IN-LB	Inch-Pound Units
SI	International System of Units

Mass Concrete—Guide

Reported by ACI Committee 207

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This guide contains a history of the development of mass concrete practice and discussion of materials and concrete mixture proportioning, properties, construction methods, and equipment. It covers traditionally placed and consolidated mass concrete for massive structures such as dams and provides information applicable to mass structural heavily reinforced concrete and for thermally controlled concrete such as bridge elements and building foundations. This guide does not cover roller-compacted concrete.

Keywords: cement; cracking; fly ash; heat of hydration; mass concrete; mixture proportioning; supplementary cementitious materials; temperature rise; thermal control plan; thermal expansion; volume change.

CONTENTS

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CHAPTER 1—INTRODUCTION AND HISTORICAL DEVELOPMENTS

1.1—Scope

Mass concrete covered by this guide generally falls into two classifications, or types. The first type is the traditional mass concrete of structures such as dams, where most of the structure is mass concrete and is constructed of intertwined placements. The second type consists of individual or distinct placements such as high-rise building foundations or bridges, and is increasingly referred to as thermally controlled concrete. Both types of mass concrete have similar principles and basic considerations; however, thermally controlled concrete is often constructed with commercial ready mixed concrete. Thus, it may be designed to be pumpable and can consist of self-consolidating, high-strength, or high-performance concrete, which typically results in concrete containing much higher cementitious materials content than traditional mass concrete. Although this guide mainly focuses on guidance for traditional mass concrete, much of the information can also be applicable to thermally controlled concrete.

The design of traditional mass concrete structures, such as dams, is generally based on durability, economy, and thermal requirements. Strength performance is often a secondary requirement, rather than a primary concern, and is sometimes specified to be achieved at an age of 56 or 90 days instead of 28 days.

The one characteristic that distinguishes mass concrete from other concrete work is thermal behavior. Because the reaction between water and cement is exothermic by nature, the temperature rise within a large concrete mass, where the heat is not quickly dissipated, can be quite high. Significant tensile stresses and strains may result from a decline in temperature as heat from hydration is dissipated at the volume extremities but not at the mass core. Measures should be taken where cracking due to thermal behavior may adversely affect structural integrity, durability, or aesthetics.

This guide contains a history of the development of mass concrete practice and a discussion of materials and concrete mixture proportioning, properties, construction methods, and equipment.

Mass concreting practices were developed largely from concrete dam construction, where temperature-related cracking was first identified. Temperature-related cracking has also been experienced in other concrete structures, including mat foundations, pile caps, bridge piers, superstructure elements, roadway patches, and tunnel linings.

High compressive strengths are not typically required in traditional mass concrete structures; however, there are some cases, such as thin arch dams, where high-strength concrete may be specified. Massive structures, such as gravity dams, resist loads primarily by their shape and mass; strength is of secondary importance. Of more importance are durability and properties connected with temperature behavior and the tendency for cracking.

The effects of heat generation, restraint, and volume changes on the design and behavior of massive reinforced elements and structures are discussed in ACI 207.2R. Cooling and insulating systems for mass concrete are addressed in ACI 207.4R.

1.2—History

Historically, mass concrete considerations evolved out of the use of concrete in dams. The first concrete dams were relatively small, and the concrete was mixed by hand. The portland cement usually had to be aged to comply with a boiling soundness test, the aggregate was bank-run sand and gravel, and proportioning was by the shovelful (Davis 1963). Tremendous progress has been made since the early 1900s, and the art and science of dam building practiced today has reached a highly advanced state. Presently, the selection and proportioning of concrete materials to produce suitable strength, durability, and impermeability of the finished product can now be predicted and controlled with accuracy.

Covered herein are the principal steps from those very small beginnings to the present. In large dam construction, there is now exact and automatic proportioning and mixing of materials. Concrete in 12 yd^3 (9 m^3) buckets can be placed by conventional methods at the rate of 10,000 yd^3/day (7650 m^3/day) at a temperature of less than 50°F (10°C) as placed, even during extremely hot weather. Grand Coulee Dam still holds the record monthly placing rate of 536,250 yd^3 (410,020 m^3), followed by the Itaipu Dam on the Brazil-Paraguay border with 440,550 yd^3 (336,840 m^3) (Itaipu Binacional 1981).

1.2.1 *Before 1900*—Before the beginning of the twentieth century, much of the portland cement used in the United States was imported from Europe. All cements were very coarse by present standards, and quite commonly they were underburned and had a high free lime content. For dams of

that period, bank-run sand and gravel were used without the benefit of washing to remove objectionable dirt and fines. Concrete mixtures varied widely in cement content and fine-to-coarse aggregate ratio. Mixing was usually done by hand and proportioning by shovel, wheelbarrow, box, or cart. The effect of the water-cement ratio (*w*/*c*) was unknown, and generally no attempt was made to control the volume of mixing water. There was no measure of consistency except by visual observation of the newly mixed concrete.

Some of the dams were of cyclopean masonry in which plums (large stones) were partially embedded in a very wet concrete. The spaces between plums were then filled with concrete. Some of the early dams were built without contraction joints and without regular lifts. There were, however, notable exceptions where concrete was cast in blocks, the height of lift was regulated, and concrete of very dry consistency was placed in thin layers and consolidated by rigorous hand tamping.

Generally, mixed concrete was transported to the forms by wheelbarrow. Where plums were employed in cyclopean masonry, stiff-leg derricks operating inside the work area moved the wet concrete and plums. The rate of placement was, at most, a few hundred cubic yards (cubic meters) a day. Generally, there was no attempt to moist cure the concrete.

An exception to these general practices was the Lower Crystal Springs Dam, completed in 1890. This dam is located near San Mateo, CA, approximately 20 miles (30 km) south of San Francisco. According to available information, it was the first dam in the United States in which the maximum permissible quantity of mixing water was specified. The concrete for this 154 ft (47 m) high structure was cast in a system of interlocking blocks of specified shape and dimensions. An old photograph indicates that hand tampers were employed to consolidate the dry concrete (concrete with a low water content and presumably very low workability). Fresh concrete was covered with planks as a protection from the sun, and the concrete was kept wet until hardening occurred.

1.2.2 *1900 to 1930*—After the turn of the century, construction of all types of concrete dams greatly accelerated. More and higher dams for irrigation, power, and water supply were built. Concrete placement by means of towers and chutes became common. In the United States, the portland cement industry became well established, and cement was rarely imported from Europe. ASTM specifications for portland cement underwent little change during the first 30 years of the century, aside from a modest increase in fineness requirement determined by sieve analysis. Except for the limits on magnesia and loss on ignition, there were no chemical requirements. Character and grading of aggregates were given more attention during this period. Very substantial progress was made in the development of methods of proportioning concrete. The water-cement strength relationship was established by Abrams (1918) from investigations before 1918. Nevertheless, little attention was paid to the quantity of mixing water. Placing methods using towers and flat-sloped chutes dominated, resulting in the use of excessively wet mixtures for at least 12 years after the importance of the *w*/*c* had been established.

Generally, portland cements were employed without admixtures. There were exceptions, such as the sand-cements used by the U.S. Reclamation Service (now the U.S. Bureau of Reclamation [USBR]) in the construction of the Elephant Butte Dam in New Mexico and the Arrowrock Dam in Idaho. At the time of its completion in 1915, the Arrowrock Dam, a gravity-arch dam, was the highest dam in the world at 350 ft (107 m). The dam was constructed with lean interior concrete and a richer exterior face concrete. The mixture for interior concrete contained approximately 376 lb/yd^3 (223 kg/m^3) of a blended, pulverized granite-cement combination. The cement mixture was produced at the site by intergrinding approximately equal parts of portland cement and pulverized granite so that no less than 90% passed the No. 200 (75 mm) mesh sieve. The interground combination was considerably finer than the cement being produced at that time.

Another exception occurred in the concrete for one of the abutments of Big Dalton Dam, a multiple-arch dam built by the Los Angeles County Flood Control District during the late 1920s. Pumicite (a pozzolan) from Friant, CA, was used as a 20% replacement by mass for portland cement.

In the early part of the twentieth century, cyclopean concrete went out of style. For dams of thick section, the maximum size of aggregate for mass concrete increased to 10 in. (250 mm). The slump test had come into use as a means of measuring consistency. The testing of 6 x 12 in. (150 x 300 mm) and 8 x 16 in. (200 x 400 mm) job cylinders became common practice in the United States. European countries generally adopted the 8 x 8 in. (200 x 200 mm) cube for testing the strength at various ages. By the end of the 1920s, mixers of 3 yd^3 (2.3 m^3) capacity were commonly used, and there were some of 4 yd^3 (3 m^3) capacity. Only Type I cement (portland cement) was available during this period. In areas where freezing-and-thawing conditions were severe, it was common practice to use a concrete mixture containing 564 lb/yd^3 (335 kg/m^3) of cement for the entire concrete mass. The construction practice of using an interior mixture containing 376 lb/yd^3 (223 kg/m^3) and an exterior face mixture containing 564 lb/yd^3 (335 kg/m^3) was developed to make the dam's face resistant to the severe climate and yet minimize the overall use of cement. In areas of mild climate, one class of concrete that contained amounts of cement as low as 376 lb/yd^3 (223 kg/m^3) was used in some dams (TVA 1940).

An exception was the Theodore Roosevelt Dam built during the years of 1905 to 1911 in Arizona. This dam consists of a rubble masonry structure faced with rough stone blocks laid in portland-cement mortar made with a cement manufactured in a plant near the dam site. For this structure, the average cement content has been calculated to be approximately 282 lb/yd^3 (167 kg/m^3). For the interior of the mass, rough quarried stones were embedded in a 1:2.5 mortar containing approximately 846 lb/yd^3 (502 kg/m^3) of cement. In each layer, the voids between the closely spaced stones were filled with a concrete containing 564 lb/

yd^3 (335 kg/m^3) of cement, into which rock fragments were manually placed. These conditions account for the very low average cement content. Construction was slow, and Roosevelt Dam represents perhaps the last of the large dams built in the United States by this method of construction.

1.2.3 *1930 to 1970*—This was an era of rapid development in mass concrete construction for dams. The use of the tower-and-chute method declined during this period and was used only on small projects. Concrete was typically placed using large buckets with cranes, cableways, railroad systems, or a combination of these. On the larger and more closely controlled construction projects, the aggregates were carefully processed, ingredients were proportioned by mass, and the mixing water was measured by volume. Improvement in workability was brought about by the introduction of supplementary cementitious materials (SCMs), air entrainment, and chemical admixtures. Slumps as low as 3 in. (76 mm) were employed without vibration, although most projects in later years of this era used large spud vibrators for consolidation.

A study of the records and actual inspection of a considerable number of dams shows that there were differences in conditions that could not be explained. Of two structures that appeared to be of similar quality subjected to the same environment, one might exhibit excessive cracking while the other, after a similar period of service, would be in near-perfect condition. The records available on a few dams indicated wide internal temperature variations due to cement hydration. To better understand the discrepancies borne from thermal behavior observed in the field, the American Concrete Institute (ACI) established a committee to study the effects of mass concrete.

ACI Committee 207, Mass Concrete, was organized in 1930 for the purpose of gathering information about the significant properties of mass concrete in dams and factors that influence these properties. Bogue (1929) had already identified the principal compounds in portland cement. Hubert Woods and his associates engaged in investigations to determine the contributions of each of these compounds to heat of hydration and to the strength of mortars and concretes (Woods et al. 1932).

By the beginning of 1930, the Hoover Dam in Nevada was in the early stages of planning. Because of the unprecedented size of the Hoover Dam, in-depth investigations were carried out to determine the effects of factors such as composition and fineness of cement, cement factor, temperature of curing, and maximum size of aggregate on the heat of hydration of cement, compressive strength, and other properties of mortars and concrete (U.S. Bureau of Reclamation 2009).

The results of these investigations led to the use of low-heat cement in the Hoover Dam. The investigations also furnished information for the design of the embedded pipe cooling system used for the first time in the Hoover Dam. Low-heat cement was first used in the Morris Dam, near Pasadena, CA, which was started a year before the Hoover Dam. For the Hoover Dam, the construction plant was of unprecedented capacity. Batching and mixing were completely automatic. The record day's output for the two concrete plants, equipped with 4 yd^3 (3 m^3) mixers, was over 10,000 yd^3 (7600 m^3). Concrete was transported in 8 yd^3 (6 m^3) buckets by cableways and compacted initially by ramming and tamping. In the spring of 1933, large internal vibrators were introduced and were used thereafter for compacting the remainder of the concrete. Within approximately 2 years, 3,200,000 yd^3 (2,440,000 m^3) of concrete were placed.

Hoover Dam marked the beginning of an era of improved practices in large concrete dam construction. Completed in 1935 at a rate of construction then unprecedented, the practices employed there, with some refinements, have been in use on most of the large concrete dams that have been constructed in the United States and in many other countries since that time.

The use of a pozzolanic material (pumicite) was given a trial in the Big Dalton Dam by the Los Angeles County Flood Control District. For the Bonneville Dam, completed by the Corps of Engineers in 1938 in Oregon, a portland cement-pozzolan combination was used. It was produced by intergrinding the cement clinker with a pozzolan processed by calcining an altered volcanic material at a temperature of approximately 1500°F (820°C). The proportion of clinker to pozzolan was 3:1 by mass. This type of cement was selected for use at Bonneville based on test results on concrete that indicated low temperature rise. This is the earliest known concrete dam in the United States in which an interground portland-pozzolan cement was used. The use of pozzolan as a separate cementitious material to be added at the mixer, at a rate of 30% or more of total cementitious materials, has come to be regular practice by the USBR, the Tennessee Valley Authority (TVA), the United States Army Corps of Engineers (USACE), and others.

The chemical admixtures that function to reduce water in concrete mixtures, control setting, and enhance strength of concrete began to be seriously recognized in the 1950s and 1960s (Wallace and Ore 1960) as materials that could benefit mass concrete. Since that time, chemical admixtures have been used in most mass concrete.

Around 1945, it became standard practice to intentionally entrain air for concrete in most structures exposed to severe weathering conditions. This practice applied to the concrete of exposed surfaces of dams as well as to concrete pavements and reinforced concrete in general. Air-entraining admixtures have been used for both interior and exterior concretes of practically all dams constructed since 1945. Air entrainment is sometimes specified in mass concrete in areas that are not affected by freezing and thawing as a method to increase workability without the addition of cement; this was used most recently on the Folsom Dam Auxiliary Control Structure in Folsom, CA (U.S. Bureau of Reclamation 1999).

1.2.4 *1970 to present*—During this era, roller-compacted concrete was developed and became the predominant method for placing mass concrete of dams. Because roller-compacted concrete is now commonly used, ACI 207.5R is a principal reference for this subject. Other concrete placement methods continue to be used for many projects, large

and small, particularly where roller-compacted concrete would be impractical or difficult to use. This often includes arch dams, large walls, and foundations, particularly where reinforcement is required.

The continuing development of chemical admixtures has allowed very large underwater placements where the concrete flows laterally up to 100 ft (30 m). Float-in construction methods where structural elements are precast or prefabricated and later filled with underwater concrete have been developed. Construction of dam sections and powerhouses have been completed in this manner.

Concern over the effects of high internal temperatures, temperature differences within elements, and placements that were traditionally not considered mass concrete began in the mid-1990s. These concerns started with a noted increase in the cracking of structures and the increased focus on long service life of transportation infrastructure. The increase in cracking was correlated with the use of concrete with high cement and SCM contents, finer cement grinding, high early compressive strength, and rapid construction practices. These practices resulted in concrete placements having high temperature gains and internal temperature differentials. Around the same time, delayed ettringite formation (DEF) came into notoriety in the precast industry and was also identified as a potential deterioration mechanism in non-precast concrete. This led to an increased focus on thermal control of concrete placements for commercial and transportation projects, with these placements being referred to as mass concrete. There is currently no standard definition of specific characteristics of concrete or placements that require control of temperatures and temperature differences, although efforts are underway to develop such a definition based on the dimensions of the concrete placement and heat generation of the concrete used in the placement.

ACI 301, ACI 350, and ACI 349 have specification requirements for placements designated to be treated as mass concrete. ACI 301 recommends that placements that are 4 ft (1.2 m) thick or greater, or capable of generating a large amount of heat at early ages should be treated as mass concrete. Placements that trap heat such as those where heat in soil does not allow placement to cool or in stacked placements with too little time provided for adequate heat dissipation should also be considered mass concrete. For such placements, ACI 301 specifies a maximum temperature limit of 160°F (70°C) and temperature differences not exceeding 35°F (20°C). Control of temperatures and temperature variations within the placement is required from the time the concrete is placed until the hottest portion of the concrete cools to the temperature, which is specified to be acceptable based on the average air temperature. This requirement is intended to maintain the maximum temperature and temperature variation within the specified limits, which often occurs if thermal controls are discontinued too early. ACI 301 also requires development and submittal of a thermal control plan to describe specific measures that will be used to control temperatures and temperature differences, and temperature monitoring to demonstrate that temperature requirements have not been exceeded. All the cited requirements of ACI 301 are the default requirements, which can be modified by the specifier or by request of the contractor, subject to review and approval by the engineer.

1.2.5 *Cement content*—During the late 1920s and early 1930s, it was practically an unwritten law that mass concrete for large dams should contain more than 376 lb/yd³ (223 kg/m³) of cement. However, some authorities of that period believed that the cement factor should never be less than 564 lb/yd³ (335 kg/m³). The cement factor for the interior concrete of Norris Dam (TVA 1940), constructed by the Tennessee Valley Authority (TVA) in 1936, was 376 lb/yd³ (223 kg/m³). The degree of cracking was excessive. The compressive strength of the wet-screened 6 x 12 in. (150 x 300 mm) job cylinders at 1 year of age was 7000 psi (48.3 MPa). Similarly, 18 x 36 in. (460 x 910 mm) core specimens drilled from the first-stage concrete containing 376 lb/yd³ (223 kg/m³) of cement at Grand Coulee Dam tested in excess of 8000 psi (55 MPa) at the age of 2 years. Judged by composition, the cement was the moderate-heat type corresponding to the present Type II. Considering the moderately low stresses within the two structures, it was evident that such high compressive strengths were unnecessary.

For Hiwassee Dam, completed by TVA in 1940, the cement content of the mass concrete was 282 lb/yd³ (167 kg/m³), an unusually low value for that time. Hiwassee Dam was singularly free from thermal cracks, which began a trend toward reducing the cement content. Since that time, the Type II cement content of the interior mass concrete has been approximately 235 lb/yd³ (140 kg/m³) and even as low as 212 lb/yd³ (126 kg/m³). An example of a large gravity dam for which the Type II cement content for mass concrete was 235 lb/yd³ (140 kg/m³) is Pine Flat Dam in California, completed by the USACE in 1954. In arch-type high dams where stresses are typically higher than large gravity dams, the cement content of the mass mixture is usually in the range of 300 to 450 lb/yd³ (180 to 270 kg/m³), with the higher cement content being used in the thinner and more highly stressed dams of this type.

Examples of cementitious contents, including pozzolan, for more recent dams are:

a) *Arch dams*—282 lb/yd³ (167 kg/m³) of cement and pozzolan in Glen Canyon Dam, a relatively thick arch dam in Arizona, completed in 1963; 373 lb/yd³ (221 kg/m³) of cement in Morrow Point Dam in Colorado, completed in 1968; and 303 to 253 lb/yd³ (180 to 150 kg/m³) of portland-pozzolan Type IP cement in El Cajon Dam on the Humuya River in Honduras, completed in 1984.

b) *Straight gravity dams*—226 lb/yd³ (134 kg/m³) of Type II cement in Detroit Dam in Oregon, completed in 1952; 194 lb/yd³ (115 kg/m³) of Type II cement and fly ash in Libby Dam in Montana, completed in 1972; and 184 lb/yd³ (109 kg/m³) of Type II cement and calcined clay in Ilha Solteira Dam in Brazil, completed in 1973.

c) *Spillways*—128 lb/yd³ (76 kg/m³) of cement and 383 lb/yd³ (227 kg/m³) of slag cement for 3000 psi (21 MPa) concrete, and 425 lb/yd³ (252 kg/m³) of cement and 180 lb/yd³ (107 kg/m³) of slag cement for 5000 psi (34.5 MPa)

concrete in Folsom Dam Auxiliary Spillway in California, completed in 2017.

Structural mass concrete mixtures will often be proportioned with higher total cementitious materials content than typical concrete mixtures used for dam construction. These proportions are often needed to meet requirements for high strength, placement in highly congested areas, pumpability, and other constraints in structural elements. For example, structural mass concrete mixtures often have a total cementitious materials content in excess of 700 lb/yd^3 (415 kg/m^3). High replacement percentages of SCMs are often used to mitigate high temperature rise in structural mass concrete mixtures. For example, replacement of cement with 40 to 50% fly ash or 70% slag cement is common.

1.2.6 *Temperature control*—The practice of precooling concrete materials before mixing to achieve a lower maximum temperature of interior mass concrete during the hydration period began in the early 1940s and has been extensively used in the construction of large dams. The first practice of precooling appears to have occurred during the construction of Norfork Dam from 1941 to 1945 by the USACE. The plan was to introduce crushed ice into the mixing water during the warmer months. By so doing, the temperature of freshly mixed mass concrete could be reduced by approximately 10°F (5.6°C).

Not only has crushed ice been used in the mixing water but coarse aggregates have also been precooled either by cold air or cold water before batching. Both fine and coarse aggregates have been precooled by various means, including vacuum saturation and liquid nitrogen injection during the batching and mixing of concrete.

It has become a common practice in the United States to use precooled concrete for mass concrete when ambient conditions are warm or hot. For dams, this placement temperature has been frequently limited to the average annual air temperature at the project site. The temperature limit was originally intended to ensure that the dam structure was in a low thermal stress condition during its service life. For thermally controlled concrete, ACI 301 originally required the concrete be no warmer than 70°F (20°C) at the time of placement; however, this limit was eventually eliminated and replaced with control of temperatures and temperature differences in the concrete placement, as thermally controlled concrete typically did not have the same restraint considerations as dams.

On some large dams, including Hoover (Boulder) Dam, a combination of precooling and post-cooling to temperatures at or below ambient by embedded pipe loops has been used (U.S. Bureau of Reclamation 1949). A good example of this practice is Glen Canyon Dam, where the ambient temperatures can be greater than 100°F (38°C) during the summer months. The temperature of the precooled fresh concrete did not exceed 50°F (10°C). Both refrigerated aggregate and crushed ice were used to achieve this low initial temperature. The maximum temperature of hardening concrete was kept below 75°F (24°C) by means of embedded pipe loops. Post-cooling is sometimes required in gravity and arch dams that contain transverse joints so that transverse joints can be opened for grouting by cooling the concrete after it has hardened. Post-cooling controls peak temperatures, which also helps in the control of thermal cracking. Post-cooling is increasingly being used in thermally controlled concrete as one way to control the high temperature rise of concrete with high cementitious contents and to reduce time to maintain thermal control measures. ACI 207.4R provides information regarding post-cooling systems.

1.2.7 *Long-term strength design*—A most significant development of the 1950s was the abandonment of the 28-day strength as a design requirement for dams. In massive structures, such as dams, maximum stresses under load do not usually develop until the concrete is at least 1 year old. Under mass curing conditions, with the cement and SCMs customarily employed, the gain in concrete strength between 28 days and 1 year is generally large. ACI 232.2R reports that the gain can range from 30% to more than 100%, depending on the quantities and proportioning of cementitious materials, properties of the aggregates, and the timeframe allotted for the strength gain. It has become the practice of some designers of dams to specify the desired strength of mass concrete at later ages, such as at 56 or 90 days or even 1 or 2 years.

CHAPTER 2—DEFINITIONS

Please refer to the latest version of *ACI Concrete Terminology* for a comprehensive list of definitions. Definitions provided herein complement that resource.

mass and thermally controlled concrete—There is currently no universally accepted definition for mass concrete based on specific characteristics of concrete or placements that require control of temperatures and temperature differences. *ACI Concrete Terminology* defines mass concrete as "any volume of structural concrete in which a combination of dimensions of the member being cast, the boundary conditions, the characteristics of the concrete mixture, and the ambient conditions can lead to undesirable thermal stresses, cracking, deleterious chemical reactions, or reduction in the long-term strength as a result of elevated concrete temperature due to heat from hydration". However, this definition does not provide any definitive guidance with respect to the physical characteristics of mass concrete.

thermally controlled concrete—referred to as mass concrete, or reinforced structural mass concrete (RSMC), is used to generally describe concrete for construction where issues with heat generation for higher design strength requirements can lead to high concrete temperatures during its curing and high risk of thermal cracking. Thermally controlled concrete is typically used to describe concrete that generally has higher cementitious materials content, higher temperature rise, smaller maximum aggregate size, and/or higher or earlier compressive strength requirements relative to traditional mass concrete (Bartojay 2012). These conditions often lead to mixtures having a potential for higher temperature rise than the mixtures typically used for traditional mass concrete.

traditional mass concrete—traditional mass concrete is a term used to generally describe mass concrete for

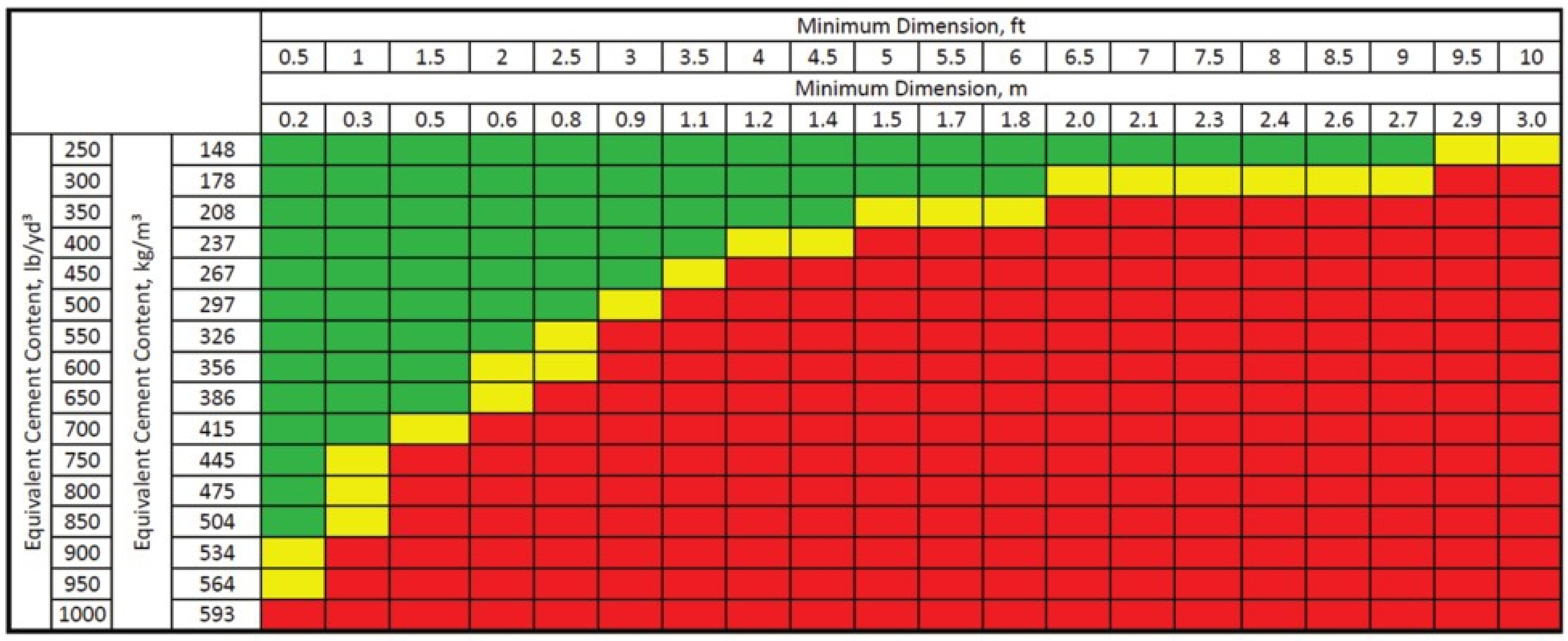

Fig. 3.1.1—Definition of mass concrete as a function of the equivalent cement content of the concrete and placement thickness (Gajda et al. 2018).

construction such as dams where issues with heat generation have been observed with sequential placements having large dimensions that trapped a lot of heat (Bartojay 2012). Traditional mass concrete typically refers to concrete used for these types of placement that generally have a relatively lower cementitious materials content, lower temperature rise, lower or later-age design strength requirements, and/or large maximum coarse aggregate size.

CHAPTER 3—MATERIALS AND MIXTURE PROPORTIONING

3.1—General

As is the case with all concrete, mass concrete is composed of cement, aggregates, water, chemical admixtures, and, frequently, supplementary cementitious materials (SCMs) such as slag cement, fly ash, silica fume, and metakaolin. The objective of mass concrete mixture proportioning is to select the combinations of materials that will produce concrete to meet the requirements of the structure. Economy, workability, dimensional stability, cracking control, low temperature rise, adequate strength, long-term durability, and, in some cases, low permeability, are of utmost importance for mass concrete structures. The limits and requirements for the aforementioned properties will be determined in consideration of the intended service and exposure conditions considered by design. Specific site conditions and construction methods are also taken in consideration when evaluating and selecting materials and proportions for the mass concrete mixture designs.

For smaller projects and those with a few placements of thermally controlled concrete, standard concretes from commercial ready mixed concrete suppliers are often used as the cost and time to develop an optimized concrete mixture for the mass concrete application may not be practical. For larger projects, the efforts to optimize the concrete mixture are often small in comparison to the benefits provided by use of the optimized concrete.

This chapter describes optimized concretes for mass concrete construction and the factors influencing their selection and proportioning. The recommendations contained herein may need to be adjusted for special uses, such as for heavily reinforced structures and tremie placements, among others.

3.1.1 *Defining mass concrete by equivalent cement method*—When developing a mass concrete mixture, one of the key considerations, in addition to strength and placeability, is the temperature rise of the concrete after placement. For traditional mass concrete, this determines the measures that will be needed to address thermal aspects such as placement height, post-cooling, and the temperature of the concrete at the time of placement. For thermally controlled concrete, this can determine if thermal control of the concrete is required (that is, if measures are necessary to mitigate stresses due to differential temperatures).

Figure 3.1.1 was developed to define mass concrete as a function of the primary influencers; the concrete mixture design and minimum placement dimension (Gajda et al. 2018). It is intended for thermally controlled concrete and not for traditional mass concrete structures with many intertwined placements. This definition can also be used to guide the development of concrete mixtures to reduce efforts needed to control temperatures and temperature differentials, or to develop a concrete mixture for a particular set of placements that does not require thermal mitigation measures. In Fig. 3.1.1, the green areas are not considered mass concrete; the red areas are mass concrete; and the yellow areas are borderline, which indicates that the placement may or may not be mass concrete. In this regard, mass concrete means that the maximum temperature in the placement is anticipated to exceed 160°F (70°C) or the temperature difference within the placement is predicted to exceed 35°F (20°C).

The equivalent cement content (ECC) of the concrete for use with Fig. 3.1.1 can be estimated from Eq. (3.1.1)*

$$\text{ECC} = \text{Cement} + 0.5\cdot\text{FAsh} + 0.8\cdot\text{CAsh} + 1.2\cdot\text{SFMK} + \text{Factor}\cdot\text{Slag} \tag{3.1.1}$$

where Cement is Type I/II portland cement, lb/yd³ (kg/m³); FAsh is Class F fly ash, lb/yd³ (kg/m³); CAsh is Class C fly ash (no distinction is made for the calcium oxide content of the fly ash, which is the main heat generating portion), lb/yd³ (kg/m³); SFMK is silica fume or metakaolin, lb/yd³ (kg/m³); Slag is slag cement (no distinction is made for Grade

100 or Grade 120), lb/yd^3 (kg/m^3); and Factor is a variable that depends on the percentage of portland cement being replaced by slag cement:

- 1.0 to 1.1 for 0 to 20% replacement;
- 1.0 for 20 to 45% replacement;
- 0.9 for 45 to 65% replacement; and
- 0.8 for 65 to 80% replacement.

3.2—Cements

Economy and low temperature rise can be achieved by reducing the total cementitious materials content and selecting cementitious materials with low heat characteristics. ACI 207.2R and 207.4R contain additional information on cement types and effects on heat generation.

The following types of hydraulic cement are suitable for use in mass concrete construction:

a) *Portland cement*—Types I, II, IV, and V, as covered by ASTM C150/C150M

b) *Blended cement*—Types IP, IS, IT, and IL, as covered by ASTM C595/C595M

c) *Hydraulic cement*—Types GU, MS, HS, MH, and LH, as covered by ASTM C1157/C1157M

Types I and GU cements are suitable for use in general construction. If these are used alone in mass concrete, they can pose a challenge to control temperature rise of concrete without other measures that can help to control temperature because of their substantially higher heat of hydration.

Types II (moderate heat) and MH cements are preferable for mass concrete construction because they typically have moderate heat of hydration, which is important to control temperature and thus mitigate thermal cracking. Type II cements are designed for moderate sulfate resistance and may have moderate heat properties. Specifications for Type II portland cement require that it contain no more than 8% tricalcium aluminate (C_3A), the compound that contributes substantially to early heat evolution in concrete. Optional specifications for Type II cement place a limit of 100% or less on the sum of $4.75 \cdot C_3A$ and C_3S or a limit on the heat of hydration to 70 cal/g (290 kJ/kg) at 7 days. When one of the optional requirements is specified, the 28-day strength requirement for cement paste under ASTM C150/C150M is reduced due to the slower rate of strength gain associated with the use of this cement.

Type III and Type HE cements, not previously listed, are not generally used in thermally controlled concrete; however, when used for specific applications, they may complicate thermal control efforts because of the high temperature rise associated with these cements.

Types IV and LH, low-heat cements, may be used where it is desired to produce low heat development in massive structures. These cements have been used in limited applications because they have been difficult to obtain and, more importantly, because experience has shown that in most cases, heat development can be satisfactorily controlled by other means. Type IV specification requirements limit the maximum amount of C_3A to 7%, the C_3S to 35%, and place a minimum on the C_2S of 40%. At the option of the purchaser, the heat of hydration may be limited to 60 cal/g (250 kJ/kg) at 7 days and 70 cal/g (290 kJ/kg) at 28 days. Type IV cement is generally not commercially available in the United States. LH is available in some markets in Canada.

Types V and HS sulfate-resistant cements are available in areas with high-sulfate soils and will often have moderate heat characteristics. They are usually both low-alkali (less than 0.6 equivalent alkalis) and low-heat (less than 70 cal/g at 7 days).

Type IP portland-pozzolan cement is a uniform blend of portland cement and fine pozzolan. They are produced either by intergrinding portland cement clinker and pozzolan or by blending portland cement and finely divided pozzolan. The pozzolan constituents can be up to 40% by mass of the portland-pozzolan cement.

Heat of hydration limit of 70 cal/g (290 kJ/kg) at 7 days is an optional requirement for Type IP by adding the suffix (MH). A limit of 60 cal/g (250 kJ/kg) at 7 days is optional for Type IP by adding the suffix (LH).

Type IS portland blast-furnace slag cement is a uniform blend of portland cement and slag cement. It is produced either by intergrinding portland cement clinker and slag cement or by blending portland cement and slag cement. The amount of slag may be up to 95% by mass of the portland-slag cement. This cement has sometimes been used with a pozzolan.

3.3—Supplementary cementitious materials

Supplementary cementitious materials (SCMs) include slag cement, fly ash, natural pozzolans, metakaolin, and silica fume. SCMs are used in mass concrete to reduce the portland cement content; lower internal heat generation and temperature rise (for certain SCMs); improve workability (for certain SCMs); reduce (improve) permeability; and, when applicable, lessen the potential for deleterious expansion caused by alkali-silica reactivity (ASR), delayed ettringite formation (DEF), and sulfate attack. It should be recognized that properties of different SCMs and different sources of the same SCM may vary widely; silica fume and metakaolin have a higher temperature rise than portland cement and reduce workability. Refer to ASTM C1778, ACI 201R, and ACI 221R for more information.

Before an SCM is used, it should be tested in combination with the project cement and aggregates to establish that the SCM will beneficially contribute to the quality and economy of the concrete. In comparison with portland cement, the strength development from certain SCMs may be slow at early ages but continues at a higher level for a longer duration. For the same water-cementitious materials ratio (*w/cm*), early strength of a concrete containing certain SCMs is generally expected to be lower (with the exception of silica fume and metakaolin) than that of a portland-cement concrete designed for equivalent strength at later ages.

When using SCMs in concrete, the materials are often batched separately at the mixing plant; however, this is not always the case, as blended cements are a combination of interground portland cement and various types of SCM.

ASTM C618 classifies and provides requirements for natural pozzolans (Class N) and fly ash (Class F or C), and

ACI 232.2R addresses the use of fly ash in concrete. Fly ash has been used to replace up to approximately 60% of the portland cement in certain applications, and 50% cement replacement by Class F fly ash has become increasingly common; however, the useful upper limit depends on the chemical composition of portland cement and fly ash. Fly ash particles are typically spherical, which aids in the workability and pumpability of the concrete. The benefit of Class F fly ash in the reduction of the temperature rise of concrete can be seen in Eq. (3.1.1); Class F fly ash has approximately half the heat of hydration as portland cement.

Slag cement is governed by ASTM C989/C989M, which specifies three grades based on slag activity index: Grade 80, Grade 100, and Grade 120. The use of slag cement in concrete is discussed in ACI 233R. The effect of slag on the temperature rise of concrete depends on the replacement level, as can be seen in Eq. (3.1.1). Slag cement is typically used to replace between 50 and 75% of the portland cement in a mass concrete mixture. Higher replacement levels have been used when specific testing is performed, as performance depends on the specific sources of portland cement and slag cement.

ACI 318 does not permit concretes where the cementitious materials content consists of more than 25% for fly ash or 50% for slag cement to be used in placements exposed to freezing and thawing in the presence of deicer chemicals (Exposure Class F3). This is based on early historical experiences where scaling occurred on some horizontal surfaces with a high SCM content and high *w/cm*, exacerbated by inadequate curing and/or improper finishing prior to salt exposure. There are many examples of successful high SCM concretes where 75% of the cementitious materials consisted of SCMs. ASTM C672/C672M, or similar newer tests such as CSA A23.2-22C, can be performed to verify scaling resistance of high-SCM concretes (Hooton and Vassilev 2012; CSA A23.2-22C).

Silica fume and metakaolin are sometimes used to increase the early-age strength of the concrete and to improve the permeability; however, the inclusion of these materials often requires additional attention to plastic shrinkage cracking and curing. Replacement of portland cement with silica fume, metakaolin, or both, typically increases the temperature rise of concrete as can be seen in Eq. (3.1.1). The use of silica fume is discussed in ACI 234R.

3.4—Chemical admixtures

Chemical admixtures provide important benefits to mass concrete in its plastic state by improving workability, increasing slump, reducing water content, delaying setting time, modifying the rate or capacity for bleeding, entraining air, reducing segregation, and reducing rate of slump loss. Chemical admixtures can provide important benefits to mass concrete in its hardened state by allowing concrete proportions to be modified and optimized (to reduce the cementitious content) to produce concrete with lower heat evolution during hardening, increased strength, increased durability, improved permeability, and improved abrasion or erosion resistance. An extensive discussion of admixtures is contained in ACI 212.3R and requirements for chemical admixtures are contained in ASTM C494/C494M.

Air-entraining admixtures produce microscopic air bubbles in concrete during mixing, which is needed for freezing-and-thawing resistance. Other benefits of air entrainment include improved workability, reduced segregation, and lessened bleeding. For these reasons, air entrainment is sometimes used even when the structure is not exposed to freezing-and-thawing cycles during its service life. In general, each 1% of entrained air permits a reduction in mixing water from 2 to 4%, with some improvement in workability and no loss in slump. For the same *w/cm*, an increase in the air content by means of entraining air will generally reduce the strength of concrete. A typical rule of thumb is that each 1% increase in the entrained air content will reduce the strength by 3 to 5%. However, where the cement content is held constant and advantage is taken of the reduced water demand imparted by the entrained air, lower *w/cm* is achieved. Air entrainment in mass concrete can have a negligible effect on strength. Some of the factors that influence the amount of entrained air in concrete for a given amount of agent include grading and particle shape of the aggregate, richness of the mixture, presence of other admixtures, mixing time, slump, and temperature of the concrete. For a given quantity of air-entraining admixture, air content increases with increases in slump, and decreases with increases in amount of fines, temperature of concrete, and mixing time. If fly ash is used that contains high carbon content, an increased dosage of air-entraining admixture is expected to be required. When air-entrained concrete is required, most specifications require the air content to be in the range of 5 to 8%; however, this depends on the coarse aggregate size. Refer to ACI 301 for specific requirements.

Accelerating admixtures are typically not used in mass concrete because they increase the rate of early heat generation and can pose a challenge in controlling temperature. When high early strength is desired and accelerating admixtures are incorporated in the mass concrete mixture, such as in a tower crane pad, testing should be performed to assess temperature rise and develop a thermal control plan with representative data. Due to its accelerating properties, similar care should be exercised where admixtures containing calcium nitrite, such as in corrosion inhibitors, are used in the concrete.

Water-reducing admixtures are used to reduce the required mixing water for a given workability to increase strength, reduce shrinkage of the concrete, increase the workability of the concrete, or produce the same strength with less cement. Set-controlling or hydration-stabilizing admixtures can be used to keep the concrete in a plastic state longer so that successive layers can be placed and consolidated together before the underlayer sets. Water-reducing admixtures reduce water content by 5% or more. If the water content is kept constant, these admixtures will increase slump instead. Depending on the composition, these admixtures can retard the set time of the concrete but not necessarily reduce the rate of slump loss. Depending on the type of concrete, composition of the cement, temperature, and other factors, the use of chemical admixtures can result in significant increases in

early and later strengths. This gain in strength can be attributed to a reduction in the *w/cm* and a better dispersion of the cementitious materials throughout the concrete, resulting in a higher degree of hydration.

3.5—Aggregates

Fine and coarse aggregate requirements are governed by ASTM C33/C33M. The combined gradation affects the paste content, and smaller maximum aggregate size generally require more paste than larger maximum aggregate size. Additional information on aggregates is contained in ACI 221R.

Fine aggregate passes the 3/8 in. (9.5 mm) sieve. It may be composed of natural material (sand), material obtained by crushing larger-size rock particles, or a mixture of the two. Fine aggregate should consist of hard, dense, durable, uncoated particles. Fine aggregate should not contain harmful amounts of clay, silt, dust, mica, organic matter, or other impurities to such an extent that, either separately or together, they render it impossible to attain the required properties of concrete when using normal proportions of the constituents. ASTM C33/C33M specifies general and widely accepted deleterious material limits in fine aggregate. The specifier should consider the exposure conditions of the mass concrete and specify deleterious limits deemed appropriate based on severity of environment.

The grading of fine aggregate strongly influences the workability of concrete and strength performance. Sand gradations for mass concrete should be within the limits in ASTM C33/C33M unless laboratory investigation shows other gradations to be satisfactory. Once the proportion is established, the gradation of the fine aggregate should be controlled to avoid variations in the concrete workability.

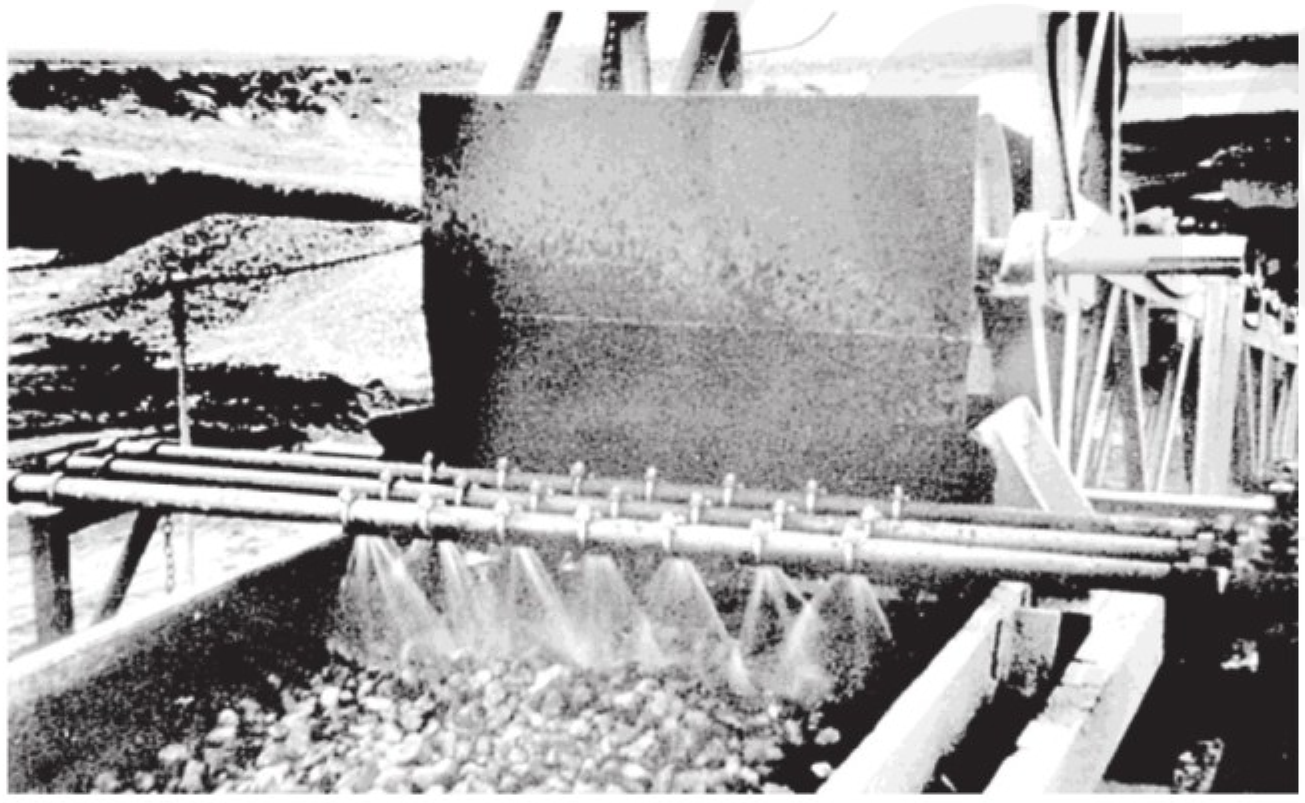

Fig. 3.5a—Coarse aggregate rewashing.

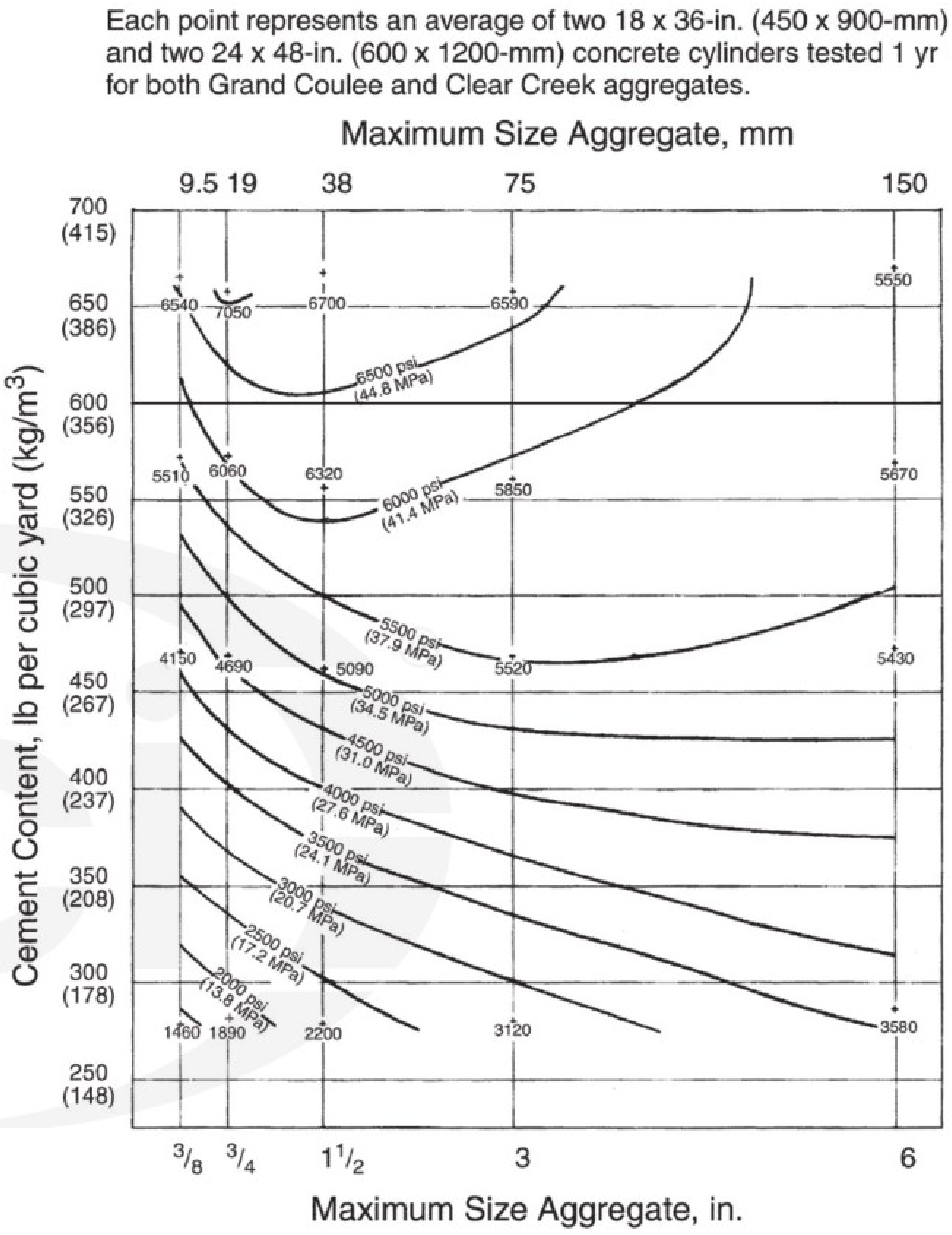

Fig. 3.5b—Effect of aggregate size and cement content on compressive strength at 1 year (adapted from Higginson et al. [1963]).

Table 3.5a—Grading guidance for cobble portion of coarse aggregate gradation (U.S. Bureau of Reclamation 1981)

<table>
<tr><th rowspan="2">Test sieve size, square mesh, in. (mm)</th><th colspan="4">Percent by mass passing designate test sieve</th></tr>
<tr><th>Cobbles: 6 to 3 in. (150 to 75 mm)</th><th>Coarse: 3 to 1-1/2 in. (75 to 37.5 mm)</th><th>Medium: 1-1/2 to 3/4 in. (37.5 to 19 mm)</th><th>Fine: 3/4 in. to No. 4 (19 to 4.75 mm)</th></tr>
<tr><td>7 (175)</td><td>100</td><td colspan="3" rowspan="10">Refer to ASTM C33/C33M</td></tr>
<tr><td>6 (150)</td><td>90 to 100</td></tr>
<tr><td>4 (100)</td><td>20 to 45</td></tr>
<tr><td>3 (75)</td><td>0 to 15</td></tr>
<tr><td>2 (50)</td><td>0 to 5</td></tr>
<tr><td>1-1/2 (37.5)</td><td></td></tr>
<tr><td>1 (25)</td><td></td></tr>
<tr><td>3/4 (19)</td><td></td></tr>
<tr><td>3/8 (9.5)</td><td></td></tr>
<tr><td>No. 4 (4.75)</td><td></td></tr>
</table>

Table 3.5b—Ranges in each size fraction of coarse aggregate that have produced workable mass concrete with cobbles (U.S. Bureau of Reclamation 1981)

Maximum size in concrete, in. (mm)	Percentage of cleanly separated coarse aggregate fractions					
	Cobbles: 6 to 3 in. (150 to 75 mm)	Coarse: 3 to 1-1/2 in. (75 to 37.5 mm)	Medium: 1-1/2 to 3/4 in. (37.5 to 19 mm)	Fine aggregate		
				3/16 (No. 4) to 3/4 in. (4.8 to 19 mm)	3/8 to 3/4 in. (9.5 to 19 mm)	3/16 (No. 4) to 3/8 in. (4.8 to 9.5 mm)
6 (150)	20 to 35	20 to 32	20 to 30	20 to 35	12 to 20	8 to 15
3 (75)	—	20 to 40	20 to 40	25 to 40	15 to 25	10 to 15
1-1/2 (37.5)	—	—	40 to 55	45 to 60	30 to 35	15 to 25
3/4 (19)	—	—	—	100	55 to 73	27 to 45

Coarse aggregate is defined as gravel, crushed gravel, crushed rock, or a mixture of these, nominally larger than the No. 4 (4.75 mm) and smaller than the 6 in. (150 mm) sieves. Coarse aggregate should consist of hard, dense, durable, uncoated particles. Figure 3.5a shows a coarse-aggregate rewashing screen at the batch plant where dust and coatings accumulating from stockpiling and handling can be removed to ensure aggregate cleanliness. ASTM C33/C33M provides deleterious particle limits in coarse aggregate sources based on concrete class designation, which is based on application.

The general rule for mass concrete is to use the largest practical maximum size of well-graded coarse aggregate to reduce the cementitious paste content needed for workability. Such aggregates may be 3 or 1.5 in. (75 or 38 mm). The use of poorly graded aggregates, those that are flat/elongated, or both, can require as much or more cementitious paste than if a smaller aggregate size is used, thus defeating the purpose of using a large maximum coarse aggregate size.

In some cases, the use of smaller aggregate may be dictated by the close spacing of reinforcement or embedded items or by the unavailability of larger aggregates. However, the smaller the aggregate size, the higher the water demand and paste content. Higher cement and paste contents can pose adverse effects on internal heat generation, cracking, and shrinkage potential. Regardless of the aggregate size, mass concrete should be workable and capable of being placed, consolidated, and finished with the selected means and methods. The maximum aggregate size should be a function of the least dimension of the structure, clear distance between reinforcing steel, and cover. It has been demonstrated (Fig. 3.5b) that to achieve the greatest cement efficiency, there is an optimum maximum aggregate size for each compressive strength (Higginson et al. 1963).

Aggregates should be composed of a blend of large, intermediate, and fine aggregates to provide a well-graded distribution. It has been found that maintaining the percent passing the 3/8 in. (9.5 mm) sieve at less than 30% in the 3/4 in. to No. 4 (19 to 4.75 mm) size fraction (preferably near zero if crushed) will greatly improve mass concrete workability and response to vibration (Tuthill 1980).

ASTM C33/C33M provides requirements for coarse aggregate gradations with a maximum size of 3.5 in. (89 mm) or less. Table 3.5a provides guidance for aggregate particles greater than 3 in. (76 mm) from the U.S. Bureau of Reclamation (1981). Table 3.5b gives examples of blended coarse aggregate gradations that have produced workable mass concrete with cobbles. In absence of experience, the gradation recommendations in Table 3.5b are a good starting point.

3.6—Water

Water used for mixing concrete should comply with ASTM C1602/C1602M.

3.7—Selection of proportions

The primary objective of proportioning studies for mass concrete is to establish economical mixtures of proper strength, durability, and permeability with the best combination of available materials that will provide adequate workability for placement and least practical rise in temperature after placement.

As noted previously, for smaller projects, concrete mixtures using standard materials from the local commercial concrete batch plant are often used for thermally controlled mass concrete placements. These mixtures may be standard concrete mixtures or may be specially developed using the available materials based on the specific needs of the project. For larger or very large projects, it is economical and beneficial to develop special concrete mixtures for the project. In some cases, nonlocal materials such as coarse and fine aggregate may be imported because they are advantageous to use over the site or local materials.

The remainder of this section is primarily for traditional mass concrete placements of dams; however, the general principles described in this section also apply to all mass concrete.

Selection of *w/cm* will establish the strength, durability, and permeability of the concrete. There should also be sufficient fine material to facilitate placement. Trial mixtures using the required *w/cm* and the observed water requirement for the job materials will demonstrate the cementitious material content that may be used to provide the required workability (Portland Cement Association 1979; Ginzburg et al. 1966).

The first step in arriving at the actual batch weights is to select the maximum aggregate size for each part of the work. Criteria for this selection are given in Section 3.5. The next step is to assume or determine the total water content needed to provide required slump, which may be as low as 1-1/2 to 2 in. (38 to 50 mm). In tests for slump, aggregate larger than 1-1/2 in. (38 mm) should be removed by promptly screening the wet concrete. For 6 in. (150 mm)

maximum-size aggregate, water contents for air-entrained minimum-slump concrete may vary from approximately 120 to 150 lb/yd^3 (71 to 89 kg/m^3) for natural aggregates, and from 140 to 190 lb/yd^3 (83 to 113 kg/m^3) for crushed aggregates. Corresponding water requirements for 3 in. (76 mm) maximum-size aggregate are approximately 20% higher. For strengths above 4000 psi (28 MPa) at 1 year, however, the 3 in. (75 mm) maximum-size aggregate may be more efficient (Fig. 3.5a).

The batch weight of the cementitious material is determined by dividing the total weight of the mixing water by the *w/cm* or, when workability governs, it is the minimum weight of cementitious materials required to satisfactorily place the concrete. With the batch weights of cementitious material and water determined and with an assumed air content of 3 to 5%, the remainder of the material is aggregate. The only remaining decision is to select the relative proportions of fine and coarse aggregate. The optimum proportions depend on aggregate grading and particle shape, and they can be finally determined only in the field. For 6 in. (150 mm) aggregate concrete containing natural sand and gravel, the percentage of fine aggregate to total aggregate by absolute volume may be as low as 21%. With crushed aggregates, the percentage may be in the range of 25 to 27%.

When an SCM is included in the concrete mixture, the mixture proportioning procedure does not change. Attention should be given to the following matters:

a) The water requirement may change

b) Early-age strength may become critical

c) For maximum economy, the age at which design strength is attained should be greater

Concrete containing most SCMs gains strength somewhat slower than concrete made with only portland cement; however, the load on mass concrete is generally not applied until the concrete is many months or years old. Therefore, mass concrete containing SCMs is commonly designed on the basis of 90-day to 1-year strengths. While mass concrete does not require strength performance at early ages to perform its design function, most systems of construction require that the forms for each lift be anchored to the next lower lift. Therefore, the early strength should be great enough to prevent pullout of the form anchors. Specially designed form anchors may be required to allow safe, rapid turnaround times for the forms, especially when large amounts of SCMs are used or when the concrete is lean and precooled.

CHAPTER 4—PROPERTIES

4.1—General

Many factors can influence the design and construction of massive concrete structures. In addition to strength, durability, and serviceability requirements, other factors can include cost, environmental conditions, placement method and associated requirements, availability of suitable materials of construction, properties of the supporting soils, and site topography. Economy may also dictate some aspects of design and construction, including the choice of type of structure for a given site.

The concrete mixture proportions selected for a mass concrete project can have a very large effect on the construction requirements. Investing in the design and testing of the concrete mixture is justified and essential for larger structures. This includes investigating and evaluating available aggregate sources, types of admixtures, and different supplementary cementitious materials (SCMs). An optimized concrete mixture can reduce the risk of cracking and reduce overall construction costs, including lowering material costs, reducing thermal mitigation requirements, allowing larger placements, and expediting construction.

To properly analyze the suitability of a concrete mixture, the following properties and characteristics may be evaluated: compressive strength, tensile strength, rate of f_c' and f_t development related to maturity, modulus of elasticity, Poisson's ratio, tensile strain capacity, creep, drying shrinkage, autogenous shrinkage, adiabatic temperature rise, coefficient of thermal expansion, specific heat capacity, thermal conductivity, and permeability. Approximate values of these properties based on computations or experience are often used in preliminary evaluations. Actual properties should be determined by testing of the actual mixture and constituents to be used for construction when practical because small inaccuracies can often affect predicted behavior. A mockup or test block representative of the construction materials and placement conditions can also provide valuable information about the temperature rise and performance of the concrete, which can be used to refine thermal analysis models.

4.1.1 A compilation of concrete proportion data on representative dams is given in Table 4.1.1a (Price and Higginson 1963; Ginzburg et al. 1966; ICOLD 1964; Harboe 1961; U.S. Bureau of Reclamation 1958; Houghton and Hall 1972; Houghton 1969, 1970). Reference is made to concrete mixtures described in Table 3.5a and in discussions of properties reported in Tables 4.1.1b through 4.1.1f.

Table 4.1.1a—Typical concrete mixture data from various dams

No.	Name of dam (country)	Year completed	Type	Cement		Pozzolan		Sand	Coarse aggregate		Maximum size aggregate, in. (mm)	Water, lb*/yd^3 (kg/m^3)	*w/cm*	Entrained air, %	Density, lb/ft^3 (kg/m^3)	WRA used
				Type	lb*/yd^3 (kg/m^3)	Type	lb*/yd^3 (kg/m^3)	lb*/yd^3 (kg/m^3)	lb*/yd^3 (kg/m^3)	Type						
1	Hoover (U.S.)	1936	Arch gravity	IV	380 (225)	—	0	931 (552)	2679 (1589)	Limestone and granite	9.0 (225)	220 (130)	0.58	0	155.9 (2498)	No
2	Norris (U.S.)	1936	Straight gravity	II	338 (200)	—	0	1264 (750)	2508 (1487)	Dolomite	6.0 (150)	227 (135)	0.67	0	156.0 (2499)	No
3	Bonneville (U.S.)	1938	Gravity	Portland pozzolan	329 (195)	—	0	1094 (649)	2551 (1513)	Basalt	6.0 (150)	251 (149)	0.76	0	156.4 (2505)	No
4	Bartlett (U.S.)	1939	Multiple arch	IV	466 (276)	—	0	1202 (713)	2269 (1346)	Quartzite and granite	3.0 (75)	270 (160)	0.58	0	154.8 (2480)	No
5	Grand Coulee (U.S.)	1942	Straight gravity	II and IV	377 (224)	—	0	982 (582)	2568 (1523)	Basalt	6.0 (150)	226 (134)	0.60	0	153.8 (2464)	No
6	Kentucky (U.S.)	1944	Straight gravity	II	338 (200)	—	0	967 (573)	2614 (1550)	Limestone	6.0 (150)	213 (126)	0.63	0	153.2 (2454)	No
7	Shasta (U.S.)	1945	Curved gravity	IV	370 (219)	—	0	906 (537)	2721 (1614)	Andesite and slate	6.0 (150)	206 (122)	0.56	0	155.7 (2494)	No
8	Hungry Horse (U.S.)	1952	Arch gravity	II	188 (111)	Fly ash	90 (53)	842 (499)	2820 (1672)	Sandstone	6.0 (150)	130 (77)	0.47	3.0	150.7 (2415)	No
9	Detroit (U.S.)	1953	Straight gravity	II and IV	226 (134)	—	0	1000 (593)	2690 (1595)	Diorite	6.0 (150)	191 (113)	0.85	5.5	152.1 (2437)	No
10	Monticello (U.S.)	1957	Arch	II LA	212 (126)	Calcinated diatomaceous clay	70 (42)	770 (457)	2960 (1756)	Graywacke sandstone quartzite	6.0 (150)	161 (96)	0.57	2.7	154.6 (2477)	No
11	Flaming Gorge (U.S.)	1962	Arch gravity	II	188 (111)	Calcined clay	94 (56)	729 (432)	2900 (1720)	Limestone and sandstone	6.0 (150)	149 (88)	0.53	3.5	150.4 (2409)	No
12	Glen Canyon (U.S.)	1963	Arch gravity	II	188 (111)	Pumicite	94 (56)	777 (461)	2784 (1651)	Limestone, chaledonic chert, and sandstone	6.0 (150)	153 (91)	0.54	3.5	148.0 (2371)	No
		1963	Arch gravity	II	188 (111)	Pumicite	90 (53)	800 (474)	2802 (1662)		6.0 (150)	140 (83)	0.50	3.5	148.9 (2385)	Yes
13	Yellowtail (U.S.)	1965	Arch gravity	II	197 (117)	Fly ash	85 (50)	890 (526)	2817 (1670)	Limestone and andesite	6.0 (150)	139 (82)	0.49	3.0	152.9 (2449)	No
14	Morrow Point (U.S.)	1967	Thin arch	II	373 (221)	—	0	634 (376)	2851 (1691)	Andesite, tuff, and basalt	4.5 (114)	156 (93)	0.42	4.3	148.7 (2382)	Yes
15	Dworshak (U.S.)	1972	Gravity	II	211 (125)	Fly ash	71 (42)	740 (439)	2983 (1770)	Crushed granite gneiss	6.0 (150)	164 (97)	0.59	3.5	154.4 (2473)	No
16	Libby (U.S.)	1972	Gravity	II	148 (88)	Fly ash	49 (29)	903 (536)	2878 (1708)	Natural quartzite gravel	6.0 (150)	133 (79)	0.68	3.5	152.3 (2439)	No
17	Lower Granite (U.S.)	1973	Gravity	II	145 (86)	Milled volcanic cinders	49 (29)	769 (456)	3096 (1837)	Natural basaltic gravel	6.0 (150)	138 (82)	0.71	3.5	155.4 (2490)	Yes

Table 4.1.1a (cont.)—Typical concrete mixture data from various dams

18	Pueblo (U.S.)	1974	Buttress	II LA	226 (134)	—	75 (44)	952 (565)	2589 (1535)	Granite, shist, limestone, dolomite	3.5 (89)	168 (100)	0.56	3.5	148.5 (2379)	Yes
19	Crystal (U.S.)	1976	Thin arch	II LA	390 (231)	—	0	829 (492)	2740 (1625)	Shist, altered volcanics	3.0 (75)	183 (109)	0.47	3.5	153.4 (2457)	Yes
20	Richard B. Russell (U.S.)	1982	Gravity	II	226 (134)	Fly ash	59 (35)	822 (488)	2958 (1755)	Crushed granite	6.0 (150)	173 (103)	0.57	3.4	157.0 (2515)	Yes
				II	173 (103)	Fly ash	73 (43)	864 (513)	2935 (1741)		6.0 (150)	177 (105)	0.67	3.4	156.0 (2499)	Yes
21	Rossens (Switzerland)	1948	Arch	I	421 (250)	—	0	—	—	Limestone	3.1 (79)	225 (133)	0.53	0	—	No
22	Pieve di Cadore (Italy)	1949	Arch gravity	Ferric-pozzolanic	253 (150)	Natural	84 (50)	1180 (700)	2089 (1239)	Limestone	4.7 (120)	213 (126)	0.63	2.0	159.9 (2560)	Yes
23	Francisco Madero (Mexico)	1949	Roundhead buttress	IV	372 (221)	—	0	893 (530)	2381 (1412)	Rhyolite and basalt	6.0 (150)	223 (132)	0.60	—	—	—
24	Chastang (France)	1951	Arch gravity	250/315	379 (225)	—	0	759 (450)	2765 (1640)	Granite	9.8 (250)	169 (100)	0.45	—	150.8 (2415)	—
25	Salmonde (Portugal)	1953	Thin arch	II	421 (250)	—	0	739 (438)	2621 (1554)	Granite	7.9 (200)	225 (133)	0.54	0	148.4 (2376)	—
26	Warragamba (Australia)	1960	Straight gravity	II	330 (196)	—	0	848 (503)	2845 (1687)	Porphyry and granite	6.0 (150)	175 (104)	0.53	0	154.2 (2469)	No
27	Krasnoiarsk (Russia / U.S.S.R.)	About 1970	Straight gravity	IV and portland blast furnace	388 (230)	—	0	—	—	Granite	3.9 (100)	213 (126)	0.55	—	—	Yes
28	Ilha Solteira (Brazil)	1974	Gravity	II	138 (82)	Calcined clay	46 (27)	788 (468)	3190 (1893)	Quartzite gravel, crushed basalt	6.0 (150)	138 (82)	0.75	3.5	159.3 (2552)	No
29	Itaipu (Brazil-Paraguay)	1982	Hollow gravity buttress	II	182 (108)	Fly ash	22 (13)	981 (582)	3096 (1837)	Crushed basalt	6.0 (150)	143 (85) 170 (101)	0.70	4.0	158.4 (2537)	No
30	Peace Site 1 (Canada)	1979	Gravity	I	158 (94)	Fly ash	105 (63)	967 (575)	2610 (1549)	Quartzite limestone sandstone	3 (75)	144 (85)	0.67	3.6	148.5 (2379)	Yes
31	Theodore Roosevelt modification (U.S.)	1995	Arch gravity	II LA	216 (128)	Fly ash	54 (32)	954 (566)	2672 (1585)	Granite	4.0 (100)	144 (85)	0.53	4.0	149.7 (2397)	Yes

*Pounds mass.

Table 4.1.1b—Mixture proportion and compressive strengths of concrete in various dams

Dam	Country	Cement or cement-SCM, lb/yd³ (kg/m³)	Water, lb/yd³ (kg/m³)	Predominant aggregate type	Maximum size aggregate, in. (mm)	*w/cm*	90-day compressive strength, psi (MPa)	Cement efficiency at 90 days, psi/lb/yd³ (MPa/kg/m³)
La Palisse	France	506 (300)	250 (148)	Granite	4.7 (120)	0.49	4790 (33.0)	9.5 (0.111)
Chastang	France	379 (225)	169 (100)	Granite	9.8 (250)	0.45	3770 (26.0)	9.9 (0.115)
L'Aigle	France	379 (225)	211 (125)	Granite	9.8 (250)	0.56	3200 (22.1)	8.4 (0.098)
Pieve di Cadore	Italy	337 (200)	213 (126)	Dolomite	4.0 (100)	0.63	6400 (44.1)	19.0 (0.220)
Forte Baso	Italy	404 (240)	238 (141)	Pophyry	3.9 (98)	0.59	4920 (33.9)	12.2 (0.141)
Cabrilo	Portugal	370 (220)	195 (116)	Granite	5.9 (150)	0.53	4150 (28.6)	11.2 (0.130)
Salamonde	Portugal	420 (249)	225 (133)	Granite	7.9 (200)	0.54	4250 (29.3)	10.1 (0.118)
Castelo Bode	Portugal	370 (220)	180 (107)	Quartzite	7.9 (200)	0.49	3800 (26.2)	10.3 (0.119)
Rossens	Switzerland	420 (249)	225 (133)	Glacial mixture	2.5 (64)	0.54	5990 (41.3)	14.3 (0.166)
Mauvoisin	Switzerland	319 (189)	162 (96)	Gneiss	3.8 (96)	0.51	4960 (34.2)	15.5 (0.181)
Zervreila	Switzerland	336 (199)	212 (126)	Gneiss	3.8 (96)	0.63	3850 (26.5)	10.5 (0.133)
Hungry Horse	United States	188-90 (111-53)	130 (77)	Sandstone	6.0 (150)	0.47	3100 (21.4)	11.2 (0.130)
Glen Canyon	United States	118-94 (111-56)	153 (99)	Limestone	6.0 (150)	0.54	3810 (26.3)	13.5 (0.160)
Lower Granite	United States	145-49 (86-29)	138 (82)	Basalt	6.0 (150)	0.71	2070 (14.3)	10.7 (0.124)
Libby	United States	148-49 (88-29)	133 (79)	Quartzite	6.0 (150)	0.68	2460 (17.0)	12.5 (0.145)
Dworshak	United States	211-71 (125-42)	164 (97)	Granite	6.0 (150)	0.58	3050 (21.0)	10.8 (0.126)
Dworshak	United States	198-67 (117-40)	164 (97)	Gneiss	6.0 (150)	0.62	2530 (17.4)	9.5 (0.111)
Dworshak	United States	168-72 (100-43)	166 (98)	Gneiss	6.0 (150)	0.69	2030 (14.0)	8.5 (0.098)
Dworshak	United States	174-46 (130-27)	165 (98)	Gneiss	6.0 (150)	0.75	1920 (13.2)	8.7 (0.084)
Pueblo	United States	226-75 (134-44)	168 (100)	Granite, limestone, dolomite	3.5 (89)	0.56	3000* (20.7)	10.0 (0.116)
Crystal	United States	390 (231)	183 (109)	Shist and altered volcanics	3.0 (75)	0.47	4000† (27.6)	10.3 (0.119)
Flaming Gorge	United States	188-94 (111-56)	149 (88)	Limestone and sandstone	6.0 (150)	0.53	3500 (24.1)	12.4 (0.144)
Krasnoiarsk	USSR/Russia	388 (230)	213 (126)	Granite	3.9 (100)	0.55	3280 (22.6)	8.5 (0.098)
Ilha Solteira	Brazil	138-46 (82-27)	132 (82)	Quartzite gravel, crushed basalt	6.0 (150)	0.75	3045 (21.0)	16.5 (0.193)
Itaipu	Brazil	182-22 (108-13)	143 (85)	Crushed basalt	6.0 (150)	0.70	2610 (18.0)	12.8 (0.149)
Theodore Roosevelt modification	United States	270 (160)	144 (85)	Granite	4.0 (100)	0.53	4500 (31.0)	16.7 (0.194)

*Strength at 80 days.

†Strength at 1 year.

Table 4.1.1c—Compressive strength and elastic properties of mass concrete

No.	Dam	Compressive strength, psi (MPa)				Elasticity properties: Modulus of elasticity, $E \times 10^6$, psi (E, GPa)				Elasticity properties: Poisson's ratio			
		Age, days				Age, days				Age, days			
		28	90	180	365	28	90	180	365	28	90	180	365
1	Hoover	3030 (20.9)	3300 (22.8)	—	4290 (29.6)	5.5 (38)	6.2 (43)	—	6.8 (47)	0.18	0.20	—	0.21
2	Grand Coulee	4780 (33.0)	5160 (35.6)	—	5990 (41.3)	4.7 (32)	6.1 (42)	—	6.0 (41)	0.17	0.20	—	0.23
3	Glen Canyon	2550 (17.6)	3810 (26.3)	3950 (27.2)	—	5.4 (37)	—	5.8 (40)	—	0.11	—	0.14	—
4	Glen Canyon*	3500 (24.1)	4900 (33.8)	6560 (45.2)	6820 (47.0)	5.3 (37)	6.3 (43)	6.7 (46)	—	0.15	0.15	0.19	—
5	Flaming Gorge	2950 (20.3)	3500 (24.1)	3870 (26.7)	4680 (32.3)	3.5 (24)	4.3 (30)	4.6 (32)	—	0.13	0.25	0.20	—
6	Yellowtail	—	4580 (31.6)	5420 (37.4)	5640 (38.9)	—	6.1 (42)	5.4 (37)	6.2 (43)	—	0.24	0.26	0.27
7	Morrow Point*	4770 (32.9)	5960 (41.1)	6430 (44.3)	6680 (46.1)	4.4 (30)	4.9 (34)	5.3 (37)	4.6 (32)	0.22	0.22	0.23	0.20
8	Lower Granite*	1270 (8.8)	2070 (14.3)	2420 (16.7)	2730 (18.8)	2.8 (19)	3.9 (27)	3.8 (26)	3.9 (27)	0.19	0.20	—	—
9	Libby	1450 (10.0)	2460 (17.0)	—	3190 (22.0)	3.2 (22)	4.0 (28)	—	5.5 (38)	0.14	0.18	—	—
10	Dworshak*	1200 (8.3)	2030 (14.0)	—	3110 (21.4)	—	3.7 (26)	—	3.8 (26)	—	—	—	—
11	Ilha Solteira	2320 (16.0)	2755 (19.0)	3045 (21.0)	3190 (22.0)	5.1 (35)	5.9 (41)	—	—	0.15	0.16	—	—
12	Itaipu	1885 (13.0)	2610 (18.0)	2610 (18.0)	2755 (19.0)	5.5 (38)	6.2 (43)	6.2 (43)	6.5 (45)	0.18	0.21	0.22	0.20
13	Peace site*	3060 (21.1)	3939 (27.2)	4506 (31.1)	4666 (32.2)	—	—	—	—	—	—	—	—
14	Theodore Roosevelt modification	2400 (16.5)	4500 (31.0)	5430 (37.4)	5800 (40.0)	4.5 (31)	5.4 (37)	—	6.2 (43)	0.20	0.21	—	0.21

*Water-reducing agent used.

Table 4.1.1d—Elastic properties of mass concrete

Age of time of loading	Instantaneous and sustained modulus of elasticity,* psi × 10^6 (GPa)														
	Grand Coulee			Shasta			Hungry Horse			Dworshak			Libby		
	E	E^1	E^2	E	E^1	E^2	E	E^1	E^2	E	E^1	E^2	E	E^1	E^2
2 days	1.7 (12)	0.83 (5.7)	0.76 (5.2)	1.4 (9.7)	0.54 (3.7)	0.49 (3.4)	2.8 (19)	1.5 (10)	1.4 (9.7)	1.4 (9.7)	0.75 (5.2)	0.70 (4.8)	1.6 (11)	1.0 (6.9)	0.9 (6.2)
7 days	2.3 (16)	1.1 (7.6)	1.0 (6.9)	2.1 (14)	1.0 (6.9)	0.96 (6.6)	4.2 (29)	1.9 (13)	1.8 (12)	2.0 (14)	1.0 (6.9)	0.90 (6.2)	3.2 (22)	1.6 (11)	1.3 (9.0)
20 days	3.5 (24)	1.8 (12)	1.6 (11)	3.5 (24)	1.8 (12)	1.6 (11)	4.5 (31)	2.6 (18)	2.4 (17)	2.8 (19)	1.4 (9.7)	1.3 (9.0)	4.1 (28)	2.2 (15)	2.0 (14)
90 days	4.1 (20)	2.5 (17)	2.3 (16)	4.4 (30)	2.7 (19)	2.5 (17)	5.2 (36)	3.2 (22)	3.0 (21)	3.8 (26)	2.2 (15)	2.0 (14)	5.2 (36)	2.9 (20)	2.7 (19)
1 year	5.0 (34)	2.5 (17)	2.3 (16)	4.4 (30)	2.7 (19)	2.5 (17)	5.2 (36)	3.2 (22)	3.0 (21)	3.8 (26)	2.2 (15)	2.0 (14)	5.2 (36)	2.9 (20)	2.7 (19)
5 years	5.3 (37)	3.6 (25)	3.4 (23)				5.9 (41)	4.0 (28)	3.8 (26)	4.9 (34)	3.0 (21)	2.9 (20)	6.4 (44)	4.3 (30)	4.1 (28)
7-1/4 years				5.6 (39)	4.3 (30)	4.1 (28)									

*All concrete mass mixed, wet screened to 1-1/2 in. (37.5 mm) maximum-size aggregate.

Note: E is instantaneous modulus of elasticity at time of loading; E^1 is sustained modulus after 365 days under load; and E^2 is sustained modulus after 1000 days under load. Instantaneous modulus of elasticity refers to static or normal load rate (1- to 5-minute duration) modulus, not a truly instantaneous modulus measured from dynamic or rapid load rate testing.

Table 4.1.1e—Volume change and permeability of mass concrete

Structure	Autogenous volume change		Drying shrinkage	Permeability, K ft/s/ft* hydraulic head	Permeability, m/s/m* hydraulic head
	90 days, microstrain	1 year, microstrain	1 year, microstrain		
Hoover	—	—	–270	1.97×10^{-12}	1.83×10^{-13}
Grand Coulee	—	—	–420	—	—
Hungry Horse	–44	–52	–520	5.87×10^{-12}	5.45×10^{-13}
Canyon Ferry	+6	–37	–397	6.12×10^{-12}	5.69×10^{-13}
Monticello	–15	–38	–998	2.60×10^{-11}	2.42×10^{-12}
Glen Canyon	–32	–61	–459	5.74×10^{-12}	5.33×10^{-13}
Flaming Gorge	—	—	–496	3.52×10^{-11}	3.27×10^{-12}
Yellowtail	–12	–38	–345	6.25×10^{-12}	5.81×10^{-13}
Dworshak	+10	–8	–510	6.02×10^{-12}	5.59×10^{-13}
Libby	+3	+12	–480	1.49×10^{-11}	1.38×10^{-12}
Lower Granite	+4	+4	—	—	—

*ft/s/ft = ft^3/ft^2 – s/ft of hydraulic head; m/s/m = m^3/m^2 – s/m of hydraulic head; measured in linear length change.

Note: Volume change specimens for Hoover and Grand Coulee Dams were 4 x 4 x 40 in. (100 x 100 x 1000 mm) prisms; for Dworshak, Libby, and Lower Granite Dams, volume change was determined on 9 x 18 in. (230 x 460 mm) sealed cylinders. Specimens for other dams were 4 x 4 x 30 in. (100 x 100 x 760 mm) prisms. Specimens for permeability for Dworshak, Libby, and Lower Granite dams were 6 x 6 in. (150 x 150 mm) cylinders. Specimens for permeability for the other dams tabulated were 18 x 18 in. (460 x 460 mm).

Table 4.1.1f—Shear properties of concrete (triaxal tests)

Dam	Age, days	*w/cm*	Compressive strength		Cohesion		tanϕ	s_s/s_c*
			psi	MPa	psi	MPa		
Grand Coulee	28	0.52	5250	36.2	1170	8.1	0.90	0.223
	28	0.58	4530	31.2	1020	7.0	0.89	0.225
	28	0.64	3810	26.3	830	5.7	0.92	0.218
	90	0.58	4750	32.8	1010	7.0	0.97	0.213
	112	0.58	4920	33.9	930	6.4	1.05	0.189
	365	0.58	8500	58.6	1880	13.0	0.91	0.221
Hungry Horse	104	0.55	2250	15.5	500	3.4	0.90	0.222
	144	0.55	3040	21.0	680	4.7	0.89	0.224
	622	0.60	1750	12.1	400	2.8	0.86	0.229
Monticello	28	0.62	2800	19.3	610	4.2	0.93	0.218
	40	0.92	4120	28.4	950	6.6	0.85	0.231
Shasta	28	0.50	5740	39.6	1140	7.9	1.05	0.199
	28	0.60	4920	33.9	1060	7.3	0.95	0.215
	90	0.50	5450	37.6	1090	7.5	1.05	0.200
	90	0.50	6590	45.4	1360	9.4	1.01	0.206
	90	0.60	5000	34.5	1040	7.2	1.00	0.208
	245	0.50	6120	42.2	1230	8.5	1.04	0.201
Dworshak	180†	0.59	4150	28.6	1490	10.3	0.44	0.359
	180†	0.63	3220	22.2	1080	7.4	0.46	0.335
	180†	0.70	2420	16.7	950	6.6	0.43	0.393
	200‡	0.59	2920	20.1	720	5.0	0.84	0.247

*Cohesion divided by compressive strength.

†Designates 18 x 36 in. (450 x 900 mm) test specimens sealed to prevent drying.

‡Designates 18 x 36 in. (450 x 900 mm) test specimens sealed to prevent drying, with 6 in. (150 mm) maximum-size aggregate.

Note: All test specimens (except † and ‡) 6 x 12 in. (150 x 300 mm) with dry, 1.5 in. (38 mm) maximum-size aggregate.

4.2—Strength

Among the most common factors that affect concrete strength are:

a) Cementitious material types, amount, composition, and fineness

b) Aggregate mineralogy, shape, quality, and gradations

c) Water content

A study of compressive strength data given in Table 4.1.1b shows a considerable variation from the direct relationship between the *w*/*cm* and strength. Lowering the *w*/*cm* will typically result in increased strength.

In general, the concrete strength in a mass concrete application should be limited to that required for the structure to meet its strength demands. Specifying excess strength will generally result in concrete proportions with increased cementitious materials content, thereby increasing temperature rise and drying shrinkage as well as possible thermal and drying shrinkage-related stresses. It may be appropriate for the design documents to specify minimum compressive strength requirements to be met at ages later than 28 days such as 56 days or even later, depending on mixture design and project requirements.

Measurement of compressive strength is often done using cylinders, and as a general rule, the cylinder diameter should be no less than three times the largest aggregate and the height-to-diameter ratio should be 2:1. For projects with concrete that use large coarse aggregates—those greater than 2 in. (50 mm)—it is often impractical to make and handle the associated large cylinders, so the concrete is typically wet screened to remove aggregate larger than 2 in. (50 mm) during the cylinder fabrication process as described in ASTM C31/C31M. This allows the use of 6 x 12 in. (150 x 300 mm) cylinders. When this practice is employed, a correlation of strength before and after wet screening is often performed early in the project, as the strength of large test specimens up to 36 x 72 in. (900 x 1800 mm) will usually be 80 to 90% of the strength of 6 x 12 in. (150 x 300 mm) cylinders tested at the same age (U.S. Bureau of Reclamation 2001).

Accelerated curing procedures, such as those described in ASTM C1768/C1768M and the alternate curing procedure in ASTM C1202, provide, at early ages, a reasonable indication of the concrete strength at later ages. The accelerated strength indicator is helpful where it satisfactorily correlates with longer-term values using companion specimens of the same concrete. Although the indicator may not accurately predict ultimate concrete properties, it can be beneficial during construction to monitor potential variations in mixing or the concrete mixture components.

For very large mass concrete projects, such as gravity dams, there are several complex factors involved in relating strength tests on small samples to the probable strength of the full-size structure. Because of these complexities, concrete strength requirements are sometimes several times the calculated maximum design stresses for mass concrete structures. For example, design criteria for gravity dams commonly used by U.S. Bureau of Reclamation (1976) and USACE EM 1110-2-2200 set the maximum allowable compressive stress for usual loading combinations at one-third of the specified concrete strength. The selection of allowable stresses and factors of safety depend on the structure type, loading conditions being analyzed, and the structure location.

Concrete with higher compressive strength tends to have higher tensile strength, but there is also large scatter in the data when plotting these strengths against each other. Tensile strength can be measured by several tests, primarily direct tension (USBR 4914 [U.S. Bureau of Reclamation 1992a]), splitting tension (ASTM C496/C496M), and modulus of rupture (ASTM C78/C78M; ASTM C293/C293M) tests. Each of these tests has a different relationship with compressive strength. The tensile strength of mass concrete varies significantly and is dependent on the age of the structure, the coarse aggregate size and type, specimen-to-aggregate ratio, and the direction the cores were taken and tested (Schrader et al. 2018). USBR data indicates that most historical dams with large coarse aggregate (in excess of 2 in. [50 mm]) in the United States have a direct tensile strength of 3.0 to 6.6% of the compressive strength (Dolen et al. 2014; Darbar et al. 2016). The tensile strength of structural concrete with small coarse aggregate (less than 2 in. [50 mm]) has typically been reported as 8 to 12% of the compressive strength (Kosmatka and Wilson 2016; Arioglu et al. 2006).

Where feasible and necessary, testing should be conducted to confirm these relationships, ideally by measurement of direct tensile strength (USBR 4914; USACE CRD-164). Measurements using various materials can and do result in factors that vary significantly from the generic factor of the aforementioned formulas, based on aggregate quality and other factors.

The strength of concrete is also influenced by the speed of loading; typical tests for concrete are static and use relatively low loading rates. Dynamic events such as earthquakes, blast, or impacts result in the development of stresses in a fraction of a second. When evaluating blast loadings, strain rate can be an important factor to consider (Schrader et al. 2018). When loaded dynamically, compressive strength of moist concrete specimens has shown increases up to 30%, and tensile strength have increased up to 50%, when compared with values obtained at standard rates of loading (Saucier 1977; Graham 1978; Raphael 1984; Harris et al. 2000). Mass concrete, however, potentially has a lower tensile capacity for lower amplitude constant stresses over a longer duration (Darbar et al. 2016). Sensitivity studies with a range of values should be considered in the absence of concrete test data to determine the effect this variation can have on a particular design or analysis.

4.3—Temperature

4.3.1 *Temperature control*—An important characteristic of mass concrete that differentiates its behavior from that of conventional concrete is its thermal behavior. This is not to say that conventional concrete does not suffer from temperature-related issues; it means that there is a recognized likelihood of temperature-related issues for concrete deemed to be mass concrete and that actions are needed to avoid such issues.

Measures to provide temperature control during construction are commonly described in thermal control plans, which include project-specific practices to control and monitor the maximum temperature and temperature difference in mass concrete. Thermal control plans are a tool to guide in planning and executing mass concrete placements during construction. Aspects related to the reinforcement configuration, structure dimensions, restraints, or all of these, in the mass concrete placements should be addressed prior to

construction in the design phase. Items typically addressed in a thermal control plan should include:

a) Concrete mixture proportions

b) Calculated or measured adiabatic temperature rise of concrete

c) Upper limit for concrete temperature at time of placement

d) Description of specific measures to control maximum temperature and maximum temperature differential

e) Calculated maximum temperature and temperature differential in placement based on expected conditions at time of placement

f) Methodology or modeling for calculation of anticipated temperature rise and temperature differentials

g) Description of equipment and procedures that will be used to monitor and log temperatures and temperature differences

h) Description of measures to address and reduce excessive temperatures and temperature differences, if they occur

i) Drawing of locations for temperature sensors in placement

j) Description of format and frequency of providing temperature data

k) Description of curing procedures, including materials, methods, and curing duration

l) Methods used for cooling concrete after placement, if used

m) Description of formwork removal procedures to ensure temperature difference at temporarily exposed surface will not exceed temperature difference limit, and how curing will be maintained

It is not always practical to avoid thermal cracking, even in situations within normal concreting practices; thus, when no cracking is required, special measures that may impact design and construction costs should be expected (Bamforth 2007). Control of cracking associated with non-thermal-related mechanisms, such as autogenous shrinkage and drying shrinkage, among other mechanisms, are not intended to be covered by thermal control plans. However, stresses from such mechanisms can increase the total stresses and can reduce the effective thermal stress that results in cracking of the concrete.

The combination of placement dimensions, timing of adjoining placements, temperature rise of the concrete, and concrete temperature at placement creates potential for significant temperatures within the interior of the placement and a large temperature difference between the interior and extremities of the placement. The four elements of an effective temperature control program, any or all of which may be used for a particular mass concrete project, are:

a) Cementitious material content control, where the choice of type and amount of cementitious materials can lessen the heat-generating potential of the concrete

b) Precooling, where cooling of ingredients achieves a lower concrete temperature as placed in the structure or forms

c) Post-cooling, where removing heat from the interior of the concrete with uniformly spaced embedded cooling pipes limits the temperature rise and differentials in the concrete structure

d) Construction management, where efforts are made to monitor temperature and protect the structure from excessive temperature differences by knowledge of insulation, concrete handling, construction scheduling, and construction procedures

The temperature control may be no more than one or two of these measures, such as using a low heat-of-hydration concrete or restricting placing operations to cool periods at night or during cool weather when the concrete temperature at placement is generally lower, these measures are easier and cheaper and, thus, are generally selected first as temperature control measures. On the other extreme, some projects may require a wide variety of separate, but complementary, temperature control of concrete measures that can include all the aforementioned strategies. The measure(s) selected are chosen by the engineer or contractor based on the predicted temperature rise and differential.

4.3.2 *Precooling and post-cooling of concrete*—Precooling of fresh concrete may be achieved by reducing the temperature of the aggregates, using chilled water or ice as part of the mixing water, and/or using liquid nitrogen to cool aggregates or the fresh concrete. Post-cooling can be achieved by use of embedded pipes in the concrete to actively cool the placement. A detailed discussion of thermal issues is contained in ACI 207.2R. Guidance in cooling systems for mass concrete can be found in ACI 207.4R.

4.3.3 *Temperature rise*—The principal means for limiting temperature rise is controlling the type and amount of cementitious materials. Controlling temperature rise of concrete is typically desired to minimize the maximum concrete temperature and reduce temperature difference of the concrete after placement during the curing period. Best practices for determining temperature rise include laboratory or mockup testing using representative construction materials and placement conditions. For preliminary design purposes, the temperature rise of the concrete in a 6 ft (1.8 m) thick or greater placement that occurs at approximately 7 to 10 equivalent age days can be roughly estimated using the following expression (Gajda et al. 2018), which is a modification of Eq. (3.1.1)

$$\text{Rise} = 0.16\cdot(\text{Cement} + 0.5\cdot\text{FAsh} + 0.8\cdot\text{CAsh} + 1.2\cdot\text{SFMK} + \text{Factor}\cdot\text{Slag}),\ ^\circ\text{F} \tag{4.3.3a}$$

$$\text{Rise} = 0.15\cdot(\text{Cement} + 0.5\cdot\text{FAsh} + 0.8\cdot\text{CAsh} + 1.2\cdot\text{SFMK} + \text{Factor}\cdot\text{Slag}),\ ^\circ\text{C} \tag{4.3.3b}$$

where the factors are the same as Eq. (3.1.1).

Equations (4.3.3a) and (4.3.3b) should not be used for final design purposes. The final thermal control plan should consider tested material properties and the effects from actual material combinations and the expected range of temperature conditions. Other methods of predicting the temperature rise of the concrete are discussed in ACI 207.2R.

4.3.4 *Maximum (peak) temperature*—Reducing the maximum temperature of the concrete decreases volume changes and tensile stresses that can lead to cracking. The maximum temperature of concrete is often limited to 160°F (70°C), mainly because certain cements can be susceptible to a durability issue known as delayed ettringite formation (DEF) when internal temperatures exceed this limit. However, not all concrete mixtures are exposed to the same level of risk for DEF when exposed to temperatures above this limit during the curing period. In certain cases, temperatures above this limit can be justified when the cementitious materials consist of a certain minimum amount of SCMs. Research (Folliard 2008; Ramlochan et al. 2003; Ferraro 2018) has suggested that maximum concrete temperatures up to 185°F (85°C) will not result in DEF when the cementitious consists of, for example:

a) 25% by mass of ASTM C618 Class F fly ash
b) 35% by mass of ASTM C618 Class C fly ash
c) 35% by mass of ASTM C989/C989M slag cement

Even though research has shown benefits in reducing DEF expansion with concrete containing SCMs, the same level of mitigation may not be always achieved with all sources of SCMs. Further information related to DEF can be found in ACI 201.2R, Day (1992), Thomas and Ramlochan (2002), Ramlochan et al. (2004), and Ferraro (2018).

4.3.5 *Temperature difference*—A large temperature difference within a concrete placement results in thermal stress. Cracking generally occurs when the tensile stresses exceed the developing tensile strength. Depending on the exposure conditions and durability requirements, concerns may arise due to cracking. Such cracking occurs at the surface of the concrete, typically first at the center of the large surfaces for concrete where restraint is primarily internal, as is the case for thermally controlled elements such as a bridge footing on steel piles and ground. External restraint can affect the location and severity of the cracking, and external restraint is addressed in detail in 207.2R. External restraint also affects thermal shrinkage stresses at the interface between placements, which is not addressed in this section.

ACI 301 requires limiting the maximum temperature difference between the center and the concrete near the surface during the curing period to a maximum of 35°F (20°C). This is a simple method that has historically worked in the absence of project-specific thermal control measures. However, this does not consider all aspects of performance.

For concretes with a low coefficient of thermal expansion, a higher temperature difference limit is sometimes used. For example, for concrete with granite coarse aggregate, a temperature difference limit of 45°F (25°C) is sometimes used, and for concrete with pure limestone coarse aggregate, a temperature difference limit of 56°F (31°C) is sometimes used (Bamforth 1984).

The aforementioned limits are simple methods intended to minimize/prevent thermal cracking of the concrete. They are by no means rigorous enough to prevent thermal cracking in all cases and may be overly conservative in most other cases. When a higher likelihood of preventing thermal cracking is required, a temperature difference limit that is a function of the developing tensile strength of the concrete is often used (Gajda 2008; Gajda and VanGeem 2002). Such a limit is based on the premise that cracking instantly occurs when the tensile stresses at the surface exceed the tensile strength at the surface of the concrete, or the tensile strain developed exceeds the tensile strain capacity of the concrete (Houghton 1972; Houghton 1976; Dusinberre 1945). The following simplified expression, derived from a series of equations, can be used to determine the limiting temperature difference associated with cracking due to internal restraint (Bamforth 1984; Crook 2006; Bamforth 2007)

$$\text{Temperature difference limit, °F (°C)} = \frac{f_t'}{E \times \text{CTE} \times R \times C} \tag{4.3.5}$$

where f_t' is tensile strength, psi (MPa); CTE is coefficient of thermal expansion, in./in/°F (mm/mm/°C); E is modulus of elasticity, psi (MPa); R is degree of restraint; and C is creep factor.

Equation (4.3.5) is used to develop an approximate relationship between the temperature difference limit and the developed in-place properties of the concrete. It is not intended to be used as a single temperature difference limit that does not change with age or in-place properties, such as the 35°F (20°C) limit. It is intended to be used with properties at the exterior surface of the concrete, which is where cracking first occurs. With Eq. (4.3.5), the restraint is typically assumed based on the placement type (Bamforth 2007; Bamforth and Price 1995). Refer to ACI 207.2R for additional detail regarding restraint. Considerations associated with early-age creep affect the temperature difference limit calculations. Because creep testing is relatively time-consuming, a creep factor is often assumed from values reported in technical literature (Bamforth 2007; Crook 2006).

Equation (4.3.5) is approximate because the material properties and restraint factor are difficult, at best, to know accurately. Developers and users of thermal control plans should be aware that the calculated temperature difference limit from Eq. (4.3.5) will only be as accurate as the accuracy of the assumptions made for the input parameters, which change with time.

Users are cautioned that Eq. (4.3.5) does not incorporate safety factors that would account for variations that are inherent to concrete.

4.4—Elastic properties

Concrete is not a truly elastic material, and the graphic stress-strain relationship for continuously increasing load is not linear. For practical purposes, however, the modulus of elasticity is often approximated as a constant within the range of stresses to which mass concrete is usually subjected, up to a maximum of 40% of the ultimate stress.

The compressive moduli of elasticity of concrete representative of various dams are given in Table 4.1.1c. These values range from 2.8 to 5.5 × 10^6 psi (1.9 to 3.8 × 10^4 MPa) at 28 days and from 3.8 to 6.8 × 10^6 psi (2.6 to 4.7 ×

10^4 MPa) at 1 year. For a given aggregate type, concrete with higher strengths tend to have higher elastic moduli. The stiffness of the aggregate is important, in part because the large aggregate constitutes a very large percentage of the concrete volume, and the aggregate stiffness can vary substantially with the mineralogy of the aggregate. Values for the tensile modulus of elasticity are generally slightly less than the compressive elastic modulus. Research has shown that tensile modulus of elasticity is approximately 10% lower than compressive modulus of elasticity (Ferraro 2009; Balendran 1995). For simplification, however, it is often approximated to be the same as the compressive modulus of elasticity (Philleo 1966).

The Poisson's ratio data given in Table 4.1.1c tend to range between the values of 0.16 and 0.20, with small increases with increasing time of cure. Extreme values may vary from 0.11 to 0.27. Poisson's ratio, like modulus of elasticity, is influenced by the aggregate, cement paste, and relative proportions of the two.

The results of several investigations indicate that the modulus of elasticity appears to be relatively unchanged whether tested at normal or dynamic rates of loading (Hess 1992). Poisson's ratio can be considered the same for normal or dynamic rates of loading (Hess 1992).

When the stress during short-term loading exceeds approximately 50% of the ultimate stress, internal microcracks develop, and Poisson's ratio and elastic moduli becomes less linear due to the increased strain.

4.5—Creep

Creep of concrete is a time-dependent deformation due to a sustained load, which reduces stress in mass concrete. Creep generally decreases with increasing concrete strength, increasing elastic modulus and over time (Smith and Hammons 1993). With all mixture parameters being equal, the risk of cracking increases as the concrete strength increases, as there is less stress relief from creep. Concrete with lower strength and lower modulus of elasticity is more likely to realize the benefits from creep. The behavior of mass concrete with a relatively lower rate of hydration was reported where a concrete mixture with high volume of fly ash had slower development of mechanical properties, which resulted in higher rates of stress relaxation (creep) and was less prone to cracking (Zhao et al. 2019). With concrete containing the same type of aggregate, the magnitude of creep is closely related to the paste content (Polivka et al. 1963) and the *w/cm* of the concrete. ACI 209R discusses the prediction of creep, shrinkage, and temperature effects in concrete structures.

The contribution of tensile creep in the mitigation of early-age cracking in mass concrete has not been well documented (Zhao et al. 2019). Therefore, other methods that can be used to indirectly account for contributions from creep are favored. One method of expressing the effect of creep is as the sustained modulus of elasticity of the concrete in which the stress is divided by the total deformation for the time under the load. The instantaneous and sustained modulus of elasticity values obtained on 6 in. (150 mm) diameter cylinders made with mass concrete that is wet screened to remove coarse aggregate larger than 1-1/2 in. (38 mm) are recorded in Table 4.1.1d. The instantaneous modulus is measured immediately after subjecting the concrete to loading. The sustained modulus, also called the effective modulus, represents values after 365 and 1000 days under loading. Table 4.1.1d shows that the sustained long-term values for the modulus of elasticity are approximately half that of the instantaneous modulus when load is applied at early ages and is a slightly higher percentage of the instantaneous modulus of elasticity when the loading age is 90 days or greater. Long-term creep of concrete appears to be approximately directly proportional to the applied stress-strength ratio, up to approximately 40% of the ultimate strength of the concrete. Above 40%, microcracking develops, which affects the modulus, as described in Section 4.4 (Liners 1987).

When examining short-term creep, such as when evaluating early hydration temperature stresses, creep values can be very high and much larger than those for long-term loading. The early creep values can also change dramatically during the first few days, and testing can be appropriate when these values could be important to the evaluation (Altoubat and Lange 2001; Bamforth 2018).

4.6—Volume change

Volume change occurs primarily from changes in concrete temperature, moisture content, chemical reactions, and applied loads. Cracks form in restrained concrete when the resulting stresses exceed the tensile strength or strain capacity of the concrete.

Cracking is more likely to occur in restrained structures, but can also occur in structures with little restraint, from large temperature differences developed due to the heat of hydration and associated temperature rise at the interior of the concrete element. Figure 4.6 shows such cracking due to a large temperature difference between the interior and surface of a mass concrete placement. Crack widths can be expected to increase over time due to combined effects of autogenous shrinkage, drying shrinkage, or both.

Fig. 4.6—Cracked concrete hammerhead pier cap.

Cracking in concrete structures, while expected, can adversely affect durability, visual appearance, and structural integrity. Volume change data for some mass concrete are given in Table 4.1.1e. Various factors influencing cracking of mass concrete are discussed in ACI 207.2R and USACE ETL 1110-2-542.

Drying shrinkage of concrete occurs at surfaces that are exposed to air. Drying shrinkage is a longer-term consideration and ranges from less than 0.02% (or 200 microstrain) for low-slump lean concrete with well-graded and high-quality aggregates, to over 0.10% (or 1000 microstrain) for rich mortars or concrete with high paste content, low-quality aggregates, and high water content (Neville 2011). Drying shrinkage occurs from the loss of moisture within the cement paste fraction, which can shrink due to drying as much as 1%. Aggregates can provide internal restraint that substantially reduces this value. The magnitude of drying shrinkage is influenced mainly by the paste and water volume, the type of aggregate, and external factors such as the prevailing ambient temperature and relative humidity. The addition of SCMs can increase drying shrinkage except where the water requirement is reduced. Some aggregates have been known to contribute to high drying shrinkage. ACI 224R and Houghton (1972) discuss the factors involved in drying characteristics of concrete.

Autogenous shrinkage is a reduction in volume produced during the hydration of cement, regardless of drying to the environment or external loading. Concretes with a *w*/*cm* greater than 0.42 have a negligible autogenous shrinkage (Holt 2001). Autogenous shrinkage occurs with increasing magnitude in concretes with a lower *w*/*cm* and can be as high as 200 to 400 microstrain when the *w*/*cm* is 0.30.

The coefficient of thermal expansion (CTE) of concrete reflects the types and amounts of constituent materials in the concrete, including the cementitious materials, water, coarse aggregate, and fine aggregate. In most conventional concretes, coarse aggregate typically is the largest volume and therefore has the most influence on the CTE of the concrete. The CTE of aggregate ranges from approximately 2 to 8 microstrain per °F (1.4 to 4.5 microstrain per °C), depending on the minerology (ACI 207.2R). The CTE of cementitious paste varies from approximately 6 to 14 microstrain per °F (3.3 to 7.8 microstrain per °C), depending on the types and amounts of the individual cementitious materials, the *w*/*cm*, and degree of saturation (Neville 2011; Helmuth 1961). The CTE of fully saturated normalweight concrete is typically in the range of 3.5 to 7 microstrain per °F (1.9 to 2.5 microstrain per °C) (Hall and Tayabji 2011).

The degree of saturation of the concrete also affects its CTE. AASHTO T 336 is typically used to measure the CTE of saturated concrete. This test method requires a 4 x 8 in. (100 x 200 mm) cylinder, which is cut to be 7 in. (175 mm) long. For concretes with coarse aggregates larger than 1 in. (25 mm), wet screening of aggregates will disproportionally increase the cementitious paste content and will usually result in a higher CTE. Testing multiple cores to obtain a statically appropriate sample may be the better approach when measuring the CTE of large aggregate concretes.

4.7—Permeability

Permeability is not typically a concern for traditional mass concrete. The typical permeability of the mass itself is low, on the order of 10^{-11} to 10^{-12} ft/s. Permeability coefficients using USBR 4913 for some mass concretes are given in Table 4.1.1e. It is important for thermally controlled concrete to protect the reinforcement from corrosion. When concrete mixtures are properly proportioned, placed, and not subject to cracking, issues related to durability from permeability are mitigated.

Air-entraining and other chemical admixtures permit significant benefits in concrete such as sufficient workability with reduced water content and, therefore, contribute to reduced permeability. The use of SCMs in mass concrete also significantly reduce the permeability of the concrete at later ages, as can be loosely correlated to ASTM C1202 test results (Tibbets et al. 2020).

4.8—Thermal properties

The main thermal material properties that influence mass concrete are specific heat, density, thermal conductivity, thermal diffusivity, coefficient of thermal expansion, and adiabatic temperature rise. The mineralogical composition of the aggregate is usually the primary factor affecting the thermal properties of a concrete (Rhodes 1978) because coarse aggregates generally comprise the largest proportion of the concrete volume. Requirements for cement, SCM, ratio of coarse to fine aggregate, and water content are modifying factors but offer a negligible effect on thermal properties. Entrained air is an insulator and reduces thermal conductivity, but not significantly. ACI 207.2R provides additional discussion on the thermal property values for mass concrete.

4.9—Shear properties

For dams, the potential failure mode of cracking and sliding within the mass concrete or at the concrete/foundation interface requires careful consideration. Sliding stability relies on the shear strength of the concrete mass itself, without reinforcement. Shear properties are especially important at horizontally oriented joints because sliding stability along these joints is often the most vulnerable part of a dam and a potential mode of failure. It is important to determine the correct site-specific concrete shear strength properties for an analysis to be accurate. Assigning realistic shear strength parameters within the parent concrete and at the joints requires laboratory testing of specimens from the subject structure or concrete mixture to be used. Engineering judgment is used in upscaling the laboratory parameters to the field scale.

Shear properties for some concrete containing 1-1/2 in. (38 mm) maximum size aggregates are listed in Table 4.1.1f. This table also includes the site-specific concrete properties of compressive strength, cohesion, and coefficient of internal friction, which are related linear functions determined from results of triaxial tests. Linear analysis of triaxial results gives a shear strength slightly above the value obtained from biaxial shear strength (U.S. Bureau of Reclamation 1992a). Past criteria have stated that the coefficient of internal fric-

tion can be taken as 1.0 and cohesion as 10% of the compressive strength (U.S. Bureau of Reclamation 1976). Investigations have concluded that assuming this level of cohesion may be unconservative (McLean and Pierce 1988), and additional testing may be necessary.

Recent work analyzing historic laboratory data has found that other laboratory-determined properties (direct tensile, splitting tensile, and uniaxial compressive strengths) are poor predictors of the shear strength of concrete (Lindenbach 2017). A recent analysis of over 1000 shear strength tests provides realistic ranges of parameters for an initial design. However, the author cautions that given the broad range of values provided and the lack of a good correlation to other properties, shear strength of concrete for design purposes should be determined through laboratory testing of specimens from the structure being analyzed (Lindenbach 2017). The shear strength relationships reported are typically analyzed using the Mohr envelope equation $Y = C + X\tan\phi$, in which C (unit cohesive strength or cohesion) is defined as the shear strength at zero normal stress; $\tan\phi$, which is the slope of the line, represents the coefficient of internal friction; and X and Y are normal and shear stresses, respectively. Note that the use of a linear Mohr-Coulomb failure envelope is a linearization of a small portion of the larger curved failure envelope. The slope and intercept (internal friction and cohesion, respectively) of the linearization depend on the normal stresses applied during testing; extrapolating shear strength parameters for normal stresses outside of those used to develop the linear envelope should be performed with caution. As the mobilized shear strength envelope has a nonlinear concave-down shape, there will be a steeper failure envelope (larger friction angle and smaller cohesion) at lower normal stresses, likely also due to the amount of particle breakage along the sliding interface.

In many cases, the shear strengths in Table 4.1.1f were higher for older specimens; however, there is not a definable trend (Harboe 1961). The ratio of triaxial shear strength to compressive strength varies from 0.19 to 0.39 for the various concretes shown. When shear strength is used for design, the confining pressures used during testing should reflect anticipated conditions in the structure. Whenever possible, direct shear tests on both parent concrete and on jointed concrete should be conducted in accordance with USBR 4915 (U.S. Bureau of Reclamation 1992b) and/or ASTM D5607 to determine valid cohesion and coefficient of internal friction values for design. It is important to note is that the mobilized shear strength has been shown to decrease significantly with increasing shear displacement (Yathon et al. 2019). This phenomenon is likely due to shear surface degradation during sliding and needs to be considered when determining the correct testing procedure and shear strength parameters for the anticipated boundary condition. Bonded horizontal construction joints may have shear strength comparable to that of the parent concrete. Unbonded joints typically have lower cohesion, but the same coefficient of internal friction, when compared with the parent concrete. If no tests are conducted, the coefficient of internal friction can be taken at 1.0 and the cohesion as 0 for unbonded joints. For bonded joints, the coefficient of internal friction can be taken as 1.0, while the cohesion may approach that of the parent concrete (McLean and Pierce 1988). Lindenbach (2017) provides a significant amount of data from an extensive library of testing, which can aid the practitioner in selecting appropriate ranges of shear strength parameters (Lindenbach 2017).

4.10—Durability

A durable concrete is one that has the ability to resist weathering action, chemical attack, abrasion, and other conditions of service. Laboratory tests can indicate relative durability of concrete, but it is usually not possible to directly predict durability in field service from laboratory testing. While the durability of the concrete itself is important, eliminating or reducing cracking can be as important as the overall durability of the structure. ACI 318 contains durability requirements for structural concrete.

Disintegration of uncracked concrete by weathering is mainly caused by the disruptive action of freezing and thawing while the concrete is critically saturated and by expansion and contraction under restraint, resulting from temperature variations and alternate wetting and drying. Entrained air improves the resistance of concrete to damage from frost action and should be specified for all concrete subject to cycles of freezing and thawing while critically saturated. Selection of high-quality materials, use of entrained air, low *w/cm*, proper mixture proportioning, proper placement techniques to provide a watertight structure, and good water curing usually provide a concrete that has excellent resistance to weathering action (ACI 201.2R).

Chemical attack occurs from exposure to acidic waters, exposure to sulfate-bearing waters, and leaching by mineral-free waters, as explained in ACI 201.2R. Typically, portland-cement concrete is not very resistant to attack by acids. Should this type of exposure occur, the concrete is best protected by surface coatings.

Sulfate attack can be rapid and severe. The sulfates react chemically with the calcium hydroxide and hydrated tricalcium aluminate in cement paste to form calcium sulfate and calcium sulfoaluminates. These reactions are accompanied by considerable expansion and disruption of the concrete. Concrete containing cement low in tricalcium aluminate (ASTM Types II, IV, and V) is more resistant to attack by sulfates (ASTM C150/C150M). SCMs also provide resistance to sulfate attack.

Calcium hydroxide is one of the products formed when cement and water combine in concrete and is readily dissolved in pure or slightly acidic water that may occur in high mountain streams. SCMs that react with calcium hydroxide liberated by cement hydration can prevent leaching. Surfaces of tunnel linings, retaining walls, piers, and other structures are often disfigured by efflorescence from water seeping through cracks, joints, and interconnected voids. With dense, low-permeability concrete, leaching is seldom severe enough to impair the serviceability of the structure.

The principal causes of erosion of concrete surfaces are cavitation and the movement of abrasive material by

flowing water. Use of increased-strength and wear-resistant concrete offers some relief, but the best solution lies in the prevention, elimination, or reduction of the causes by proper design, construction, and operation of the concrete structure (ACI 207.6R). Refer to ACI 207.6R for additional information about erosion resistance and cavitation in mass concrete structures.

Alkali-aggregate reactivity (AAR) is the chemical reaction between alkalis (sodium and potassium) from portland cement or other sources, and certain constituents of some aggregates that, under certain conditions such as high moisture, produces deleterious expansion of the concrete. These reactions include alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR), which are discussed elsewhere in this document. Refer to ACI 201.2R for further information on durability.

CHAPTER 5—CONSTRUCTION

5.1—Batching

Proper batching of mass concrete requires the same accurate, consistent, reliable batching that is essential for other types of concrete. Batch plants should be calibrated to ensure accurate and precise production in conformance with specified requirements. Certification of batch plants may be required to meet industry standards, such as National Ready Mixed Concrete Association (NRMCA). ACI 304R presents information on the handling, measuring, and batching of all the materials used in making concrete. ACI 221R presents information on selection and use of aggregates in concrete. Batching tolerances are provided in ASTM C94/C94M.

The desirability of restricting the temperature rise of mass concrete by limiting the cementitious content of the mixture creates a continuing construction challenge to maintain workability in the plastic concrete. Efficiently optimized mixtures for mass concrete typically contain unusually low portions of cementitious materials, sand, and water. Thus, the workability of these mixtures for conventional placement is more sensitive to normal variations in batching. Consistency in the gradation of coarse and fine aggregate as well as consistency in the cleanliness of the aggregate is extremely helpful in producing consistent uniform concrete. In addition, constant knowledge of aggregate moisture and measures to maintain a relatively constant moisture are also extremely helpful; this typically involves constantly sprinkling aggregate stockpiles (during nonfreezing conditions) and moisture probes in coarse and fine aggregate bins.

Usually, the production of large quantities of mass concrete is like an assembly-line operation, particularly in dam construction, where the performance of repetitive functions makes it economically prudent to use specialty equipment and efficient construction methods.

When ice replaces batch water, for the most efficient use of ice, it should be less than 32°F (0°C) and be brittle-hard, dry, and finely broken. For maximum efficiency, use automated equipment. Ice should be batched by weighing from a well-insulated storage bin, with quick discharge into the mixer along with the other ingredients.

Liquid admixtures are generally batched by volume, although weighing equipment has also been used successfully. Reliable admixture batching equipment is available from admixture or batch plant manufacturers. Means should be provided for making a visual accuracy check. Provisions should be made for preventing batching of an admixture while the discharge valve is open. Interlocks should also be provided that will prevent inadvertent over-batching of admixtures. Any irregularities in batching admixtures can cause troublesome variations in slump, air content, or both. When several liquid admixtures are to be used, they should be batched separately into the mixer or in accordance with the admixture manufacturer's recommendations. For continuing good operation, equipment should be maintained and kept clean. Timed-flow systems should not be used. Also, it is important to provide winter protection for storage tanks and related delivery lines where necessary.

Each size fraction of aggregates should be stored in separate stockpiles and bins to prevent segregation and to ensure consistency. One example is that a stockpile of ASTM C33/C33M No. 467 aggregate tends to segregate; therefore, it is better to have separate stockpiles and bins for No. 4 and No. 67 aggregates.

5.2—Mixing

Mixers for traditional mass concrete should be capable of discharging low-slump concrete quickly and with consistent distribution of large aggregate throughout the batch. This is best accomplished with large, tilting mixers in stationary central plants. The most common capacity of the mixer drum is 4 yd^3 (3 m^3), but good results have been achieved with mixers as small as 2 yd^3 (1.5 m^3) and as large as 12 yd^3 (9 m^3). Truck mixers are not suited to the mixing and discharging of low-slump large-aggregate concrete. Turbine-type and compulsory mixers are commonly used mixers may be used for mass concrete containing 3 in. (75 mm) in size. Mass concrete used in buildings and bridges commonly has a 3/4 to 1 in. (19 to 25 mm) maximum aggregate size and can be dry batched, mixed, and delivered as a typical concrete with truck mixers. Batching and delivering equipment should be certified by the NRMCA or appropriate agency.

Specifications for traditional mass concrete, include mixing time range from a minimum of 1 minute for the first cubic yard plus 15 seconds for each additional cubic yard (80 seconds for first cubic meter plus 20 seconds for each additional cubic meter) of mixer capacity (ACI 304R; ASTM C94/C94M) to 1.5 minutes for the first 2 yd^3 plus 30 seconds for each additional cubic yard (1.5 minutes for the first 1.5 m^3 plus 40 seconds for each additional cubic meter) of capacity (U.S. Bureau of Reclamation 1981). Blending the materials by ribbon feeding during batching makes it possible to reduce the mixing period. Some of the mixing water and coarser aggregate should lead other materials into the mixer to prevent sticking and clogging. Mixing time may be reduced based on results of mixer uniformity testing (ASTM C94/C94M). Mixing time is best controlled by a timing device that prevents release of the discharge mechanism until the mixing time has elapsed.

During mixing, the batch should be closely observed to ensure the desired workability is consistently produced. The operator and inspector should be alert and attentive. Tuthill (1950) discussed effective inspection procedures and facilities. The operator should be able to easily see, either firsthand or with cameras installed near the mixer, the concrete batch and be able to judge whether its slump is correct so that adjustments can be made prior to discharge. Amperage meters are often used to assist visual observations.

Continuous batching and mixing (pugmill) have been used successfully in roller-compacted concrete for years, and in some instances has also been used for traditional mass concrete with satisfactory performance. Generally, the maximum aggregate size for this method is limited to 3 in. (75 mm) or possibly 4 in. (100 mm). ACI 207.5R and ACI 304R discuss continuous batching and mixing in more detail.

5.3—Placing

Placing includes preparation of horizontal construction joints (mainly for traditional mass concrete), transportation, handling, placement, and consolidation of the concrete (ACI 304R; U.S. Bureau of Reclamation 2001).

The most appropriate preparation of horizontal joint surfaces begins with the activities of topping out the placement. The surface should be left free from protruding rock, deep footprints, vibrator holes, and other surface irregularities. In general, the surface should be relatively even, with a gentle slope for drainage. This slope makes the surface cleanup easier prior to placement of the subsequent lift. As late as is feasible, but before adjoining concrete is placed, laitance and contamination should be removed to expose a fresh, clean mortar and aggregate surface. Overcutting to deeply expose aggregate is unnecessary and wasteful of good material. Bond strength is accomplished by exposing and allowing direct contact of cement grains, not by exposing the coarse aggregate. Joint shear strength is determined both by this bond and by interface friction (Lindenbach 2017; McLean and Pierce 1988). The friction contribution is affected by confining pressure and coarse aggregate interlock. Usually, removal of approximately 0.1 in. (a few millimeters) of a material will reveal a satisfactory surface suitable for bonding, depending on how the construction joint was cured.

The best method for obtaining suitable construction joints for traditional mass concrete is to green cut the concrete in a timely manner using high-pressure water blasting (Neeley and Poole 1996). The use of raking or roughening the surface of the fresh concrete should be avoided, as bond and the ability to clean the surface are degraded.

Surface laitance, poorly embedded aggregate, and other contaminants are typically removed from hardened concrete by hydro-blasting prior to placement of adjoining concrete. Depending on the water temperature, the concrete interior temperature, and the surface temperature, the use of cold water during the hydro-blasting operation may need to be avoided to prevent thermal shock. This will be exacerbated during periods of high winds or if the curing water evaporates.

Fig. 5.3a—Before and after horizontal construction joint cleanup using hydro blasting.

Before and after horizontal construction joint cleanup with hydro-blasting is illustrated in Fig. 5.3a. Clean joints are essential to good bond and watertightness.

The clean concrete surface should be free from excess surface moisture at the time new concrete is placed on it (USACE 1959, 1963; Tynes and McCleese 1973; Neeley and Poole 1996; Neeley et al. 1998). The concrete should be close to saturated surface-dry (SSD) and free of ponding or standing water. Testing has shown superior strength and watertightness of joints that are clean and free of excess water when the overlying concrete is placed. Tests have also shown that the practice of placing mortar on the joint ahead of the concrete is not necessary for either strength or permeability of the joint (Houghton and Hall 1972). The mortar coat, although widely used in the past, is no longer commonly used in mass concrete construction. Equivalent results can be obtained without the mortar if the first layer of the plastic concrete is thoroughly vibrated over the joint area and all rock clusters at batch-dump perimeters are carefully scattered.

Selection of equipment for transporting and placing mass concrete is strongly influenced by the maximum size of the aggregate. Concrete for mass placements, such as in dams, often contains cobbles, which are defined as coarse-aggregate particles larger than 3 in. (75 mm) and smaller than 12 in. (300 mm). The tendency of cobbles to segregate from the mixture as a result of their greater inertia when in motion may dictate the use of large, 2 to 12 yd^3 (1.5 to 9 m^3) capacity buckets. Railcars, trucks, cableways, cranes, or some combination of these may be used to deliver the buckets to the point of placement. For concrete containing coarse aggregate 3 in. (75 mm) and larger, a bucket size of 4 to 8 yd^3 (3 to 6 m^3) is preferable because smaller buckets do not discharge as readily, and each delivery is too small to work well with a high-production placement scheme. On the other hand, the 12 yd^3 (9 m^3) bucket puts such a large pile in one place that

(a)

(b)

Fig. 5.3b—(a) Placement of mass concrete by conveyor belt in a traditional mass concrete placement; and (b) in a thermally-controlled mass concrete placement.

much of the crew's time is devoted to vibrating for spreading instead of for consolidation. To preclude these piles being larger than 4 yd^3 (3 m^3), one agency requires controllable discharge gates in buckets carrying more than 4 yd^3 (3 m^3). Extra care should be taken to ensure ample vibration deep in the center of these piles and at points of contact with concrete previously placed. Mass concrete of proper mixture proportions and low slump does not separate by settlement during such transportation over the short distances usually involved. Care should be taken, however, to prevent segregation at each transfer point.

Traditional mass concrete may also be transported in dumping rail cars and non-agitating trucks and placed by use of conveyors. Placing mass concrete with conveyors has been most successful and economical when the aggregate size is 4 in. (100 mm) or less. The point of discharge from conveyors should be managed so that concrete is discharged onto fresh concrete and immediately vibrated to prevent stacking. Placement of mass concrete by conveyor is shown in Fig. 5.3b. Additional information on placing concrete with conveyors is contained in ACI 304.4R.

Availability and job conditions may preclude the use of aggregates larger than 1-1/2 in. (38 mm) or specialized placement equipment. Concrete in such structures may be placed with more conventional equipment such as smaller crane buckets, concrete pumps, or conveyors. The selection of placing equipment should be predicated on its ability to successfully place concrete that has been proportioned for mass concrete considerations as defined in Section 3.7, which emphasizes the reduction of heat evolution. Placing capacity and concrete supply should be great enough to avoid cold joints and undesirable exposure to extremes of heat and cold at placement surfaces. This is usually accomplished by using many pieces of placing equipment. Additional information on pumping of concrete is contained in ACI 304.2R.

Traditional mass concrete is best placed in successive layers. The maximum thickness of the layer depends on the ability of the vibrators to properly consolidate the concrete and combine the layers. A 6 in. (150 mm) diameter vibrator produces satisfactory results with 4 to 6 in. (100 to 150 mm) nominal maximum-size aggregate and less than 1.5 in. (40 mm) slump in layers 18 to 20 in. (460 to 510 mm) thick placed with 4 to 8 yd^3 (3 to 6 m^3) buckets. Smaller-diameter vibrators will produce satisfactory results with 3 to 4 in. (75 to 100 mm) nominal maximum-size aggregate and less than 2 in. (50 mm) slump placed in 12 to 15 in. (300 to 380 mm) layers with smaller buckets. Shallower layers, rather than deeper layers, give better assurance of satisfactory consolidation and freedom from rock pockets at joint lines, corners, and other form faces, as well as within the block itself.

The layer thickness should be an even fraction of the lift height or of the depth of the block. For low-slump concrete, the layers are carried forward in a stair-step fashion in the block by means of successive discharges, so there will be a setback of approximately 5 ft (1.5 m) between the forward edges of successive layers, as shown in Fig. 5.3b. Placement of the steps is organized to expose a minimum of surface to lessen warming of the concrete in warm weather and reduce the area affected by rain during wet weather. A setback greater than 5 ft (1.5 m) unnecessarily exposes cold concrete to heat gain in warm weather and, in rainy weather, increases the danger of water damage. A narrower setback will cause concrete above it to sag when the step is vibrated to make it monolithic with the concrete placed later against that step. This stepped front progresses forward from one end of the block to the other until the form is filled and the lift placement is completed.

Vibration is the key to the successful placement of mass concrete, particularly when the concrete is low slump and contains large aggregate (Tuthill 1953). Ineffective equipment is more costly to the contractor because of a slower placing rate and the hazard of poor consolidation. Vibration should be systematic and should thoroughly cover and deeply penetrate each layer. Particular attention should be paid to ensure full vibration where the perimeters of two discharges join because the outer edge of the first batch is not vibrated until the next batch is placed against it. The two discharges can then be vibrated monolithically together without causing either edge to flow downward. Proper vibration of large-aggregate mass concrete is shown in Fig. 5.3c. To ensure

Fig. 5.3c—Consolidation of low-slump mass concrete placed by bucket.

proper consolidation, the vibrators should penetrate the lower layer for 2 to 4 in. (50 to 100 mm) and be maintained in a vertical position at each penetration during vibration. To prevent imperfections along lift lines and layer lines at form faces, these areas should be systematically deeply revibrated as each layer advances from the starting form, along each of the side forms, to the other end form. Any visible clusters of separated coarse aggregate should be scattered on the new concrete before covering with additional concrete. Vibration is unlikely to fill and solidify unseparated aggregate clusters with mortar. During consolidation, the vibrators should remain at each penetration point until large air bubbles have ceased to rise and escape from the concrete. The average time for one vibrator to fully consolidate 1 yd^3 (3/4 m^3) of concrete may be as much as 1 minute (80 seconds for 1 m^3). Over-vibration of low-slump mass concrete is unlikely; however, over-vibration of higher-slump concrete will cause segregation.

To simplify cleanup operations, the top of the uppermost layer should be leveled and made reasonably even by means of vibration. Holes from previous vibrator insertions should be closed. Large aggregate should be almost completely embedded. If such work is done by manual means, boards should be laid on the surface in sufficient number to prevent deep footprints. Ample and effective vibration equipment should be available and in use during the placement of mass concrete. Specific recommendations for mass concrete vibration are contained in ACI 309R.

Mass concrete for underwater placements is done without vibration. Generally, the mixture is proportioned with a relatively high cementitious materials content and a reduced aggregate size to promote the required lateral flow of the mixture after the mixture is introduced into the placement area by tremie pipe or pumpline. It is more common to incorporate an anti-washout admixture and water-reducing admixtures into the mixture to increase the flow of the mixture, decrease washout of the paste, and improve consolidation of the mixture. Typical applications include bridge pier tremie seals, repair of stilling basins and other in-water structures, and placement of float-in structures.

5.4—Curing

The use of water for curing mass concrete should be avoided. However, mass concrete can be cured with water as long as this does not cause the exterior surface to rapidly cool and create a condition of an excessively high temperature difference between the surface and interior of the placement. Even when not the most practical or economical alternative, heated water may be specified for curing in some cases. Moisture retention curing practices, which are more commonly used, are typically preferred and recommended.

In mixtures with very low cement content, which exhibit low temperature rise, water curing can provide additional cooling benefit in warm weather. Water curing can also be beneficial to mass concrete surfaces that will be exposed to rapid flowing water or possible erosive, abrasive conditions.

Except when evaporation rates are high (during hot or cold weather, or when winds are significant), little curing is needed beyond the moisture provided to prevent the concrete from drying prior to installation of moisture retention curing methods; however, the concrete should not be saturated when it is exposed to freezing temperatures. In above-freezing weather, when moisture is likely to be lost from the concrete surfaces, mass concrete should be protected from losing moisture.

Curing methods for mass and thermally controlled concrete should prevent cracking of the concrete. The duration of curing for mass concrete is primarily controlled by the items addressed within the thermal control plan. Curing procedures should consider temperature control and formwork removal procedures to ensure maximum temperature difference limits are not exceeded.

The use of a liquid-membrane curing compound is an acceptable alternative where water curing is not practical, such as in below-freezing conditions, where the application of water may result in rapid cooling of the surface, or a high temperature difference. If a curing compound is used on construction joints, it should be completely removed by sandblasting or water-blasting to prevent reduction or loss of bond.

5.5—Forms

Forms for mass concrete have the same basic requirements for strength, mortar-tightness, accuracy of position, and generally good surface condition, as described in ACI 347R. Formwork for traditional mass concrete may differ somewhat from other formwork because of the comparatively low height normally required for each placement, which is traditionally 5 to 10 ft (1.5 to 3 m). There may be some increase of form pressures due to the use of low-temperature concrete and the impact of dumping large buckets of concrete near the forms, despite the relieving effect of the generally low slump of mass concrete. Form pressures depend on the methods used and the care exercised in placing concrete adjacent to the form. Further information about formwork pressure can be found in ACI 347R.

For traditional mass concrete, form ties connected to standard anchors in the previous lift and braces have long been used. Many large jobs are now equipped with forms supported by cantilevered strongbacks anchored firmly into the lift below. Additional support of cantilevered forms may be provided by form ties, particularly when the concrete is low in early strength. Cantilevered forms are raised by hydraulic, air, or electric jacking systems. Care should be taken to avoid spalling concrete around the anchor bolts in the low-early-strength concrete of the lift being stripped of forms because these bolts will be used to provide horizontal restraint in the next form setup. High-lift, mass concrete formwork is comparable to that used for standard structural concrete work except that ties may be 20 to 40 ft (6 to 12 m) long across the lift rather than 12 to 40 in. (0.3 to 1.0 m). To facilitate placement by bucket, widely spaced large-diameter, high-tensile-strength ties should be used to permit passage of the concrete buckets.

Beveled grade strips and 1 in. (25 mm) or larger triangular toe fillets can be used to mask offsets that sometimes occur at horizontal joint lines in traditional mass concrete placements. This will generally improve the appearance of formed surfaces. When used at the top and bottom of the forms, this can create an effective and aesthetically pleasing groove. A 1 in. (25 mm) or larger chamfer should also be used in the corners of the forms at the upstream and downstream ends of construction joints for the sake of appearance and to prevent chipping of the edges; otherwise, sharp corners of the block are often damaged and cannot be effectively repaired. Such chamfers also prevent pinching and spalling of joint edges caused by high surface temperatures.

Sloping forms, when used, often extend over the construction joint to the extent that it is difficult to position buckets close enough to place and adequately consolidate the concrete. Such forms may be hinged so the top half can be held in a vertical position until concrete is placed up to the hinged elevation. The top half is then lowered into position and concrete placement is continued. Sloping forms are subject to less outward pressure, but uplift should be considered in their anchorage.

A common forming problem for spillway sections of gravity dams is encountered in the sloping and curved portions of the crest and bucket. The curved or sloped surfaces can be effectively shaped, thoroughly consolidated, and finished with the use of temporary forms rather than using screed guides and strike-off. A well-proportioned regular mass concrete face mixture can be finished as easily as a mixture with small aggregate; however, sometimes a different concrete mixture is required on the spillway face for durability reasons. The desired shape is achieved with skilled finishers, as shown in Fig. 5.5. Certified flatwork finishers are recommended for these complicated geometric shapes. A curved guide rail and roller screed have also been successfully used for ogee crests and spillway curves. Finishing of concrete surfaces for water conveyance structures such as spillways and rollways are of critical importance to mitigate the risk of deterioration caused by cavitation.

Fig. 5.5—Finishing the curved spillway of the Minidoka Dam.

5.6—Placement height and time between adjoining placements

Control of temperature rise is a function of various factors, including placement heights and placing frequency. Influencing factors are size and type of massive structure, concrete properties and cementitious content, prevailing climate during construction and during service, construction schedule, and other specified temperature controls. Placement heights range from 2.5 ft (0.75 m) for multiple stacked placement just above foundations to 5 and 7.5 ft (1.5 and 2.3 m) in many gravity dams, and to 10 ft (3 m) or more in thin arch dams, piers, pile caps, and abutments.

From the standpoint of construction, higher placement heights result in fewer construction joints. Regarding past experience of hardened concrete temperature in cold weather, the shallower the placement, the higher the percentage of the total heat of hydration that will escape before the next placement is made. In hot weather with lean mixtures and precooling, the opposite may be true. When the placement thickness is increased above 10 ft (3 m), heat losses from the upper surface become a decreasing percentage of the total heat generated within the full depth of the placement. Hence, with very deep placement, the internal temperature reached by the concrete is not significantly influenced by the length of the time interval between placements. In such infrequent extreme cases, minimal time between thick stacked placements may be preferable, especially as a means of minimizing joint cleanup, mitigating the risk of cracking, or permitting the use of slipforms, such as for massive piers. In large blocks, such as in dam construction, the loss of heat from a placement surface in cold weather does not justify extended exposure to the environment, provided a means of internal cooling is provided. A long exposure of placement surfaces to changes in ambient temperature may initiate cracking. Where stress mitigation measures are implemented, the best construction schedule consists of regular placement on each block, at the shortest time interval, with the least practical height differential between adjacent blocks—again, if a means of internal cooling is provided.

For thermally controlled concrete, where internal concrete temperatures can be high shortly after placement, care should be taken to avoid creating a large temperature difference in the existing placement by the placement of adjoining concrete or in the new placement by transferring heat too rapidly into the existing placement.

High-lift mass concrete construction was adopted by some authorities, particularly in Canada during the 1950s and 1960s in an attempt to reduce potential leakage paths and minimize cracking in dams built in cold, and even subzero, weather. The procedure is no longer in common use because of cracking concerns. In its extreme form, the method provides for continuous placing of lifts up to 50 ft (15 m) high using wood or insulated forms with housings and steam heat. Under these placing conditions, the adiabatic temperature rise (ATR) of the concrete and the maximum temperature drop to low, stable temperatures are approximately equal. For control of cracking due to thermal shock, most design criteria restrict this maximum drop to 25 to 35°F (14 to 20°C), which should be adhered to unless placement specific analysis shows that higher temperature drops are acceptable. This requires concrete with a low ATR and a low concrete temperature at placement. Design requirements can be met under these conditions by controlling, through mixture proportioning, the adiabatic rise to these levels (Klein et al. 1963). With precooled 50°F (10°C) mass concrete of low cement content in a warm climate, ambient heat removes the advantage of shallower lifts and is the reason 7.5 ft (2.3 m) or even 10 ft (3 m) lifts have been permitted by specifications on several dam projects in recent years.

5.7—Cooling and temperature control

Currently, it is common practice to precool mass concrete before placement. The concrete temperature at placement is typically determined via a thermal study. Efficient equipment is now available to produce concrete at temperatures less than 45°F (7°C) in practically any weather. Using finely chipped ice to replace part of the mixing water and shading damp (but not wet) aggregate will reduce the concrete placing temperature to a value approaching 50°F (10°C) in moderately warm weather. For efficient distribution of admixtures during initial mixing, ice should not replace 100% of the mixing water; for practical purposes, typical ice replacements can be maximized to approximately 75% of the mixing water available for batching after aggregate moisture corrections are performed.

Aggregate storage bins and aggregate piles should be shaded as illustrated in Fig. 5.7a. Aggregates can be cooled by evaporation through vacuum, inundation in cold water, cold air circulation (ACI 207.4R; ACI 305R), or liquid nitrogen. Liquid nitrogen is injected directly into the central mixer or truck mixer (Gajda and Sumodjo 2012). Figure 5.7b shows the cooling of coarse aggregate by spraying and inundation with chilled water immediately before placing in the batch plant bins. Figure 5.7c shows the cooling of batched concrete by liquid nitrogen into the drum of a concrete truck. Caution should be used with chilling concrete with liquid nitrogen as, if executed incorrectly, damage to concrete truck drums can result.

Cooling of concrete to the lowest practical temperature reduces the maximum (peak) temperature of the concrete but not the temperature rise of the concrete, which is a function of the types and quantities of cementitious materials used in the concrete. Surface insulation is recommended to mitigate heat loss from exterior surfaces, thereby inhibiting the development of large temperature differences in the mass concrete placement, especially in colder ambient conditions for traditional mass concrete, and virtually all temperature conditions for thermally controlled concrete. During placement in warm weather, absorption of heat by cold concrete can be minimized by placing at night; managing placement so that minimum areas are exposed; and, if placement will be in the sun, by fog-spraying the work area to mitigate drying and convective heat absorption.

Cooling pipes embedded in the concrete placement can be used to control the temperature rise in the concrete when maximum temperatures cannot be limited by other cooling measures. Embedded cooling pipes can be required to ensure the minimum opening of contraction joints needed when grouting joints in dams is necessary. Additional information

Fig. 5.7a—Metal cover over drained fine aggregate stockpile to reduce heat absorption.

Fig. 5.7b—Cooling coarse aggregate by chilled water spray and inundation.

Fig. 5.7c—Cooling batched concrete by liquid nitrogen injection.

on aggregate and concrete precooling, insulation, protection, and post-cooling considerations are provided in ACI 207.4R.

5.8—Instrumentation

The specific goals of data collection, transmittal, processing, review, and action procedures are to provide accurate and timely evaluation of data for potential remedial action relating to the safety of a structure. Enough instrumentation should be installed to provide confirmation of all important data. It is often desirable to use more than one type of instrument to facilitate the analysis. Instrumentation is also required in cases where it is necessary to correlate with or confirm an unusual design concept related to either the structure or the service condition, or where the instrumentation results may lead to greater refinements for future design.

Instrumentation should be part of the design and construction of any mass concrete structure wherever a future question may arise concerning the safety of the structure. Also, preparations essential for an accurate evaluation of the instrumentation results should be made through long-term, laboratory-sample studies to determine progressive age relationships for properties of the actual project concrete (refer to Chapter 4).

Factors or quantities that are often monitored in mass concrete dams and other massive structures include structural displacements, deformations, settlement, seepage, and uplift pressures in the foundation and the structure. A wide variety of instruments can be used in a comprehensive monitoring program. Instruments installed in dams in the United States have been primarily of the unbonded resistance-wire or Carlson-type meter and vibrating wire. Several manuals on instrumentation of concrete dams are available (USACE EM 1110-2-4300 1985; USSD 2002; U.S. Bureau of Reclamation 1987).

5.8.1 *Temperature measuring devices*—To properly monitor ambient and in-place temperature of a mass concrete placement, it is often necessary to collect instrumentation data over extended periods. The monitoring equipment should be simple, rugged, and durable, and be maintained in satisfactory operating condition. Thermal data loggers are installed prior to placing concrete, typically at the center of the placement to measure maximum concrete temperatures and at approximately 1 to 3 in. (25 to 75 mm) from the exterior surfaces, secured to the reinforcing bar below the concrete surface, to measure concrete temperatures near the surface for the determination of the temperature difference at locations representative of the concrete mass behavior. Temperature sensors should be electrically and thermally isolated from reinforcing bar and other internal components. Concrete temperatures are monitored and downloaded on a daily (or more frequent) basis. Monitoring of thermal data allows adjusting thermal control measures during the protection period.

5.9—Grouting contraction joints in dams

Large arch dams were historically constructed in a series of distinct blocks with means to post-cool the concrete to below the average air temperature. This opened the vertical joints between the blocks, which were subsequently filled with grout to secure the dam into the abutting rock surface. Grouting also distributes in-service stresses more evenly throughout the structure. Prior to grouting, the concrete should be cooled by post-cooling to allow the joint to open to at least 0.04 in. (1 mm) to ensure complete filling with grout. Warner (2004) describes the grouting systems and grouting operations for grouting contraction joints. Silveira et al. (1982) describe a grouting system that employs packers to permit reuse of the piping system. The use of embedded instrumentation across the joint is the only way to accurately determine the magnitude of the joint opening (Carlson 1979; Silveira et al. 1982).

In some gravity dams, the vertical contraction joints were not grouted. In these dams, as a cost-saving measure, it was decided that an upstream waterstop backed up by a vertical drain would prevent visible leakage, and that grout would therefore be unnecessary because there was no transverse stress. In recent years, however, the appearance of vertical

cracks, parallel to the contraction joints, has prompted reconsideration of this practice.

CHAPTER 6—REFERENCES

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ACI 201.2R-16—Guide to Durable Concrete

ACI 207.2R-07—Report on Thermal and Volume Change Effects on Cracking of Mass Concrete

ACI 207.4R-05(12)—Cooling and Insulating Systems for Mass Concrete

ACI 207.5R-11—Report on Roller-Compacted Mass Concrete

ACI 207.6R-17—Report on the Erosion of Concrete in Hydraulic Structures

ACI 209R-92(08)—Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

ACI 212.3R-10—Report on Chemical Admixtures for Concrete

ACI 221R-96(01)—Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete

ACI 224R-01(08)—Control of Cracking in Concrete Structures

ACI 233R-17—Guide to the Use of Slag Cement in Concrete and Mortar

ACI 232.2R-18—Report on the Use of Fly Ash in Concrete

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ACI 301-16—Specifications for Structural Concrete

ACI 304R-00(09)—Guide for Measuring, Mixing, Transporting, and Placing Concrete

ACI 304.2R-17—Guide to Placing Concrete by Pumping Methods

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ACI 305R-10—Guide to Hot Weather Concreting

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ACI 311.5-04—Guide for Concrete Plant Inspection and Testing of Ready-Mixed Concrete

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ACI 350-06—Code Requirements for Environmental Engineering Concrete Structures

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