Adiabatic Temperature Rise of Mass Concrete in Florida

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BY

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16. Abstract

Currently FDOT mandates contractors to provide an analysis of the anticipated thermal developments in mass concrete elements. Contractors typically use Schmidt's method in conjunction with adiabatic temperature rise curves published by ACI 207 Committee for concrete with different cement types and placement temperatures. These curves were developed a few decades ago by testing concrete mixes made with ASTM approved cements. Currently, FDOT specifies AASHTO approved cements, which have different chemical composition and fineness. In addition, ACI 207 curves should be modified for calculating the heat development when pozzolanic materials such as fly ash or slag are used to replace cement. The objective of this project was to develop and recommend a set of adiabatic temperature rise curves for typical mass concretes used in the State of Florida. Adiabatic temperature-rise tests of concrete made with AASHTO type II cements, pozzolan, and locally available coarse aggregates were performed in the laboratory under conditions that represent those that will occur in the field. A total of 20 mixes with cements from two different sources and various percentages of pozzolanic materials were placed at two different temperatures and tested for adiabatic temperature rise, thermal diffusivity and compressive strength. The Heat of hydration of cement samples and blends of cement and pozzolan were also determined. The results of this study showed that the reduction of peak temperature due to replacing cement with pozzolan depends on the percentage of pozzolan and concrete placing temperature. For mixes with 73ºF placing temperature, the addition of pozzolan had a larger reducing effect on the peak temperature than those placed at 95ºF. A pozzolan modification factor was developed based on the type and percentage of pozzolan in the mix and its placing temperature. This factor represents the percentage of heat that pozzolan produces compared to the cement that it replaces. The average thermal diffusivity for the mixes in this research project was determined to be 0.80 ft^2/day, which is about 35% less than the 1.22 ft^2/day reported in ACI 207. The results also showed that thermal diffusivity of concrete reduces when Portland cement is replaced with high percentage of pozzolans (50% or higher). Comparison between the 28-day compressive strength of concrete samples cured at room temperature and those cured at high temperature (160-190ºF) revealed that high curing temperature reduces compressive strength. For 73ºF placing temperature, this reduction was 20 percent for plain cement concrete. Addition of high percentage of pozzolans reduced the negative effect of high curing temperature on compressive strength. The results of this study showed that the current method used by the FDOT contractors to predict thermal developments in mass concrete elements underestimates the maximum temperature rise. It is recommended to develop an analytical model that can more accurately predict the rate of heat generation and the maximum temperature rise of a mass concrete element based on its mix design, placement temperature, geometry, and environmental conditions.

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EXECUTIVE SUMMARY

FDOT Structures Design Guidelines defines mass concrete as “any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat and attendant volume change so as to minimize cracking.” Mass concrete is used in many projects related to the Florida Department of Transportation such as bridge foundations, bridge piers, and concrete abutments. Thermal action, durability and economy are the main factors in the design of mass concrete structures. The most important characteristic of mass concrete is thermal behavior. Hydration of Portland cement is exothermic and a large amount of heat is generated during the hydration process of mass concrete elements. Since concrete has a low conductivity, a great portion of generated heat is trapped in the center of a mass concrete element and escapes very slowly. This situation leads to a temperature difference between the center and outer part of the mass concrete element. Temperature difference is a cause for tensile stresses, which forms thermal cracks in concrete structure. Thermal cracks may cause loss of structural integrity and shortening of the service life of the concrete element.

The rate and magnitude of heat generation of the concrete depends on the amount per unit volume of cement and pozzolan, the compound composition and fineness of cement, the shape of the concrete element and its volume to surface ratio, the initial temperature of the concrete, the ambient temperature, and the other surrounding conditions.

Currently FDOT mandates contractors to provide an analysis of the anticipated thermal developments in mass concrete elements. Contractors typically use Schmidt’s method in conjunction with ACI adiabatic temperature rise curves. Schmidt’s method is

one of the most frequently used in connection with temperature studies for mass concrete structures in which the temperature distribution is to be estimated. This method uses the adiabatic temperature rise curves that the ACI 207 Committee has published for concrete with different cement types and placement temperatures. These curves were developed a few decades ago by testing concrete mixes made with ASTM approved cements [ACI 207.1 R-96]. However, FDOT specifies AASHTO approved cements, which have different chemical composition and fineness. In addition, ACI 207 curves should be modified for calculating the heat development when pozzolanic materials such as fly ash or slag are used to replace cement. A rule of thumb suggested by ACI 207 to assume that pozzolan produces only about 50 percent as much heat as the cement that it replaces is only for preliminary computations and is not accurate.

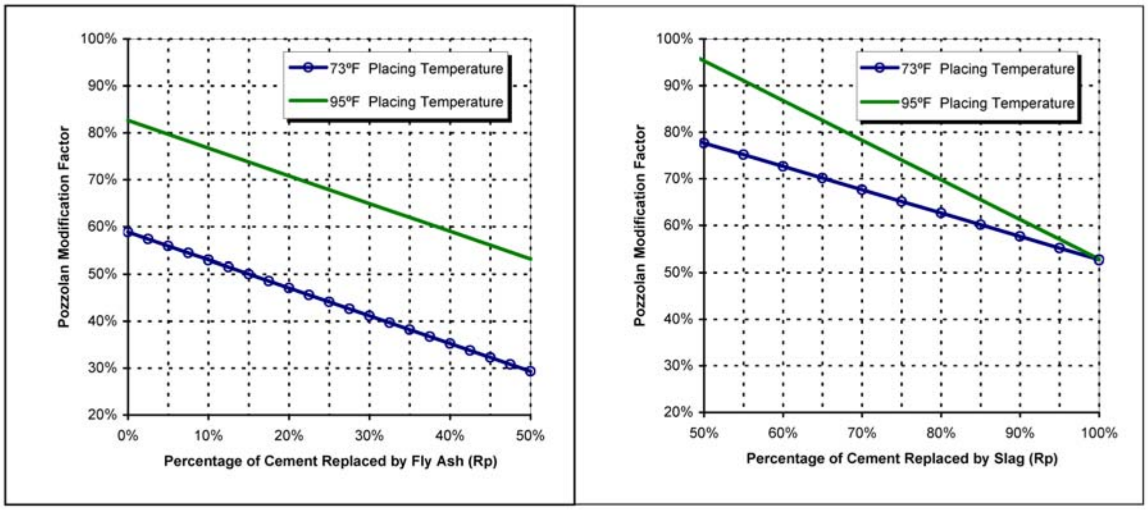
The objective of this project was to develop and recommend a set of adiabatic temperature rise curves for typical mass concretes used in the State of Florida. These curves will be used to predict the expected temperature increase in mass concrete structures, which is of primary importance with regard to controlling heat of hydration.

A literature review was conducted to identify the factors affecting temperature rise in concrete and to study previous works in this field. The literature review showed that in recent years with the advancements in computing technology several attempts have been made to develop numerical methods and computer software to predict the temperature rise in a mass concrete element.

A previous survey on mass concrete showed that only nine states including Florida have mass concrete specifications (California, Idaho, Illinois, Kentucky, North Carolina, South Carolina, Texas and Virginia). The State Materials Engineers in these states were contacted through e-mail to investigate any recent changes they have made in their specifications and to seek their opinions regarding the accuracy and use of ACI 207 curves. Of the four states that responded two are using ACI 207 curves and believe they are fairly accurate (Kentucky, South Carolina), Illinois does not require contractors to submit a mass concrete plan, and California leans to the side of performance specifications and requires the contractor to deliver crack free concrete.

Adiabatic temperature-rise tests of concrete made with AASHTO type II cements, pozzolan, and locally available coarse aggregates were performed in the laboratory under conditions that represent those that will occur in the field. A total of 20 mixes with cements from two different sources, two different placing temperatures and various percentages of pozzolanic materials were tested for adiabatic temperature rise, thermal diffusivity and compressive strength. The Heat of hydration of cement samples and blends of cement and pozzolan were also determined.

It is generally believed that replacing cement with pozzolan has a reducing effect on the peak temperature of concrete. However, the amount of reduction has been reported differently in various sources. The results of this study showed that the amount of reduction depends on the percentage of pozzolan and concrete placing temperature. For mixes with 73ºF placing temperature, the addition of pozzolan had a larger reducing effect on the peak temperature than those placed at 95ºF. For concrete mixes with 95ºF placing temperature replacement of cement with 50% slag did not reduce the peak temperature. Based on the results of this study a pozzolan modification factor (α_p) was developed based on the type and percentage of pozzolan in the mix (R_p) and its placing temperature (see figure below). This factor represents the percentage of heat that pozzolan produces compared to the cement that it replaces. It also can be used to calculate equivalent cement content which is needed to convert the adiabatic temperature rise curve of the base mix (plain cement) into the adiabatic temperature rise curve for a mix with pozzolan.



Relationship Between α_p and R_p

The following table shows how placing temperature and type and percentage of pozzolan affect the pozzolan modification factor.

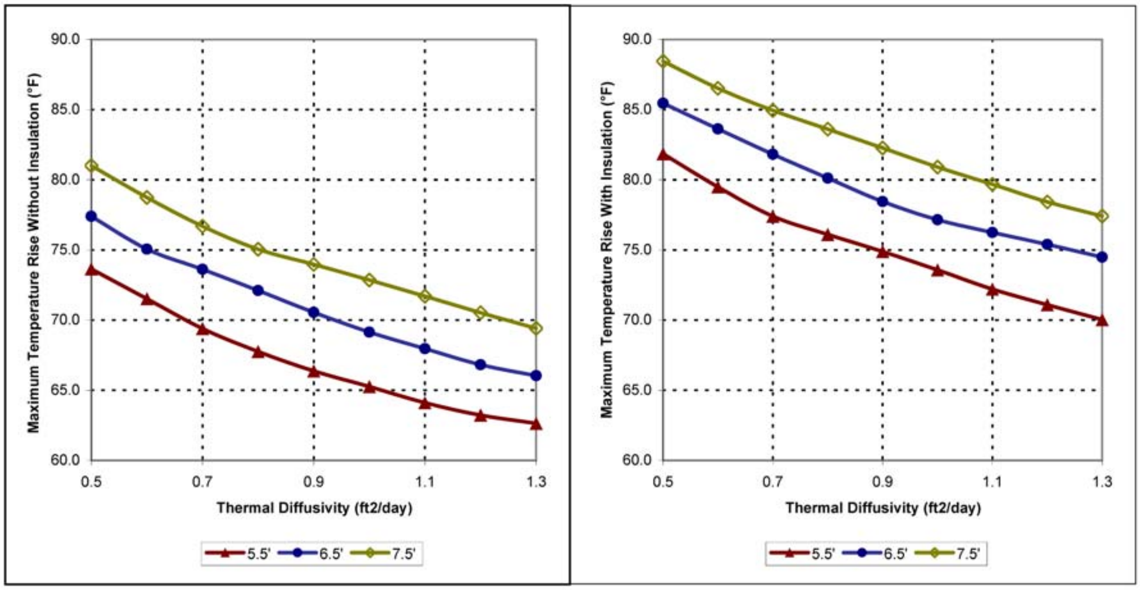
Pozzolan Modification Factor for Mixtures tested in this Study

R_p	20% FA	20% FA	35% FA	35% FA	50% SL	50% SL	70% SL	70% SL
Placing Temp.	73ºF	95ºF	73ºF	95ºF	73ºF	95ºF	73ºF	95ºF
α_p	0.47	0.71	0.38	0.62	0.78	0.95	0.68	0.78

Currently, FDOT contractors assume that fly ash and slag produce respectively 50 and 75 percent as much heat as the cement that they replace. They do not consider the effects of concrete placing temperature and percentage of pozzolan on the value of pozzolan modification factor.

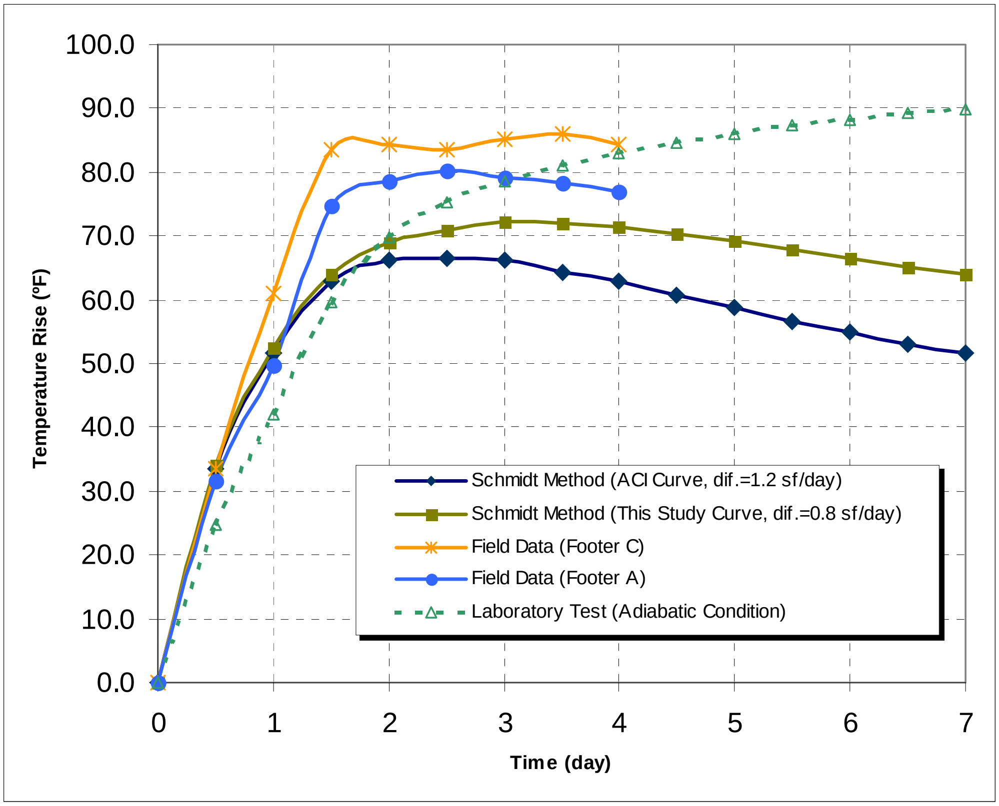
The laboratory test results were compared to the field temperature recordings of the core of footers of the Bella Vista Bridge (a bridge over I-4 west of Memorial Blvd in Lakeland) to verify that the hydration chambers used in this experiment simulate the real conditions of the mass concrete elements in the field. The average peak temperature rise recorded for the three footings after 64 hours was 81.5ºF. The laboratory test of the samples taken from the field and kept in the heat chambers showed a temperature rise of 78.5ºF after 64 hours, which is consistent with the field recordings.

Another factor that affects prediction of the mass concrete peak temperature is thermal diffusivity of concrete. The ACI 207 recommended diffusivity value for concrete made with limestone aggregate is 1.22 ft^2/day. However, no information has been provided as to the maximum aggregate size of concrete used to determine thermal diffusivity. The number reported by ACI 207 may be originated from measuring diffusivity of concrete samples used in dams where coarse aggregates occupy more than 80% of the concrete volume. A coarse aggregate in FDOT mass concrete mixes occupies about 50% of the mix volume. The rest is filled with a mixture of cement, fine aggregate and water with a thermal diffusivity of about 0.4 ft^2/day. The average thermal diffusivity for the mixes in this research project was determined to be 0.80 ft^2/day, which is about 35% less than the 1.22 ft^2/day reported in ACI 207. The results also showed that thermal diffusivity of concrete reduces when Portland cement is replaced with high percentage of pozzolans (50% or higher). Thermal diffusivity number affects the calculations of the maximum temperature and temperature difference (thermal ingredient) of a mass concrete element. The following figures show that for footers with different thicknesses and boundary conditions, changing thermal diffusivity from 1.2 ft^2/day to 0.8 ft^2/day will increase the maximum temperature rise by 5ºF.



Effect of Thermal Diffusivity on Maximum Temperature Rise

For footers of the Bella Vista Bridge the maximum temperature recorded in the field is almost 20ºF higher than the maximum temperature predicted using Schmidt Method in conjunction with ACI 207 curves and 1.22 ft^2/day suggested diffusivity number (see figure below).



Temperature Rise Prediction and Field Records for a 5.5' Footing

Comparison between the 28-day compressive strength of concrete samples cured at room temperature and those cured at high temperature (160-190ºF) revealed that high curing temperature reduces compressive strength. For 73ºF placing temperature, this reduction was 20 percent for plain cement concrete (see table below). Addition of high percentage of pozzolans reduced the negative effect of high curing temperature on compressive strength. However, this was not true for 95ºF placing temperature.

Percentage of Reduction in Compressive Strength due to High curing Temperature

Type of Mix	73ºF Placing Temp.	95ºF Placing Temp.	Overall
Plain Cement	20%	10%	16%
20% Fly Ash	12%	12%	12%
35% Fly Ash	03%	20%	09%
50% Slag	11%	17%	14%
70% Slag	02%	06%	04%

The results of this study showed that the current method used by the FDOT contractors to predict thermal developments in mass concrete elements underestimates the maximum temperature rise. The data used by the contractors (ACI 207 adiabatic temperature rise curves, concrete thermal diffusivity, pozzolan modification factors) are approximate and do not represent the materials used in Florida mass concrete mixes. It is recommended to develop an analytical model that can more accurately predict the rate of heat generation and the maximum temperature rise of a mass concrete element based on its mix design, placement temperature, geometry, and environmental conditions. Data obtained in this study could be used for verification of such analytical model; however, more data is needed to verify the accuracy of the model and make it comprehensive. The pozzolan modification factor developed in this study should be verified for several mass concrete mixes and revised if necessary. It is important to note that this factor was developed for the type of fly ash and slag used in this study. Changing the source of pozzolans may affect the pozzolan modification factor.

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CHAPTER 1
INTRODUCTION

Objectives

Mass concrete is used in many projects related to the Florida Department of Transportation (FDOT) such as bridge foundations, bridge piers, and concrete abutments. Since cement hydration is an exothermic reaction, the temperature rise within a large concrete mass can be quite high. As a result, significant tensile stresses and strains may develop from the volume change associated with the increase and decrease of temperature within the mass concrete. It is, therefore, necessary to predict the temperature rise and take measures to prevent cracking due to thermal behavior. Cracks caused by thermal gradient may cause loss of structural integrity and monolithic action or shortening of service life of the structures.

The prediction of temperature rise is important in controlling the heat of hydration. The objective of this research is to develop the adiabatic temperature rise curves of different types of mass concrete used in FDOT projects. These curves will be used to predict the expected temperature rise in mass concrete structures used in FDOT projects.

Background

FDOT Structures Design Guidelines defines mass concrete as “any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat and attendant volume change so as to minimize cracking” (FDOT, 2002). Thermal action, durability and economy are the main factors in the design of mass concrete structures. The most important characteristic of mass concrete is thermal behavior. Hydration of Portland cement is exothermic and a large amount of heat is generated during the hydration process of mass concrete elements. Since concrete has a low conductivity, a great portion of generated heat is trapped in the center of mass concrete element and escapes very slowly. This situation leads to a temperature difference between center and outer part of the mass concrete element. Temperature difference is a cause for tensile stresses, which forms thermal cracks in concrete structure. These cracks are called thermal cracks. Thermal cracks may cause loss of structural integrity and shortening of the service life of the concrete element.

Predicting the maximum temperature of mass concrete has always been the main concern of designers and builders of mass concrete structures. One of the earliest efforts to predict the maximum temperature of the mass concrete were carried in late 20's and early 30's during the design phase of Hoover Dam (Blanks, 1933). Later on various studies were performed to develop methods to predict the maximum temperature in mass concrete elements.

One of the most popular methods to predict the mass concrete peak temperature rise is using adiabatic temperature rise curves. These curves have been developed for concrete with different cement types and placing temperature. American Concrete Institute (ACI) Committee 207 has published adiabatic temperature rise curves that are widely used.

Currently FDOT mandates contractors to provide temperature rise predictions for mass concrete pours using ACI adiabatic temperature rise curves. These curves were developed a few decades ago by testing concrete mixes made with American Society for Testing Materials (ASTM) approved cements (ACI 207.1 R-96). However, FDOT specifies American Association of State Highway Officials (AASHTO) approved cements, which have different chemical composition and fineness.

Research is needed to investigate if the temperature rise predictions using ACI curves are accurate for mass concrete mixes used in Florida projects. The objective of this research is to study adiabatic temperature rise in mass concrete for concrete mixes which are used in Florida.

Scope of Work

Literature Review

A comprehensive review of the previously performed research on adiabatic temperature rise of mass concrete was undertaken. In this review researches on thermal diffusivity of concrete were also studied.

Mix Design Selection

A comprehensive list of concrete mix designs approved by the FDOT since 1990 was compiled. The list of mix designs was analyzed and different designs were categorized based on cement type, aggregate type, type and ratio of pozzolanic materials, cement suppliers, and pozzolanic materials suppliers.

More frequently used mixes were chosen as representative mixes in each category. Representative mixes were tested to develop adiabatic temperature rise curves.

Survey of State Highway Agencies

A group of states were selected based on previous studies about mass concrete regulations. A questioner was send via e-mail to Material Engineers in selected states concerning each state's regulation on temperature rise in mass concrete to see if they have ever undertaken any research to develop adiabatic temperature raise curves for mass concrete. The results of this survey are presented in Appendix A.

Concrete Testing

In this phase, concrete mixes were prepared based on different mix designs selected earlier. Samples were tested for slump, air content, adiabatic temperature rise, compressive strength, and thermal diffusivity.

Data Analysis

After collecting data from the tests, adiabatic temperature rise curves were developed for each mix and the new curves were compared to ACI curves for similar concrete mixes. Also a correction factor was determined to predict the adiabatic temperature rise curves for the mixes with pozzolanic materials.

Concrete thermal diffusivity test results led to a lower diffusivity number for concretes used in Florida compared to ACI suggested numbers.

CHAPTER 2
LITERATURE REVIEW

Introduction

Before focusing on the adiabatic temperature rise of mass concrete, it is necessary to review the definition and characteristics of mass concrete and its components. Also, methods to predict the temperature rise of mass concrete are reviewed.

Mass Concrete

As mentioned before, 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 the cement and attendant volume change to minimize cracking" (ACI 116R). In the design of mass concrete structures thermal action, durability and economy are the main factors that are taken into consideration, while strength is often a secondary concern. Since water cement reaction is an exothermic reaction and mass concrete structures have large dimensions, the most important characteristic of mass concrete is thermal behavior. A large amount of heat is generated during the hydration process of cementitious material in mass concrete elements. A great portion of generated heat that is trapped in the center of mass concrete element escapes very slowly because concrete has a low conductivity. This situation leads to a temperature difference between center and outer part of the mass concrete element. Temperature difference is a cause for tensile strains, which in turn is a source for tensile stress. Tensile stress forms cracks in concrete structure. These cracks are called thermal cracks. Thermal cracks may cause loss of structural integrity and shortening of the service life of the concrete element.

Thermal cracks were first observed in dam construction. Other thick section concrete structures including mat foundations, pile caps, bridge piers, thick walls and tunnel linings also experienced temperature related cracks (ACI 207.1 R-96).

Mass Concrete History

During years 1930 to 1970 mass concrete construction developed rapidly. Some records are available from this period of using mass concrete in few dams. These records show wide internal temperature variation due to cement hydration. The degree of cracking was associated with temperature rise (ACI 207.1 R-96).

Hoover Dam was in the early stages of planning by the beginning of 1930. Since there were no examples of such a large concrete structure before Hoover Dam, an elaborate investigation was undertaken to determine the effects of composition and fineness of cement, cement factor, temperature of curing and, maximum size of aggregate on heat of hydration of cement, comprehensive strength, and other properties of concrete. Blanks (1933) reported some of the findings of these investigations. The results of these investigations led to the use of low heat cement and embedded pipe cooling system in Hoover dam. Low heat cement was used for the first time in construction of Morris Dam, near Pasadena, California, a year before Hoover Dam (ACI 207.1 R-96).

Chemical admixtures as materials that could benefit mass concrete were recognized in the 1950s. Wallace and Ore (1960) published their report on the benefit of these materials to lean mass concrete in 1960. Since then, chemical admixtures have come to be used in most mass concrete (ACI 207.1 R-96).

Use of purposely-entrained air for concrete became a standard practice in about 1945. Since then, air-entraining admixtures were used in concrete dams and other structures such as concrete pavements (ACI 207.1 R-96).

Placement of conventional mass concrete has not largely changed since the late 1940s. Roller-compacted concrete is the new major development in the field of mass concrete (ACI 207.1 R-96).

Portland Cement Types

AASHTO standard specifications for Portland cement (M85-93) cover different types of Portland cement.

Table 2-1 shows AASHTO requirements for Type I, Type II, Type III, Type IV and Type V cements are shown.

Table 2-1 AASHTO M 85 Standard Requirements for Portland Cement

Type of Cement	SiO_2 min	Al_2O_3 max	Fe_2O_3 max	SO_3 $C_3A<8$	SO_3 $C_3A>8$	C_3S max	C_2S min	C_3A max
Type I When Special properties specified for any other type are not required	-	-	-	3	3.5	-	-	-
Type II When moderate sulfate resistance or moderate heat of hydration is desired	20	6	6	3	-	58*	-	8
Type III When high early strength is desired	-	-	-	3.5	4.5	-	-	15
Type IV When low heat of hydration is desired	-	-	6.5	2.3	-	35	40	7
Type V When high sulfate resistance is desired	-	-	-	2.3	-	-	-	5

* Not required for ASTM C 150

- Type I Portland cement is commonly used in general construction. It is not recommended for use by itself in mass concrete without other measures that help to control temperature problems because of its substantially higher heat of hydration (ACI 207.1 R-96).
- Type II Portland cement is suitable for mass concrete construction because it has a moderate heat of hydration important to control cracking.
- Type IP portland-pozzolan cement is a uniform blend of Portland cement or Portland blast-furnace slag cement and fine pozzolan.

Composition and Hydration of Portland Cement

Portland cement is a composition of several different chemicals: SiO_2, Al_2O_3, Fe_2O_3, MgO, SO_3, C_3A, C_3S, C_2S and C_3AF are the main components of Portland cement. The proportions of these components change in different types of cements.

The major compounds of Portland cement are C_3S, C_2S, C_3A and C_3AF. These constituents have different contributions in heat of hydration of cement. Table 2-2 shows heat of hydration of main components of Portland cement as reported by Cannon (1986). These numbers have been originally determined by heat of solution method by Lerch and Bogue (1934).

Table 2-2 Specific Heat of Hydration of Individual Compounds of Portland Cement

Compound	Specific Heat of Hydration (cal/gr)
C_3S	120
C_2S	62
C_3A + gypsum	320
C_3AF	100

Heat of hydration of Portland cement can be calculated as the sum of specific heat of each compound weighted by the mass percentage of the individual compound (Swaddiwudhipong et al., 2002).

C_3A reaction with gypsum to form ettringite releases about 150 cal/g. After the depletion of gypsum, C_3A reacts with ettringite forming a more stable monosulphate and releases additional heat of hydration of 207 cal/g. Therefore the total heat of hydration of C_3A and gypsum is 357 cal/g (Swaddiwudhipong et al., 2002). However, Swaddiwudhipong (2002) suggests the total heat of hydration of C_3A and gypsum be considered as 320 cal/g. This suggestion is based on a series of least square analyses carried out by Detwiler (1996).

Fly Ash and Blast Furnace Slag

In most of the FDOT approved mix designs, fly ash or slag has been used. Fly ash is the flue dust from burning ground or powdered coal. Suitable fly ash can be an excellent pozzolan if it has a low carbon content, a fineness about the same as that of Portland cement, and occurs in the form of very fine, glassy spheres. Because of its shape and texture, the water requirement is usually reduced when fly ash is used in concrete.

Class F fly ash is designated in ASTM C 618 and originates from anthracite and bituminous coals. It consists mainly of alumina and silica and has a higher loss on ignition (LOI) than Class C fly ash. Class F fly ash also has a lower calcium content than Class C fly ash. Additional chemical requirements are listed in Table 2-3.

Table 2-3 Class F Fly Ash Chemical Properties

Property	ASTM C618 Requirements, %
SiO_2 plus Al_2O_3 plus Fe_2O_3, min	70
SO_3, max	5
Moisture content, max	3
Loss on Ignition, max	6

Finely ground granulated iron blast-furnace slag may also be used as a separate ingredient with Portland cement as cementitious material in mass concrete (ACI 207.1 R-96).

Ground granulated blast furnace slag (GGBFS) is designated in ASTM C 989 and consists mainly of silicates and aluminosilicates of calcium. GGBFS is divided into three classifications based on its activity index. Grade 80 has a low activity index and is used primarily in mass structures because it generates less heat than Portland cement. Grade

100 has a moderate activity index, is most similar to Portland cement with respect to cementitious behavior, and is readily available. Grade 120 has a high activity index and is more cementitious than Portland cement. To be used in cement, GGBFS must have the chemical requirements listed in Table 2-4.

Table 2 4 GGBFS Chemical Composition

Chemical	Maximum Requirements (ASTM 989), %
Sulfide sulfur (S)	2.5
Sulfate ion reported as SO_3	4

Thermal Properties of Concrete

It is essential to know the thermal properties of concrete to deal with any problem caused by temperature rise. These properties are specific heat, conductivity and diffusivity. The main factor affecting the thermal properties of concrete is the mineralogical composition of the aggregate (Rhodes, 1978).

The specific heat is defined as the amount of thermal energy required to change the temperature per unit mass of material by one degree. Values for various types of concrete are about the same and vary from 0.22 to 0.25 Btu's/pound/°F. Lu (Lu et. al., 2001) reported that changes in aggregate types, mixture proportions, and concrete age did not have a great influence on the specific heat of ordinary concrete at normal temperature, as concrete volume is mainly occupied by aggregates with thermal stability.

The thermal conductivity of a material is the rate at which it transmits heat and is defined as the ratio of the flux of heat to the temperature gradient. Water content, density, and temperature significantly influence the thermal conductivity of a specific concrete. Typical values are 2.3, 1.7, and 1.2 British thermal units (Btu)/hour/foot/Fahrenheit degree (°F) for concrete with quartzite, limestone, and basalt aggregates, respectively (USACE, 1995).

Thermal Diffusivity is described as an index of the ease or difficulty with which concrete undergoes temperature change and, numerically, is the thermal conductivity divided by the product of specific heat and density (USACE, 1995). Aggregate type largely affects concrete thermal diffusivity. In ACI 207.1R-96 typical diffusivity values for concrete are from 0.77 ft^2/day for basalt concrete to 1.22 ft^2/day for limestone concrete to 1.39 ft^2/day for quartzite concrete. Other sources report slightly different numbers for concrete thermal diffusivity. Vodak and associates (1997) report thermal diffusivity numbers between 0.73 to 1.16 ft^2/day for various siliceous concretes.

Thermal diffusivity of cement paste and mortar (cement + sand) is lower than concrete. Xu and Cheng (2000) measured cement paste and mortar thermal diffusivity with laser flash method. In the laser flash method a laser beam is flashed to one side of the specimen (a disc with 13mm diameter and 2mm in thickness). Temperature of the other side of the specimen is measured by a thermocouple and thermal diffusivity is calculated from the temperature vs. time curve. Thermal diffusivity of cement paste without silica fume and mortar without silica was determined to be 0.33 ft^2/day and 0.41 ft^2/day respectively (Xu and Cheng, 2000).

Factors Affecting Heat Generation of Concrete

The total amount of heat generated during hydration of concrete, as well as the rate of heat generation, is affected by different factors.

The first factor affecting heat generation and the total amount of heat generated is the type of the cement used. As mentioned before, the ratio of chemical compounds is different in each cement type. Heat of hydration is also different for each of the cement chemical compounds. Therefore combination of these compounds with various portions results in different heats of hydration for each cement type (See Figure 2-1). Tricalcium Silicate (C_3S) and Tricalcium Aluminate (C_3A) generate more heat and at a faster rate than other cement compounds (Copland et. al., 1960). Therefore concrete mixes containing cement types with higher percentage of C_3S and C_3A generate more heat.

Cement content of concrete is the next factor in heat generation. As cement is the main source of heat generation during the hydration process, larger portion of cement leads to larger amount of heat generated.

Another factor that affects the thermal behavior of concrete is cement fineness. The cement fineness affects the rate of heat generation more than the total generated heat (Price, 1982). Greater fineness increases the surface available for hydration, causing more rapid generation of heat (the fineness of Type III is higher than that of Type I cement) (U.S. Dept. Trans. 1990). This causes an increase in rate of heat liberation at early ages, but may not influence the total amount of heat generated in several weeks.

Figure 2-2 shows how cement fineness affects rat of heat generation in cement paste. These curves were developed by Verbeck and Foster for cement paste specimen cured at 75°F (ACI 207.2R).

Placing temperature of concrete is another effective element on the maximum temperature of concrete. A higher initial temperature results in higher maximum temperature. The inner part of a mass concrete element is in a semi-adiabatic condition, which means heat exchange with the outer environment is very difficult. Therefore the initial heat entraps and the heat of hydration adds to the initial temperature. Placing temperature also affects the rate of adiabatic temperature rise of concrete.

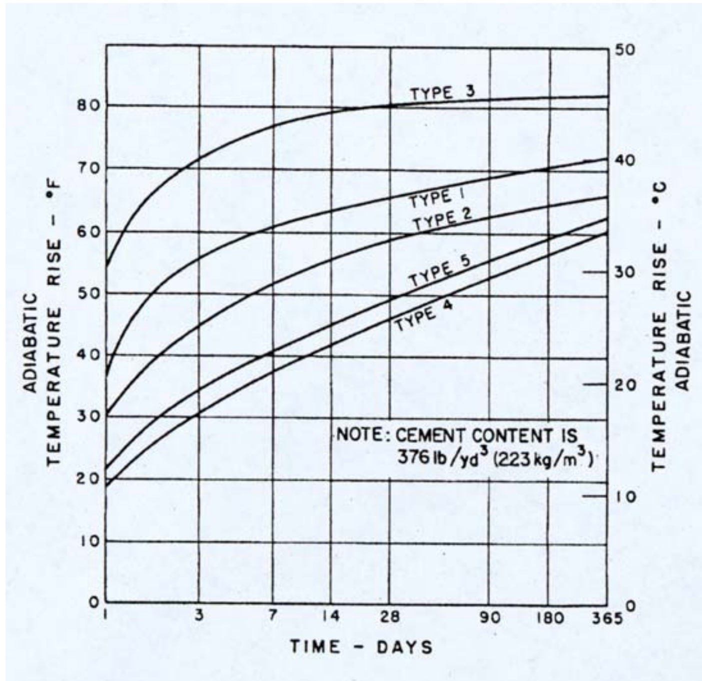


Figure 2-1 Adiabatic Temperature Rise in Different Types of Concrete (ACI 207.1R-96)

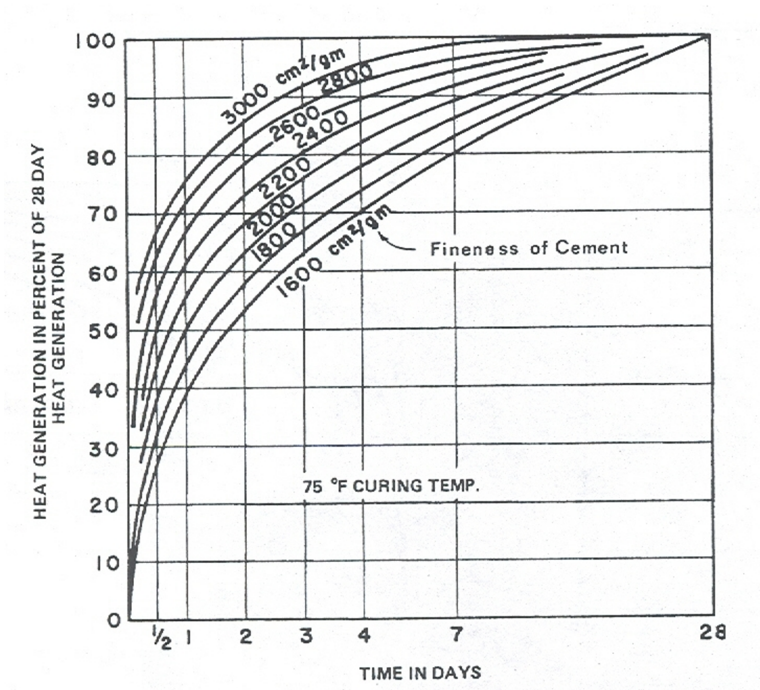


Figure 2-2 Rate of Heat Generation as Affected by Fineness of Cement (ACI 207.2R-96)

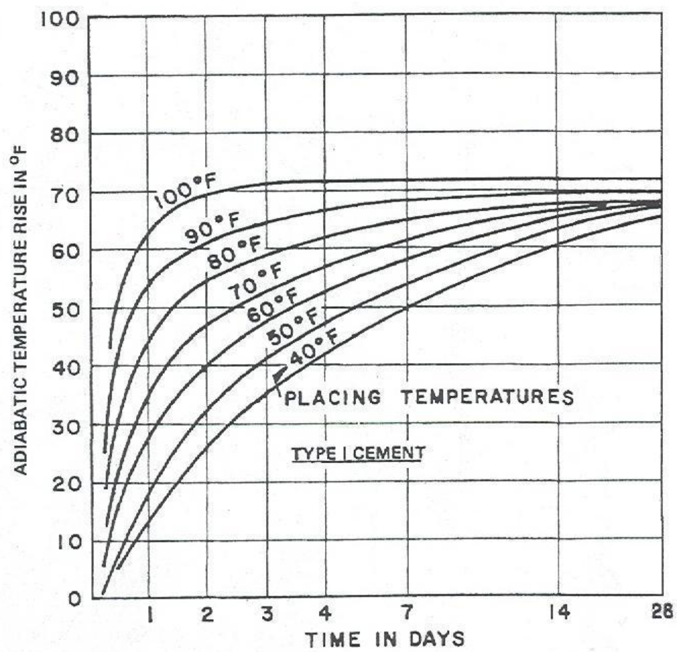


Figure 2-3 Effect of Placing Temperature and Time on Adiabatic Temperature Rise of Mass Concrete Containing 376 lb/yd^3 of Type I Cement (ACI 207.2R-96)

As it is shown in Figure 2-3 concrete mixes with higher placing temperatures reach the maximum temperature faster than concrete mixes with low placing temperature. However the maximum temperature is not significantly different within the mixes with different placing temperatures (ACI 207.2R-96).

Replacement of cement with pozzolanic materials in the concrete mix reduces maximum temperature. One of the earliest indication of effects of fly ash on heat generation was made by Davis et. al. (1937). The first use of fly ash in mass concrete was reported by Philleo (1967).

Bamforth (1980) performed an extensive study on effects of using fly ash or GGBFS (Slag) on the performance of mass concrete. He monitored temperature and strain in three pours of mass concrete each 14.75 ft deep, which formed part of foundation of a grinding mill in the United Kingdom. The total cementitious material in each pour was 675 lb/yd^3 In one of the pours, 30% of cement was replaced by fly ash. In another pour, 75% of cement was replaced bay GGBFS. OPC (Ordinary Portland Cement) was used in the mixes. As reported, the cement contained 13.6% of C_3A so it will be categorized as Type III cement in the AASHTO standard (See Table). Temperature rise of concrete was measured by Copper/Constantan thermocouples placed in foundations. Initial temperature of the concrete was measured immediately after each thermocouple was covered by concrete. Temperature rise was recorded for 40 days. The results are shown in Figure 2-4.

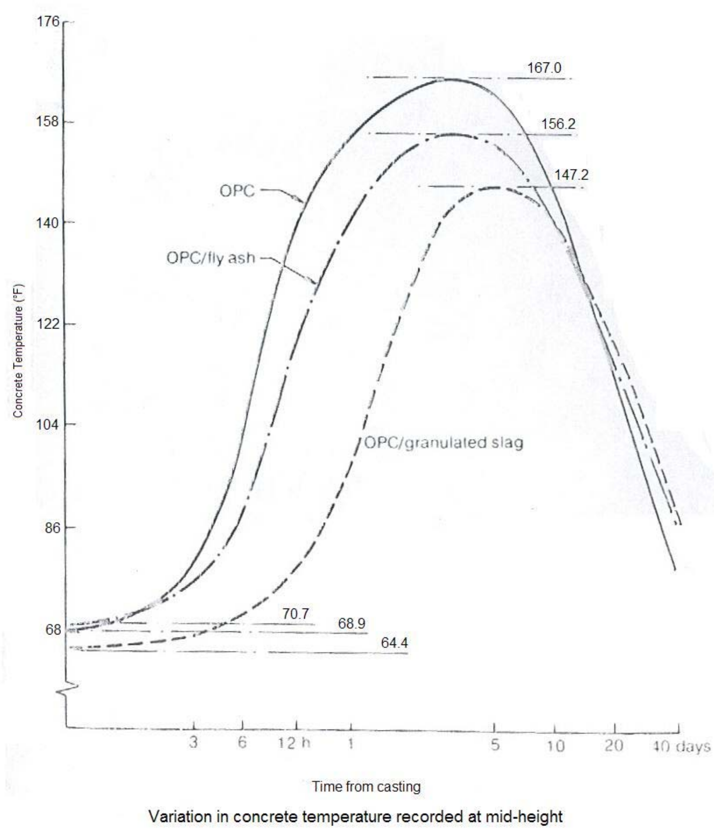


Figure 2-4 Variation In Maximum Temperature as Reported by Bamforth (1980)

In pour 1 (OPC only), 98.1°F rise in temperature was recorded. Temperature rise of 85.5°F and 82.8°F was recorded for Pour 2 (OPC/Fly Ash) and Pour 3 (OPC/GGBFS) respectively. Maximum temperature was reduced by 12.8% for Pour 2 and 15.5% for Pour 3. This shows that replacement of cement with 30% fly ash has almost the same effect as replacing cement with 75% slag. Bamforth (1980) believes this reduction in maximum temperature was smaller than what was observed in smaller concrete pours with lower cement content. He also reports from a survey of data relating to the use of slag in concrete mixes that as the size of pour is increased, the effectiveness of using slag to reduce the maximum concrete temperature reduces. However, he reports that in pours of up to 6.5 ft deep, reduction of 50% in maximum temperature has been achieved by a 70% replacement. Bamforth (1980) believes that in smaller pours the initial mix temperature is of greater significance. He reports the same trend in mixes with fly ash replacement. Bamforth (1980) recommends the minimum replacement percentage for pozzolanic materials, as shown in Table 2-5, to achieve benefits in reduced thermal stress. He believes that, in pours deeper than 8.2 ft, the benefit of reduced temperature resulting from replacement of cement with pozzolanic material is unlikely to be sufficient to offset the increased stiffness of concrete.

Table 2-5 Minimum Level of Replacement Percentage

Pour Thickness (ft)	Fly Ash	GGBFS (Slag)
Up to 3.3	20	40
3.3-4.9	25	50
4.9-6.6	30	60
6.6-8.2	35	70

A study (Atiş, 2002) on high-volume fly ash concrete showed that using 50% fly ash causes a reduction of 23% in the peak temperature. The same study showed 70% fly ash replacement leads to 45% reduction in the peak temperature (Figure 2-5). Atiş (2002) believes that 20-30% of fly ash replacement may not cause significant reduction in the maximum temperature of concrete. According to him, with 20-30% of fly ash replacement a reduction of 10-15% in the maximum temperature is expected. He also reports that changes in W/C ratio influence the temperature rise in concrete. A study on concretes with 675 lb/yd^3 OPC and W/C ratios of 0.35, 0.45, and 0.55 showed 104.4, 108.3 and 111.9 ºF of peak temperature in nonadiabatic conditions with no insulation (Atiş, 2002).

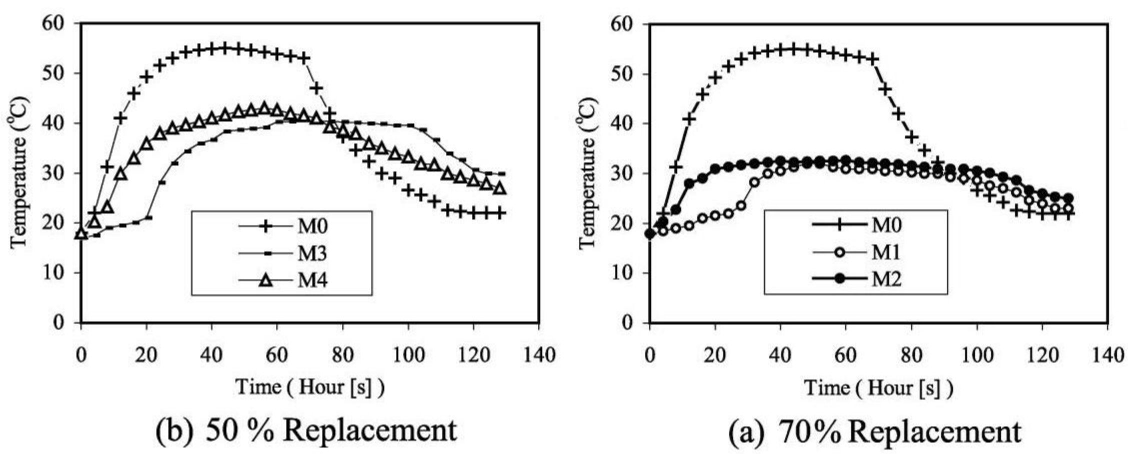


Figure 2-5 Adiabatic Temperature Rise Curves Developed by Atiş (2002)

(M0: Control Mix 675lb/yd3 Cement, M1: 70% Fly Ash with Superplasticizer, M2: 70% Fly Ash without Superplasticizer, M3:50% Fly Ash with Superplasticizer, M4: 50% Fly Ash without Superplasticizer)

ACI 207.2R-96 recommends that the total quantity of heat generation is directly proportional to an equivalent cement content (C_{eq}), which is the total quantity of cement plus a percent to total pozzolan content. The contribution of pozzolans to heat generation varies with age of concrete, type of pozzolan, the fineness of pozzolan compared to the cement and pozzolans themselves. ACI 207.2R-96 suggests to test the cement and pozzolan mixes to determine the fineness and heat of hydration of the blend. A rule of thumb that has worked fairly well on preliminary computations has been to assume that pozzolan produces only about 50 percent as much heat as the cement that it replaces (ACI 207.1 R-96).

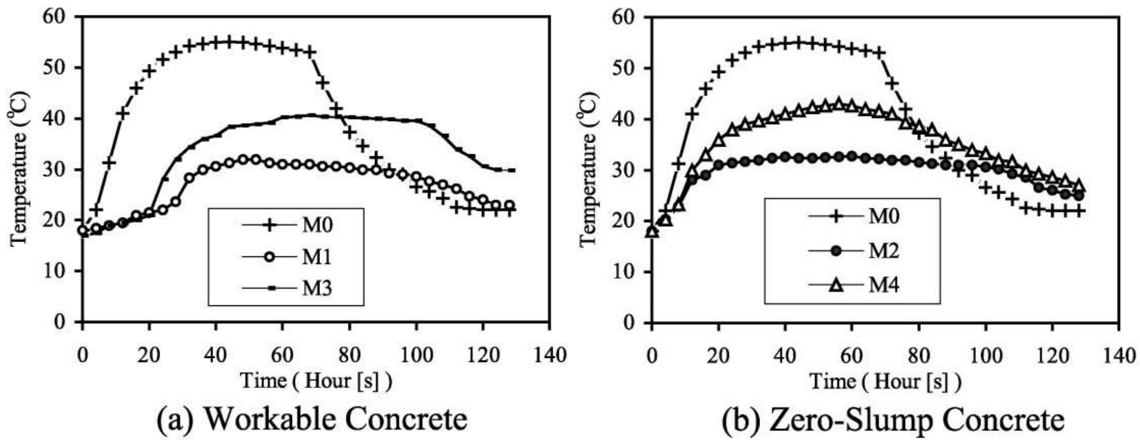


Figure 2-6 Effect of Admixtures on Heat Generation

(Atiş, 2002)(M0: Control Mix 675lb/yd^3 Cement, M1: 70% Fly Ash with Superplasticizer, M2: 70% Fly Ash without Superplasticizer, M3:50% Fly Ash with Superplasticizer, M4: 50% Fly Ash without Superplasticizer)

Using chemical admixtures as water-reducing, set-retarding agents affects the concrete mix in the first 12 to 16 hours after mixing. These chemicals do not alter the total heat generated in the concrete after the first 24 hours (ACI 207.2R-96). However, for studies involving a large amount of mass concrete the proposed mix should be tested for adiabatic temperature rise (ACI 207.1 R-96). Results from study performed by Atiş (2002) on using high-volume fly ash concretes with low water cement ratios and superplasticizers showed that concrete mixes with similar ingredients and different amounts of superplasticizers have the same peak temperature. Mixes with superplasticizers showed a delay in reaching the maximum temperature (Figure 2-6). This fact is caused by the retarding effect of superplasticizers on cement hydration (Atiş, 2002).

The size of the element and ambient temperature are factors that affect the concrete peak temperature in nonadiabatic conditions. However Bamforth (1980) showed that mixes with high percentage of pozzolan in large concrete members may be affected by cement replacement. As mentioned before, he believes pozzolan replacement in large members increases concrete stiffness.

Methods to Predict Temperature Rise in Mass Concrete

One of the main problems of mass concrete construction is the necessity for controlling the heat entrapped within it as the cement hydrates. Both the rate and the total adiabatic temperature rise differ among the various types of cement (ACI 207.1 R-96).

Predicting the maximum temperature of mass concrete has always been the main concern of designers and builders of mass concrete structures. As mentioned before, planning phase for the construction of Hoover Dam was a turning point in mass concrete studies. One of the earliest efforts to predict the maximum temperature of the mass concrete is reported by Blanks (1933). He describes a series of tests on adiabatic temperature rise of concretes with different cement types. Those test had been supported by the U. S. Bureau of Reclamation. To develop the adiabatic temperature rise curves, cylinders of full mass concrete with 188lb/yd^3 cement content were cast in place in accurately controlled adiabatic calorimeter rooms and immediately sealed by soldering a cover on the light sheet metal mold. The temperature of the air in the room and the temperature of the specimen were measured by resistance thermometers. These thermometers were connected to an input/output controller system, which maintained the air temperature in the calorimeter room within 0.10°F of the temperature of the specimen (Blanks, 1933).

Figure 2-7 shows the results of tests reported by Blanks (1933). At that time the final objective of the tests was to find the most proper commercial cement for the construction of Hoover Dam. ASTM standard cements had not been defined therefore different cements were designated by numbers (See Table 2-6).

Table 2-6 Compound Composition of Cements Represented in Figure 2-7

Cement No.	C_3S	C_2S	C_3A	C_4AF	MgO	$CaSO_4$	Free Lime
I-1	49.1	21.9	13.6	10.3	0.7	2.8	0.9
R-2252	57.8	17.4	6.7	10	2.6	3.6	1.2
Y-9	52.9	26.5	8.8	5.9	1.6	2.3	-
U-2	23.2	50.1	12.9	6.2	3.5	2.9	0.3
S-310	Not a True Portland Cement						
R-2249	25.6	46.2	2.8	18.1	2.4	4.1	1.0

Figure 2-1 shows adiabatic temperature rise curves for mass concrete containing 376lb/yd^3 of various types of cement. These curves are published in ACI 207.1R and are widely used to predict the adiabatic temperature rise in mass concrete. ACI curves have been traced to early 1960s.

As mentioned before, when a portion of the cement is replaced by pozzolan, the temperature rise curves are greatly modified, particularly in the early ages. Depending on the composition and fineness of the pozzolan and cement used in combination, the effect of pozzolans differs greatly.

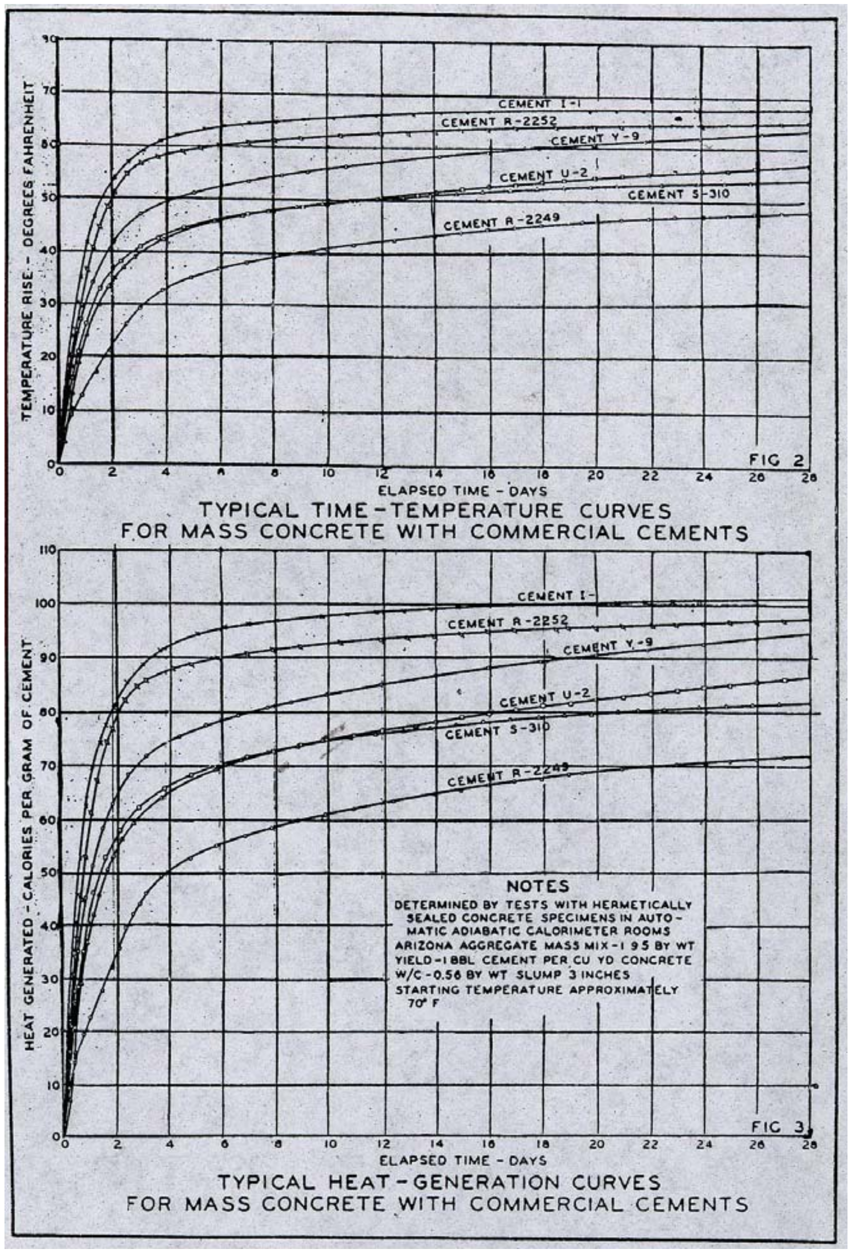


Figure 2-7 Typical Heat-Generation Curves (Blanks, 1933)

Specifications for mass concrete often require particular cement types, minimum cement contents, and maximum supplementary cementitious material contents. Once this information is available, the process of predicting maximum concrete temperatures and temperature differences can begin. Several options are available to predict maximum concrete temperatures.

Gajda (2002) reports a simplistic method, which is briefly described in a Portland Cement Association document. This method is useful if the concrete contains between 500 and 1000 lbs of cement per cubic yard of concrete and the minimum dimension is greater than 6 ft. For this approximation, every 100 lbs of cement increases the temperature of the concrete by 12.8 F. Using this method, the maximum concrete temperature of a concrete element that contains 900 lb of cement per cubic yard and is cast at 60°F is approximately 175°F. This PCA method does not, however, consider surface temperatures or supplementary cementitious materials (Gajda et al., 2002).

A more precise method is known as Schmidt's method. This method is most frequently used in connection with temperature studies for mass concrete structures in which the temperature distribution is to be estimated. Determining the approximate date for grouting a relatively thin arch dam after a winter's exposure, the depth of freezing, and temperature distributions after placement are typical applications of this step-by-step method. Different exposure temperatures on the two faces of a theoretical slab and heat of hydration of cement can be taken into consideration (Townsend, 1981).

In its simpler form, Schmidt's Method assumes no heat flow normal to the slab and is adapted to a slab of any thickness with any initial temperature distribution. Schmidt's Method states that the temperature, t_2 , of an elemental volume at any subsequent time is dependent not only upon its own temperature but also upon the temperatures, t_1 and t_3 , of the adjacent elemental volumes. At time Δt , this can be expressed as:

$$t_{2,\Delta t} = [t_1 + (M-2)\, t_2 + t_3] / M$$

Where $M = [C\rho(\Delta x)^2]/[K\Delta t] = (\Delta x)^2/(h^2\Delta t)$, since the diffusivity of concrete, h^2 (ft^2/hr), is given as $K/C\rho$.

K= Concrete Conductivity. Btu/ft.hr.F

C= Specific Heat, Btu/lb.F

ρ= Density of Concrete, lb/ft^3

If $\Delta t = (\Delta x)^2/(2h^2)$, then M=2. Therefore the temperature, t_2 at time Δt, becomes $t_{2,\Delta t} = (t_1+t_3)/2$. It means that the subsequent temperature of an elemental volume is simply the average of the two adjacent elemental temperatures.

The principal objection to the Schmidt Method of temperature is the time required to complete the step-by-step computation. This has been overcome by the use of computer programs (Townsend, 1981).

In recent years, there have been some efforts to develop models to simulate the hydration process.

Construction Technology Laboratories (CTL) staff have developed a software based on Schmidt's method and have validated it by field calibrations since early 1990s (Gajda et al., 2002). Gadja (2002) describes this software as being capable of predicting maximum concrete temperature and temperature differences for any concrete mix proportion under various placing conditions. He also indicates that CTL's software has the ability to thermally analyze a concrete element 1-, 2- and 3-dimensionaly.

Bentz and associates (1998) used a 3-D microstructural model to predict the adiabatic temperature rise. They tested a series of conventional and high performance concrete with and without silica fume. Before mixing, the materials were placed in a room having a regulated temperature equal to that of the adiabatic calorimeter to ensure thermal equilibrium at the beginning of the test.

Cement was imaged using scanning electron microscope/X-ray analysis to obtain a two-dimensional image. Each phase of the cement was uniquely identified in the image. This image and measured particle size distribution for the cement were used to reconstruct a three-dimensional representation of the cement (Bentz et al., 1998).

The cellular automatom-based 3-D cement hydration and microstructural model operates as a sequence of cycles, each consisting of dissolution, diffusion, and reaction steps.

Bentz and associates (1998) concluded that the 3-D microstructural model had successfully predicted the adiabatic temperature rise and there have been a reasonable relation between the developed model and experimental work. However, the accuracy of the model's prediction is restricted to correct computation of kinetic constants, activation energies, and reaction product stoichiometries.

Swaddiwudhipong and associates proposed a numerical model to simulate the exothermic hydration process of cement and temperature rise in mass concrete pours (Swaddiwudhipong et al. 2002). In their model the hydration reaction of each major mineral compound found in Portland cement, C_3S, C_2S, C_3A and C_4AF, is considered. The hydration of each mineral compound is characterized by its thermal activity and the reference rate of heat of hydration. Reference rate of heat of hydration is the rate of heat of hydration per unit mass of mineral compound in cement under specified hydration conditions.

In this model the influence of various factors on the exothermic hydration process is taken into consideration. The applicability of the proposed model is verified by a series of adiabatic temperature rise tests. Swaddiwudhipong and associates (2002) believe that with the establishment of this approach, it is possible to simulate the exothermic hydration process of Portland cement and the temperature rise directly on the basis of intrinsic mechanism of hydration, chemical composition of cement, and mix proportion of concrete mixture.

They concluded, "Compared with other empirical methods, the proposed model serves as a more reasonable and effective tool to predict the evolution of heat of hydration, the degree of hydration and the temperature rise in concrete mixtures" (Swaddiwudhipong et al.,2002).

Ballim (2004) developed a finite difference heat model for predicting time-based temperature profiles in mass concrete elements. In this study, a model representing a two-dimensional solution to the Fourier heat flow equation was developed. This model runs on a commercially available spreadsheet package. The model uses the results of a heat rate determination using an adiabatic calorimeter together with Arrhenius maturity function to indicate the rate and extent of hydration at any time and position within the concrete element. Ballim (2004) reports that this model is able to predict the temperature within 3.6ºF throughout temperature monitoring period.

Experimental Methods to Measure the Heat of Hydration of Concrete

There are normally four methods to measure the heat of hydration of concrete. (Gibbon et al., 1997).

- *Heat of Solution Test*: This method determines the total heat produced by the binder content of the concrete over a 28-day period, but does not indicate the rate of heat production at any point in time.

- *Conduction Calorimetry:* In this method heat removed from a sample of hydrating cementitious paste is measured. Since the rate of hydration is dependent on temperature, this method does not allow the sample to attain temperatures that it would in a concrete structure and therefore does not simulate the true condition.

- *Adiabatic Calorimetry:* This method allows determination of both the total heat and the rate of heat generation. In this method, there is no heat transfer from or into the test sample.

- *Isothermal Method:* This method is similar to adiabatic calorimetry but uses a Dewar or thermos flask to prevent heat loss, instead of an adiabatic control system. The heat loss from the flask is difficult to determine and will affect the hydration process.

CHAPTER 3
METHODOLOGY

Introduction

In this chapter, the materials and the test methods to study the adiabatic temperature rise of mass concrete are presented. In the first section of the chapter procedures that were undertaken to choose sample concrete mixes’ materials and their proportion are explained. Test methods and equipment used to measure the temperature rise and other characteristics of concrete samples are presented in the second section. In the third section, test procedures to determine adiabatic temperature rise, concrete thermal diffusivity, compressive strength, and heat of hydration are described.

Mix Design Selection

The first step to prepare a concrete sample is to design the mix proportions and choose the materials. There are many different mass concrete mix designs that have been approved and used in various FDOT projects in the past. The goal was to choose a mix design which is a representative of the majority of the mixes used in FDOT mass concrete projects. To achieve this goal a comprehensive list of 87 FDOT approved mix designs used for mass concrete elements in the time interval between 1990 and 2000 was compiled. Based on the information gathered about these mix designs, concrete class, cement type, proportion of pozzolanic material, and coarse and fine aggregates were selected.

Concrete Class

The breakdown of concrete classes of the mixes used in mass concrete projects in Florida is shown in Figure 3-1. The majority of the mixes were FDOT Class IV (5500 psi) concrete. It was therefore decided to use a Class IV concrete mix.

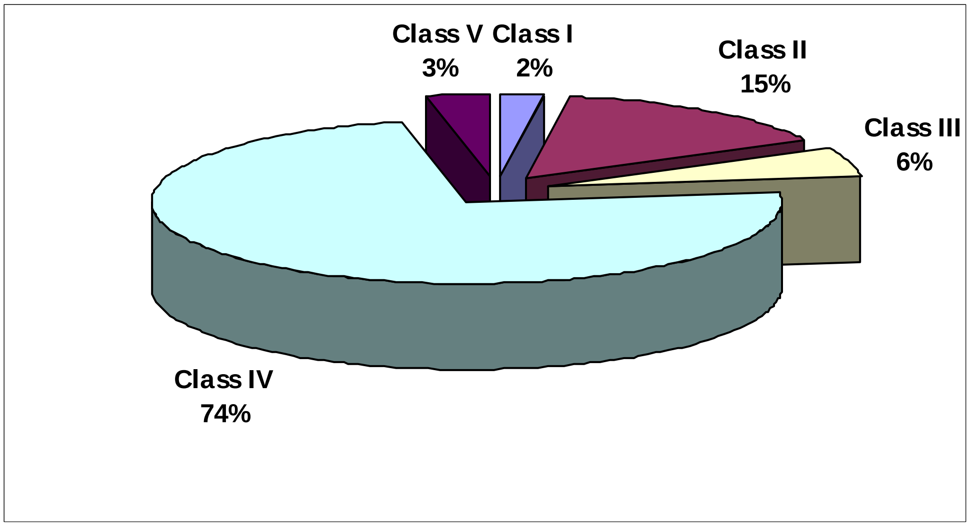


Figure 3-1 Mix Design Breakdown By Concrete Class

Cement Type

The next step was to choose the cement type. Table 3-1 shows the distribution of different types of cement used in 87 FDOT approved mass concrete mixes. Cements from two different sources that satisfy the AASHTO criteria for type II cement were used.

Table 3-1 Cement Type Distribution in FDOT Approved Mixes

Cement type	Number	Percentage	Comment
Type IP	10	11	
Type I	5	6	
Type II	72	83	← **Selected**
Total	**87**	**100**	

Pozzolanic Materials Proportion

Pozzolanic materials (Fly Ash or Slag) are generally used in mass concrete mixes. The following approach was used to determine the percentage of pozzolanic materials to be used in this project's mix designs.

Figure 3-2 shows the percentage of mixes made with different ratios of fly ash in FDOT approved mass concrete mixes. As one can readily observe, the ratio of fly ash to total cementitious material varies from 18% to 40%. It was decided to make two mixes with two different percentages of fly ash to have good representatives of the mixes. Mixes were divided into two groups. First group included mixes with 18% to 22% of fly ash. Based on weighted average and frequency of fly ash percentage, 20% was chosen for this group. The second group consisted of mixes with 30% to 40% of fly ash. The proportion of fly ash in this group was determined to be 35%.

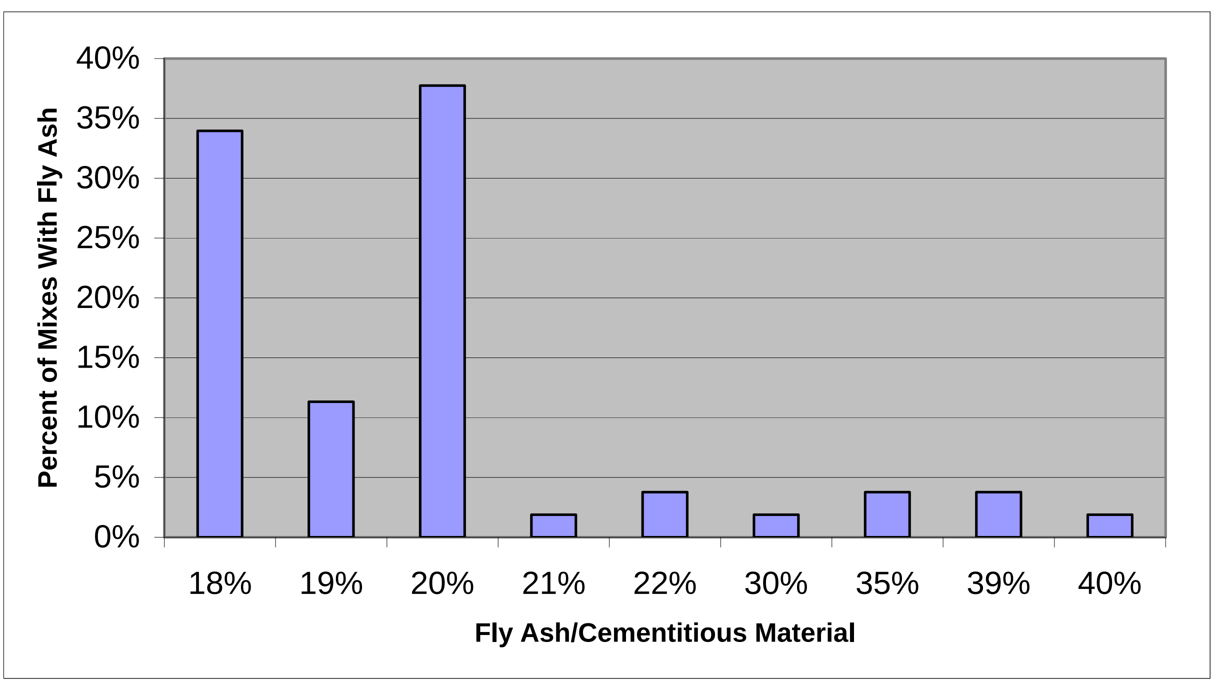


Figure 3-2 Fly Ash Percentage In FDOT Approved Mixes With Fly Ash

As shown in Figure 3-3, within the FDOT approved mass concrete mixes with slag, slag to cementitious materials percentage was 50%, 60% or 70%. It was decided to test two different mix designs with slag. For this purpose, 50% and 70% of slag in the mix were chosen. These proportions are the most frequent ones.

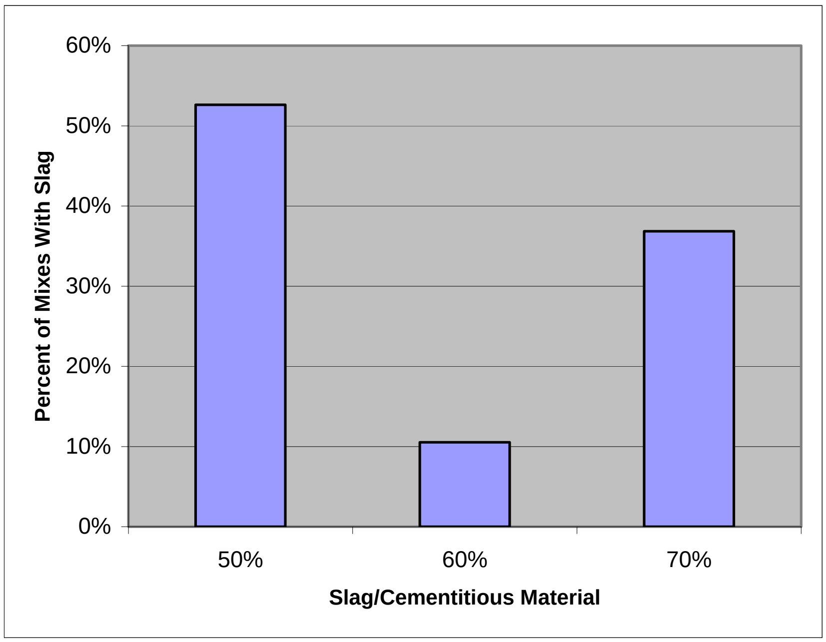


Figure 3-3 Slag Percentage In FDOT Approved Mixes With Slag

Cement Source

Data from FDOT approved mass concrete mixes showed that cements used are generally produced by the following cement manufacturers:

1. Rinker Materials (Miami)
2. Florida Rock Ind. (Newberry)
3. Florida Mining and Materials (Cemex in Brooksville)
4. Tarmac (Miami)

Samples from all four cements were tested for chemical analysis, physical analysis and, heat of hydration. In Table 3-2 results of chemical and physical analysis tests are presented. The final selection was based on the factors that are believed to affect the heat generation in cement. As mentioned in Chapter 2, Tricalcium Silicate (C_3S) and Tricalcium Aluminate (C_3A) have the largest contribution to the heat of hydration. Study of the chemical analysis test results showed that the cement from source 2 has the maximum C_3S content. Chemical analysis test also showed that the cement from source 3 has the maximum C_3A content while the other samples contain an equal percentage of C_3S. Based on the chemical analysis tests, cements from sources 2 and 3 were selected preliminarily.

Table 3-2 Results of Chemical and Physical Analysis for the Cement Samples

Cement Source		**Source Number**			
		1	**2**	**3**	**4**
		Rinker Materials (Miami)	Florida Rock Ind. (Newberry)	Florida Mining and Materials (Cemex in Brooksville)	Tarmac (Miami)
Chemical Analysis					
Loss of Ignition		1.5%	1.7%	0.1%	1.5%
Insoluble Residue		0.20%	0.21%	0.15%	0.16%
Sulfur Trioxide		2.7%	2.9%	2.9%	2.8%
Magnesium Oxide		0.9%	0.7%	0.9%	0.9%
Tricalcium Aluminate (C_3A)		6%	6%	**7%**	6%
Total Alkali as Na_2O		0.34%	0.36%	0.48%	0.44%
Silicon dioxide		21.3%	20.1%	21.5%	21.2%
Aluminum Oxide		4.6%	5.0%	5.1%	5.1%
Ferric Oxide		3.8%	4.2%	3.8%	3.8%
Tricalcium Silicate (C_sS)		56%	**58%**	48%	50%
Physical Analysis					
Compressive strength (psi)	3	3330	2740	2860	3050
	7	4360	3490	4110	4130
Fineness (m^2/kg)		390	401	350	370
Setting time Gilmore (minutes)	Initial	148	123	137	133
	Final	208	183	235	216
Soundness - Autoclave		0.00	-0.02	0.00	0.00
Normal Consistency		-	-	-	-
Comments			**Selected A**	**Selected B**	

The results of heat of hydration test showed cement 2 has the minimum heat of hydration at 7days and 28 days (see Table 3-3). Cement 3 did not have the maximum heat of hydration but its heat of hydration at 7 an 28 days was only 2.3% and 0.7% less than the heat of hydration of the cement 4 with the maximum heat of hydration. Thus cements 2 (Florida Rock Ind.) and 3 (Florida Mining and Material) were selected as cement A and cement B respectively.

Table 3-3 Results of Heat of Hydration Tests

No	Cement Source	Heat of Hydration @ 7 days (cal/g)	Heat of Hydration @ 28 days (cal/g)
1	Rinker (Miami)	77.6	88.2
2	Florida Rock Ind. (Newberry)	**66.2**	**84.1**
3	Florida mining and materials (Cemex in Brooksville)	**78.2**	**94.4**
4	Tarmac (Miami)	80.0	95.1

Fly Ash Source

Table 3-4 shows the percentage of mixes with fly ash from different sources. Monex/Boral fly ash has the highest frequency of use in FDOT approved mass concrete mixes and was selected to be used in this project.

Table 3-4 Percentage of Mixes With Fly Ash From Different Sources

Source	**Percent**
Monex/Boral	30%
Ash Management (Carbo)	18%
Florida Fly Ash	16%
Ash Management	14%
Florida Mining and Materials	10%
Ash Management (St Johns Power)	6%
Monier	2%
JTM (Jacksonville)	2%
Conversion System	2%

Blast Furnace Slag Source

Table 3-5 shows the percentage of mixes with blast furnace slag from different sources. Although Blue Circle's Newcem slag has the highest frequency of use, Lafarge slag was used in this project because of the difficulty in obtaining Newcem.

Table 3-5 Percentage of Mixes With Slag From Different Sources

Source	**Percent**
Blue Circle (Newcem)	70%
Lafarge Florida inc	15%
Pencem Pennsuco	15%

Table 3-6 Percentages of Coarse Aggregates From Different Sources

	Coarse Aggregate Source	S.G.	Pit Number	Number of Mixes	Percentage
Type II Cement with Fly Ash	Rinker Materials	2.451	87-090	18	39%
	Vulcan/Ica	2.455	08-005	8	17%
	Florida Rock Ind.	2.493	08-004	4	9%
	Vulcan/Ica	2.738	AL-149	4	9%
	S & S Materials	2.620	AL-288	3	7%
	Cabbage Grove	2.480	38-268	2	4%
	Harper Brothers	2.500	12-260	2	4%
	Florida Rock Ind.	2.370	TM-478	1	2%
	Rinker Materials	2.420	87-035	1	2%
	Tarmac	2.430	87-145	1	2%
	Florida Rock Ind.	2.450	08-012	1	2%
	White Construction	2.630	38-036	1	2%
	Total			**46**	**100%**
Type II Cement with Slag	Rinker Materials	2.434	87-090	8	40%
	Tarmac	2.430	87-145	8	40%
	Florida Rock Ind.	2.375	87-049	2	10%
	Florida Rock Ind.	2.430	08-004	2	10%
	Total			**20**	**100%**

Coarse Aggregate Source

The other important component of a mix is coarse aggregate. Based on data collected from FDOT approved mass concrete mixes, the most frequently used coarse aggregate was produced by Rinker Materials (Pit Number 87). Table 3-6 shows source, specific gravity, and pit number of coarse aggregates used in FDOT approved mass concrete mixes with fly ash or slag. Coarse aggregate produced by Rinker Materials from Pit Number 87-090 was therefore chosen for the test.

Fine Aggregate Source

Fine aggregate selected for this project was from the same source as the coarse aggregate (Rinker Materials).

Total Cementitious Materials

The amount of cementitious material is one of the important factors in generating heat of hydration in the concrete mix. Considering Table 3-7 and the fact that higher total cementitious material generates more heat of hydration, 760 lbs/yd^3 total cementitious materials was used in this project.

Table 3-7 Total Cementitious Materials In FDOT Approved Class IV Concrete Mixes

Total Cementitious Materials (lbs)	Number	Percentage
650-709	27	47%
710-769	26	46%
770-869	4	7%

Mix Temperature

Concrete placing temperature also affects the heat of hydration. To demonstrate how placing temperature affects heat of hydration, two placing temperatures, 73°F and 95°F, were used in this project.

Number of Mixes

The objective of this study was to develop adiabatic temperature rise for typical mass concrete mixes made with Florida materials. To accomplish this objective 20 different mixes were made using two different cement sources, two placing temperatures, and different percentages of pozzolanic materials. Table 3-8 exhibits the type and amount of pozzolanic materials, cement source, and placing temperature for these 20 mixes.

Table 3-8 Number of Test Mixes

Binder	Placing Temp (°F)	Cement Source	Mix Designation
Type II Cement 0% Pozzolanic Material	73	A	73A00P
	73	B	73B00P
	95	A	95A00P
	95	B	95B00P
80% Type II Cement + 20% Fly Ash	73	A	73A20F
	73	B	73B20F
	95	A	95A20F
	95	B	95B20F
65% Type II Cement + 35% Fly Ash	73	A	73A35F
	73	B	73B35F
	95	A	95A35F
	95	B	95B35F
50% Type II Cement + 50% Slag	73	A	73A50S
	73	B	73B50S
	95	A	95A50S
	95	B	95B50S
30% Type II Cement + 70% Slag	73	A	73A70S
	73	B	73B70S
	95	A	95A70S
	95	B	95B70S
Total Number of Mixes			**20**

Test Methods and Equipments

This research project involved development of adiabatic temperature rise curves for FDOT approved concrete mixes, measuring concrete thermal diffusivity, and testing compressive strength at different ages. In addition, the heat of hydration for plain and blended cements used in different mixes was measured.

Adiabatic Temperature Rise

There is no standard method to measure the adiabatic temperature rise of mass concrete. Literature review showed that the basis of all previous tests to monitor and record the adiabatic temperature rise of mass concrete was to provide an adiabatic or semi adiabatic condition for the concrete sample and at the same time record the temperature rise by thermocouples placed in the core of the concrete samples. Different methods have been used to provide an adiabatic condition for concrete samples. These methods range from building an isolated room with controlled temperature to small chambers or cylinders that are connected to heaters and thermocouples and are monitored by computers.

Apparatus

For this study Sure Cure system was used. This system had been previously used by FDOT in similar research projects. The system consists of a Hydration Chamber, Cylinder Molds, I/O Controller Cabinet and Personal Computer.

Hydration Chamber

Hydration Chamber (Figure 3-4) is a caped cylinder that holds a fresh concrete sample approximately equal to a 6"x12" cylinder. It has an insulated wall and cap which prevents heat exchange between the concrete and the outside ambient. Two thermocouples are mounted in the Hydration Chamber. The first one is called “Inner” and is placed inside a cone in the middle of the cylinder (see Figure 3-6). This thermocouple records the temperature of the core of the concrete sample placed in the Hydration Chamber. The second thermocouple, called “Outer,” is placed in the wall of the cylinder and monitors the temperature of the surface of the sample. These thermocouples are connected to the I/O controller. Two heaters are placed in the wall and the cap of the Hydration Chamber. Each heater is connected to the I/O Controller separately. Whenever the temperature recorded by the Inner thermocouple is smaller than the one recorded by the Outer, heaters will start heating the sample to eliminate the temperature gap between the inside and the outside of the sample.

Figure 3-4 Hydration Chamber

Cylinder Molds

A cylinder mold holds a 4" by 8" concrete sample. A thermocouple is placed in each mold and relays the temperature to the controller. The controller sends the data to the computer and the control software compares the temperature of the mold to the temperature recorded from the Inner thermocouple of the Hydration Chamber. If the temperature of the cylinder mold is lower than the temperature of the Inner thermocouple, the software orders the controller to turn on the heater inside the cylinder mold. Each cylinder mold is insulated by flexible polyurethane insulation to protect the mold from the environment (Figure 3-5). The specimens from the cylinder molds were used for compressive strength test.

Figure 3-5 Cylinder Mold

I/O Controller Cabinet and Personal Computer

The I/O Controller can monitor and control up to 15 thermocouple channels and 8 heater channels. The computer provides complete time vs. temperature information. The PC can control cylinder molds to follow the specimen in the Hydration Chamber or any pre-programmed time/temperature curve. The thermocouple and 120V power circuits from the Hydration Chambers and the cylinder molds can be plugged directly into the I/O cabinet of the PC controller. As specified by the Sure Cure System Manufacturer, the system controls the temperature of the cylinder molds within ±2ºF of the "master" thermocouple. In these tests the Inner thermocouple of the Hydration Chamber was the "master" thermocouple.

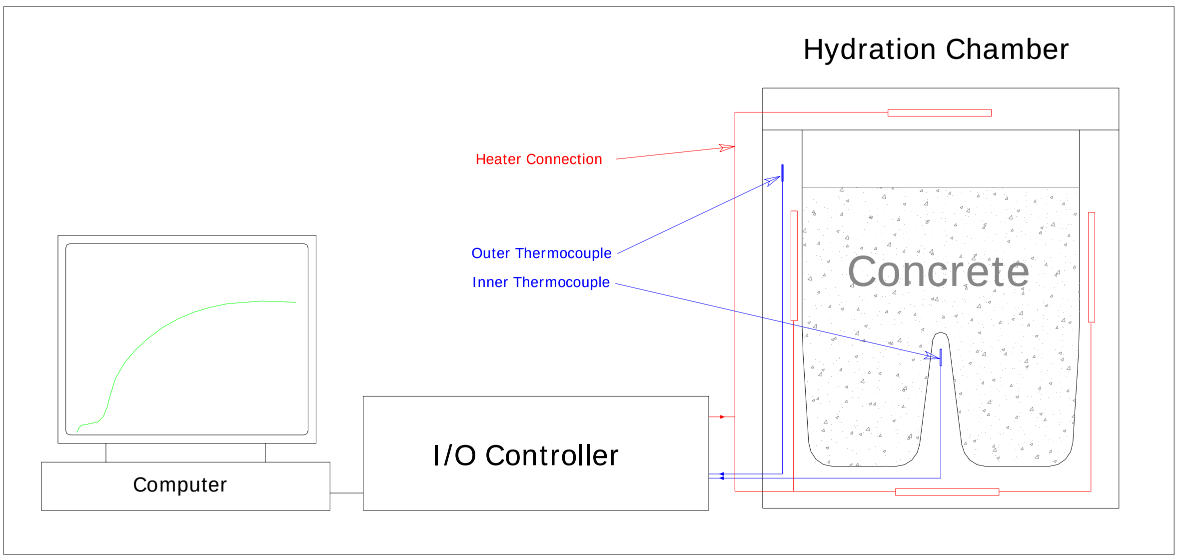


Figure 3-6 Sure Cure System

Concrete is placed in the Hydration Chamber. Immediately after the concrete placement, the temperature data from Inner and Outer thermocouples is read every 6 seconds by I/O Controller. The software that monitors the I/O Controller compares the core temperature and the outer surface temperature. As soon as the difference between the core temperature and the surface temperature is more than a user-defined amount, the software orders the I/O Controller to turn on the heaters connected to the chamber and heat the specimen surface to reduce the temperature difference between the core and the surface of the specimen (Figure 3-6). The software records temperature readings from thermocouples for a specific period of time every minute. Recorded data is saved on the PC and is later used to develop the adiabatic temperature rise curves.

For each mix two specimens in the Hydration Chamber and six in the Cylinder Molds were tested. The setup of the system is shown in Figure 3-7.

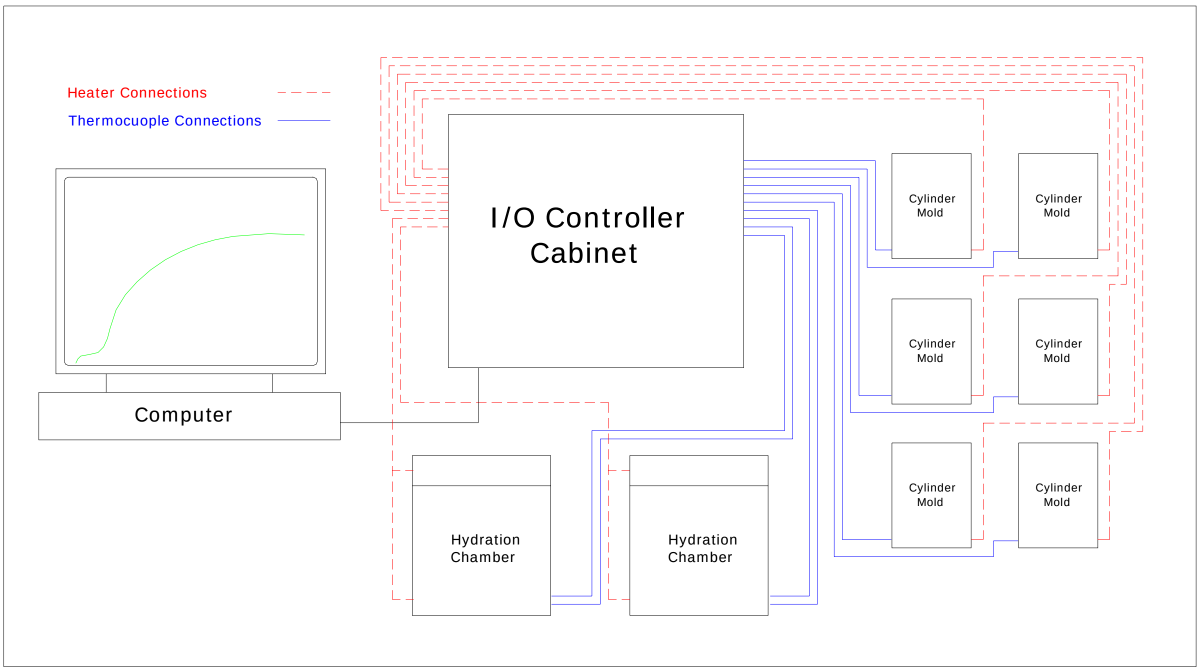


Figure 3-7 Sure Cure System Setup

Thermal diffusivity of concrete

To measure concrete thermal diffusivity the CRD-C 36-73 Method (Method of Test for the Thermal Diffusivity of Concrete) developed by Army Corps of Engineers was used.

Scope

This test is used to determine the thermal diffusivity of concrete. The thermal diffusivity is equal to the thermal conductivity divided by the heat capacity per unit volume and may be used as an index of the facility with which the material will undergo temperature change.

Apparatus

The apparatus consists of Bath, Diffusion Chamber and Temperature Indicating and recording instrument and Timer.

Bath.

A heating bath (Figure 3-8) in which concrete cylinders can be raised to uniform high temperature (212 °F).

Diffusion Chamber

A chamber containing running cold water (Figure 3-9).

Temperature Indicating and recording instrument and Timer

This device records time and temperature (Figure 3-10).

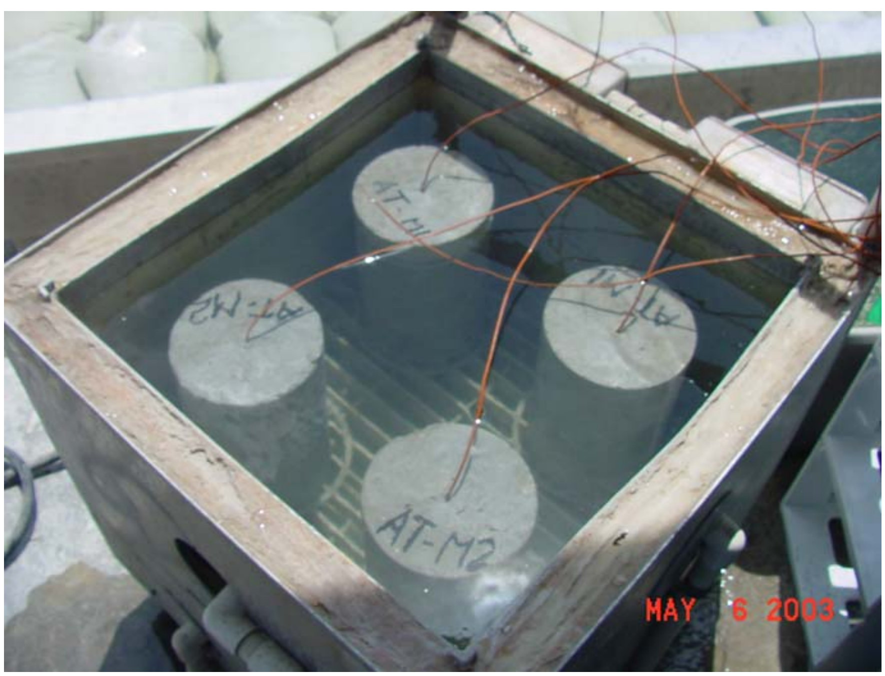


Figure 3-8 Heating Bath

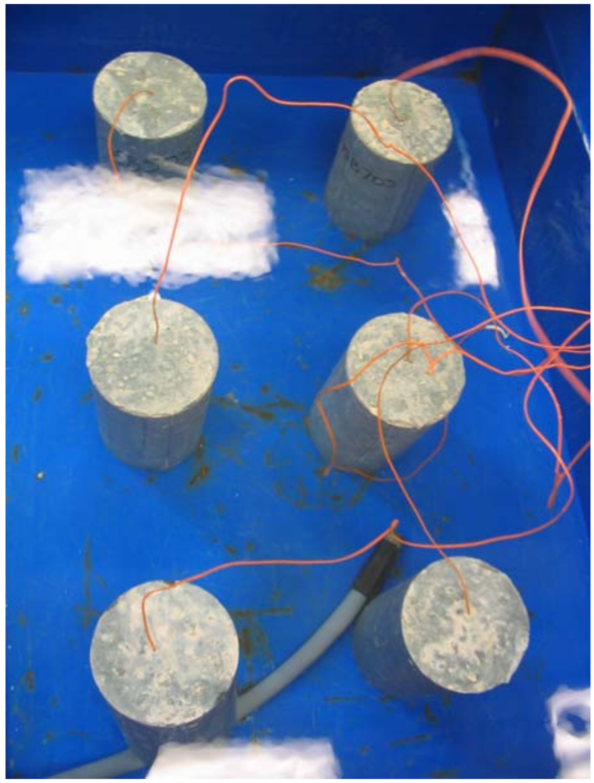


Figure 3-9 Diffusion Chamber

Figure 3-10 Temperature Recorder and Timer

Procedure

Preparation of Specimen

The test specimens were 6 by 12 in. cylinder. Molded specimens were moist-cured for 28 days prior to testing.

Heating

Each specimen was heated to the same temperature by continuous immersion in boiling water until the temperature of the center reached 212° F (100°C). The specimen was then transferred to a bath of running cold water, and suspended in the bath so that the entire surface of the specimen is in contact with the water. The temperature of the cold water was determined by means of another thermocouple.

Cooling

The cooling history of the specimen was obtained from readings of the temperature of the interior of the specimen at 1-min intervals from the time the temperature difference between the center and the water reached 120 °F (67 °C) until the temperature difference between the center and water reached 8 °F (4 °C). The data was recorded. Two such cooling histories were obtained for each test specimen, and the diffusivities were calculated within ± 0.002 ft^2/h.

Calculations

The temperature difference in ºF was plotted against the time in minutes on a semi logarithmic scale. The best possible straight line was then drawn through the points so obtained. A typical graph is shown in Figure 3-11. The time elapsed between the temperature difference of 80 °F and 20 °F was read from the graph, and this value was inserted in equation below from which the thermal diffusivity was calculated:

$\alpha = 0.812278/(t1 - t2)$ Equation 3.1

Where:

α = thermal diffusivity, ft2/ h r

And,

(t1 - t2) = elapsed time between temperature differences of 80 °F and 20 °F in minutes, and

0.812278 = numerical factor applicable to 6- by 12-in cylinder.

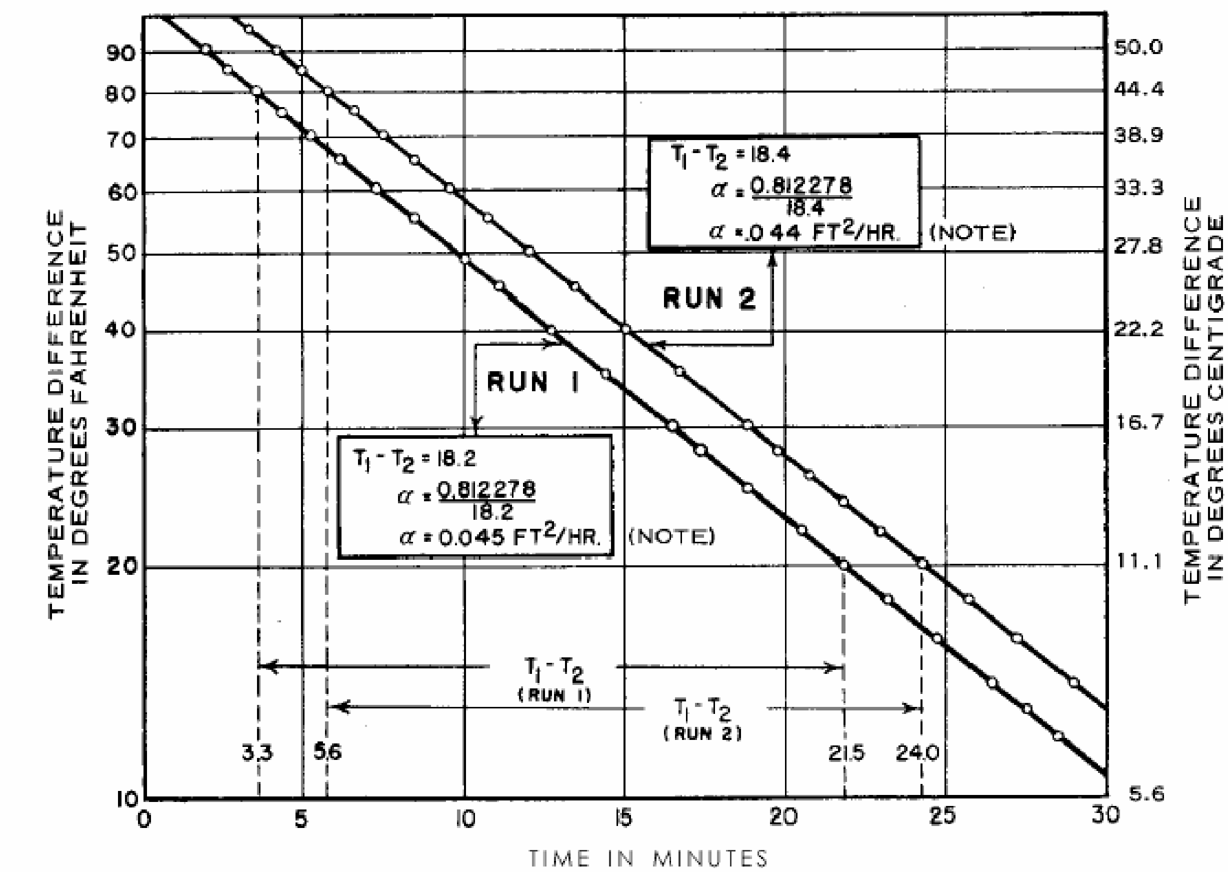


Figure 3-11 Example Showing Calculation Of Thermal Diffusivity Of A Concrete Cylinder

Compressive Strength

The compressive strengths of the samples were determined at ages of 14 and 28 days. The compressive strengths were determined according to ASTM C39/C 39M-01, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*, using FDOT physical laboratory equipment.

Heat of Hydration

The heat of hydration for cementitious materials was determined at 7 and 28 days. The heat of hydration was determined according to ASTM C186, *Standard Test Method for the Determination of Heat of Hydration of Hydraulic Cement*, by Construction Technology Laboratories (CTL) in Skokie, Illinois.

Test Procedures

In this section test procedures to determine adiabatic temperature rise, thermal diffusivity, and compressive strength for concrete and the heat of hydration for plain and blended cement paste are described. Concrete tests were performed at the FDOT physical laboratory, while Construction Technology Laboratories performed the heat of hydration test for cement.

Tests Performed For Each Mix

The following tests were performed for each mix:

1-Fresh Concrete Properties

2-Adiabatic Temperature Rise

3-Compressive Strength at different ages

4- Concrete Thermal Diffusivity

5- Cementitious Materials Heat of Hydration

Procedures for each test will be described in the following sections.

Size and Number of Specimens

For each mix different sizes of specimen were prepared.

Hydration Chamber: 2 Hydration Chambers were used to monitor the adiabatic temperature rise for each mix.

Cylinder Molds: These molds are 4"x8" cylinder shaped and are connected to the Sure Cure System I/O Controller. They are heated by electric heaters to follow temperature rise of concrete in Hydration Chambers. Six cylinder molds were used for each mix. Three of the specimens were tested for compressive strength at 14 days along with three specimens cured at room temperature. The other three specimens were kept in the moisture room for another 14 days. Three of the specimens cured at room temperature were also kept in the moisture room for another 14 days. These specimens were tested for compressive strength at 28 days.

Plastic Molds: *6"x12" Cylinder*: Six samples were prepared. Three were used for compressive strength at 28 days and the other three were used for concrete diffusivity test.

4"x8" Cylinder: Six samples were prepared. All of them were kept in the molds in room temperature and were covered to prevent moisture loss. At the age of 14 days molds were stripped. Three specimens were broken for compressive strength test. The other three specimens were transferred to moisture room and kept for 14 days. As explained earlier, these specimens were tested for compressive strength at 28 days along with high temperature cured specimens.

<u>Adiabatic Temperature Rise</u>

As explained earlier in this chapter, Sure Cure System was used to measure the adiabatic temperature rise.

Laboratory Tests

Preparation of Mix Materials

Concrete materials were prepared a day before the mix date with proportions determined in mix design calculations (Figure 3-12). For 73°F placing temperature tests, materials were kept in the laboratory area with a controlled temperature of 72-75°F. For the mixes with 95°F placing temperature, fine aggregates were placed in an oven with 200°F temperature for 24 hours.

Figure 3-12 Materials Prepared For The Next Day Test

Concrete Mixing and Placing Stage

For each test a 3 Cubic Feet mix was prepared. As suggested by Hydration Chamber manufacturer, there should not be a direct contact between concrete and the chamber's wall. Therefore concrete was placed in a plastic bag which was inside the chamber (Figure 3-13)

Figure 3-13 Hydration Chamber

Immediately after concrete placing, Hydration Chambers were connected to the controller and computer system and temperature recording started. At the same time concrete was also placed in metal cylinder molds (Figure 3-5) and they were also connected to the controller and computer. Hydration Chambers and cylinder molds were placed on a table for the period of temperature rise monitoring (Figure 3-14).

Figure 3-14 Cure Chambers and Metal Molds Connected to the Computer

Temperature Rise Monitoring Stage

Temperature monitoring started immediately after placing concrete in Hydration Chambers. For first five mixes (73A00P, 73A20F, 73A35F, 73A50S and 73A70S) temperature rise were monitored for 28 days. It was realized that there is not a significant temperature gain after the second week, so for the following tests temperature rise was monitored for 14 days.

At the end of the temperature rise monitoring stage, recorded temperature rise log was saved for later analysis. Samples from Hydration Chambers were transferred to moisture room and kept there for future use in microcrack test. Samples from metal molds were used for compressive strength test.

Field Tests

In addition to the laboratory tests, it was necessary to take samples from mass concrete projects and monitor their temperature rise with hydration Chambers to evaluate the correlation between the laboratory results and the field temperature readings.

The first step was to find FDOT mass concrete projects throughout the State of Florida. Different regional offices of FDOT were contacted. Finally Interstate 4 project was chosen. Test equipments were mounted in a minivan and were ready to be dispatched to the project site at the notice from the project administrator (Figure 3-15).

Figure 3-15 Hydration Chambers, Controller and Computer in Minivan

On April 2nd 2004, the first group of samples were taken from Bella Vista Bridge (a bridge over I-4 west of Memorial Blvd.) footing A in Lakeland. However because of equipment malfunction temperature monitoring could not be started immediately after sample placement so the results from this test was not considered reliable. On April 9th, footing C of the same project was poured, so samples were taken again at this date.

Sample Preparation

It was decided to take samples from the center of the concrete pour. This decision was based on the fact that the middle core of a mass concrete element will most likely reach the maximum curing temperature, in addition, temperature sensors to monitor temperature rise of concrete are installed at the center of the pour.

Concrete was poured directly from the truck mixer to a wheel borrow. Immediately concrete from the wheel borrow was placed in Hydration Chambers. Hydration Chambers were transferred to the minivan and connected to the controller and the computer. Power was supplied from minivan through a converter (Figure 3-16)

Figure 3-16 Cure Chambers in the Minivan

The samples were not moved during the initial set time of the concrete. After four hours (initial set time) samples were moved to FDOT laboratory located about 140 miles from the project site. Upon arrival at the FDOT facility, Hydration Chambers, controller, and the computer were transferred to a stable location and temperature rise was monitored for 14 days.

Thermal Diffusivity

Test Procedure

The method to measure concrete thermal diffusivity was described in the previous section. Samples were tested for thermal diffusivity at 90 days age. The method to measure thermal diffusivity was not identified before the first group of samples was about 90 days age. To keep the consistency, other samples were tested at this age as well. Also literature review did not show any indication of the effect of age of concrete on its thermal diffusivity.

Sample Preparation

For each mix three 6"x12" cylinders were cast. Within each cylinder a wire thermocouple was placed. Specimens were kept in moisture room before they were tested for thermal diffusivity.

Heating

Specimens were placed in a heating bath at the age of 90 days. Specimens remained in the heating bath until their core temperature reached 212°F.

Figure 3-17 Specimens Transferred From Heating Bath to Diffusion Chamber

Cooling

Specimens were then transferred to a diffusion chamber. Diffusion chamber contained a relatively large body of water and was connected to running water. These factors kept water temperature constant during concrete cooling period.

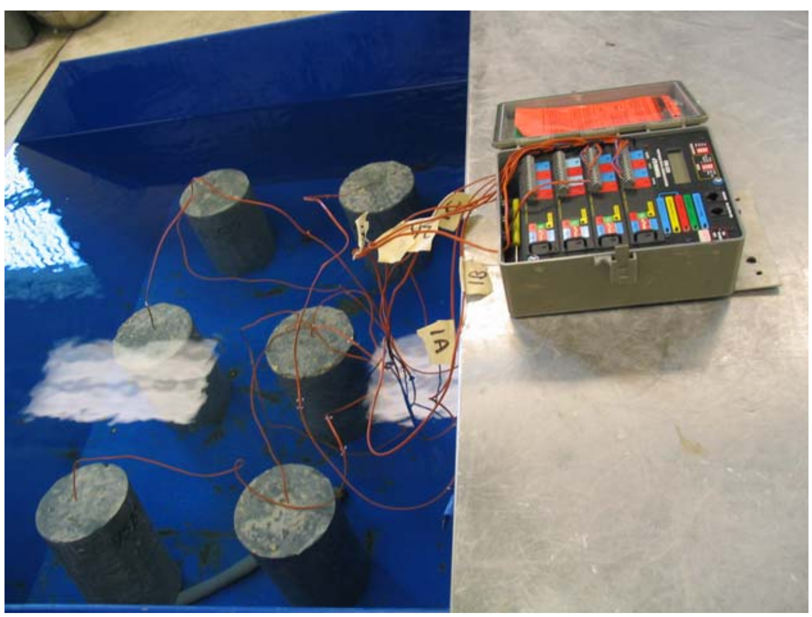


Figure 3-18 Specimens Connected To Temperature Recorder During Cooling Stage

Core temperature of the specimen was recorded in one-minute intervals from the time specimen was placed in the diffusion chamber until the temperature difference between the specimen core and water was less than 8°F. Temperature drop history was later used to calculate the thermal diffusivity.

Compressive Strength

Compressive strength test was performed for the specimens with different ages and sizes as mentioned in previous sections according to ASTM C39.

Test Procedure

Sample Preparation

In total, 18 samples were cast for compressive strength tests. All specimens were placed in molds after placing concrete in Cure Chambers.

Testing of Samples

Samples were tested at 2 different ages: 14 and 28 days.

14-Day Tests: Two groups of specimens were tested at this age. Each group consisted of three 4″x8″ cylinders. The first group was cured in high temperature. The second group was kept in molds in room temperature. Test results for two groups were compared to study the effect of high curing temperature on compressive strength.

28-Day Tests: Two series of tests were performed at this age. The first series of tests included two groups of specimens. Each group consisted of three 4"x8" cylinders. The first group was cured in high temperature for 14 days and was then cured in moisture room for another 14 days. The second group was kept in molds in room temperature and later was moved to the moisture room and was kept there for additional 14 days. Test results for two groups were compared to study the effect of high curing temperature on compressive strength. For the second series, three 6"x12" samples were cured in moisture room and were tested at 28 days age.

Heat of Hydration

Two series of Heat of Hydration tests were performed by CTL in August 2003 and April 2004. ASTM C186 method was used to determine Heat of Hydration at 7 and 28 days.

Test Series I

In the first series four samples of plain cement (from four different sources) and samples of fly ash and blast furnace slag were sent to CTL laboratory in Illinois to perform the test. The results of Heat of Hydration tests were used to choose the two cement sources to use for mix preparation. Also to study the correlation of Heat of Hydration and maximum adiabatic temperature rise, blends of 80% cement A with 20% fly ash and 50% cement A with 50% slag were tested for Heat of Hydration.

Test Series II

The first supply of cement A was finished during the test period due to the repetition of some of the test mixes. It was necessary to measure Heat of Hydration for a new supply of cement A. Therefore, sample of this cement was sent to CTL to perform Heat of Hydration test. Along with the cement sample, fly ash and slag samples were also sent to CTL. In this series, Heat of Hydration of cement A, 65% cement A with 35% fly ash and 30% cement A with 70% slag was determined.

CHAPTER 4
TEST RESULTS

Introduction

This chapter provides the results of the various tests performed to measure the properties of mass concrete mixes. Mixture properties can be found in the first section. The second section provides fresh concrete properties measured for each mix. The heat of hydration data, as reported by CTL is shown in the third section. In the forth section the temperature of concrete samples recorded by Sure Cure system as well as adiabatic temperature rise curves for different mixes are shown. In the fifth section results for the thermal diffusivity tests are presented. In this section a sample calculation to determine the thermal diffusivity of concrete is also shown. In the last section of the chapter, compressive strength tests results are presented.

Mixture Properties

In Table 4-1 mixture properties of different test mixes are shown. The proportions of cement, pozzolanic materials (fly ash or blast furnace slag), water, fine aggregate, coarse aggregate, air entertainer and admixture for each mix can be found in this table. Darex® air entertainer and WARDA® 56 water reducer were used in all of the mixes.

Table 4-1 Concrete Mixture Properties

FDOT Class IV (5500 psi) (Mix proportions per cubic yard)									
No.	Mix Designation	Cement (lb)	Fly Ash (lb)	Slag (lb)	Water (lb)	Fine Aggregate (lb)	Coarse Aggregate (lb)	Air Entertainer (oz)	Admixture (oz)
1	73A00P	760.00	0.00	0.00	279.00	1033.00	1736.00	2.30	68.20
2	73A20F	608.00	152.00	0.00	279.00	1033.00	1708.00	4.00	68.20
3	73A35F	494.00	266.00	0.00	279.00	980.00	1687.00	4.00	68.20
4	73A50S	380.00	0.00	380.00	279.00	1021.00	1724.00	2.30	68.20
5	73A70S	228.00	0.00	532.00	279.00	1021.00	1724.00	2.30	68.20
6	73B00P	760.00	0.00	0.00	279.00	1033.00	1736.00	2.30	45.65
7	73B20F	608.00	152.00	0.00	279.00	1033.00	1708.00	4.00	45.65
8	73B35F	494.00	266.00	0.00	279.00	980.00	1687.00	4.00	45.65
9	73B50S	380.00	0.00	380.00	279.00	1021.00	1724.00	2.30	68.20
10	73B70S	228.00	0.00	532.00	279.00	1021.00	1724.00	2.30	30.40
11	95A00P	760.00	0.00	0.00	279.00	1033.00	1736.00	2.30	60.86
12	95A20F	608.00	152.00	0.00	279.00	1033.00	1708.00	4.00	68.20
13	95A35F	494.00	266.00	0.00	279.00	980.00	1687.00	4.00	60.86
14	95A50S	380.00	0.00	380.00	279.00	1021.00	1724.00	2.30	25.87
15	95A70S	228.00	0.00	532.00	279.00	1021.00	1724.00	2.30	68.20
16	95B00P	760.00	0.00	0.00	279.00	1033.00	1736.00	2.30	60.86
17	95B20F	608.00	152.00	0.00	279.00	1033.00	1708.00	4.00	30.40
18	95B35F	494.00	266.00	0.00	279.00	980.00	1687.00	4.00	68.20
19	95B50S	380.00	0.00	380.00	279.00	1021.00	1724.00	2.30	38.06
20	95B70S	228.00	0.00	532.00	279.00	1021.00	1724.00	2.30	68.20

Fresh Concrete Properties

Table 4-2 shows the results of Slump and Air Content tests. The slump tests were performed in accordance with ASTM C143. The objective was to maintain the slump in 3″±1 ½″. Air Content tests were run according to ASTM C231. Air temperature and mix temperature were measured at the time of placing concrete in hydration Chambers. Water to Cementitious Material Ratio was calculated using following equation:

$WC_r = W_w / W_c$ Equation 4-1

Where:

WC_r= Water to Cementitious Material Ratio

W_w= Weight of Water in lbs from Table 4-1

W_c= Weight of Total Cementitious Materials (Cement + Fly Ash or Slag) in lbs from Table 4-1

Weight of Total Cementitious Materials (Cement + Fly Ash or Slag) and water was constant, therefore WC_r equal to 0.37 in all mixes.

Unit weight was calculated by dividing the summation of all materials weights from Table 4-1 by mix volume. For example unit weight of mix 73A00P was calculated as follows:

Weight of Cement = 760 lbs

Weight of Water = 279 lbs

Weight of Fine Aggregate = 1033 lbs

Weight of Coarse Aggregate = 1736 lbs

Weight of Air Entertainer + Admixture = 2.3+68.20 = 70.50 oz = 4.41 lbs

Total of Weight of Mix Materials = 3812.41 lbs   Mix Volume = 1 CY = 27 CF

⇒ Mix Density = 3812.41 lb/CY = 141.2 lb/CF   □

Table 4-2 Fresh Concrete Properties

No	Mix	Slump (in.)	Air Content (%)	Mix Temp. (ºF)	Air Temp. (ºF)	Water to Cementitious Material Ratio	Unit Weight (lb/CF)
1	73A00P	2.75	4.25	73	72	0.37	141.2
2	73A20F	2.00	2.75	73	72	0.37	140.2
3	73A35F	3.25	2.25	72	72	0.37	137.4
4	73A50S	3.50	3.75	81	80	0.37	140.3
5	73A70S	1.75	3.25	74	72	0.37	140.2
6	73B00P	7.25	3.25	73	72	0.37	141.1
7	73B20F	6.50	3.50	73	75	0.37	140.1
8	73B35F	6.00	2.00	75	72	0.37	137.4
9	73B50S	3.50	3.25	73	73	0.37	140.3
10	73B70S	3.75	3.00	73	72	0.37	140.2
11	95A00P	3.25	3.25	93	72	0.37	141.2
12	95A20F	2.75	2.25	95	72	0.37	140.2
13	95A35F	4.00	1.75	94	68	0.37	137.4
14	95A50S	3.50	2.25	96	70	0.37	140.3
15	95A70S	1.75	2.75	93	72	0.37	140.2
16	95B00P	4.25	5.50	99	73	0.37	141.1
17	95B20F	1.75	2.25	98	70	0.37	140.1
18	95B35F	0.50	2.00	101	70	0.37	137.4
19	95B50S	3.00	3.50	101	70	0.37	140.3
20	95B70S	2.75	2.50	101	72	0.37	140.2
Repeated Tests							
21	73A00P-R	3.75	3.50	73	72	0.37	141.2
22	73A20F-R	3.75	2.00	74	73	0.37	140.2
23	73A35F-R	3.50	2.25	74	72	0.37	137.4
24	73A50S-R	3.75	2.75	68	68	0.37	140.3
25	73A70S-R	3.00	2.50	69	70	0.37	140.2
26	73B50S-R	3.00	3.50	73	73	0.37	140.3
27	73B70S-R	2.50	3.50	74	72	0.37	140.2
28	95A70S-R	3.25	2.50	97	72	0.37	140.2

Heat of Hydration

Heat of Hydration test results are shown in Tables 4-3 and 4-4 as reported by CTL. Two samples of cement from Florida Rock Industries Newberry plant were submitted to CTL. The heat of Hydration of the second sample was higher than the first sample. This shows that heat of hydration of different batches of cements even from the same plant varies.

Table 4-3 Heat of Hydration Test Results for Cement

No	Cement Source	Heat of Hydration @ 7 days (cal/g)	Heat of Hydration @ 28 days (cal/g)
1	Florida Rock Ind. (Sample 1) (Newberry)	66.2	84.1
2	Florida Rock Ind. (Sample 2) (Newberry)	76.7	91.3
3	Rinker (Miami)	77.6	88.2
4	Florida mining and materials (Cemex in Brooksville)	78.2	94.4
5	Tarmac (Miami)	80.0	95.1

Table 4-4 Heat of Hydration for Cement and Pozzolanic Material Blends

No	Cement Source	Heat of Hydration @ 7 days (cal/g)	Heat of Hydration @ 28 days (cal/g)
1	20% Fly ash + 80 % Florida Rock Ind. Sample 1	56.8	68.9
2	50% Slag + 50 % Florida Rock Ind. Sample 1	59.2	75.9
3	35% Fly ash + 65 % Florida Rock Ind. Sample 2	61.4	65.4
4	70% Slag + 30 % Florida Rock Ind. Sample 2	47.5	58.6

Adiabatic Temperature Rise

In this section test results for adiabatic temperature rise of different mixes are presented. For each mix, temperature was monitored immediately after placing concrete in Cure Chamber. To calculate temperature rise, initial temperature reading as recorded by the computer was deducted from succeeding temperature readings. After calculating the temperature rise for data from each Cure Chamber, numbers were averaged. Data was recorded every six minutes for 14 days. In Tables 4-5 to 4-8 temperature rise data of the different mixes are reported for every 12 hours. For the first five mixes, data was recorded for 28 days. Tests were repeated for the following mixes: 73A00P, 73A20F, 73A35F, 73A50S, 73A70S, 73B50S, 73B70S and 95A705.

Laboratory Tests

Test results are divided into four groups based on cement source and placing temperature. The results for each group are presented in a separate table.

Cement A with 73ºF Placing Temperature

Tests started with mixes 73A00P and 73A20F. The second group of mixes that were tested consisted of mixes 73A35F and 73A50S. An Equipment malfunction occurred during the second run of tests that halted the data recording for mix 73A35F . In the third group of tests, mixes 73A35F and 73A70S were tested. These mixes were monitored for 28 days, while the temperature rise was recorded. Continuous temperature rise with a relatively high rate was noticed which was unexpected. The manufacturer of the Sure Cure system was contacted for advice. The manufacturer suggested adjusting one of the control parameters of the Sure Cure system in a way that always 1.5 to 2.0 degrees of Fahrenheit temperature difference between heaters and core temperature is kept. The logic behind this provision is that there is a good possibility that if the heaters are set to reach the exact same temperature as that of the sample core, due to small size of the sample, extra heat is transferred to sample core. This extra heat, which is not generated by cement hydration, causes core temperature to rise. A rise in core temperature is relayed to the controller software and software in turn orders the heaters to warm up to keep sample's outer surface in a same temperature as the core. When heaters start working again, extra heat is transferred to the core and the procedure repeats. These events form a cycle of heating and thus a continuous temperature rise would be recorded. The proceeding tests were run in accordance with this suggestion. To keep the consistency in the results, it was decided to repeat the test for the first five mixes.

Table 4-5 Adiabatic Temperature Rise Data for Concrete with Cement A and 73ºF Placing Temperature

Time (Day)	Concrete with Cement A (73ºF Placing Temperature)									
	Plain Cement		20% Fly Ash		35% Fly Ash		50% Slag		70% Slag	
	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)
	1st Test	2nd Test	1st Test	2nd Test	1st Test	2nd Test	1st Test	2nd Test	1st Test	2nd Test
0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.5	9.40	34.50	5.60	21.60	4.00	6.80	4.50	19.90	13.45	10.70
1.0	36.80	60.80	19.30	49.10	24.20	34.00	22.90	40.50	31.50	23.15
1.5	60.00	70.25	35.90	61.20	42.30	48.40	47.00	61.40	56.20	42.40
2.0	80.00	75.45	55.20	66.90	52.55	56.00	69.10	75.80	68.10	57.50
2.5	87.00	79.05	66.70	71.50	58.95	61.00	80.00	83.10	73.53	64.10
3.0	93.40	81.60	75.70	75.50	64.95	66.00	83.70	86.70	76.20	67.05
3.5	95.90	83.50	82.00	78.80	69.63	70.40	85.50	88.90	77.65	69.20
4.0	97.50	85.05	84.70	80.80	72.35	73.40	86.50	90.40	78.75	70.60
4.5	98.60	86.55	86.10	82.00	73.80	75.50	87.40	91.80	79.78	71.10
5.0	99.50	87.65	87.00	82.60	74.90	77.10	87.90	93.00	80.53	71.10
5.5	100.20	88.60	87.60	83.20	75.83	78.30	88.30	93.80	81.15	71.10
6.0	100.70	89.55	88.10	83.80	76.48	79.30	88.30	94.70	81.60	71.10
6.5	101.30	90.30	88.70	83.80	77.30	80.00	88.40	95.20	81.63	71.10
7.0	101.10	91.05	88.60	84.10	77.83	80.80	88.20	95.80	82.18	71.10
7.5	101.50	91.60	88.80	84.10	78.38	81.40	88.30	96.20	81.95	71.10
8.0	101.30	92.05	88.70	84.00	79.08	81.50	88.50	96.50	82.15	71.10
8.5	102.00	92.10	89.00	84.20	79.23	81.80	88.50	96.90	82.50	71.10
9.0	102.30	92.15	89.30	84.20	79.65	81.60	88.60	97.00	82.73	71.10
9.5	102.60	92.15	89.70	84.30	79.95	81.70	88.70	97.10	82.98	71.10
10.0	103.00	92.15	90.00	84.30	80.30	81.80	88.70	97.10	83.25	71.10
10.5	103.30	92.20	90.40	84.30	80.75	81.90	88.70	97.10	83.15	71.10
11.0	103.60	92.25	90.70	84.30	81.05	81.90	88.80	97.10	83.35	71.10
11.5	103.90	92.25	91.00	84.40	81.33	82.00	89.00	97.10	83.65	71.10
12.0	104.30	92.30	91.30	84.40	81.65	82.10	88.90	97.10	83.75	71.10
12.5	104.50	92.40	91.60	84.40	81.65	82.10	88.90	97.10	83.75	71.10
13.0	104.80	92.45	91.90	84.40	81.65	82.30	88.90	97.10	83.75	71.10
13.5	104.90	92.60	92.10	84.40	81.65	82.20	88.90	97.10	83.75	71.10
14.0	105.00	92.60	92.20	84.40	81.65	82.10	88.90	97.10	83.75	71.10

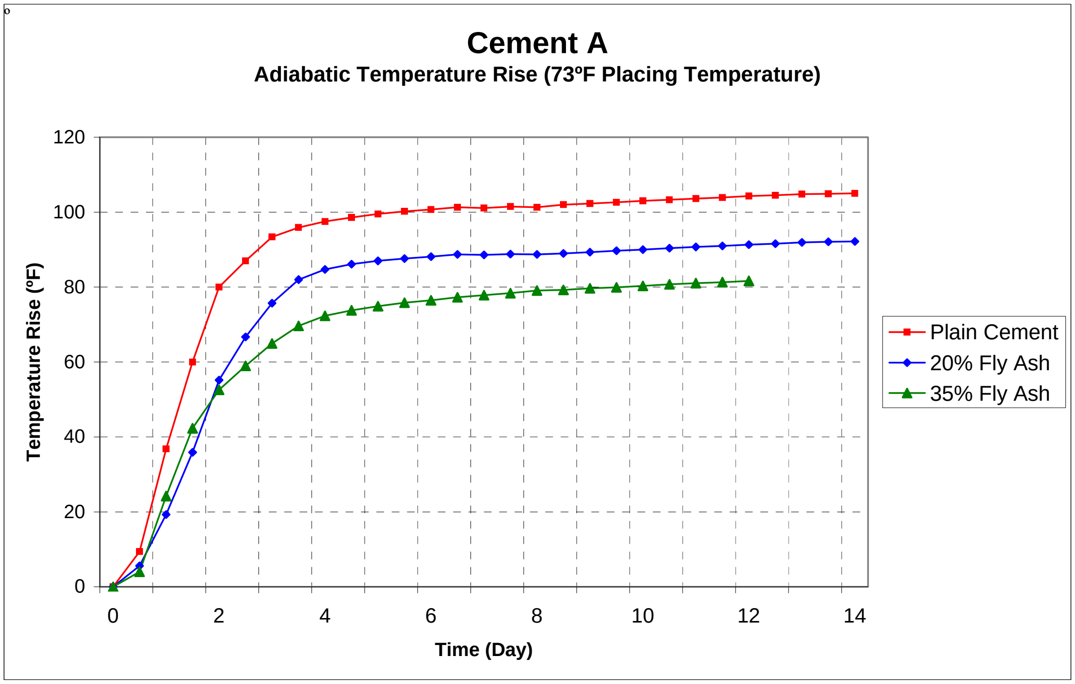


Figure 4-1 Temperature Rise for Concrete Mixtures of Cement A With Different Percentages of Fly Ash (First Run)

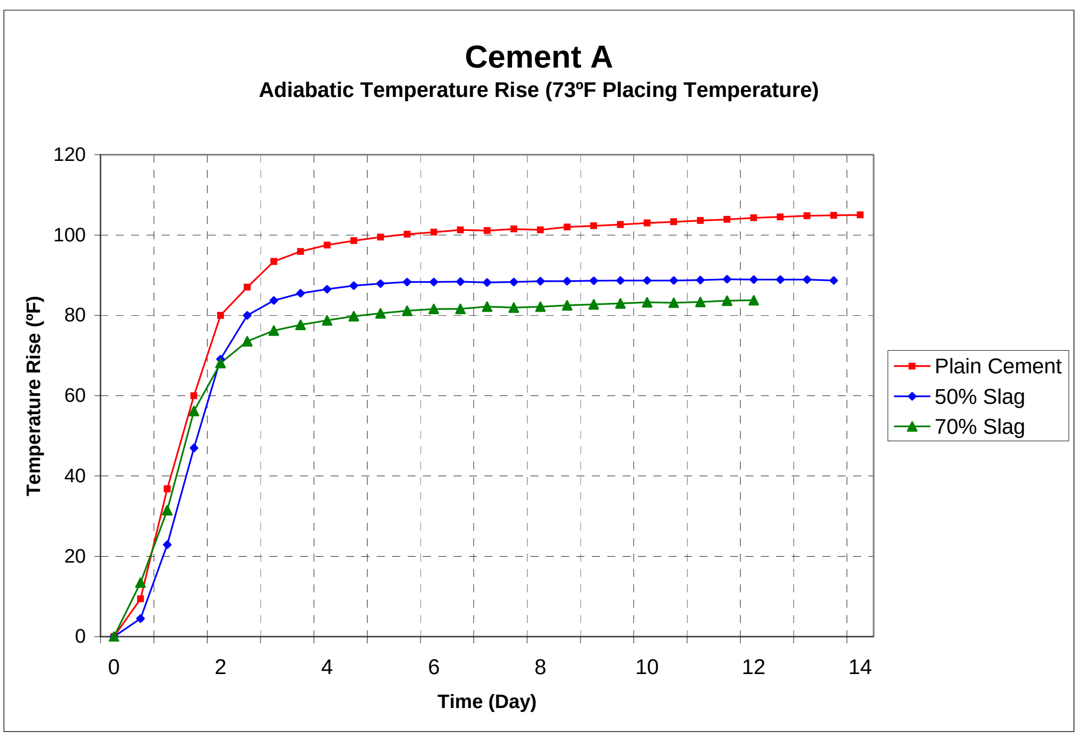


Figure 4-2 Temperature Rise for Concrete Mixtures of Cement A With Different Percentages of Slag (First Run)

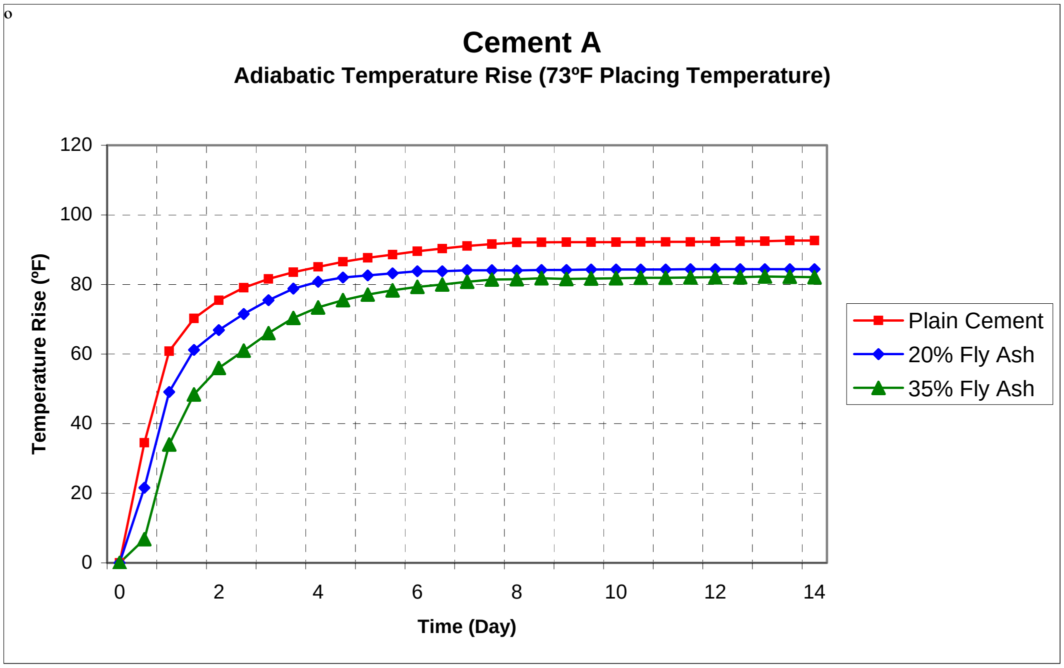


Figure 4-3 Temperature Rise for Mixtures of Cement A With Different Percentages of Fly Ash (Second Run)

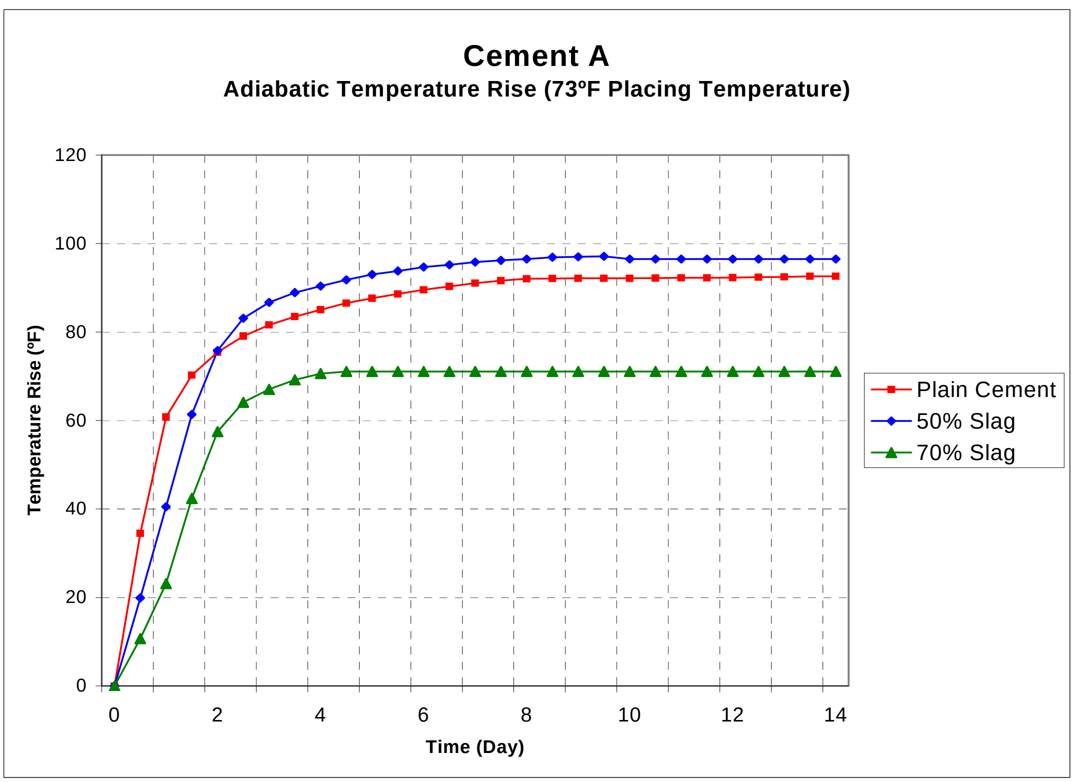


Figure 4-4 Temperature Rise for Concrete Mixtures of Cement A With Different Percentages of Slag (Second Run)

As it is shown in Figure 4-4, the second run of test for the mix 73A50S reached the maximum temperature rise of 97.10 ºF, which is even higher than the maximum temperature rise of the mix 73A00P without any pozzolanic material. The only explanation for this unexpected result is the test equipment or controller software malfunction. This set of data was not considered reliable.

Cement B with 73ºF Placing Temperature

In Table 4-1 temperature rise data for mixes with cement B and 73ºF placing temperature is presented. Test for mixes 73B50S and 73B70S were repeated because the first set of collected temperature rise records was stopped after 7.5 days due to equipment malfunction.

Table 4-1 Adiabatic Temperature Rise Data for Concrete With Cement B and 73ºF Placing Temperature

Time (Day)	Concrete with Cement B (73ºF Placing Temperature)						
	Plain Cement	20% ºFly Ash	35% ºFly Ash	50% Slag		70% Slag	
	Temp. Rise (ºF)	Temp. Rise (ºF	Temp. Rise (ºF	Temp. Rise (ºF) 1st Test	Temp. Rise (ºF) 2nd Test	Temp. Rise (ºF) 1st Test	Temp. Rise (ºF) 2nd Test
0.0	0.00	0.00	0.00	0.00	0	0.00	0.00
0.5	28.90	15.50	25.10	23.85	20.07	13.85	15.65
1.0	69.85	61.10	45.40	47.30	49.95	32.30	40.00
1.5	80.10	74.75	54.20	68.15	67.51	57.00	61.22
2.0	84.40	82.70	60.00	78.15	72.48	67.80	66.52
2.5	87.65	87.35	64.10	83.65	75.48	73.40	69.39
3.0	90.60	88.25	67.60	87.05	77.91	76.55	71.57
3.5	92.70	90.00	70.00	87.95	79.89	77.95	73.48
4.0	94.50	91.35	71.80	88.55	81.47	79.25	74.96
4.5	95.85	92.35	73.00	88.75	82.85	80.20	76.35
5.0	97.15	93.20	74.00	88.75	83.86	81.55	77.22
5.5	98.10	93.75	74.50	88.75	84.47	82.05	77.65
6.0	99.05	94.35	75.10	88.75	85.05	82.25	78.05
6.5	99.95	94.70	75.40	88.75	85.35	82.80	78.05
7.0	100.80	95.30	75.80	88.75	85.63	82.80	78.05
7.5	101.60	95.75	76.00	88.75	85.90	82.80	78.05
8.0	102.45	96.05	76.20	-	86.18	-	78.05
8.5	102.85	96.05	76.50	-	86.32	-	78.05
9.0	103.30	96.15	76.60	-	86.47	-	78.05
9.5	103.65	96.10	76.80	-	86.58	-	78.05
10.0	104.20	96.25	77.00	-	86.77	-	78.05
10.5	104.35	96.30	77.10	-	86.82	-	78.05
11.0	104.55	96.30	77.30	-	86.88	-	78.05
11.5	104.70	96.45	77.40	-	86.93	-	78.05
12.0	104.90	96.45	77.50	-	87.00	-	78.05
12.5	105.20	96.60	77.60	-	87.10	-	78.05
13.0	105.45	96.65	77.50	-	87.18	-	78.05
13.5	105.60	96.80	77.70	-	87.23	-	78.05
14.0	105.95	96.95	78.30	-	87.35	-	78.05

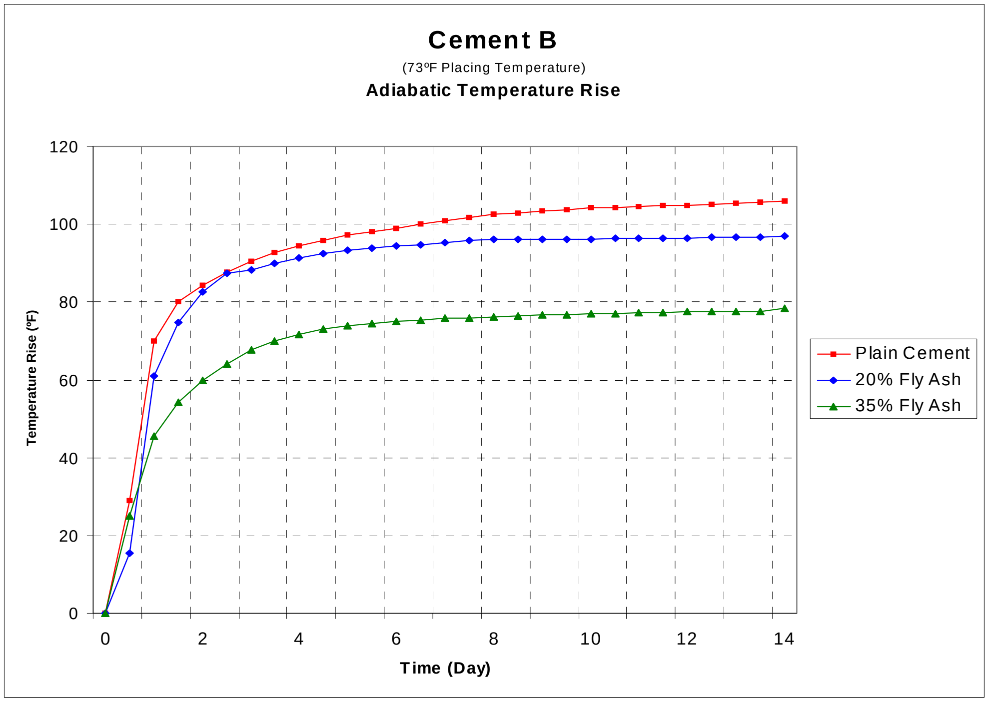


Figure 4-5 Temperature Rise for Concrete Mixtures of Cement B With Different Percentages of Fly Ash

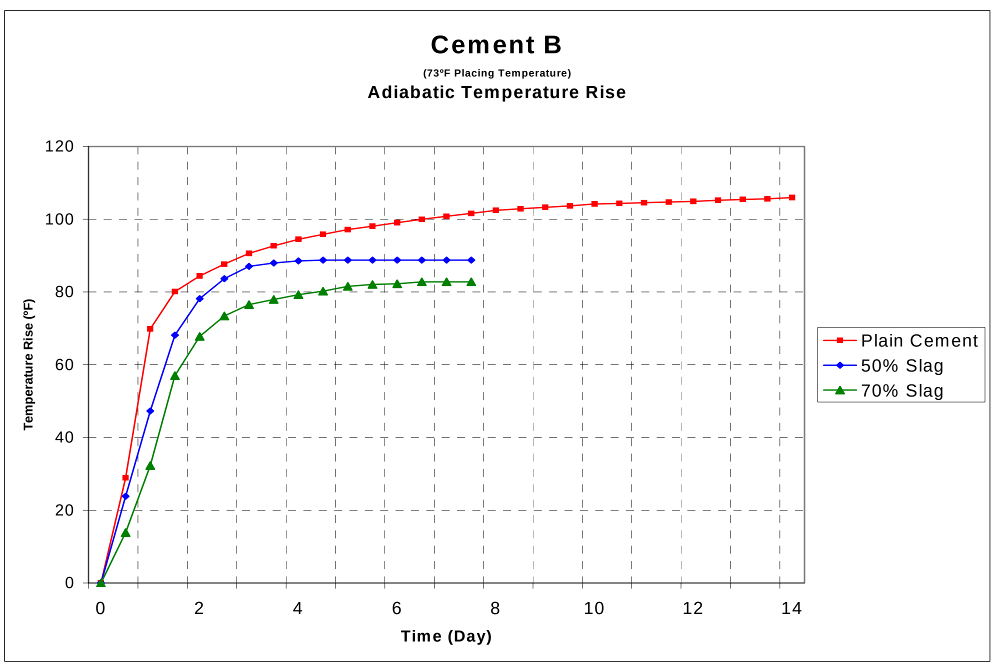


Figure 4-6 Temperature Rise for Concrete Mixtures of Cement B With Different Percentages of Slag (First Run)

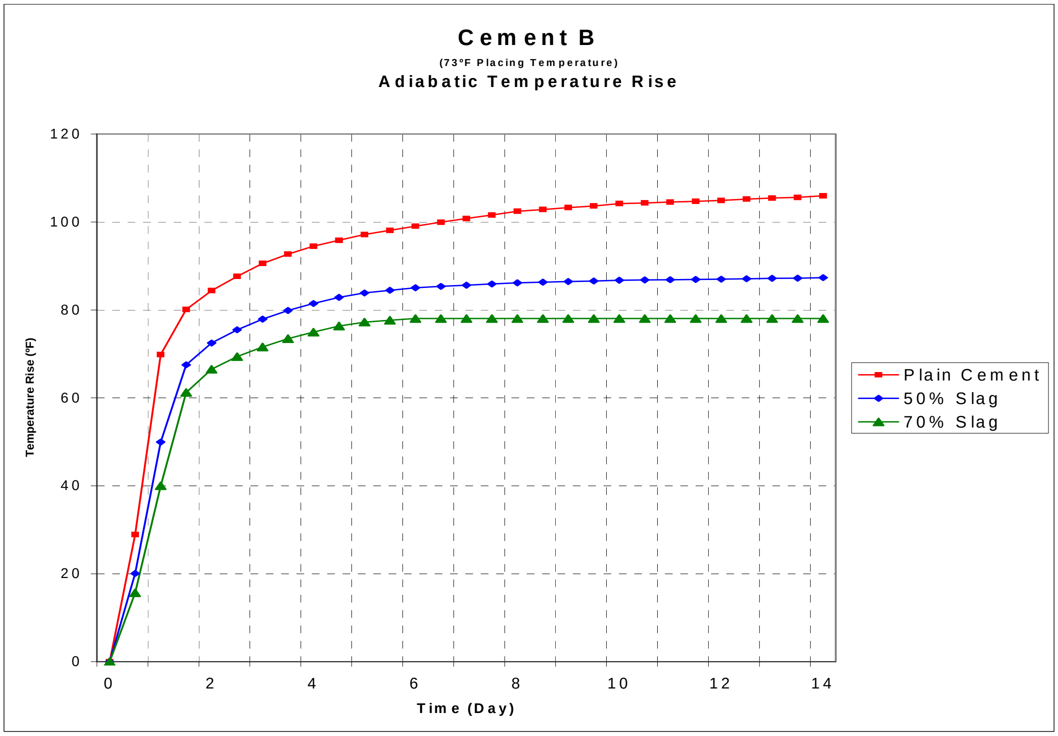


Figure 4-7 Temperature Rise for Concrete Mixtures of Cement B With Different Percentages of Slag (Second Run)

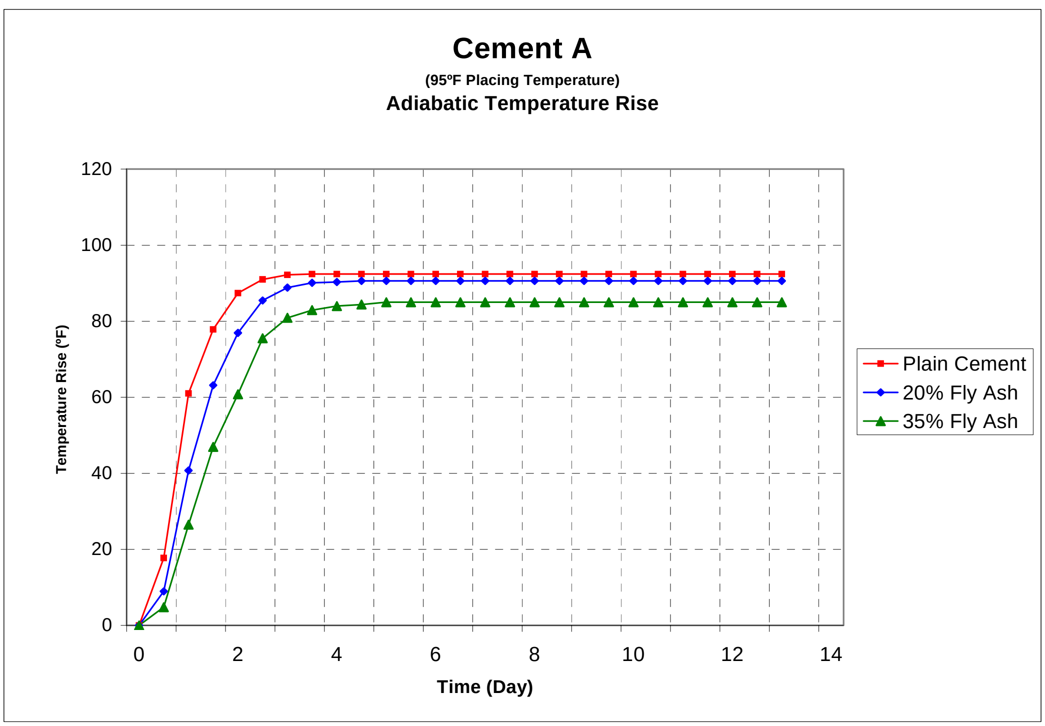


Figure 4-8 Temperature Rise for Concrete Mixtures of Cement A With Different Percentages of Fly Ash

Cement A with 95ºF Placing Temperature

Five mixes were made with cement A with placing temperature ranging from 93ºF to 96ºF (See Table 4-2). The first run of the test for the mix 95A70S showed unexpected results. Temperature rise in the first 24 hours was only about 6ºF which is much lower than similar mixes. It was decided to run this test again. The results of both tests are presented in Table 4-2, Figures 4-9 and 4-10.

Table 4-2 Adiabatic Temperature Rise Data for Concrete with Cement A and 95ºF Placing Temperature

Time (Day)	Concrete with Cement A (95ºF Placing Temperature)					
	Plain Cement	20% Fly Ash	35% Fly Ash	50% Slag	70% Slag	
	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF) 1st Test	Temp. Rise (ºF) 2nd Test
0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	17.75	8.95	4.80	19.30	5.10	4.30
1.0	61.00	40.75	26.50	54.50	5.30	16.80
1.5	77.85	63.15	46.95	82.70	12.40	49.50
2.0	87.40	76.95	60.80	91.00	42.80	57.70
2.5	91.00	85.45	75.50	92.20	56.05	60.50
3.0	92.20	88.85	80.90	92.20	58.85	62.10
3.5	92.40	90.10	82.90	92.20	60.15	63.60
4.0	92.40	90.30	84.00	92.20	61.00	64.40
4.5	92.40	90.60	84.40	92.30	61.55	65.40
5.0	92.40	90.60	85.00	92.30	62.30	66.10
5.5	92.40	90.60	85.00	92.30	62.50	66.70
6.0	92.40	90.60	85.00	92.30	63.15	67.10
6.5	92.40	90.60	85.00	92.30	63.25	67.50
7.0	92.40	90.60	85.00	92.30	63.40	67.90
7.5	92.40	90.60	85.00	92.30	63.75	68.20
8.0	92.40	90.60	85.00	92.30	63.80	68.60
8.5	92.40	90.60	85.00	92.30	63.95	68.90
9.0	92.40	90.60	85.00	92.30	64.05	69.00
9.5	92.40	90.60	85.00	92.30	64.25	69.30
10.0	92.40	90.60	85.00	92.30	64.25	69.40
10.5	92.40	90.60	85.00	92.30	64.25	69.60
11.0	92.40	90.60	85.00	92.30	64.25	69.80
11.5	92.40	90.60	85.00	92.30	64.25	69.90
12.0	92.40	90.60	85.00	92.30	64.25	70.10
12.5	92.40	90.60	85.00	92.30	64.25	70.10
13.0	92.40	90.60	85.00	92.30	64.25	70.40
13.5	92.40	90.60	85.00	92.30	64.25	70.50
14.0	92.40	90.60	85.00	92.30	64.25	70.80

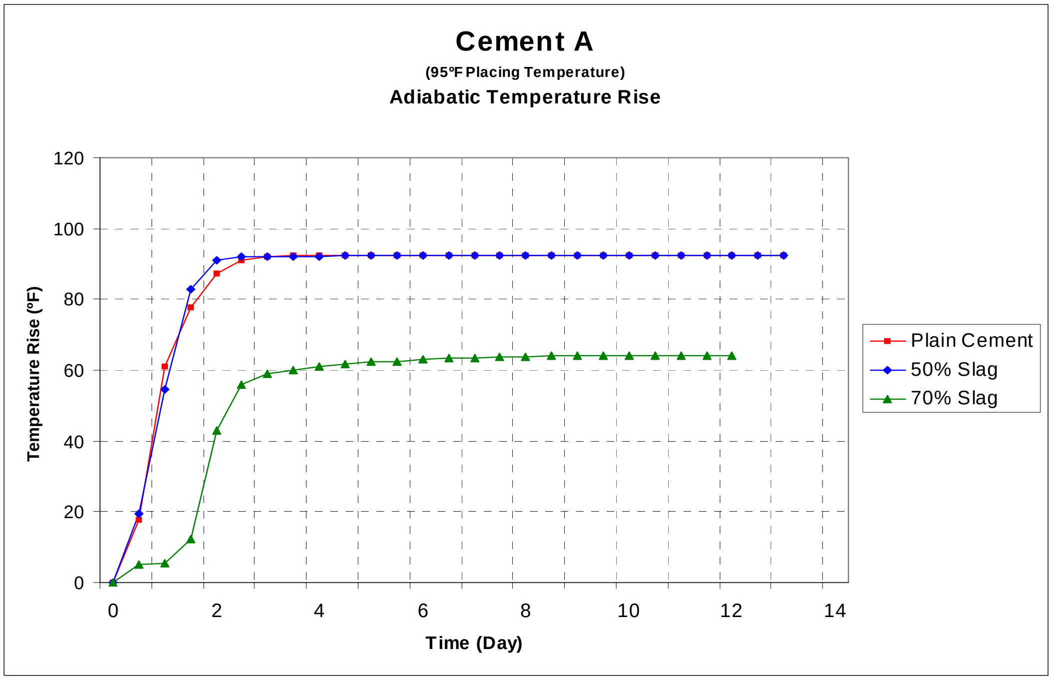


Figure 4-9 Temperature Rise for Concrete Mixtures of Cement A With Different Percentages of Slag

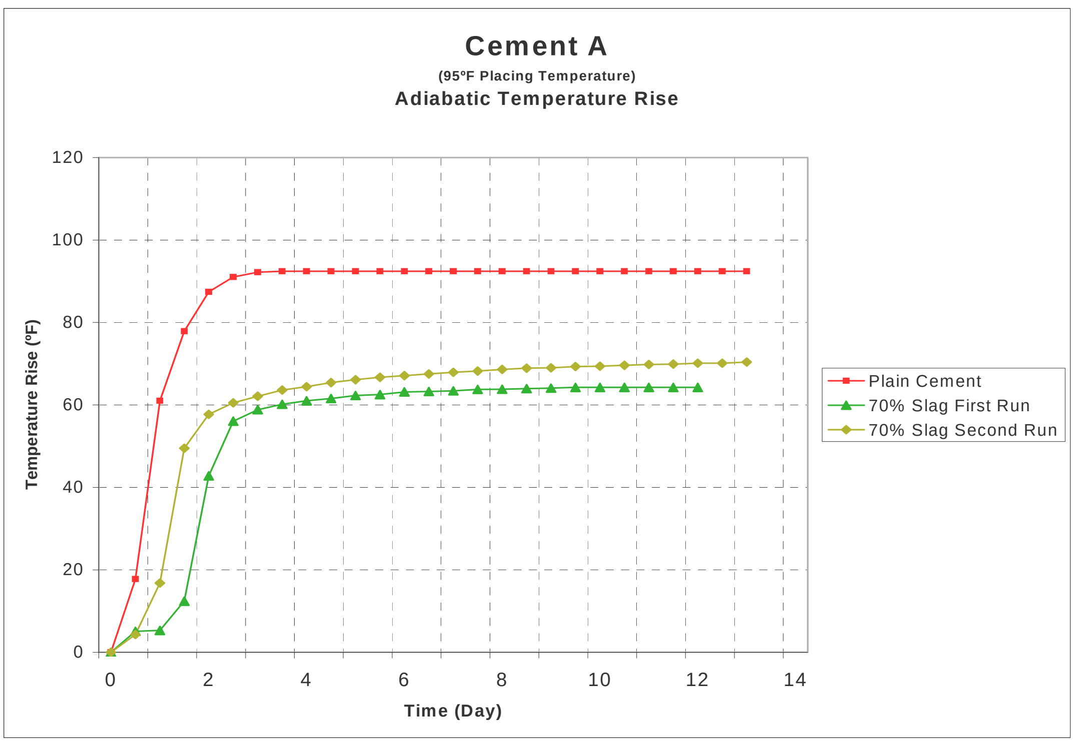


Figure 4-10 Temperature Rise for Concrete Mixtures of Cement A With Different Percentages of Slag (First & Second Run)

Cement B with 95ºF Placing Temperature

In Table 4-3 temperature rise data for mixes with cement B and 95ºF placing temperature is presented. A review of test results and Figures 4-11 and 4-12 shows that temperature rise for the mix without pozzolanic material was very slow in the first 12 hours.

Table 4-3 Adiabatic Temperature Rise Data for Concrete with Cement B and 95ºF Placing Temperature

Time (Day)	Concrete with Cement B (95ºF Placing Temperature)				
	Plain Cement	20% Fly Ash	35% Fly Ash	50% Slag	70% Slag
	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)
0.0	0.00	0.00	0.00	0.00	0.00
0.5	2.60	56.80	50.35	58.60	45.00
1.0	72.15	72.90	61.90	76.70	69.40
1.5	84.20	77.40	67.85	80.70	73.90
2.0	85.60	77.60	68.90	81.80	76.30
2.5	85.65	77.60	69.20	82.00	77.90
3.0	85.95	77.60	69.30	82.00	78.90
3.5	85.95	77.60	69.60	82.00	79.50
4.0	85.95	77.60	69.90	82.00	79.90
4.5	85.95	77.60	69.95	82.00	79.90
5.0	85.95	77.60	69.95	82.00	79.90
5.5	85.95	77.60	69.95	82.00	79.90
6.0	85.95	77.60	69.95	82.00	79.90
6.5	85.95	77.60	69.95	82.00	79.90
7.0	85.95	77.60	69.95	82.00	79.90
7.5	85.95	77.60	69.95	82.00	79.90
8.0	85.95	77.60	69.95	82.00	79.90
8.5	85.95	77.60	69.95	82.00	79.90
9.0	85.95	77.60	69.95	82.00	79.90
9.5	85.95	77.60	69.95	82.00	79.90
10.0	85.95	77.60	69.95	82.00	79.90
10.5	85.95	77.60	69.95	82.00	79.90
11.0	85.95	77.60	69.95	82.00	79.90
11.5	85.95	77.60	69.95	82.00	79.90
12.0	85.95	77.60	69.95	82.00	79.90
12.5	85.95	77.60	69.95	82.00	79.90
13.0	85.95	77.60	69.95	82.00	79.90
13.5	85.95	77.60	69.95	82.00	79.90
14.0	85.95	77.60	69.95	82.00	79.90

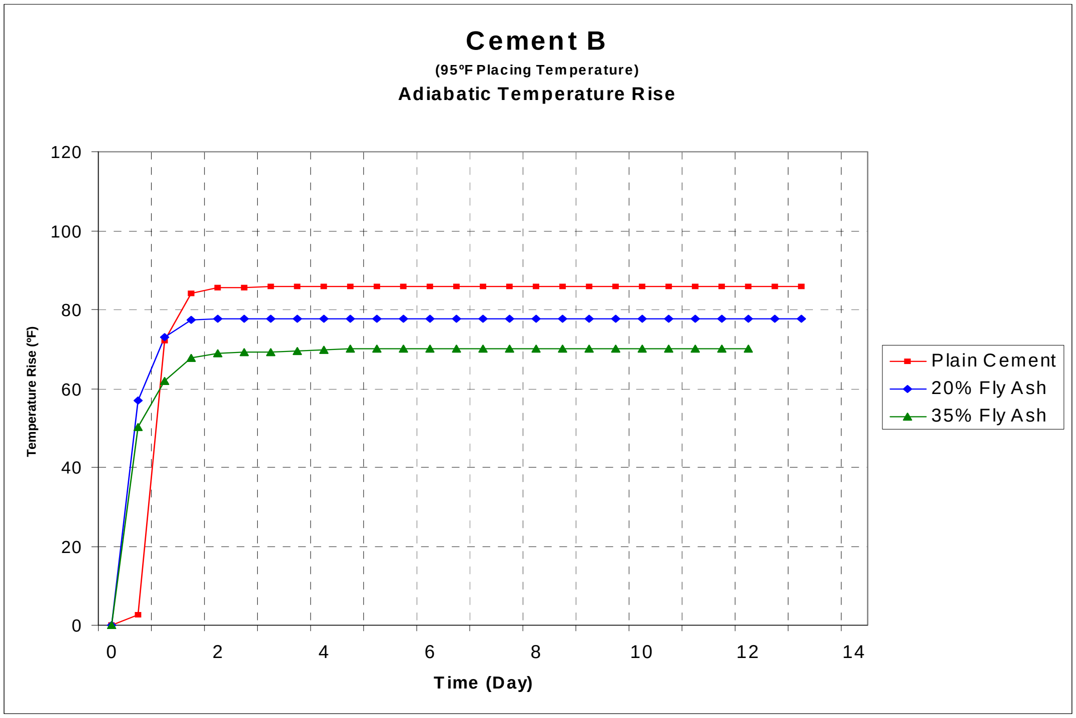


Figure 4-11 Temperature Rise for Concrete Mixtures of Cement B With Different Percentages of Fly Ash

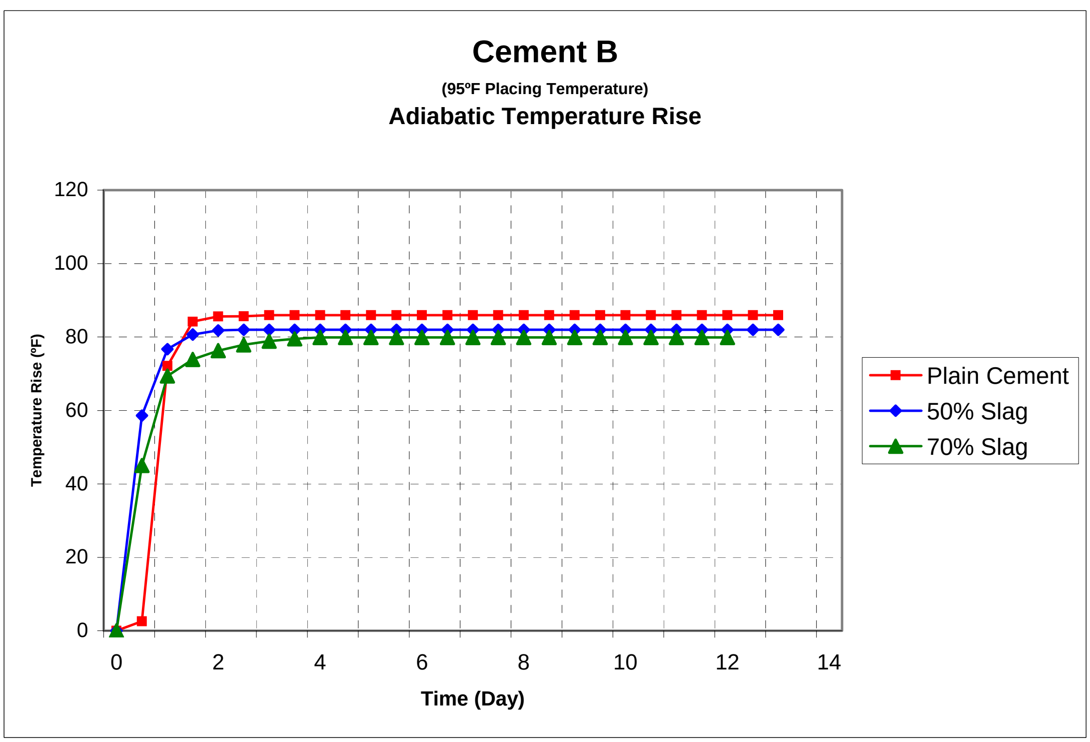


Figure 4-12 Temperature Rise for Concrete Mixtures of Cement B With Different Percentages of Slag

Field Tests

In Table 4-6 the temperature rise readings for two series of tests run on samples from a mass concrete pour are presented. The samples were taken from footings of Bella Vista Bridge (a bridge over I-4 west of Memorial Blvd.) in Lakeland, Florida. Due to the malfunction of computer system, it was not possible to monitor the temperature rise in samples immediately after concrete placing. It was decided to bring back the samples after placing in Cure Chambers to the FDOT laboratory in Gainesville. The samples were moved while they were fresh and were not connected to the controller for about 2 hours. These two factors may affect the heat generation, although it is not proven. However, comparing the results from the first and the second tests do not show a significant difference. For the first sample, temperature rise was 26.00°F in the first 12 hours, while the second sample gained 25.65°F during the same period. Both samples continued to generate heat with the same rate in the following days. The first sample was monitored for 6 days. After six days, test equipment was needed for the second run of field tests, therefore temperature rise monitoring was stopped.

Concrete for both samples was mixed according to FDOT approved mix design No. 01-0632 with 360 lb/yd^3 of Type II cement from Florida Rock Newberry (Same source as cement A used in laboratory tests) and 360 lb/yd^3 slag (See Table 4-4). The sample temperature was 70°F for the first test and 72°F for the second one.

Table 4-4 Mix Proportions of Field Test Samples

No.	Mix Designation	Cement (lb/CY)	Fly Ash (lb/CY)	Slag (lb/CY)	Water (lb/CY)	Fine Aggregate (lb/CY)	Coarse Aggregate (lb/CY)	Air Entertainer (oz/CY)	Admixture (oz/CY)
1	Site50S-1	360.00	0.00	360.00	252.00	1168.00	1708.00	5.00	28.8
2	Site50S-2	360.00	0.00	360.00	260.00	1140.00	1696.00	5.00	28.8

Table 4-5 Fresh Concrete Properties of Field Test Samples

No	Mix	Slump (in.)	Air Content (%)	Mix Temp. (ºF)	Air Temp. (ºF)	Water to Cementitious Material Ratio	Unit Weight (lb/CF)
1	Site50S-1	3.00	3.00	70	68	0.35	142.6
2	Site50S-2	3.25	3.00	72	71	0.38	141.4

Table 4-6 Adiabatic Temperature Rise for Samples Taken from the Field

Time (Day)	Temp. Rise (ºF) 1st Test (Site50S-1)	Temp. Rise (ºF) 2nd Test (Site50S-2)
0.0	0.00	0.00
0.5	26.00	24.65
1.0	43.80	41.95
1.5	62.60	59.50
2.0	74.03	69.75
2.5	79.66	75.20
3.0	82.54	78.60
3.5	84.32	81.10
4.0	85.58	83.00
4.5	86.65	84.55
5.0	87.56	85.95
5.5	88.29	87.25
6.0	88.92	88.26
6.5	-	89.15
7.0	-	89.94
7.5	-	90.75
8.0	-	91.23
8.5	-	91.65
9.0	-	91.98
9.5	-	92.10
10.0	-	92.15
10.5	-	92.17
11.0	-	92.18
11.5	-	92.18
12.0	-	92.18
12.5	-	92.18
13.0	-	92.18
13.5	-	92.18
14.0	-	92.18

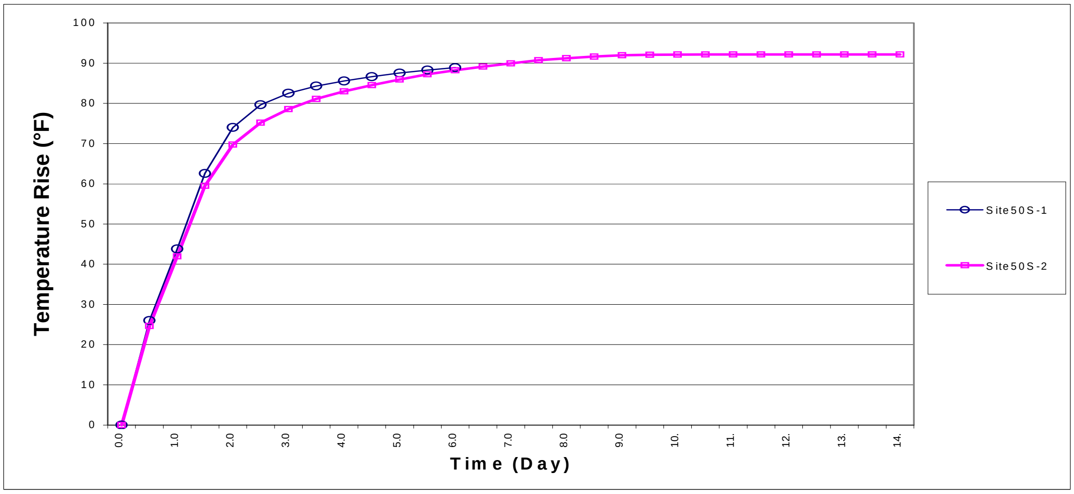


Figure 4-13 Temperature Rise for Field Test Samples

Thermal Diffusivity

In Table 4-7 thermal diffusivity values for different test mixes are shown. These numbers were calculated based on method presented in Chapter 3.

Table 4-7 Thermal Diffusivity of Concrete Samples

Mix Designation	Diffusivity (ft^2/day)	Diffusivity (ft^2/hr)
73A00P	**0.72**	**0.030**
73A20F	**0.70**	**0.029**
73A35F	0.79	0.033
73A50S	**0.69**	**0.029**
73A70S	0.77	0.032
73B00P	0.82	0.034
73B20F	0.82	0.034
73B35F	0.80	0.033
73B50S	0.73	0.030
73B70S	0.71	0.030
95A00P	0.82	0.034
95A20F	0.81	0.034
95A35F	0.77	0.032
95A50S	0.76	0.031
95A70S	0.76	0.032
95B00P	0.79	0.033
95B20F	0.79	0.033
95B35F	0.79	0.033
95B50S	0.75	0.031
95B70S	0.73	0.031

In Figure 4-14 thermal diffusivity of cements A and B with different placing temperatures and percentage of pozzolanic materials are drawn. As it is shown, three of the curves follow the same pattern while one curve (Cement A 73°F placing temperature) has a very different pattern. All of the diffusivity tests were run with the same apparatus and under the same condition except the first three tests (73A00P, 73A20F and 73A50S; shown in bold letters in Table 4-7). For the first three tests a smaller diffusing chamber (Figure 4-15) was used compared to the diffusivity chamber used for other tests (Figure

4-16). The smaller body of water in the first diffusivity chamber has a lower capacity to absorb heat from the concrete samples. It therefore takes longer for the samples to cool down. A longer cool down time leads to a smaller diffusivity number. This fact explains the different shape of curve for cement A with 73°F placing temperature. As mentioned earlier, three of the points in this curve were results of the test with a different apparatus while other two points (35% and 70%) were calculated based on data collected from a test with the larger diffusion chamber. Because of this flaw in the test procedure, diffusivity numbers for mixes 73A00P, 73A20F and 73A50S are not comparable to the other results and are lower than the actual diffusivity numbers. In Figure 4-14, it can be seen that mixes with higher percentage of pozzolanic material have a lower diffusivity number.

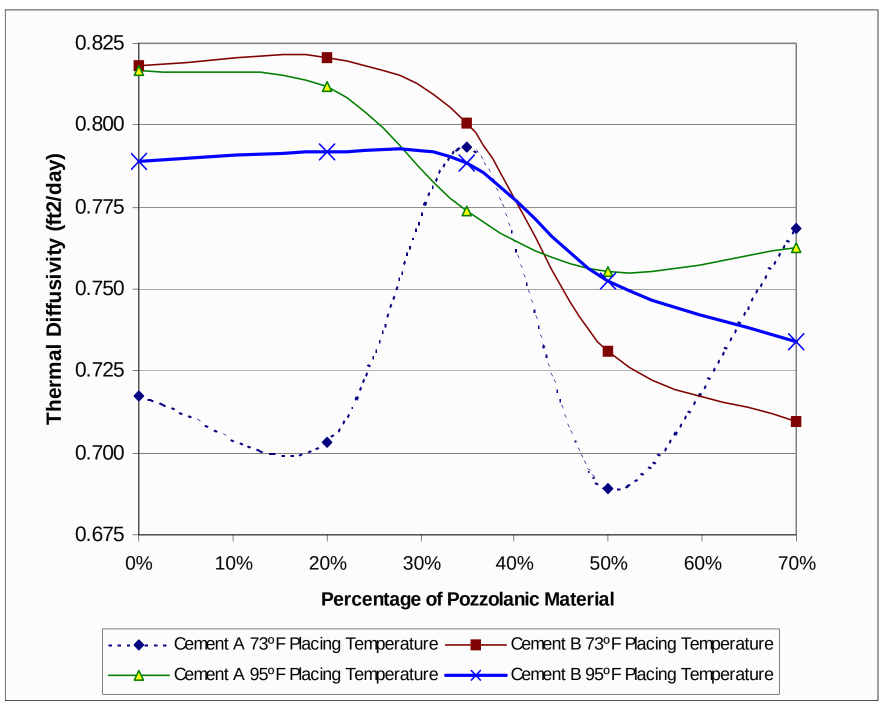


Figure 4-14 Thermal Diffusivity of Concrete Samples with Different Percentage of Pozzolanic Materials

Figure 4-15 Small Diffusion Chamber

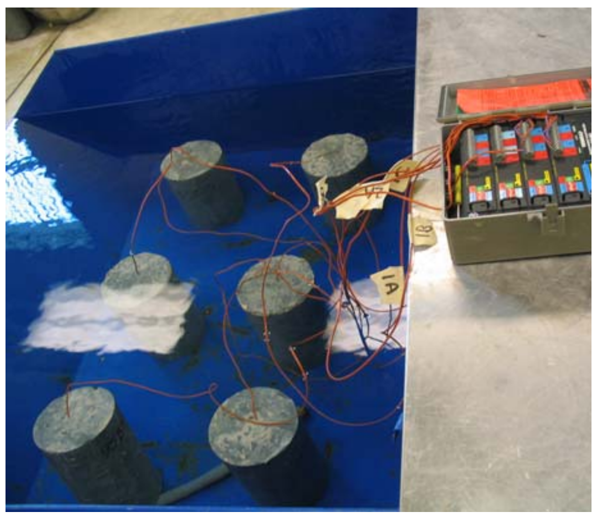


Figure 4-16 Large Diffusion Chamber

Calculation Example for Concrete Thermal Diffusivity

In this section thermal diffusivity calculation method for mix 73B00P is presented. Based on the data in Table 4-8 the elapsed time between the moment that temperature difference is 80°F to the moment that it is 20°F equals to 23.98 minutes. Thermal diffusivity for the mix is calculated as follows:

Thermal Diffusivity = $\alpha = 0.812278/(t_1 - t_2)$ and $(t_1 - t_2)$=23.98 minutes

$\Rightarrow \alpha = 0.812278/23.98 = 0.034\ ft^2/hr = \underline{0.81\ ft^2/day}$ ▯

Table 4-8 History of Temperature Difference Between Concrete Specimen Core and Water

Time (Minutes)	Temperature Difference Between Specimen and Water (°F)	Time (Minutes)	Temperature Difference Between Specimen and Water (°F)
1	84.0	14	40.3
2	79.8	15	38.0
3	75.4	16	35.7
4	71.3	17	33.7
5	67.4	18	31.7
6	63.7	19	30.0
7	60.4	20	28.3
8	57.0	21	26.8
9	53.9	22	25.4
10	51.1	23	23.8
11	48.0	24	22.3
12	45.6	25	21.0
13	42.9	26	19.7

Compressive Strength

Compressive strength tests were performed at different specimen ages and with 4″x8″ and 6″x12″ specimens. Compressive strength test results of 4″x8″ specimens are modified to be comparable to results from tests with 6″x12″ specimens.

In Table 4-9 test results for high temperature cured specimens are presented. For each mix 3 specimens were broken and the average number is reported. In Table 4-10 test results for room temperature cured specimens are shown. These numbers are also the average of results from three tests.

Table 4-9 Compressive Strength of High Temperature Cured Concrete Specimens

Mix	Compressive Strength (psi)	Age (Day)	Max. Temp (ºF)	Compressive Strength (psi)	Age (Day)	Max. Temp (ºF)
73A00P	7283	28	200.0	7417	28	190.0
73A20F	7930	28	180.0	7255	28	200.0
73A35F	7496	28	164.4	7428	28	154.1
73A50S	7846	28	170.0	7986	28	198.0
73A70S	8216	28	158.2	7055	28	150.8
73B00P	5679	15	181.9	5051	28	175.8
73B20F	6854	15	181.4	6508	28	171.5
73B35F	6702	15	151.9	6462	28	153.0
73B50S	7595	15	161.8	7433	28	164.4
73B70S	7431	15	155.8	7324	28	159.7
95A00P	7482	15	185.4	8623	28	190.2
95A20F	7671	15	185.6	7395	28	192.1
95A35F	6056	15	179.0	5182	28	189.8
95A50S	7334	15	186.2	6698	28	190.4
95A70S	8553	15	152.7	6378	28	161.8
95B00P	7369	15	184.6	6868	28	185.3
95B20F	8052	15	167.5	7114	28	183.7
95B35F	7287	15	170.8	6711	28	171.1
95B50S	7251	15	177.8	6843	28	188.2
95B70S	8332	15	173.9	7541	28	187.9
73A00P-R	7080	15	163.7	7391	28	167.5
73A20F-R	8675	15	157.2	8055	28	159.6
73A35F-R	8150	15	152	7980	28	160.2
73A50S-R	7946	15	163.2	7311	28	167.0
73A70S-R	8369	15	136.5	8485	28	143.7
73B50S-R	8862	15	150.1	9145	28	159.3
73B70S-R	8332	15	154	8710	28	150.1
95A70S-R	8716	15	164.5	9635	28	171.1

Table 4-10 Compressive Strength of Room Temperature Cured Concrete Specimens

Mix	Compressive Strength (psi)	Age (Day)	Max. Temp (ºF)	Compressive Strength (psi)	Age (Day)	Max. Temp (ºF)
73A00P	8985	28	80	8584	28	80
73A20F	8436	28	80	8854	28	80
73A35F	7441	28	80	-	-	-
73A50S	9308	28	75	-	-	-
73A70S	8036	28	75	-	-	-
73B00P	6705	15	75	6906	28	75
73B20F	7292	15	75	7372	28	75
73B35F	6174	15	74	7040	28	74
73B50S	7101	15	73	7960	28	73
73B70S	6997	15	75	7909	28	75
95A00P	8239	15	93	8394	28	93
95A20F	7065	15	95	8392	28	95
95A35F	4951	15	94	7565	28	94
95A50S	7252	15	96	8107	28	96
95A70S	7360	15	93	7213	28	93
95B00P	7306	15	99	9025	28	99
95B20F	7902	15	98	8172	28	98
95B35F	6754	15	101	8394	28	101
95B50S	8118	15	101	8392	28	101
95B70S	8028	15	101	7565	28	101
73A00P-R	7395	15	75	8180	28	75
73A20F-R	7108	15	75	8132	28	75
73A35F-R	6522	15	75	7654	28	75
73A50S-R	6435	15	75	7690	28	75
73A70S-R	7413	15	76	8181	28	76
73B50S-R	8107	15	75	9124	28	75
73B70S-R	7224	15	75	9131	28	75
95A70S-R	8871	15	97	7819	28	97

In Figures 4-17 through 4-24 changes in compressive strength for different mixes with different curing temperatures are shown. First four figures show the data for compressive strength at 15 days age, while the other figures show compressive strength for specimens with 28 days age.

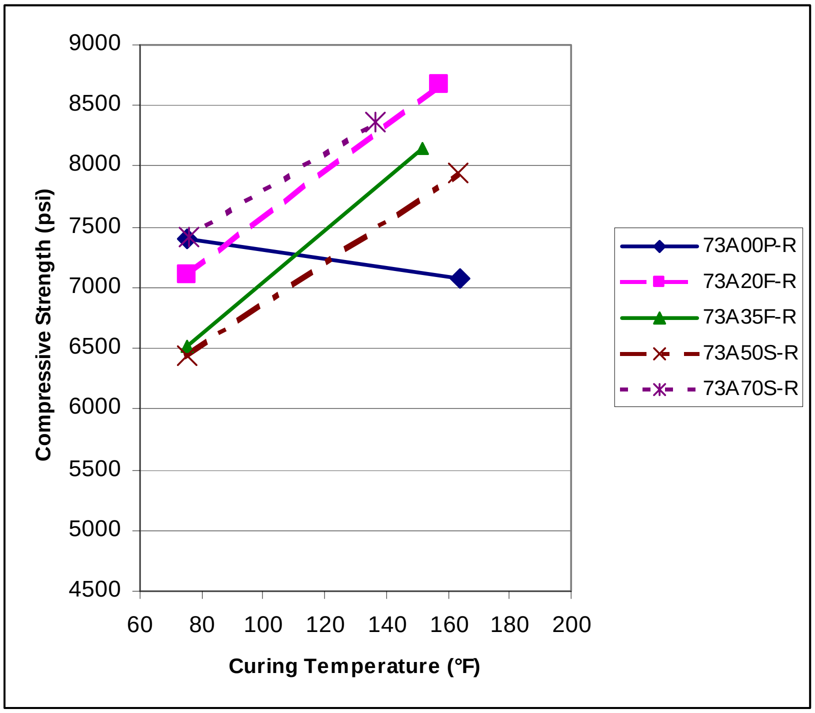


Figure 4-17 Compressive Strength for Concrete Specimens with Cement A and 73°F Placing Temperature at 15 Days Age

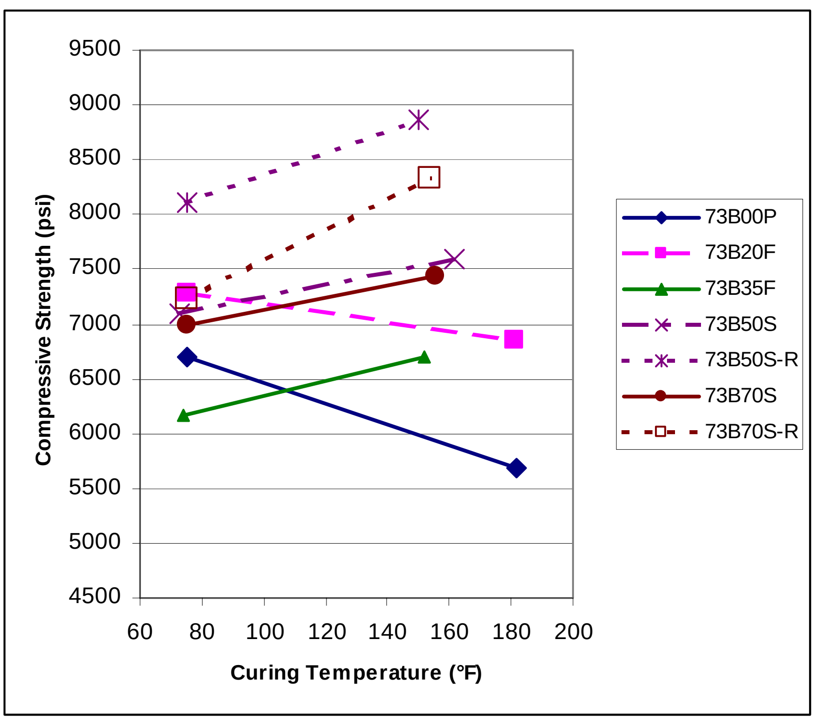


Figure 4-18 Compressive Strength for Concrete Specimens with Cement B and 73°F Placing Temperature at 15 Days Age

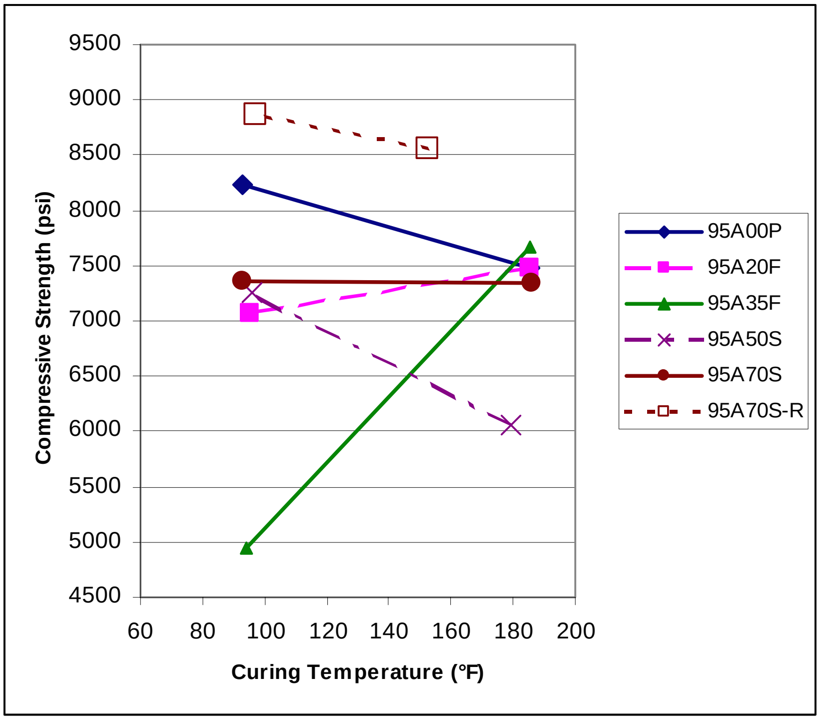


Figure 4-19 Compressive Strength for Concrete Specimens with Cement A and 95°F Placing Temperature at 15 Days Age

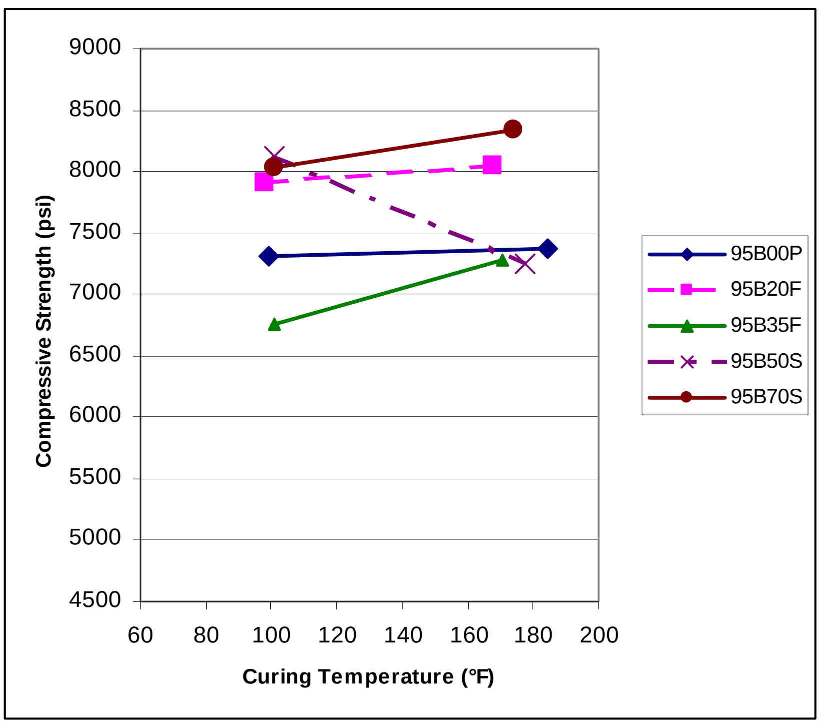


Figure 4-20 Compressive Strength for Concrete Specimens with Cement B and 95° F Placing Temperature at 15 Days Age

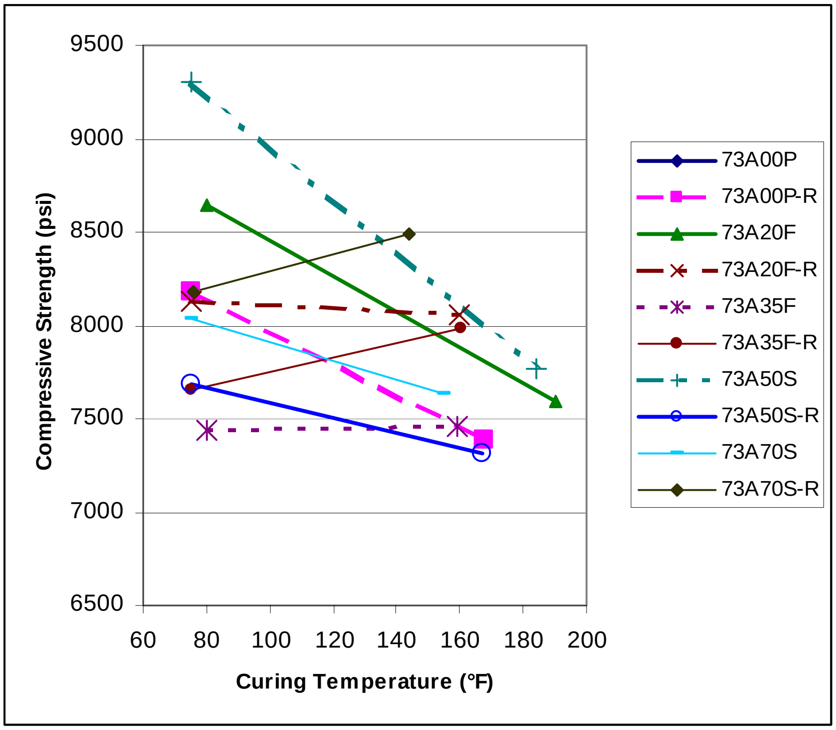


Figure 4-21 Compressive Strength for Concrete Specimens with Cement A and 73° F Placing Temperature at 28 Days Age

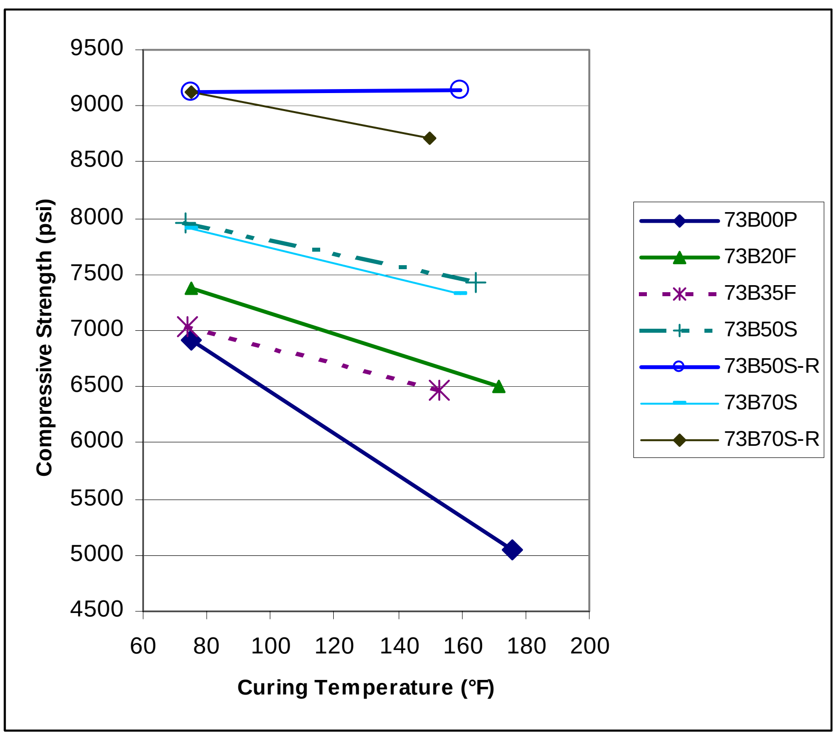


Figure 4-22 Compressive Strength for Concrete Specimens with Cement B and 73° F Placing Temperature at 28 Days Age

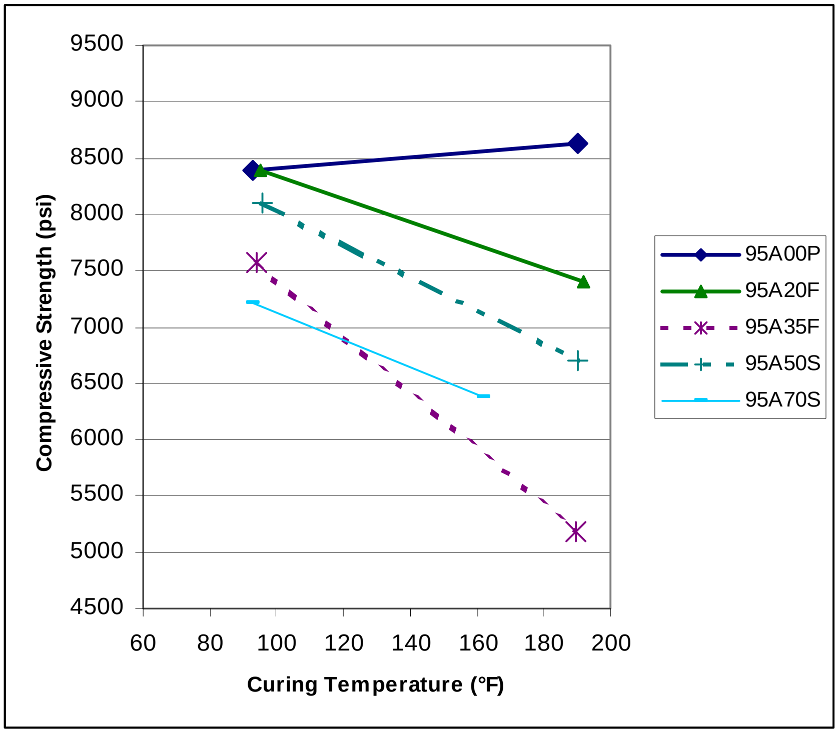


Figure 4-23 Compressive Strength for Concrete Specimens with Cement A and 95° F Placing Temperature at 28 Days Age

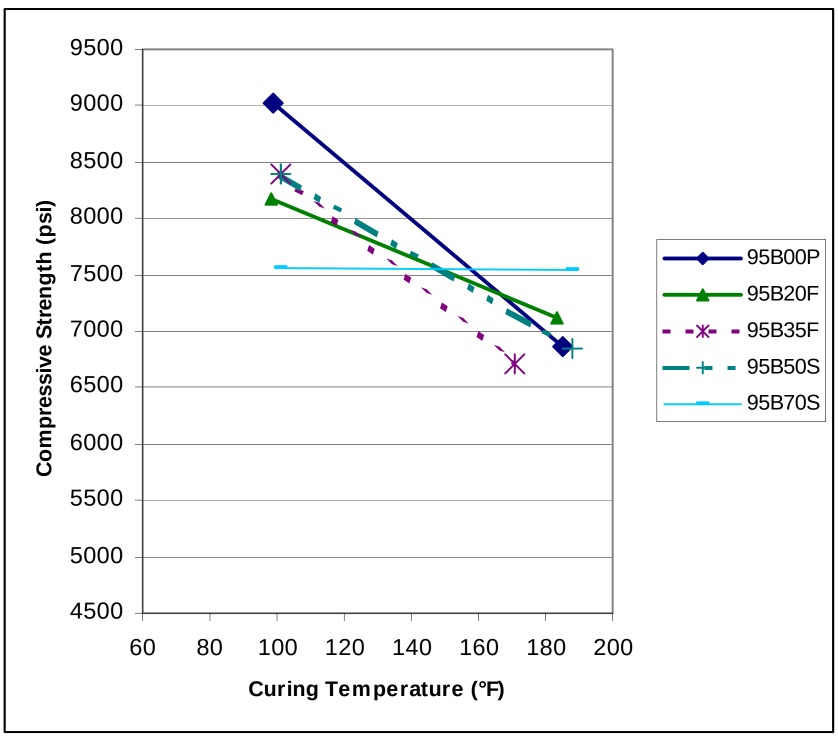


Figure 4-24 Compressive Strength for Concrete Specimens with Cement B and 95° F Placing Temperature at 28 Days Age

These figures show that at 15 days age, mixes with pozzolanic materials have higher compressive strength if they are cured in high temperatures while at 28 days age they show a lower strength compared to room temperature cured specimens.

In Table 4-11 the compressive strength of test mixes for moisture room cured samples are shown. Each number is the average of three compressive strength test results for 6″x12″ cylinder specimens that have been cured for 28 days.

Table 4-11 Compressive Strength at 28 Days for Moisture Room Cured Concrete Specimens

Mix	Compressive Strength (psi)	Age (Day)	Mix	Compressive Strength (psi)	Age (Day)
73A00P	8945	28	95A70S	8762	28
73A20F	7689	28	95B00P	7989	28
73A35F	7389	28	95B20F	8175	28
73A50S	8691	28	95B35F	7229	28
73A70S	8580	28	95B50S	8720	28
73B00P	7025	28	95B70S	8117	28
73B20F	7987	28	73A00P-R	8073	28
73B35F	6767	28	73A20F-R	7556	28
73B50S	7657	28	73A35F-R	9462	28
73B70S	7921	28	73A50S-R	8117	28
95A00P	8885	28	73A70S-R	7573	28
95A20F	8313	28	73B50S-R	8361	28
95A35F	6556	28	73B70S-R	9112	28
95A50S	7799	28	95A70S-R	7784	28

CHAPTER 5
DISCUSSION OF RESULTS

Introduction

In this chapter the results of the adiabatic temperature rise, the heat of hydration, thermal diffusivity and compressive strength tests are summarized and compared for different mixes. The effects of placing temperature and pozzolan content on properties of concrete within the scope of this research project are studied. Also a group of equations are introduced to calculate the modification factor for the replacement of pozzolans in concrete mixes with different placing temperatures.

Adiabatic Temperature Rise Curves

In total, 20 adiabatic temperature rise tests were run. As explained in Chapter 4, some of the tests were repeated. The main reason to repeat the tests was a change in the test procedure to prevent the effect of heat from Hydration Chamber heaters on specimen temperature rise. After reviewing the test results and taking into consideration the suggestions made by Sure Cure system's manufacturer, it can be said that heat from heaters can affect temperature rise when the rate of temperature rise in concrete is less than 5ºF/day. This stage normally occurs when concrete was three days old. Therefore it would be reasonable to average the temperature rise data that have been repeated when the temperature rise rate is more than 5ºF/day and use the data from that point through day 14 from the second run of the test for a specific mix. Using this method the final numbers for temperature rise data for each mix was determined.

In Tables 5-1 to 5-4, the adiabatic temperature rise data for test mixes are presented. The temperature rise for repeated mixes is the average of two tests.

Table 5-1 Adiabatic Temperature Rise Data for Concrete with Cement A and 73ºF Placing Temperature

Time (Day)	Concrete with Cement A (73ºF Placing Temperature)				
	Plain Cement	20% Fly Ash	35% Fly Ash	50% Slag	70% Slag
	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)
0.0	0.0	0.0	0.0	0.0	0.0
0.5	22.0	13.6	5.4	12.2	12.1
1.0	48.8	34.2	29.1	31.7	27.3
1.5	65.1	48.6	45.4	54.2	49.3
2.0	77.7	61.1	54.3	72.5	62.8
2.5	83.0	69.1	60.0	81.6	68.8
3.0	87.5	75.6	65.5	85.2	71.6
3.5	89.4	80.4	70.0	87.0	73.4
4.0	91.0	82.4	72.9	88.0	74.7
4.5	92.5	83.6	75.0	88.9	75.4
5.0	93.6	84.2	76.6	89.4	75.8
5.5	94.5	84.8	77.8	89.8	76.1
6.0	95.5	85.4	78.8	89.8	76.4
6.5	96.2	85.4	79.5	89.9	76.4
7.0	97.0	85.7	80.3	89.7	76.5
7.5	97.5	85.7	80.9	89.8	76.6
8.0	98.0	85.6	81.0	90.0	76.7
8.5	98.0	85.8	81.3	90.0	76.9
9.0	98.1	85.8	81.1	90.1	77.0
9.5	98.1	85.9	81.2	90.2	77.1
10.0	98.1	85.9	81.3	90.2	77.3
10.5	98.1	85.9	81.4	90.2	77.4
11.0	98.2	85.9	81.4	90.3	77.5
11.5	98.2	86.0	81.5	90.5	77.6
12.0	98.2	86.0	81.6	90.4	77.6
12.5	98.3	86.0	81.6	90.4	77.6
13.0	98.4	86.0	81.8	90.4	77.6
13.5	98.5	86.0	81.7	90.4	77.6
14.0	98.5	86.0	81.6	90.4	77.6

Table 5-2 Adiabatic Temperature Rise Data for Concrete with Cement B and 73ºF Placing Temperature

Time (Day)	Concrete with Cement B (73ºF Placing Temperature)				
	Plain Cement	20% Fly Ash	35% Fly Ash	50% Slag	70% Slag
	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)
0.0	0.0	0.0	0.0	0.0	0.0
0.5	28.9	15.5	25.1	22.0	14.8
1.0	69.9	61.1	45.4	48.6	36.2
1.5	80.1	74.8	54.2	67.8	59.1
2.0	84.4	82.7	60.0	75.3	67.2
2.5	87.7	87.4	64.1	79.6	71.4
3.0	90.6	88.3	67.6	82.0	74.1
3.5	92.9	90.0	70.0	84.0	75.7
4.0	94.7	91.4	71.8	85.6	77.1
4.5	96.1	92.4	73.0	86.9	78.3
5.0	97.4	93.2	74.0	87.9	79.4
5.5	98.6	93.8	74.5	88.6	79.9
6.0	99.7	94.4	75.1	89.1	80.2
6.5	100.7	94.7	75.4	89.4	80.4
7.0	101.5	95.3	75.8	89.7	80.4
7.5	102.3	95.8	76.0	90.0	80.4
8.0	103.0	96.1	76.2	90.3	80.4
8.5	103.5	96.3	76.5	90.4	80.4
9.0	104.0	96.4	76.6	90.6	80.4
9.5	104.3	96.5	76.8	90.7	80.4
10.0	104.6	96.6	77.0	90.8	80.4
10.5	104.8	96.6	77.1	90.8	80.4
11.0	105.1	96.6	77.3	90.9	80.4
11.5	105.3	96.8	77.4	90.9	80.4
12.0	105.5	96.8	77.5	91.0	80.4
12.5	105.7	96.9	77.6	91.0	80.4
13.0	105.8	97.0	77.5	91.0	80.4
13.5	105.9	97.0	77.7	91.0	80.4
14.0	106.0	97.0	78.3	91.0	80.4

Table 5-3 Adiabatic Temperature Rise Data for Concrete with Cement A and 95ºF Placing Temperature

Time (Day)	Concrete with Cement A (95ºF Placing Temperature)				
	Plain Cement	20% Fly Ash	35% Fly Ash	50% Slag	70% Slag
	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)
0.0	0.0	0.0	0.0	0.0	0.0
0.5	17.8	9.0	4.8	19.3	4.3
1.0	61.0	40.8	26.5	54.5	16.8
1.5	77.9	63.2	47.0	82.7	49.5
2.0	87.4	77.0	60.8	91.0	57.7
2.5	91.0	85.5	75.5	92.2	60.5
3.0	92.2	88.9	80.9	92.2	62.1
3.5	92.4	90.1	82.9	92.2	63.6
4.0	92.4	90.3	84.0	92.2	64.4
4.5	92.4	90.6	84.4	92.3	65.4
5.0	92.4	90.6	85.0	92.3	66.1
5.5	92.4	90.6	85.0	92.3	66.7
6.0	92.4	90.6	85.0	92.3	67.1
6.5	92.4	90.6	85.0	92.3	67.5
7.0	92.4	90.6	85.0	92.3	67.9
7.5	92.4	90.6	85.0	92.3	68.2
8.0	92.4	90.6	85.0	92.3	68.6
8.5	92.4	90.6	85.0	92.3	68.9
9.0	92.4	90.6	85.0	92.3	69.0
9.5	92.4	90.6	85.0	92.3	69.3
10.0	92.4	90.6	85.0	92.3	69.4
10.5	92.4	90.6	85.0	92.3	69.6
11.0	92.4	90.6	85.0	92.3	69.8
11.5	92.4	90.6	85.0	92.3	69.9
12.0	92.4	90.6	85.0	92.3	70.1
12.5	92.4	90.6	85.0	92.3	70.1
13.0	92.4	90.6	85.0	92.3	70.4
13.5	92.4	90.6	85.0	92.3	70.5
14.0	92.4	90.6	85.0	92.3	70.8

Table 5-4 Adiabatic Temperature Rise Data for Concrete with Cement B and 95ºF Placing Temperature

Time (Day)	Concrete Cement B (95ºF Placing Temperature)				
	Plain Cement	20% Fly Ash	35% Fly Ash	50% Slag	70% Slag
	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)	Temp. Rise (ºF)
0.0	0.0	0.0	0.0	0.0	0.0
0.5	2.6	56.8	50.4	58.6	45.0
1.0	72.2	72.9	61.9	76.7	69.4
1.5	84.2	77.4	67.9	80.7	73.9
2.0	85.6	77.6	68.9	81.8	76.3
2.5	85.7	77.6	69.2	82.0	77.9
3.0	86.0	77.6	69.3	82.0	78.9
3.5	86.0	77.6	69.6	82.0	79.5
4.0	86.0	77.6	69.9	82.0	79.9
4.5	86.0	77.6	70.0	82.0	79.9
5.0	86.0	77.6	70.0	82.0	79.9
5.5	86.0	77.6	70.0	82.0	79.9
6.0	86.0	77.6	70.0	82.0	79.9
6.5	86.0	77.6	70.0	82.0	79.9
7.0	86.0	77.6	70.0	82.0	79.9
7.5	86.0	77.6	70.0	82.0	79.9
8.0	86.0	77.6	70.0	82.0	79.9
8.5	86.0	77.6	70.0	82.0	79.9
9.0	86.0	77.6	70.0	82.0	79.9
9.5	86.0	77.6	70.0	82.0	79.9
10.0	86.0	77.6	70.0	82.0	79.9
10.5	86.0	77.6	70.0	82.0	79.9
11.0	86.0	77.6	70.0	82.0	79.9
11.5	86.0	77.6	70.0	82.0	79.9
12.0	86.0	77.6	70.0	82.0	79.9
12.5	86.0	77.6	70.0	82.0	79.9
13.0	86.0	77.6	70.0	82.0	79.9
13.5	86.0	77.6	70.0	82.0	79.9
14.0	86.0	77.6	70.0	82.0	79.9

Comparison between Temperature Rise Curves

Effect of Percentage of Pozzolans

It is generally believed that replacing cement with pozzolan has a reducing effect on the peak temperature of concrete. However, the amount of reduction has reported differently in various sources. In the first part of this section, the adiabatic temperature rise curves from this study are presented. In the second part, the percentage of reduction in the temperature at 14 days is calculated and changes in the percentage of reduction in the temperature are shown.

Adiabatic Temperature Rise Curves

In figures 5-1 and 5-2, the effect of replacing cement with fly ash is shown. As it can be seen, adding pozzolan to the mix reduces the peak temperature. The amount of reduction, however, has been different for the mixes with cements from the different sources and also different placing temperatures. A comparison between Figures 5-1 and 5-2 shows that in mixes with lower placing temperature, the addition of pozzolan had a larger reducing effect on the peak temperature. In one case (cement A with 95ºF placing temperature) replacing cement with 20% fly ash did not have a significant effect on the peak temperature. The other observation is that replacing more fly ash did not result in the same amount of reduction in the peak temperature.

In Figures 5-3 and 5-4, the effect of replacing cement with slag is shown. The same trend as replacement of fly ash can be seen in these figures. In Figure 5-4 it can be seen that replacing 50% of cement with slag did not have a reducing effect on the peak temperature, while using 70% slag reduced the peak temperature significantly. For this cement and with 95ºF placing temperature, it could be said that replacing up to 50% of cement with slag does not lead to a reduction in the peak temperature. The replacement of cement with slag in the mixes with cement B did not show a significant reduction in the peak temperature either. From these observations it could be said that slag in high temperature placing conditions does not have a significant reducing effect on the peak temperature.

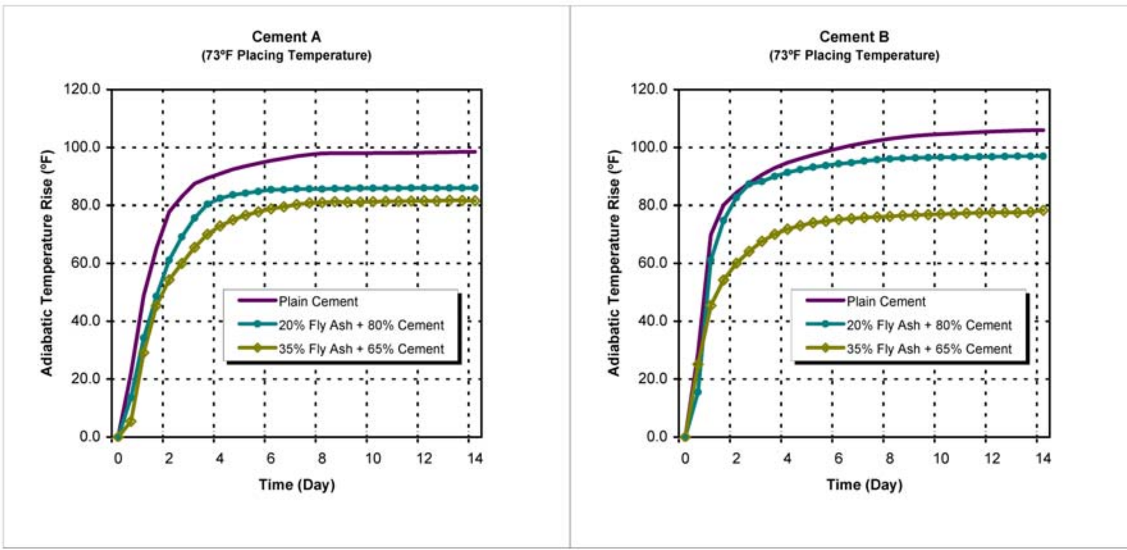


Figure 5-1 Effect of Replacing Cement With Fly Ash on Adiabatic Temperature Rise (73ºF Placing Temperature)

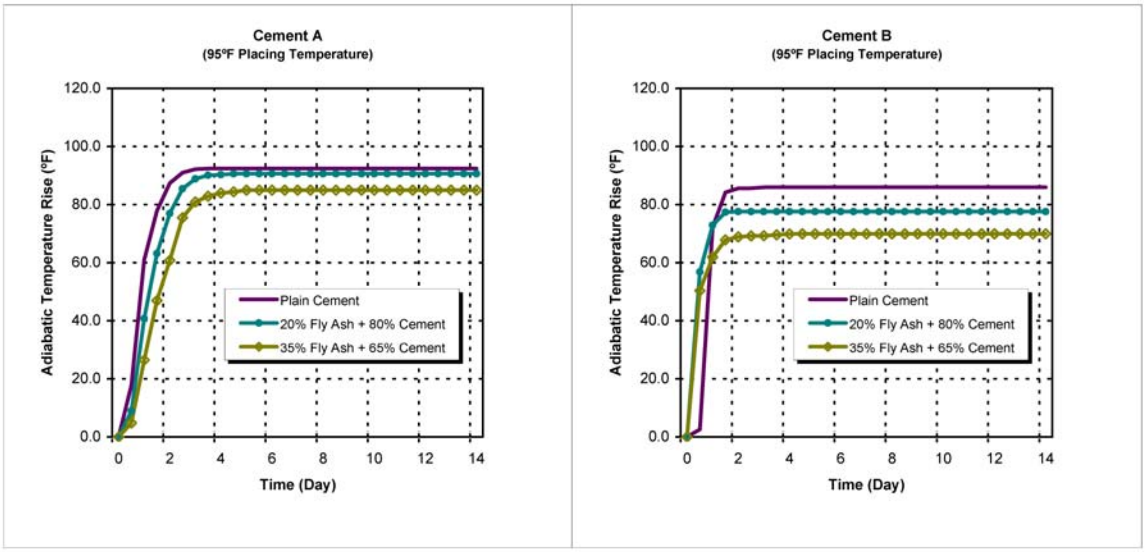


Figure 5-2 Effect of Replacing Cement With Fly Ash on Adiabatic Temperature Rise (95ºF Placing Temperature)

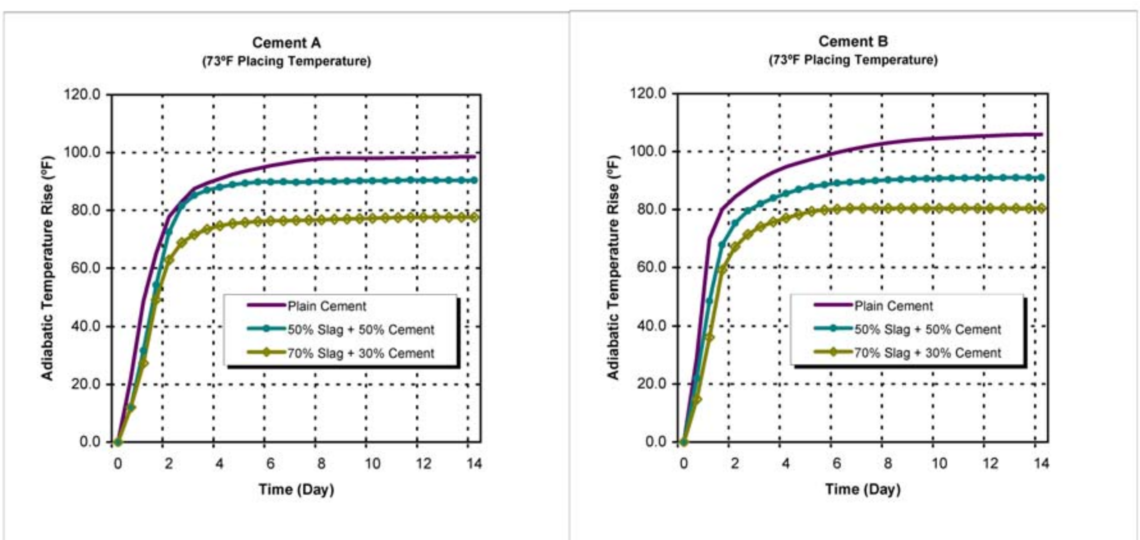


Figure 5-3 Effect of Replacing Cement With Slag on Adiabatic Temperature Rise (73ºF Placing Temperature)

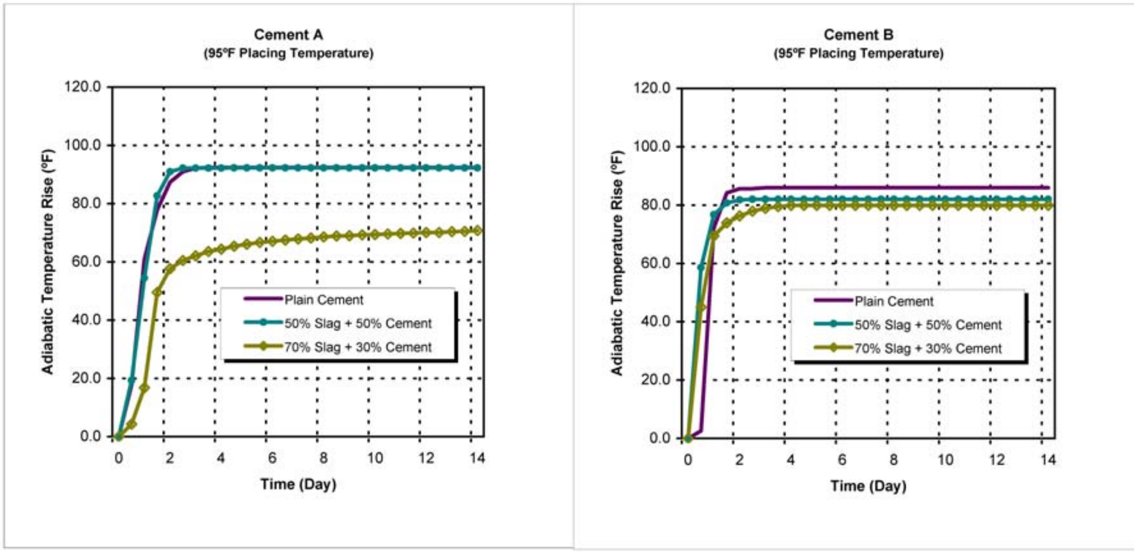


Figure 5-4 Effect of Replacing Cement With Slag on Adiabatic Temperature Rise (95ºF Placing Temperature)

Reduction in the Peak Temperature

The percentage of reduction in the peak temperature (P_{rp}) for each mix with pozzolan was calculated using the following equation.

$P_{rp}=(T_{cm} - T_{pm})/ T_{cm}$ Equation 5-1

Where:

P_{rp} : Percentage of reduction in the peak temperature,

T_{cm} : Peak temperature of the control mix (the mix with plain cement),

T_{pm} : Peak temperature of the mix with pozzolan (20% fly ash, 35% fly ash, 50% slag, 70% slag)

Using P_{rp} eliminates the possible errors in the peak temperatures recorded by the Sure Cure system because all the mixes were made and tested in the same condition. The ratio between peak temperatures should therefore be correct even if an error exists in the peak temperature number. In 5-5, P_{rp} for different mixes is shown. The numbers could be averaged because both cements A and B are Type II AASHTO cement.

Table 5-5 Effect of Pozzolans on the Peak Temperature of Concrete

Cement Source	Placing Temperature	% Reduction in Peak Temperature (at 14 days)			
		20% Fly Ash	35% Fly Ash	50% Slag	70% Slag
A	73ºF	12.7	17.2	8.2	21.2
B	73ºF	8.5	26.1	14.1	24.1
Average for Cements A & B		**10.6**	**21.7**	**11.2**	**22.7**
A	95ºF	1.9	8.0	0.1	23.4
B	95ºF	9.7	18.6	4.6	7.0
Average for Cements A & B		**5.8**	**13.3**	**2.4**	**15.2**

In Figures 5-5 and 5-6, the effect of using pozzolans on temperature at different ages of concrete is shown. The percent of reduction in temperature (P_r) was calculated as:

$P_r = (T_c - T_p) / T_c$ Equation 5-2

Where:

P_r : Percentage of reduction in the temperature,

T_c : Temperature of the control mix (the mix with plain cement),

T_p : Temperature of the mix with pozzolan (20% fly ash, 35% fly ash, 50% slag, 70% slag)

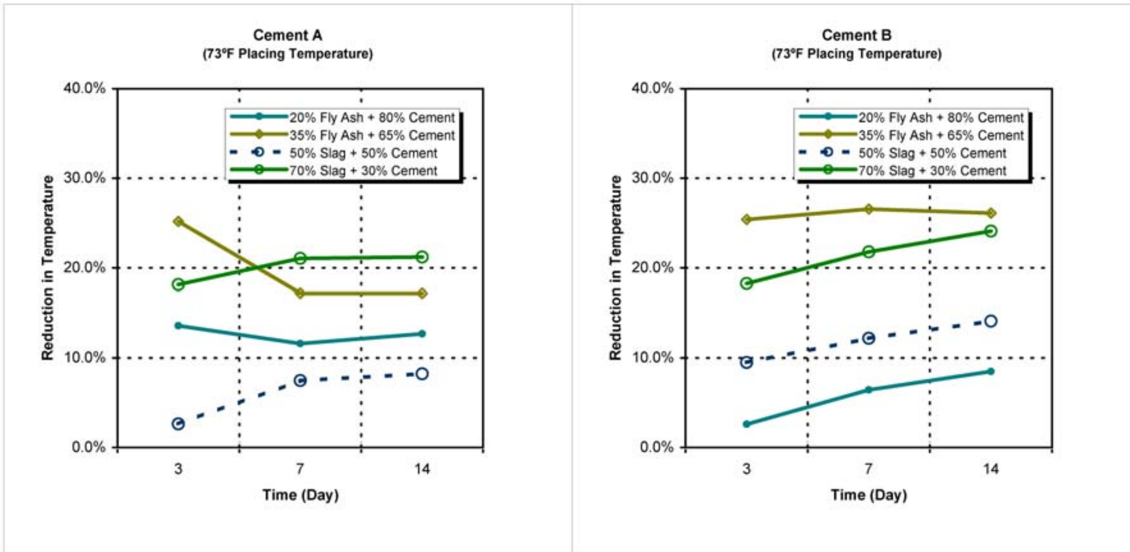


Figure 5-5 P_r for Mixes with 73ºF Placing Temperature

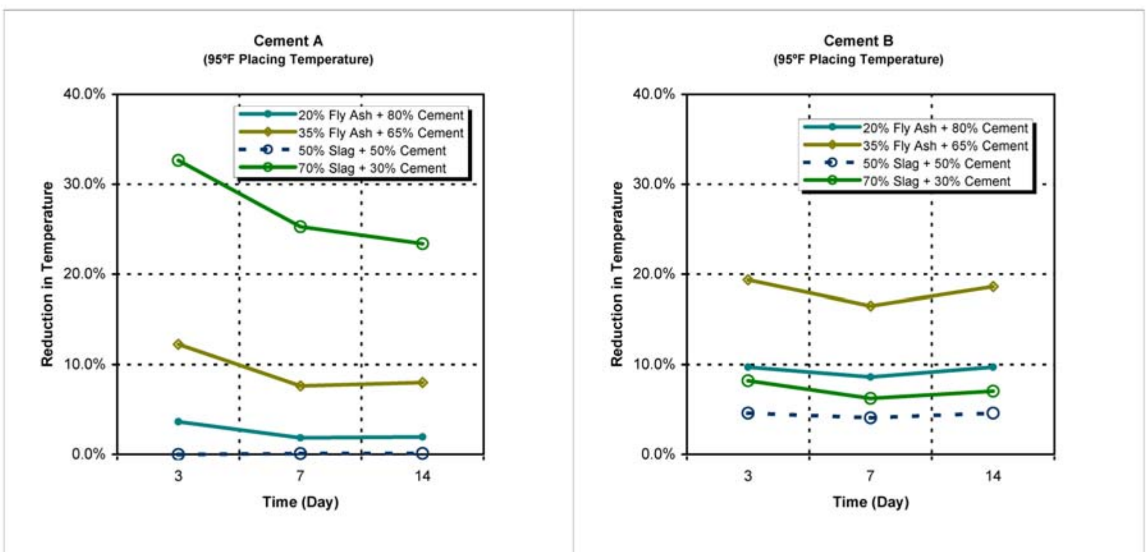


Figure 5-6 P_r for Mixes with 95ºF Placing Temperature

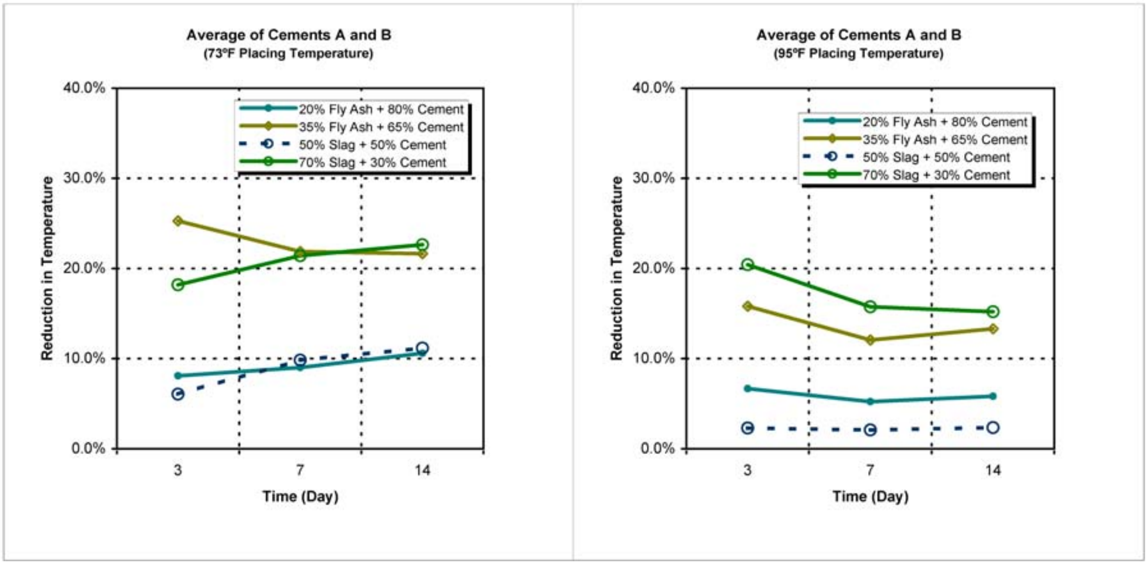


Figure 5-7 Average P_r for Mixes with 73ºF and 95ºF Placing Temperatures

Figure 5-7 shows the average P_r for mixes with different placing temperatures. The first conclusion from this figure is that fly ash has a stronger effect on the temperature reduction. In other words, a smaller percentage of fly ash replacement has the same effect as that of a larger percentage of slag replacement. As it is shown, 20% fly ash replacement has almost the same effect as 50% slag replacement and, in a similar fashion, 35% fly ash replacement reduces the temperature the same as does 70% slag replacement. Also it can be said that placing temperature has an effect on the amount of reduction in the temperature. A higher placing temperature weakens the reducing effect of pozzolans on concrete temperature.

The data in Table 5-5 can be divided to four groups:

1. Average of cements A and B with fly ash and 73ºF placing temperature,
2. Average of cements A and B with fly ash and 95ºF placing temperature,
3. Average of cements A and B with slag and 73ºF placing temperature,
4. Average of cements A and B with slag and 95ºF placing temperature,

A second degree polynomial to calculate P_{rp} for any given percentage of cement replacement by pozzolan for each group can be determined if three points of the equation are known. For each group, the data of the three points are known: two points in Table 5-5 and the third point would be P_{rp}= 0 in the mix with plain cement.

The general format of the equation is:

$P_{rp} = a(R_p)^2 + b(R_p) + c$ Equation 5-3

Where:

P_{rp} : Percentage of reduction in the pick temperature,

a, *b* and *c*: Constants,

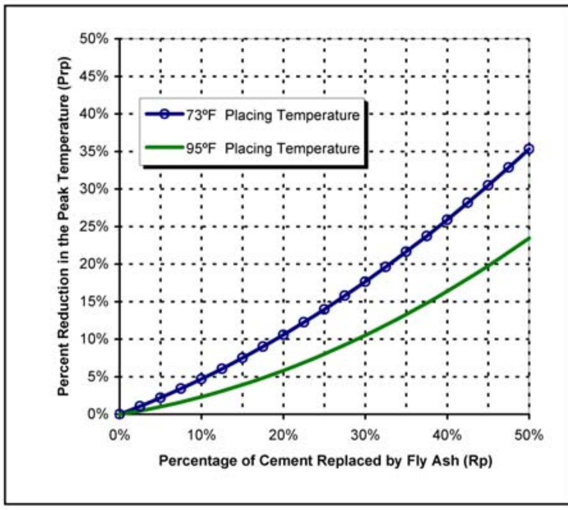
R_p : Percentage of pozzolan in the mix.

a, *b* and *c* constants were determined by solving the equation for the known three points. The amount of contents for each data group are shown in

Table 5-6 Constant Factors of P_{rp} Equation

No.	Data Group Description	Constants		
		a	*b*	*c*
1	Cement + Fly Ash, 73ºF Placing Temperature	0.59	0.41	0.00
2	Cement + Fly Ash, 95ºF Placing Temperature	0.59	0.17	0.00
3	Cement + Slag, 73ºF Placing Temperature	0.50	-0.03	0.00
4	Cement + Slag, 95ºF Placing Temperature	0.85	-0.38	0.00

The relationship between P_{rp} , $R_{p,}$ and Different Placing Temperatures is shown in Figure 5-8. As it can be seen, higher placing temperature has a smaller reducing effect on the peak temperature.



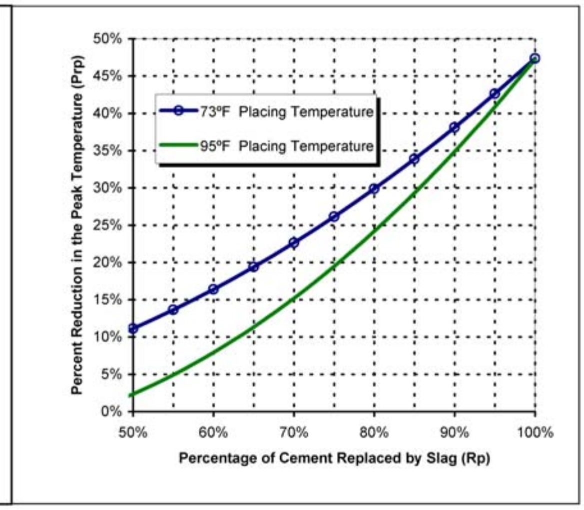


Figure 5-8 The Relationship Between P_{rp}, $R_{p,}$ and Different Placing Temperatures

As suggested by ACI committee 207, the adiabatic temperature rise curves must be modified for mixes with pozzolans. One way is to calculate equivalent cement content (C_{eq}) and use it to determine a modification factor (k_c) for the adiabatic temperature rise curves published in the ACI 207.1R and ACI 207.2R reports. It is also possible to develop the adiabatic temperature rise curve for a mix with only plain cement (base mix) and then apply k_c to generate the adiabatic temperature rise curve for any given mix with different pozzolan contents. C_{eq} is calculated as:

$C_{eq} = C_c + \alpha_p C_p$ Equation 5-4

Where:

C_{eq} : Equivalent Cement Content

C_c : Cement content of the mix (lb/yd^3)

α_p : Pozzolan modification factor

C_p : Pozzolan content of the mix (lb/yd^3)

And

$k_c = C_{eq} / C_b$ Equation 5-5

Where:

k_c : Adiabatic temperature rise curve modification factor

C_b : Cement content of the base mix (lb/yd^3)

The relationship between α_p and P_{rp} is determined as follows:

P_{rp}=($T_{cm} - T_{pm}$)/ $T_{cm} \Rightarrow P_{rp}$= 1 - T_{pm}/ T_{bm} Equation 5-6

According to ACI 207.1R-96, there is a linear relationship between the cementitious material content and the peak temperature so:

$T_{pm}/ T_{cm} = (C_c + \alpha_p C_p)/ C_t$ Equation 5-7

Where:

C_t : Cement content of the control mix which is equal to the summation of the amounts of C_c and C_p in lb/yd^3 (Ct = Cc + Cp)

Therefore:

$T_{pm}/ T_{cm} = (C_c + \alpha_p C_p)/ (C_c + C_p)$ Equation 5-8

Equations 5-7 and 5-8 $\Rightarrow P_{rp} = 1 - (C_c + \alpha_p C_p)/ (C_c + C_p) \Rightarrow P_{rp} = C_p(1- \alpha_p)/(C_c + C_p) \Rightarrow$

$(1- \alpha_p) = P_{rp}(C_c + C_p)/ C_p \Rightarrow (1- \alpha_p) = P_{rp}(C_c /C_p) + P_{rp}$ Equation 5-9

$R_p = C_p / (C_c + C_p) \Rightarrow C_c/ C_p = 1/ R_p - 1$ Equation 5-10

Combining Equations 5-9 and 5-10 results:

$(1- \alpha_p) = P_{rp} (1/ R_p - 1) + P_{rp} \Rightarrow \alpha_p = 1- P_{rp}/ R_p$ Equation 5-11

From Equations 5-3 and 5-11 it is concluded that:

$$\alpha_p = 1 - a(R_p) - b - c/R_p \qquad \text{Equation 5-12}$$

Using the constant factors from Table 5-6 the equations to calculate α_p would be as follows:

For mixes with fly ash and 73ºF placing temperature: $\alpha_p = 0.59 - 0.59R_p$

For mixes with fly ash and 95ºF placing temperature: $\alpha_p = 0.83 - 0.59R_p$

For mixes with Slag and 73ºF placing temperature: $\alpha_p = 1.03 - 0.50R_p$

For mixes with Slag and 95ºF placing temperature: $\alpha_p = 1.38 – 0.85R_p$

Figure 5-9 shows the graphic presentation of the above equations. It can be seen that lower placing temperature leads to a lower pozzolan modification factor. A lower pozzolan modification factor is more desirable while pozzolans are added to concrete. Also it is shown that fly ash and slag do not have similar effect on the pozzolan modification factor.

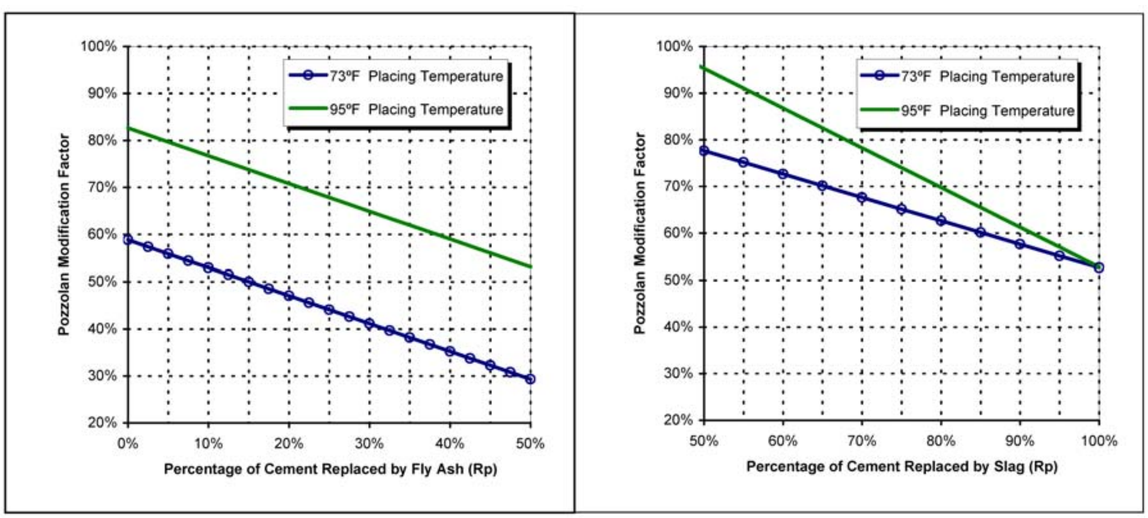


Figure 5-9 Relationship between α_p and R_p

Effect of Placing Temperature

In the previous section, effects of placing temperature on the peak temperature in combination with the type of pozzolan are shown. As it is shown in Figures 5-10 to 5-14, all the mixes with high placing temperature (95°F) reached the peak temperature in a shorter time compared to the mixes with low placing temperature (73°F). This observation is consistent with the curves in ACI 207.2R-96. It is believed that higher initial temperature accelerates the cement hydration as well as the pozzolan hydration.

As it can be seen in Figures 5-10 to 5-14, it is not possible to conclude if a higher placing temperature results in a higher peak temperature. Some mixes (95A20F, 95A35F and 95A50S) showed a higher peak temperature compared to the similar mixes with low placement temperature. The test results show a stop in temperature rise in all the mixes; therefore the condition has not been fully adiabatic. This fact may affect the peak temperature numbers and cause the different peak temperature in the mixes with different placing temperature.

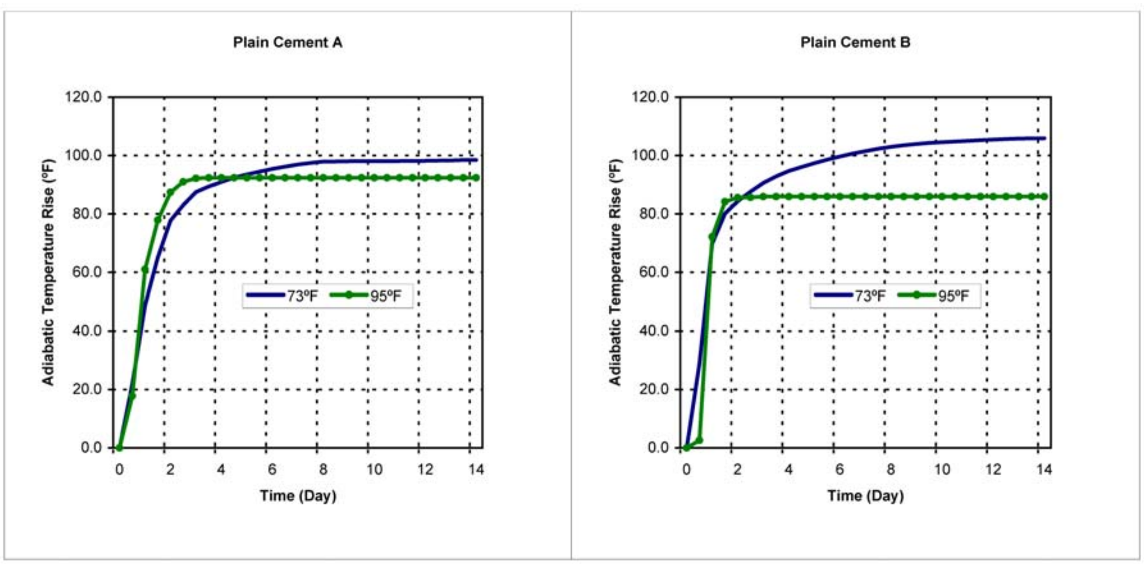


Figure 5-10 Effect of Placing Temperature on Adiabatic Temperature Rise for Mixes with Plain Cement

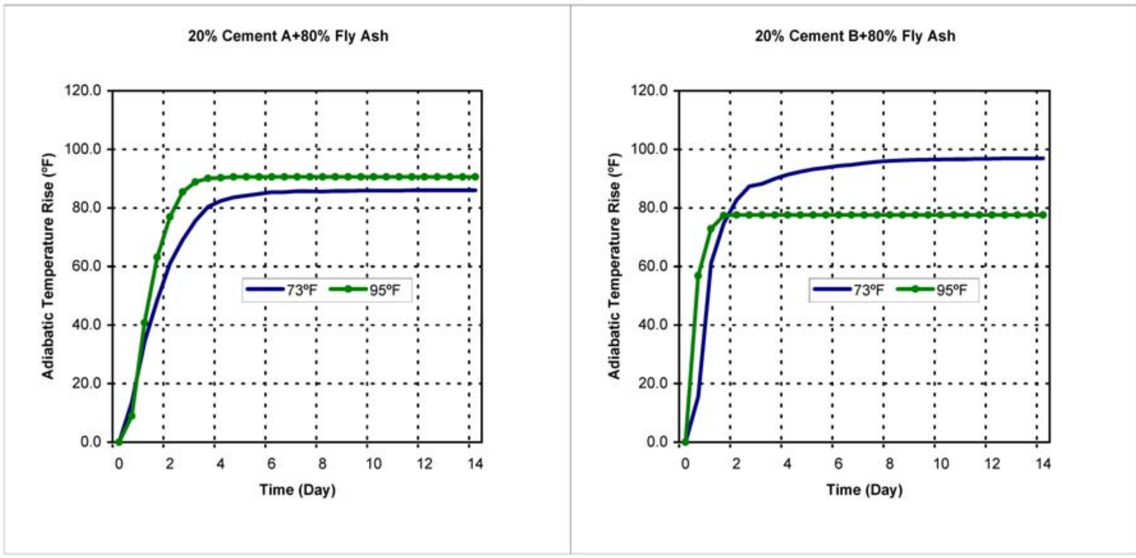


Figure 5-11 Effect of Placing Temperature on Adiabatic Temperature Rise for Mixes with 20% Fly Ash

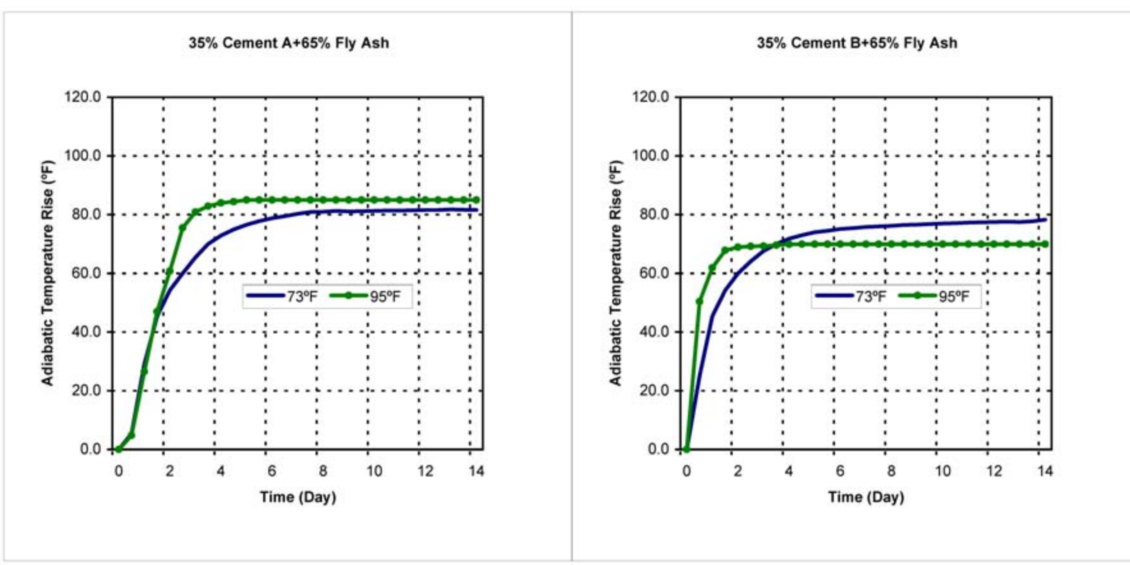


Figure 5-12 Effect of Placing Temperature on Adiabatic Temperature Rise for Mixes with 35% Fly Ash

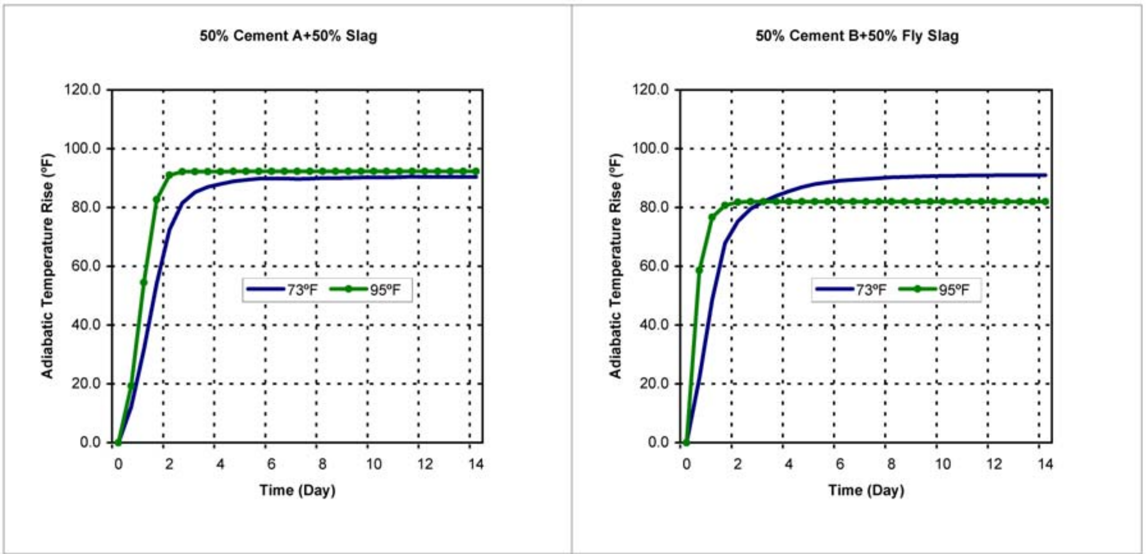


Figure 5-13 Effect of Placing Temperature on Adiabatic Temperature Rise for Mixes with 50% Slag

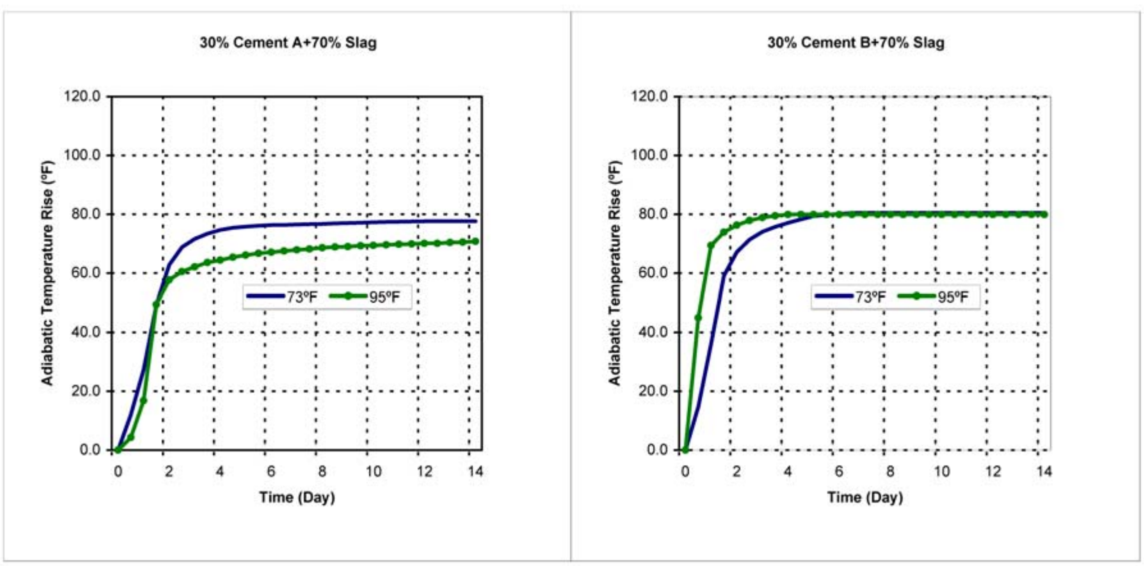


Figure 5-14 Effect of Placing Temperature on Adiabatic Temperature Rise for Mixes with 70% Slag

Comparison Between Field and Laboratory Test Results

It is necessary to compare the laboratory test results to the field temperature recordings to determine how similar is the condition in the Hydration Chambers to the real conditions of the core of a mass concrete element. Based on FDOT requirements, contractors are required to record the temperature rise in any mass concrete element from the time that the element is poured to the time it reaches the peak temperature. The data for the temperature records of the core of the footers A, B, and C of the Bella Vista Bridge was provided by the project administrator and is presented in Table 5-7. All footers have a width and length of 17 feet and are 5 ½ feet in depth.

Table 5-7 Temperature Rise of the Core of the Bella Vista Bridge Footers

Footer A		Footer B		Footer C	
Time (hour)	Temperature Rise (°F)	Time (hour)	Temperature Rise (°F)	Time (hour)	Temperature Rise (°F)
0	0.0	0	0.0	0	0.0
13	34.2	9	26.1	8	27.1
19	45.1	15	35.1	14	36.9
25	50.1	21	39.4	20	51.4
31	64.3	27	54.0	26	66.0
37	75.4	33	58.2	32	80.1
43	76.7	39	65.7	38	85.1
49	78.6	45	71.6	44	86.8
55	79.6	51	74.9	50	83.0
61	**80.3**	57	76.1	56	80.1
67	80.2	**63**	**77.3**	62	85.1
73	79.1	69	75.1	**68**	**86.8**
79	76.2	75	70.5	74	84.3
85	78.5	81	73.1	80	86.4
91	77.3	87	75.2	86	85.6

The data from Table 5-7 is also presented graphically in Figure 5-15. As it is shown all three footings reached the peak temperature approximately 64 hours after placing concrete. The peak temperature for three footings is also close (average 81.5 °F).

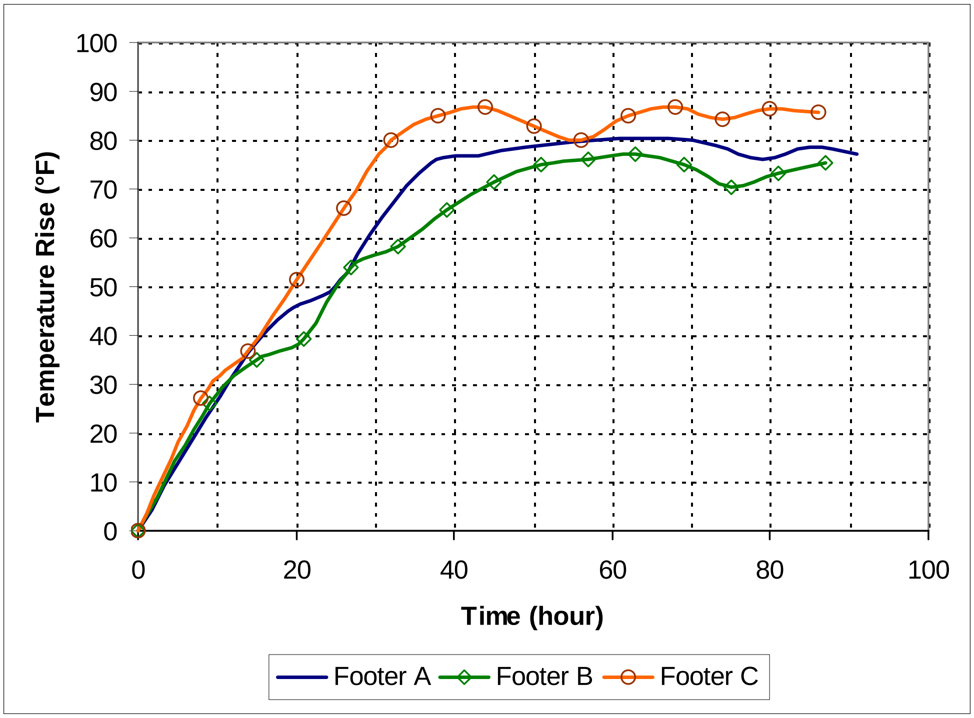


Figure 5-15 Temperature Rise For Footers A, B and C

Footers A and B were placed on April 2nd, 2004, while footer C was placed on April 7th the same year. The same mix with the same materials was used for all the three footings. The mix contained 50% type II cement and 50% slag. As it can be seen in Figure 5-15, some fluctuations have recorded in the temperature rise of footer C. This unexpected behavior may be caused by a systemic error in the recording device.

Samples were taken from the same concrete that was used to pour the footers and were tested by the Sure Cure system to determine the temperature rise curves. In Table 5-8 the adiabatic temperature rise data for laboratory tests are shown. After 64 hours the average sample temperature rise is 78.5°F, which is consistent with field recordings. However, the temperature rises after this time in the laboratory, while it drops in the field. The difference is due to the fact that in the field the condition is semi-adiabatic and therefore concrete looses heat. Whenever the rate of heat loss is larger than the rate of heat gain, the temperature drops.

Table 5-8 Adiabatic Temperature Rise for Samples Taken from the Field

Time (hour)	Temp. Rise (ºF) 1st Test (Site50S-1)*	Temp. Rise (ºF) 2nd Test (Site50S-2)	Time (hour)	Temp. Rise (ºF) 1st Test (Site50S-1)	Temp. Rise (ºF) 2nd Test (Site50S-2)
0.0	0.00	0.00	168.0	-	89.94
12.0	26.00	24.65	180.0	-	90.75
24.0	43.80	41.95	192.0	-	91.23
36.0	62.60	59.50	204.0	-	91.65
48.0	74.03	69.75	216.0	-	91.98
60.0	79.66	75.20	228.0	-	92.10
64.0	**80.62**	**76.33**	240.0	-	92.15
72.0	82.54	78.60	252.0	-	92.17
84.0	84.32	81.10	264.0	-	92.18
96.0	85.58	83.00	276.0	-	92.18
108.0	86.65	84.55	288.0	-	92.18
120.0	87.56	85.95	300.0	-	92.18
132.0	88.29	87.25	312.0	-	92.18
144.0	88.92	88.26	324.0	-	92.18
156.0	-	89.15	336.0	-	92.18

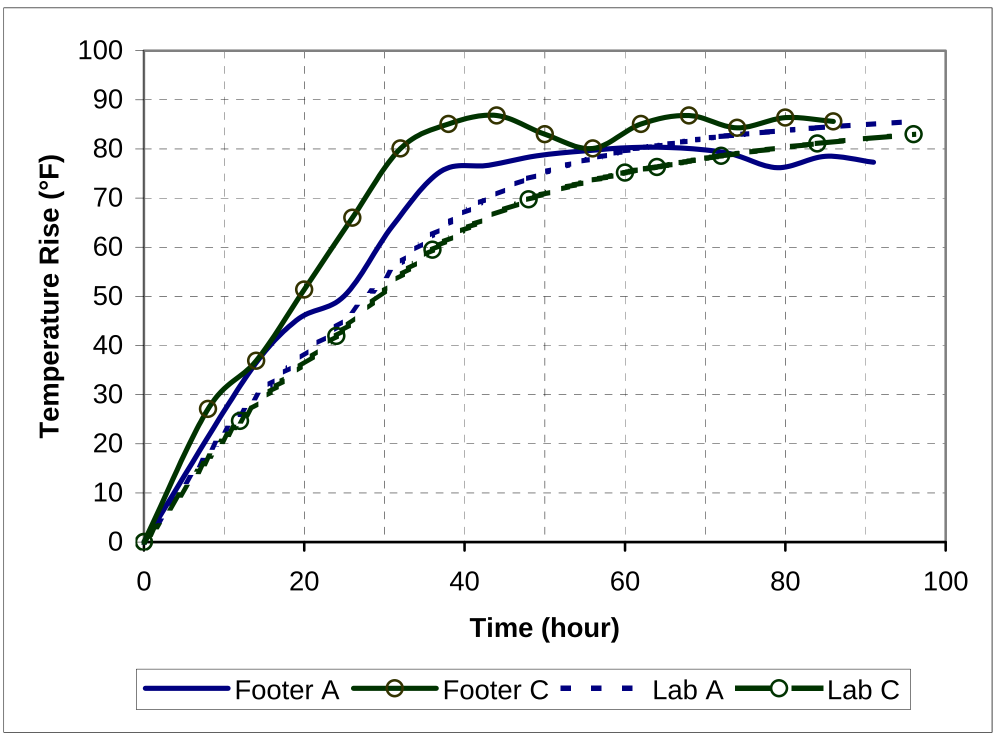


Figure 5-16 Comparison of Temperature Rise Between Field and Laboratory Tests

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* As mentioned before, the data from this test may not be accurate because of the initial equipment malfunction. However, the results of the first and the second test are not significantly different.

The correlation between the field and the laboratory data for the first 96 hours after placing concrete was 0.978 and 0.966 for footers A and C respectively. The high correlation between the filed and the laboratory results shows that the results from the Sure Cure system can be considered as an acceptable simulation of a mass concrete pour.

Comparison between the Test Results and ACI Curves

In ACI 207.1R-96 (Same as Figure 2-1) the adiabatic temperature rise curves of concrete for concrete samples with 376 lb/yd^3 of cement from different types are shown. The second column in Table 5-9 shows the data from the ACI curve for low placing temperature. The third column presents the modified ACI data for the higher cement content of the test mixes. Test results for concrete mixes containing plain cements A and B were compared to ACI curve. Cement A showed a lower temperature rise during the test compared to the ACI curve. Lower than average heat of hydration of the cement A may contribute to this fact. Cement B showed a higher temperature rise in the first three days of the test. After that, the temperature rise of cement B was lower than the ACI curve. The difference for both cements was ascending during the test. This is due to the fact that the condition in the Hydration Chambers is not completely adiabatic when the temperature gain of the concrete is lower than 1.5 to 2.0 °F per day. Ever ascending ACI curves shows that these curves were developed in a complete adiabatic condition.

Table5-9 Comparison Between ACI Curves and Test Results

Time (day)	Temperature Rise (°F)		Cement A (73°F Placing Temperature)		Cement B (73°F Placing Temperature)	
	ACI Type II 376 lb/yd^3	ACI Type II 760 lb/yd^3	Temperature Rise (°F)	Difference Comparing to ACI Curve	Temperature Rise (°F)	Difference Comparing to ACI Curve
0	0.0	0.0	0.0	0.0%	0.0	0.0%
1	30.8	62.2	48.8	21.6%	69.9	-12.3%
2	40.2	81.3	77.7	4.4%	84.4	-3.8%
3	44.7	90.4	87.5	3.2%	90.6	-0.2%
4	47.1	95.2	91.0	4.4%	94.7	0.5%
5	49.5	100.0	93.6	6.4%	97.4	2.6%
6	50.3	101.6	95.5	6.0%	99.7	1.9%
7	51.6	104.3	97.0	7.0%	101.5	2.6%
8	52.6	106.4	98.0	7.9%	103.0	3.2%
9	53.2	107.5	98.1	8.7%	104.0	3.2%
10	53.7	108.5	98.1	9.6%	104.6	3.6%
11	54.2	109.6	98.2	10.4%	105.1	4.1%
12	55.0	111.2	98.2	11.7%	105.5	5.1%
13	55.3	111.7	98.4	11.9%	105.8	5.3%
14	55.8	112.8	98.5	12.7%	106.0	6.0%
Correlation With ACI Curve			0.991		0.994	

Heat of Hydration

The results from the heat of hydration test are to a large extent consistent with the concrete temperature rise tests. As it is shown in Table 5-10, fly ash has a stronger effect on the reduction of the heat of hydration. The test results showed that replacing cement with 20% fly ash reduces the heat of hydration at 7 days by 14.2%, while the replacement of 50% reduces the heat of hydration by 10.2% at 7 days. However, 20% jump in the amount of slag replacement significantly reduces the heat of hydration. (38.1% at 7 days and 35.8% at 28 days).

Table 5-10 Effect of Pozzolans on Heat of Hydration

Cement Source	Time	% Reduction in Heat of Hydration			
		20% Fly Ash	35% Fly Ash	50% Slag	70% Slag
A-1	7 Days	14.2	-	10.2	-
A-2	7 Days	-	19.9	-	38.1
A-1	28 Days	18.1	-	9.1	-
A-2	28 Days	-	28.4	-	35.8

Thermal Diffusivity

The average thermal diffusivity number for the mixes was determined to be 0.80 ft^2/day which is about 35% less than the amount suggested in ACI 207.1R-96 for concrete mixes with lime stone aggregate (ACI suggested number is 1.22 ft^2/day). ACI 207.1R-96 does not specify the maximum aggregate size of the samples that were used to determine the concrete thermal diffusivity. Since the major application of mass concrete is in dams, the numbers reported by ACI 207.1R-96 may be originated from the samples with very large aggregate size. However, in FDOT approved mass concrete mix designs coarse aggregate occupies about 45% of the mix volume. The rest is filled with cement, fine aggregate, and water. The mixture of cement and sand has a thermal diffusivity of about 0.40 ft^2/day (Xu and Cheng, 2000). The thermal diffusivity of water is 0.13 ft^2/day. The thermal diffusivity of limestone has been reported between 1.00 to 1.40 ft^2/day in different sources with an average 1.20 ft^2/day. Based on these diffusivity numbers it is reasonable for a concrete containing about 45% limestone and the rest containing materials with much lower thermal diffusivity to have a thermal diffusivity lower than that of the limestone. Another observation of theses tests (as it was shown in Chapter 4) was that concrete thermal diffusivity reduces with the replacement of higher percentage of pozzolans.

The thermal diffusivity number affects the results of calculations to determine the maximum temperature and temperature difference (thermal gradient) in a mass concrete element. In Figure 5-17 the effect of thermal diffusivity on the maximum temperature rise for mass concrete elements with thickness of 5.5, 6.5 and 7.5 feet is shown. The maximum temperature rise was calculated using Schmidt method. Two different curing conditions were assumed. At first it was assumed that no insulation is used during the curing period. For the second time, the boundary conditions were modified to simulate the use of insulation. As it is shown, higher diffusivity numbers result in lower maximum temperature rise. Therefore, if the thermal diffusivity number of the concrete is assumed higher than what it really is, the maximum temperature will be predicted lower than the number that it will reach. In Figure 5-18 the effect of thermal diffusivity on thermal gradient is shown. The model showed that if insulation is not used, changes in the thermal diffusivity will affect the thermal gradient but if insulation is used, changes in the thermal diffusivity do not affect the thermal gradient.

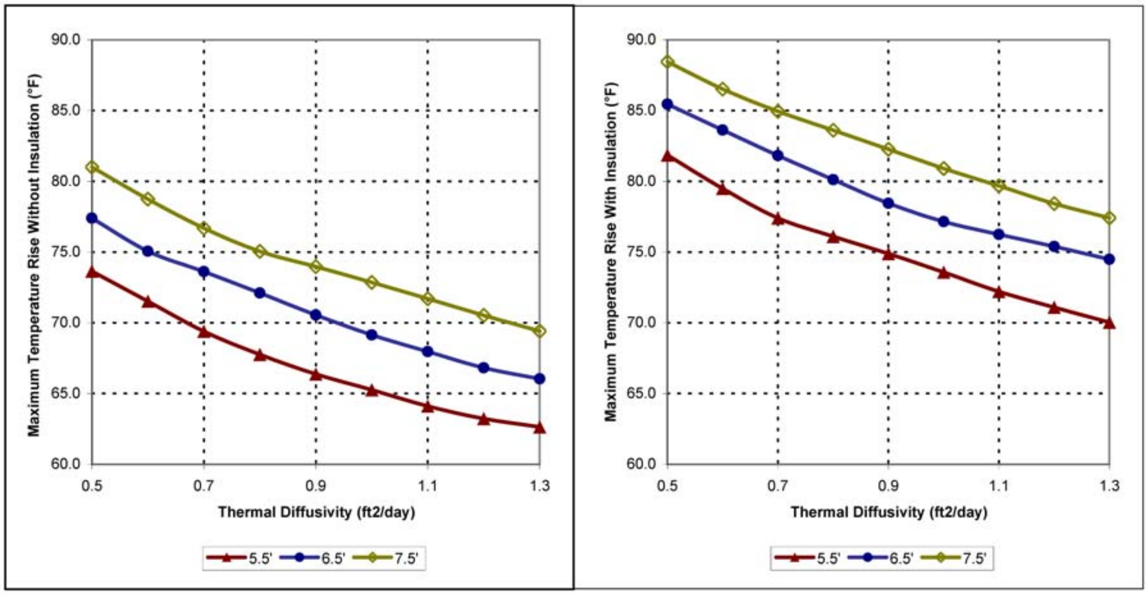

Figure 5-17 Effect of Thermal Diffusivity on Maximum Temperature Rise

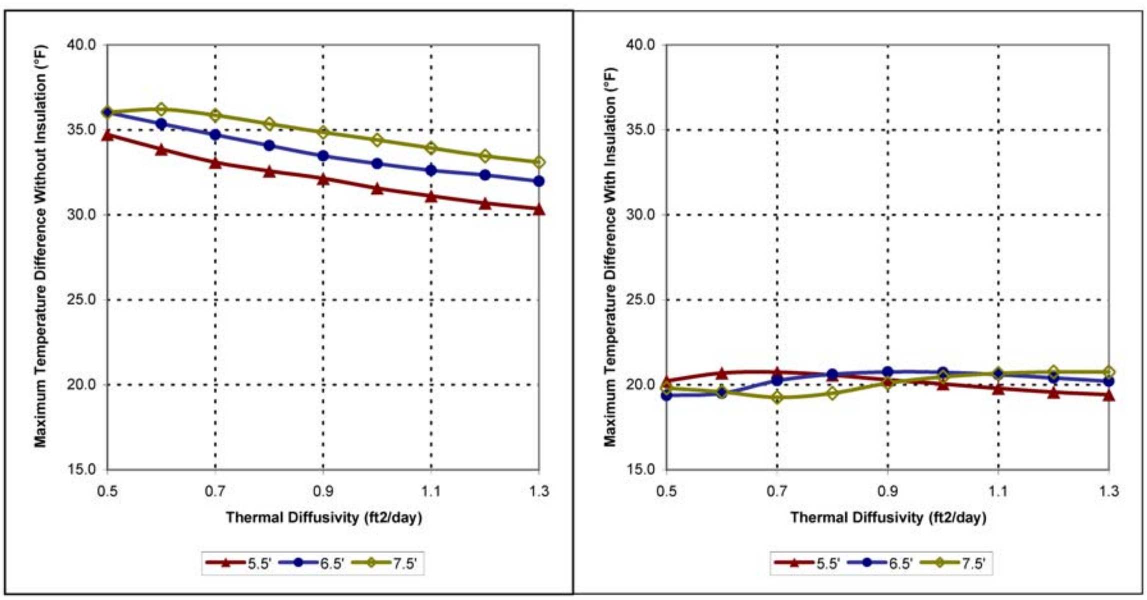

Figure 5-18 Effect of Thermal Diffusivity on Thermal Gradient

Compressive Strength

It was shown in Chapter 4 that using pozzolans increases short term (15 day) compressive strength of high temperature cured samples. However, after 28 days, the compressive strength of high temperature cured samples was lower than the room temperature cured samples.

Table 5-11 and Figure 5-19 show the summary of 28-day compressive strength results for 6″x12″ specimens cured in moisture room. The numbers are average of compressive strength tests for similar mixes. For specimens with 73°F a significant difference in 28-day compressive strength was not resulted. It can also be seen that replacing a portion of cement with slag helps the 28-day compressive strength of concrete. The results for high temperature placed concrete showed that replacing cement with pozzolans (slag or fly ash) reduces the 28-day compressive strength 2 to 3% in the mixes with 20% fly ash, 50% and 70% slag. However, in the mixes with 35% fly ash a significant (18%) loss of 28-day compressive strength was observed.

Table 5-11 Effect of Pozzolan Content on 28-day Compressive Strength

Placing Temperature	Pozzolan Type	Pozzolan (%)	Compressive Strength (psi)	Compressive Strength Ratio (As a ratio of 0% pozzolan mix)
73°F	-	0	8014	**1.00**
	Fly Ash	20	7744	**0.97**
		35	7873	**0.98**
	Slag	50	8207	**1.02**
		70	8297	**1.04**
95°F	-	0	8437	**1.00**
	Fly Ash	20	8244	**0.98**
		35	6893	**0.82**
	Slag	50	8260	**0.98**
		70	8221	**0.97**

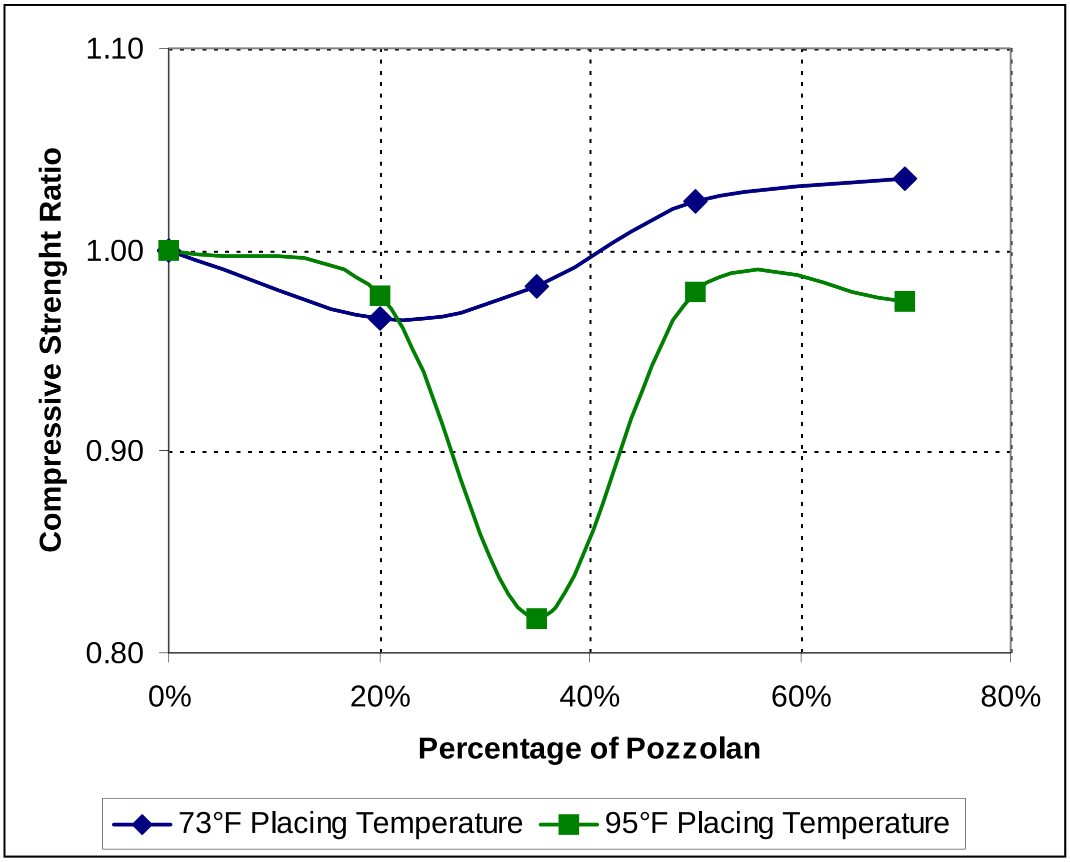

Figure 5-19 Effect of Placing Temperature and Pozzolan Content on 28-day Compressive Strength

CHAPTER 6
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

The main objective of this study was to develop the adiabatic temperature rise curves for mass concrete made with Florida materials. In addition, the thermal diffusivity of Florida concrete and the effect of high placing and curing temperature on compressive strength of concrete were studied. The following is a summary of steps undertaken to achieve the goals of the project.

A literature review was conducted to identify the factors affecting temperature rise in concrete and to study previous works in this field. The literature review showed that the temperature rise in concrete has been an issue of concern since early 1930's when mass concrete was first used in dam projects. Various methods have been developed to predict the temperature rise in mass concrete. One of the most practical methods is to use ACI curves in combination with the Schmidt method to predict the temperature rise in mass concrete elements. The literature review also showed that in recent years with the advances in computing devices some attempts have been made to develop numerical methods and computer software to predict the temperature rise in a mass concrete element.

The first step in the experimental phase of the project was to determine the concrete mix designs and materials which needed to be tested. A total of 20 mixes with cements from two different sources, with two different placing temperatures and various percentages of pozzolanic materials were tested for adiabatic temperature rise, thermal diffusivity and compressive strength. The Heat of hydration of cement samples and blends of cement and pozzolan were also determined.

The results of the tests were analyzed. The effect of placing temperature, curing temperature and pozzolan content on temperature rise curves, thermal diffusivity and compressive strength were studied.

Conclusion

The following conclusions can be made after executing the aforementioned activities for this study and analyzing the results:

- Using fly ash and slag as a replacement for AASHTO Type II cement reduces the peak temperature in mass concrete pours.
- Higher placing temperature reduces the effectiveness of fly ash and slag in peak temperature reduction.
- Higher placing temperature accelerates the hydration of cement; therefore the concrete reaches the peak temperature earlier.
- A Pozzolan Modification Factor (α_p) was developed that can be used to modify ACI curves for mixes with different cement and pozzolan contents.
- Thermal diffusivity of concrete reduces when cement is replaced with high percentage of pozzolanic materials.
- Thermal diffusivity of mass concrete mixes used in Florida is approximately 33% lower than the number suggested by ACI (0.8 ft^2/day vs 1.22 ft^2/day).
- Concrete mixes with pozzolan have a higher short term compressive strength if cured in higher temperature. However, after 28 days the mixes with lower curing temperature show larger compressive strength.

Recommendations

The results of this study showed that the current method used by the FDOT contractors to predict thermal developments in mass concrete elements underestimates the maximum temperature rise. It is therefore recommended that:

- An analytical model be developed that can more accurately predict the rate of heat generation and maximum temperature rise of a mass concrete element based on its mix design (type and amount of cement, type and amount of pozzolanic materials, and type of aggregate), placement temperature, geometry, and environmental conditions.

- Data obtained in this study could be used for verification of such analytical model; however, additional data is needed to assure the model is comprehensive and can predict the thermal development of typical mass concrete mixes used in Florida. The current study used only one type of fly ash and slag and two types of cement. Data for mixes with other types of cement and pozzolanic materials commonly used in mass concrete elements in Florida must be generated.

- The Pozzolan Modification Factor (α_p) developed in this study should be verified for pozzolanic materials supplied by other sources and revised if necessary.

- The diffusivity number suggested by ACI 207 for concrete made with limestone (1.22 ft^2/day) should be replaced with the one measured in this study (0.8 ft^2/day) in prediction of mass concrete peak temperature.

APPENDIX A

SURVEY OF STATE HIGHWAY AGENCIES

A previous study[1] on mass concrete showed that only nine states including Florida have mass concrete specifications. The purpose of this survey was to figure out the other states' regulation on mass concrete pours and possible studies that have been done. Following questions were sent to state Material Engineers of California, Idaho, Illinois, Kentucky, North Carolina, South Carolina, Texas and Virginia via e-mail

1. Do you require contractors to provide you with calculations showing mass concrete temperature rise prediction?
2. If the answer to question number 1 is yes, what method is used to predict the temperature rise? Do you use ACI curves?
3. Have you ever developed adiabatic temperature rise curves for concrete mixes that are usually used in mass concrete projects? If yes, can you provide us with your own curves?
4. If you have not developed adiabatic temperature rise curves, do you think it is necessary to develop curves for concrete mixes made of local materials or you think that ACI curves are accurate enough?
5. Do you have any suggestion regarding ACI curves or adiabatic temperature rise in mass concrete?

Four responses were received. Following is the description of responses.

<u>California</u>

Caltrans (California Department of Transportation) does very few projects where mass concrete is involved. Of the ones that mass concrete was involved, Caltrans

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[1] Chini, Abdol R., Muszynski, Larry C., Acquaye, Lucy, Tarkhan, Sophia, 2003 *"Determination of the Maximum Placement and Curing Temperatures in Mass Concrete to Avoid Durability Problems and DEF",* Final Report Submitted to The Florida Department of Transportation, Gainesville, Florida

typically leans to the side of performance specifications and requires the contractor to deliver crack free concrete. Full responsibility of the mass pour is put on the contractor.

If the contractor does have a heat removal plan, it must be approved by the engineer and the regional water quality control board (if they are in fact involved). Caltrans's approval does not imply that the plan will work, it is rather to check if it is reasonable, follows "best management practices," and is workable.

One thing Caltrans does to help the contractor is to make sure to give mix parameters that are conducive to low heat. For instance on the new San Francisco Oakland Bay Bridge, California's "banner project," currently in construction, Caltrans gives contractors a mix design which has 50% slag and/or 50% fly ash requirement.

Illinois

Illinois Department of Transportation does not require contractors to submit a mass concrete plan. Contractors are required to monitor the temperatures, report the results, and stay within the specification limitations for temperature difference.

Kentucky

Kentucky Department of Transportation does have a requirement for contractors to submit a mass concrete. They use ACI 207 curves and do not think that they need to develop their own curves because ACI are believed to be fairly accurate.

South Carolina

South Carolina Department of Transportation requires contractors to submit a mass concrete plan. The requirements are as follows:

702.16 Mass Concrete Placement. Mass concrete placement shall be defined as any pour in which the concrete being cast has dimensions of 5 feet or greater in three different directions. For pours with a circular cross-section, a mass concrete placement shall be defined as any pour that has a diameter of 6 feet or greater and a length of 5 feet or greater. For all mass concrete pours, the mix temperature shall not exceed 80°F as measured at discharge into the forms. Further, the Contractor shall be required to maintain a temperature differential of 35°F or less between the interior and exterior of all mass pour elements during curing. Before placing mass concrete, the Contractor shall submit, to the Engineer for review and acceptance, a Mass Concrete Placement Plan containing, but not limited to, the following:

1. An analysis of the anticipated thermal developments within mass pour placements using the proposed materials and casting methods.

2. A plan outlining specific measures to be taken to control the temperature differential within the limits noted above.

3. Details of the Contractor's proposed monitoring system. If the Contractor is proposing a special concrete mix design as part of the temperature control plan, this mix design should also be submitted for review. The Contractor shall provide temperature monitoring devices to record temperature development between the interior and exterior of the element at points approved by the Engineer and shall monitor the mass pours to measure temperature differential. Temperature monitoring shall continue until the interior temperature is within 35°F of the lowest ambient temperature or a maximum of two (2) weeks. The Engineer shall be provided with a copy of each set of readings as they are taken and a temperature chart for each mass pour element showing temperature readings vs. time. If the monitoring indicates that the proposed measures are not controlling the concrete temperature differential within the 35°F specified, the Contractor shall make the necessary revisions to the plan and submit the revised plan for review. The Contractor shall assume all risks connected with placing a mass pour of concrete. Review of the Contractor's plan will in no way relieve the Contractor of the responsibility for obtaining satisfactory results. Should any mass concrete placed under this specification prove unsatisfactory, the Contractor will be required to make the necessary repairs or remove and replace the material at the Contractor's expense. All costs associated with special temperature controls for mass concrete placement shall be included in the unit cost of the concrete cast, and will be without additional specific compensation. The control of temperatures in mass concrete pours shall be in addition to any other requirements found on the plans and/or in the special provisions that may apply to the work in question.

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