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Evaluating Thermal Behavior and Use of Maturity Method in Mass Concrete

Tahsin Alper Yikici

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**Evaluating Thermal Behavior and Use of Maturity Method in
Mass Concrete**

Tahsin Alper Yikici

**Dissertation 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
Doctor of Philosophy in
Civil Engineering**

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ABSTRACT

Evaluating Thermal Behavior and Use of Maturity Method in Mass Concrete

Tahsin Alper Yikici

Large concrete placements with increased amount of cement contents result in higher peak temperatures as well as higher temperature differentials between the concrete surface and the interior. Such high thermal differentials can result in large temperature-induced stresses and increases the risk of early age cracking. To minimize this risk, temperature development within the structure must be known. Throughout the project, fourteen different sub-structures from six different bridge projects and four 6-ft cube blocks, in total of eighteen structural elements in nine different districts were instrumented successfully with sacrificial loggers, and temperature-time histories of these elements were monitored. Laboratory studies involved determination of concrete heat generation, activation energy and compressive strength development at different curing temperatures. In order to predict temperature distribution within large concrete structures, a 3D numerical analysis methodology was developed using finite volume method in which variable heat conductivity and capacity can be handled at early ages. MATLAB® was then employed to generate a program that solves the governing heat transfer equation. Analysis results were validated with temperature-time histories collected from fourteen different sub-structures at six different bridge projects and four 6-ft cube blocks. Laboratory studies were conducted to determine concrete heat generation, activation energies and compressive strength development at different curing temperatures.

Additionally, equivalent age method was implemented to estimate in-place strength of mass concrete placements. Four inch diameter core samples, with 6-foot (1.8 m) in length, were taken from the 6-ft cubes and the core strengths were compared with the predicted concrete strengths. It was found out that the predicted in-place concrete strength was always higher than the actual core strength on top surface locations and core strength from the bottom section were generally higher than the predicted values.

Overall, the numerical model has proven to produce accurate predictions in 2D and 3D temperature analysis within the concrete elements at early ages. Using the concrete mixture information and the measured concrete hydration properties, this study shows that the temperature predictions can be correlated reasonably well with the field data by means of finite volume model. Moreover, ASTM C1074 Maturity Method was employed successfully to estimate measured core strength for mass concrete structures.

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1 INTRODUCTION

1.1 Introduction

Modern concrete technology allows contractors to produce high performance concrete with high cement content to increase the rate of strength gain in order to reduce formwork removal time for accelerated construction schedules. Consequently, concrete placements with increased amount of cement contents result in higher peak temperatures, as well as temperature differentials between the concrete surface and the interior. The hydration of concrete is an exothermic process producing significant amount of heat within concrete elements. When heat of hydration slows down, surface of concrete tends to cool down much faster than the inside. Therefore, tensile stresses occur from the restraining volume of concrete which can result in thermal cracking at early ages (Figure 1.1).

Figure 1.1. Thermal cracking in a pier stem in Martinsburg, West Virginia.

At the same time, higher curing temperature will speed up the hydration process and the concrete matures faster at early age. This concept was introduced to the concrete industry as maturity concept in order to predict concrete strength development in terms of temperature and time by monitoring the in-place concrete temperatures in real time (Saul, 1951).

1.2 Background

The one characteristic that distinguishes mass concrete from other concrete elements is its thermal behavior. There are differing statements in transportation agency specifications as to what defines a structure to be deemed mass concrete. According to the West Virginia Division of Highways (WVDOH), “Concrete placements whose least dimension exceeds 48.0 inches, excluding drilled caissons and tremie seals, shall be considered as mass concrete”. However, there are differences in opinion amongst those within the industry, concrete suppliers, and the state agencies concerning where it is necessary to apply mass concrete procedures.

1.2.1 Current status of mass concrete in United States

In an effort to determine the existing status of mass concrete specifications and what other states are requiring and specifying about mass concrete, a brief survey questionnaire was sent to U.S. transportation agencies of other states by WVDOH (Mance, Mass concrete survey, 2010) . They were asked about the parameters to define mass concrete structure, such as minimum dimension and type of an element, and also about the requirement for temperature control. Additionally, a literature search was conducted and all together the following information was summarized:

1. Parameters used to define mass concrete, such as size, maximum temperature and maximum temperature differential
2. Requirements for concrete mix proportions, such as type of cement, maximum cement dosage, use of supplemental cementitious materials
3. Details of the thermal control plan

In general, requirements in mass concrete specifications of the U.S. transportation agencies vary considerably. But, mass concrete is usually defined by minimum dimension criteria such that specific elements above the suggested size limits are being considered as “Mass Concrete”.

Different states have different regulations for the mass concrete projects. There is no standard classification for mass concrete regarding to element type. In some states, drilled shafts, caissons, tremie seals and foundation seals are disregarded as mass concrete components. Some state agencies do not classify non- structural components as mass concrete. For example, in South Carolina the contractors include all costs associated with temperature control for mass concrete placement in the unit cost of the concrete. In Florida, mass concrete elements are noted in the plans used for bidding so the contractor is aware of the specific elements that are identified as mass concrete. Thus, the final bid includes the additional cost for the thermal control plan developed by a specialty engineer. Moreover, none of the agencies evaluate the added cost of having mass concrete elements in a project against the increased service life of the bridge.

Contractors are typically required to submit a temperature control plan and monitor the maximum concrete temperature in the center, and the temperature differential between the furthest from the center of the elements which are designated as mass concrete in the construction plans. Concrete temperature and temperature differentials are required to be controlled using different methods including precooling, formwork removal, insulation blankets, and cooling pipes. WVDOH has a temperature monitoring special provision that explains required temperature monitoring system with detailed instructions. A sample special provision from a bridge project conducted in 2010 is given in Appendix A.

Requirements for mass concrete placements in different states are summarized in Table 1.1. The minimum dimension criteria in the specifications is ranging from three feet to seven feet. Limitations for the initial concrete temperatures ranges between 70°F (21°C) to 95°F (35°C) as delivered to the field. Additionally, the maximum temperature is limited to between 154°F (68°C) to 180°F (82°C). Currently, there are two approaches for limiting maximum temperature

differentials. Delaware and Iowa are following a daily increasing limitation rule so that the temperature differential limit increases with time. Others, require a fixed maximum temperature differential of 35°F - 38°F (20°C - 21°C).

Table 1.1. Requirements for Mass Concrete Placements

State	Minimum Dimensions, feet (m)	Maximum Temperature, °F (°C)	Maximum ΔT, °F (°C)
Arkansas	-	75 (24) initial	<36 (20)
California	Structure: >7 (2.1)	160 (70)	Based on TCP
Delaware	-	160 (70)	0-24 hours <30 24- 48 hours <40 2-7 days <50 7-14 days <60
Florida	Structure: >3 (0.9) Drilled shaft: >6 (1.8) v/s: >1 (0.3)	180 (82)	<35 (20)
Georgia	>5 (1.5) >6 (1.8) drilled caisson	158 (70) 85(29) initial	<35 (20)
Idaho			<35 (20)
Illinois	>4 (1.2) or 5 (1.5)		
Iowa	>6.5 (2)	160 (70) 70 (21) initial	0-24 hours <20 24-48 hours <30 48-72 hours <40 >72 hours <50
Kentucky	>5 (1.5)	160 (70)	<35 (20)
Maryland	>6 (1.8)	160 (70)	<35 (20)
Massachusetts	>4 (1.2)	154 (68)	<38 (21)
Minnesota	>4 (1.2)	-	-
New Jersey	>3 (0.9) (or v/s: >1)	-	-
New York	None	-	-
North Carolina	>6 (1.8) footing	-	<35 (20)
North Dakota	5 (1.5) by 5 (1.5)	-	-
Rhode Island	4 (1.2)	-	-
South Carolina	Structures: >5 (1.5), Circular: diameter >6 (1.8), length >5 (1.5)	80 (27) (initial)	<35 (20)
Texas	>5 (1.5)	160 (70) 75 (24) initial	<35 (20)
Virginia	>5 (1.5)	95(35) initial 160 (70) 170 (77) with slag	<35 (20)
West Virginia	>4 (1.2)	160 (70)	<35 (20)

Additionally, there are other requirements for the mix design and ultimate concrete strength that can be used in mass concrete placements. For example, Arkansas requires 3,000 psi

compressive strength at 28 days (3,500 psi at 90 days) with minimum 0.49 water to cement ratio. Fly ash may be used up to 120 pounds per cubic yards (70 kg/m^3), GGBFS is allowed up to 25%. Formwork can be removed 4 days after placement, all exposed surfaced are moist cured for 14 days (Arkansas State Highway and Transportation Department, 2015). Delaware has a performance specification for mass concrete. According to the specification, “Design-Builder’s” are responsible to determine which elements will be considered as mass concrete and to provide a thermal control plan to ensure that no thermal cracking occurs. Slag or fly ash are allowed up to 75% by weight of total cementitious material in the mix (State of Delaware Department of Transportation, 2008). Virginia allows up to 40% fly ash or 75% slag replacement for mass concrete mixtures and requires hourly temperature monitoring for at least 3 days. Florida requires the temperatures to be monitored at a maximum of 6 hour intervals until the maximum temperature is achieved. Use of fly ash and slag for mass concrete mixtures is allowed within specified limitations where the maximum cementitious material content is limited to 752 pounds per cubic yard (445 kg/m^3) (Florida Department of Transportation, 2013). Iowa requires minimum cement content of 562 pounds per cubic yard (330 kg/m^3) and maximum water to cementitious ratio of 0.45 (Iowa Department of Transportation, 2010). North Carolina has project based special provisions. There is no maximum temperature limitations, but a thermal control plan is required with minimum six temperature sensors; one sensor at the center of mass and a second sensor 2 inches (50 mm) from the farthest surface recording hourly temperatures. Maximum amount of cementitious material is 690 pounds per cubic yard (410 kg/m^3) with a range of water to cementitious ratio between 0.37 and 0.41. Minimum 28 day strength is 5,000 (35 MPa) psi (Bobko, Seracino, Zia, & Edwards, 2014). Texas requires temperature monitoring only at two

locations: the core and the surface for the first 4 days. Fly ash is allowed up to 45% in mass concrete placements (Pruski & Browne, 2008).

1.2.2 Research needs

WVDOH special provisions for mass concrete (Appendix A) requires a thermal control plan to be implemented for structures with at least 48.0 inches in one dimension, excluding drilled caissons and tremie seals. A temperature monitoring and recording system is also required to record concrete temperatures at the center, the top surface and the side surface of the structure. The maximum allowable temperature differential shall be limited to 35°F (20°C) and the maximum allowable concrete temperature shall be limited to 160°F (70°C). Contractors violating these requirements are imposed financial penalties, such that paying given amount of dollars per cubic yard per degree Fahrenheit that exceeds the limits. Eventually, application of mass concrete procedures needs extra effort as well as an extra cost in the project specifications. Therefore, mass concrete has been the subject of interest within WVDOH. Varying mass concrete procedures have been used on bridge construction projects. Research is needed to investigate early-age thermal behavior of concrete elements. Temperature development and distribution within the large structures must be known, especially for the concrete produced using local materials available in West Virginia. It is also necessary to predict the temperature-time histories in order to prevent cracking which may cause loss of structural integrity and durability, shortening the service life of the infrastructure.

1.3 Objectives of this study

The main objective of this study is collecting data from the field and the laboratory, and develop a computational program that will allow the researchers and construction engineers to

evaluate thermal behavior of selected bridge elements i.e. maximum temperature differentials and peak temperature, so that specifications for mass concrete should be applied. Such analysis methodology in terms of a prediction model, can help engineers and contractor to make decisions beforehand, such as concrete mix proportioning, pre-cooling, insulation, curing and formwork removal time. Other research objectives of this study can be listed as follows:

- Identify the appropriate test methods to determine heat of hydration in concrete mixtures.
- Determine the rate of heat generation of commonly used concrete mixtures in West Virginia that can be utilized for predicting temperature development.
- Develop and implement an analysis method to predict temperature-time history of concrete elements and compare analysis results with field data.
- Establish the relationship between temperature history and strength development of in-place concrete.

1.4 Outline of Dissertation

The dissertation consists of eight chapters. Chapter 2 is a review of the literature summarizing recent studies related with thermal analysis and maturity method in mass concrete. Concrete maturity concept is described in detail and effect of curing temperatures at early ages are highlighted in this chapter. Chapter 3 shows the data collected from the field studies including temperature time histories and concrete properties from selected bridge elements and 6-ft cube constructions. This chapter also includes the procedures of monitoring and collecting temperatures from the field. Chapter 4 includes the finite volume numerical method used for 2D and 3D thermal analysis of concrete structures. This chapter also includes information about the basic principles for the temperature prediction model, including available models for concrete thermal properties as well as other input parameters for the analysis. Concrete heat generation and available methods to quantify rate of heat generation in concrete are thoroughly described. Experimental data for different concrete mixtures obtained in the laboratory are presented. Chapter 5 presents analysis

results in comparison with temperature data collected from bridges as well as 6-ft cubes. Chapter 6 discusses application of maturity method in mass concrete using equivalent age predictions of locally produced concrete mixtures and the verification of in-place concrete strength using 6-ft core samples. Chapter 7 summarizes the conclusions drawn from this study and Chapter 8 provides recommendations and future study.

2 LITERATURE REVIEW

2.1 Introduction

According to the ACI, mass concrete is defined as “any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of cement and attendant volume change to minimize cracking” (ACI Committee 207, 2005). In addition to that, ACI Committee 301 presents an optional requirements checklist to assist the engineers in selecting and specifying project requirements in the specification. Roughly, mass concrete is defined by a minimum dimension of 2.5 feet (0.75 m) or a minimum cement content of 600 pounds per cubic yard (350kg/m³) (ACI Committee 301, 2005).

Portland Cement Association (PCA) describes mass concrete as any concrete placement with a minimum dimension of 36 inches (915 mm), or concrete batched with a minimum of 600 pounds per cubic yard (350 kg/m³) TYPE III or high-early-strength cement regardless its dimension that could cause high internal temperatures exceeding 158°F (70°C). Additionally, concrete placements should be considered as mass concrete where thermal cracking may occur due to high temperature differentials between the center and the surface of the structure (Gajda J. , 2007).

Nowadays, any concrete placement that leads to thermal cracking is considered as mass concrete regardless its dimensions. Therefore, estimation of thermal behavior has become necessary to minimize or eliminate cracking of concrete due to thermal issues in order to ensure long term durability for extended service life of the concrete structures.

2.2 Controlling mass concrete temperatures

Construction practices of how to control mass concrete temperatures vary. Prescriptive specifications with simplistic methods have been found effective controlling temperatures to

minimize cracking in mass concrete placements. These methods include limiting maximum cementitious content (600 pounds per cubic yard), incorporating fly ash (25%-30%), limiting initial concrete temperatures (75°F-80°F), limiting maximum concrete temperatures (160°C) and temperature differentials (35°C). It requires concrete mix prequalification before placement and temperature monitoring during placement (ACI Committee 207, 2005).

ACI recommends using TYPE II or TYPE IV cement or replacing cement with supplementary cementitious materials such as fly ash, ground granulated blast furnace slag (GGBFS) in order to obtain less heat of hydration. Additionally, decreasing the cement dosage is recommended by ACI to reduce the heat of hydration when 56 or 90-day strength is acceptable for service conditions. Water-curing is also recommended for extra cooling during summer time. In addition, pre-cooling the constituent materials, and post-cooling the structure using cooling pipes are suggested in ACI 207.1R for more effective temperature control. Furthermore, ACI suggests preparing a thermal control plan that restricts the temperature differential between the surface and the center of the structure so that thermal cracking can be minimized. When a high maximum concrete temperatures is expected, it is recommended to add reinforcement to minimize the crack width. ACI 207.1R provides detailed information about the constituent materials, concrete mixtures, mechanical and thermal properties of in-place concrete, construction methods and equipment that are necessary for mass concrete applications. Key points about batching, mixing, placing and curing mass concrete are explained thoroughly. The guideline states that controlling the type and the amount of cementitious material is the key for limiting temperature rise inside the concrete (ACI Committee 207, 2005) (ACI Committee 301, 2005).

ACI 207.2R describes typical values for structural properties such as tensile strength and creep, and thermal properties such as specific heat, thermal conductivity, and thermal diffusivity

of mass concrete mixtures that can be used to predict cracking potential. The guidelines provide several example problems to estimate initial concrete temperature, temperature rise and final concrete temperature in mass concrete structures using given charts and tables consisting empirical data. Schmidt Method for estimating the temperature rise and the differentials was described (ACI Committee 207, 2007).

ACI 207.4R summarizes the methodology of different construction procedures that can be used to control temperature development in mass concrete structures. Pre-cooling is one of the most effective method for reducing the initial concrete temperature. The batch water can be chilled or substituted with ice, and the aggregate piles can be shaded or water cooled to minimize the fresh concrete temperature. Alternatively, liquid nitrogen can be used to reduce fresh concrete temperature. Concrete temperatures can be also reduced using post-cooling methods such as embedded water pipe and shading (ACI Committee 207, 2005).

2.2.1 Allowable temperature difference

In mass concrete specifications the maximum allowable temperature difference is often limited to 35°F (20°C). This limit was proposed by FitzGibbon M. E. in 1970's. In his study, the cracking strain of concrete was reached when the temperature differential exceeds 35°F (20°C). According to the study, thermal cracking in mass concrete occurs by two different mechanisms, as shown in Figure 2.1. First, thermal cracking occurs because of the instant surface cooling therefore early formwork removal is one of the main reasons for external thermal cracking. Second, thermal cracking occurs when the rate of the surface temperature rise is lot smaller than the inside during the first day or two after concrete placement (Fitzgibbon, 1976).

Figure 2.1. Thermal cracking in mass concrete structures (Fitzgibbon, 1976)

Lately, researchers discuss if the allowable temperature difference of 35°F (20°C) is still viable, since in some cases thermal cracking has not been encountered even at higher temperature difference, and in other cases, cracking was observed when the temperature difference is below 35°F (Gajda & Vangeem, 2002).

2.3 Current Research Regarding Mass Concrete

There are several studies related to thermal properties and early age cracking issues of mass concrete placements. Some of the most recent studies are summarized in this section.

In 2002, the FHWA Mobile Concrete Laboratory (MCL) conducted a project during the Ohio Vermilion Bridge Construction to evaluate the potential for thermal cracking. FHWA analysis was based on the maximum temperature and temperature gradient predictions. The semi-adiabatic calorimetry analysis was used to predict temperature developments of 12 feet diameter caissons after concrete placement and results were compared with field data using maximum temperature limit of 160°F (70°C) and maximum temperature differential of 35°F (20°C) (FHWA Mobile Concrete Laboratory (I), 2002). Additionally, MCL performed thermal analysis for the pile caps used in Woodrow Wilson Bridge Foundation, Maryland in 2002. Temperature development within the pile caps were analyzed under varying air temperatures, river water

temperatures, concrete temperatures and placement times. Depths of these pile caps range from a minimum of 9 feet to a maximum depth of 16 feet and widths of the pile caps range from 40 feet to 53 feet. According to the simulation results, the initial concrete temperatures and the size of the placement had significantly affected the maximum concrete temperatures and temperature differentials. The air temperatures and the river water temperatures did not influence the maximum concrete temperatures at the center of the pile caps. However, river water temperatures had a significant effect on the temperature differentials (FHWA Mobile Concrete Laboratory (II), 2002).

Typically, FHWA recommends replacing cement, with fly ash and GGBFS which can reduce the heat of hydration of concrete and decrease heat rise problems in mass concrete placement. Use of Grade 80 GGBFS with substitution rates greater than 50 percent was recommended (FHWA, 2011).

Iowa State University conducted a research project funded by the FHWA and published technical reports in 2006 and 2007. Firstly, they investigated existing test methods that have been used in-situ and in the laboratory for monitoring the heat of hydration evolution of concrete mixtures. Later on, they performed experimental work to evaluate the commercially available calorimetry equipment and developed a faster calorimetry test for concrete heat of hydration (Wang, Ge, Grove, Ruiz, & Rasmussen, 2006) (Wang, et al., 2007).

Folliard et al. (2008) developed a software known as ConcreteWorks, which can be used to analyze concrete structures to control thermal cracking. They built an empirical hydration model that can simulate the temperature time history of different types of structures using concrete mixtures batched with various cementitious materials.

Tia et al. (2010) conducted studies on finite element modeling for mass concrete structures, particularly for bridge footings. University of Florida researchers analyzed the early age behavior

of 3.5-ft concrete blocks and predicted the thermal behavior accurately using finite element modeling. They suggested performing isothermal calorimetry test for each concrete mixture to be used as the heat generation function in their model. They also recommended running a finite element analysis for mass concrete applications to make sure that the tensile stresses due to temperature gradients will not exceed the tensile strength of the concrete structure. According to their analysis results, it was suggested to improve the formwork insulation in order to reduce the temperature gradients at early ages.

2.4 Use of Maturity Method in Mass Concrete

2.4.1 Concrete maturity concept

According to the maturity concept, concrete specimens will have same compressive strength if they have the same calculated maturity from the temperature-time history (Carino, 2004). The method assumes that the temperature-time history of concrete can be used to develop a strength-maturity curve that is specific to each mix design. By preparing these correlation curves, the strength of in-place concrete can be estimated by monitoring the concrete temperatures in real time. Consequently, this information can be used to make decisions (e.g. time of formwork removal) that save time and reduce the construction cost.

There are two alternative methods to calculate the maturity of concrete using the temperature history. First one is known as Nurse-Saul maturity function and assumes that the chemical reaction rate in concrete increases with increased temperature. It is simply the area under the temperature-time history (Carino, 2004).

$$M(t) = \sum (T_a - T_0)\Delta t \quad 2.1$$

where: M is the temperature-time factor or maturity index, T_a is the average concrete temperature, T_0 is the datum temperature and Δt is the time interval. The datum temperature is considered to

be the lowest temperature at which the hydration will occur and can be determined following ASTM C 1074 procedure (Ferraro, 2009).

The second maturity function was proposed first by Freiseleben-Hansen, and Pedersen in 1977 and can be specifically used to convert the actual age of concrete into the equivalent age (t_e), which is defined as a function of time at a specified temperature. This function is known as “Arrhenius Equation” within the industry:

$$t_e = \sum e^{\frac{E_a}{R}(\frac{1}{T_a} - \frac{1}{T_s})} \Delta t \quad 2.2$$

where; t_e is the equivalent age, E_a is the activation energy, R is the universal gas constant, T_a is the average temperature of the concrete during time interval, T_s is the specified (reference) temperature and Δt is the time interval. The reference temperature is generally assumed to be 68°F (20°C) in European standards and 73°F (23°C) in ASTM. Using the given maturity function a calibration curve can be prepared from strengths of cylinders that were cured under laboratory conditions.

The calibration curve that represents the strength gain of the concrete can be modeled using appropriate equations. The hyperbolic model was suggested by ASTM to analyze the strength data is as follows:

$$S = S_u \frac{k(t - t_0)}{1 + k(t - t_0)} \quad 2.3$$

where; S is the strength at age t , S_u is the limiting strength, k is the rate constant, 1/day, and t_0 age at start of strength development. The limiting strength, S_u , is the asymptote for the function that fits the data. The parameters S_u , k and t_0 can be obtained by least-squares curve fitting analysis using the compressive strength test results. At least six data point are required for the analysis.

Another widely used formulation for modeling strength development is the exponential function (Carino, 2004):

$$S = S_u \exp \left[- \left(\frac{\tau_s}{t} \right)^{\beta_s} \right] \quad 2.4$$

where; τ_s is the time constant and β_s is the shape constant. Similarly, the parameters can be obtained by curve fitting analysis and the rate constant is calculated as the inverse of the time constant.

For each concrete mixture, a strength-maturity relationship needs to be established by laboratory tests. At the same time, the temperature-time history of the in-place concrete is recorded. Using the strength-maturity relationship and the calculated in-place maturity index the in-place strength can be estimated (ASTM Standard C1074, 2011). Using the given maturity function, the actual age of the concrete can be converted to the equivalent age at a specified temperature and the calibration curve can be used to estimate the in-place concrete strength if temperature history of the structure is known.

2.4.2 Activation energy of concrete

The Arrhenius Equation requires determination of the apparent activation energy for the calculation of the equivalent age functions. Activation energy is defined as the energy that a molecule requires to initiate a reaction (Glasstone, Laidler, & Eyring, 1941). Concrete activation energy can be simply defined as the sensitivity of concrete properties at different curing temperatures (D'Aloia & Chanvillard, 2002). Activation energy of concrete is mix design specific and the value of activation energy depends on factors that affect the rate of strength gain of the concrete. Cement chemistry, cement fineness and chemical admixtures added to the concrete can be considered as main factors affecting activation energy (Carino, 2004). Recommended values of activation energy in the literature range from 40 to 45 kJ/mol for concrete mixtures batched

with ordinary TYPE I cement without any other additives or admixtures (Carino & Lew, 2001). Activation energy values can be obtained experimentally by means of compressive strength or isothermal calorimetry tests. Besides, different prediction models have been proposed in the literature based on experiments and theoretical analysis, and conflicting results have been drawn regarding the value of the apparent activation energy.

Freiesleben-Hansen and Pedersen (1977) proposed the apparent activation energy according to the following equations (Freiesleben & Pedersen, 1977):

$$\begin{aligned} E(\theta) &= 33,500 \text{ J/mol}, \theta \geq 20^\circ\text{C} \\ E(\theta) &= 33,500 + 1470(20 - \theta) \text{ J/mol}, \theta < 20^\circ\text{C} \end{aligned} \tag{2.5}$$

where, θ is the temperature in degree-Celsius.

According to Kim et al. (2001) the variation of initial apparent activation energy (E_0) with curing temperature (T^c) can be presented as:

$$E_0 = 42.830 - 43T^c \text{ J/mol} \tag{2.6}$$

Han et al. (2003) proposed a model to estimate the activation energy of fly ash concrete. They produced research data for fly ash concrete with various water-binder and fly ash replacement ratios and proposed estimation curves for the initial activation energy. The results are divided into two groups:

$$\begin{aligned} \text{Water/Binder} &\leq 0.40 & \text{Water/Binder} > 0.40 \\ E_a &= 39,720 - 119FA & E_a = 42,920 + 90FA \end{aligned} \tag{2.7}$$

where; FA is the fly ash replacement ratio (%).

Another research conducted by Barnett et al. (2006) investigated the effect of GGBFS replacement level on apparent activation energy. It was found out that the apparent activation

energy (E_a) is highly dependent on the GGBFS level and vary approximately linearly from 34 kJ/mol to around 60 kJ/mol:

$$E_a = 32,200 + 400r \quad 2.8$$

where; r represents the GGBFS replacement ratio (%).

Schindler and Folliard (2005) proposed a regression model to predict the activation energy for each cement type that accounts for different curing temperatures. The change in cement chemical composition, fineness and use of supplementary cementitious materials were considered in the given formula to estimate activation energy (Schindler & Folliard, 2005).

$$E = 22,100f_E p_{C3A}^{0.3} p_{C4AF}^{0.25} Blaine^{0.35} \quad 2.9$$

where; p_{C3A} is the weight ratio of C3A, p_{C4AF} is the weight ratio of C4AF, f_E is the activation energy modification factor for supplementary cementitious materials defined as:

$$f_E = 1 - 0.05f_{FA} \left(1 - \frac{p_{FACao}}{0.4}\right) + 0.4p_{slag} \quad 2.10$$

where; p_{FACao} is the weight ratio of the CaO content of the fly ash and p_{slag} is the weight ratio of the slag.

Later on, Poole (2007) proposed an updated regression model to calculate concrete heat of hydration parameters including apparent activation energy based on the concrete mixture proportions and constituent material properties, and developed an equation based on the concrete isothermal calorimetry test results. A similar equation is being used in ConcreteWorks program to calculate concrete heat of hydration parameters (Poole J. L., 2007).

$$\begin{aligned} E_a = & 41,230 + 1,416,000 \cdot [(C_3A + C_4AF) \cdot p_{\text{cement}} \cdot SO_3 \cdot p_{\text{cement}}] \\ & - 347,000 \cdot Na_2O_{eq} - 19.8 \cdot Blaine + 29,600 \cdot p_{\text{FlyAsh}} \cdot p_{\text{CaO-FlyAsh}} + 16,200 \cdot p_{\text{GGBFS}} \\ & - 51,600 \cdot p_{\text{SF}} - 3,090,000 \cdot WRRET - 345,000 \cdot ACCL \end{aligned} \quad 2.11$$

where; p_{cement} is the % cement in mixture, p_{FlyAsh} = % fly ash in mixture, $p_{\text{CaO-FlyAsh}}$ is the % CaO in fly ash, p_{GGBF} is the % GGBF slag in mixture, p_{SF} is the % silica fume in mixture, $\text{Na}_2\text{O}_{\text{eq}}$ is the % Na₂O equivalent alkali in cement ($0.658 \times \% \text{K}_2\text{O} + \% \text{Na}_2\text{O}$), C₃A is the % C₃A in cement, C₄AF is the % C₄AF in cement, SO₃ is the % SO₃ in cement, WRRET is the ASTM Type A-D water reducer/retarder, % solids per gram of cementitious material, ACCL is the ASTM Type C calcium-nitrate based accelerator, % solids per gram of cementitious material.

2.4.3 Research regarding maturity method in mass concrete

Maturity method has been used world-wide for decades in different construction projects. Many state DOT's have instituted procedures to implement the maturity method to predict in-place concrete strength for structural applications and pavements. However, there are no standard specification or procedure recommending the use of maturity method in mass concrete applications. On the contrary, Arizona Department of Transportation has provisions for prediction of concrete strength using maturity method that specifically does not recommend using this method for mass concrete (Arizona Department of Transportation, 2010). Nevertheless, use of maturity method for mass concrete was studied by different researchers in order to find out how early age high temperatures affects concrete strength development. Ahmad et al. (2006) proposed new procedures for using maturity method in Florida as a reliable quality control and quality assurance tool, so that the conventional cylinder testing method can be partly replaced for strength verification.

In 2008, Wade et al. employed maturity method on several precast, prestressed girders and a bridge deck in Alabama. It was concluded that the method can be used accurately for estimating in-place concrete strength up to an equivalent age of seven days. Anderson et al. (2009) reported that the maturity method was used in three different Portland cement concrete pavement (PCCP)

projects in state of Washington to open traffic faster. Similarly, Hosten and Johnson (2011) evaluated maturity method for use in pavements in Maryland. It was concluded that the procedure is very sensitive to the constituent materials and concrete mixtures. Extreme pre-cautions in order to obtain maximum accuracy when using maturity method for the field applications. Connecticut recently implemented maturity method in PCC specifications and considering to expand its use in all type of structural applications (Henault, 2012). Nevertheless, the maturity concept has been used to estimate in-place concrete strength development for over 40 years (Carino, 2004).

Poole (1996) from U.S. Army Engineers investigated the applicability of maturity method for estimating in-place strength of high volume fly ash concrete used in Red River Waterway Lock and Dam No.4 construction. It was found out that the estimated concrete strength following Nurse-Saul method were up to 50% lower than the measured concrete strength from the core samples. When using equivalent age method the difference became smaller at early ages but the later age estimations were still underestimating in-place concrete results, having an error close to 40% at early ages. It was concluded that in-place concrete strength cannot be properly predicted using maturity method in mass concrete structures where high concrete temperatures may be developed (Poole T. S., 1996).

Tepke et al. (2004) showed results of concrete maturity method performed in different highway bridge structures constructed with high performance concrete. It was concluded that high curing temperatures significantly reduced the long-term compressive strength, however the strength-maturity relationship was found to be appropriate in predicting in-place strength of bridge piers, where high concrete temperatures recorded.

Kim (2004) investigated the effects of variable curing temperatures on the strength development of two different concrete mixtures. According to the results, the normal strength

concrete cured at higher temperatures indicated lower results at later ages compared to regularly cured specimens. It was also stated that specimens cured at constant high temperatures show lower strength results compared to the specimens subjected to variable temperature curing. The study also claimed that the crossover effect does not exist for high strength concrete mixtures (Kim T. , 2004).

2.4.4 Effect of curing temperatures on strength development

The main concern for using maturity method in mass concrete applications is the large amount of heat released during hydration. Consequently, effects of early high temperatures on the concrete strength development has been investigated in many research studies (Carino, "The Maturity Method", Chapter 5 in Handbook on Nondestructive Testing of Concrete, 2004) (Chanvillard & D'Aloia, 1997) (Kim & Rens, Concrete maturity method using variable temperature curing for normal and high strength concrete. Part I: Experimental study., 2008).

Chini et al. (2003) explained the theory behind this interaction between high early temperature and concrete strength. According to this theory, concrete cured at high temperatures will hydrate at a higher rate and the increased rate of hydration does not allow sufficient time for the proper distribution of hydration products. This will result in lower concrete strength at later ages. On the other hand, the hydration products are more uniformly distributed when concrete is cured at lower temperatures. More hydration products formed due to higher temperatures result in high early strength. However, those hydration products form a barrier type of structure around the unhydrated cement particles, which prevents further hydration. Therefore, concrete that is cured at constant high temperature shows lower strength results in later ages.

Literature has shown evidence that the development of the concrete strength is not only dependent on the concrete age but also curing conditions. Specifically, the long-term strength is

reduced at higher curing temperatures. But, current maturity functions does not consider such effect on strength development. However, maturity method may work appropriately before strength losses start to occur and the maturity equations can be implemented for early-age results (concrete equivalent age less than 4 days).

3 FIELD TEMPERATURE MONITORING

3.1 Selecting Bridges for Instrumentation

One of the tasks during this study was the selection of bridge constructions from different districts in West Virginia for thermal monitoring. Choosing different construction projects from all over the state with different concrete mix design, different types and brands of cement, and different types of aggregates was a good opportunity to investigate the effects of the concrete materials and the environmental conditions. After looking through many ongoing construction projects without mass concrete special provisions, the following bridges listed below in Table 3.1 were selected. These selected bridges were instrumented with temperature sensors right before concrete placement and concrete temperatures were monitored for 28 days after casting. All the selected bridges are non-mass concrete bridges constructed using regular Class B or Class B modified concrete.

Table 3.1. List of Bridges

DOH DISTRICT #	BRIDGE	PROJECT #
DISTRICT 10	CLEAR FORK ARCH BRIDGE #2	S355-6-7.64
DISTRICT 10	CLEAR FORK ARCH BRIDGE #1	S355-6-5.95
DISTRICT 7	LUCILLE STALNAKER BRIDGE	S311-17-01000-07
DISTRICT 4	ICES FERRY BRIDGE	S331-857-102000-AF

DISTRICT 3	SOUTH MINERAL WELLS INTERCHANGE	S354-14-7.47 00
DISTRICT 2	5TH AVENUE BRIDGE	S306-T2-0.08

Specific bridge elements such as abutments, footings, pier stems, and pier caps were selected from each bridge. A special provision for the selected bridge projects was developed with collaboration of the project monitors. The special provisions outlined the general requirements about the temperature monitoring system that was going to be used during the construction of the selected bridges. The special provisions included all necessary information about the monitoring process as well as instructions for installing temperature sensors and a mass concrete temperature monitoring form.

To be able to get additional data from existing normal concrete mixtures in the areas where no bridges are being monitored, 6-foot concrete cube blocks were constructed with the help of the bridge division at four different WVDOT Districts. Similarly temperature sensors were placed in these concrete blocks and temperature data were collected; a detailed instruction plan for the construction and instrumentation of the 6-ft cube study was developed and implemented.

3.2 Temperature monitoring special provisions

An instruction plan explained basically how to install the temperature sensors and the monitoring procedures were developed (Appendix B). It was specific for each bridge and provided details of each temperature sensor location and name designation with drawings. The locations of the sensors were decided before hand according to the bridge plans and construction schedules. For simple data collection each sensor was designated after its locations, such as D7C103-P indicates District 7 (D7), Pier cap (C1), sensor

location number 3 (03) and primary sensor (P). An example drawing is given here in Figure 3.1.

Figure 3.1. General layout for the instrumentation of a pier cap in District 7.

Additionally, a mass concrete temperature form was outlined that needed to be filled out by the contractor for each element installed with temperature sensors including information during construction such as formwork type, curing, and concrete mixture used for that placement and on-site observation.

3.3 Instructions for the 6-ft Cubes

The instructions for construction and testing plan for 6-ft cubes was prepared to describe the sequences and methodology of 6-foot cube casting at District 1, 9, 5 and 6. A 6-foot cube was constructed at each WVDOH district office. A copy of the instrumentation plan for the 6-ft Cube experiment is attached in the APPENDIX C.

3.4 Temperature monitoring training seminar

A training seminar on how to install and use the temperature sensors were organized and the WVDOH engineers and contractors who were involved with the project were invited to this training session. A picture of the data-logger and temperature sensors can be seen in Figure 3.2. The project special provisions regarding the instrumentation plan prepared for respective bridge projects were also explained during the seminar.

Figure 3.2. Temperature monitoring equipment, handheld reader and loggers.

3.5 Summary of field temperatures

Table 3.2 shows the summary of all bridge structures that were monitored throughout this research project. Each structure was instrumented at several different locations with a primary and a secondary sensor recording hourly concrete temperature for about 28 days after concrete placement. An extra sensor located at the construction site was usually used to record ambient temperature. When ambient temperature was not recorded, temperature data from the closest weather station was retrieved online. All temperature data collected from the bridge structures is presented in Appendix D.

Table 3.2. Bridge Element Summary

Element	Mix Design	Dimensions			Type of Formwork	Formwork Removed, days	Curing
		Length	Width	Height			
Lucille Stalnaker Pier Stem#1	Class B Flyash (1390229)	-	6' (diameter)	~17' - 7"	Metal	7	wet burlap
Lucille Stalnaker Pier Cap#1	Class B Flyash (1390229)	24'-6"	4'	8'	Plywood	8	wet burlap
Lucille Stalnaker Abutment#2	Class B Fly Ash (1390229)	26'-6"	3'-9"	~5'	Plywood	7	wet burlap
5 th Avenue Footer#2	Class B GGBFS (1399055)	16'-0"	16'-0"	4'-0"	Plywood	4	wet burlap
5 th Avenue Pier Stem#2	Class B Mod. (1399056)	-	8'-0" (Diameter)	-	Steel	3	wet burlap
5 th Avenue Pier Cap#2	Class B Mod. (1399056)	37'-3"	5'-0"	11'1 ⁵ / ₈ "	Steel	28	wet burlap
5 th Avenue Abutment#2	Class B GGBFS (1399055)	39'-7"	3'-9"	~4'	Plywood	7	wet burlap
Clear Fork Arch#2 Abutment#1 Footing	Class B Fly-Ash (1385966)	29'-8"	11'-0"	3'(min)	Steel	-	-
Clear Fork Arch#2 Abutment#2 Stem	Class B Fly-Ash (1385966)	32'-0"	9'-4"	3'-0"	Steel	-	-
Clear Fork Arch#1 Abutment#2 Footing	Class B	35'-2 ⁵ / ₈ "	3'-0"	5'0"	N.A.	N.A.	N.A.
S. Mineral Wells Pier#2 Stem#2	Class B Fly-Ash (1420219)	-	7'-0" (Diameter)	-	Steel	5	wet burlap on top
S. Mineral Wells Pier Cap#2	Class B Fly-Ash (1420219)	57'-8"	5'-0"	10'2"	Steel	17	wet burlap on top
S. Mineral Wells Abutment#2	Class B Fly-Ash (1390254)	58'6"	3'0"	~4'	Plywood	-	-
Ices Ferry Pier Cap#2	Class B Mod.	37'-0"	6'-0"	10'2"	Steel	N.A.	N.A.

3.6 Six Feet Cubes Construction

Other than the bridge constructions, four 6-ft Cubes were constructed at four different WVDOH districts (D1, D5, D6 and D9), in Charleston (D1), Lewisburg (D9), Martinsburg (D5), and Wheeling (D6), pouring approximately 9 cubic-yard concrete provided by local ready-mix concrete plants. The purpose of the 6-ft Cube is to monitor the maximum temperature and maximum temperature differential of a larger volume of

normal Class B concrete in order to obtain parameters to predict the temperature profile of mass concrete. In addition to that, the accuracy and the limitations of the maturity method for Mass Concrete placements was determined by estimating concrete in-place strength according to ASTM C 1074.

A two-day trip was made for each of the cube's construction. The temperature sensors were instrumented and a cube block was constructed 2 feet (0.6 m) in the ground on a 2-inch (50 mm) layer of #57 limestone. At the same time, fresh concrete properties were determined and 6x12 inch (300x150 mm) cylinders were taken for the maturity test. A schematic of the sensor locations is given in Figure 3.3.

Figure 3.3. Schematic of the sensor locations.

Concrete was poured directly from the concrete mixer truck without pumping and then was subjected to vibration in order to get sufficient compaction. Ordinary surface finish using wood-float rubbing was applied on the top surface. The concrete surface was maintained completely and continuously moist by burlap during seven day curing period. After the concrete placement the top of the block was covered with white polyethylene

sheeting. If necessary, concrete blankets were used on top surface as well as around the formwork.

3.6.1 District 1 Cube construction

District 1 Cube was constructed in Charleston, WV on August 15, 2011 using wooden formwork. Temperature sensors were placed and activated in thirteen locations (Figure 3.4). Nine cubic yards of Class B Fly Ash Concrete containing 470 pounds per cubic yard (278 kg/m³) cement and 75 pounds per cubic yard (44 kg/m³) Fly Ash was delivered to the construction site. The mixture had 2.5 inches (65 mm) slump and 6.8% air content as measured on the job site. After the concrete placement the top of the block was covered only with white polyethylene sheeting. The concrete casting started at 1:00 PM with an initial concrete temperature of 81°F (27°C).

Figure 3.4. District 1 Cube instrumentation.

Temperature sensors collected hourly data at thirteen locations for the first 28 days after casting. The maximum temperature observed for this cube was 145°F (63°C) and occurred 24-25 hours after casting in the core of the 6-ft cube (Sensor #3). At the same time, the surface sensors, about 2-inch inside the surface (Sensors #1 and #6, located on the top surface, Sensor #11, located at the center of the side surface, and Sensors #5 and

#8, located at the bottom surface) show a maximum of either 118°F (48°C) or 120°F (49°). The maximum temperature differential was 32°F (18°) and occurred after about 45 hours after casting between the center and the side surface of the cube. The two corner sensors located at the diagonally opposite corners show the lowest temperature value at around same time; Sensor #12 located bottom corner was 95°F (35°C) and Sensor #13 top corner was 98°F (36.5°C), and it can be observed that those two sensors are influenced most heavily by the ambient temperatures. In addition, the polyethylene sheeting was not effective enough to prevent the temperature fluctuations on the top surface (Figure 3.5).

Figure 3.5. Temperature-time history, District 1 Cube.

3.6.2 District 9 Cube construction

The 6-ft cube at District 9 was constructed on August 26, 2011 in Lewisburg, WV, using Class B concrete with 564 pounds per cubic yard (334 kg/m^3) TYPE I/II cement and 0.45 water-cement ratio. Wooden formwork and #4 rebar cage were prepared, and temperature sensors were attached at thirteen locations. Concrete was delivered to the field in two batches with two mixing trucks. Slump was measured 5 inches (125 mm) from the first truck and 7 inches (180 mm) from the second truck and the air content was 7.8%

and 9.5% on the job site. The initial concrete temperature was recorded 82°F (28°C). Concrete poured directly from the mixers into the 6-ft cube and consolidated with hand held vibrators. The top of the block was covered with burlene (Figure 3.6).

Figure 3.6. District 9 Cube Instrumentation.

The maximum temperature of 165°F (74°C) occurred in the core of the 6-ft cube (Sensor #3) at 31-32 hours after casting. At the same time, Sensor #1 and Sensor#6 that are located on the top surface reached 127°F (53°C) and 133°F (56°C), respectively, and Sensor #11, which is located at the center of the side surface, was 133°F (56°C) as well. The maximum temperature differential was 38°F (21°C) between the top surface and the center of the cube at about 35 hours after casting. The two corner sensors located at the diagonally opposite corners show the lowest temperature value at around same time; Sensor #12 located bottom corner was 99°F (37°C) and Sensor #13 top corner was 108°F (42°C), and these two sensors would be most influenced by the ambient temperatures (Figure 3.7).

Figure 3.7. Temperature-time history, District 9 Cube.

3.6.3 District 5 Cube construction

Another 6-ft Cube was constructed at District 5 Martinsburg, WV, next to the Shenandoah River Bridge Construction. The same Class B concrete that was placed into the bridge footers and piers was used in the cube construction (Class B GGBFS). According to the instrumentation plan the surface sensors were placed inside the concrete cover which is usually 2 inches inside the cube. This time, one more temperature sensor (#14) was placed on the side surface as close as possible to the formwork (approximately ½ inch (12 cm) from the surface) to determine the effect of the concrete cover on temperature differentials between the core and the closest surface of the cube.

The rebar cage and the steel formwork were prepared and temperature sensors were placed accordingly. Nine cubic yards of Class B GGBFS Concrete containing 423 pounds per cubic yard (250 kg/m³) cement and 141 pounds per cubic yard (83 kg/m³) Slag was delivered to the job site (Figure 3.8). The water-cementitious ratio of the mixture was 0.48. The initial temperature of the concrete was measured 79°F (26°C), the slump was 2.5 inches (65 mm) and air content was measured at 5.6%.

Figure 3.8. District 5 Cube instrumentation.

The maximum temperature recorded in the cube was 149°F (65°C) at around 28-30 hours after concrete casting. At the same time, the surface sensors that are embedded with a concrete cover thickness of 2 inches shows 129°F (54°C) at the top surface (#3), 118°F (48°C) at the bottom surface (#5), and 113°F (45°C) at the side surface (#11). The sensor #14 was embedded closer to the formwork, approximately 0.5 inches (13 mm) inside concrete. Temperature differential at that time was 36°F (20°C) between the core (#3) and the side surface sensor (#11), but it reaches 45°F (25°C) when #14 is considered as the concrete surface temperature. The maximum temperature differential is 54 °F (30°C) and occurs approximately after 42 hours after concrete placement between the center sensor (#3) and the side surface (#14). The two sensors located at the diagonally opposite corners show the lowest temperature value at around same time; Sensor #12 located bottom corner was 89°F (32°C) and Sensor #13 top corner was 79°F (26°C), and it can be observed from temperature-time history that those two sensors are influenced most heavily by the ambient temperatures and therefore show lowest concrete temperatures (Figure 3.9).

Figure 3.9. Temperature-time history, District 5 Cube.

3.6.4 District 6 Cube construction

The D6 cube was constructed on February 21, 2012 in Moundsville, WV using Class B Modified concrete with 658 pounds per cubic yard (390 kg/m^3) Type I cement. Temperature sensors were installed at 15 different locations inside the wooden formwork and concrete temperatures was recorded up to 28 days (Figure 3.10). The slump of the concrete was measured 3 inches (75 mm) at first, and $1\frac{3}{4}$ inch (45 mm) after about 45 minutes. The air content was 4.8% and the initial temperature was recorded 67°F (19°).

For this construction, additional temperature sensors was placed on the side surface as close as possible to the timber formwork (approximately $\frac{1}{2}$ inch (13 mm) from the surface) to determine the effect of the concrete cover on temperature differentials between the core and the surface of the cube, similar to the D5 Cube case which was using steel formwork.

The maximum temperature (T3) recorded at the center inside the cube was 156°F (69°C) appeared between 25 to 42 hours. The side surface temperature (T15) recorded at the same time period was decreasing from 133°F (56°C) to 124°F (51°C) and the maximum

temperature differential between the center sensor and the side sensor was about 32°F (18°C) at around 42 hours after concrete placement. However, the largest temperature differential was 45°F between the bottom sensor (T8) and the center sensor (T3) at around 38 hours after casting. It can be observed that the temperature difference between the top sensor (T1) and the center sensor jumps instantaneously up to 20°F when the insulation blanket was removed during 4-day core sampling (Figure 3.11).

Figure 3.10. District 6 Cube (a) instrumentation (b) curing blankets.

Figure 3.11. Temperature monitoring, District 6 Cube.

The cube was covered with an insulation blanket right after concrete placement and protected against daily temperature changes. It can be clearly observed from sensor #6

(located close to top surface) reading that the surface temperature fluctuates when the blanket was removed after about two weeks (Figure 3.11).

3.6.5 Effect of Honeycombing on Concrete Surface Temperatures

Some honeycombed areas on the side surface of the District 6 cube were observed after formwork removal. In literature, honeycombing refers to voids that occurred in concrete due to failure of the mortar to fill the spaces in between coarse-aggregates. It usually occurs due to low slump concrete and/or poor vibration quality (Figure 3.12). Incidentally, sensor T14 and T11 are located on the side surface of the cube where the largest honeycombing observed and therefore the sensor was not completely embedded inside the concrete like the surface sensor on the other side of the cube (T15). The honeycombing made the measured concrete surface temperature close to the surface air temperature inside the wooden formwork. The maximum temperature differential between the center and these three sensors are shown in Figure 3.13.

Figure 3.12. Honeycombing on the concrete side surface.

Figure 3.13. Effect of honeycombing on temperature differential.

3.7 Summary of the Field Results

Six bridge projects and four 6-ft cubes, in total fourteen different elements, from all over the state with different concrete mix design, different types and brands of cement, and different types of aggregates and different size of structures were instrumented with loggers for temperatures monitoring. All of these construction projects were already awarded as non-mass concrete projects, meaning thermal control plan was not required, and the contractors were using regular concrete. A detailed instruction for installing temperature sensors in the structures was prepared and distributed to each bridge project. Moreover, a training seminar on how to install and use the temperature sensors was provided.

These elements were constructed using common practice typically applying on regular concrete construction. Concrete was produced and delivered by local ready mix concrete companies. During the cold winter, hot water was added to the mix as needed, and during the hot summer, ice bags were added to the mixer trucks to control initial concrete temperatures. Wet burlap, white polyethylene and blankets were used on the

forms for curing and protecting concrete surface. The location and the type of the structure, the minimum dimension of the element, the concrete cementitious material content, casting date and time, concrete initial temperature (T_0), the maximum concrete temperature at the center of the structure and the time of the maximum temperature, the maximum temperature differential between the center and the surface, as well as the time of the maximum differential appeared are summarized in Table 3.3.

A total of eleven different WVDOH Class B Concrete mixtures were used during this study. Initial concrete temperatures varied from 52°F (11°C) to 82°F (28°C) and the maximum concrete temperatures varied from 97°F (36°) to 165°F (74°C). The minimum concrete temperature rise at the center of the structures was calculated 34°F (19°C), and the highest center temperature rise was 95°F (53°C). The critical maximum temperature differential encountered was 75°F (42°C) and the lowest was 17°F (9.5°C). Eleven cases exceeded 35°F (20°C) maximum allowable temperature differential limit, and two of them exceed 160°F (70°C) maximum allowable temperature limit. It was observed that the maximum temperature increases with increased concrete initial temperature. Additionally, higher initial concrete temperature increases the possibility of higher temperature differentials. It was concluded that the placement temperature should be controlled under hot weather conditions.

Table 3.3. Summary of the Field Results

Place	Element	Mix Design	Casting	T ₀ °F	Max. Temp °F @ hrs	ΔT °F	Temperature Gradient	
			Date				°F @ hrs	between
Lucille Stalnaker	Pier Stem #1 (Circular)	6 Bag w/13% FA	11/1/2010 9:55 A.M.	65	129 @ 20	64	35 @ 16 40 @ 20	side surface
Lucille Stalnaker	Pier Cap #1	6 Bag w/13% FA	1/17/2011 8:45 A.M.	52	118 @ 28	66	36 @ 15	side surface
Lucille Stalnaker	Abutment #2	6 Bag w/13% FA	3/30/2011 10:00 A.M.	59	118 @ 21	59	34 @ 16	bottom surface
5 th Avenue	Footer #2	6 Bag w/45% Slag	2/3/2011 10:00 A.M.	62	99 @ 57	37	18@18 38@110	side surface
5 th Avenue	Pier Stem#2	7 Bag	2/11/2011 1:00 P.M.	68	158 @37	90	25 @ 15 41 @91	side surface
5 th Avenue	Pier Cap #2	7 Bag	3/28/2011 11:00 A.M.	69	163 @ 35	94	35 @ 18 76 @ 66	side surface
5 th Avenue	Abutment #2	6 Bag w/45% Slag	4/1/2011 10:00 A.M.	65	99 @ 30	34	20 @ 24	side surface
Clear Fork Arch #2	Abutment#1 Footing	6 Bag w/14% FA	1/18/2011 9:10 A.M.	55	122 @ 23	67	35 @ 28 52 @ 41	side surface
Clear Fork Arch #2	Abutment#2 Stem	6 Bag w/14% FA	2/15/2011 12:50 P.M.	56	115 @ 27	59	35 @ 17 40 @ 22	top surface
Clear Fork Arch #1	Abutment #2	6 Bag	1/4/2011	56	97 @ 18	41	25 @ 15	bottom surface
S. Mineral Wells	Pier2Stem2	6 Bag w/14% FA	7/20/2011 6:10 A.M.	78	156 @ 25	78	38 @ 48	side surface
S. Mineral Wells	Pier Cap #2	6 Bag w/14% FA	8/22/2011 7:30 A.M.	81	145 @ 23	64	31 @ 23	side surface
S. Mineral Wells	Abutment#2 Stem	6 Bag w/13% FA	11/02/2011 12:30 P.M.	68	116 @ 17	48	32 @12	bottom surface
Ices Ferry	Pier Cap #2	7 Bag w/12% FA	10/20/2011 12:30 P.M.	65	135 @ 21	70	31 @ 22	side surface
D#1	6-ft Cube	6 Bag w/14% FA	08/15/2011 1:00 P.M	81	145 @ 24	64	32 @ 45	side surface
D#9	6-ft Cube	6 Bag	08/26/2011 1:00 P.M	82	165 @ 31	83	38 @ 35	top surface
D#5	6-ft Cube	6 Bag w/25% Slag	09/15/2011 1:00 P.M	78	149 @ 28	71	35 @ 29 43 @ 45	side surface
D#6	6-ft Cube	7 Bag	02/21/2012 1:00 P.M	67	156 @ 20	89	39 @31	bottom surface

The temperature-time histories from the structures were analyzed and the observations are summarized below:

- Supplementary cementitious materials significantly reduce the temperature rise and the peak concrete temperatures. The least maximum concrete temperature was obtained through Class B GGBFS concrete mixture, 6 bag concrete mix with 45 percent slag replacement. Incorporation of fly ash up to 15 percent appears to be effective reducing concrete peak temperatures.

- Peak concrete temperatures are significantly affected by initial concrete temperatures, generally they were successfully kept around mid 60's (60°F) during the winter time and low 80's (80°F) during the summer time.
- The largest temperature differential occurs most of the time at the side surface of the structure. Bottom temperature appeared to be critical for bridge abutments, and temperature of the sub-base greatly influenced the maximum temperature differential which may exceed 35°F (20°C) at very early ages (as low as 12 hours after concrete placement).
- When concrete surfaces were not protected well enough due to insufficient formwork insulation, especially during the winter conditions, outer surface of the structure remained cold while core temperatures were rising, regardless of the structure type and structure dimensions. As a result, concrete differentials were reaching 35°F (20°C) in less than 24 hours after concrete placement. Temperature differentials could be effectively reduced with adequate insulated formwork.
- Early formwork removal causes faster temperature drop on the exposed surfaces.
- Effect of the formwork type should be considered, as well as the effect of the insulation blankets and curing method.
- Sensor location accuracy has a great effect on the temperature-time history. Temperature difference between the sensor located at the very surface and located inside the concrete cover may have up to 12°F (7°C) temperature differential (D5 Cube sensor #14 - 0.5 inch (13 mm) (inside the surface and #11- 2 inch (51 mm) inside the surface). Location of each sensor has to be verified before concrete placement.

In addition to that, factors affecting concrete temperature development are summarized in Figure 3.14. Concrete temperature is mainly affected by the mix design, structure geometry, formwork type and ambient temperature. These factors will be considered in the thermal analysis presented in Chapter 4.

Figure 3.14. Factors affecting concrete temperature development.

4 NUMERICAL MODELING

4.1 Introduction

In recent years, it has become a common practice for contractors to use concrete with relatively high cement content increase the rate of strength gain in order to reduce formwork removal time, thus accelerating construction schedules. Concrete placements of large structures with increased amount of cement contents result in higher peak temperatures as well as higher temperature differentials between the concrete surface and the interior. It is well known that high thermal differentials can result in large temperature-induced stresses and increases the risk of early age cracking. Therefore, prediction of temperature-time histories in mass concrete has always been of great interest for both contractors and project engineers. Most important properties for modeling temperature development inside the concrete element besides cement hydration can be listed as the thermal properties of the concrete, geometry of the structure, formwork, insulation materials and ambient settings (Gajda J. , 2007) (Emborg, 1998).

4.2 Temperature prediction methods

There are several different tabular, empirical and numerical methods, being used to predict the maximum temperatures and temperature differentials in mass concrete structures. Earliest methods for predicting temperatures in concrete structures are tabular methods which were using simplified finite difference solutions to calculate the temperatures for single nodes by step-by-step time increments. Carlson's method (Carlson, 1937) and Schmidt's method are the two simple procedures that were applied for mass concrete structures during pre-computer era (Rawhouser, 1945). In both method, concrete was divided into small elements and the temperature of each element was found as the

average of the temperature of adjacent elements, plus adiabatic temperature rise due to heat generation during the time step. ACI 207.2R also provides a detailed example of how to use the Schmidt's method to calculate the temperature rise and the gradients (ACI Committee 207, 2007). Another simple method suggested by ACI is a graphical solution method which is based on empirical results for different types of concrete containing 376 pounds of cement per cubic yard (222 kg/m^3) to predict maximum temperature in mass concrete (ACI Committee 207, 2007).

4.2.1 Computational methods

There are also several models associated with numerical methods used for temperature predictions in mass concrete. Ballim (2003) developed a finite difference method (FDM) for predicting temperature-time histories. A practical spreadsheet program was designed to solve the PDE that governs the heat equation and perform a 2D transient heat conduction analysis. FDM was implemented using a macro generated on Microsoft Excel to predict temperature histories in mass concrete. For this model, the third dimension of the structure was assumed to be much larger than the other dimensions and the heat was assumed to be unaffected over the third dimension. It can be simply applied to rectangular elements which is typical for most bridge pier caps and abutments (Ballim, 2004). The rate of heat generation was obtained experimentally using adiabatic calorimeter and a maturity based heat rate data was used as input into the numerical model (Gibbon, Ballim, & Grieve, 1997). In addition, equivalent age method was used to generate the maturity based heat data so that internal heat generation and degree of hydration at any position in the concrete element would be distinguished by the temperature-time history at that point (Ballim & Graham, 2003). Other necessary thermal properties such as concrete

conductivity, specific heat capacity were selected as constant values based on the concrete mix design properties. Furthermore, a simplified model was implemented to approximate the ambient temperature (T_A) at any time using daily minimum (T_{min}) and maximum (T_{max}) temperatures (Ballim, 2004):

$$T_A = -\sin\left(\frac{2\pi(t_d + t_w)}{24}\right)\left(\frac{T_{max} - T_{min}}{2}\right) + \left(\frac{T_{max} + T_{min}}{2}\right) \quad 4.1$$

where, t_d is the time of the day (0-24 h) and t_w is the time at which minimum overnight temperature occurs. The program does not take into account other ambient settings such as cloud cover, wind effect or humidity.

Another FDM based software program that can perform thermal analysis in mass concrete elements was developed by University of Texas at Austin. This software is called “ConcreteWorks” and incorporates effects of the constituent materials properties, concrete mix proportion, structure geometry, formwork type, subgrade material, curing options and environmental conditions (Riding K. , 2007).

The thermal conductivity was assumed to be changing linearly with the degree of hydration (α). It decreases from 1.33 times the ultimate thermal conductivity (k_{uc}) to the thermal conductivity (k_c).

$$k_c(\alpha) = k_{uc}(1.33 - 0.33\alpha) \quad 4.2$$

The specific heat capacity of concrete (C_p) was modeled based on degree of hydration, mixture proportions, and temperature.

$$C_p = \frac{1}{\rho}(W_c\alpha C_{ref} + W_c(1 - \alpha)C_c + W_aC_a + W_wC_w) \quad 4.3$$

where, W_c , W_a , W_w are the amount by weight of cement, aggregate and water in kg/m^3 , C_c , C_a , C_w are specific heat of cement, aggregate and water in $\text{J/kg}^\circ\text{C}$ and C_{ref} is specific heat of hydrated cement in $\text{J/kg}^\circ\text{C}$ that can be calculated by: $8.4T_c + 339$.

The internal heat generation was handled by creating a heat of hydration curve using pre-defined hydration parameters based on the concrete mix design and mainly cement composition. Element geometries were pre-defined, such as rectangular column, pier cap and footing. The program performs two-dimensional analysis, but for the footing case, three-dimensional analysis can be also performed. Some elements can be modeled as either submerged or with soil on the sides, such as an underwater column or a footing with clay or soil serving as the formwork (Folliard, et al., 2008).

Furthermore, there are comprehensive prediction models using commercial software programs. Tia et al. performed thermal and structural analysis of bridge footing element using finite element method (FEM) based TNO DIANA model in order to predict temperature profiles, resulting temperature differentials and associated thermal strain and stress. Additionally, a method was developed for determining the effects of insulation on temperature development. The required amount of insulation for bridge footings to prevent early-age cracking were studied (Do, Lawrance, Tia, & Bergin, 2013) (Do, Lawrance, Tia, & Bergin, 2014).

Another FEM based computer simulation software is ANSYS. Malkawi et al (2003) used ANSYS to analyze the thermal behaviour of a roller compacted dam (Malkawi, Mutasher, & Qui, 2003). Some other commercial programs that were used in infrastructure projects, especially in European countries, can be listed as 4C Temp&Stress and b4cast (Gokce, Koyama, Tsuchiya, & Gencoglu, 2009) (Shaw, Jahren, Wang, & Li, 2014).

In this study, a three-dimensional numerical analysis method was developed using finite volume method (FVM) in which variable heat conductivity and capacity can be handled at early ages. MATLAB® was employed to generate finite volume numerical model to solve the governing heat transfer equation (Appendix E). This model was developed to improve the thermal analysis capabilities of the pre-existing programs such as the spreadsheet program by Ballim (2003). Temperature-time histories data collected from actual bridge constructions and the 6-ft cube castings were presented in comparison with the analysis results. Furthermore, sensitivity analyses were conducted with regards to the effect of early age thermal properties, specific heat and thermal conductivity.

4.3 Basic principles for the temperature prediction model

Thermal behavior of hardening concrete is a transient heat transfer problem and it can be described by the partial differential equation (PDE) shown as:

$$\rho c_p \frac{\partial T}{\partial t} = \frac{\partial}{\partial x} \left(k_x \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left(k_y \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left(k_z \frac{\partial T}{\partial z} \right) + \dot{q} \quad 4.4$$

where, ρ is the density of the concrete, c_p is the specific heat, k_x , k_y , k_z are thermal conductivities in the x,y,z-directions, $\dot{q}$ is the rate of internal heat generation and x, y and z are the coordinates in the element.

Different discretization methods can be used to solve Equation 4.4 , such as finite differences, finite elements and finite volume methods. These methods are based on the idea of discretizing the PDE with proper boundary conditions and then solving the problem with computational methods. Finite difference method is the oldest discretization method that can be used to approximate derivatives by transforming the differential equation into a set of algebraic equations on a grid of mesh. Finite element method is commonly used for structural stress analysis using simple piecewise functions to describe the local

variations of unknown variables. Finite volume method considers a set of regions, or elements, and considers the average temperature in each region. This method is locally conservative because it is based on energy balance approach. A local balance is obtained on each control volume considering heat flux over the boundaries and heat generation within the control volume. The finite volume and finite difference discretization for uniform one-dimensional meshes provides the same results. However, finite volume method is more effective for 2D and 3D problems.

In this study, finite volume method was implemented to solve Equation 4.4 in order to simulate thermal behavior of large bridge elements at early ages. First, thermal properties of concrete were approximated using predetermined models from literature. Those properties are mainly associated by the proportions of the constituent materials and their properties. Concrete is simply a mix of water, cement and aggregates. When first mixed, concrete is a dense suspension. During early stages of the cement hydration, it transforms into a solid durable mass while mechanical properties as well as thermal properties rapidly evolve. The concrete material properties used in the analysis are described as follows:

4.3.1 Density

Density of concrete usually can simply be measured in the fresh state using the total weight of batched materials and the absolute volume. Typical density values for normal weight concrete vary between 140 to 150 pounds per cubic feet (2,240 to 2,400 kg/m³) [68].

4.3.2 Thermal conductivity

Thermal conductivity of concrete (k_c) is greatly influenced by the types of aggregates and the concrete density as well as temperature and moisture conditions of the specimen (Kim, Jeon, Kim, & Yang, 2003). Therefore, values for concrete thermal conductivity reported in the literature vary significantly. Since aggregates comprise 60% to 80% of a typical concrete mix, the thermal conductivity of concrete is mostly affected by the aggregates. ACI Committee 207 presented a table of the typical thermal conductivity values for mass concrete mixtures selected by type of aggregates (Table 4.1).

Table 4.1. Thermal Conductivity of Mass Concrete (ACI Committee 207, 2007)

Aggregate Type	Thermal conductivity, BTU/h-ft-°F (W/m-K)
Quartzite	2 (3.46)
Dolomite	1.83 (3.17)
Limestone	1.5 (2.6) to 1.91 (3.32)
Granite	1.5 (2.6) to 1.58 (2.74)
Rhyolite	1.25 (2.16)
Basalt	1.08 (1.87) to 1.25 (2.16)

Thermal conductivity values of different concrete mixtures that were used at various mass concrete projects were also summarized in that report. Depending on the coarse aggregate type used in the mix design, thermal conductivity values were measured between 0.94 BTU/h-ft-°F (1.63 W/m-K) to 2.13 BTU/h-ft-°F (3.68 W/m-K) following CRD-C 44 method (ACI Committee 207, 2007).

Khan (2002) measured thermal conductivity of concrete specimen batched with different aggregates using hot wire method. According to his results, concrete made with aggregates with less thermal conductivity (limestone) produced lowest value, 1.17 BTU/h-ft-°F (2.03 W/m-K) and the concrete with more conductive aggregates (quartzite) produced highest value 1.6 BTU/h-ft-°F (2.77 W/m-K). It was also reported that the moisture content

of the concrete had a significant influence on the thermal conductivity of concrete. Thermal conductivity values of fully saturated quartzite concrete specimen were measured 2.41 BTU/h-ft-°F (4.18 W/m-K) and limestone concrete specimen were measured 1.69 BTU/h-ft-°F (2.92 W/m-K) (Khan, 2002). Kim et al (2003) measured thermal conductivity of concrete batched with local river sand and crushed stone aggregates at 3, 7, 14, and 28 days using a probe method and reported values of 1.33 BTU/h-ft-°F (2.3 W/m-K), 1.36 BTU/h-ft-°F (2.35 W/m-K), 1.34 BTU/h-ft-°F (2.33 W/m-K) and 1.32 BTU/h-ft-°F (2.29 W/m-K), respectively. It was concluded that thermal conductivities were not affected by age, after concrete reached a certain maturity.

Wadsö et al. (2012) produced concrete mixtures with “special” aggregates such as magnetite, graphite and copper to increase thermal conductivity and obtained values up to 70% higher than regular concrete with a thermal conductivity value of 1.3 BTU/h-ft-°F (2.24 W/m-K) (Wadsö, Karlsson, & Tammo).

Thermal conductivity of concrete (k_c) can also be estimated using average thermal conductivities of the constituents by mass per unit volume used in the mix shown as:

$$k_c = \frac{W_{cem}k_{cem} + W_{agg}k_{agg} + W_wk_w}{W_{cem} + W_{agg} + W_w} \quad 4.5$$

where, W_{cem} , W_{agg} and W_w represent the mass (by weight) and k_{cem} , k_{agg} and k_w are the thermal conductivity of cement, aggregate and water, respectively.

Alternatively, thermal conductivity of concrete can be determined using the Hashin-Shtrikman method (Hashin & Shtrikman, 1962) which can be written for a two-phase composite material such as concrete (aggregate and hydrated cement paste). The lower (k_L) and upper (k_U) bounds of thermal conductivity can be estimated assuming that the thermal conductivity of the aggregates (k_{agg}) and the cement paste (k_{cem}) are known.

Also, volume fractions of the aggregates (v_{agg}) and the cement paste (v_{cem}) need to be calculated from the concrete composition. However, the influence of air voids is not taken into account in this model.

$$k_L = k_{cem} + \frac{v_{agg}}{\frac{1}{k_{agg} - k_{cem}} + \frac{v_{cem}}{3k_{cem}}} \quad 4.6$$

$$k_U = k_{agg} + \frac{v_{cem}}{\frac{1}{k_{cem} - k_{agg}} + \frac{v_{agg}}{3k_{agg}}} \quad 4.7$$

Bentz (2007) applied Hashin-Shtrikman method to estimate thermal conductivity of regular limestone concrete assuming a value of 0.58 BTU/h-ft-°F (1.0 W/m-K) for the hydrated cement paste and obtained a concrete thermal conductivity value of 1.21 BTU/h-ft-°F (2.1 W/m-K) (Bentz, 2007). Wadsö et al. (2012) also successfully used Hashin-Shtrikman method to predict concrete thermal conductivity of 1.31 BTU/h-ft-°F (2.27 W/m-K) by assuming a value of 0.34 BTU/h-ft-°F (0.58 W/m-K) for cement paste.

De Schutter and Taerwe (1995) developed a test method to determine thermal diffusivity of concrete during hardening and concluded that thermal diffusivity decreases linearly with increasing degree of hydration. Since thermal diffusivity is proportional to thermal conductivity, an equation was proposed for thermal conductivity as shown (De Schutter & Taerwe, 1995):

$$k_c(\alpha) = k_{uc}(1.10 - 0.10\alpha) \quad 4.8$$

where, $k_c(\alpha)$ is the current thermal conductivity, k_{uc} is the ultimate thermal conductivity and α is the degree of hydration.

4.3.3 Specific heat

Specific heat capacity of hardened concrete is generally influenced by the amount and type of aggregates, water to cement ratio of the mix, temperature and moisture level (De Schutter, 2002)). According to ACI 207 specific heat values vary between 0.20 to 0.25 BTU/lb-°F (840 to 1050 J/kg-°C). Based on experimental results, specific heat has been reported to decrease about 20% linearly with time. DeSchutter and Taerwe (1995) also reported a linear decrease with regards to the degree of hydration and formulated an experimental relationship given as (De Schutter & Taerwe , 1995):

$$c_p(\alpha) = c_{up}(1.15 - 0.15\alpha) \quad 4.9$$

Where, $c_p(\alpha)$ is the current specific heat and c_{up} is the ultimate specific heat.

Most commonly, specific heat of concrete is being calculated by adding specific heat of each constituent material per mass fraction. The specific heat of water is 1 BTU/lb-°F (4186 J/kg-K) which is higher than any other constituent in a concrete mixture. Therefore, water content in the mixture plays a very important role.

Bentz et al (2011) measured specific heat capacity of individual constituents using transient plane source method. According to the results, specific heat capacity of TYPE I/II cement was found 0.18 BTU/lb-°F (750 J/kg-K), Class F fly ash 0.17 BTU/lb-°F (720 J/kg-K), siliceous sand 0.17 BTU/lb-°F (710 J/kg-K), limestone sand 0.18 BTU/lb-°F (760 J/kg-K). A law of mixtures was proposed to estimate specific heat capacity of concrete mixtures as follows:

$$c_{up} = 4.18M_f^{water} + 0.75M_f^{cem} + 0.72M_f^{flyash} + 0.71M_f^{fine} + 0.76M_f^{coarse} \quad 4.10$$

where; M_f^{water} , M_f^{cem} , M_f^{flyash} , M_f^{fine} , M_f^{coarse} are the mass fraction of water, cement, fly ash, fine aggregate and coarse aggregate, respectively (Bentz, 2007).

4.4 Concrete heat generation

The rate of heat generation of concrete is the key point to predict the temperature profiles precisely. The heat development in concrete starts immediately after water is added to the cementitious material and continues until it reaches the steady state. Each concrete mixture shows different hydration characteristics based on the cement type and chemical composition, supplementary cementitious materials used in the mix design, water-cementitious ratio, initial concrete temperature, curing history and chemical admixtures, such as accelerators and retarding agents. The amount of heat generation in concrete mainly depends on the composition, fineness and quantity of the cement and the cement type. Cement types containing higher percentage of C3S, and C3A generate naturally more heat. Besides, increasing the cement fineness will accelerate the hydration since the specific surface area is increased (ACI Committee 207, 2005). Additionally, the water to cement ratio (w/c) is an important factor that directly affects the ultimate degree of hydration of concrete (Bensted, 1983).

Curing temperature is another key factor affecting the concrete hydration because it influences the rate of the reaction. Hydration occurs faster with increased temperatures. In practice, concrete is produced with an initial concrete temperature between 50°F (10°C) and 90°F (32°C) depending on the seasons of the year. Concrete mixtures with higher initial temperatures reach the ultimate hydration faster than concrete mixes with lower initial temperatures. However very high early temperatures, such as 122°F (50°C) and above may have negative effect on the hydration process by reducing the ultimate degree

of hydration of the concrete mix (Carino, "The Maturity Method", Chapter 5 in Handbook on Nondestructive Testing of Concrete, 2004).

Use of cementitious materials and natural pozzolans such as fly ash and slag is a very common practice in the concrete industry. Cementitious materials do not react primarily with water but they react with calcium hydroxides formed during cement hydration and silicas to form hydration products (Neville, 1995). There are many advantages of incorporating cementitious materials as supplementary or as partially replacement for cement. In practice, replacement of cement with such cementitious materials in the concrete mix is preferred to lower total heat production and thus the maximum temperatures.

Another factor that affects hydration development is chemical admixtures added to the concrete mix. The chemical admixtures used in concrete such as plasticizers, water-reducers, air entraining agents and retarders have very little effect on the total heat generated, however they affect the reaction rate by slowing down or speeding up the hydration (ACI Committee 207, 2007).

4.4.1 Quantifying maximum available heat within concrete

The hydration of cement is an exothermic reaction which releases energy of up to 500 kilojoules per kilogram of cement; more than enough to boil a liter of water at room temperature. This reaction is a complex exothermic process that starts when cement mixed with water. In presence of water the silicates and the aluminates starts forming hydration products releasing a significant amount of heat to the environment. The total heat generated during the hydration depends on the amount and composition of the cement and the amount of water present.

Each cement compound has a unique contribution to the heat generation. A very common method to determine the total heat of hydration of cement (H_{cem}) at complete hydration is known as Bogue method (Bogue, 1955). The heat contribution of each compound in terms of total cement content and the total available heat can be calculated using the heat of hydration values as recommended by this equation:

$$H_{cem} = 500(C3S) + 200(C2S) + 866(C3A) + 420(C4AF) + 624(SO_3) + 1186(CaO_{free}) + 850(MgO) \quad 4.11$$

where (H_{cem}) is total heat of hydration of cement (kJ/kg), $C3S$ is tricalcium silicate (alite), $C2S$ is dicalcium silicate (belite), $C3A$ is tricalcium aluminate (aluminate), $C4AF$ is tetracalcium aluminoferrite (ferrite), SO_3 is sulfur trioxide (sulfate) and CaO_{free} free lime in terms of weight ratio of the cement content.

In West Virginia, TYPE I and TYPE I/II cements are widely available and being used in many concrete applications. The chemical compositions of cement from different sources that are being used in West Virginia are shown in Table 4.2.

Table 4.2. Chemical Composition of Locally Available Cements

Test Value, %	Armstrong, TYPE I ¹	Essrock, TYPE I/II ²	Cemex TYPE I ³	LeHigh TYPE I/II ⁴
CaO	63.81	62.90	63.70	61.25
SiO ₂	19.76	20.60	20.50	19.25
Al ₂ O ₃	5.35	4.60	4.40	4.79
Fe ₂ O ₃	3.87	3.10	3.30	3.32
MgO	1.54	1.00	3.20	3.23
SO ₃	2.78	3.00	3.20	1.09
Loss of ignition	1.00	1.40	1.26	2.64
C3S	60.0	55.5	60.0	53.1
C2S	11.2	17.1	14.0	15.1
C3A	7.6	7.0	6.0	7.1
C4AF	11.8	9.4	10.0	10.1
Na ₂ O Equiv.	0.52	0.73	0.63	0.45
Blaine Fineness, cm ² /g	3605	3850	3980	3690
1. Armstrong Cement&Supply Certification Letter, March 2006. 2. Essrock Cement Certification, 4/14/2011. 3. Cemex/Cosmos Cement Mill Test Report, 10/20/2011. 4. LeHigh Mill Test Certificate Report, 9/7/2011.				

The heat contribution for each cement compound and the total heat of hydration is calculated using Equation 4.11 and presented in Table 4.3. The amount of free lime (CaO_{free}) which is usually less than 1% in Portland cement was disregarded in the calculations. According to the Bogue method, a typical cement that is used in West Virginia produces total heat of 457 to 474 J/g.

Table 4.3. The Heat Contribution of Each Compound in Terms of Total Cement Content

Contribution to heat of cement, J/g	C3S	C2S	C3A	C4AF	MgO	SO3	Total (Hcem)
Armstrong TYPE I	302.4	29.2	66.1	49.3	13.1	17.3	462.5
Essrock, TYPE I/II	279.4	44.4	60.2	39.5	22.1	18.7	464.3
Cemex, TYPE I/II	302.0	34.8	52.7	42.0	22.1	20.0	473.6
LeHigh, TYPE I/II	257.5	42.9	61.3	42.3	33.3	20.1	457.5

The total heat that is available (H_u) in a concrete mix design after incorporating effects of the other cementitious materials such as slag and fly ash can be estimated in kJ/kg using the following equation (Riding, Poole, Folliard, Juenger, & Schindler, 2012):

$$H_u = H_{cem}p_{cem} + 461p_{slag} + 1800p_{FA-CaO}p_{FA} + 330p_{SF} \quad 4.12$$

where, p_{cem} , p_{slag} , p_{FA-CaO} , p_{FA} and p_{SF} are the mass fractions of cement, slag, CaO in fly ash, fly ash and silica fume of the total cementitious content.

4.4.2 Degree of hydration

The degree of hydration (α) can be defined as the ratio between the hydrated cementitious material and the total cementitious material and it is a function of time and temperature. Degree of hydration is equal to one in case of complete hydration of all the cement. It can be measured experimentally or it can be assumed as the ratio of the heat generated by the mixture $Q(t)$ to the total heat available in the mixture (H_u), also referred to as Q_{ult} (Riding, Poole, Folliard, Juenger, & Schindler, 2012).

$$\alpha(t) = \frac{Q(t)}{Q_{ult}} \quad 4.13$$

Since degree of hydration can be estimated using the heat generated by the specific mix design, it can be defined as degree of heat generation. Methods to quantify concrete heat generation will be explained and the calculated degree of heat generation for commonly used concrete mixtures will be presented in the following section.

4.4.3 Methods for obtaining heat generation

There are mathematical models to express the heat generation of concrete as well as experimental methods to determine concrete heat of hydration. In this section some of the commonly used methods will be summarized.

4.4.3.1 Experimental methods to determine concrete heat of hydration

Heat generation in concrete can be obtained experimentally via calorimetric tests. Isothermal, semi-adiabatic and adiabatic calorimetry are the standardized techniques that has being used by the cement and concrete industry. Isothermal calorimetry method measures the heat production rate at constant temperature conditions. Either paste or mortar specimens can be prepared to determine total heat of hydration of the cementitious materials up to 7 days (ASTM Standard C1679, 2009). Studies show that isothermal calorimetry is an efficient technique to measure concrete heat generation (Xiong & Van Breugel, 2001) (Wadso, 2003).

Semi-adiabatic calorimeters measure temperature to calculate the heat produced during hydration. This method principally relies on the insulation around the sample to slow down the rate of heat loss to the surroundings. The heat loss throughout the experiment has to be calculated accurately in order to obtain adiabatic temperature rise. Additionally, the heat capacity of the system has to be measured via calibration procedures.

This method is described in several different standards and procedures (RILEM TC-119, 1997) (NT BUILD 388, 1992). There are four main parameters to be determined in order to reconstruct the adiabatic temperature rise curve when using semi-adiabatic calorimetry; the heat capacity of the concrete sample and the calorimeter, the activation energy (E_a) of the concrete mix, and the heat loss coefficient (thermal conductivity) of the calorimeter.

The heat produced during hydration is partly being used to increase the temperature of the sample and the insulation material inside the calorimeter, and the remainder is released to the environment through the insulation. To be able to calculate the total heat loss, heat capacity of the calorimeter and the heat loss coefficient needs to be determined. When calculating the total heat generation, the influence of the concrete temperature on the hydration can be considered by means of maturity method using Arrhenius equation. Activation energy is needed to calculate the equivalent age of concrete which is a simple correction on the time parameter. The real time, t , recorded during experiment is converted into equivalent age (t_e), by applying following Arrhenius equation:

$$t_e = \sum e^{-\frac{E_a}{R} \left(\frac{1}{296} - \frac{1}{273+T_s} \right)} \Delta t \quad 4.14$$

where: t_e is equivalent age, E_a is activation energy, R is the gas constant, T_s is specimen temperature and Δt the time interval. The activation energy values are mix design specific and can be obtained from strength development curves at developed at different curing temperatures; high (104°F), low (50°F), and laboratory temperature (73°F).

Adiabatic calorimetry method relies on the principle where no heat transfer between the sample and the environment occurs. In an adiabatic calorimeter, the core temperature of the sample is measured and feed it back to a heating or cooling system that keeps the surrounding environment at that same temperature. Practically, true adiabatic conditions

are very difficult and costly to achieve. Therefore, the calorimeters are designed such as where heat losses are prevented or minimized by controlling the temperature of the concrete samples surroundings. In order to satisfy that, the environment where the sample kept is being heated or cooled externally. Although adiabatic calorimeters are not commercially available Ballim et al. (1997) described the design and instructions for use of a low-cost, computer-controlled adiabatic calorimeter that can be used for the determination of the heat generation of concrete and mortar mixtures.

4.4.3.2 Modeling of hydration and microstructure development

There are mathematical and empirical models to express the rate of heat generation of concrete. Maekawa and Kishi (1995) proposed a heat generation model based on the heat generation rate of each mineral.

$$\bar{H}_i = \gamma_i \beta_i \mu \bar{H}_{i,T_0} \exp\left(\frac{E_i}{R} \left(\frac{1}{T} - \frac{1}{T_0}\right)\right) \quad 4.15$$

where, $\bar{H}_i$ is the rate of heat generation, γ is the parameter for delaying effect of chemical admixture and fly ash during early hydration project, β_i for expressing the reduction in heat generation due to the reduced availability of free water, μ for expressing the effects of mineral compositions.

Schindler and Folliard (2005) presented a best fit mathematical model for rate of heat generation shown in equation 4.16. The rate of heat generation Q_H is dependent on total available heat for reaction, degree of hydration, time and concrete temperature.

$$Q_H(t) = H_u C_c \left(\frac{\tau}{t_e}\right)^\beta \left(\frac{\beta}{t_e}\right) \alpha(t_e) \exp\left(\frac{E_a}{R} \left(\frac{1}{273 + T_r} - \frac{1}{273 + T_c(t)}\right)\right) \quad 4.16$$

where H_u is total heat available for reaction (kJ/kg), C_c is weight of cementing material per unit volume (Kg/m^3), $\alpha(t_e)$ is degree of hydration, τ and β are hydration parameters,

t_e is equivalent age of concrete, R is universal gas constant, $T_c(t)$ is concrete temperature at time t , E_a is activation energy (J/mol). As discussed previously, ConcreteWorks was developed to predict the thermal behavior of concrete considering the heat conduction in the concrete, the heat generation from the hydration process, and the heat exchanged at the boundary of the structural element (Riding K. , 2007).

Other hydration models have been developed based on the evolution of the microstructure of cement-based materials. Van Breugel (1991) developed HYMOSTRUC3D, a three-dimensional numerical model based on the formation of microstructure during hydration process. In this model, cement hydration can be simulated as a function of the particle size distribution and the chemical composition of the cement, water to cement ratio and temperature (Van Breugel, 1991) (Koenders, 1997) (Ye, 2003).

Bentz and Garboczi (1990) from National Institute of Standards and Technology (NIST) used digital image-based microstructure (SEM) to simulate cement hydration (Bentz & Garboczi, 1990). Bentz et al (1998) modified the three-dimensional model to incorporate the pozzolanic reaction of silica fume and to simulate hydration under adiabatic conditions. Later on, Bullard at NIST developed another simulation program, called HydratiCA, based on a probability analysis of the hydration process. This program uses very small time steps, 0.2 mili-seconds, to make detailed predictions of the chemical and structural changes during hydration (Bullard, 2007).

4.5 Experimental study to determine heat generation

During this study, tests were conducted to determine heat of hydration of the concrete mixtures commonly used in West Virginia. Constituent materials of each mix

design were shipped from different locations in West Virginia to produce identical concrete that was used in pre-determined bridge projects and 6-ft cubes (Table 4.4).

Table 4.4. Mix Design Proportions, per cubic yard (kg/m³)

Items, lb (kg)	Class B Fly Ash	Class B GGBFS	Class B	Class B Modified	Class B Fly Ash Mod.
Cement (TYPE I/II)	470 ^a (278)	423 ^b (250)	564 ^a (334)	658 ^a (390)	564 ^c (334)
Fly Ash (TYPE F)	75 (45)	--	--	--	75 (45)
GGBFS (Grade 100)	--	141 (83)	--	--	--
Water	245 (145)	276 (163)	262 (155)	260 (154)	240 (142)
Coarse Aggregate (#57)	1775 (1050)	1815 (1074)	1723 (1019)	1750 (1035)	1740 (1030)
Fine Aggregate	1255 (743)	1225 (725)	1299 (769)	1111 (657)	1240 (733)
w/cm	0.45	0.49	0.45	0.40	0.38
Source of the cement used in concrete mix: a. Essrock, b. LeHigh, c. Cemex					

4.5.1 Semi-adiabatic calorimetry

A semi-adiabatic calorimeter was constructed filling an open-headed 55 gallon steel drum with four pounds density pour-in-place polyurethane foam. A small space was left in the center of the drum for a 6 by 12 inch concrete cylinder to fit (The R-value of the foam is 5.6 per inch). PT100 resistance temperature sensors were used to measure the temperatures of the ambient and the concrete sample. The sampling rate was set to be 5 minutes throughout the experiment. Complete test setup can be seen in Figure 4.1.

Figure 4.1. Semi-adiabatic drum calorimeter.

The heat generation of concrete was calculated using a spreadsheet following RILEM TC-119 method (RILEM TC-119, 1997). Theoretically, the heat released by the concrete hydration was broken down into portions as:

$$\Delta Q_t = \Delta Q_{acc} + \Delta Q_{loss} \quad 4.17$$

where, ΔQ_{acc} is the change of accumulated heat that increases specimen temperature during the time interval, Δt , and ΔQ_{loss} is the change of transmitted heat from the concrete discharged outside during the time interval, Δt .

In order to quantify the heat loss during the semi-adiabatic test a cooling factor needs to be determined. Concrete specimen used for the semi-adiabatic test were heated up to 130-140°F (55-60°C) inside an oven and placed back into the semi-adiabatic drum to measure the spontaneous cooling behavior. Using the Newton's cooling law the average cooling factor (α) was calculated as 320 J/°C/h. The total heat generated, $Q(t)$ can be calculated by:

$$\frac{dT}{dt} = a(T_c(t) - T_a) \quad 4.18$$

$$Q(t) = mc_p \left[(T_c(t) - T_0) + a \int_0^t (T_c(t) - T_a) dt \right] \quad 4.19$$

Results from these five concrete mixtures that are being used in West Virginia are given in Figure 4.2. The total heat generated (Q_t) and the rate of heat generation (q_{rate}) were determined and results were plotted versus equivalent age.

Figure 4.2. Heat of hydration test result, semi-adiabatic analysis (a) Class B Fly Ash (b) Class B (c) Class B Slag (d) Class B Modified (e) Class B Fly Ash Modified.

4.5.2 Adiabatic calorimetry

An adiabatic calorimeter similar to the one described in Gibbon et al. was used to determine the adiabatic temperature rise. Detailed information for the setup can be obtained from Lin and Chen (2013).

The total heat generated by the concrete specimen with mass m was calculated from the adiabatic temperature rise measurement such that:

$$Q_t(t) = mc_p[T_c(t) - T_0] \quad 4.20$$

Where, $T_c(t)$ is the temperature of the concrete specimen at time t , during the adiabatic test, and T_0 is the initial temperature. The total heat generated (Q_t) and the rate of heat generation (q) of each mixture were plotted versus equivalent age as shown in Figure 4.3.

Figure 4.3. Heat of hydration test result, adiabatic analysis (a) Class B Fly Ash (b) Class B (c) Class B Slag (d) Class B Modified (e) Class B Fly Ash Modified.

In general test results show that semi-adiabatic calorimetry produces less ultimate heat of hydration compared to adiabatic calorimetry mainly due to an error dependent on the effectiveness of the heat insulation provided in semi-adiabatic calorimeter.

Degree of heat generation development that is being used in Equation 4.8 and Equation 4.9 to estimate the variation in heat conductivity and specific heat for concrete mixtures is calculated and given in Figure 4.4.

Figure 4.4. Degree of heat generation development.

4.6 Boundary conditions of the thermal analysis

Initial values and boundary conditions need to be determined considering field conditions such as initial concrete temperature, ambient temperature, type of insulation and type of formwork used. In the current model, concrete placement temperature was set as the initial temperature and daily ambient temperature was predicted following the model used in Ballim (2004) (Ballim, 2004). The advantage of this model is that the ambient temperatures can be modeled as a function of time based on daily maximum and minimum temperatures which can be easily obtained from weather forecast online.

Surface convection plays an important role predicting temperature distribution in mass concrete. Convection heat transfer occurs between concrete surface and the ambient proportional to the temperature difference and it can be described by Newton's cooling law (Jiji, 2009):

$$q = h(T_c - T_a) \quad 4.21$$

where, q is the heat flux at the surface, h is the convection coefficient, T_c is the concrete surface temperature and T_a is the ambient temperature.

In practice, heat transfer usually occurs as a combination of two mechanisms. Heat conducted through concrete is for example removed from the surface by a combination of convection and radiation. The energy balance can be shown as:

$$-k_x \frac{dT}{dx} = h(T_c - T_a) + \sigma \epsilon (T_c^4 - T_a^4) \quad 4.22$$

where, σ is the Stefan-Boltzmann constant and ϵ is the emissivity of the surface.

Radiation is mainly transmitted by electromagnetic waves. Structure geometry, shape, surface area, orientation and emissivity and absorptivity of surfaces plays an important role in radiation heat transfer (Jiji, 2009).

Ryding (2007) stated that concrete surface temperature can be effected by different types of radiation such as solar and atmospheric radiation and radiation from the surrounding surfaces (Riding K. , 2007). Although, radiation can be critical for exposed concrete slabs and pavements, the effect is minimum in case of mass concrete. Especially, when concrete surfaces are covered with insulation blankets made of light colored and shiny material. Therefore, based on the literature and the above explanation, the radiation effect was neglected in the current model.

4.6.1 Predicting a heat transfer coefficient

During hydration, concrete surfaces lose heat to the surrounding air. If the heated air is constantly replaced by the cooler wind, the rate of heat loss may increase. Therefore, convection coefficient is usually predicted as a function of wind speed. A commonly used relationship for smooth surfaces was proposed by McAdams (1954) (McAdams, 1954):

$$\begin{aligned} h &= 5.7 + 3.8v & \text{for } v \leq 5m/s \\ h &= 7.2v^{0.78} & \text{for } v \geq 5m/s \end{aligned} \quad 4.23$$

Other important parameters affecting heat transfer coefficient at the boundaries are the use of formwork and insulation. Based on the thickness (L_i) and the thermal conductivity (k_i) of the materials used as formwork and insulation layers the efficient heat transfer coefficient (h_{eff}) can be estimated as follows (Riding K. , 2007):

$$h_{eff} = \left[\frac{1}{h} + \sum_{i=1}^n \frac{L_i}{k_i} \right]^{-1} \quad 4.24$$

4.7 Numerical implementation with finite volume method

In this study, the governing equation given in 3D form, together with a set of boundary conditions, was discretized by using finite volume method.

$$\rho c_p \frac{\partial T}{\partial t} = \frac{\partial}{\partial x} \left(k_x \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left(k_y \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left(k_z \frac{\partial T}{\partial z} \right) + \dot{q} \quad 4.25$$

The principle of conservation of energy is applied properly to a finite control volume. The heat transfer at the boundaries was also accounted for balancing the energy in the control volume where temperature variation occurs. In order to obtain a discretized equation at a general node P at the center of the control volume, integration of the governing equation has to be performed with respect to time and space. The general form of the integration over the control volume for transient problems in n -dimensional space as given in literature is (Patankar, 1980) (Versteeg & Malalasekera, 2007):

$$\int_t^{t+\Delta t} \int_{CV} \rho c_p \frac{\partial T}{\partial t} dV dt = \int_t^{t+\Delta t} \int_{CV} \frac{\partial}{\partial n} \left(k_n \frac{\partial T}{\partial n} \right) dV dt + \int_t^{t+\Delta t} \int_{CV} \dot{q} dV dt \quad 4.26$$

4.7.1 Discretization for 2D

The 2D grid used for the discretization of the internal nodes is shown in Figure 4.5. The grid consist a general node (P) at the center and the neighboring nodes to the west (W),

east (E), north (N) and south (S). The control volume is positioned such that the boundaries are located right between two adjacent nodes.

Figure 4.5. Two-dimensional grid for internal nodes.

Integrating the equation over the control volume explicitly gives the discretized solution as:

$$\rho c_{p,i,j} \left(\frac{T_{i,j}^{n+1} - T_{i,j}^n}{\Delta t} \right) \Delta x \Delta y = \left(k_{i+1/2,j} \frac{T_{i+1,j}^n - T_{i,j}^n}{\Delta x} - k_{i-1/2,j} \frac{T_{i,j}^n - T_{i-1,j}^n}{\Delta x} \right) + \left(k_{i,j+1/2} \frac{T_{i,j+1}^n - T_{i,j}^n}{\Delta y} - k_{i,j-1/2} \frac{T_{i,j}^n - T_{i,j-1}^n}{\Delta y} \right) + \dot{q}_{i,j} \Delta x \Delta y \quad 4.27$$

Where, the superscript is the time step and the subscripts indicate the node of the mesh according to Figure 4.5. Upon re-arranging, the standard discretization equation for the new temperature can be written as:

$$T_{i,j}^{n+1} = a_{i+1,j} T_{i+1,j}^n + a_{i-1,j} T_{i-1,j}^n + a_{i,j+1} T_{i,j+1}^n + a_{i,j-1} T_{i,j-1}^n + S_c^n \quad 4.28$$

where S_c^n is referred as the source term in time step, n and, a is a coefficient of variable thermal properties as shown in Table 4.5.

$$S_c^n = T_{i,j}^n \left[1 - (a_{i+1,j} + a_{i-1,j} + a_{i,j+1} + a_{i,j-1}) \right] + \frac{\dot{q}_{i,j}^n \Delta t}{\rho c_{p_{i,j}}} \quad 4.29$$

Table 4.5. Summary of the Coefficients for 2D Discretization

Coefficient	Thermal conductivity	Specific heat
$a_{i+1,j} = \frac{k_{i+1/2,j} \Delta t}{\rho c_{p_{i+1/2,j}} \Delta x^2}$	$k_{i+1/2,j} = \frac{k_{i,j} + k_{i+1,j}}{2}$	$c_{p_{i+1/2,j}} = \frac{c_{p_{i,j}} + c_{p_{i+1,j}}}{2}$
$a_{i-1,j} = \frac{k_{i-1/2,j} \Delta t}{\rho c_{p_{i-1/2,j}} \Delta x^2}$	$k_{i-1/2,j} = \frac{k_{i,j} + k_{i-1,j}}{2}$	$c_{p_{i-1/2,j}} = \frac{c_{p_{i,j}} + c_{p_{i-1,j}}}{2}$
$a_{i,j+1} = \frac{k_{i,j+1/2} \Delta t}{\rho c_{p_{i,j+1/2}} \Delta y^2}$	$k_{i,j+1/2} = \frac{k_{i,j} + k_{i,j+1}}{2}$	$c_{p_{i,j+1/2}} = \frac{c_{p_{i,j}} + c_{p_{i,j+1}}}{2}$
$a_{i,j-1} = \frac{k_{i,j-1/2} \Delta t}{\rho c_{p_{i,j-1/2}} \Delta y^2}$	$k_{i,j-1/2} = \frac{k_{i,j} + k_{i,j-1}}{2}$	$c_{p_{i,j-1/2}} = \frac{c_{p_{i,j}} + c_{p_{i,j-1}}}{2}$

In case of a surface boundary, the volume element represented by the surface node becomes half size $(\Delta x/2 \times \Delta y \times 1)(\Delta x \times \Delta y/2 \times 1)$ and it is subjected to convection from the ambient (T_A^n) and conduction from the adjacent nodes (N, S, E) (Figure 4.6). The heat flux at the surface boundary can be calculated as:

$$k_{i,j} \frac{T_{i+1,j}^n - T_{i-1,j}^n}{2\Delta x} = h(T_A^n - T_{i,j}^n) \quad 4.30$$

The temperature at the fictitious node (W') can be expressed by re-arranging Equation 4.30 as:

$$T_{i-1,j}^n = \frac{2\Delta x h}{k_{i,j}} (T_{i,j}^n - T_A^n) + T_{i+1,j}^n \quad 4.31$$

The general discretized form of the governing equation on the surface boundary can be re-written by using Equation 4.28 and Equation 4.31:

$$T_{i,j}^{n+1} = T_{i+1,j}^n (a_{i+1,j} + a_{i-1,j}) + a_{i,j+1} T_{i,j+1}^n + a_{i,j-1} T_{i,j-1}^n + a_{i-1,j} \frac{2\Delta x h}{k_{i,j}} (T_A^n - T_{i,j}^n) + S_c^n \quad 4.32$$

where S_c is referred as the source term in old time for the surface nodes:

$$S_c^n = T_{i,j}^n [1 - (a_{i+1,j} + a_{i,j+1} + a_{i,j-1})] + T_{i-1,j}^n a_{i-1,j} \frac{2\Delta x h}{k_{i,j}} + \frac{\dot{q}_{i,j}^n \Delta t}{\rho c_{p,i,j}} \quad 4.33$$

Figure 4.6. Two-dimensional grid for surface nodes.

For the bottom surface two separate situations can be considered. First, bottom surface can be treated as a surface node as shown in Figure 4.6. Second, if the structure is placed on top of soil, rock or previously cast concrete, a fictitious node S' with conductivity of k_s and temperature of T_s was added to the grid as shown in Figure 4.7. The temperature of the node S' was assumed to be as the minimum ambient temperature of the previous day (Ballim, 2004).

Figure 4.7. Two-dimensional grid for bottom surface nodes on soil or concrete.

The standard discretization equation for the new temperature can be written as:

$$T_{i,j}^{n+1} = a_{i+1,j}T_{i+1,j}^n + a_{i-1,j}T_{i-1,j}^n + a_{i,j+1}T_{i,j+1}^n + a_{i,j-1}T_{i,j-1}^n + S_c^n \quad 4.34$$

where S_c^n is referred as the source term in old time and a is a coefficient of variable thermal properties as shown in Table 4.5.

$$S_c^n = T_{i,j}^n [1 - (a_{i+1,j} + a_{i-1,j} + a_{i,j+1})] + 2a_{i,j-1} \frac{k_{i,j-1}}{k_{i,j+1}} (T_{i,j-1}^n - T_{i,j}^n) + \frac{\dot{q}_{i,j}^n \Delta t}{\rho c_{p,i,j}} \quad 4.35$$

Finally, temperatures of the corner nodes were simply calculated as the average of two neighboring surface nodes such as:

$$T_{i,j}^{n+1} = (T_{i+1,j}^{n+1} + T_{i,j+1}^{n+1})/2 \quad 4.36$$

After completion of the finite volume formulation the maximum time step that can be used to solve the problem was determined. The smallest primary coefficient of $T_{i,j}^n$ from the new temperature equations was selected to express the stability criterion for this explicit problem; such as:

$$\Delta t \leq \frac{\rho c_p \Delta x^2}{2k} \quad 4.37$$

4.7.2 Discretization for 3D

Two more neighbors T and B (top and bottom) for the z-axis can be added to complete the 3D structure as shown in Figure 4.8. Integrating the governing equation over the control volume explicitly gives the discretized solution for 3D:

$$\begin{aligned} \rho c_p \left(\frac{T_{i,j,k}^{n+1} - T_{i,j,k}^n}{\Delta t} \right) \Delta x \Delta y \Delta z = & \left(\mathbf{k}_{i+1/2,j,k} \frac{T_{i+1,j,k}^n - T_{i,j,k}^n}{\Delta x} - \mathbf{k}_{i-1/2,j,k} \frac{T_{i,j,k}^n - T_{i-1,j,k}^n}{\Delta x} \right) + \\ & \left(\mathbf{k}_{i,j+1/2,k} \frac{T_{i,j+1,k}^n - T_{i,j,k}^n}{\Delta y} - \mathbf{k}_{i,j-1/2,k} \frac{T_{i,j,k}^n - T_{i,j-1,k}^n}{\Delta y} \right) + \\ & \left(\mathbf{k}_{i,j,k+1/2} \frac{T_{i,j,k+1}^n - T_{i,j,k}^n}{\Delta z} - \mathbf{k}_{i,j,k-1/2} \frac{T_{i,j,k}^n - T_{i,j,k-1}^n}{\Delta z} \right) + \dot{q}_{i,j,k}^n \Delta x \Delta y \Delta z \end{aligned} \quad 4.38$$

Figure 4.8. Three-dimensional discretization scheme.

Using Table 4.6 the general discretization equation for interior nodes can be written as:

$$\begin{aligned} T_{i,j,k}^{n+1} = & a_{i+1,j,k} T_{i+1,j,k}^n + a_{i-1,j,k} T_{i-1,j,k}^n + a_{i,j+1,k} T_{i,j+1,k}^n \\ & + a_{i,j-1,k} T_{i,j-1,k}^n + a_{i,j,k+1} T_{i,j,k+1}^n + a_{i,j,k-1} T_{i,j,k-1}^n + S_c^n \end{aligned} \quad 4.39$$

where S_c^n is referred as the source term in old time and a is a coefficient of variable thermal properties as shown in Table 4.6.

$$S_c^n = T_{i,j,k}^n \left[1 - (a_{i+1,j,k} + a_{i-1,j,k} + a_{i,j+1,k} + a_{i,j-1,k} + a_{i,j,k+1} + a_{i,j,k-1}) \right] + \frac{\dot{q}_{i,j,k}^n \Delta t}{\rho c_p} \quad 4.40$$

Table 4.6. Summary of the Coefficients for 3D Discretization

Coefficient	Thermal conductivity	Specific heat
$a_{i+1,j,k} = \frac{k_{i+1/2,j,k} \Delta t}{\rho c_{p_{i+1/2,j,k}} \Delta x^2}$	$k_{i+1/2,j,k} = \frac{k_{i,j,k} + k_{i+1,j,k}}{2}$	$c_{p_{i+1/2,j,k}} = \frac{c_{p_{i,j,k}} + c_{p_{i+1,j,k}}}{2}$
$a_{i-1,j,k} = \frac{k_{i-1/2,j,k} \Delta t}{\rho c_{p_{i-1/2,j,k}} \Delta x^2}$	$k_{i-1/2,j,k} = \frac{k_{i,j,k} + k_{i-1,j,k}}{2}$	$c_{p_{i-1/2,j,k}} = \frac{c_{p_{i,j,k}} + c_{p_{i-1,j,k}}}{2}$
$a_{i,j+1,k} = \frac{k_{i,j+1/2,k} \Delta t}{\rho c_{p_{i,j+1/2,k}} \Delta y^2}$	$k_{i,j+1/2,k} = \frac{k_{i,j,k} + k_{i,j+1,k}}{2}$	$c_{p_{i,j+1/2,k}} = \frac{c_{p_{i,j,k}} + c_{p_{i,j+1,k}}}{2}$
$a_{i,j-1,k} = \frac{k_{i,j-1/2,k} \Delta t}{\rho c_{p_{i,j-1/2,k}} \Delta y^2}$	$k_{i,j-1/2,k} = \frac{k_{i,j,k} + k_{i,j-1,k}}{2}$	$c_{p_{i,j-1/2,k}} = \frac{c_{p_{i,j,k}} + c_{p_{i,j-1,k}}}{2}$
$a_{i,j,k+1} = \frac{k_{i,j,k+1/2} \Delta t}{\rho c_{p_{i,j,k+1/2}} \Delta z^2}$	$k_{i,j,k+1/2} = \frac{k_{i,j,k} + k_{i,j,k+1}}{2}$	$c_{p_{i,j,k+1/2}} = \frac{c_{p_{i,j,k}} + c_{p_{i,j,k+1}}}{2}$
$a_{i,j,k-1} = \frac{k_{i,j,k-1/2} \Delta t}{\rho c_{p_{i,j,k-1/2}} \Delta z^2}$	$k_{i,j,k-1/2} = \frac{k_{i,j,k} + k_{i,j,k-1}}{2}$	$c_{p_{i,j,k-1/2}} = \frac{c_{p_{i,j,k}} + c_{p_{i,j,k-1}}}{2}$

5 EXPERIMENTAL ANALYSIS

5.1 Introduction

In this chapter, the computational model was benchmarked by comparing 2D analysis results with the spreadsheet program. The program was also verified by comparing 2D and 3D Analysis results produced by the computational model with field results.

5.2 Validation of the computational model

The benchmarking of the computational model developed using MATLAB® was conducted by comparing 2D analysis results with the spreadsheet program developed by Ballim (Ballim, 2004). The spreadsheet program was successfully used in earlier study to predict temperature development within a large concrete pier-cap structure at early ages (Yikici & Chen, 2015a). It should be noted that the thermal properties such as specific heat capacity and thermal conductivity were taken as constant values for benchmarking. Moreover, the implemented model was used to predict temperature histories of large bridge elements and 6-ft cube blocks constructed in different location in West Virginia.

5.2.1 Benchmarking example

For benchmarking purpose, a 4 feet (1.2 m) by 8 feet (2.4 m) cross-section was analyzed using 6 bag Class B concrete mix proportions. The pre-set rate of heat profile was selected for the straight cement mix that was embedded into the program. Thermal conductivity of concrete was taken 1.8 BTU/hr-ft-°F (3.1 W/m-K) pre-set for limestone aggregate concrete. The specific heat of concrete was calculated to be 0.27 BTU/lb-°F (1134 J/kg-K). The heat transfer coefficient was set to be 0.88 BTU/hr-ft²-°F (5 W/m²-K) for covered surfaces (formwork). The space interval was chosen 4 inches (10 cm) and the

time step was set to 0.1 hours. The initial concrete temperature was taken 70°F (21°C) and daily ambient maximum and minimum temperatures were taken 77°F (25°C) and 50°F (10°C), respectively. Results obtained from the 2D model matched well with results from the spreadsheet program as shown in Figure 5.1.

Figure 5.1. Benchmarking finite volume model with the spreadsheet program.

5.3 Predicting field temperatures

Analysis results produced with the computational model were compared with field results. In every time step, the source term ($\dot{q}$) was selected for each nodal point based on the equivalent age heat rate data. Also, the specific heat capacity and thermal conductivity of each node were updated based on degree of heat generation. The overall procedure for the thermal analysis is briefly outlined in Figure 5.2. The operations needed for predicting the temperature development are shown in sequential order as programmed in MATLAB®.

Figure 5.2. Flowchart of the thermal analysis.

The internal heat source was determined experimentally and the rate of heat generation of each mix design was calculated in equivalent age using the adiabatic calorimeter test results presented earlier in Figure 4.3. Activation energy values (E_a) used to calculate the equivalent age of the concrete mixtures were determined experimentally according to ASTM C 1074 which were described in Yikici and Chen (Yikici & Chen, 2015b). The fresh concrete densities were measured on the field with other fresh concrete properties.

Degree of heat generation based relationship was implemented to consider effects of temperature and time on thermal properties of concrete for each nodal point. Ultimate thermal conductivity of different concrete mixtures were selected based on ACI 207 reported values shown in Table 4.1. Depending on the coarse aggregate type used in the mix design, ultimate thermal conductivity values varied between 0.94 BTU/h-ft-°F (1.63

W/m-K) to 2.13 BTU/h-ft-°F (3.68 W/m-K). Initial specific heat values were calculated from the specific heat capacity of each component in the mix design as shown in Equation 4.10.

5.3.1 2D Thermal Analysis for Pier Caps

Most bridge structures have relatively simple geometries, usually with one very large dimension, such as pier caps. Therefore, the middle cross sections of the larger dimension of the pier caps were analyzed, simulating a 2D heat transfer problem. The boundary conditions were specified as convection into the surface or convection out of the surface from the surroundings. A combined convection coefficient was estimated to be 0.88 BTU/hr-ft²-°F (5.0 W/m²-K) for the concrete surfaces covered with steel formwork and protected with insulation blankets. Input parameters used in 2D analysis for pier cap examples are given in Table 5.1. The analysis was done in 3 minute time steps with 2 inches (5 cm) space interval.

Table 5.1. Input Parameters Used in the 2D Analysis

Property	Values		
	D3	D4	D7
Specific heat (c_{up}), BTU/lb-°F (J/kg-K)	0.231 (967)	0.226 (945)	0.227 (952)
Density (ρ), lb/ft ³ (kg/m ³)	141 (2250)	143 (2290)	142.5 (2280)
Thermal conductivity (k_{uc}), BTU/hr-ft-°F (W/m-K)	1.79 (3.1)	1.79 (3.1)	1.79 (3.1)
Convection coefficient (h_c), BTU/hr-ft ² -°F (W/m ² -K)	0.88 (5.0)	0.88 (5.0)	0.88 (5.0)
Activation energy (E_a), kJ/mol	45.7	44.0	45.7
Initial concrete temperature (T_0), °F (°C)	81 (27)	66 (19)	55 (13)

Figure 5.4a, Figure 5.6a and Figure 5.8a show temperature predictions from three pier caps in comparison with the field data and Figure 5.4b, Figure 5.6b and Figure 5.8b

show the temperature distribution throughout the mid cross-section at around 30 hours after casting. FVM_{Mid} and FVM_{Side} represent analysis results from the center and the side surface locations. T_{Amb} is the ambient temperature obtained using Equation 4.1 suggested by Ballim (2004).

The first bridge is called the South Mineral Wells Bridge which is a three-span bridge located in District 3, Wood County (Figure 5.3a). This bridge has a total length of 433 feet (132 m) with the largest span length of 180 feet (54.9 m). The pier cap (D3 Pier Cap) is a double-pier hammerhead design with approximate dimensions of 58 feet (17.7 m) in length, 5 feet (1.5 m) in width, and 7 feet (2.2 m) in height (Figure 5.3b). Class B concrete mix that with 470 lb/yd³ (278 kg/m³) of cement, 75 lb/yd³ (44 kg/m³) of fly ash, and 0.45 water to cementitious ratio was used for the concrete placement. Steel formwork was used for the construction, and the top surface was covered with wet burlap for seven days.

Figure 5.3. D3 Pier Cap on South Mineral Wells Bridge (a) Picture of D3 Pier on South Mineral Wells Bridge (b) Schematic drawing and sensor locations.

According to the temperature records, the initial concrete temperature was measured 81°F (27°C) and the core temperature (D3C202) reached 145°F (63°C) at about 23-24 hours after concrete placement while the temperature measurement at the side surface (D3C204) was 115°F (46°C). The largest temperature differential was 30°F (16.7°C) between the core and the side surface. The maximum calculated temperature at the center of the D3 Pier Cap cross-section is 145°F (63°C) after 29 hours and the maximum temperature difference between the center and the side surface is 28°F (15.7°C) after 28 hours (Figure 5.4a).

Figure 5.4. D3 Pier Cap concrete temperatures compared with the analysis results
(a) Temperature vs time (b) Temperature contour at 30 hours, °F.

The second bridge is called the Ices Ferry Bridge. The Ices Ferry Bridge is located in District 4, Monongalia County, WV. It has 3 piers in water and has a total length of 828 feet (252 m) with the largest span of 258 feet (79 m). Figure 5.5a shows the temperature

sensor in the pier cap of the Ices Ferry Bridge prior to concrete placement. The D4 Pier Cap shown in Figure 5.5b is a 37 feet (11.2 m) long, 6 feet (1.8 m) wide and 10 feet (3 m) high hammerhead pier cap. Schematic drawing of sensor locations within this pier cap is shown in Figure 5.5c. The Ices Ferry Bridge pier cap was constructed using Class B Modified Concrete with 564 lb/yd³ (334 kg/m³ TYPE I/II cement and 75 lb/yd³ (44 kg/m³) fly ash. The water to cementitious material ratio was 0.38.

Figure 5.5. D4 Pier Cap on Ices Ferry Bridge (a) Installing temperature sensors (b) Picture of D4 Pier Cap during construction (c) Sensor locations.

The initial concrete temperature was recorded 65°F (18.3°C). The maximum concrete temperature recorded in the center of the pier cap (D4C102) was 135°F (57.2°C) at approximately 21 hours after concrete placement. The analysis results showed that the maximum predicted temperature at the center of the D4 Pier Cap cross-section is 135°F (57.2°C) 27 hours after casting and the maximum temperature difference between the center and the side surface is 33°F (18.3°C) 41 hours after casting (Figure 5.6a).

(b)

Figure 5.6. D4 Pier Cap concrete temperatures compared with the analysis results (a) Temperature vs time (b) Temperature contour at 30 hours, °F.

The third bridge is called the Lucille Stalnaker Bridge which is a three-span bridge located in District 7, Gilmer County. This bridge has a total length of 233 feet (71 m) with the largest span length of 100 feet (30.5 m). The pier cap (D7 Pier Cap) is a single-pier design with approximate dimensions of 24 ½ feet (7.5 m) in length, 4 feet (1.2 m) in width, and 8 feet (2.4 m) in height (Figure 5.7).

Figure 5.7. Lucille Stalnaker Bridge (a) D7 Pier Cap (b) Schematic drawing and sensor locations.

The initial concrete temperature was recorded as 53°F-54°F (11.5°C) at the concrete plant. The maximum temperature was recorded 118°F (48°C) in the center of the pier stem (D7C102) at about 27 hours after concrete placement. The minimum temperature measurement at that time was 91°F (33°C) at the side surface (D7C104), and the temperature differential was 27°F (15°C). The analysis results showed that the maximum predicted temperature at the center of the D7 Pier Cap cross-section is 121°F (49.4°C) 34 hours after casting and the maximum temperature difference between the center and the side surface is 41°F (22.7°C) 49 hours after casting (Figure 5.8a).

Figure 5.8. D7 Pier Cap concrete temperatures compared with the analysis results
(a) Temperature vs time (b) Temperature contour at 30 hours, °F.

Overall, the results showed that the temperature predictions can be correlated reasonably well with the measured temperature time histories at the center as well as at the side surface for the pier caps. It can be observed from these figures that the predicted rate of the temperature rise and the maximum concrete temperatures are comparable with the

field measurement. Moreover, the influence of the ambient temperature oscillations on the side surface temperatures can be simulated reasonable well by the calculation. Furthermore, the calculated temperatures at the edges of the pier caps are the lowest, thus the largest temperature differentials occur between the center and the edges which identify the center of the top edges on the pier caps be the location of possible crack initiation due to thermal stresses. Temperature differentials could be reduced with improved insulation at the edges to minimize the possibility of thermal cracking.

5.3.2 3D Thermal Analysis for 6-ft Cubes

For convenient computational speeds, time step was increased to 6 minutes and space interval was taken $2\frac{3}{4}$ inches (6 cm) in 3D analysis. Input parameters used in 3D analysis for the 6-ft cubes are given in Table 5.2.

Table 5.2. Input Parameters Used in the 3D Analysis

Property	Values			
	D1	D9	D5	D6
Specific heat (c_{up} , BTU/lb-°F (J/kg-K))	0.231 (967)	0.234 (980)	0.227 (948)	0.232 (969)
Density (ρ), lb/ft ³ (kg/m ³)	141 (2250)	142.5 (2280)	144 (2300)	141 (2250)
Thermal conductivity (k_{uc}), BTU/hr-ft-°F (W/m-K)	1.33 (2.3)	1.79 (3.1)	1.79 (3.1)	1.79 (3.1)
Convection coefficient (h_c), BTU/hr-ft ² -°F (W/m ² -K)	0.76 ⁺ (4.3)	0.71 ⁺ (4.0)	0.88* (5.0)	0.71 ⁺ (4.0)
Activation energy (E_a), kJ/mol	45.7	41.1	44.7	40.5
Initial concrete temperature (T_0), °F (°C)	84 (29)	79 (26)	81 (27)	73 (23)
+ wood formwork, *steel formwork				

Temperature-time histories of the center location (T3) as well as the side surface (T11) from D1, D9, D5 and D6 cubes were plotted in comparison with field data in Figure 5.10a, Figure 5.11a, Figure 5.12a, and Figure 5.13a, respectively. At the same time, the ambient temperature (T_{Amb}) was calculated using Equation 4.1. Additionally, a 3D plot of

the center cross-section was shown at 30 hours (Figure 5.10b, Figure 5.11b, Figure 5.12b, Figure 5.13b). Moreover, temperature distribution along the x-axis at the center of the cubes were plotted in comparison with concrete temperatures obtained from temperature loggers T3, T9, T10 and T11 (Figure 5.10c, Figure 5.11c, Figure 5.12c, Figure 5.13c).

Figure 5.9. Embedded temperature loggers along the x-axis.

D1 Cube analysis results are shown in Figure 5.10. According to the results, the maximum predicted temperature at the center (FVM_{T3}) of the D1 Cube is 146°F (63.3°C) 30 hours after casting and the maximum temperature difference between the center and the side surface is 30°F (16.7°C) 26 hours after casting.

Figure 5.10. D1 Cube concrete temperatures compared with the analysis results (a) Temperature vs time (b) Temperature contour at 30 hours, °F (c) Temperature along the x-axis at the center of the cube.

D9 Cube analysis results are presented in (Figure 5.11). Result show that the maximum predicted temperature at the center (FVM_{T3}) and the side surface (FVM_{T3}) of the D9 Cube cross-section is 164°F (73.3°C) and 136°F (57.7°C) 34 hours after concrete placement, respectively. Maximum temperature difference was predicted to be 34°F (19°C) around 70 hours after placement (Figure 5.11a).

Figure 5.11. D9 Cube concrete temperatures compared with the analysis results (a) Temperature vs time (b) Temperature contour at 30 hours, °F (c) Temperature along the x-axis at the center of the cube.

It can be observed from Figure 5.12 the maximum predicted temperature at the center of the D5 Cube cross-section is 149.5°F (65.3°C) and the maximum temperature at the side surface is 114.5°F (45.8°C) 26 hours after casting. The temperature difference was calculated 35°F (20°C).

Figure 5.12. D5 Cube concrete temperatures compared with the analysis results (a) Temperature vs time (b) Temperature contour at 30 hours, °F (c) Temperature along the x-axis at the center of the cube.

It can be observed from Figure 5.13 the maximum predicted temperature at the center of the D6 Cube cross-section is 151°F (66.1°C) 31 hours after casting. The temperature difference is 38°F (45.8°C) at about 53 hours after casting.

Figure 5.13. D6 Cube concrete temperatures compared with the analysis results (a) Temperature vs time (b) Temperature contour at 40 hours, °F (c) Temperature along the x-axis at the center of the cube.

5.4 Implementation of variable thermal properties

Concrete thermal conductivity and specific heat capacity were defined as function of degree of heat generation as shown in Equation 4.8 and Equation 4.9, respectively.

Figure 5.14. Implementation of variable thermal properties with respect to equivalent age.

Figure 5.14 illustrates the process of the use of degree of heat generation to determine concrete thermal properties variable with respect to equivalent age. At each time step (Δt) new values for concrete thermal conductivity and specific heat are being calculated for every node based on the calculated equivalent age of the node. The equivalent age can be determined from the temperature-time history of each location. Next, degree of heat generation can be selected and the value can be substituted in the corresponding equations. Figure 5.15 shows the variation of thermal conductivity (k_c) and specific heat capacity (c_p) of D3 Pier Cap analysis with time.

Figure 5.15. Variation of concrete thermal properties with time (a) thermal conductivity (b) specific heat.

5.4.1 Effect of variable k

The influence of variable thermal conductivity on temperature development inside the 6-ft cube was investigated by modifying the previously given Equation 4.8:

$$k_c(\alpha) = k_{uc}(1.10 - m\alpha)$$

5.1

In this part of the study, three models were considered to demonstrate different settings for thermal conductivity. The first model used a constant thermal conductivity ($m=0$) and the second model considered 10% variation ($m=0.1$) and the third model considered 30% variation ($m=0.3$) in thermal conductivity. All the other input parameters were identical. Specific heat and ambient temperature were set constant for the analysis and convection coefficient was same for all six surfaces. Figure 5.16 shows the core temperature results for three models, k_{10} , k_{30} and k_{con} representing 10% variation, 30% variation and constant thermal conductivity, respectively.

It can be observed in all three models that temperature histories were not affected up to 30 hours of analysis. Also, use of constant thermal conductivity value resulted in slightly lower maximum temperature. Additionally, maximum and final temperatures slightly increased with decreased thermal conductivity. Therefore, it can be concluded that the variable thermal conductivity generally has smaller influence on temperature histories.

Figure 5.16. Effect of variable conductivity on temperature development.

5.4.2 Effect of variable C_p

The effect of variable specific heat was investigated by modifying the previously given Equation 4.9:

$$c_p(\alpha) = c_{up}(1.10 - m\alpha) \quad 5.2$$

Similarly, three models were considered to demonstrate different settings for specific heat. The first model used a constant value ($m=0$) and the second model considered 10% variation ($m=0.1$) and the third model considered 30% variation ($m=0.3$) along the degree of heat generation. All the other input parameters were identical for each model. Thermal conductivity and ambient temperature were set constant for the analysis and convection coefficient was same for all six surfaces.

It can be observed from Figure 5.17 that maximum temperatures were increased by 2°F (1.1°C) and increased by 4.5°F (2.5°C) when using model cp₁₀ and cp₃₀, respectively. Considering that, most specifications require 35°F (20°C) limitation for peak

temperatures, such difference can be significant. Effect of variable specific heat should be considered when predicting concrete temperature development, especially in larger elements.

Figure 5.17. Effect of variable specific heat on temperature development.

5.5 Summary and Conclusions

In summary, a FVM was developed using MATLAB® programming language for numerical computation to predict temperature-time histories in mass concrete elements at early ages. This program can be implemented for 1D, 2D and 3D temperature analysis of different structure sizes while mesh size can also be selected differently in x, y, and z coordinates. Using MATLAB® computational capabilities, number of nodal points were maximized while time step was taken as low as 3 minutes. For 3D analysis, operation time of the program is less than 1 minute using an office desktop computer.

Input parameters needed for analysis include concrete density, specific heat, thermal conductivity, rate of heat generation, initial temperature, convection coefficient

(considering formwork and insulation materials), daily maximum and minimum ambient temperatures and concrete placement time. Concrete heat generation is considered as the most important and effective factors for temperature development in concrete at early ages. Therefore, each concrete mix used for the analysis was tested in the laboratory using semi-adiabatic and adiabatic calorimetry methods and the calculated rate of heat generation property was used as input in the model. Arrhenius equivalent age method was employed to determine rate of heat generation to consider non-uniform effect of time and temperature on the hydration. Consequently, apparent activation energy values were determined in the laboratory to transform the heat generation rate relationship obtained experimentally into equivalent age.

The numerical model was successfully benchmarked with the 2D finite difference spreadsheet program that was successfully employed in an earlier study (Yikici & Chen, 2015a). Moreover, simulation results were validated with the temperature-time histories collected from bridge pier cap constructions (2D analysis) and 6-ft cube blocks (3D analysis). The maximum concrete temperatures as well as temperature differentials can be predicted reasonably well using the developed model.

In conclusion, using the concrete mix information and the measured concrete heat generation, this study shows that the temperature predictions can be correlated reasonably well with the field data. This program can provide useful information for engineers to take preventive measures during the design and construction stage and to make critical construction decisions such as selecting suitable and more economical concrete mix design for large elements, formwork removal time, curing methods, and pre and/or post-cooling methods.

5.6 Limitations and Future Work

There are several important points that can be improved to make this program more user-friendly, to obtain better results and to implement in other applications. The following items are some of the known limitations of the program and proposed future work ideas:

- Heat between the concrete and the surrounding environment is a complex phenomenon. This model does not consider some of the thermal effects such as sun radiation, curing, wind speed and external cooling systems. Also, heat convection coefficients were assumed constants based on models from other research studies. Experimental study is needed to determine such effects in order to implement actual parameters into the model.

- Thermal concrete properties, specific heat and thermal conductivity were not tested experimentally. Variation of those parameters at early ages need to be investigated more in depth.

- The most important input parameter is the concrete heat generation. A larger data base consisting “thermally friendly” mixtures can be generated using locally available cementitious materials: i.e. fly ash, slag and silica fume.

- In this study semi-adiabatic and adiabatic methods for concrete were used to determine concrete heat of hydration. However, both methods are not standardized by ASTM yet. Alternatively, isothermal calorimetry is a common method to investigate hydration properties of cementitious materials because of its ease to use. There are standard test equipments commercially available in the market, and a standard practice for the use of isothermal calorimetry for cementitious mixtures can be found in ASTM C1679.

- In the analysis, the corner and the edge node temperatures were calculated by the average of two neighboring nodes. There were no boundary conditions assigned for these nodes. For more accurate analysis results, corner and the edge boundary conditions can be specified and temperatures can be calculated accordingly.

- Formwork removal option is not available. This program can be modified so that the user can input the formwork removal time to assess how concrete temperatures are affected.

- This program can only model rectangular shape elements. Modifications need to be made in order to analyze more complicated geometry.

- The program does not consider cooling pipes for post-cooling or sequential concrete placement for thermal effects of each lift or nearby elements.

6 USE OF MATURITY METHOD TO ESTIMATE COMPRESSIVE STRENGTH OF MASS CONCRETE

6.1 Introduction

The strength of properly batched, placed and cured concrete can be expressed as a function of temperature-time history that relates to the concrete hydration. Higher curing temperature will speed up the hydration process and the concrete could gain strength faster at early age. This concept is known as the maturity concept (Carino, 2004). According to this concept, an empirical relationship can be established between temperature-time history and concrete strength development in order to predict the strength during the curing period by monitoring the in-place concrete temperatures in real time. Consequently, this information can be used to help decision making (e.g. time of formwork removal, time of post-tensioning, or open the pavement to traffic) that save time and reduce the construction cost (ASTM Standard C1074, 2011).

According to the West Virginia Division of Highways (WVDOT) survey results conducted in 2007, twenty-five out of thirty-six states used the maturity concept mainly as a substitute for early cylinder compressive strength to allow formwork to be removed or pavements to be opened to traffic (Mance, 2013). Since then, many state transportation agencies in United States have instituted procedures or are still conducting research projects to implement the maturity method to predict in-place concrete strength. However, there are concerns about the accuracy of maturity method in structural concrete applications, especially when constructing mass concrete elements where variable concrete temperatures throughout the concrete section affect the curing history. Furthermore, the “crossover” effect has been reported in the literature that limits the applicability of maturity method in predicting the behavior of concrete that has high early-age temperature.

Specifically, high curing temperatures ($>40^{\circ}\text{C}$) at early-age lead to a lower ultimate strength values as compared to an initial lower early-age curing temperature (Poole T. S., 1996) (Byfors, 1980) (Carino & Lew, Temperature Effects on Strength-Maturity Relations of Mortar, 1983) (Wild, Sabir, & Khatib, 1995). Therefore, some models were suggested to improve the maturity method by adjusting datum temperature, apparent activation energy values, or integrating additional functions to eliminate the crossover effect (Kim, Han, & Lee, 2001) (Tepke, Tikalsky, & Scheetz, 2004) (Chanvillard & D'Aloia, 1997) (Kim & Rens, 2008) (Carino & Tank, 1992) (Kjellsen & Detwiler, 1993) (Kim, Moon, & Eo, 1998) (Kim, Han, & Song, 2002) (Kim, Han, & Park, 2002) (Brooks, Schindler, & Barnes, 2007) (Kim & Rens, 2008). Nevertheless, the maturity concept has been used to estimate in-place concrete strength development for over 40 years (Carino, 2004).

This chapter is to investigate the applicability of maturity method to estimate the in-place concrete strength of large bridge sub-structure elements, such as piers, footers, pier caps or abutments, using WVDOH approved Class B concrete. Class B concrete, as described in WVDOH Standard Specifications, has a minimum 3,000 psi (20 MPa) 28-day design strength with optimum 4-inches (102 mm) slump and 7% target air. It may be designed using supplementary cementitious materials such as fly ash, ground granulated blast furnace slag (GGBFS) or micro-silica with 564 pound per cubic yard (330 kg/m³) target cement content and 0.49 maximum water-cementitious ratio. This part of the study presents test results from four different 6-ft concrete cube constructions and the predicted in-place concrete strength using a maturity function based on concrete equivalent age.

6.2 Experimental program

Four six-foot concrete cube blocks were constructed in various locations throughout West Virginia, using Class B concrete delivered from local ready-mix plants and following their common practice for placement and curing for concrete construction. Sacrificial temperature sensors that are self-contained, battery operated, microprocessor based loggers were instrumented inside the cubes. A handheld reader was used later to collect the hourly temperature data.

Fresh concrete properties were determined before placement and 6 by 12 inch (150 by 300 mm) concrete cylinders were collected for the strength-maturity calculations. Core samples were taken from the hardened 6-ft cubes and the measured compressive strengths from the core samples were compared to the predicted strengths from equivalent age calculations. Apparent activation energy values were determined based on the ASTM C 1074 testing method.

6.2.1 Six-foot cube construction

One of the purposes of the six-foot cube constructions was to investigate strength development of in-place concrete by monitoring the temperature distribution in concrete and investigate the applicability and the limitations of the maturity concept for large concrete placements throughout West Virginia. The cubes were constructed at four different WVDOH districts, District 1, 5, 9 and 6, casting approximately nine cubic-yards (6.9 m^3) of concrete in each cube, provided by the local ready-mix concrete plants in that district. These four districts located in the south (D1), east (D5), south-east (D9) and north (D6) of West Virginia. The theoretical concrete mix design for each casting is given in Table 6.1. Cubes were instrumented with temperature loggers attached on a rebar cage

(Figure 6.1a) right before concrete casting. A schematic of the sensor locations is given in Figure 6.1b. Concrete was poured directly from the mixer truck without pumping and then was subjected to mechanical vibration to achieve sufficient compaction. Ordinary surface finishing was performed using wood-float rubbing which was applied on the top surface. The concrete surface was maintained completely and continuously moist during the seven-day curing period. After the concrete placement the top of the block was covered with white polyethylene sheeting. Concrete blankets were used on top surface as well as around the formwork on the side surfaces when necessary.

Figure 6.1. Six-ft cube casting (a) Instrumentation of the test cube (b) schematic of the sensor locations.

Table 6.1. Concrete Mix Proportions, per cubic yard (kg/m^3)

Item, lbs (kg)	D1	D5	D9	D6
	Class B Fly Ash	Class B GGBFS	Class B	Class B Modified
Cement (TYPE I/II)	470 (278)	423 (250)	564 (334)	658 (390)
Fly Ash (TYPE F)	75 (45)	--	--	--
GGBFS (Grade 100)	--	141 (83)	--	--
Water	245 (145)	276 (163)	262 (155)	260 (154)
Coarse Aggregate (#57)	1775 (1050)	1815 (1074)	1723 (1019)	1750 (1035)
Fine Aggregate	1255 (743)	1225 (725)	1299 (769)	1111 (657)
w/cm	0.45	0.49	0.45	0.40

6.3 Experiments

6.3.1 Determination of activation energy

In this study equivalent age method was implemented to estimate concrete maturity. This method requires determination of the activation energy (E_a) for the calculation of the

equivalent age. Concrete activation energy can be simply defined as the sensitivity of concrete properties at different curing temperatures (D'Aloia & Chanvillard, 2002). Activation energy of concrete is mix design specific and can be determined experimentally by means of calorimetric methods or compressive strength test (Wirquin, Broda, & Duthoit, 2002).

Activation energies of concrete mixtures given in Table 6.1 were determined following the procedure given ASTM C 1074-10 A1. The use of mortar specimens instead of concrete cylinders are allowed and the mortar was proportioned to have a fine-aggregate to cement ratio equal to the coarse-aggregate to cement ratio of the concrete mixture (Table 6.2) (ASTM Standard C1074, 2011). Specimens were cured at three different temperatures: high (104°F), low (50°F), and laboratory temperature (73°F) and the compressive strength versus age relationship of 2-inch mortar cubes was established accordingly. In total, three specimens were tested at six different times in compression following the recommended test schedule (Tank, 1988), based on equal temperature-time factors for different curing temperatures (Figure 6.2a).

Table 6.2. Mortar Mix Proportions, lbs (1 lbs= 0.45 kg)

	D1	D5	D9	D6
Sand	20.48	23.60	16.80	14.6
Cement	5.50	5.50	5.50	5.50
Fly Ash	0.88	-	-	-
Slag (GGBFS)	-	1.83	-	-
Water	3.16	3.67	2.55	2.45

Upon the completion of the compressive strength tests, a hyperbolic equation was used to fit the set of data to determine the best fit regression parameters, such as the limiting strength, S_u , the rate constant of strength gain, k , and the dormant period t_0 , for those three

different curing temperatures (ASTM Standard C1074, 2011). The regression parameters are shown on in Table 6.3.

$$S = S_u \frac{k(t - t_0)}{1 + k(t - t_0)} \quad 6.1$$

where; S is the average strength of the cubes, t is the test age in hours, S_u is the limiting strength, t_0 is the offset time (age when strength development assumed to begin) and k is the rate constant of the strength gain.

Table 6.3. Hyperbolic Regression Analysis Results for Mortar Cubes

Mix	Temp, °F (°C)	Su, psi (MPa)	k, day ⁻¹	t ₀ , days	R ²
D1	50 (10)	4646 (32.0)	0.51	0.87	0.98
	73 (23)	4747 (32.7)	1.01	0.14	0.93
	104 (40)	4223 (29.1)	3.00	0.06	0.99
D9	50 (10)	4728 (32.5)	0.49	0.86	0.98
	73 (23)	4677 (32.2)	1.35	0.24	0.98
	104 (40)	4453 (30.7)	4.04	0.16	0.97
D5	50 (10)	4445 (30.6)	0.26	0.40	0.99
	73 (23)	4184 (28.8)	0.47	0.25	0.99
	104 (40)	4448 (30.7)	1.62	0.16	0.99
D6	50 (10)	6425 (44.3)	0.28	1.48	0.99
	73 (23)	7615 (52.5)	0.64	0.29	0.99
	104 (40)	5659 (39.0)	1.48	0.16	0.99

Natural logarithm of the k -values versus reciprocal curing temperature in Kelvin was plotted (Figure 6.2b). The negative slope of the line equals to the value of the activation energy divided by the universal gas constant ($R= 8.3145$ J/K-mol), also known as Q . This calculation is based on the Arrhenius function that is being used to explain the temperature dependence of the rate constant, k (Carino, 2004).

Figure 6.2. Determination of apparent activation energy (a) D1 mix mortar cube strength data with best fit curves (b) $\ln k$ versus $1/\text{Absolute Temperature}$ plot.

It was found that the hyperbolic strength-age function can properly model the strength development for each set of experiment. The apparent activation energy values were calculated as 45,700 J/mol and 44,750 J/mol for Class B Fly Ash (D1) and Class B GGBFS (D5) mixtures, respectively, and 41,150 J/mol and 40,050 J/mol for Class B (D9) and Class B Modified (D6) mixtures, respectively.

6.3.2 Determination of in-place strength

In order to establish the strength-maturity relationship of each mix, eighteen 6 by 12 inch (150 by 300 mm) cylinders were cast during each 6-ft cube construction. Additionally, two cylinders were embedded with commercial temperature loggers recording hourly temperature history (Figure 6.3a). All cylinders were placed inside insulated containers to reduce the effect from the ambient conditions overnight and then transported the next day to temperature controlled curing tanks at the District material laboratory (Figure 6.1b). Average compressive strength of the concrete was determined in accordance with ASTM C39 at 1, 3, 7, 14, 28 and 56 days, testing at least two cylinders at each age (Table 6.4).

Table 6.4. Average Strength Values from Standard Cylinder Specimens, psi (MPa)

Age, days	D1	D5	D9	D6
1	2140 (15)	1240 (9)	1510 (10)	3000 (21)
3	2940 (20)	2220 (15)	2500 (17)	4560 (31)
7	3500 (24)	3260 (22)	2810 (19)	5370 (37)
14	4130 (28)	4540 (31)	3070 (21)	6260 (43)
28	4190 (29)	5560 (38)	3800 (26)	6650 (46)
56	5240 (36)	6240 (43)	3860 (27)	7260 (50)

All concrete samples satisfied the 3,000 psi (20 MPa) 28 days compressive design strength required by the specifications. Amongst them, D9 mix showed the lowest ultimate strength due to a relatively higher air content of 9.5% recorded on-site; the required air content for this mixture was 7%.

Figure 6.3. D5 Cube construction and sampling (a) Maturity cylinders (b) Cylinders in the field overnight.

In addition to that, at 4, 28 and 56 days, 4-inch-diameter by 6-foot long (10 by 180 cm) core samples were taken parallel to the casting direction from each hardened concrete cube. A schematic drawing that shows the core locations and specimen designations is presented in Figure 6.4.

Figure 6.5 shows the extraction of the 6-ft core samples from District 6 cube. Each core was placed inside a plastic tubing and transported to the laboratory for specimen preparation. A total of six 4 by 8 inch (100 by 200 mm) cylinder specimens were extracted from each core along the 6-ft (180 cm) length (Figure 6.6), designated as 1C to 6C. The core specimens were prepared and tested for compression at the same day in accordance with ASTM C 42 (ASTM Standard C42, 2011) (Appendix F). The core strength was used to represent the in-place compressive strength of the concrete cube at different depth.

Figure 6.4. Schematic of the coring locations from the top of the cubes.

Figure 6.5. Six-ft cube coring. (a) Coring process (b) 6-ft core sample.

Figure 6.6. Core specimen cut locations and designations.

6.4 Test results and discussion

Equivalent age approach was used to establish the maturity relationships. The actual age of the concrete was converted to its equivalent age at a specified temperature following the Arrhenius Equation:

$$t_e = \sum e^{-Q(\frac{1}{T_a} - \frac{1}{T_s})} \Delta t \quad 6.2$$

where, Q is the activation energy divided by the universal gas constant.

The strength-maturity relationship of each mix was determined using the cylinders cast on-site with recorded temperature history. All cylinders were tested at the District material laboratory to obtain the compressive strength of the concrete at different ages (Table 6.4). The equivalent age was calculated accordingly using the recorded concrete temperature with the activation energy value obtained. The strength vs. equivalent age relationships for each concrete mix is plotted in Figure 6.7 and the best-fit curve parameters are listed in Table 6.5.

The best-fit curves were obtained by regression analysis following hyperbolic equation shown on Equation 6.1. The curing temperature of the cylinders was 23°C (73°F) throughout the testing period. For each set of data, the limiting strength was estimated by considering the data for tests beyond 7 days, and the offset time (t_o) was assumed to be equal to the concrete final setting time measured by the penetration resistance method (Carino, 1984) (ASTM Standard C403, 2008). It can be observed that the D6 mix batched with 7 bag straight cement had the largest k value and the D5 mix batched with 6 bag cementitious material replacement had the lowest because the 25% GGBFS replacement lowers the rate of the strength gain.

Table 6.5. Best-fit Curve Parameters for Strength-Maturity Development

Mix	S_u , psi (MPa)	k , 1/day	t_o , hours	R-squared value
D1	5373 (37)	0.3375	0.24	0.81
D5	7068 (49)	0.1359	0.22	0.99
D9	4392 (30)	0.3193	0.15	0.81
D6	7501 (52)	0.5209	0.14	0.93

Figure 6.7. Strength versus equivalent age for cylinders cured at laboratory with best-fit curves.

6.4.1 Core strength

The core test has been an effective method to determine the in-situ concrete strength. However, the core strength can be generally less than that of a corresponding test cylinder at the same age mainly due to the drilling process (Neville, 1995). It was also noted that the use of concrete vibration compaction during casting might have led to non-uniform concrete properties and possible segregation. Figure 6.8 shows the concrete strength from the core samples extracted at different ages (4, 28 and 56 days) along the depth direction.

It was observed from Figure 6.8 that the core strength at the bottom have a tendency to be higher than the core strength at the top. Results show that there is a significant strength difference of the core sample along the depth regardless of the concrete mix designs from four different sites, between the top (1C) position and the bottom (6C) position. Concrete at 1C position appears to be the weakest and 5C and 6C positions are

the strongest (Figure 6.8). The concrete strength at the bottom was always greater than the strength at the top.

Figure 6.8. Concrete core strength along the depth direction below top surface, plotted at 4, 28 and 56 days of age (a) D1 (b) D5 (c) D9 (d) D6.

The core test results also indicate the variations from the conditions occurred during concrete placement. During D9 cube construction, concrete was delivered in two separate trucks and the air content measured on-site was 7.8% and 9.5%, respectively. The unexpected difference in air content may be the reason that shows a large variation in strength between the cores 3C and 4C positions. During D6 cube construction the slump

of the fresh concrete was only 1¾ inches (4.5 cm) making proper consolidation very difficult. Therefore, honeycombing was observed at the mid-height section from the concrete surfaces. The effect of the segregation and honeycombing on the core strengths was detected between core samples 3C and 4C in the D6 cube.

6.4.2 In-place concrete strength prediction via maturity method

In order to estimate the in-place concrete strength, temperature sensors were installed at specific locations in the 6-ft cubes. The locations were selected to be representative of the temperatures at the locations of coring. The equivalent ages of the core samples were calculated using Equation 6.2 based on the temperature-time history of the concrete at these locations inside the cubes, corresponding to sensor T3 (center), T6 (top section), T7 (mid-section) and T8 (bottom section). The test age of each 6-ft core are shown in comparison with equivalent age in Table 6.6.

Table 6.6. Core Specimen Age versus Equivalent Age.

	D1		D9		D5		D6	
Sensor #	t, days	te, days	t, days	te, days	t, days	te, days	t, days	te, days
T6	4.0	14.7	4.1	18.3	4.3	12.3	4.0	19.0
T7	28.1	47.6	28.1	41.7	28.3	31.8	28.0	36.0
T3	28.9	45.2	28.0	42.9	28.3	36.0	28.0	41.0
T8	56.1	69.9	56.2	62.7	75.6	53.0	56.0	56.5
T6	4.0	19.7	4.1	27.9	4.3	16.5	4.0	21.5
T7	28.1	57.5	28.1	55.8	28.3	36.4	28.0	36.8
T3	28.9	51.4	28.0	62.7	28.3	41.9	28.0	42.1
T8	56.1	76.1	56.2	75.7	75.6	58.1	56.0	57.2
T6	4.0	14.7	4.1	19.9	4.3	11.2	4.0	10.5
T7	28.1	49.1	28.1	45.7	28.3	30.6	28.0	25.5
T3	28.9	45.7	28.0	47.2	28.3	33.8	28.0	28.9
T8	56.1	70.5	56.2	65.6	75.6	53.0	56.0	45.9

For a large concrete element, the early-age temperature of the in-place concrete (Figure 6.9) can be expected to be much higher than that of the cylinders cured in the

laboratory temperature. The peak temperatures at the center (T3) of D1, D5, D9 and D6 6-ft cubes reached to 145°F (63°C), 149 °F (65°C), 165°F (74°C) and 156 °F (69°C), respectively, within 20 to 30 hours after concrete placement. Top surface temperatures (T6) were generally influenced by the ambient temperature (T_Amb) fluctuations, except D6 case where concrete blankets were used on the top surface. It can be seen from Fig. 9 that for D6 cube, the temperature differentials between the top surface and the center of the cube are lower than the rest of the cubes; a lower temperature differential also indicates a lower possibility of thermal cracking on the top surface. It can also be observed that the temperatures at different locations inside the concrete cubes would usually converge at about 10 to 14 days to a value close to the average of the ambient temperature.

Figure 6.9. Temperature-time histories from (a) D1, (b) D5, (c) D9 and (d) D6 cubes (T_Amb obtained from closest weather station (Weather Underground, 2013)).

According to the maturity method, in-place concrete strength along the depth of the 6-ft cubes was calculated using the temperature-time histories and the established strength-

maturity relationships for each concrete mixture as shown in Figure 6.10. Corresponding equivalent ages for 4, 28 and 56 day core samples were calculated based on the temperature-time history of each core center location.

Figure 6.10. Calculated concrete core strength via maturity method along the depth direction below top surface, plotted at 4, 28 and 56 days of age (a) D1 (b) D5 (c) D9 (d) D6.

The calculated strength values were compared to the actual measured concrete strengths from the core samples taken at 4, 28 and 56 days. Core sample 1C representing the top position, 3C and 4C representing the middle position, and 6C representing the bottom position of the cubes. Figure 6.11 shows the predicted concrete strength compared

with the core sample strength. The results showed that the top surface predictions using the maturity method are always higher than the actual core strength at all four cubes. For D1, D5 and D9 cubes, core strength at the middle position were within $\pm 15\%$ of the predicted strength at any given age, however, the core strength were higher than the predicted values at the bottom position. It was also noticed that in D6 case the core strengths are always lower than the predicted strengths at all the position, mainly due to the on-site construction quality control related to compaction, in-situ water-cement ratio, air content, and finishing.

Figure 6.11. In-place concrete strength prediction compared with the strength from the core samples (a) D1 (b) D5 (c) D9 (d) D6.

6.4.3 Effect of high curing temperature on strength development

According to the literature, the strength development of the concrete does not only depend on the temperature-time history, it also depends on the magnitude of the curing temperature. Therefore, laboratory tests were performed to determine the effects of high temperature curing on the concrete maturity estimations. Class B concrete mixtures were tested in Laboratory to determine the effects of high temperature curing on strength development. 4x8 inch cylinder specimens were collected and all specimens were kept under laboratory conditions until concrete reach final set. After that, specimens were placed inside the curing tanks at 73°F and 122°F. At least two cylinders were tested for each time and strength versus age relationships are given in Figure 6.12 and Figure 6.13 for Class B Fly Ash (6 bag) and Class B Fly Ash Modified (7 bag) concrete mixtures, respectively. The specimens cured at higher temperature show faster strength gain and higher strength values compared to regularly cured concrete specimens.

Figure 6.12. Strength-maturity relationship of Class B Fly Ash concrete.

Figure 6.13. Strength-maturity relationship of Class B Fly Ash Modified concrete.

When the strength-age curves obtained, the equivalent age of the concrete samples were calculated according to the maturity concept and the age of the concrete samples were adjusted accordingly. Best fit curve parameters were determined using the hyperbolic equation (

Table 6.7) and strength-maturity relationship is shown. It can be observed from the test results that when applying maturity method high temperature curing applications, the concrete strength can be matched approximately only up to 24 hours and the after that regular curing data crosses over the high temperature curing data and the strength in regular curing temperature is higher at all times compared to high curing temperatures. This part of our study is still inconclusive, but preliminary data clearly shows that use of traditional maturity method cannot predict concrete strength at later ages when concrete is being cured in high temperatures such as 122°F (50°C) constantly.

Table 6.7. Summary of Regression Parameters from Cylinder Tests

Concrete Mix	Curing Temperature, °F (°C)	Su, psi (MPa)	k, day ⁻¹	t ₀ , days	R ²
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Class B Fly Ash	73 (23)	4,842 (33.4)	0.2974	0.4010	0.99
	122 (50)	3,689 (25.4)	0.3035	0.2837	0.97
Class B Fly Ash Mod.	73 (23)	6,042 (41.7)	0.7841	0.295	0.99
	122 (50)	5,823 (40.1)	0.2807	-0.6144	0.99

In another effort, a batch of Class B concrete similar to the D9 cube mix design with 0.42 w/c ratio and air content of 6.4% was reproduced in the laboratory and 6 by 12 inch (150 by 300 mm) cylinder specimens were cast. Cylinders were then kept under laboratory conditions until concrete reached final set. After that, specimens were placed inside the curing tanks at 73°F (23°C), 104°F (40°C) and 122°F (50°C). The compressive strength development versus equivalent age is shown in Figure 6.14.

Figure 6.14. Strength-maturity relationship of Class B concrete.

Test results showed that concrete specimens cured at 104°F (40°C) and 73°F (23°C) exhibit same strength development curve when plotted by equivalent age which indicates that maturity method works well for concrete cured at this temperature. However, when specimens were cured at 122°F (50°C), the ultimate strengths (S_u) were 10% lower and the rate of strength gain was also slower (Figure 6.14). The best-fit curve using the hyperbolic relationship shown on Equation 6.1 is also plotted on Figure 6.1.

Although results shows that maturity method may overestimate concrete strength when the concrete was cured at a constant high temperature of 122°F (50°C), such difference may not be reflected on the core samples where maximum temperatures only exceeded 122°F's (50°C) at the initial few days after constructions and the time to peak temperatures were between 20 to 30 hours after casting (Figure 6.9). The core strength test results indicate that the estimated in-place concrete strength was not affected by the variable curing temperatures, especially for those samples close to the top surface of the cubes. Laboratory cured specimens' results show that the maturity method is able to predict the concrete strength on the top surface of the cubes. The strength of the core samples from the cubes that always exhibits lower value than the maturity prediction is likely attributed to core drilling process and on-site compaction.

6.5 Summary

Four different concrete mix designs were investigated using 6-ft concrete cubes constructed at four different districts in West Virginia. Strength-maturity calibration curves for these concrete mixtures were established. Concrete temperatures inside the cubes were monitored and the equivalent-age of concrete at various locations in the cubes was calculated using the measured activation energy values. The in-place concrete strength was determined by testing core samples extracted from cubes at 4, 28 and 56 days and the results were compared with the predicted values. Based on the test results the following conclusions can be made:

1. Compressive strength-age curve of concrete can be established by testing the corresponding mortar mixture following ASTM C 1074-A1. The hyperbolic strength-age relationship can be used to model strength development at different

temperatures. Activation energy values for four concrete mixtures including supplementary cementitious materials were determined.

2. The temperature at the center of the cubes is significantly higher than that of the top and the bottom surface. Use of concrete blankets immediately after concrete placement was found useful to reduce the temperature differentials between the top surfaces and the middle section.
3. The test data show that the core strength in vertical direction increases with depth. Core strength obtained from the samples near the top surface was significantly lower than those from the bottom position.
4. The results showed that the top surface predicted strength using the maturity method was always higher than the actual core strength at all four cubes. For D1, D5 and D9 cubes, core strength values at the middle position were within $\pm 15\%$ of the predicted strength at any given age, and the core strength values at the bottom position were always higher than the predicted values.

7 CONCLUSIONS

During this study, six bridge projects and four 6-ft cubes, in total, fourteen different elements from West Virginia with different concrete mix design, different types and brands of cement, and different types of aggregates and different size of structures were instrumented with loggers for temperature monitoring. A pre-liminary analysis of mass concrete structures was conducted and factors affecting concrete temperature development were observed and summarized accordingly.

In summary, a detailed thermal control plan with precise thermal analysis is necessary to control maximum concrete temperatures and temperature differentials. Therefore, in this dissertation a finite volume numerical method for thermal analysis of large concrete elements (pier caps, pier footers) has been presented. A computational program was developed using MATLAB® programming language for numerical computation to predict temperature-time histories in mass concrete elements. The model has proven to produce accurate predictions in 2D and 3D temperature analysis within the concrete elements at early ages. The model requires to enter initial basic information such as concrete density, specific heat, thermal conductivity, initial temperature, convection coefficient (considering formwork and insulation materials) and concrete placement time.

For the temperature analysis, concrete heat generation was obtained experimentally using calorimetry analysis and Arrhenius method was employed to consider non-uniform effect of time and temperature on the hydration rate of heat generation. Concrete specific heat and thermal conductivity was modeled as a function of heat generation, so that thermal properties of concrete vary in every location in time based on the particular temperature. Consequently, apparent activation energy values were determined in the laboratory in order

to implement Arrhenius method. Also, ambient temperature was modeled using daily maximum and minimum ambient temperatures which can be either obtained experimentally or weather forecast reports. Simulation results were validated with the temperature-time histories collected from three bridge pier cap constructions (2D analysis) and four 6-ft cube blocks (3D analysis). The results showed that, maximum concrete temperatures as well as temperature differentials can be predicted reasonably well using the developed model. This program allows the user to analyze mass concrete placements using different concrete mix design, different formwork type and ambient settings. This calculation can provide useful information during the pre-design stage to take preventive measures in order to make critical construction decisions such as selecting suitable and more economical concrete mix design for large elements, formwork removal time, curing methods, and pre-cooling and post-cooling methods. Overall, the program satisfied the main objective of this study of developing an analysis methodology that will allow the researchers and construction engineers to evaluate thermal behavior of selected bridge elements.

As a part of this study, in-situ concrete strength development of mass concrete structures was investigated. To consider non-uniform maturity development of the mechanical properties throughout the concrete element, core samples were taken vertically along the 6-ft cubes and in-place compressive strength development was measured at different ages. It was found that in-place concrete strength along the height of the 6-ft cubes was not uniform due to inefficiency of conventional concrete compaction. Especially four day old cores showed strength difference of about 1,500 psi from top to the bottom core specimen. Consequently, results showed that the top surface predicted

strength was always higher than the actual core strength at all times. For D1, D5 and D9 cubes, core strength values at the middle position were within $\pm 15\%$ of the predicted strength at any given age, and the core strength values at the bottom position were always higher than the predicted values. Still, all results satisfy 28 days design compressive strength requirement. ASTM C1074 Maturity Method was employed successfully to predict measured core strength.

8 RECOMMENDATIONS AND FUTURE STUDY

Results obtained from this study summarized in Chapter 3 coincides with other research findings. Consequently, standard thermal control procedures should be applied in mass concrete construction in order to reduce maximum concrete temperatures and temperature differentials. Decreasing the initial concrete temperature and cooling the structure during concrete casting with external methods are effective for reducing maximum temperatures. Using thermal curing blankets or insulated formworks is helpful to reduce temperature differentials between the surface and the interior. Additionally, sudden change in surface concrete temperatures such as early formwork removal or cold water spraying shouldn't be allowed to prevent thermal shock effect. Furthermore, use of supplementary cementitious materials such as fly ash and slag is certainly the best solution to reduce or slow down the heat generation of concrete. Using low heat cements with lower C3S contents similarly will reduce the total heat produced by cement and cements with lower Blaine fineness will slow down the rate of heat production. The total heat of hydration should be determined and the adiabatic temperature rise needs to be calculated individually to ensure low heat generation beforehand. Finally, a data base for the heat generation characteristics can be constructed including commonly used concrete mixtures in West Virginia. It is known that simulation programs for thermal analysis can provide valuable information during concrete mix prequalification to predict temperature development of the actual mass concrete structures. Data obtained from thermal analysis can be used to optimize the mix design and help determine the critical locations within the elements where temperatures has to be monitored.

Possible suggestions for improving the temperature prediction program and other areas of future work could include:

- Verifying predicted inputs such as specific heat and thermal conductivity by means of laboratory experiments
- Improving the developed temperature prediction model to account for sun radiation, curing, wind speed and external cooling systems
- Adding more tools to increase the capability of the program such as using cooling pipes for post-cooling and sequential concrete placement for thermal effects of each lift or nearby element
- Conducting a validation study using data from other states
- Developing a model to predict cracking risk based on concrete temperature distribution and validate 35°F (20°C) maximum allowable temperature difference
- Improving the maturity method for mass concrete structures using match-cure laboratory specimens.

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APPENDIX A

Sample Special Provision

November 16, 2010

**WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
SPECIAL PROVISION
FOR**

State Project Number: _____
Federal Project Number: _____

**FOR
SECTION 601 – STRUCTURAL CONCRETE
MASS CONCRETE**

601.1-DESCRIPTION:

ADD THE FOLLOWING SUBSECTION:

601.1.1-Mass Concrete:

Concrete placements whose least dimension exceeds 48.0 inches, excluding Drilled Caissons, tremie seals and Class D Concrete, shall be considered mass concrete and shall conform to the details shown on the plans and these special provisions. For this project, mass concrete placements shall be applicable to the elements identified in the plans.

Compensation for conforming to these requirements will be at no additional cost and shall be included in Pay Items for individual elements identified in the plans.

601.2-MATERIALS:

IN THE TABLE, REMOVE THE FOLLOWING ROW:

MATERIAL	SECTION OR SUBSECTION
* Portland Cement	701.1, 701.3

IN THE TABLE, ADD THE FOLLOWING ROW:

MATERIAL	SECTION OR SUBSECTION
* Portland Cement	701.1, ASTM C150 Type II

**** All coarse aggregate used in mass concrete placements shall be limestone

DELETE THE FOLLOWING SENTENCE:

Unless otherwise permitted by the Engineer, only one source of a pozzolanic additive shall be used in any one structure.

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REPLACE WITH THE FOLLOWING SENTENCE:

Sources of each type of pozzolanic additive shall be approved by the Engineer. Multiple sources of the same type of pozzolanic additive shall not be permitted.

601.3-PROPORTIONING:

ADD THE FOLLOWING TO SUBSECTION 601.3.1:

601.3.1 – Mix Design Requirements:

For Mass Concrete placements, the Design Mix shall meet the 28-day compressive strength as specified in the plans. If the 28-day compressive strength obtained in the field does not meet the design 28-day compressive strength requirement, acceptance may be based on a 56-day compressive strength test, if approved by the Engineer after considering the stresses resulting from the construction sequence proposed by the Contractor. Acceptance shall be in accordance to Section 601.4.4 of the Standard Specifications and of this Special Provision, and per the approval of the Engineer.

For Mass Concrete placements, pozzolanic additives may be a combination of the following additives at the substitution rate shown in the following table:

Cementitious Materials	Maximum percent of total cementitious materials by mass**
Class F Fly Ash	25
Ground Granulated Furnace Slag	50
Total of Fly Ash and Slag	50*

** Total cementitious materials include the summation of Portland Cement, fly ash, slag.

* Fly Ash shall not constitute more than 25-percent of cementitious materials.

A combination exceeding more than two types of pozzolanic additives will not be permitted.

601.4 TESTING

ADD THE FOLLOWING TO SUBSECTION 601.4.4:

601.4.4 – Compressive Strength Tests for Acceptance: Compressive strength acceptance criteria pertaining to Class of Concrete for mass concrete elements may be based on 56-day compressive strength if approved by the Engineer after considering the stresses resulting from the construction sequence proposed by the Contractor. 601.12-CURING AND PROTECTING CONCRETE:

ADD THE FOLLOWING SUBSECTION:

601.12.4-Mass Concrete: All mass concrete elements shall be kept completely and continuously moist by means of moisture retention. White polyethylene sheeting meeting the requirements of 707.10 shall be used. The sheeting shall be installed, and joints shall be sealed, so as to prevent as much moisture loss as possible. Water curing shall not be permitted. Curing shall be continued for a period of at least 7 calendar days.

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Surfaces may have coverings temporarily removed for finishing, but the covering shall be restored as soon as possible. The installation of curing blankets on mass concrete elements will be required if thermal cracking becomes an issue.

601.12.4.1-Temperature Monitoring System: The temperature monitoring and recording system for mass concrete shall consist of temperature sensors connected to a data acquisition system capable of printing, storing, and downloading data to a computer. Temperature sensors shall be located such that the maximum temperature difference within a mass concrete element can be monitored. As a minimum, for each mass concrete element placement, concrete temperatures shall be monitored at the center of the element, the center of the top face of the element, and at the center of the side face which is furthest from the center of the element.

Temperature readings shall be automatically recorded on an hourly or more frequent basis. A redundant set of sensors shall be installed near the primary set. Provision shall be made for recording the redundant set, but records of the redundant sensors need not be made if the primary set is operational.

Methods of concrete consolidation shall prevent damage to the temperature monitoring and recording system. Wiring from temperature sensors cast into the concrete shall be protected to prevent movement. Wire runs shall be kept as short as possible. The ends of the temperature sensors shall not come into contact with either a support or concrete form, or reinforcing steel.

When any equipment used in the temperature control and monitoring and recording system fails during the mass concrete construction operation, the Contractor shall take immediate remedial measures to correct the situation.

601.12.4.2-Construction: Temperature readings will begin when casting is complete. Temperature readings will continue for 28 days from the time of placement. Data shall be printed and submitted to the Engineer daily. A copy shall be submitted to the Materials Control, Soils and Testing Division for informational purposes.

601.12.4.3-Temperature Control Requirements: The Contractor shall verify to the Division in writing and provide all documentation daily that the temperature control requirements as specified below are met:

Temperature Control Requirements for each mass concrete elements:

- i. The Maximum Allowable Temperature Differential shall be limited to 40 degrees F. The temperature differential between the interior (center of the element) and exterior (furthest from the center) portions of the designated mass concrete elements during curing will be maintained to be less than or equal to this Maximum Allowable Temperature Differential, and
- ii. The Maximum Allowable Concrete Temperature shall be limited to 160 degrees F.

A change to the Temperature Control Requirements specified above may be proposed by the Contractor and shall be submitted to the Engineer for approval prior to any pour. This submission will include the new proposed Maximum Allowable

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Temperature Differential, along with all necessary data providing evidence to satisfactorily demonstrate to the Engineer that the deleterious effects to the concrete can be avoided. The Contractor shall allow seven (7) days for approval.

If the monitoring indicates that the Temperature Control Requirements have been exceeded then a penalty shall be assessed for bullets (i) and (ii) above, independently as follows:

\$100 / degree F or fraction there-of the allowable temperature range multiplied by the number of yards in the element.

No extension of time or compensation will be made for any rejected or penalized mass concrete element.

APPENDIX B

Temperature Monitoring Special Provisions

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION

DIVISION OF HIGHWAYS

SPECIAL PROVISION

FOR

STATE PROJECT: S311-17-0.10

FEDERAL PROJECT: BR-0017 (082) D

FOR

SECTION 601 – STRUCTURAL CONCRETE

TEMPERATURE MONITORING OF CONCRETE

ADD THE FOLLOWING SUBSECTIONS:

601.12.4.1-Temperature Monitoring System:

The temperature monitoring and recording system shall consist of temperature sensors connected to a data acquisition system capable of printing, storing, and downloading data to a computer. Temperature sensors shall be installed at the locations shown in the attached drawings. A sensor shall also be located in an area close to the concrete placement, and this sensor shall be used to record the corresponding ambient temperature.

Temperature readings shall be automatically recorded on an hourly or more frequent basis. A redundant set of sensors shall be installed near the primary set. Provision shall be made for recording the redundant set, but records of the redundant sensors need not be made if the primary set is operational.

Methods of concrete consolidation shall prevent damage to the temperature monitoring and recording system. Wiring from temperature sensors cast into the concrete shall be protected to prevent movement. Wire runs shall be kept as short as possible. The ends of the temperature sensors shall not come into contact with either a support or concrete form, or reinforcing steel.

When any equipment used in the temperature control and monitoring and recording system fails during the mass concrete construction operation, the Contractor shall take immediate remedial measures to correct the situation.

601.12.4.2-Construction:

Temperature readings will begin when casting is complete. Temperature readings will continue for 28 days from the time of placement.

601.12.4.3-Reporting:

Within two weeks of the completion of the concrete placements for the elements being monitored for the project, the Contractor shall compile all temperature data obtained in section 601.12.4.1 and submit it electronically to the following e-mail addresses:

Roger.Chen@mail.wvu.edu

Mike.A.Mance@wv.gov

The data for each concrete element being monitored shall be compiled separately and shall include the following:

1. The name and number of the concrete element (i.e.: Pier 1 footer, Pier 2 column, etc.) being monitored.
2. The dimensions of each concrete element being monitored and, if applicable, the dimensions and volume of each concrete placement within that element.
3. The total number of sensors in each element for each placement.
4. The date of each concrete placement and if applicable, the location of the placement within the element (i.e.: first placement of a column, etc.).
5. An illustration of each concrete element being monitored which shows the identification number and location of each sensor, and the distance of each sensor from each edge of the element. A photograph of the location where each sensor is placed shall also be included.
6. The class of concrete and a copy of the approved concrete mix design used in the elements being monitored.
7. A table containing all of the temperatures recorded at each sensor, the corresponding ambient temperature, and the time at which each reading was obtained.
8. A summary which includes the maximum temperature and the maximum temperature differential within each concrete placement and the time at which they both occurred.
9. The method which was used to cure the concrete (wet burlap, insulated forms, etc.), and the duration of curing. This shall include the times when curing begins and ends and the time at which any insulation is placed and removed.
10. Documentation including photographs and maps of any cracks in the concrete elements being monitored. This shall be done after the forms and insulation, if applicable, are removed and also immediately prior to completion of the project. The dates on which any cracks are first noted, shall also be included in this documentation.

INSTRUCTIONS FOR INSTALLING TEMPERATURE SENSORS

1. Temperature sensor locations and designations for District 7 – LUCILLE STALNAKER BRIDGE – S311-17-0.10 (please see Appendix A for drawings):

Table 1 Sensor designation for each location

	Top Center	Mid Center (core)	Bottom Center	Closest Side Center	Far Side Center
Abutment #2	D7A201	D7A202	D7A203	D7A204	D7A205
Pier Stem #1	D7S101	D7S102	D7S103	D7S104	-
Pier Cap #1	D7C101	D7C102	-	D7C103	D7C104

Notation: D7A203 indicates District 7 (D7), Abutment #2 (A2), sensor location number 3 (03).

Totally 26 sensors at 13 different locations.

2. At each location, two sensors will be installed. One will be the primary sensor (i.e. D8A201-P), and the other will be the secondary (i.e. D8A201-S). The cables that are connected to the sensors should be designated and labeled as primary and secondary. Both sensors will collect data from the beginning.
3. The necessary equipment including the sensors, cables and read-out units will be supplied by WVDOH. Upon completion of all monitoring, the read-out units, any extra sensors and substantial lengths of cable should be returned to WVDOH personnel.
4. All sensors shall be placed within the established clear cover for the respective member. The side surface sensors should be located at the cover depth at mid-height of the member. No direct contact of the sensor head with the reinforcement or formwork should be permitted. Plastic ties should be used to tie the sensors with the reinforcement to avoid displacement during concreting.
 - a. If there is no available reinforcement to attach the sensors, an extra rebar should be placed at the location so that the sensor can be attached at the desired location. This rebar should adhere to clear cover requirements.
 - b. If avoidable, sensors should not be attached directly to rebar that protrudes from the concrete surface of the current phase of construction.
5. The lead wires must be tied to the side of the rebar with plastic ties and positioned carefully to avoid any damage during construction and extended outside the concrete to collect data.
 - a. Considerations for construction procedures shall be made when placing all sensors and lead wires to assure that these remain intact during all phases of construction.
 - b. In the case that a significant change in direction is required when running lead wires (angle greater than 45 degrees), the wires should be secured within six inches on either side of the bend. If no bends are necessary, wires should be secured in intervals of no more than three feet.
6. Within 2 hours before placement of concrete sensors should be tested. If the sensor doesn't work, it should be replaced before concrete placement. If the primary and secondary sensors at a particular location both fail, the wires must be examined for breaks and if a break is found, it should be repaired. Once tested, the sensors shall remain activated.
7. The "Mass Concrete Temperature Monitoring Form" (F-002-RP257-REV00) should be completed and submitted with the temperature data obtained during the construction.
8. Additionally, at least one sensor shall be activated and used to monitor the ambient temperature at the project. This sensor shall be located in a shaded location next to the project field office. The ambient temperature at the project shall be monitored throughout the duration of all of the 28-day concrete temperature monitoring periods during the entire project.

Appendix A: Drawings for sensor locations

Figure 1. General Layout for the Instrumentation (Abutment #2)

Figure 2. General Layout for the Instrumentation (Pier Stem #1)

Figure 3. General Layout for the Instrumentation (Pier Cap #1)

Mass Concrete Temperature Monitoring Form																							
<p>General Information</p> <p>District # _____ Project # _____</p> <p>Type of Concrete Member and Element #</p> <p>☐ Rectangular Pier Stem # _____ ☐ Circular Pier Stem # _____ ☐ Footing # _____ If footing, type of subbase (soil, rock): _____ ☐ Abutment # _____</p> <p>Construction Techniques</p> <p>Date and Time of Concrete Placement: _____ Brand and Type of Formwork: _____ Time of Formwork removal hrs from start of the pour: _____ Curing Procedure (time, type etc) : _____</p> <hr/> <p>Concrete Mix Design</p> <p>Approved Mix Design Number: _____</p> <table style="width: 100%;"> <tr> <td style="width: 50%;">Cement, lb/yd3: _____</td> <td style="width: 50%;">Type and brand of cement: _____</td> </tr> <tr> <td>Fly Ash, lb/yd3: _____</td> <td>Type and source of fly ash: _____</td> </tr> <tr> <td>Slag, lb/yd3: _____</td> <td>Slag brand and grade: _____</td> </tr> <tr> <td>Coarse agg, lb/yd3: _____</td> <td>Type and source: _____</td> </tr> <tr> <td>Fine agg, lb/yd3: _____</td> <td>Type and source: _____</td> </tr> <tr> <td>Air, %: _____</td> <td></td> </tr> <tr> <td>w/cm: _____</td> <td></td> </tr> </table> <p>Please attach the chemical analysis report of cement and other cementitious materials if available.</p> <p>Chemical Admixtures</p> <table style="width: 100%;"> <tr> <td style="width: 50%;">Brand & Model: _____</td> <td style="width: 50%;">Dosage: _____</td> </tr> <tr> <td>_____</td> <td>_____</td> </tr> <tr> <td>_____</td> <td>_____</td> </tr> <tr> <td>_____</td> <td>_____</td> </tr> </table> <p>Cracking occurred after formwork removal ? Yes No</p> <p>If yes: Location(s) of the crack(s) _____ Crack widths and lengths, inch _____</p> <p>Please take a picture of cracked section and e-mail to: Roger.Chen@mail.wvu.edu</p>		Cement, lb/yd3: _____	Type and brand of cement: _____	Fly Ash, lb/yd3: _____	Type and source of fly ash: _____	Slag, lb/yd3: _____	Slag brand and grade: _____	Coarse agg, lb/yd3: _____	Type and source: _____	Fine agg, lb/yd3: _____	Type and source: _____	Air, %: _____		w/cm: _____		Brand & Model: _____	Dosage: _____	_____	_____	_____	_____	_____	_____
Cement, lb/yd3: _____	Type and brand of cement: _____																						
Fly Ash, lb/yd3: _____	Type and source of fly ash: _____																						
Slag, lb/yd3: _____	Slag brand and grade: _____																						
Coarse agg, lb/yd3: _____	Type and source: _____																						
Fine agg, lb/yd3: _____	Type and source: _____																						
Air, %: _____																							
w/cm: _____																							
Brand & Model: _____	Dosage: _____																						
_____	_____																						
_____	_____																						
_____	_____																						

F-002-RP257-REV01

APPENDIX C

Construction and Installation Quality Plan for 6 feet Cube Trial Casting

1. Purpose and Scope of the Work

The objective is to find the accuracy and the limitations of the maturity method for mass concrete placements by estimating concrete in-place strength according to the “ASTM C 1074 Standard Practice for Estimating Concrete Strength by the Maturity Method”.

The purpose of this Construction and Installation Quality Plan is to define the sequences and methodology of 6 feet Cube Trial Casting.

2. Method and Sequence of the Works

The 6 feet Cube casting will be performed at WVDOH District Office. Approximately 8 cubic yard ready-mix Class B concrete will be used for this project.

Figure 1. Sequence of the works.

2.1 Preparation of 6 feet Cube

A cube block with dimensions 6 feet by 6 feet by 6 feet will be prepared using wooden forms and #4 rebar. There are 28 six feet long and 3 three feet long #4 rebar needed to build the rebar cage. Before casting of concrete, the formwork needs to be prepared and inspected as mentioned in WVDOH Construction Manual Section 601 (601.4.1).

Figure 2. Six feet cube formwork and rebar cage in 3D.

2.2 Placing and Compacting

Concrete will be poured directly from the mixer truck without pumping and then concrete will be subjected to vibration in order to get sufficient compaction as mentioned in WVDOH Construction Manual Section 601 (601.5.9).

2.3 Sampling and Testing

Sampling from fresh concrete and fresh concrete testing will be performed on site by WVU Research Group with the assistance of WVDOH personnel according to the ASTM standards. Slump, entrained air, unit weight, setting time will be determined.

Totally twenty-four 6 inch by 12 inch and eight 4 inch by 8 inch cylinder specimens will be prepared, temperature sensors will be embedded into two 6 inch by 12 inch cylinder. All cylinders will be placed under controlled environment conditions for one day, and then moist cured in temperature controlled curing tanks for the rest of the time. 1, 3, 7, 14 and 28 day compression tests will be performed on 6 inch cylinders and 4 inch cylinders will be tested at the age of 3 day and 28 day at the nearby district laboratory, testing two cylinders at each age.

Eight of the 6 inch cylinder and two of the 4 inch cylinder will be shipped to WVU Laboratories the day after casting.

2.4 Temperature Monitoring

Temperature sensors will be attached by WVU Research Group to the rebar cage before concrete placement.

Figure 3. Temperature sensor locations.

2.5 Finishing, Curing and Formwork Removal

Class 1– Ordinary surface finish using wood-float rubbing will be applied on the top surface as mentioned in WVDOH Construction Manual Section 601 (601.6.4).

It is important that the concrete surface be maintained completely and continuously moist during curing period. After the concrete placement the top of the block will be covered with white polyethylene sheeting. When the surface is finished application of wet-burlap is required. Ensure that the white polyethylene sheeting will be restored after all.

The formwork will be removed seven days after casting and additional curing will be provided right after the removal of forms by applying a liquid membrane-forming curing compound (Linseed oil).

2.6 Coring

At 4, 28 and 56 days after concrete placement, 4 inch by 6 feet core samples will be taken from hardened concrete and 4 inch by 8 inch specimens will be prepared for compression test by WVDOH. These specimens will be prepared and tested immediately after coring in WVDOH facilities.

Figure 4. Coring locations.

Figure 5. Coring positions in 3D.

2.7 Surface Check

Surface of the concrete cube will be checked by WVDOH personnel visually after formwork removal to define the surface defects and cracks. Pictures will be taken if necessary.

APPENDIX D

D1. Lucille Stalnaker Bridge Monitoring

A pier-stem, a pier-cap and an abutment from Lucille Stalnaker Bridge located in District 7 Gilmer County was monitored with temperature sensors during concrete placement. The circular pier stem and the pier cap were constructed with metal forms and the abutment was constructed with wooden forms. Class B Fly Ash concrete poured with 470 pounds per cubic yard (278 kg/m³) TYPE I/II cement and 70 pounds per cubic yard (40 kg/m³) Type F Fly Ash was used for the construction. Limestone sand sub-base was laid over the soil underneath the abutment. Wet burlap and plastic cover were placed after concrete placement at least for 3 days. $\frac{3}{4}$ inch (1.9 cm) thick concrete blankets (R-Value 2.77) were placed after concrete placement on the top surface for at least five days to protect the concrete from severe ambient temperatures. Each element was instrumented at four different locations with a total of eight sensors recording hourly concrete temperature for about 28 days after concrete placement. An extra sensor located at the construction site was used to record air temperature. The initial concrete temperature was recorded 65°F (18°C). The maximum temperature was 129°F (54°C) recorded in the center of the pier stem (CC) at about 20-22 hours after concrete placement (Figure D.1). The minimum temperature measurement at that time was 89°F (31.5°) at the side surface (SC), and the maximum temperature differential was 40°F (22°C) occurred at around 20 hours after casting.

Figure D.1. Temperature-time history from Lucille Stalnaker Pier Stem#1.

The sensors on Pier Cap #1 were activated on 01/10/2011 at 11:30 AM. But the concrete pour was delayed one week, as seen in Figure D.2. The initial concrete temperature was recorded as 54°F (11.5°C) at the concrete plant. The maximum temperature was recorded 118°F (48°C) in the center of the pier stem (CC) at about 27 hours after concrete placement. The minimum temperature measurement at that time was 91°F (33°C) at the side surface (SC), and the temperature differential was 27°F (15°C). However, the maximum temperature differential was 36°F (20°C) occurred at approximately 62 hours after casting (Figure D.2).

Figure D.2. Temperature-time history from Lucille Stalnaker Pier Cap#1.

The abutment was poured on March 30th 2011 using Class B Fly Ash Concrete batched with 470 pounds per cubic yard (278 kg/m³) TYPE I/II cement and 70 pounds per cubic yard (41 kg/m³) Type F Fly Ash. Initial concrete temperature was measured between 59°F (15°C) and 62°F (17°C) for several batches at the concrete plant; hot water (100°F (38°C)) was added to the mixture to obtain the initial concrete temperature. Temperature-time history of the structure for 28 days is shown in Figure D.3. Limestone sand sub-base was laid over the soil underneath the abutment. Wooden forms made from plywood kept in place for about one week and ¾ inch thick blankets (R-Value 2.77) on the top surface stayed for about five days to protect the concrete from severe ambient temperatures. The maximum temperature was 118°F (48°C) recorded in the center of the pier stem (CC) at about 21 hours after concrete placement. The minimum temperature measurement at that time was 84°F (29°C) at the bottom surface (BC). The maximum temperature differential of 34°F (19°C) started occurring at around 16 hours after casting. The effect of the ambient temperature after one week can be observed from the Figure D.3. The influence of the ambient temperature was especially evident on the top (TC) and the side surfaces (SC).

Figure D.3. Temperature-time history from Lucille Stalnaker Abutment#2.

D2. 5th Avenue Bridge Monitoring

The 5th Avenue Bridge is located in District 2, Huntington, WV. Four different structures were instrumented with sensors and monitored for 28 days. The bridge Footer #2 and Abutment #2 were cast using Class B GGBSF concrete that contains 330 pounds per cubic yard (195 kg/m³) of TYPE I/II cement and 260 pounds per cubic yard (154 kg/m³) slag on February 3 and April 1 2011, respectively. The top of concrete was covered with wet burlap, and the forms were wooden. The wooden forms were sheeted with ¾ inch (19 mm) plywood and the forms and burlap stayed on the footer for four days and on the abutment for one week.

The highest temperature reached 99°F (37°C) at the center (CC) of the Footer #2 after about 56-57 hours after concrete placement. The minimum temperature measurement at that time was 81°F (27°C) on the side surface (SC), and the temperature differential was 17°F (9.5°C). It can be observed from the temperature-time history that the entire structure was protected well against the colder ambient temperatures until the formwork removed. The largest temperature differential was 40°F (22°C) and occurred after formwork removal, at about 5 to 6 days age, between the core and the side surface of the footer, since the ambient temperature increases the

rate of cooling on the concrete surface. It can be observed that the top sensor (TC) and the mid sensor (CC) temperatures are raising again after about 8 days after concrete casting which corresponds to the day of Pier Stem #2 casting on February 11th. The temperatures on the side surfaces (SC, FSC) became stable ($\sim 52^{\circ}\text{F}$) after 10 days, so that the temperatures were not influenced by the ambient temperatures. According to the phone conversations with the field engineers it was known that the river was flooded and the footer remained under water during that time (Figure D.4).

Figure D.4. Temperature-time history from 5th Avenue Bridge Footer #2.

Abutment #2 was poured on 2 to 3 inches of limestone sub-base with Class B GGBSF Concrete on April 1st 2011. Wooden formwork was used for the construction and the top of concrete was covered with wet burlap after concrete placement. According to the measurement, the core temperature (CC) reached 99°F (37°C) at about 33-34 hours after concrete placement. The minimum temperature measurement at that time was 79°F (26°C) at the bottom surface (BC), and the temperature differential was 20°F (11°C). The temperature differential between the back surface (SC) and the center of the abutment was 14°F (8°C). Although maximum temperature

differential does not exceed 20°F, the effect of ambient temperatures on the concrete surface can be observed after formwork removal (Figure D.5).

Figure D.5. Temperature-time history from 5th Avenue Abutment#2.

Pier Stem #2 was constructed on February 11th using Class B Modified Concrete. Steel formwork was removed after three days and the concrete was cured with wet burlap. The highest temperature was recorded as 158°F (70°C) after 45 hours at the bottom surface of the column (BC). The core temperature at the same time reaches up to 150°F (66°C) (CC) and the temperature at the side surface was 129°F (54° C) (SC). The differential between the bottom sensor and the side sensor was 29°F (16°C) at that time. However, the maximum temperature differentials occur after about four days. The differential between side surface and the center of the pier stem was 41°F (23°C) and the bottom of the pier stem was 51°F (28°C) (Figure D.6).

Figure D.6. Temperature-time history from 5th Avenue Pier Stem #2.

Pier Cap #2 was constructed on March 28th using steel formwork. Class B Modified Concrete was poured with an initial concrete temperature 66°F (19°C). The top of the pier cap was cured with wet burlap afterwards. Concrete temperatures reach 163°F (73°C) in the center (CC) and 104°F (40°C) on the side surface (SC) of the pier cap after about 35 hours. The temperature difference between the center and the side surface of the pier cap became 76°F (42°C) after about 66 hours. The largest temperature differential occurs at the same time between the center and the far side surface (the end of pier cap), 92°F (51°C). It is the largest temperature differential encountered throughout this study. According to the engineer in the field, the river was flooded the day after the pier-cap casting and the cold water temperature rapidly lowered the concrete surface temperatures (Figure D.7).

Figure D.7. Temperature-time history from 5th Avenue Pier Cap #2.

D3. Clear Fork Arch Bridge #2 Monitoring

One abutment footing and one abutment stem were instrumented and temperature data was collected from the Clear Fork Arch Bridge#2 located in District 10, Wyoming County, WV. The Abutment#1 Footing was constructed on January 18, 2011. Class B Fly Ash concrete with 470 pounds per cubic yard (278 kg/m³) TYPE I/II cement and 70 pounds per cubic yard (41 kg/m³) Type F Fly Ash was used at the construction. Hot water was added to the mixture to increase the initial concrete temperatures up to 60°F (16°C). The average slump and the air content were 3.5 inches (90 mm) and 6.8%, respectively.

Figure D.8. Temperature-time history from Clear Fork Arch#2 Abutment#1 Footing.

According to the field records, the center temperature (CC) reached 122°F (50°C) at about 41 hours after concrete placement. The minimum temperature measurement at that time was 73°F (23°C) at the side surface (SC), and the temperature differential was 49°F (27°C). The largest temperature differential 52°F (29°C) occurred between the core and the side surface of the structure at around 49 hours. The top sensor (TC) temperature is rising significantly after about 7 days after concrete casting which corresponds to the day of Abutment#1 Stem casting (Figure D. 8).

The Abutment #2 Stem was constructed on February 15, 2011 using the same concrete mixture as the footing. The initial temperature was 54°F (12°C), the slump was 4.5 inches (115 mm) and the air content was 4.7%. The highest temperature reached 115°F (46°C) at the center (CC) after about 27 hours after concrete placement. The minimum temperature measurement at that time was 75°F (24°C) on the top surface (TC), and the temperature differential was 40°F (22°C). The effect of the day-night temperature differentials can be observed from the side sensor (SC, FSC) temperatures at early ages (Figure D.9).

Figure D.9. Temperature-time history from Clear Fork Arch#2 Abutment#2 Stem.

D4. Clear Fork Arch Bridge#1 Monitoring

The Clear Fork Arch Bridge#1 Abutment #2 Footing was constructed on January 4, 2011. Class B concrete with 564 pounds per cubic yard (334 kg/m^3) TYPE I cement was used in this construction. The average initial concrete temperature was 52°F (11°C) and the air content was measured 5.8%. Temperature data up to only fifteen days was obtained from the contractor. The maximum concrete temperature recorded after about 18 hours after concrete placement in the center (CC) of the abutment was 97°F (36°C). The minimum concrete temperatures were recorded at the bottom sensor (BC) and the maximum temperature differential was 25°F (14°C) between the core and the bottom center of the abutment (Figure D.10).

Figure D.10. Temperature-time history from Clear Fork Arch#1 Abutment#2 Footing.
D5. South Mineral Wells Bridge Monitoring

South Mineral Wells Interchange Bridge was constructed in District 3, Wood County, WV. A pier stem, pier cap and abutment stem were instrumented with loggers and monitored for 28 days after concrete placement. The South Mineral Wells Interchange Bridge Pier#2 Stem#2 was cast on July 20, 2011 with Class B Fly Ash Concrete that contains 470 pounds per cubic yard (278 kg/m³) TYPE I/II cement and 75 pounds per cubic yard (109 kg/m³) fly ash. The pier stem with seven foot large diameter was poured using steel forms and the top of the stem was covered with wet burlap after concrete casting. The burlap and formworks were removed at 125 hours after casting. According to the concrete batch reports, the initial concrete temperature was 78°F (25°C) to 86°F (30°C) for different batches, slump was measured at 4 inches (100 mm), and the air content was determined to be 4.6%.

The pier stem was 40 feet (12 m) in height above the caisson, therefore concrete temperatures were monitored at three different levels: bottom section (Level I), mid-section (Level II) and top section (Level III). The casting started in the early morning; the first truck was unloaded

at around 6:50 AM and the last truck was unloaded at around 10 AM. The highest temperature was recorded as 156°F (69°C) after 27 hours at the center of the top section (L3-TC). The core temperature of the bottom section (L1-CC) and the mid-section (L2-CC) at the same time reach up to 149°F (65°C). The differential between the center sensor and the side sensor was 34°F (19°C) at that time. The maximum temperature differential is 38°F (21°C) and occurs between the side surface and the center of the pier stem Level III after about 48 hours (Figure D.11).

Figure D.11. Temperature-time history from South Mineral Wells Pier#2 Stem#2.

The 5-foot (1.5 m) thick South Mineral Wells Bridge Pier Cap#2 was poured on August 22, 2011 using the same Class B Fly Ash concrete used in pier stem construction. Steel formwork was used for the construction, and the top of the stem was covered with wet burlap after concrete casting for about a week. According to the concrete batch reports, the initial concrete temperature was between 78°F (25.5°C) to 81°F (27°C) for different batches. One bag of ice (22 lbs each) per cubic yard concrete was added to the mixer truck to control the initial concrete temperature. The slump was measured 2.5 inches (65 mm) to 4.75 inches (120 mm) and the air content was determined to be 7.9%. Steel formwork was removed after about seventeen days and the top surface of the concrete was cured with wet burlap for seven days. According to the field records,

the core temperature (CC) reached 145°F (63°C) at about 23-24 hours after concrete placement. The minimum temperature measurement at that time was 115°F (46°C) at the side surface (SC), and the largest temperature differential was 30°F between the core and the side surface. The ambient temperature was obtained from one of the closest weathercast stations (Figure D.12).

Figure D.12. Temperature-time history from South Mineral Wells Pier Cap#2.

The abutment was constructed on October 2, 2011 using the Class B Fly Ash as well. Wooden formwork was used for the concrete placement. According to the concrete batch tickets average initial concrete temperature was 68°F (20°C), slump was 4 inches (100 mm) and the air content was 9%. The maximum temperature was 117°F (47°C) after 18 hours at the center of the abutment. The maximum temperature differential reaches 34°F (19°C) at around 16 hours between the center sensor and the bottom sensor (Figure D.13).

Figure D.13. Temperature-time history from South Mineral Wells Abutment#2.

E6. Ices Ferry Bridge Monitoring

The Ices Ferry Bridge pier cap #2 was constructed on October 20, 2011 using Class B Modified concrete with 564 pounds per cubic yard (334 kg/m^3) cement and 75 pounds per cubic yard (44 kg/m^3) fly ash. The initial temperature was recorded 65°F (18°C). The slump was measured 4.5 inches (115 mm) and the air content was 4.7%. Concrete temperature was monitored for 28 days. The job site was visited before concrete placement to see whether all the sensors were placed following the instrumentation plan. Unfortunately, most of the surface sensors were placed incorrectly and no location had a secondary sensor attached. Figure D.14 shows a picture of the pier cap from top view. The bottom sensor marked in the picture is attached to the stirrup which is approximately 1 foot (0.3 m) inside the formwork. The project engineer was informed and extra sensors were placed right before the concrete placement, but they were unable to retrieve data from those back-up sensors.

Figure D.14. Sensor attached to the stirrup, Pier Cap #2.

The maximum concrete temperature recorded in the very center (CC) of the pier cap was 135°F (57 °C) around 23-25 hours after concrete placement. The minimum concrete temperature at the same time was 104°F (40°C) at the side surface (SC) of the pier cap and the maximum temperature differential was 31°F (17°C). All other surface sensors including top center (TC), bottom center (BC), and far side center (FSC) were embedded inside the concrete more than required (2 inches) which leads to a higher maximum temperature for that location (Figure D.15).

Figure D.15. Temperature-time history from Ices Ferry Pier Cap#2.

E1. Creating 3D Mesh

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```

delx = xtot/(nx-1);
dely = ytot/(ny-1);
delz = ztot/(nz-1);

for i = 2:nx-1
    x(i) = x(i-1) + delx;
end
for i = 2:ny-1
    y(i) = y(i-1) + dely;
end
for i = 2:nz-1
    z(i) = z(i-1) + delz;
end

```

E2. Solution for the Internal Nodes

```

%----- Internal nodes -----
for i = 2:nx-1
for j = 2:ny-1
for kk = 2:nz-1

k_e = (k(i,j,kk)+k(i+1,j,kk))/2;
k_w = (k(i,j,kk)+k(i-1,j,kk))/2;
k_n = (k(i,j,kk)+k(i,j+1,kk))/2;
k_s = (k(i,j,kk)+k(i,j-1,kk))/2;
k_b = (k(i,j,kk-1)+k(i,j,kk))/2;
k_t = (k(i,j,kk)+k(i,j,kk+1))/2;

cp_e = (cp(i,j)+cp(i+1,j))/2;
cp_w = (cp(i,j)+cp(i-1,j))/2;
cp_n = (cp(i,j)+cp(i,j+1))/2;
cp_s = (cp(i,j)+cp(i,j-1))/2;
cp_b = (cp(i,j,kk-1)+cp(i,j,kk))/2;
cp_t = (cp(i,j,kk)+cp(i,j,kk+1))/2;

aeast = (k_e*dt)/(rho*cp_e*delx.^2);
awest = (k_w*dt)/(rho*cp_w*delx.^2);
anorth = (k_n*dt)/(rho*cp_n*dely.^2);
asouth = (k_s*dt)/(rho*cp_s*dely.^2);
abottom = (k_b*dt)/(rho*cp_b*delz.^2);
atop = (k_t*dt)/(rho*cp_t*delz.^2);

sc = phi_old(i,j,kk)*...
(1-(aeast+awest+anorth+asouth+abottom+atop))...
+source_old(i,j,kk)*dt./(rho*cp(i,j,kk));
phi(i,j,kk) = aeast*phi_old(i+1,j,kk) ...
+awest*phi_old(i-1,j,kk) ...
+anorth*phi_old(i,j+1,kk) ...
+asouth*phi_old(i,j-1,kk) ...
+abottom*phi_old(i,j,kk-1)...
+atop*phi_old(i,j,kk+1)...
+sc;
end

```

APPENDIX F

Table F.1. Six-Ft Cube Core Specimen Test Results

Specimen Designation		1C	2C	3C	4C	5C	6C
Depth from the surface, inches		2"-10"	14"-22"	26"-34"	38"-46"	50"-58"	62"-70"
D1 CUBE	4 Days	3,160	4,670	4,830	4,690	4,850	4,930
	28 Days (center)	4,750	5,640	5,600	4,950	6,460	6,540
	29 Days (1 ft away)	4,370	5,600	5,640	5,490	6,070	5,900
	56 Days	4,690	6,130	5,920	5,820	6,370	6,410
D9 CUBE	4 Days	2,420	2,660	-	3,620	3,670	4,010
	28 Days (center)	2,960	2,670	2,520	3,710	3,630	4,120
	28 Days (1-ft away)	3,150	2,625	2,510	3,790	3,740	4,210
	56 Days	3,350	2,730	2,640	4,000	3,840	4,330
D5 CUBE	4 Days	3,880	4,790	4,790	4,870	4,790	5,300
	28 Days (center)	4,460	6,080	5,820	5,570	5,630	6,960
	28 Days (1 ft away)	4,510	4,800	5,150	6,040	5,700	6,590
	76 Days*	4,180	5,750	5,580	5,310	6,090	7,430
D6 CUBE	4 Days	4,460	5,710	4,100	3,310	5,250	5,250
	28 Days (center)	6,010	6,440	5,150	6,490	6,210	6,230
	28 Days (1 ft away)	5,730	6,160	5,450	5,980	6,090	6,400
	56 Days	5,390	6,530	6,160	6,590	6,630	6,440
Note: 1 psi = 6.89 kPa							