Mass Concrete for Buildings and Bridges

By John Gajda

Concrete Thinking
for a sustainable world

PCA
Portland Cement Association

Mass Concrete for Buildings and Bridges

By
John Gajda

PCA
Portland Cement Association
5420 Old Orchard Road
Skokie, Illinois 60077-1083
847.966.6200 Fax 847.966.9781
www.cement.org

An organization of cement companies to improve and extend the uses of portland cement and concrete through market development, engineering, research, education, and public affairs work.

Abstract: Mass concrete has been historically associated with large structures such as dams and other large volume placements. However, due to the increasingly common use of fast-track construction practices and high-performance concretes with high cementitious contents, mass concrete issues are being experienced in typical bridge and building placements. Understanding the causes of mass concrete issues (high internal temperatures and temperature-related cracking) is the key to producing a structure that provides many years of service. This document provides practical guidance to aid in understanding mass concrete, how to manage concrete temperatures, and prevent or minimize temperature-related cracking.

Keywords: bridges, buildings, columns, delayed ettringite formation (DEF), fly ash, foundations, heat of hydration, mass concrete, metakaolin, moisture shrinkage cracking, restraint cracking, silica fume, slag cement, supplementary cementitious materials (SCMs), temperature rise, thermal cracking, thermal shock

Reference: Gajda, John, *Mass Concrete for Buildings and Bridges,* EB547, Portland Cement Association, Skokie, Illinois, USA, 2007, 44 pages.

About the Author: John Gajda, PE, Principal Engineer, CTLGroup, Inc., 5400 Old Orchard Road, Skokie, IL 60077, USA, Phone: 847-972-3140, Email: JGajda@CTLGroup.com.

Cover photo:

Upper Left Photo: A large bridge column from the Clearwater Causeway in Florida. (IMG25430)

Center Photo: A partially completed pier table of the San Francisco-Oakland Bay Bridge in California. Insulated formwork and cooling pipes are visible in the photo. (IMG25431)

Lower Right Photo: Large columns of a commercial high-rise building. (IMG13863)

SN3035

ISBN 978-0-89312-261-4

EB547

WARNING: Contact with wet (unhardened) concrete, mortar, cement, or cement mixtures can cause SKIN IRRITATION, SEVERE CHEMICAL BURNS (THIRD DEGREE), or SERIOUS EYE DAMAGE. Frequent exposure may be associated with irritant and/or allergic contact dermatitis. Wear waterproof gloves, a long-sleeved shirt, full-length trousers, and proper eye protection when working with these materials. If you have to stand in wet concrete, use waterproof boots that are high enough to keep concrete from flowing into them. Wash wet concrete, mortar, cement, or cement mixtures from your skin immediately. Flush eyes with clean water immediately after contact. Indirect contact through clothing can be as serious as direct contact, so promptly rinse out wet concrete, mortar, cement, or cement mixtures from clothing. Seek immediate medical attention if you have persistent or severe discomfort.

Table of Contents

Preface

Mass Concrete for Buildings and Bridges is a practical guide that provides pertinent information related to mass concrete issues, which are increasingly being experienced in seemingly normal (but large) concrete placements. This publication updates and supersedes PCA's *Concrete for Massive Structures* (IS128). The intended audience for this publication includes contractors, designers, engineers, specification writers, and owners; however, anyone who is involved with mass concrete construction may benefit from reading this publication.

With information pertaining to material selection, thermal control calculation methods, and construction practices for mass concrete placements; this publication supplies the reader with a wealth of information. Information in this publication is based on current practice and technical information, some of which differs significantly from that which were developed for the mass concrete of dams. As with all technical publications, it is assumed that the reader has an understanding of concrete technology, which is necessary to avoid potential issues and other safety hazards.

Acknowledgement

The author would like to collectively thank all of those who have contributed to this publication.

The following individuals provided quality photographs which help clarify the text: Chris Carson (Carson Mitchell), Mike Fox (Engius), and Fred Parkinson (PB).

The following colleagues contributed significantly to the accuracy and review of this publication: Rick Bohan (PCA), Terry Collins (PCA), Beatrix Kerkhoff (PCA), Steve Kosmatka (PCA), Paul Tennis (PCA Consultant), George Seegebrecht (CTLGroup), Jackie Shearer (CTLGroup), Martha VanGeem (CTLGroup), Randy Bass (Schnabel Engineering), Charles Ishee (Florida DOT), Al Kaufman (ConcreteRx), Brian Mapel (FCI Constructors), Jon Poole (WJE), Henry Prenger (Lafarge North America), Grant Scott (Kiewit), and Wahid Tadros (California Engineering).

In addition, thanks are due to Cheryl Taylor (PCA Consultant) and Arlene Zapata (PCA) for their professional expertise in the design layout and artwork for this publication. Finally, special thanks go to Michelle Wilson (PCA) for her significant contribution as technical editor for this publication.

Chapter 1 – Introduction

Mass concrete is defined by the American Concrete Institute 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 207.1R).

Traditionally, mass concrete has been associated with massive structures (the Hoover Dam for example) and other extremely large volume placements. However, these are not the only cases where mass concrete measures must be taken. The size of placements for structures, such as buildings and bridges, are routinely increased for purposes of economy or to carry greater loads (Figure 1).

Figure 1. Mass concrete columns and footings for the James River Bridge. (Courtesy of Fred Parkinson, PB.)

It is increasingly common today to use high-performance concretes with high cement contents for added durability and service life, rapid strength gain, or higher load carrying capacity. The combined effect is that mass concrete symptoms are now experienced in seemingly typical placements.

Placements treated as mass concrete include: drilled shafts, caissons, mat foundations, footings, bridge piers, columns, column caps, and beams (Figure 2). Heat is generated as a result of the hydration of cementitious materials. The majority of heat is generated during the first few days after placement. In most placements, the heat escapes almost as quickly

(IMG25511)

Figure 2. Mass concrete columns for the San Francisco-Oakland Bay Bridge.

as it is generated. However, in mass concrete placements, the heat energy is generated faster than it can escape. This causes the temperature of the concrete to increase. Depending on the mix design and the size of the placement, internal concrete temperatures can greatly exceed 70°C (158°F) and sometimes approach the boiling point of water, 100°C (212°F). Additionally, temperature differences between the interior and exterior surface of the placement can result in significant thermal stresses that can crack the concrete. This cracking can reduce the anticipated service life of the concrete and compromise structural integrity.

Treatment of a placement as mass concrete essentially requires control of concrete temperatures and the temperature difference within a placement until the concrete is adequately cooled. This cooling time may extend well beyond the normal curing period. Mass concrete placements are often a challenging experience for many contractors, ready-mix suppliers, engineers, and owners because of the potentially significant preplanning and thermal control efforts that may be required. If not identified early on, significant costs may be incurred due to rework and delays.

Mass concrete placements typically use concrete from local ready-mix suppliers and are often placed by concrete pump (Figure 3). The volume of concrete required for these placements and projects is usually not large enough to justify the cost of a dedicated onsite batch plant. Local ready-mix suppliers typically do not stock special materials; therefore, the concrete mix designs used for these placements will most likely use locally available cements and aggregates of commonly used sizes and gradations. Additionally, concrete suppliers often have limited capabilities for precooling concrete. Traditional temperature control strategies successfully used for massive concrete placements may not be practical for mass concrete in bridge and building placements.

The purpose of this document is to provide practical guidance to aid in the understanding, planning, placement, and thermal control of mass concrete placements in bridges and buildings.

Figure 3. Placement of a chimney foundation (23.75 m [78 ft] octagon in plan, 3.1 m [10 ft] thick). (Courtesy of Chris Carson, Carson Mitchell Inc.)

Chapter 2 – Key Considerations

Mass concrete placements can be damaged by high internal temperatures and by cracking from thermal stresses. When temperatures are kept within reasonable limits, thermal stresses can be controlled.

Effects of High Internal Temperatures

Elevated temperatures alter the hydration reactions of the cementitious materials that are responsible for the strength and durability of the concrete. The hydration reactions are temperature dependent; at elevated temperatures, hydration reactions are accelerated. The acceleration causes heat to be generated much more rapidly. As a result, high internal temperatures can occur in a very short period of time.

Delayed Ettringite Formation

At temperatures above 70°C (158°F), *delayed ettringite formation* (DEF) can occur in some concretes. DEF is a rare but valid concern in regard to the durability of concrete. When DEF occurs, the concrete paste expands and cracks the concrete with detrimental results (Kosmatka 2002). DEF has reportedly been found in mass concrete structures in the United States and Europe (Scrivener 2004).

DEF can occur in a mass concrete placement because of unusually high internal temperatures. Although DEF is still being studied (Shimada 2005), the mechanism appears to follow this sequence: (1) the high temperatures disrupt the normal formation of ettringite, causing the sulfate and alumina to be adsorbed by calcium silicate hydrate (C-S-H) gel in the cement paste; (2) after the concrete has cooled to ambient conditions, the sulfate can later desorb in the presence of moisture and react with calcium monosulfoaluminate to form ettringite; (3) this "delayed" ettringite can then exert great pressure because it forms in the limited space of a rigid structure in an expansive reaction; and (4) these high pressures within the paste can cause internal micro-cracking and macro-cracking. DEF is characterized by expanding paste that becomes detached from aggregates (Figure 4).

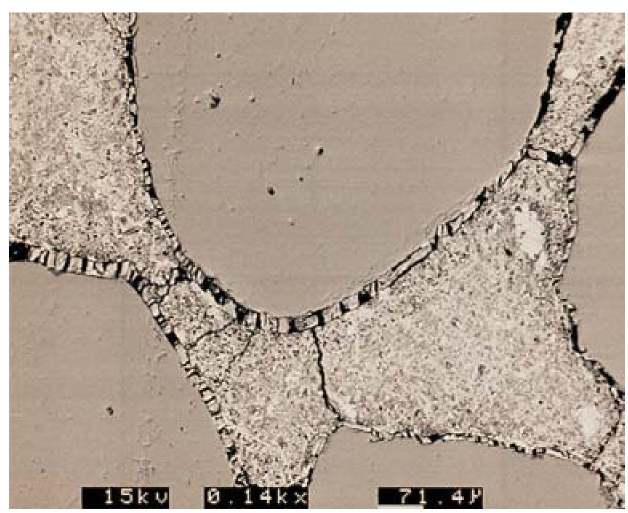


Figure 4. DEF is characterized by expanding paste that becomes detached from non-expansive components, such as aggregates, creating gaps at the paste-aggregate interface. The gap can subsequently be filled with larger opportunistic ettringite crystals as shown here. (Photo courtesy of Zhaozhou Zhang and Jan Olek, Purdue University).

The reformation of ettringite requires a substantial quantity of water. Without free water, the DEF reaction cannot readily occur. Some concretes may never be exposed to large volumes of externally available water. This "reasoning" has been used to justify internal temperatures above 70°C (158°F). However, allowing higher internal temperature limits (>70°C) may not be appropriate depending on the intended use or exposure conditions. Examples might include below-grade structures or structures in an outdoor environment. The possibility that the intended use for a structure might change, along with its exposure to moisture, should also be considered.

Concretes that contain supplementary cementitious materials (SCMs) may have a reduced risk of DEF (Miller 2003, and Silva 2006). In these studies, concrete mixtures containing 20% slag and 20% Class F fly ash were found to suppress the formation of expansive DEF.

While the use of SCMs in concrete mixtures and isolation of the concrete from water can prevent expansive DEF, the most effective way to ensure that DEF will not occur is to prevent the internal temperature of the concrete from exceeding 70°C (158°F). Refer to PCI guidance on specifiable criteria to avoid DEF.

Temperature Effect on Other Concrete Properties

Compressive Strength

In general, concrete cured at elevated temperatures reaches the specified strength faster than concrete cured at lower temperatures. Maturity relationships can be used to estimate the in-place concrete strength as a result of elevated temperatures. Maturity is discussed in the section on ***Fast-Track Construction.***

In mass concrete placements, research has shown that there is no temperature effect on the compressive strength within the interior of a concrete placement provided temperatures are kept under 82°C (180°F) (Burg 1999). At temperatures above 85°C (185°F), there are conflicting data regarding high temperature effects on in-place strength and elastic modulus. Much of this work was conducted on compressive strength test cylinders where some surface drying may have occurred as a result of the high temperatures. There is some undocumented information on cores removed from mass concrete placements that reached internal temperatures near 95°C (205°F) resulting in a reduction in physical properties.

Permeability

The development of desirable concrete properties is diminished when drying occurs; curing ceases when the relative humidity in concrete falls below 80%. Elevated temperatures may result in localized surface drying unless proper moisture is maintained during curing. This localized surface drying can have a detrimental effect on the permeability and strength at the concrete surface. Development of adequate resistance to permeability at the surface is necessary to ensure that the concrete's service life requirements can be met.

Cracking

Mass concrete can be damaged during construction due to elevated temperatures. Damage from cracking can be caused by thermal movement (thermal cracking), thermal shock, moisture loss (drying shrinkage cracking), and restraint. This cracking is generally preventable if precautions are taken.

Thermal Cracking

In all mass concrete placements, the interior of the concrete is typically hotter than the exposed surfaces. This temperature difference, also called a thermal gradient or temperature delta, creates thermal stresses in the concrete because the interior expands more than the surface. This results in large thermal stresses at the surface of the concrete. Cracking occurs immediately when the thermal stress exceeds the localized tensile strength of the concrete. This cracking is referred to as *thermal cracking,* and is preventable.

In many cases, thermal cracking presents a durability issue because the cracking often extends from the concrete surface to the depth of the reinforcing steel providing an easy pathway for air and water to reach reinforcing steel and start corrosion. In some cases, where thermal stresses are significant, the cracking may be extensive enough to extend through the full thickness of the placement. Full depth cracking can affect the structural integrity of the placement.

Thermal cracking is similar in appearance to drying shrinkage cracking. The typical crack pattern is that of random map cracking on the surface of the concrete, as shown in Figure 5. Crack widths are usually minimal, but will depend on how much the thermal stresses exceed the localized tensile strength of the concrete. In extreme cases, where the thermal stresses greatly exceed the tensile strength, tensile cracking may appear as one or more wider cracks rather than map cracking, as shown in Figure 6. In slab or wall-type placements, this cracking may be random, or it may align primarily in a longitudinal or transverse direction. In column-type placements, this cracking is typically vertical.


(IMG25514)

Figure 5. Thermal cracking manifested as map-cracking on the surface of a base slab.


(IMG25515)

Figure 6. A more severe case of thermal cracking in a concrete footing.

Thermal crack widths are proportional to the temperature difference between the interior and surface of the concrete. When the interior of the concrete cools to near ambient conditions, crack widths should decrease accordingly. The exception occurs when the thermal stresses are large enough to locally yield the reinforcing steel. In some placements, crack widths may vary throughout the year due to thermal cycling with seasonal weather changes.

Limiting the temperature difference within a mass concrete placement can control or prevent thermal cracking. A simple approach often used is to ensure that the temperature difference between the center and surface of the concrete does not exceed 20°C (35°F). This approach is based on studies with a series of small-unreinforced concrete dams constructed in England more than 50 years ago (FitzGibbon 1976). A 20°C (35°F) limit is often used because it is easy to measure, and usually effective in preventing thermal cracking.

However, because of its inherent simplicity, limiting the temperature difference to 20°C (35°F) can be conservative. Some concretes, especially high-strength concretes, or concretes containing low-thermal expansion aggregates, like granite or limestone, can withstand temperature differences much greater than 20°C (35°F) without thermal cracking. In these cases, higher temperature difference limits may be justified. Others have suggested that 25°C (45°F) would be a more appropriate temperature gradient limit for concrete with granite aggregates, and 31°C (56°F) for concrete with limestone aggregates (Bamforth 1981).

Because thermal cracking occurs when thermal stresses exceed concrete tensile strength – with both values changing within the first few weeks after concrete is placed – it is often advantageous and cost-effective to use a performance-based temperature difference limit tailored both to the properties of the concrete and the placement. This method is used to keep the thermal stresses less than the in-place tensile strength of the concrete at the surface. An example of a performance-based temperature difference limit is shown in Figure 7. Finite element modeling and detailed calculations are often used to develop the temperature difference limit.

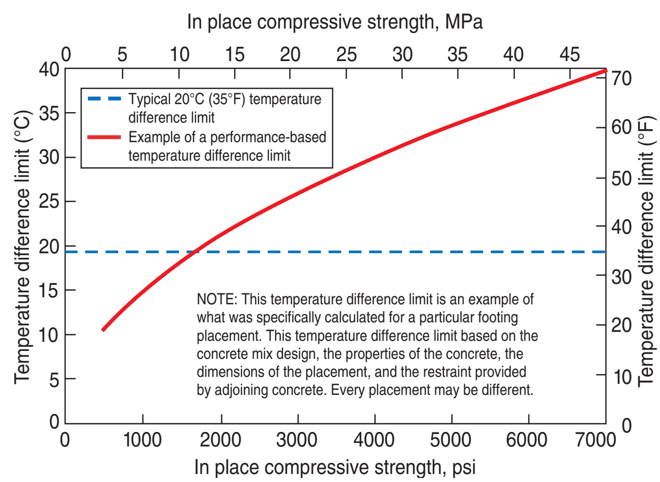


Figure 7. Example of a performance-based temperature difference limit.

While it is generally desirable to prevent thermal cracking, in some cases, a certain amount of thermal cracking may be tolerable. Table 1 shows tolerable crack widths based on various exposure conditions (ACI 224R). The allowance for some thermal cracking generally means higher temperature difference limits. For instance, in the new Benicia-Martinez Bridge, near Concord, California, tolerable crack widths of up to 0.15 mm (0.006 in.) were allowed. This resulted in a temperature difference limit about 6°C (10°F) higher when

Table 1. Tolerable Crack Widths per ACI 224R*

Exposure condition	Crack width mm	Crack width in.
Dry air or protective membrane	0.41	0.016
Humidity, moist air, soil	0.30	0.012
Deicing chemicals	0.18	0.007
Seawater/seawater spray, wetting/drying	0.15	0.006
Water-retaining structures†	0.10	0.004

* It should be expected that a portion of the cracks in the structure will exceed these values. With time, a significant portion can exceed these values. These are general guidelines for design to be used in conjunction with sound engineering judgement.

† Excluding nonpressure pipes.

compared to the limits given if thermal cracking was not allowed. Finite element modeling and equations presented in ACI 207.2R can be used to account for the crack width limiting effects of reinforcing steel.

Thermal Shock

Thermal shock occurs when the surface of the concrete is rapidly cooled relative to the interior. Like thermal cracking, thermal shock cracking occurs when the thermal stresses at the surface exceed the localized concrete tensile strength. This type of cracking is preventable.

Cracking from thermal shock ranges from minor to extensive. Minor thermal shock cracking can affect the concrete's durability. It often happens when formwork and thermal controls (such as surface insulation) are prematurely removed. Extensive cracking most likely occurs from unplanned events, such as flooding of a cofferdam because of a pump failure, or high winds that blow the thermal insulation off the concrete and expose the surface to cold ambient temperatures.

Engineers should ensure that the temperature difference between the interior and the surface of the placement is maintained until the interior of the concrete has cooled to within the temperature difference limit of the ambient temperature. For example, given a specified 20°C (35°F) temperature difference limit: if the interior of the concrete is 57°C (135°F) and the average ambient air temperature around the placement is 10°C (50°F), then the thermal controls (for instance: insulation) should not be removed from the concrete until the interior cools to less than 30°C [the sum of 10°C and 20°C] (85°F) [the sum of 50°F and 35°F]. This may require that thermal control be maintained for a significantly longer time than the required period of moist curing. To reduce this time, a performance-based temperature difference limit can be used. Other practices, including precooled concrete, or active post-cooling (with cooling pipes) can also be helpful. These practices are discussed in Chapter 3 – **Influencing Factors**.

Moisture Shrinkage Cracking

Loss of surface moisture can cause cracking in mass concrete placements. Higher curing temperatures increase the evaporation rate of surface moisture. Concrete shrinks due to moisture loss which adds to the stresses at the surface of the concrete. The combination of lost moisture and thermal stresses may exceed the tensile strength of the concrete and cause localized cracking.

Additionally, if adequate moisture is not retained, curing at the surface essentially stops and strength development ceases (see Figure 8). When the tensile strength stops developing, the likelihood of thermal cracking increases because thermal stresses are not related to moisture loss. To prevent drying shrinkage cracking, proper moisture retention curing practices must be followed.

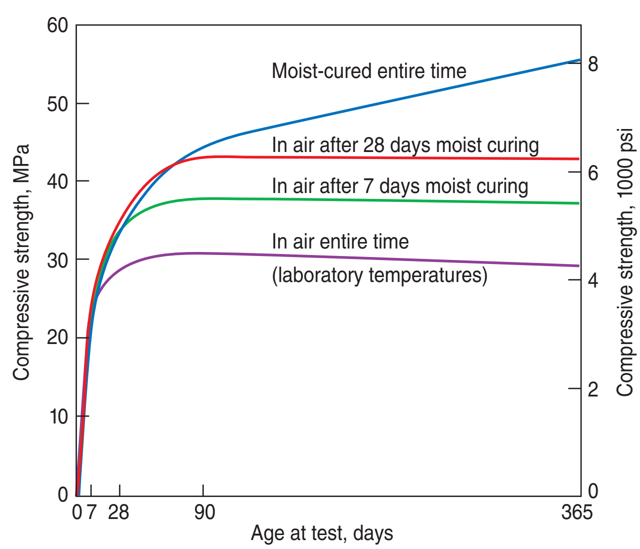


Figure 8. Effect of curing on strength development (Gonnerman and Shuman 1928).

External Restraint

Cracking due to external restraint (restraint cracking) occurs when the thermal movement of a mass concrete element is partially or fully restrained by one or more adjoining elements. The adjoining element or elements prevent the thermal movement from freely occurring. If thermal expansion does occur, the compressive or tensile loading of the adjoining elements would normally absorb some loading while early-age creep of the mass concrete would absorb the remainder. Once the maximum temperature occurs in the placement, the concrete begins to thermally contract. Any restraint from adjoining placements may prevent this contraction from fully occurring. That portion of the thermal expansion absorbed by early-age creep is not recoverable and results in cracking.

In long wall placements, restraint cracking typically manifests itself as a series of vertical cracks widest near the base. These cracks may extend up the wall. The first crack usually occurs about mid-length in the wall. Subsequent cracks appear mid-way between the first crack and the ends of the placement, as shown in Figure 9. Additional cracks may appear midway between the existing cracks. The length and cumulative width of the cracks can be used to approximate the stresses and tensile strength of the concrete at the time of cracking. Similar cracking may occur in beams, but typically appear as uniformly spaced cracks perpendicular to the longest dimension of the beam.

This type of cracking is prevented by limiting the effects of restraint, using low-thermal expansion aggregates, or by reducing the heat generated from the concrete.

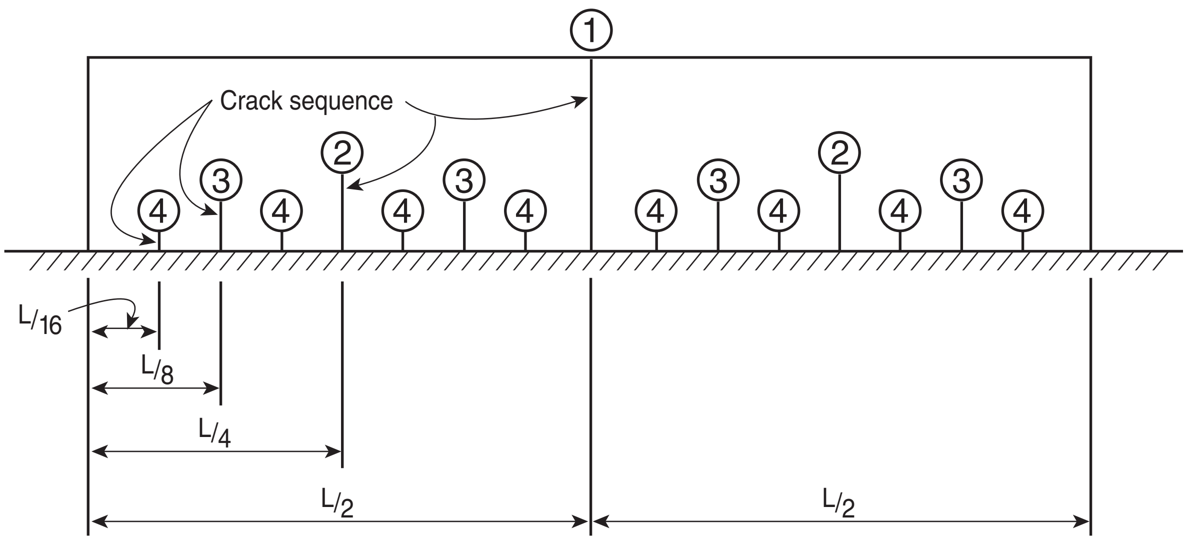


Figure 9. Continuous base restraint sequence of crack propagation (adapted from ACI 207.2R).

Chapter 3 – Influencing Factors

Factors that affect whether the maximum temperature will be excessive, or if cracking will occur, include: the dimensions of the placement, concrete mix design, initial concrete temperature, thermal control measures, and curing methods. These factors are all interrelated. For example, a concrete with high heat energy may require insulation to prevent thermal cracking along with significant precooling to avoid excessive internal temperatures. These influencing factors are discussed in the following sections.

Placement Dimensions

The dimensions of the concrete are one of the main factors that determine whether or not a concrete placement will behave as mass concrete. The *minimum dimension,* that is the smallest of the length, width, and thickness of the placement is the critical dimension. In placements with a small minimum dimension, the heat of hydration dissipates almost as rapidly as it is generated. When this occurs, the temperature rise in the placement is minimal. For placements with a larger minimum dimension, internal heat cannot escape as rapidly as it is generated. This can result in high internal temperatures.

There is no clear consensus regarding the minimum dimension that requires a concrete placement be treated as mass concrete. Minimum dimensions of 1 to 2 meters (3 to 7 ft) are commonly prescribed in mass concrete specifications. Some specifications use a volume-to-surface ratio instead of a minimum dimension requirement. There is confusion over the units used and in determining which concrete should be included in the calculation and in the volume-to-surface ratio specification. Because of this confusion, the volume-to-surface ratio is rarely specified.

In an attempt to standardize the requirement, the following guidance is provided: *Any placement of structural concrete with a minimum dimension equal to or greater than 915 mm (36 in.) should be considered mass concrete. Similar considerations should be given to other concrete placements that do not meet this minimum dimension, but contain Type III or HE cement, accelerating admixtures, or cementitious materials in excess of 355 kg/m³ (600 lb/yd³) of concrete.*

Using the definition noted above, a bridge or building column that is 940 mm (37 in.) in diameter would meet the minimum dimension criteria and should be treated as mass concrete. The same would be true for an 890 mm (35 in.) diameter column using a concrete with 359 kg/m³ (605 lb/yd³) of cementitious materials. If the 890 mm (35 in.) diameter column used a concrete with 341 kg/m³ (575 lb/yd³) of cementitious materials, the placement would not meet the mass concrete definition.

Consideration should also be given to placements that entrap heat. An example of such a placement would include a slab placed on top of a recently placed slab, where the combined thickness of the two slabs meets the mass concrete definition for minimum dimension.

Concrete Mixture Proportions

As indicated in the above definition for mass concrete, the concrete mixture proportions are a factor that determines whether the placement will behave as mass concrete. Concretes with a lower heat energy will not increase in temperature as much as a concrete with a higher heat energy, and therefore will have less mass concrete issues. The heat energy is mainly determined by the type and quantity of cementitious materials. Mixture properties, design strength, spacing of reinforcing steel, and required service life all affect the required quantity of cementitious materials.

Design Strength

Structural designs are based primarily on the compressive strength of concrete. The use of 28-day compressive strength requirements is standard for most concretes. To ensure that 28-day strengths are met, concretes are often significantly over-designed. This over-design often results in design strengths that are met at 7 to 14 days rather than at 28 days. This is counter-productive for mass concrete because it increases the cementitious materials content (Figure 10). Higher strength requirements generally also limit the use of large amounts of SCMs (such as fly ash and slag) due to their slower strength generation even though they can decrease the heat energy.

Most mass concrete placements will not experience design loads for periods of 56 days or more after the placement. Given this delay between placement and loading, concrete for mass concrete placements should be specified and proportioned based on the use of 56 or 90-day design strengths. These later age strength criteria would enable the use of lower cementitious materials contents and higher proportions of SCMs.

If early age strength is required, the use of a concrete with later-age design strength may still be viable. Early age strength criteria may still be acceptable because the temperatures within most mass concrete placements are elevated to the point where the in-place compressive strength develops much more quickly than that of companion laboratory-cured cylinders. Depending on the in-place temperatures, the design strength can be achieved in a relatively short period of time. An example of this is shown in Figure 11 for compressive strengths stored at laboratory conditions and then adjusted for different temperatures using maturity.

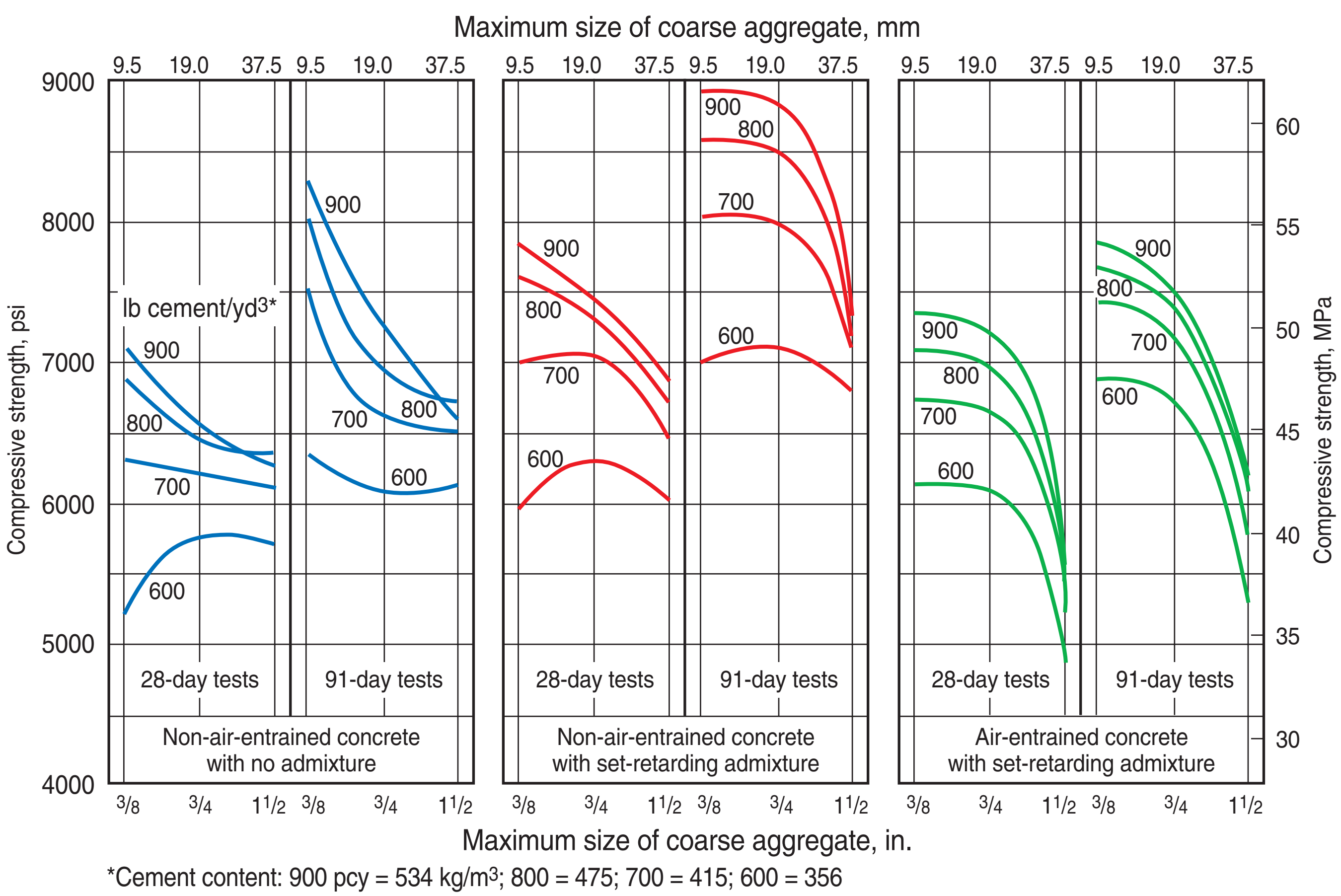


Figure 10. Cementitious content versus 28 and 91 day strength (adapted from Farny 1994).

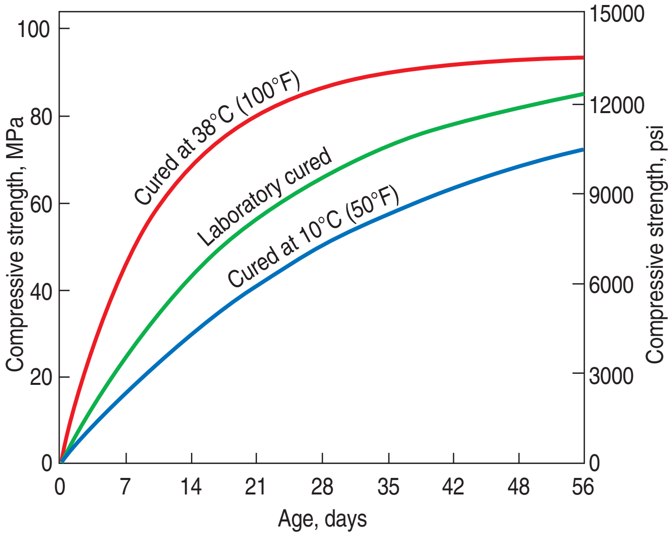


Figure 11. Effect of temperature on compressive strength development of a high strength mass concrete mix design.

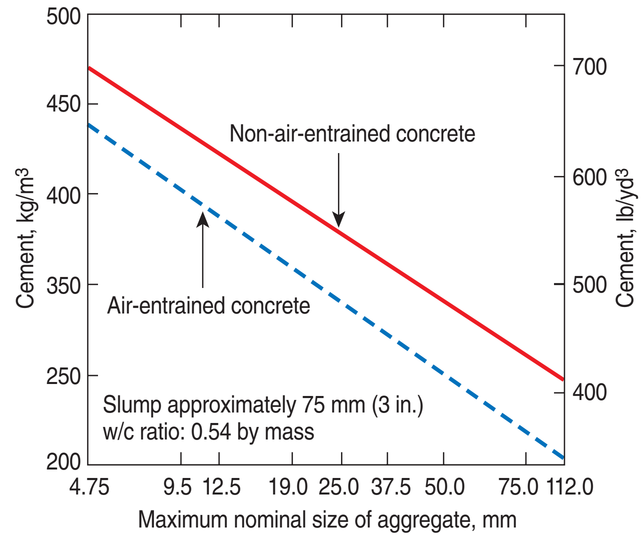


Figure 12. Aggregate size versus cementitious material content (Bureau of Reclamation 1981).

Reinforcement Spacing

Many mass concrete placements contain congested reinforcing steel. To help ensure that the concrete flows around the reinforcing steel, the maximum size of coarse aggregate should not exceed one-fifth the narrowest dimension of a concrete member, nor three-fourths the clear space between individual reinforcing bars or wire, bundles of bars, or prestressing tendons or ducts (ACI 318). It is also good practice to limit aggregate size to not more than three-fourths the clear space between reinforcement and forms. These requirements can significantly restrict the maximum size of aggregate that can be used.

In most cases, pumping considerations, and available materials will limit the maximum aggregate size to less than 38 mm (1.5 in.). In some highly congested placements, smaller size aggregates, such as 9.5 mm to 12.5 mm (⅜ in. to ½ in.), are used. Note that the use of smaller size aggregates will increase the required minimum cementitious material content of the concrete. These smaller size aggregates typically result in cementitious materials contents in the range of 300 to 420 kg/m^3 (500 to 700 lb/yd^3), as shown in Figure 12.

Durability

Durability and service life concerns are more prevalent today than ever before. Some owners now require concrete to achieve a design life of 100 to 150 years. A long design service life is possible with crack-free, low permeability concrete. While some owners specify that durability be demonstrated using performance-based test methods, such as ASTM C1202 *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration,* other owners simply require a minimum cementitious material content of 385 to 400 kg/m^3 (650 to 675 lb/yd^3) or higher. Lowering the permeability of concrete is best accomplished by a low w/cm and the use of SCMs.

Concrete Components

Once the design parameters of the concrete are determined, the next step is to design an optimal concrete for the project. Mass concrete must be proportioned to produce a material that can be readily placed, economical, and have low heat energy. Anything done to remove heat energy from the concrete will also reduce the efforts required to control temperatures and temperature differences after the concrete is placed. When the overall cost of thermal control is considered, mix design changes are often the most economical approach.

Cement Types

There are several types of hydraulic cements available in the United States as designated by ASTM C150, C595, and C1157. The most common types generally used are ASTM C150 Type I and Type II portland cement. Because of their availability, these cements are used in most mass concrete placements. ASTM C150 specifies a low heat cement, Type IV cement, which is rarely produced in the U.S. However, if available, Type II cement with an optional low heat of hydration of 293 kJ/kg (70 cal/g) or less at 7 days should be used. This cement's heat of hydration is generally much lower than that of either a Type I or a conventional Type II cement (Figure 13).

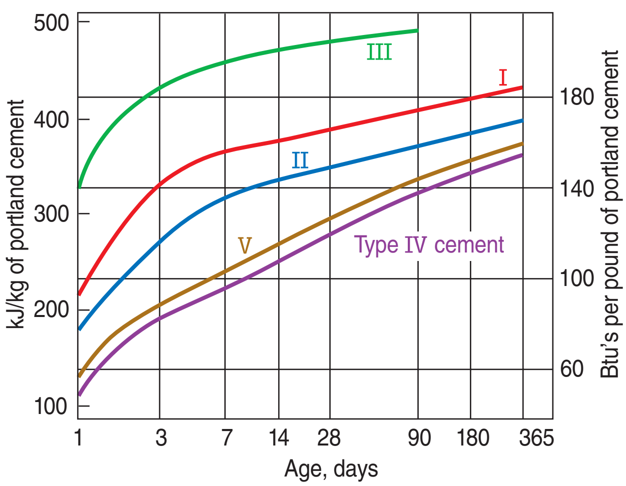


Figure 13. Heat of hydration versus cement type (Kosmatka 1988).

Some ASTM C150 Type V cements may also have lower heats of hydration values, but this is not always the case. Type V cements are designed to be highly resistant to sulfate attack and are used primarily in regions where soils or groundwater have high sulfate content. The fineness of the cement also plays a role. In some cases, where high early strength is needed, ASTM C150 Type III cement has been used in mass concrete placements. However, since the heat of hydration is proportional to the rate of strength development of the cement, use of a Type III cement may exacerbate the amount of heat generated.

All cements are unique and the values for the heat of hydration of individual components shown in Table 2 should be considered averages over a wide range of samples. There are several methods to measure the heat of hydration of the cement if a more exact value is needed, including ASTM C186 *Standard Test Method for Heat of Hydration of Hydraulic Cement,* and calorimetry (Poole 2007 and Robbins 2007). These methods can also be used to measure the heat of hydration of blended cements (ASTM C595 and C1157) or blends of portland cements and SCMs. In many cases, the effect of different water to cementitious materials ratios (w/cm) and admixture combinations can also be measured.

If measuring the heat of hydration of cement is not practical, it can be roughly estimated using the cement mill certification report. With this method, the heat energy of the cement is estimated from the weight percent of the heat energy of its individual components of C_3S, C_2S, C_4AF, and C_3A (Table 2). There may be significant errors and uncertainty in using this method (Lea 1971). If possible, it is always better to measure the heat energy of the concrete directly. Note that the use of an approximate heat of hydration estimated from individual weight percentages is not applicable for blended cements.

Supplementary Cementitious Materials

Supplementary cementitious materials (SCMs), such as fly ash or slag, are often used in mass concrete. SCMs are beneficial because they generally reduce the permeability of the concrete (making it more durable), increase the ultimate strength, and reduce the heat energy of the concrete. Other SCMs, such as metakaolin and silica fume, are often used to increase durability and strength; however, they do not reduce the heat energy of the concrete. Effects on the heat energy are shown in Table 3.

Table 2. Heat of Hydration of Individual Components (Adapted from Lea 1971)

Compound		3 days		7 days		28 days		90 days	
Actual formula	Cement notation	kJ/kg	cal/g	kJ/kg	cal/g	kJ/kg	cal/g	kJ/kg	cal/g
$3CaO\ SiO_2$	C_3S	243±33	58±8	222±46	53±11	377±29	90±7	435±21	104±5
$2CaO\ SiO_2$	C_2S	50±21	12±5	42±29	10±7	105±17	25±4	176±13	42±3
$3CaO\ Al_2O_3$	C_3A	888±117	212±28	1557±163	372±39	1377±96	329±23	1302±71	311±17
$4CaO\ Al_2O_3{\cdot}Fe_2O_3$	C_4AF	289±113	69±27	494±155	118±37	118±37	118±22	410±67	98±16

The ± value indicates the calculated probable error on the deduced value.

Table 3. Effect of SCMs on Concrete Heat Energy

SCM	Fly Ash ASTM C618 (AASHTO M 295)	Slag Cement ASTM C989 (AASHTO M 302)	Silica Fume ASTM C1240 (AASHTO M 307)	Metakaoli ASTM C618 (AASHTO M 295)
Effect on Heat Energy	Reduces heat energy. Varies depending on the dosage and the CaO content of the fly ash. The lower the CaO content, the greater the reduction.	Reduces heat energy. Varies depending on the fineness and dosage.	Little to no effect with small dosages. Increases with higher dosages.	Little to no effect with small dosages. Increases with higher dosages.

Slag Cement

Ground granulated blast-furnace slag, also called slag, is a by-product of the iron manufacturing process. Slag is a nonmetallic hydraulic cement consisting essentially of silicates and aluminosilicates of calcium. It is ground to a fine material with Blaine surface area fineness of about 400 to 600 m^2/kg. Its relative density (specific gravity) is in the range of 2.85 to 2.95. For comparison, Type I cements generally have a Blaine fineness between 330 to 430 m^2/kg and a relative density of about 3.15. In the presence of water and an activator, such as portland cement, slag hydrates much the same way portland cement does.

ASTM C989 (AASHTO M 302) classifies slag by its reactivity as Grade 80, 100, or 120. The grade is based on the activity index, which is the ratio of the 28-day mortar cube strength of a 50:50 blend of slag and portland cement to the 28-day mortar cube strength of the control portland cement. The reactivity is a function of the fineness, chemistry, and granulation method of the slag. Slag is commonly available in Grades 100 and 120 in the U.S.

Slag imparts many benefits to concrete. In the plastic state, slag can reduce bleeding, increase workability, pumpability, and finishability, and delay the time of set. Depending on the temperature of the concrete and the proportion of slag in the concrete mixture, delaying the time of set using slag can be a significant benefit. In the hardened state, slag reduces permeability (increases durability), increases ultimate strength, and reduces the potential for distress due to alkali-silica reactivity (ASR) and DEF. Because of these benefits, slag is used in quantities of up to 50% of the cementitious materials content in general purpose concrete (Luther 1998).

In mass concrete placements, slag commonly makes up 50% to 75% of the cementitious materials. However, ACI 318 limits slag cement to 50% of the total cementitious materials in placements that will be exposed to deicing chemicals.

When used in sufficient quantities (>50%), slag reduces the temperature rise of concrete. The amount of reduction is based on the proportion of slag in the concrete. Different combinations of slag cement and portland cement will behave differently.

Fly Ash

Fly ash, a by-product of the combustion of pulverized coal in electric power generating plants, is the most widely used SCM in concrete. Fly ash is primarily a silicate glass containing silica, alumina, iron, and calcium. Its minor constituents include magnesium, sulfur, sodium, potassium, and carbon. Crystalline compounds are present in small amounts. The surface area is typically 300 to 500 m^2/kg, although some fly ashes can have surface areas as low as 200 m^2/kg and as high as 700 m^2/kg. The relative density (specific gravity) of fly ash generally ranges between 1.9 and 2.8.

ASTM C618 (AASHTO M 295) Class F and Class C fly ashes are commonly used in general purpose concrete. Class F fly ashes are lower in calcium (less than 10% CaO), with carbon contents usually less than 5%, but some may be as high as 10%. Class C fly ashes are higher in calcium (10% to 30% CaO), with carbon contents less than 2%. Some fly ashes meet the requirements of both Class F and Class C classifications. Fly ash may be processed to remove the unburned carbon.

Fly ash provides benefits to the fresh and hardened properties of concrete similar to that of slag. In general purpose concrete, Class F fly ash is often used at dosages of 15% to 25% by mass of cementitious material. Class C fly ash is used at dosages of 15% to 40% by mass of cementitious material. ACI 318 limits fly ash to 25% of the total cementitious materials in placements that will be exposed to deicing chemicals.

In mass concrete placements, Class F fly ash is sometimes used at dosages as high as 35% to 50% of the cementitious materials. Higher proportions have been used experimentally, but significant prequalification would be required to ensure that durability and structural properties could be routinely achieved (Marceau 2002).

Class F fly ash typically reduces the heat of hydration by approximately 50% at all ages (Gajda 2002). This value is an accepted approximation. Since all cements and fly ashes are different, if specific data is needed, then the heat of hydration should be measured as discussed under **Cement Types**. Class C fly ash is less commonly used in mass concrete because its heat of hydration is higher than that of Class F. The heat of hydration of Class C fly ash depends on the CaO content. For low-CaO fly ashes, the heat of hydration may be similar to that of Class F fly ash; however for higher-CaO fly ashes, the heat of hydration may be similar to that of cement.

■ Silica Fume

Silica fume is a processed by-product of silicon or ferrosilicon alloy manufacture. Silica fume is essentially silicon dioxide (usually more than 85%) in a noncrystalline (amorphorous) form. It has a surface area of about 20,000 m^2/kg and a relative density generally in the range of 2.2 to 2.5.

Silica fume is primarily available in a densified powder form and is used in amounts ranging between 5% and 10% by mass of the total cementitious material. It is used in applications where a high degree of impermeability is needed and in high-strength concrete. Silica fume must meet the requirements of ASTM C1240 (AASHTO M 307).

Silica fume typically has a heat of hydration that is equal to or somewhat greater than that of portland cement depending on the dosage level. Unless specific testing is done to determine the heat of hydration, an acceptable approximation is that the heat of hydration of silica fume is 100% to 120% that of portland cement (Pinto 1999).

■ Metakaolin

Metakaolin is an engineered material that is obtained by calcining pure kaolin (clay), a natural pozzolan. Natural pozzolans are classified by ASTM C618 (AASHTO M 295) as Class N pozzolans. The fineness of metakaolin falls within that of fly ash and silica fume. The ground product is highly reactive and provides benefits similar to that of silica fume. The relative density of metakaolin is about 2.6. Metakaolin is typically used at dosages of 5% to 15% of the cementitious materials. It is used in special applications where very low permeability (high durability) and/or very high strength are needed. Metakaolin also mitigates the effects of ASR.

Typically, metakaolin has a heat of hydration equal to or somewhat greater than that of portland cement. A good estimate is that the heat of hydration of metakaolin is 100% to 125% that of portland cement. To get a more exact determination of the heat of hydration, specific testing should be done.

Aggregates

For most mass concrete placements, a local ready mix supplier will provide the concrete. The choice of available aggregates sources, sizes, and gradations may be limited. The reinforcing steel spacing and layout may add further limitations to the size of aggregates that can be used (see section on **Reinforcement Spacing**).

If different sources of aggregates are available, the use of aggregates with a lower coefficient of thermal expansion can be beneficial, as shown in Table 4. The use of an aggregate with a lower thermal expansion reduces the thermal volume change of concrete, which also reduces thermal stresses and the likelihood of thermal cracking.

Table 4. Effect of Aggregate Type on the Thermal Expansion of Concrete

Aggregate type (from one source	Coefficient of expansion, millionths per °C	Coefficient of expansion, millionths per °F
Quartz	11.9	6.6
Sandstone	11.7	6.5
Gravel	10.8	6.0
Granite	9.5	5.3
Basalt	8.6	4.8
Limestone	6.8	3.8

Coefficients of concretes made with aggregates from different sources may vary widely from these values, especially those for gravels, granites, and limestones (Davis 1930).

Chemical Admixtures

Most mass concrete mixtures utilize admixtures to reduce water content, increase slump, delay setting time, and/or entrain air. Other admixtures may also be included, depending on the requirements of the concrete.

To prevent bleeding and segregation, mass concretes typically have a low water content. This produces a low-slump concrete, typically 50 mm (2 in.). High-range water reducing admixtures (superplasticizers) are used to make these concretes placeable. With the use of high-range water reducers, slumps of 175 mm to 225 mm (7 in. to 9 in.) are commonplace. When self-consolidating concrete (SCC) is used, slump flows of 500 mm to 700 mm (20 in. to 28 in.) are common. SCC has been used in many mass concrete placements, including the footings of the San Francisco-Oakland Bay Bridge and the foundation of the Trump Tower in Chicago (Baker 2006). Additional requirements are needed for SCC, the admixture manufacturer or a qualified design professional should be consulted in regards to proportioning the concrete mixture.

Water-reducing admixtures can also be used to reduce the amount of cementitious material. This has the benefit of reducing the heat energy of the concrete without reducing the compressive strength.

Admixtures that retard the setting time can be beneficial for large placements, such as footings and foundations, to reduce the likelihood of cold joints. Retarders are also commonly used in drilled shafts, where the time of set is often specified to be twice the time of placement. In smaller placements, retarders have sometimes been used to reduce the temperature rise of the concrete. It should be noted in larger mass concrete placements retarders have little to no effect on reducing the temperature rise of the concrete.

Air-entraining admixtures provide durability for concrete that will be exposed to freezing and thawing conditions. Entrained air also reduces segregation and bleeding. However, air entrainment will reduce the concrete's overall compressive strength. As a rule of thumb, for every percent of entrained air above 4%, the concrete compressive strength will be reduced approximately 5% to 6%. Additional cementitious materials may be used to increase strength, but this will increase the heat energy of the concrete. As an alternative, the w/cm ratio can be reduced.

Concrete Temperature

Initially, the temperature of concrete as placed in the forms is about 3°C to 6°C (5°F to 10°F) greater than the average daily ambient air temperature. The effects of pumping (concrete) are included in this estimate. Somewhat lower concrete temperatures may be expected during early predawn hours, and somewhat higher concrete temperatures may occur during the afternoon.

The initial temperature of the concrete as placed can have a significant effect on the maximum temperature of the concrete after placement. For concrete mixtures with a high cementitious material content, or placements with a minimum dimension greater than 2 m (6 ft), every 1° change in the initial concrete temperature has an equivalent effect on the maximum concrete temperature. For placements with smaller minimum dimensions, or concrete mixtures with less cementitious material, the effects on the maximum temperature may be somewhat reduced.

Because of this relationship, precooling of the concrete mix can be a very effective way of reducing the maximum concrete temperature after placement. Precooling is also beneficial because it reduces the water demand and rate of slump loss, and extends the setting time.

Precooling can be accomplished by cooling individual concrete materials (prior to mixing) or precooling the concrete mixture (after mixing). Aggregates, which make up about 60% to 75% of the total volume of concrete, can be precooled by several methods, including evaporative cooling and shading. Such methods can reduce aggregate temperatures by about 6°C (10°F). Unfortunately, these methods are not always practical for ready mix plants in urban environments, where limited space is available.

Of all the materials in concrete, water is the easiest to cool. Mix water can be precooled to as low as 1°C (33°F) by refrigeration. In most cases, the use of chilled water can be expected to reduce the initial concrete temperature by about 3°C (5°F).

When additional precooling is needed, ice can be substituted for part of the mixing water. Ice reduces the temperature of the mixing water, and also lowers the concrete temperature by extracting heat (heat of fusion) during the phase change from ice to water. Ice is substituted for mixing water on a mass for mass basis, that is, 1 kg (1 lb) of ice replaces 1 kg (1 lb) of mixing water.

The ice used to cool concrete can be flaked, crushed, or cubed, and can be added directly to the ready-mix truck mixer, stationary, or central mixer. When a large project requires large quantities of ice, a dedicated ice production facility may be justified. In most cases, ice is used for up to

80% of the total mixing water. The actual amount of cooling depends on the mix design, but ice is typically expected to reduce the concrete temperature by up to about 11°C (20°F). All of the ice must be completely melted before the concrete is discharged at the jobsite. For large placements, ice may need to be ordered weeks or even months ahead of time.

The approximate temperature of concrete when using ice or chilled water in a particular mixture can be determined through use of the following equation:

$$T\ (°C) = \frac{0.22(T_a M_a + T_c M_c) + T_w M_w + T_{wa} M_{wa} - 80 M_i}{0.22\,(M_a + M_c) + M_w + M_{wa} + M_i}$$

$$T\ (°F) = \frac{0.22(T_a M_a + T_c M_c) + T_w M_w + T_{wa} M_{wa} - 112 M_i}{0.22\,(M_a + M_c) + M_w + M_{wa} + M_i}$$

where: T_a, T_c, T_w, and T_{wa} are the temperatures of the aggregates, cementitious materials, mixing water, and free water on the aggregates, respectively, and M_a, M_c, M_w, M_{wa}, and M_i are the mass in kg (lb) of the aggregates, cementitious materials, mixing water, free water on the aggregates, and ice, respectively (Kosmatka 2002). When using this equation with estimated temperatures, it should be noted that the temperature of the cement is often about 15°C to 40°C (30°F to 70°F) warmer than the average ambient air temperature.

If further precooling is needed, liquid nitrogen can be injected directly into the drum of a ready mix truck (Figure 14). The temperature of liquid nitrogen is -195°C (-320°F). There are several advantages to using liquid nitrogen for cooling concrete: 1) less personnel are required for cooling, 2) concrete can be cooled to any temperature as low as about 1°C (33°F), and 3) the concrete can be recooled multiple times, if needed. Its drawbacks are: 1) it can be expensive, 2) cooling with liquid nitrogen can be dangerous and requires the use of highly trained personnel, 3) the drum of the truck mixer must be spinning at full speed and must be fully loaded, and, 4) it requires advanced planning. A specialty subcontractor is usually hired to cool the concrete with liquid nitrogen. An infrared temperature gun, aimed at the lower portion of the exterior of the drum, is often used to measure the concrete temperature.

(IMG25516)

Figure 14. Liquid nitrogen precooling of concrete.

Curing Methods

Water curing is sometimes specified for mass concrete. Typically, this is inappropriate because it artificially cools the surface only, which increases the likelihood of thermal cracking. When water curing is specified, heated water must be used. The temperature of the curing water should be high enough that it does not cool the concrete. ACI 308R states that the temperature of the curing water should be within 11°C (20°F) of the concrete temperature. This can be dangerous and costly when the interior of the mass concrete is anticipated to be near 70°C (158°F).

In most cases, water retention curing methods are more appropriate for mass concrete. Such methods include, form curing, membrane curing, and the use of a curing compound.

Insulation

While it may seem counterintuitive to insulate mass concrete, insulation is actually used on most mass concrete placements (Figures 15 and 16). Insulation slows the escape of heat, which warms the concrete surface and reduces the temperature difference between the surface and the interior. When the minimum dimension of the concrete is over about 1.5 m (5 ft) thick, the use of insulation has virtually no effect on the maximum concrete temperature.

Insulation with an R-value in the range of 0.35 to 0.9 $m^2 \cdot K/W$ (2 to 5°F hr ft^2/Btu) is typically used (Gajda 2006 and ACI 207.4R). Additional insulation may be needed in northern climates during winter placements.

Many materials may be used as insulation. Typically, concrete insulating blankets are used to cover finished surfaces and

(IMG25517)

Figure 15. Insulation on bridge pier base placements; extruded polystyrene on the far pier base (left) and insulating blankets and extruded polystyrene on the near pier base (right).

(IMG25518)

Figure 16. Concrete insulating blankets on a column.

formwork. For repetitive placements, formwork is sometimes coated with spray-applied polyurethane closed-cell foam insulation, or built of sandwich construction with high-density plywood inside, rigid polystyrene in the middle, and rough plywood outside.

Insulation may be required for several weeks or longer, especially on thicker placements. During this time, it may be possible to temporarily remove insulation to perform other work. This can be done during the window of time when the temperature difference between the interior of the placement and the surface is less than the specified temperature difference limit.

Heat Removal Methods

Sometimes removing heat from the concrete prior to placement is neither practical nor possible. Heat can be removed from the concrete after it is placed but it requires appropriate planning and preparation. Removing insulation only cools the surface, which increases the temperature difference from center to surface of the concrete. There are methods available to safely remove heat from concrete after placement including cooling pipes and use of lifts.

Cooling Pipes

Internal cooling pipes can be used to remove heat from the interior of a concrete placement, reducing the maximum concrete temperature, the temperature difference, and/or the period of time that surface insulation is required. The cooling pipe system can be designed to accommodate any or all of these needs.

Cooling pipes typically consist of a uniformly distributed array of 19 mm to 25 mm (¾ in. to 1 in.) diameter pipes embedded in the concrete as shown in Figure 17. Plastic pipes are typically used because they are less expensive to purchase and install than steel pipes. A pipe spacing of 1 m (3 ft) center-to-center is typical. Closer pipe spacing is sometimes used to more rapidly remove heat from the concrete. This can cool the concrete more rapidly or reduce the maximum temperature in the concrete.

(IMG25519)

Figure 17. Cooling pipes preinstalled in a column cage.

Pipes must be secured properly so that they are not damaged during the concrete placement. Prior to concrete placement, the pipes should be tested for leaks. During placement, the pipes should be filled with water to reduce the potential for floating, and repair materials should be onsite to quickly fix any damage that may occur. Shut-off valves are recommended to isolate an individual pipe in the event of damage.

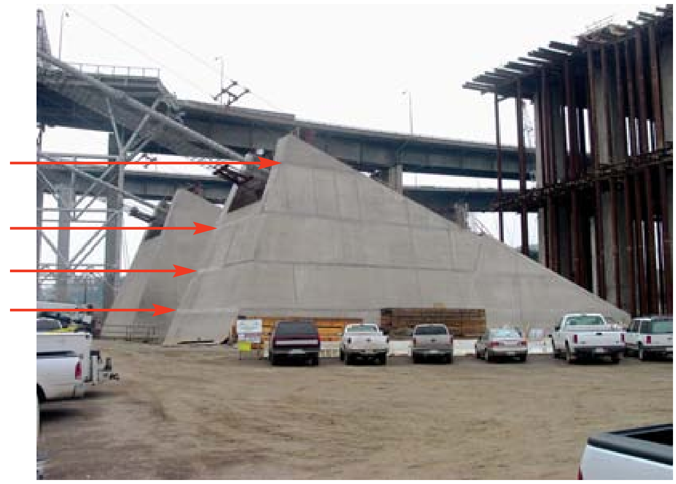
Mass concrete placements can contain vast amounts of internal heat; therefore, cooling pipes are most costeffective when a large supply of water is available, such as a nearby lake, river, or ocean. When these sources of water are not available, a chiller system can be set up to moderate the temperature of the cooling water.

The cooling system should be designed and operated so that the concrete is uniformly cooled. When this is done, large placements can be safely cooled at rates that exceed 5°C (10°F) per day. Attention should be paid to the temperature of water in the pipes to ensure uniform cooling. If the cooling is not uniform, some portions of the placement may cool more rapidly than others, and this can result in excessive differential thermal movement and cracking.

Use of Lifts

Large mass concrete placements are often divided up into several smaller placements by using lifts. This is done to make the overall placement more manageable, and/or as a way to help cool the concrete. Placements with a smaller minimum dimension are quicker to cool and may have a lower maximum temperature.

The massive bridge anchorage shown in Figure 18 was placed as a series of 3 m (10 ft) thick lift placements to reduce the cooling time of the concrete. The visible construction joints highlighted by arrows show the limits of each placement.



(IMG25520)

Figure 18. Bridge anchorage constructed with multiple placements (lifts are highlighted by arrows).

To be effective, this approach requires careful planning. Depending on the heat energy of the concrete and size of the placement, significant time may be needed between successive lifts. If insufficient time is allotted, cooling of the individual lifts may be minimal. Additionally, when placing cooler concrete on existing hotter concrete, thermal cracking of the existing concrete may occur at the construction joint and the new concrete may not fully bond to the existing concrete.

Chapter 4 – Thermal Control Plan

The thermal control plan is the most important submittal item for a mass concrete specification. The following is a generalized example of a thermal control plan. The purpose of this example is to demonstrate the type of information that is typically contained in a thermal control plan. Each thermal control plan must be tailored to the specific requirements of the specifications and project.

Thermal Control Plan Overview

A thermal control plan is similar to a quality control plan. The purpose of a thermal control plan is to document the means and methods used to ensure that cracking of the mass concrete placement (both during and after the time of construction) will be avoided and its long term durability will not be compromised. This is accomplished through the control of temperatures and temperature differences within the concrete – from the time of placement through the time that the concrete has adequately cooled to near-ambient conditions (or to the point that damage from thermal stresses is no longer a possibility).

The maximum temperature is typically limited to prevent delayed ettringite formation, but may also be limited so that durability and structural properties are not adversely affected. The temperature difference is limited to prevent or minimize thermal cracking. The thermal control plan should address all aspects of the placement(s) from initial placement of concrete through curing and formwork removal. The following is an overview of the key elements of the thermal control plan:

A. Mix Design

State the mix design name and number. Attach the mix design to the thermal control plan.

B. Concrete Placements

State the mass concrete placements addressed by the thermal control plan.

C. Temperature Ranges

State the range of concrete placement temperatures covered by the thermal control plan.

D. Thermal Modeling

Briefly discuss the method used to predict temperatures and temperature differences.

E. Methods for Limiting the Maximum Temperature

Specifications generally require that the maximum concrete temperature (after placement) be limited so that it does not exceed a predetermined limit such as 70°C (158°F). State whether the mass concrete placement will exceed recommended maximum temperature limit. If the maximum temperature limit will be exceeded without thermal controls, then state in simple terms how the maximum temperature will be controlled so that the temperature does not become excessive.

F. Methods for Limiting the Maximum Temperature Difference

Specifications require that the temperature difference be controlled. Sometimes a particular value (such as 20°C [35°F]) is specified, while other times a performance-based criteria (of no or limited cracking) is specified. State the temperature difference limit that will be used. State whether this maximum temperature difference limit will be exceeded. If this maximum temperature difference limit will be exceeded, then state in simple terms how the temperature difference will be controlled so that it does not become excessive.

G. Temperature Monitoring

Temperature monitoring is required to verify that the temperature and temperature difference are not excessive. Briefly describe the details of the temperature monitoring system, the equipment, its operation, and the reporting of data.

H. Corrective Measures

Briefly state what will be done if temperatures or temperature differences are excessive, or if cracking of the mass concrete placement occurs.

Specified Temperature Limits

Reiterate the project mass concrete specifications. At a minimum, state the specified maximum temperature, the specified temperature difference limit, and the specified length of time that the limits are in force. Also state any other specifications that will affect the mass concrete.

If the specifications require that the contractor propose a temperature difference limit for the project concrete (to prevent thermal cracking, or to limit cracks to prescribed widths), fully describe the proposed temperature difference limit.

Concrete Placements

Describe the mass concrete placement(s) addressed by this thermal control plan. State the dimensions, number of placements, locations. Discuss the construction joint locations and timing of placement of new concrete on existing mass concrete. Simple sketches or drawings of the mass concrete placements would be helpful to include in this section. References to project drawings including column lines and project elevations would also be helpful.

Concrete Mix and Properties

State the concrete mix design, including the supplier and mix name/number. State the source, type, and quantities of cementitious materials. State the size and type of aggregates.

State the adiabatic temperature rise (ATR) of the concrete and indicate whether this is estimated or measured. If the ATR is estimated, then state the assumption method and data used for the assumptions. If the ATR was measured, then briefly describe the test method used.

If the ATR is used in any thermal modeling, provide mechanical and thermal properties of the concrete. A partial list *may* include properties such as thermal diffusivity, compressive strength versus age, elastic modulus, tensile strength, and coefficient of thermal expansion.

Thermal Modeling

Thermal modeling may use simple methods or specifically designed software to predict temperatures and temperature differences in the mass concrete. Discuss the thermal modeling method, and the thermal control measures that were included in the modeling. Thermal control measures (called "thermal controls") may include precooling the concrete, insulating the formwork, and post cooling with internal cooling pipes.

Temperature Assumptions

State the range of temperature assumptions (for the delivered concrete and the surrounding environment) used in the modeling. At a minimum, the assumptions should include the concrete temperature at the time of placement and the air temperature during placement and during the thermal control period.

If the concrete is placed against existing concrete, soil/subgrade, sheet pile, water, or any other material that will affect its internal or surface temperature, then state the range of assumed temperatures of the surrounding material(s). If cooling pipes will be used, then state the assumed temperature of the coolant in the pipes.

Cooling Pipe Assumptions

If cooling pipes are to be used, state the cooling pipe size, layout, spacing, and material. State the modeling assumptions such as when the cooling pipes will be turned on, when they will be turned off, and the temperature of the coolant. State any other assumptions used.

Insulation Assumptions

If insulation is to be used, state the type of insulation that will be used, the surfaces that will be insulated, and the R-value of the insulation. State when the insulation will be installed and when it will be removed. State any other assumptions used.

Details of the Thermal Modeling

Provide any additional details of thermal modeling. State any other assumptions used.

Results of the Thermal Modeling

Provide results of the thermal modeling. If results are extensive it is suggested that they be presented in an appendix, and that a summary of the results be presented in this section. Also consider including example charts with explanatory notes showing the results of thermal modeling for typical placements.

Discuss specific measures and recommendations for thermal control to ensure that temperatures and temperature differences will not be excessive.

Measures For Thermal Control (During Construction)

Discuss specific means and methods for thermal control that will be used to ensure that temperatures and temperature difference will not be excessive, and thermal cracking will be avoided.

Placement of Concrete

Discuss the method of concrete placement (pump, chute, bucket, or conveyor).

Concrete Placement Temperature

From the thermal modeling, discuss the maximum temperature of the concrete at the time of placement. State that concrete above this temperature will be rejected.

Curing

Some curing methods artificially cool the surface of the concrete which can cause cracking. Discuss the curing method that will be used and the length of curing period. Compare this to the specified curing method.

Installation of Insulation

Many materials can be used as insulation, including some types of formwork. Detail the type of insulation that will be used, its R value, when it will be installed, and to what surfaces it will be installed. If insulation is to be temporarily removed, state guidelines that will be followed to ensure that the temperature difference will not become excessive. State when the insulation will be permanently removed.

Cooling Pipe System and Operation

If cooling pipes are to be used, state the cooling pipe size, layout, spacing, and pipe materials. Drawings that detail the installation must also be included. Discuss the operation of the cooling pipe system, such as when it will be turned on and when it will be turned off, and how uniform cooling will be ensured. Discuss measures to test for leaks prior to placement, and indicate how leaks will be fixed, if they occur during placement.

Measurement and Termination of Thermal Control

Temperatures and temperature differences must be monitored to ensure that they are not excessive. The purpose of this section is to describe temperature monitoring, reporting, and when thermal control is no longer required.

Temperature Monitoring

Describe the equipment that will be used for measuring temperatures. Provide specifications or “cut sheets” for this equipment. Discuss its installation and operation. Discuss how proper operation of this equipment will be determined.

Discuss where temperatures will be monitored and the frequency of monitoring. Provide detailed drawings showing where temperatures will be monitored. Discuss what will be done to remove or replace a temperature sensor that is damaged by the placement of concrete.

Discuss the reporting of temperature data, including the format (tabular and/or graphical), and when it will be provided to the Owner/Engineer.

Completion of Thermal Control

Thermal control is typically complete when the portion of the concrete with the highest recorded temperature cools sufficiently enough that thermal cracking is no longer a concern (even if insulation is removed and cooling pipes are turned off). After this time, temperature monitoring, cooling pipes, and insulation are no longer required. Cooling pipes are typically filled with a non-shrink grout. Describe the criteria that will be used to determine when thermal control is complete.

Corrective Measures

Describe what will be done if it appears that the temperature will closely approach or exceed either the specified maximum temperature or the specified maximum temperature difference or, alternately, what will be done if cooling pipes or the temperature monitoring system fails. Other job-specific corrective measures should also be included.

Summary

Briefly reiterate the major points of the thermal control (these same points should be noted as items A through H at the front of the thermal control plan). Provide contact information for the thermal control plan author, so that he/she can be contacted with questions that may arise during review and construction.

Attachments

Attach the following items to the end of the thermal control plan:

1. concrete mix design;
2. detailed results of thermal modeling for the range of placement conditions;
3. drawings for the cooling pipe layout (if used);
4. drawings for the temperature monitoring system layout;
5. temperature monitoring equipment cut sheets.

This is an example of a thermal control plan, which is a submittal document that details measures that will be used to control temperatures and temperature differences within mass concrete so that cracking will be avoided and durability will not be compromised.

Chapter 5 – Temperature Prediction

Many methods exist to predict temperatures and temperature differences in a concrete placement. These methods range from simplistic to highly refined and accurate thermal models. In each of these methods, the maximum temperature in the concrete is the initial temperature of the concrete plus the adiabatic temperature rise minus the heat loss to the surroundings. The adiabatic temperature rise is a measure of the heat energy in the concrete if no heat is lost to the surroundings. The differences in the various methods are based on how the temperature rise and heat loss to the surroundings are handled.

Simplistic Method For Total Temperature Rise

One very simple way to estimate the maximum temperature of a mass concrete placement is to ignore the early age heat loss to the surroundings. This method is applicable to placements that are over 2 m (6 ft) thick. In placements of this size, the maximum temperature is reached before the effects of heat loss become noticeable. In this approach, the maximum temperature is the initial temperature plus the adiabatic temperature rise.

The adiabatic temperature rise of concrete is estimated by converting the cementitious content of the concrete to an "equivalent cement content;" this is then multiplied by a factor to provide the adiabatic temperature rise of the concrete in degrees. When metric units are used, the equivalent cementitious content is given in kg/m^3, the factor used is 0.13, and the temperature rise is in degrees Celsius. When U.S. customary units are used, the equivalent cementitious content is given in lb/yd^3, the factor used is 0.14, and the temperature rise is in degrees Fahrenheit (Gajda 2006). Thinner placements will have a somewhat lower temperature rise.

To approximate the "equivalent cement content" of a concrete with Type I or Type II cement:

1 kg/m^3 (lb/yd^3) of cement is equated with 1 kg/m^3 (lb/yd^3) of cement;

1 kg/m^3 (lb/yd^3) of Class F fly ash is equated with 0.5 kg/m^3 (lb/yd^3) of cement;

1 kg/m^3 (lb/yd^3) of Class C fly ash is equated with 0.8 kg/m^3 (lb/yd^3) of cement;

1 kg/m^3 (lb/yd^3) of silica fume or metakaolin is equated with 1.25 kg/m^3 (lb/yd^3) of cement;

1 kg/m^3 (lb/yd^3) of slag (50% cement replacement) is equated with 0.9 kg/m^3 (lb/yd^3) of cement; and

1 kg/m^3 (lb/yd^3) of slag (75% cement replacement) is equated with 0.8 kg/m^3 (lb/yd^3) of cement.

The adiabatic temperature rise of the concrete can also be measured directly using specially designed laboratory equipment, or it can be calculated from sufficiently large insulated mockup placements. It can also be calculated from heat of hydration or calorimetric testing, or by using data in Table 2 with the following equation:

$$T = cH/S$$

Where c is the total cementitious material content of the concrete in kg/m^3 (lb/yd^3) divided by the density (unit weight) of the concrete in kg/m^3 (lb/yd^3); H is the heat of hydration in kJ/kg (cal/g) at any particular age; and S is the specific heat of the concrete, typically assumed to be 0.92 kJ/kg/°C (0.22 Btu/lb/°F).

The adiabatic temperature rise is affected by the temperature of the concrete, as illustrated in Figure 19. Warmer temperatures accelerate the hydration of the cementitious materials in concrete. The reverse is true of cooler temperatures. The effects of concrete temperature can be quantified through the maturity method, which is described in the section on **Fast-Track Construction**. Adding the adiabatic temperature rise to the initial concrete temperature provides a prediction for the maximum concrete temperature in the concrete.

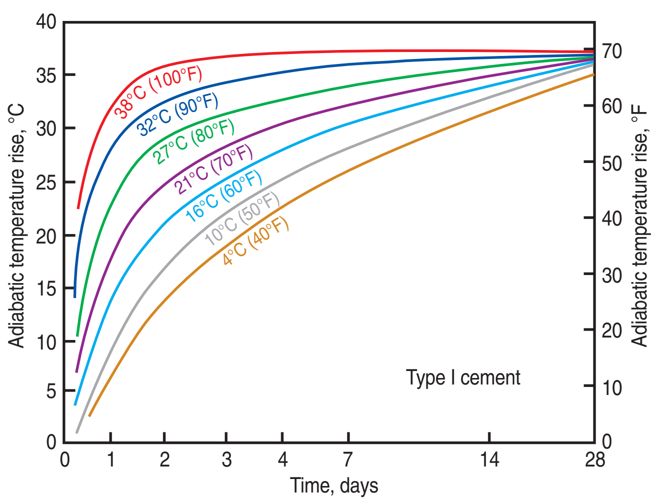


Figure 19. The effect of concrete-placing temperature on temperature rise in mass concrete with 223 kg/m³ (376 lb/yd³) of cement. Higher placing temperatures accelerate temperature rise (ACI 207.2R).

In most cases, the concrete can be expected to be at or near its maximum temperature within 1 to 3 days after placement. The actual time will depend on factors such as the type of cementitious materials and the quantity of SCMs. Depending on the minimum dimension of the placement, the concrete may not begin to cool until several days later. To estimate when insulation can be permanently removed, assume that a 2 m (6 ft) thick concrete placement will cool at a rate of about 1°C to 1.5°C (2°F to 3°F) per day. This provides only a very rough estimate because the actual cooling rate depends on the insulation, the minimum dimension of the placement, and a host of other variables.

Schmidt Method

Another method used to predict concrete temperatures is called the Schmidt Method. This method, described in ACI 207.1R, can be used to predict the temperatures and temperature differences within a concrete placement. Timing of the maximum temperature, maximum temperature difference, and cooling rates can also be predicted. For slab-type placements, where most of the heat is lost in one dimension, this method can be easily implemented using a spreadsheet. For column-type placements, and other placements that lose heat in two dimensions (both vertically and horizontally or in both plan and elevation), this method can still be implemented using a spreadsheet, but not as easily as for a slab-type placement.

Although not discussed in ACI 207.1R, the effect of adjacent concrete can be determined if treated as non-heating concrete. The effects of surface insulation can also be estimated by modeling it as an equivalent thickness of non-heating concrete.

The keys to a reasonably accurate temperature prediction when using the Schmidt method are: (1) understanding the limitations of the method; (2) using a reasonable element size of 50 mm to 75 mm (2 in. to 3 in.); and (3) using representative adiabatic temperature rise data that is adjusted for the temperature in the concrete.

Thermal Modeling

Finite element modeling is often used to model temperatures in mass concrete placements. In most cases, specially designed modeling software is used to account for temperature dependant effects on the adiabatic temperature rise. This modeling can also predict the time and temperature-dependent properties of concrete. This is especially useful if a performance based temperature difference limit is used to prevent thermal cracking or limit cracks to "tolerable" widths (see the section on **Thermal Cracking**). Finite element modeling can also account for the effects of internal cooling pipes.

Like all temperature prediction methods, the accuracy of the modeling depends on the inputs and assumptions. Critical inputs include the adiabatic temperature rise, its temperature dependence, and the weather (if insulation is not used). If concrete properties are estimated, the temperature dependence of these properties must be included.

Chapter 6 – Construction

Planning is the key to a successful mass concrete placement. The importance of planning cannot be over-emphasized. The extent of planning depends on the size of the placement. For larger placements, additional planning is needed.

Planning

When planning for a mass concrete placement, considerations must be given to the following: site access; collection of truck-mixer batch tickets; measurement of concrete properties (compressive strength, slump, etc.); handling of concrete test samples; ensuring a continuous supply of concrete; and contingency planning for interruptions caused by; accidents, bad weather, or equipment breakdowns. Larger placements may take as much as 18 to 30 hours to complete (Figure 20). Scheduling of personnel and trades is critical.

Typically, the planning process also includes a thermal control plan as discussed in Chapter 4. This allows for preplanning so that the initial concrete temperature limit can be determined, measures to control temperatures and temperature differences can be defined, and contingency plans for dealing with excessive temperatures and temperature differences can be developed.

As part of the planning, a mockup is often desirable. Mockups can test the planned thermal control measures, ensure that the concrete is adequate for the task, and assess the thoroughness of the planning.

(IMG12289)

Figure 20. Planning for large mass concrete placements is essential.

Fast-Track Construction

Today, more than ever, construction is fast-tracked. Waiting for a mass concrete placement to adequately cool is not practical or feasible in most cases. Additionally, there may not be enough time to iteratively develop a low-heat concrete that could reduce the cooling time of the concrete.

In these cases, cooling pipes are extremely useful. A properly designed cooling pipe system can reduce the cooling time of a mass concrete placement by 50% or more. The effect of using cooling pipes is illustrated in Figure 21.

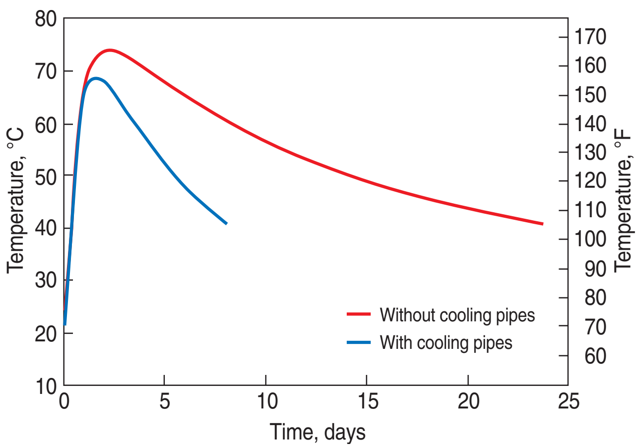


Figure 21. Rapid cooling through use of cooling pipes.

The use of maturity methods can also reduce the cooling time of concrete because these methods allow for the estimation of the in-place properties of the concrete. The strength of concrete is directly related to curing time and temperature. The benefit of maturity is that the in-place concrete strength can be predicted from simple measurements of concrete temperature taken over time. This is especially important when a performance-based temperature difference limit is used. This type of temperature difference limit ensures that the localized thermal stresses do not exceed the tensile strength of the concrete. Maturity is also useful for determining when the concrete has developed sufficient strength so that formwork can be permanently removed. The implementation of maturity in mass concrete placements is relatively easy because temperatures are typically already being monitored.

Maturity is discussed in ASTM C1074 and in various ACI documents (such as ACI 306R). There are two maturity functions needed to estimate the in-place concrete strength. The first is the Nurse-Saul function, also called the *Time-Temperature Factor*. This method is simple and very popular. However, it doesn't recognize that maturity increases disproportionately at elevated temperatures and that this increase depends on the cementitious materials and water-to-cementitious materials ratio. The result is that the Time-Temperature Factor method typically **underestimates** the strength development at elevated temperatures. The second maturity function is based on the Arrhenius equation; this function presents maturity in terms of equivalent age of curing at a specified temperature.

The *Time-Temperature Factor* method presents maturity in terms of °C•hr. Most maturity equipment uses a datum temperature of 0°C, which further simplifies the calculation. Given this simplification, maturity is typically calculated using metric units. To use maturity, a maturity to strength relationship must first be developed. This is accomplished by casting a group of cylinders using concrete identical to that used in the structure and measuring the temperature in the cylinders to calculate maturity (Figure 22). At various times, the compressive strength of the cylinders is measured and the corresponding maturity is noted. A maturity relation may be similar to that shown in Figure 23.

Figure 22. Measuring concrete temperatures to develop a maturity curve. (Courtesy of Mike Fox, Engius.)

Time-Temperature Factor $M = \sum_{0}^{t} (T - T_0)\Delta t$	M = maturity index, °C-hours T = average concrete temperature, °C, during the time interval Δt T_0 = datum temperature (usually taken to be 0°C) t = elapsed time, hours Δt = time intervals, hours
Equivalent Age $t_e = \sum_{0}^{t} e^{\frac{-E}{R}\left(\frac{1}{T_r} - \frac{1}{T}\right)} \Delta t$	t_e = equivalent age at the reference temperature E = apparent activation energy, J/mol (see ASTM C 1074 for typical values) R = universal gas constant, 8.314 J/mol-K T = average concrete temperature, Kelvin, during the time interval Δt T_r = absolute reference temperature, Kelvin Δt = time intervals, hours

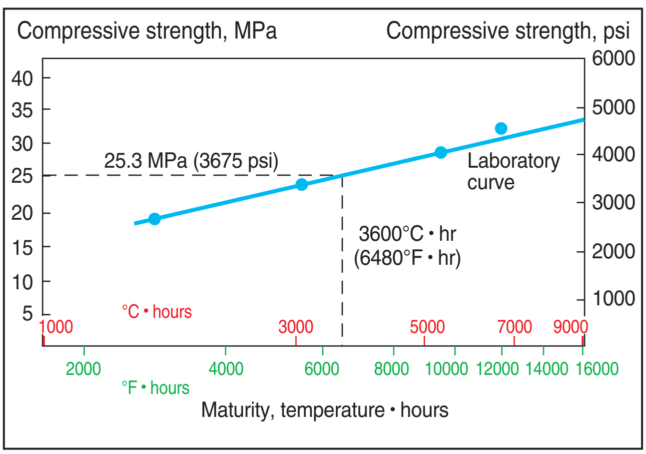


Figure 23. Maturity relation example.

Temperature Monitoring

Temperature monitoring of the mass concrete placement is required to ensure that the maximum temperature limit and the temperature difference limit are not exceeded. Monitoring also provides useful information to help guide thermal control efforts.

Temperatures can be monitored by hand or with automated equipment (Figure 24). When monitored by hand, temperatures should be recorded at least every 4 hours and more frequently during the first few days when concrete temperature is changing rapidly. Because hand monitoring is labor intensive, automated equipment is typically used with hourly temperature recording. Regardless of the method used, temperatures must be monitored throughout the entire placement and until thermal control is complete. Thermal control is complete when the hottest portion of the concrete cools to within the temperature difference limit of the ambient temperature. This requirement is necessary to prevent thermal shock.

Figure 24. Temperature monitoring. (Courtesy of Mike Fox, Engius.)

In a mass concrete placement that is uniformly insulated, temperatures should be monitored in at least two locations: (1) The location in the concrete anticipated to reach the highest temperature, which is generally at the geometric midheight, midlength, and midwidth of the placement. If cooling pipes are used, this location would be at the center of the location with the fewest number of pipes. (2) The center of the exterior or exposed surface closest to the first location, at a depth of 50 mm to 75 mm (2 in. to 3 in.) inside the surface. Placements that are not uniformly insulated or that utilize a non-uniform cooling pipe layout will normally require additional temperature monitoring locations. Temperatures should also be monitored at surfaces that will later be covered by additional concrete, such as the top surface of the individual placements shown in Figure 18.

A variety of temperature monitoring equipment is available, including specially designed proprietary systems and home made systems assembled from separately purchased off the shelf components including thermocouples and thermocouple loggers. Thermocouple loggers are generally designed for use indoors; if used outdoors, special enclosures are needed to isolate the loggers from the effects of temperature and weather changes. In general, systems designed for these specific applications are more reliable.

Chapter 7 – Case History – San Francisco Oakland Bay Bridge

The mass concrete footing placements for the San Francisco–Oakland Bay Bridge signature span present a practical, yet impressive, example of thermal control measures for mass concrete construction. Each of the footings measured about 19 m x 19 m (62 ft x 62 ft) in plan and 10 m (33 ft) thick (Figure 25).

(IMG25522)

Figure 25. Partially completed excavations for the "W2."

Specifications for the Oakland Bay Bridge called for the "W2" footings to be treated as mass concrete and required that a thermal control plan be developed to control maximum temperatures and temperature differences in the footing concrete. Concerns with thermal shrinkage, and the possibility that the footings could become debonded from the rock during an earthquake, ultimately determined that a maximum temperature limit of 50°C (122°F) be specified. A specific temperature difference limit was not specified, but the specifications did require that thermal modeling be performed to determine an acceptable temperature difference limit to prevent thermal cracking.

Although the foundations were fast-tracked, adequate time was available to optimize the concrete mix design. Project specifications required a relatively high cementitious material content of about 400 kg/m^3 (675 lb/yd^3) for both durability and service life. The selected concrete had a relatively low adiabatic temperature rise of 44°C (79°F) at 28 days, and a relatively high compressive strength of 41 MPa (6000 psi) at 21 days, and exceeded 55 MPa (8000 psi) at 56 days. The concrete also had a relatively low coefficient of thermal expansion of 8 millionths per °C (4.4 millionths per °F). These properties helped reduce the efforts needed to limit maximum temperature and temperature differences.

A number of progressive measures were utilized to reduce the time and cost of construction. A performance-based temperature difference limit (similar to that shown in Figure 7) was developed and used. This limit ensured that thermal cracking did not occur and reduced both the time that insulation was required and the efforts needed to control temperature differences. To reduce construction time further, a maturity-related procedure accounted for the increased rate of compressive strength development that occurs at elevated temperatures. Cooling pipes were also employed to rapidly remove internal heat and reduce the cooling time of the concrete (Figure 26).

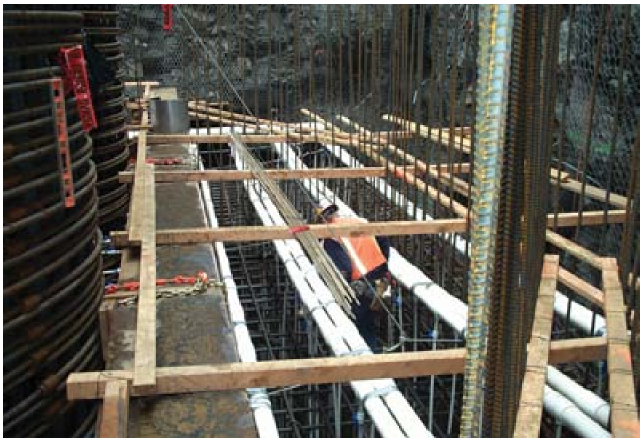


Figure 26. Cooling pipes were an effective method of reducing the concrete's internal heat.

The 50°C (122°F) maximum temperature limit meant that the footing concrete would require precooling prior to placement. Liquid nitrogen cooling was used because it did not reduce the hourly supply of concrete. It was available in sufficient quantities to adequately cool the anticipated 550 truckloads of ready-mixed concrete needed for each footing. Precooling was performed at the jobsite to ensure a 10°C (50°F) concrete temperature at the time of placement (Figure 27).

Figure 27. Five liquid nitrogen cooling stations were set up to provide a consistent flow of concrete.

Figure 28 shows the recorded temperatures in the first footing. Note that the maximum concrete temperature after placement was about 40°C (104°F), which is well below the specified 50°C (122°F) limit. The temperature difference was also well below the calculated allowable temperature difference. Temperature monitoring was discontinued after 7 days. By this time the concrete had sufficiently cooled so that the cooling pipes could be turned off and the insulation removed without causing thermal cracking. After removing the insulation and turning off the cooling pipes, the contractor began work on the columns.

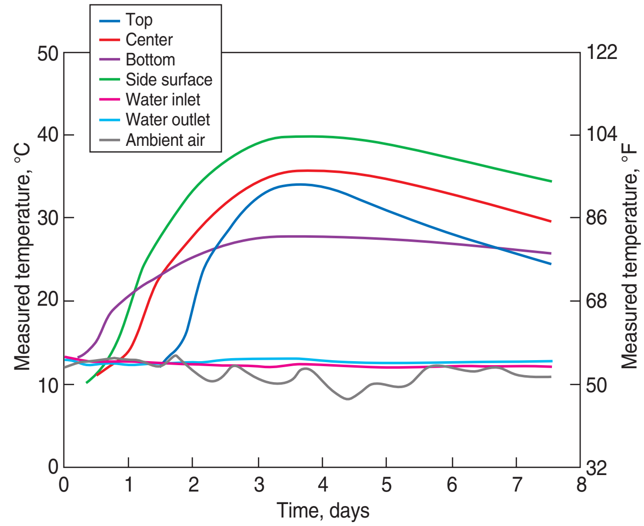


Figure 28. Measured temperatures in the first of the footings.

Both the low concrete temperature and the short cooling time are the result of using the proper concrete mix design, adhering to the performance-based temperature difference limit using maturity-related procedures, taking advantage of cooling pipes, and precooling the concrete before placement. Without these measures, an elapsed time of 100+ days would have been necessary to meet the same project requirements, instead the project was successfully completed in seven days (Figure 29).

Figure 29. View of completed footing and column from across the San Francisco Bay.

References and Related Publications

ACI Committee 207, *Mass Concrete*, ACI 207.1R-96, American Concrete Institute, Farmington Hills, Michigan, 1996, 42 pages.

ACI Committee 207, *Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete*, ACI 207.2R-95, reapproved 2002, American Concrete Institute, Farmington Hills, Michigan, 1995, 26 pages.

ACI Committee 207, *Cooling and Insulating Systems for Mass Concrete*, ACI 207.4R-05, American Concrete Institute, Farmington Hills, Michigan, 2005, 15 pages.

ACI Committee 224, *Control of Cracking in Concrete Structures,* ACI 224R-90, American Concrete Institute, Farmington Hills, Michigan, 1990, 43 pages.

ACI Committee 306, *Cold-Weather Concreting,* ACI 306R-88, reapproved 1997, American Concrete Institute, Farmington Hills, Michigan, 1997, 23 pages.

ACI Committee 308, *Standard Practice for Curing Concrete,* ACI 308-92 reapproved 1997, American Concrete Institute, Farmington Hills, Michigan, 1997, 11 pages.

ACI Committee 318, *Building Code Requirements for Structural Concrete and Commentary,* ACI 318-99, American Concrete Institute, Farmington Hills, Michigan, 1999, 369 pages.

Baker, William; Korista, Stan; Sinn, Robert; Pennings, Karl; and Rankin, Dane, "Trump International Hotel and Tower," *Concrete International*, Vol. 28, No. 7, American Concrete Institute, Farmington Hills, Michigan, 2006, pages 28 to 32.

Bamforth, P.B., "Large Pours," letter to the editor, *Concrete*, Cement and Concrete Association, Wexham Springs, Slough, England, February 1981.

Bureau of Reclamation, *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, Denver, 1981.

Burg, R.G., and Fiorato, A.E., *High-Strength Concrete in Massive Foundation Elements*, Research and Development Bulletin RD117, Portland Cement Association, Skokie, Illinois, 1999, 22 pages.

Burg, R.G., and Ost, B.W., *Engineering Properties of Commercially Available High-Strength Concretes (Including Three-Year Data)*, Research and Development Bulletin RD104, Portland Cement Association, Skokie, Illinois, 1994, 62 pages.

Concrete for Massive Structures, IS128.03T, Portland Cement Association, Skokie, Illinois, 1987, 11 pages.

Davis, R.E., "A Summary of the Results of Investigations Having to Do with Volumetric Changes in Cements, Mortars, and Concretes Due to Causes Other Than Stress," *Proceedings of the American Concrete Institute*, American Concrete Institute, Farmington Hills, Michigan, Vol. 26, 1930, pages 407 to 443.

Ettringite Formation and the Performance of Concrete, IS417, Portland Cement Association, Skokie, Illinois, 2001, 16 pages.

Farny, James A., and Panarese, William C., *High Strength Concrete,* Engineering Bulletin, EB114, Portland Cement Association, Skokie, Illinois, 1994, 56 pages.

FitzGibbon, Michael E., "Large Pours for Reinforced Concrete Structures," Current Practice Sheets No. 28, 35, and 36, *Concrete*, Cement and Concrete Association, Wexham Springs, Slough, England, March and December 1976 and February 1977.

Gajda, J., and VanGeem, M., "Controlling Temperatures in Mass Concrete," *Concrete International*, Vol. 24, No. 1, American Concrete Institute, Farmington Hills, Michigan, January 2002.

Gajda, J., and Alsamsam, I.M., "Engineering Mass Concrete Structures," *Structural Engineer*, November 2006, 7 pages (www.gostructural.com/se-pdh).

Gajda, J.; Kaufman, A.; and Sumodjo, F., "Precooling Mass Concrete," *Concrete Construction*, August 2005, pages 36 to 38.

Gajda, J., "Great Thickness Requires Great Lengths," Roads and Bridges, January 2005, pages 34 to 37.

Gajda, J., and VanGeem, M.G., "A New Concept in Bridge Construction: Mass Concrete," *Concrete Bridge Conference*, May 17, 2004.

Gajda, J., "Save Time and Money on Mass Concrete Construction," *South East Asia Construction*, September-December 2003, pages 34 to 37.

Gajda, J., "Cracking & Crack Prevention in Mass Concrete," presentation at Session No. 113 of The 82nd Transportation Research Board (TRB) Conference, Washington, D.C. Jan. 12, 2003.

Gonnerman, H.F., and Shuman, E.C., "Flexure and Tension Tests of Plain Concrete," Major Series 171, 209, and 210, *Report of the Director of Research*, Portland Cement Association, November 1928, pages 149 and 163.

Kerkhoff, B., and Gajda, J., "Maturity of Concrete: Concept, Tests, and Applications," *Concrete Technology Today*, Portland Cement Association, Skokie, Illinois, December 2005.

Kosmatka, Steven H.; Kerkhoff, Beatrix; and Panarese, William C., *Design and Control of Concrete Mixtures,* 14th Edition, Portland Cement Association, Skokie, Illinois, 2002, 358 pages.

Kosmatka, Steven H., and Panarese, William C., *Design and Control for Concrete Mixtures,* 13th Edition, Portland Cement Association, Skokie, Illinois, 1988, 220 pages.

Lea, F.M., *The Chemistry of Cement and Concrete,* 3rd Edition, Chemical Publishing Co., Inc., New York, NY, 1971, 727 pages.

Luther, M.D., "Memo for the Record about Ground Granulated Blast Furnace Skag (GGBFS) and Mass Concrete," Holnam, Inc., Chesterfield, MO, October 8, 1998.

Marceau, M.L.; Gajda, J.; and VanGeem, M.G., *Use of Fly Ash in Concrete: Normal and High Volume Ranges,* PCA R&D Serial No. 2604, Portland Cement Association, Skokie, Illinois, 2002.

Miller, F.M., and Conway, T., "Use of Ground Granulated Blast Furnace Slag for Reduction of Expansion Due to Delayed Ettringite Formation," *Cement, Concrete, and Aggregates*, Vol. 25, No. 2, December 2003, Paper ID CCA11227_252, www.astm.org.

NRMCA, *Cooling Ready Mixed Concrete*, NRMCA Publication No. 106, National Ready Mixed Concrete Association, Silver Spring, Maryland, 1962.

Pinto, Roberto C.A., and Hover, K.C., "Superplasticizer and Silica Fume Addition Effects on Heat of Hydration of Mortar Mixtures with Low Water-Cementitious Materials Ratio," *ACI Materials Journal,* V. 96, No. 5, American Concrete Institute, Farmington Hills, Michigan, September-October 1999.

Poole, Toy, *Revision of Test Methods and Specifications for Controlling Heat of Hydration in Hydraulic Cement,* SN3007, PCA, Skokie, Illinois, 2007, 42 pages.

Robbins, M., *Predicting Temperature Rise and Thermal Cracking in Concrete,* SN3030, Portland Cement Association, Skokie, Illinois, 2007, 194 pages.

Scrivener, K., and Skalny, J.; editors, *Internal Sulfate Attack and Delayed Ettringite Formation*, Conference Proceedings Pro035, RILEM, 2004, 316 pages.

Shimada, Y.; Johansen, V.C.; Miller, F.M.; and Mason, T.O., *Chemical Path of Ettringite Formation in Heat-Cured Mortar and Its Relationship to Expansion: A Literature Review*, RD136, Portland Cement Association, Skokie, Illinois, 2005, 51 pages.

Silva, A.S.; Ribeiro, A.B.; Jalali, S.; and Divet, L., "The Use of Fly Ash and Metakaolin for the Prevention of Alkali-Silica Reaction and Delayed Ettringite Formation in Concrete," *International Rilem Workshop*, Madrid, Spain, 2006 – "Performance-Based Evaluation and Indicators for Concrete Durability" *Proceedings of the International Rilem Workshop*." https://repositorium.sdum.uminho.pt/bitstream/1822/5893/1/35-018.pdf.

ASTM and AASHTO References

ASTM C150 Standard Specification for Portland Cement (AASHTO M 85)

ASTM C186 Standard Test Method for Heat of Hydration of Hydraulic Cement

ASTM C595 Standard Specification for Blended Hydraulic Cements (AASHTO M 240)

ASTM C618 Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete (AASHTO M 295)

ASTM C989 Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars (AASHTO M 302)

ASTM C1074 Standard Practice for Estimating Concrete Strength by the Maturity Method

ASTM C1157 Standard Performance Specification for Hydraulic Cement

ASTM C1202 Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

ASTM C1240 Standard Specification for Use of Silica Fume for Use as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar, and Grout (AASHTO M 307)

Portland Cement Association
5420 Old Orchard Road
Skokie, Illinois 60077-1083
847.966.6200 Fax 847.966.9781

500 New Jersey Avenue NW, 7th Floor
Washington, DC 20001-2066
202.408.9494 Fax 202.408.0877
www.cement.org

An organization of cement companies to improve and extend the uses of portland cement and concrete through market development, engineering, research, education and public affairs work.

EB547