Mass Concrete

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Synopsis

Mass concrete is "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking." The design of mass concrete structures is generally based on durability, economy, and thermal action, with strength often being a secondary concern. Since the cement-water reaction is exothermic by nature, the temperature rise within a large concrete mass, where the heat is not dissipated, can be quite high. Significant tensile stresses may develop from the volume change associated with the increase and decrease of temperature within the mass. Measures should be taken where cracking due to thermal behavior may cause loss of structural integrity and monolithic action, or may cause excessive seepage and shortening of the service life of the structure, or may be esthetically objectionable. Many of the principles in mass concrete practice can also be applied to general concrete work whereby certain economic and other benefits may be realized.

This report contains a history of the development of mass concrete practice and discussion of materials and concrete mix proportioning, properties, construction methods and equipment, and thermal behavior. It covers traditionally placed and consolidated mass concrete, and does not cover roller-compacted concrete. Mass concrete practices were largely developed from concrete dam construction, where temperature-related cracking was first identified. Temperature-related cracking has also been experienced in other thick-section concrete structures, including mat foundations, pile caps, bridge piers, thick walls, and tunnel linings.

Keywords: admixtures; aggregate gradation; aggregate size; aggregates; air entrainment; arch dams; batching; bridge piers; cements; compressive strength; concrete construction; concrete dams; cooling; cracking (fracturing); creep; curing; diffusivity; durability; fly ash; formwork (construction); gravity dams; heat generation; heat of hydration; history; instrumentation; mass concrete; mix proportioning; mixing; modulus of elasticity; permeability; placing; Poisson's ratio; pozzolans; shear properties; shrinkage; strains; stresses; temperature control; temperature rise (in concrete); thermal expansion; thermal gradient; thermal properties; vibration; volume change.

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CHAPTER 1—INTRODUCTION AND HISTORICAL DEVELOPMENTS

1.1—Scope

1.1.1—"Mass concrete" is defined in ACI 116R as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking." The design of mass concrete structures is generally based principally on durability, economy, and thermal action, with strength often being a secondary rather than a primary concern. The one characteristic that distinguishes mass concrete from other concrete work is thermal behavior. Since the cement-water reaction is exothermic by nature, the temperature rise within a large concrete mass, where the heat is not quickly dissipated, can be quite high (see 5.1.1). Significant tensile stresses and strains may develop from the volume change associated with the increase and decrease of temperature within the mass. Measures should be taken where cracking due to thermal behavior may cause loss of structural integrity and monolithic action, or may cause excessive seepage and shortening of the service life of the structure, or may be esthetically objectionable. Many of the principles in mass concrete practice can also be applied to general concrete work whereby certain economic and other benefits may be realized.

This report contains a history of the development of mass concrete practice and discussion of materials and concrete mix proportioning, properties, construction methods and equipment, and thermal behavior. This report covers traditionally placed and consolidated mass concrete, and does not cover roller-compacted concrete. Roller-compacted concrete is described in detail in ACI 207.5R.

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

High compressive strengths are usually not required in mass concrete structures; thin arch dams are exceptions. Massive structures, such as gravity dams, resist loads by virtue of their shape and mass, and only secondarily by their strength. 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. Mixture proportioning for mass concrete is discussed in ACI 211.1.

1.2—History

1.2.1—When concrete was first used in dams, the dams were 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 days, and the art and science of dam building practiced today has reached a highly advanced state. The selection and proportioning of concrete materials to produce suitable strength, durability, and impermeability of the finished product can be predicted and controlled with accuracy.

1.2.2—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 the hottest weather. Grand Coulee Dam still holds the all-time record monthly placing rate of 536,250 yd^3 (410,020 m^3) followed by the more recent achievement at Itaipu Dam on the Brazil-Paraguay border of 440,550 yd^3 (336,840 m^3) (Itaipu Binacional 1981). Lean mixes are now made workable by means of air-entraining and other chemical admixtures and the use of finely divided pozzolanic materials. Water-reducing, strength-enhancing, and set-controlling chemical admixtures are effective in reducing the required cement content to a minimum as well as in controlling the time of setting. With the increased attention to roller-compacted concrete, a new dimension has been given to mass concrete construction. The record monthly placing rate of 328,500 yd^3 (250,200 m^3) for roller-compacted concrete was achieved at Tarbela Dam in Pakistan. Placing rates for no-slump concrete, using large earth-moving equipment for transportation and large vibrating rollers for consolidation, appear to be limited only by the size of the project and its plant's ability to produce concrete. Those concerned with concrete dam construction should not feel that the ultimate has been reached, but they are justified in feeling some satisfaction with the progress that has been made.

1.2.3 *Prior to 1900*—Prior to 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 benefit of washing to remove objectionable dirt and fines. Concrete mixes varied widely in cement content and in sand/coarse aggregate ratio. Mixing was usually by hand and proportioning by shovel, wheelbarrow, box, or cart. The effect of water-cement ratio 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, also very wet. Some of the early dams were built without contraction joints and without regular lifts. However, there were 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 a day. Generally, there was no attempt to moist cure.

An exception to these general practices was the Lower Crystal Springs Dam completed in 1890. This dam is located near San Mateo, California, about 20 miles 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. Fresh concrete was covered with planks as a protection from the sun and the concrete was kept wet until hardening occurred.

Only a few of the concrete dams built in the United States prior to 1900 remain serviceable today, and most of them are small. Of the nearly 3500 dams built in the United States to date, fewer than 20 were built prior to 1900. More than a third of these are located in the states of California and Arizona where the climate is mild. The others survive more rigorous climates thanks to their stone masonry facing.

1.2.4 *Years 1900 to 1930*—After the turn of the century, the construction of all types of concrete dams was greatly accelerated. More and higher dams for irrigation, power, and water supply were the order of the day. Concrete placement by means of towers and chutes became the vogue. 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 this 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 was 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 Duff Abrams and his associates from investigations prior to 1918 when Portland Cement Association (PCA) Bulletin 1 appeared. 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 mixes for at least 12 years after the importance of the water-cement ratio had been established.

Generally, portland cements were employed without admixtures. There were exceptions such as the sand-cements employed by the U.S. Reclamation Service, now the U.S. Bureau of Reclamation, in the construction of Elephant Butte and Arrowrock dams. 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

*. See 6.2 for references.

concrete. The mixture for interior concrete contained approximately 376 lb of a blended, pulverized granite-cement combination per yd^3 (223 kg/m^3). The cement mixture was produced at the site by intergrinding about equal parts of portland cement and pulverized granite such that not less than 90 percent passed the 200 (75 μm) 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, California, was employed as a 20 percent replacement by weight for portland cement.

During the 1900-1930 period, cyclopean concrete went out of style. For dams of thick section, the maximum size of aggregate for mass concrete was increased to as large as 10 in. (250 mm). As a means of measuring consistency, the slump test had come into use. 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. Mixers of 3-yd^3 (2.3-m^3) capacity were in common use near the end of this period and there were some of 4-yd^3 (3-m^3) capacity. Only Type I cement (normal portland cement) was available during this period. In areas where freezing and thawing conditions were severe it was common practice to use a concrete mix containing 564 lb of cement per yd^3 (335 kg/m^3) for the entire concrete mass. The construction practice of using an interior mix containing 376 lb/yd^3 (223 kg/m^3) and an exterior face mix containing 564 lb/yd^3 (335 kg/m^3) was developed during this period 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.

An exception was Theodore Roosevelt Dam built during 1905-1911. It is 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 about 846 lb of cement per yd^3 (502 kg/m^3). In each layer the voids between the closely spaced stones were filled with a concrete containing 564 lb of cement per yd^3 (335 kg/m^3) into which spalls were spaded by hand. These conditions account for the very low average cement content. Construction was laboriously slow, and Roosevelt Dam represents perhaps the last of the large dams built in the United States by this method of construction.

1.2.5 *Years 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, and/or railroad systems. On the larger and more closely controlled construction projects, the aggregates were carefully processed, ingredients were proportioned by weight, and the mixing water measured by volume.

Improvement in workability was brought about by the introduction of finely divided mineral admixtures (pozzolans), 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 employed large spud vibrators for consolidation.

A study of the records and actual inspection of a considerable number of dams show that there were differences in condition which could not be explained. Of two structures that appeared to be of like quality subjected to the same environment, one might exhibit excessive cracking while the other, after a like period of service, would be in near-perfect condition. The meager records available on a few dams indicated wide internal temperature variations due to cement hydration. The degree of cracking was associated with the temperature rise.

ACI Committee 207, Mass Concrete, was organized in 1930 (originally as Committee 108) for the purpose of gathering information about the significant properties of mass concrete in dams and factors which influence these properties. Bogue (1949) and his associates under the PCA fellowship at the National Bureau of Standards had already identified the principal compounds in portland cement. Later, 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.

By the beginning of 1930, Hoover Dam was in the early stages of planning. Because of the unprecedented size of Hoover Dam, investigations much more elaborate than any that had been previously undertaken were carried out to determine the effect of composition and fineness of cement, cement factor, temperature of curing, maximum size of aggregate, etc., on heat of hydration of cement, compressive strength, and other properties of mortars and concrete.

The results of these investigations led to the use of low-heat cement in Hoover Dam. The investigations also furnished information for the design of the embedded pipe cooling system employed for the first time in Hoover Dam. Low-heat cement was first used in Morris Dam, near Pasadena, California, which was started a year before Hoover Dam.

For 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 about two 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 which have been constructed in the United States and in many other countries all over the world since that time.

The use of a pozzolanic material (pumicite) was given a trial in Big Dalton Dam by the Los Angeles County Flood Control District. For Bonneville Dam, completed by the Corps of Engineers in 1938, a portland cement-pozzolan combination was employed for all of the work. It was produced by intergrinding the cement clinker with a pozzolan processed by calcining an altered volcanic material at a temperature of about 1500 F (820 C). The proportion of clinker to pozzolan was 3:1 by weight. This type of cement was selected for use at Bonneville on the basis of results of tests on concrete which indicated large extensibility and low temperature rise. This is the only known completed concrete dam in the United States in which an interground portland-pozzolan cement has been employed. The use of pozzolan as a separate cementing material to be added at the mixer, at a rate of 30 percent, or more, of total cementitious materials, has come to be regular practice by the Bureau of Reclamation, the Tennessee Valley Authority, the Corps of Engineers, and others.

The group of 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 as materials that could benefit mass concrete. In 1960, Wallace and Ore published their report on the benefit of these materials to lean mass concrete. Since this time, chemical admixtures have come to be used in most mass concrete.

It became standard practice about 1945 to use purposely entrained air for concrete in most structures that are exposed to severe weathering conditions. This practice was applied to the concrete of exposed surfaces of dams as well as concrete pavements and reinforced concrete in general. Air-entraining admixtures introduced at the mixer have been employed for both interior and exterior concretes of practically all dams constructed since 1945.

Placement of conventional mass concrete has remained largely unchanged since that time. The major new development in the field of mass concrete is the use of roller-compacted concrete.

1.2.6 *1970 to present: roller-compacted concrete*—During this era, roller-compacted concrete was developed and became the predominant method for placing mass concrete. Because roller-compacted concrete is now so commonly used, a separate report, ACI 207.5R, is the principal reference for this subject. Traditional mass concrete 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 wall, and some foundation works, particularly where reinforcement is required.

1.2.7 *Cement content*—During the late 1920s and the early 1930s, it was practically an unwritten law that no mass concrete for large dams should contain less than 376 lb of cement per yd^3 (223 kg/m^3). Some of the authorities of that period were of the opinion that the cement factor should never be less than 564 lb/yd^3 (335 kg/m^3). The cement factor for the interior concrete of Norris Dam (Tennessee Valley Authority 1939) constructed by the Tennessee Valley Authority in 1936, was 376 lb/yd^3 (223 kg/m^3). The degree of cracking was objectionably great. The compressive strength of the wet-screened 6 x 12-in. (150 x 300-mm) job cylinders at one-year age was 7000 psi (48.3 MPa). Core specimens 18 x 36-in. (460 x 910-mm) drilled from the first stage concrete containing 376 lb of cement per yd^3 (223 kg/m^3) at Grand Coulee Dam tested in the excess of 8000 psi (55 MPa) at the age of two years. Judged by composition, the cement was of 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 quite unnecessary. A reduction in cement content on similar future constructions might be expected to substantially reduce the tendency toward cracking.

For Hiwassee Dam, completed by TVA in 1940, the 376 lb/yd^3 (223 kg/m^3) cement-content barrier was broken. For that structure the cement content of the mass concrete was only 282 lb/yd^3 (167 kg/m^3), an unusually low value for that time. Hiwassee Dam was singularly free from thermal cracks, and there began a trend toward reducing the cement content which is still continuing. Since this time, the Type II cement content of the interior mass concrete has been on the order of 235 lb/yd^3 (140 kg/m^3) and even as low as 212 lb/yd^3 (126 kg/m^3). An example of a large gravity dam for which the Type II cement content for mass concrete was 235 lb/yd^3 (140 kg/m^3) is Pine Flat Dam in California, completed by the Corps of Engineers in 1954. In high dams of the arch type where stresses are moderately high, the cement content of the mass mix is usually in the range of 300 to 450 lb/yd^3 (180 to 270 kg/m^3), 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:

Arch dams

1. 282 lb/yd^3 (167 kg/m^3) of cement and pozzolan in Glen Canyon Dam, a relatively thick arch dam in Arizona, completed in 1963.
2. 373 lb/yd^3 (221 kg/m^3) of cement in Morrow Point Dam in Colorado, completed in 1968.
3. 420 lb/yd^3 (249 kg/m^3) of cement in El Atazar Dam near Madrid, Spain, completed in 1972.
4. 303 to 253 lb/yd^3 (180 to 150 kg/m^3) of portland-pozzolan Type IP cement in El Cajon Dam on the Humuya River in Honduras, completed in 1984.

Straight gravity dams

1. 226 lb/yd^3 (134 kg/m^3) of Type II cement in Detroit Dam in Oregon, completed in 1952.
2. 194 lb/yd^3 (115 kg/m^3) of Type II cement and fly ash in Libby Dam in Montana, completed in 1972.
3. 184 lb/yd^3 (109 kg/m^3) of Type II cement and calcined clay in Ilha Solteira Dam in Brazil, completed in 1973.

1.3—Temperature control

1.3.1—To achieve a lower maximum temperature of interior mass concrete during the hydration period, the practice of precooling concrete materials prior to mixing was started in the early 1940s and has been extensively employed in the construction of large dams beginning in the late 1940s.

1.3.2—The first serious effort to precool appears to have occurred during the construction of Norfork Dam in 1941-1945 by the Corps of Engineers. 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 about 10 F (5.6 C). On later works not only has crushed ice been used in the mixing water, but coarse aggregates have been precooled either by cold air or cold water prior to batching. Recently, both fine and coarse aggregates in a moist condition have been precooled by various means including vacuum saturation and liquid nitrogen injection. It has become almost standard practice in the United States to employ precooling for large dams in regions where the summer temperatures are high, to assure that the temperature of concrete as it is placed in the work does not exceed about 50 F (10 C).

1.3.3—On some large dams, including Hoover (Boulder) Dam, a combination of precooling and postcooling refrigeration by embedded pipe has been used (U.S. Bureau of Reclamation 1949). A good example of this practice is Glen Canyon Dam, where at times during the summer months the ambient temperatures were considerably greater than 100 F (38 C). 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 temperature. By means of embedded-pipe refrigeration, the maximum temperature of hardening concrete was kept below 75 F (24 C). Postcooling is sometimes required in gravity and in arch dams that contain transverse joints, so that transverse joints can be opened for grouting by cooling the concrete after it has hardened. Postcooling is also done for control of peak temperatures, to control cracking.

1.4—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. Maximum stresses under load do not usually develop until the concrete is at least one year old. Under mass curing conditions, with the cement and pozzolans customarily employed, the gain in concrete strength between 28 days and one year is generally large. The gain can range from 30 percent to more than 200 percent, depending on the quantities and proportioning of cementitious materials and properties of the aggregates. It has become the practice of some designers of dams to specify the desired strength of mass concrete at later ages such as one or two years. For routine quality control in the field, 6 x 12-in. (150 x 300-mm) cylinders are normally used with aggregate larger than $1^1/_2$ in. (37.5 mm) removed by wet screening. Strength requirements of the wet-screened concrete are correlated with the specified full-mix strength by laboratory tests.

CHAPTER 2—MATERIALS AND MIX PROPORTIONING

2.1—General

2.1.1—As is the case with other concrete, mass concrete is composed of cement, aggregates, and water, and frequently pozzolans and admixtures. The objective of mass concrete mix proportioning is the selection of combinations of materials that will produce concrete to meet the requirements of the structure with respect to economy, workability, dimensional stability and freedom from cracking, low temperature rise, adequate strength, durability, and—in the case of hydraulic structures—low permeability. This chapter will describe materials that have been successfully used in mass concrete construction and factors influencing their selection and proportioning. The recommendations contained herein may need to be adjusted for special uses, such as for massive precast beam segments, for tremie placements, and for roller-compacted concrete. Guidance in proportioning mass concrete can also be found in ACI 211.1, particularly Appendix 5 which details specific modifications in the procedure for mass concrete proportioning.

2.2—Cements

2.2.1—ACI 207.2R and ACI 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 C 150.

(b) Blended cement: Types P, IP, S, IS, I(PM), and I(SM) as covered by ASTM C 595.

When portland cement is used with pozzolan or with other cements, the materials are batched separately at the mixing plant. Economy and low temperature rise are both achieved by limiting the total cement content to as small an amount as possible.

2.2.2—Type I portland cement is commonly used in general construction. It is not recommended for use by itself in mass concrete without other measures that help to control temperature problems because of its substantially higher heat of hydration.

2.2.3—Type II portland cement is suitable for mass concrete construction because it has a moderate heat of hydration important to the control of cracking. Specifications for Type II portland cement require that it contain no more than 8 percent tricalcium aluminate (C_3A), the compound that contributes substantially to early heat development in the concrete. Optional specifications for Type II cement place a limit of 58 percent or less on the sum of tricalcium aluminate and tricalcium silicate, 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 C 150 is reduced due to the slower rate of strength gain of this cement.

2.2.4—Type IV portland cement, also referred to as “low heat” cement, may be used where it is desired to produce low heat development in massive structures. It has not been used in recent years because it has been difficult to obtain and, more importantly, because experience has shown that in most cases heat development can be controlled satisfactorily by other means. Type IV specifications limit the C_3A to 7 percent, the C_3S to 35 percent, and place a minimum on the C_2S of 40 percent. 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 V sulfate-resistant portland cement (Canadian Type 50) is available both in the United States and in Canada usually at a price premium over Type I. It is usually both low alkali and low heat.

2.2.5—Type IP portland-pozzolan cement is a uniform blend of portland cement or portland blast-furnace slag cement and fine pozzolan. Type P is similar but early strength requirements are lower. They are produced either by intergrinding portland cement clinker and pozzolan or by blending portland cement or portland blast-furnace slag cement and finely divided pozzolan. The pozzolan constituents are between 15 and 40 percent by weight of the portland-pozzolan cement, with Type P having the generally higher pozzolan content.

Type I(PM) pozzolan-modified portland cement contains less than 15 percent pozzolan and its properties are close to those of Type I cement. A heat of hydration limit of 70 cal/g (290kJ/kg) at 7 days is an optional requirement for Type IP and Type I(PM) by adding the suffix (MH). A limit of 60 cal/g (250 kJ/kg) at 7 days is optional for Type P by adding the suffix (LH).

2.2.6—Type IS portland blast-furnace slag cement is a uniform blend of portland cement and fine blast-furnace slag. It is produced either by intergrinding portland cement clinker and granulated blast-furnace slag or by blending portland cement and finely ground granulated blast-furnace slag. The amount of slag used may vary between 25 and 70 percent by weight of the portland blast-furnace slag cement. This cement has sometimes been used with a pozzolan. Type S slag cement is finely divided material consisting essentially of a uniform blend of granulated blast-furnace slag and hydrated lime in which the slag constituent is at least 70 percent of the weight of the slag cement. Slag cement is generally used in a blend with portland cement for making concrete.

Type I(SM) slag-modified portland cement contains less than 25 percent slag and its properties are close to those of Type I cement. Optional heat of hydration requirements can be applied to Type IS, and I(SM), similar to those applied to Type IP, I(PM), and P.

2.2.7—Low-alkali cements are defined by ASTM C 150 as portland cements containing not more than 0.60 percent alkalies calculated as the percentage of Na_2O plus 0.658 times the percentage of K_2O. These cements should be specified when the cement is to be used in concrete with aggregate that may be deleteriously reactive. The use of low-alkali cement may not always control highly reactive noncrystalline siliceous aggregate. It may also be advisable to use a proven pozzolan to insure control of the alkali-aggregate reaction.

2.3—Pozzolans and ground slag

2.3.1—A pozzolan is generally defined as a siliceous or siliceous-and-aluminous material which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Pozzolans are ordinarily governed and classified by ASTM C 618, as natural (Class N), or fly ash (Classes F or C). There are some pozzolans, such as the Class C fly ash, which contain significant amounts of compounds like those of portland cement. The Class C fly ashes likewise have cementitious properties by themselves which may contribute significantly to the strength of concrete.

Pozzolans react chemically with the calcium hydroxide or hydrated lime liberated during the hydration of portland cement to form a stable strength-producing cementitious compound. For best activity the siliceous ingredient of a pozzolan must be in an amorphous state such as glass or opal. Crystalline siliceous materials, such as quartz, do not combine readily with lime at normal temperature unless they are ground into a very fine powder. The use of fly ash in concrete is discussed in ACI 226.3R, and the use of ground granulated blast-furnace slag is discussed in ACI 226.1R.

2.3.2—Natural pozzolanic materials occur in large deposits throughout the western United States in the form of obsidian, pumicite, volcanic ashes, tuffs, clays, shales, and diatomaceous earth. These natural pozzolans usually require grinding. Some of the volcanic materials are of suitable fineness in their natural state. The clays and shales, in addition to grinding, must be activated to form an amorphous state by calcining at temperatures in the range of 1200 to 1800 F (650 to 980 C).

2.3.3—Fly ash is the flue dust from burning ground or powdered coal. Suitable fly ash can be an excellent pozzolan if it has a low carbon content, a fineness about the same as that of portland cement, and occurs in the form of very fine, glassy spheres. Because of its shape and texture, the water requirement is usually reduced when fly ash is used in concrete. There are indications that in many cases the pozzolanic activity of the fly ash can be increased by cracking the glass spheres by means of grinding. However, this may reduce its lubricating qualities and increase the water requirement of the concrete. It is to be noted that high-silica Class F fly ashes are generally excellent pozzolans. However, some Class C fly ashes may contain such a high CaO content that, while possessing good cementitious properties, they may be unsuitable for controlling alkali-aggregate reaction or for improving sulfate resistance of concrete. Additionally, the Class C fly ash will be less helpful in lowering heat generation in the concrete.

2.3.4—Pozzolans in mass concrete may be used to reduce portland cement factors for better economy, to lower internal heat generation, to improve workability, and to lessen the potential for damage from alkali-aggregate reactivity and sulfate attack. It should be recognized, however, that properties of different pozzolans may vary widely. Some pozzolans may introduce problems into the concrete, such as increased drying shrinkage as well as reduced durability and low early strength. Before a pozzolan is used it should be tested in combination with the project cement and aggregates to establish that the pozzolan will beneficially contribute to the quality and economy of the concrete. Compared to portland cement, the strength development from pozzolanic action is slow at early ages but continues at a higher level for a longer time. Early strength of a portland cement-pozzolan concrete would be expected to be lower than that of a portland cement concrete designed for equivalent strength at later ages. Where some portion of mass concrete is required to attain strength at an earlier age than is attainable with the regular mass concrete mixture, the increased internal heat generated by a substitute earlier-strength concrete may be accommodated by other means. Where a pozzolan is being used, it may be necessary temporarily to forego the use of the pozzolan and otherwise accommodate the increased internal heat generated by the use of straight portland cement. However, if there is a dangerous potential from alkali-aggregate reaction, the pozzolan should be used, while expedited strength increase is achieved by additional cement content.

Pozzolans, particularly natural types, have been found effective in reducing the expansion of concrete containing reactive aggregates. The amount of this reduction varies with the chemical makeup and fineness of the pozzolan and the amount employed. For some pozzolans, the reduction in expansion may exceed 90 percent. Pozzolans reduce expansion by consuming alkalies from the cement before they can enter into deleterious reactions with the aggregates. Where alkali-reactive aggregates are used, it is considered good practice to use both a low-alkali cement and a pozzolan of proven corrective ability. Alkali-aggregate reactions are discussed in ACI 221R.

Some experiments conducted by the Corps of Engineers (Mather 1974) indicate that for interior mass concrete, where stresses are moderately low, a much higher proportion of pozzolan to cement may be used when there is an economic advantage in doing so and the desired strength is obtained at later ages. For example, the results of laboratory tests indicate that an air-entrained mass concrete, containing 94 lb/yd^3 (53 kg/m^3) of cement plus fly ash in an amount equivalent in volume to 188 lb (112 kg) of cement has produced a very workable mixture, for which the water content was less than 100 lb/yd^3 (60 kg/m^3). The one-year compressive strength of wet-screened 6 x 12-in. (150 x 300-mm) cylinders of this concrete was on the order of 3000 psi (21 MPa). For such a mixture the mass temperature rise would be exceedingly small. For gravity dams of moderate height, where the material would be precooled such that the concrete as it reaches the forms will be about 15 F (8 C) below the mean annual or rock temperature, there is the possibility that neither longitudinal nor transverse contraction joints would be required. The maximum temperature of the interior of the mass due to cement hydration might not be appreciably greater than the mean annual temperature.

The particle shapes of concrete aggregates and their effect on workability has become less important because of the improved workability that is obtainable through the use of pozzolans, and air-entraining and other chemical admixtures. The development of new types of pozzolans, such as rice hull ash and silica fume, may find a promising place in future mass concrete work.

2.3.5—Finely ground granulated iron blast-furnace slag may also be used as a separate ingredient with portland cement as cementitious material in mass concrete. Requirements on finely ground slag for use in concrete are specified in ASTM C 989. If used with Type I portland cement, proportions of at least 70 percent finely ground slag of total cementitious material may be needed with an active slag to produce a cement-slag combination which will have a heat of hydration of less than 60 cal/g (250 kJ/kg) at 7 days. The addition of slag will usually reduce the rate of heat generation due to a slightly slower rate of hydration. Finely ground slag also produces many of the beneficial properties in concrete that are achieved with suitable pozzolans, such as reduced permeability, control of expansion from reactive aggregate, sulfate resistance, and improved workability. However, finely ground slag is usually used in much higher percentages than pozzolan to achieve similar properties.

2.4—Chemical admixtures

2.4.1—A full coverage of admixtures is contained in ACI 212.3R. The chemical admixtures that are important to mass concrete are classified as follows: (1) air-entraining; (2) water-reducing; and (3) set-controlling.

2.4.2—Accelerating admixtures are not used in mass concrete because high early strength is not necessary in such work and because accelerators contribute to undesirable heat development in the concrete mass.

2.4.3—Chemical admixtures can provide important benefits to mass concrete in its plastic state by increasing workability and/or reducing water content, retarding initial setting, modifying the rate of and/or capacity for bleeding, reducing segregation, and reducing rate of slump loss.

2.4.4—Chemical admixtures can provide important benefits to mass concrete in its hardened state by lowering heat evolution during hardening, increasing strength, lowering cement content, increasing durability, decreasing permeability, and improving abrasion/erosion resistance.

2.4.5—Air-entraining admixtures are materials which produce minute air bubbles in concrete during mixing—with resultant improved workability, reduced segregation, lessened bleeding, lowered permeability, and increased resistance to damage from freezing and thawing cycles. The entrainment of air greatly improves the workability of lean concrete and permits the use of harsher and more poorly graded aggregates and those of undesirable shapes. It facilitates the placing and handling of mass concrete. Each one percent of entrained air permits a reduction in mixing water of from 2 to 4 percent, with some improvement in workability and with no loss in slump. Durability, as measured by the resistance of concrete to deterioration from freezing and thawing, is greatly improved if the spacing of the air bubble system is such that no point in the cement matrix is more than 0.008 in. (0.20 mm) from an air bubble.

2.4.6—Entrained air generally will reduce the strength of most concretes. Where the cement content is held constant and advantage is taken of the reduced water requirement, air entrainment in lean mass concrete has a negligible effect on strength and may slightly increase it. Among the factors that influence the amount of air entrained in concrete for a given amount of agent are: grading and particle shape of the aggregate, richness of the mix, 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 up to 6 in. (150 mm) and decreases with increases in amount of fines, temperature of concrete, and mixing time. If fly ash is used that contains activated carbon, an increased dosage of air-entraining admixture will be required. Most specifications for mass concrete now require that the quantity of entrained air, as determined from concrete samples wet sieved through the $1^1/_2$-in. (37.5-mm) sieve, be about 5 percent, although in some cases as high as 8 percent. Requirements for air-entraining admixtures are contained in ASTM C 260.

2.4.7—Water-reducing and set-controlling admixtures generally consist of one or more of these compounds: (1) lignosulfonic acid; (2) hydroxylated carboxylic acid; (3) polymeric carbohydrates; or (4) naphthalene or melamine types of high-range water reducers.

Set-controlling admixtures can be used to keep the concrete plastic longer in massive blocks so that successive layers can be placed and vibrated before the underlayer sets. Water-reducing admixtures are used to reduce the mixing water requirement, to increase the strength of the concrete or to produce the same strength with less cement. Admixtures from the first three families of materials above generally will reduce the water requirement up to about 10 percent, will retard initial set at least 1 hr (but not reduce slump loss), and will increase the strength an appreciable amount. When a retarder is used, the strength after 12 hr is generally comparable to that of concrete containing no admixture. Depending upon the richness of the concrete, composition of cement, temperature and other factors, use of chemical admixtures will usually result in significant increases in 1-, 7-, 28-day, and later strengths. This gain in strength cannot be explained by the amount of the water reduction or by the degree of change in the water-cement ratio; the chemicals have a favorable effect on the hydration of the cement. Admixtures of the carboxylic acid family augment bleeding. The high-range water-reducing family of admixtures does not have a well-established record in mass concrete construction, although these admixtures were used in some mass concrete in Guri Dam in Venezuela, and have been used in reinforced mass concrete foundations. However, in view of their strong plasticizing capability, they may hold a promising role in adding workability to special mass concreting applications where workability is needed. Requirements for chemical admixtures are contained in ASTM C 494.

2.5—Aggregates

2.5.1—Coarse and fine aggregate as well as terms relating to aggregates are defined in ASTM C 125. Additional information on aggregates is contained in ACI 221R.

2.5.2—Fine aggregate is that fraction "almost entirely" passing the No. 4 (4.75 mm) sieve. It may be composed of natural grains, manufactured grains 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 employing normal proportions of the ingredients. Deleterious substances are usually limited to the percentages by weight given in Table 2.5.2. For bridge piers, dams, and other hydraulic structures, the maximum allowable percentage of the deleterious substance should be 50 percent lower for face concrete in the zone of fluctuating water levels. It can be 50 percent higher for concrete constantly immersed in water and for concrete in the interior of massive dams.

Table 2.5.2— Maximum allowable percentages of deleterious substances in fine aggregate (by weight)

Clay lumps and friable particles	3.0
Material finer than No. 200 (75-µm sieve:	
For concrete subject to abrasion	3.0*
For all other concrete	5.0*
Coal and lignite:	
Where surface appearance of concrete is of importance	0.5
All other concrete	1.0

*In the case of manufactured sand, if the material passing the No. 200 (75-µm) sieve consists of the dust of fracture, essentially free of clay or shale, these limits may be increased to 5 percent for concrete subject to abrasion and 7 percent for all other concrete.

2.5.3—The grading of fine aggregate strongly influences the workability of concrete. A good grading of sand for mass concrete will be within the limits shown in Table 2.5.3. Laboratory investigation may show other gradings to be satisfactory. This permits a rather wide latitude in gradings for fine aggregate.

Although the grading requirements themselves may be rather flexible, it is important that once the proportion is established, the grading of the sand be maintained reasonably constant to avoid variations in the workability of the concrete.

Table 2.5.3— Fine aggregate for mass concrete*

Sieve designation	Percentage retained, individual by weight
$^3/_8$ in. (9.5 mm)	0
No. 4 (4.75 mm)	0-5
No. 8 (2.36 mm)	5-15
No. 16 (1.18 mm)	10-25
No. 30 (600 µm)	10-30
No. 50 (300 µm)	15-35
No. 100 (150 µm)	12-20
Pan fraction	3-7

*U.S. Bureau of Reclamation 1981

2.5.4—Coarse aggregate is defined as gravel, crushed gravel, or crushed rock, or a mixture of these nominally larger than the No. 4 (4.75 mm) and smaller than the 6 in. (150 mm) sizes for large structures. Massive structural concrete structures, such as powerhouses or other heavily-reinforced units that are considered to be in the mass concrete category, have successfully used smaller-sized coarse aggregates, usually of 3 in. (75 mm) maximum size but with some as small as $1^1/_2$ in. (37.5 mm). The use of smaller aggregate may be dictated by the close spacing of reinforcement or embedded items, or by the unavailability of larger aggregates. This results in higher cement contents with attendant adverse effects on internal heat generation and cracking potential that must be offset by greater effort to reduce the cement requirement and concrete placing temperatures. The maximum size of coarse aggregate should not exceed one-fourth of the least dimension of the structure nor two-thirds of the least clear distance between reinforcing bars in horizontal mats or where there is more than one vertical reinforcing curtain next to a form. Otherwise, the rule for mass concrete should be to use the largest size of coarse aggregate that is practical.

2.5.5—Coarse aggregate should consist of hard, dense, durable, uncoated particles. Rock which is very friable or which tends to degrade during processing, transporting, or in storage should be avoided. Rock having an absorption greater than 3 percent or a specific gravity less than 2.5 is not generally considered suitable for exposed mass concrete subjected to freezing and thawing. Sulfates and sulfides, determined by chemical analysis and calculated as SO_3, should not exceed 0.5 percent of the weight of the coarse aggregate. The percentage of other deleterious substances such as clay, silt, and fine dust in the coarse aggregate as delivered to the mixer should in general not exceed the values outlined in Table 2.5.5.

Fig. 2.5.5 shows a coarse aggregate rewashing screen at the batch plant where dust and coatings accumulating from stockpiling and handling can be removed to assure aggregate cleanliness.

Table 2.5.5— Maximum allowable percentages of deleterious substances in coarse aggregate (by weight)

Material passing No. 200 sieve (75 μm)	0.5
Lightweight material	2.0
Clay lumps	0.5
Other deleterious substances	1.0

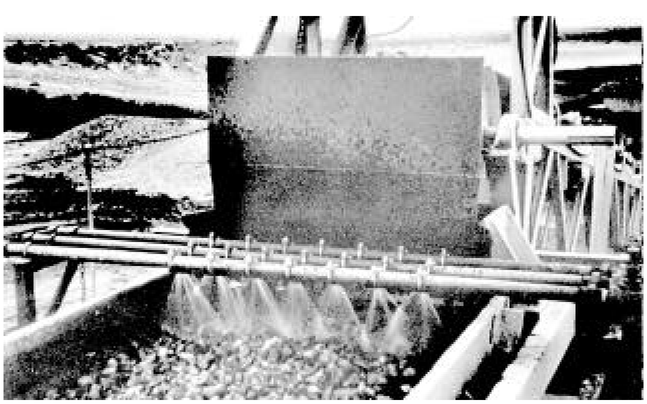


Fig. 2.5.5—Coarse aggregate rewashing

2.5.6—Theoretically, the larger the maximum aggregate size, the less cement is required in a given volume of concrete to achieve the desired quality. This theory is based on the fact that with well-graded materials the void space between the particles (and the specific surface) decreases as the range in sizes increases. However, it has been demonstrated (Fig. 2.5.6) that to achieve the greatest cement efficiency there is an optimum maximum size for each compressive strength level to be obtained with a given aggregate and cement (Higginson, Wallace, and Ore 1963). While the maximum size of coarse aggregate is limited by the configuration of the forms and reinforcing steel, in most unreinforced mass concrete structures these requirements permit an almost unlimited maximum aggregate size. In addition to availability, the economical maximum size is therefore determined by the design strength and problems in processing, batching, mixing, transporting, placing, and consolidating the concrete. Large aggregate particles of irregular shape tend to promote cracking around the larger particles because of differential volume change. They also cause voids to form underneath them due to bleeding water and air accumulating during placing of concrete. Although larger sizes have been used on occasion, an aggregate size of 6 in. (150 mm) has normally been adopted as the maximum practical size.

2.5.7—The particle shape of aggregates has some effect on workability and consequently, on water requirement. Rounded particles, such as those which occur in deposits of stream-worn sand and gravel, provide best workability. However, modern crushing and grinding equipment is capable of producing both fine and coarse aggregate of entirely adequate particle shape from quarried rock. Thus, in spite of the slightly lower water requirement of natural rounded aggregates, it is seldom economical to import natural aggregates when a source of high quality crushed aggregate is available near the site of the work. It is necessary to determine that the crushing equipment and procedures will yield a satisfactory particle shape. One procedure to control particle shape is to specify that the flat and elongated particles cannot exceed 20 percent in each size group. A flat particle is defined as one having a ratio of width to thickness greater than three, while an elongated particle is defined as one having a ratio of length to width greater than three.

2.5.8—The proportioning of aggregates in the concrete mixture will strongly influence concrete workability and this is one factor that can readily be adjusted during construction. To facilitate this, aggregates are processed into and batched from convenient size groups. In United States practice it is customary, for large-aggregate mass concrete, to divide coarse aggregate into the fractional sizes listed in Table 2.5.8 (Tuthill 1980).

Sizes are satisfactorily graded when one-third to one-half of the aggregate within the limiting screens is retained on the middle size screen. Also, it has been found that maintaining the percent passing the $^3/_8$-in. (9.5-mm) sieve at less than 30 percent 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.

2.5.9—Experience has shown that a rather wide range of material percentage in each size group may be used as listed in Table 2.5.9. Workability is frequently improved by reducing the proportion of cobbles called for by the theoretical

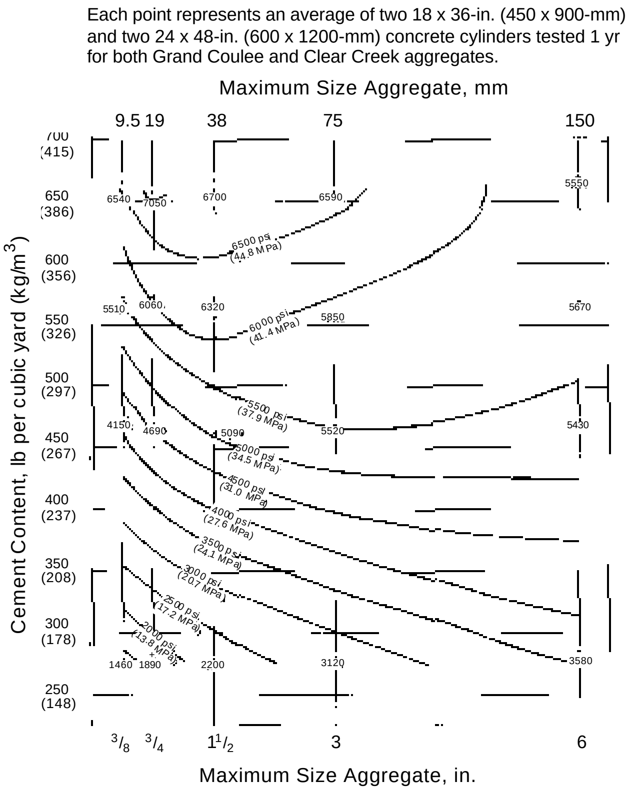


Fig. 2.5.6—Effect of aggregate size and cement content on compressive strength at one year (adapted from Higginson, Wallace, and Ore 1963)

Table 2.5.8— Grading requirements for coarse aggregate

Test sieve size, sq. mesh, in. (mm)	Percent by weight passing designated test sieve			
	Cobbles 6-3 in. (150 - 75 mm)	Coarse $3\text{-}1\frac{1}{2}$ in. (75 - 37.5 mm)	Medium $1\frac{1}{2}$ - $\frac{3}{4}$ in. (37.5 - 19 mm)	Fine $\frac{3}{4}$ - No. 4 in. (19 - 4.75 mm)
7 (175)	100			
6 (150)	90-100			
4 (100)	20-45	100		
3 (75)	0-15	90-100		
2 (50)	0-5	20-55	100	
$1\frac{1}{2}$ (37.5)		0-10	90-100	
1 (25)		0-5	20-45	100
$\frac{3}{4}$ (19)			1-10	90-100
$\frac{3}{8}$ (9.5)			0-5	30-55
No. 4 (4.75)				0-5

Table 2.5.9— Ranges in each size fraction of coarse aggregate that have produced workable concrete*

Maximum size in concrete, in. (mm)	Percentage of cleanly separated coarse aggregate fractions				
	Cobbles 6-3 in. (150-75 mm)	Coarse $3\text{-}1\frac{1}{2}$ in. (75-37.5 mm)	Medium $1\frac{1}{2}\text{-}\frac{3}{4}$ in. (37.5-19 mm)	Fine $\frac{3}{4}\text{-}\frac{3}{8}$ (19-9.5 mm)	Fine $\frac{3}{8}$-No. 4 (9.5-4.75 mm)
6 (150)	20-30	20-32	20-30	12-20	8-15
3 (75)		20-40	20-40	15-25	10-15
$1\frac{1}{2}$ (37.5)			40-55	30-35	15-25
$\frac{3}{4}$ (19)				30-70	20-45

*U.S. Bureau of Reclamation 1981.

gradings. When natural gravel is used, it is economically desirable to depart from theoretical gradings to approximate as closely as workability permits the average grading of material in the deposit. Where there are extreme excesses or deficiencies in a particular size, it is preferable to waste a portion of the material rather than to produce unworkable concrete. The problem of waste usually does not occur when the aggregate is crushed stone. With modern two- and three-stage crushing it is normally possible to adjust the operation so that a workable grading is obtained. Unless finish screening is employed, it is well to reduce the amount of the finest size of coarse aggregate since that is the size of the accumulated undersize of the larger sizes. However, finish screening at the batching plant, on horizontal vibrating screens and with no intermediate storage, is strongly recommended for mass concrete coarse aggregates. With finish screening there is little difficulty in limiting undersize to 4 percent of the cobbles, 3 percent of the intermediate sizes, and 2 percent of the fine coarse aggregates. Undersize is defined as that passing a test screen having openings five-sixths of the nominal minimum size of the aggregate fraction. Undersize larger than this five-sixths fraction has no measurable effect on the concrete (Tuthill 1943).

2.5.10—In some parts of the world "gap" gradings are used in mass concrete. These are gradings in which the material in one or more sieve sizes is missing. In United States practice, continuous gradings are normally used. Gap gradings can be used economically where the material occurs naturally gap-graded. But comparisons which can be made between concretes containing gap-graded aggregate and continuously graded aggregate indicate there is no advantage in purposely producing gap gradings. Continuous gradings produce more workable mass concrete with somewhat lower slump, less water, and less cement. Continuous gradings can always be produced from crushing operations. Most natural aggregate deposits in the United States contain material from which acceptable continuous gradings can be economically prepared.

2.6—Water

2.6.1—Water used for mixing concrete should be free of materials that significantly affect the hydration reactions of portland cement (Steinour 1960). Water that is fit to drink may generally be regarded as acceptable for use in mixing concrete. Potability will preclude any objectionable content of chlorides. However, chloride content tests should be made on any questionable water if embedded metals are present. Limits on total chloride for various constructions are contained in ACI 201.2R. When it is desirable to determine whether a water contains materials that significantly affect the strength development of cement, comparative strength tests should be made on mortars made with water from the proposed source and with distilled water. If the average of the results of these tests on specimens containing the water being evaluated is less than 90 percent of that obtained with specimens containing distilled water, the water represented by the sample should not be used for mixing concrete. If a potential water source lacking a service record contains amounts of impurities as large as 5000 ppm or more, then, to insure durable concrete, tests for strength and volume stability (length change) may also be advisable.

2.6.2—Waters containing up to several parts per million of ordinary mineral acids, such as hydrochloric acid or sulfuric acid, can be tolerated as far as strength development is concerned. Waters containing even small amounts of various sugars or sugar derivatives should not be used as setting times may be unpredictable. The harmfulness of such waters may be revealed in the comparative strength tests.

2.7—Selection of proportions

2.7.1—The primary objective of proportioning studies for mass concrete is to establish economical mixes of proper strength, durability, and impermeability with the best combination of available materials that will provide adequate workability for placement and least practical rise in temperature after placement. Trial mix methods are generally used following procedures in ACI 211.1, Appendix 5.

2.7.2—Selection of the water-cement ratio or water-cementitious material ratio will establish the strength, durability, and permeability of the concrete. There also must be sufficient fine material to provide proper placeability. Experience has shown that with the best shaped aggregates of 6 in. (150 mm) maximum size, the quantity of cement-size material required for workability is about 10 percent less than for a concrete containing angular aggregates. Trial mixes using the required water-cementitious material ratio and the observed water requirement for the job materials will demonstrate the cementitious material content that may be safely used to provide the required workability (Portland Cement Association 1979; Ginzburg, Zinchenko, and Skuortsova 1966).

2.7.3—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 2.5. The next step is to assume or determine the total water content needed to provide required slump which may be as low as 1-$^1/_2$ in. (38 mm) to 2 in. (50 mm). In tests for slump, aggregate larger than 1$^1/_2$ in. (38 mm) must 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 about 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 percent higher. However, for strengths above 4000 psi (28 MPa) at 1 year the 3-in. (75 mm) maximum-size aggregate may be more efficient. (See Figure 2.5.6).

2.7.4—The batch weight of the cement is determined by dividing the total weight of the mixing water by the water-cement ratio or, when workability governs, it is the minimum weight of cement required to satisfactorily place the concrete (see 2.7.2). With the batch weights of cement and water determined and with an assumed air content of 3 to 5 percent, 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 ratio of fine aggregate to total aggregate by absolute volume may be as low as 21 percent. With crushed aggregates the ratio may be in the range 25 to 27 percent.

2.7.5—When a pozzolan is included in the concrete as a part of the cementitious material, the mixture proportioning procedure does not change. Attention must be given to the following matters: (a) water requirement may change, (b) early-age strength may become critical, and (c) for maximum economy the age at which design strength is attained should be greater. Concrete containing most pozzolans gains strength somewhat more slowly than concrete made with only portland cement. However, the load on mass concrete is generally not applied until the concrete is relatively old. Therefore, mass concrete containing pozzolan is usually designed on the basis of 90-day to one-year strengths. While mass concrete does not require strength 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 must 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 pozzolan are used or when the concrete is lean and precooled.

2.8—Temperature control

2.8.1—The four elements of an effective temperature control program, any or all of which may be used for a particular mass concrete project, are: (1) cementitious material content control, where the choice of type and amount of cementitious materials can lessen the heat-generating potential of the concrete; (2) precooling, where cooling of ingredients achieves a lower concrete temperature as placed in the structure; (3) postcooling, where removing heat from the concrete with embedded cooling coils limits the temperature rise in the structure; and (4) construction management, where efforts are made to protect the structure from excessive temperature differentials by knowledgeable employment of concrete handling, construction scheduling, and construction procedures. The temperature control for a small structure may be no more than a single measure, such as restricting placing operations to cool periods at night or during cool weather. On the other extreme, some projects can be large enough to justify a wide variety of separate but complementary control measures that additionally can include the prudent selection of a low-heat-generating cement system including pozzolans; the careful production control of aggregate gradings and the use of large-size aggregates in efficient mixes with low cement contents; the precooling of aggregates and mixing water (or the batching of ice in place of mixing water) to make possible a low concrete temperature as placed; the use of air-entraining and other chemical admixtures to improve both the fresh and hardened properties of the concrete; using appropriate block dimensions for placement; coordinating construction schedules with seasonal changes to establish lift heights and placing frequencies; the use of special mixing and placing equipment to quickly place cooled concrete with minimum absorption of ambient heat; evaporative cooling of surfaces through water curing; dissipating heat from the hardened concrete by circulating cold water through embedded piping; and insulating surfaces to minimize thermal differentials between the interior and the exterior of the concrete.

It is practical to cool coarse aggregate, somewhat more difficult to cool fine aggregate, and practical to batch a portion or all of the added mixing water in the form of ice. As a result, placing temperatures of 50 F (10 C) and lower are practicable and sometimes specified. Lower temperatures are obtainable with more difficulty. Injection of liquid nitrogen into mix water has also been effectively used to lower concrete temperature for mass concrete work. In most cases a placing temperature of less than 65 F (18 C) can be achieved with liquid nitrogen injection. Cooled concrete is advantageous in mixture proportioning since water requirement decreases as temperature drops. Specified placing temperatures should be established by temperature studies to determine what is required to satisfy the design. Guidance in cooling systems for mass concrete can be found in ACI 207.4R.

2.8.2—The chief means for limiting temperature rise is controlling the type and amount of cementitious materials. The goal of concrete proportioning studies is to reach a cementitious material content no greater than is necessary for the design strength. The limiting factor in reaching this low cementitious material level is usually the need to use some minimum amount of cement-sized particles solely to provide workability in the concrete. Without the use of supplemental workability agents—such as pozzolans, air-entraining, or other chemical admixtures—a mass concrete project can experience a continuing struggle to maintain workability while holding to the low cementitious material content that best protects against cracking. The ASTM specification for Type II portland cement contains an option which makes it possible to limit the heat of hydration to 70 cal/g (290 kJ/kg) at 7 days. Use of a pozzolan as a replacement further delays and reduces heat generation. This delay is an advantage—except that when cooling coils are used, the period of postcooling may be extended. If the mixture is proportioned so that the cementitious materials content is limited to not more than 235 lb/yd^3 (139 kg/m^3), the temperature rise for most concretes will not exceed 35 F (19 C). A complete discussion of temperature control is given in Chapter 5.

CHAPTER 3—PROPERTIES

3.1—General

3.1.1—The design and construction of massive concrete structures, especially dams, is influenced by site topography, foundation characteristics, and the availability of suitable materials of construction. Economy, second only to safety requirements, is the most important single parameter to consider. Economy may dictate the choice of type of structure for a given site. Proportioning of the concrete is in turn governed by the requirements of the type of structure and such properties as the strength, durability, and thermal properties. For large structures extensive investigations of aggregates, admixtures, and pozzolans are justified. Concrete mixture investigations are necessary to determine the most economical proportions of selected ingredients to produce the desired properties of the concrete. Within recent years an increasing utilization has been made of finite element computer programs for thermal analysis (Polivka and Wilson 1976; U.S. Army Corps of Engineers 1994). Determination of tensile strain capacity has also lead to a better understanding of the potential for cracking under rapid and slow loading conditions (Houghton 1976).

3.1.2—The specific properties of concrete which should be known are compressive strength, tensile strength, modulus of elasticity, Poisson’s ratio, tensile strain capacity, creep, volume change during drying, adiabatic temperature rise, thermal coefficient of expansion, specific heat, thermal conductivity and diffusivity, permeability, and durability. Approximate values of these properties based on computations or past experience are often used in preliminary evaluations. Useful as such approximations may be, the complex heterogeneous nature of concrete and the physical and chemical interactions of aggregate and paste are still not sufficiently known to permit estimation of reliable values. For this reason, it is again emphasized that extensive laboratory and field investigations must be conducted to assure a safe structure at lowest cost. In addition, the moisture condition of the specimens and structure, and the loading rate required, must be known, as these factors may dramatically affect some concrete properties. Specimen size and orientation effects on mass concrete test properties can also be significant.

3.1.3—A compilation of concrete proportion data on representative dams is given in Table 3.1.3 (Price and Higginson 1963; Ginzburg, Zinchenko, and Skuortsova 1966; ICOLD 1964; Harboe 1961; U.S. Bureau of Reclamation 1958; Houghton and Hall 1972; Houghton 1970; Houghton 1969). Reference will be made to concrete mixes described in Table 3.1.3 in discussions of properties reported in Tables 3.2.1, 3.3.2, 3.4.2, 3.5.1, 3.7.1, and 3.8.1.

3.2—Strength

3.2.1—The water-cementitious material ratio to a large extent governs the quality of the hardened portland cement binder. Strength, impermeability, and most other desirable properties of concrete are improved by lowering the water-cementitious material ratio. A study of compressive strength data given in Table 3.2.1 shows a considerable variation from the direct relationship between water-cementitious material ratio and strength. Factors, totally or partially independent of the water-cementitious material ratio, which affect the strength are: (1) composition and fineness of cement, (2) amount and type of pozzolan, (3) surface texture and shape of the aggregate, (4) the mineralogic makeup and strength of the aggregate, (5) aggregate grading, and (6) the improvement of strength by admixtures above that attributable to a reduction in water-cementitious material ratio.

3.2.2—High strengths are usually not required in mass concretes except in thin arch dams. Concrete proportioning should determine the minimum cement content for adequate strength to give greatest economy and minimum temperature rise. Cement requirements for adequate workability and durability rather than strength frequently govern the portland cement content.

Table 3.1.3—Typical concrete mix data from various dams

No.	Name of dam (country)	Year completed	Type	Cement Type	Cement lb/yd^3 (kg/m^3)	Pozzolan Type	Pozzolan lb/yd^3 (kg/m^3)	Sand lb/yd^3 (kg/m^3)	Coarse aggregate lb/yd^3 (kg/m^3)	Coarse aggregate Type	Maximum size aggregate, in. (mm)	Water, lb/yd^3 (kg/m^3)	W/(C+P) or W/C	Entrained air, percent	Density lb/ft^3 (kg/m^3)	Water-reducing admixture used
1	Hoover (USA)	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 (USA)	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 (USA)	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 (USA)	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 (USA)	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 (USA)	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 (USA)	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 (USA)	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 (USA)	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 (USA)	1957	Arch	II LA	212 (126)	Calcined 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 (USA)	1962	Arch gravity	II	188 (111)	Calc, shale	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 (USA)*	1963	Arch gravity	II	188 (111)	Pumicite	94 (56)	777 (461)	2784 (1651)	Limestone, chalcedonic chert and sandstone	6.0 (150)	153 (91)	0.54	3.5	148.0 (2371)	No
12a		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	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 (USA)	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 (USA)	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 (USA)	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

* 0.37 percent admixture added

Table 3.1.3—Typical concrete mix data from various dams, continued

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/(C+P) or W/C	Entrained air, percent	Density lb/ft^3 (kg/m^3)	Water-reducing admixture 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						
17	Lower Granite (USA)	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
18	Pueblo (USA)	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 (USA)	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 (USA)	1982	Gravity	II II	226 (134) 173 (103)	Fly ash Fly ash	59 (35) 73 (43)	822 (488) 864 (513)	2958 (1755) 2935 (1741)	Crushed granite	6.0 (150) 6.0 (150)	173 (103) 177 (105)	0.57 0.67	3.4 3.4	157.0 (2515) 156.0 (2499)	Yes 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* (150)	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	Salamonde (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 (USSR)	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, cr. 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)	0.70	4.0	158.4 (2537)	No
30	Peace Site I (Canada)	1979	Gravity	I	158 (94)	Fly ash	105 (63)	967 (575)	2610 (1549)	Quartzite limestone sandstone	3 (75)	170 (101)	0.67	3.6	148.5 (2379)	Yes
31	Theo. Roosevelt Modification (USA)	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

* 25 percent pozzolan interground with cement for summer months.

Table 3.2.1—Cement/water requirements and strengths of concrete in various dams

Dam	Country	Cement or cement-pozzolan, lb/yd^3 (kg/m^3)	Water, lb/yd^3 (kg/m^3)	Predominant aggregate type	Maximum size aggregate, in. (mm)	W/(C+P) or W/C	90-day strength, psi (MPa)	Cement efficiency at 90 days, psi/lb/yd^3 (MPa/kg/m^3)
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)	Porphyry	3.9 (98)	0.59	4920 (33.9)	12.2 (0.141)
Cabril	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	Switz.	420 (249)	225 (133)	Glacial mix	2.5 (64)	0.54	5990 (41.3)	14.3 (0.166)
Mauvoisin	Switz.	319 (189)	162 (96)	Gneiss	3.8 (96)	0.51	4960 (34.2)	15.5 (0.181)
Zervreila	Switz.	336 (199)	212 (126)	Gneiss	3.8 (96)	0.63	3850 (26.5)	10.5 (0.133)
Hungry Horse	USA	188-90 (111-53)	130 (77)	Sandstone	6.0 (150)	0.47	3100 (21.4)	11.2 (0.130)
Glen Canyon	USA	188-94 (111-56)	153 (91)	Limestone	6.0 (150)	0.54	3810 (26.3)	13.5 (0.160)
Lower Granite	USA	145-49 (86-29)	138 (82)	Basalt	6.0 (150)	0.71	2070 (14.3)	10.7 (0.124)
Libby	USA	148-49 (88-29)	133 (79)	Quartzite	6.0 (150)	0.68	2460 (17.0)	12.5 (0.145)
Dworshak	USA	211-71 (125-42)	164 (97)	Granite	6.0 (150)	0.58	3050 (21.0)	10.8 (0.126)
Dworshak	USA	198-67 (117-40)	164 (97)	Gneiss	6.0 (150)	0.62	2530 (17.4)	9.5 (0.111)
Dworshak	USA	168-72 (100-43)	166 (98)	Gneiss	6.0 (150)	0.69	2030 (14.0)	8.5 (0.098)
Dworshak	USA	174-46 (130-27)	165 (98)	Gneiss	6.0 (150)	0.75)	1920 (13.2)	8.7 (0.084)
Pueblo	USA	226-75 (134-44)	168 (100)	Granite limestone dolomite	3.5 (89)	0.56	3000* (20.7)	10.0 (0.116)
Crystal	USA	390 (231)	183 (109)	Shist and altered volanics	3.0 (75)	0.47	4000† (27.6)	10.3 (0.119)
Flaming Gorge	USA	188-94 (111-56)	149 (88)	Limestone and sandstone	6.0 (150)	0.53	3500 (24.1)	12.4 (0.144)
Krasnoiarsk	USSR	388 (230)	213 (126)	Granite	3.9 (100)	0.55	3280 (22.6)	8.5 (0.098)
Ilha Solteira	Brazil	138-46 (82-27)	138 (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)
Theo. Roosevelt Modification	USA	270 (160)	144 (85)	Granite	4.0 (100)	0.53	4500 (31.0)	16.7 (0.194)

* Strength at 180 days
† Strength at one yr

3.2.3—Mass concrete is seldom required to withstand substantial stress at early age. Therefore, to take full advantage of the strength properties of the cementing materials, the design strength is usually based on the strength at ages from 90 days to one year; and sometimes up to two years. Job control cylinders must of necessity be tested at an earlier age if they are to be useful in exercising control and maintaining consistency during the progress of the construction. For the sake of convenience, job control test specimens are usually 6 x 12-in. (150 x 300-mm) cylinders containing concrete wet screened to $1^1/_2$ in. (37.5 mm) maximum size. It is important that correlation tests be made well in advance of construction to compare the strength of wet-screened concrete tested at the control age with appropriate-size test specimens containing the full mass concrete tested at the design test age. The strength of large test specimens will usually be only 80 to 90 percent of the strength of 6 x 12-in. (150 x 300-mm) cylinders tested at the same age. Accounting for the continued strength development beyond 28 days, particularly where pozzolans are employed, the correlation factors at one year may range from 1.15 to 3.0 times the strength of the wet-screened control specimens tested at 28 days.

3.2.4—Accelerated curing procedures set forth in ASTM C 684 yield compression test results in 24 to 48 hr that can provide an indication of potential concrete strength. However, the use of these procedures should be limited to detecting variations in concrete quality and judging the effectiveness of job control measures. The accelerated strength indicator is helpful where satisfactory correlation has been established with longer-term values using companion specimens of the same concrete. Although the indicator may have dubious relationship to the actual future strength in the concrete structure, it can be helpful during construction.

3.2.5—The factors involved in relating results of strength tests on small samples to the probable strength of mass concrete structures are several and complex and still essentially unresolved. Because of these complexities, concrete strength requirements are usually several times the calculated maximum design stresses for mass concrete structures. For example, design criteria for gravity dams commonly used by the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers 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 (U.S. Bureau of Reclamation 1976; U.S. Army Corps of Engineers 1990).

Table 3.3.2— Compressive strength and elastic properties of mass concrete

No	Dam	Compressive strength psi (MPa) Age, days				Elastic properties: Modulus of elasticity, $E \times 10^6$ psi ($E \times 10^4$ MPa) Age, days				Poisson's ratio 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 (3.8)	6.2 (4.3)	—	6.8 (4.7)	0.18	0.20	—	0.21
2	Grand Coulee	4780 (33.0)	5160 (35.6)	—	5990 (41.3)	4.7 (3.2)	6.1 (4.2)	—	6.0 (4.1)	0.17	0.20	—	0.23
3	Glen Canyon	2550 (17.6)	3810 (26.3)	3950 (27.2)	—	5.4 (3.7)	—	5.8 (4.0)	—	0.11	—	0.14	—
3a	Glen Canyon*	3500 (24.1)	4900 (33.8)	6560 (45.2)	6820 (47.0)	5.3 (3.7)	6.3 (4.3)	6.7 (4.6)	—	0.15	0.15	0.19	—
4	Flaming Gorge	2950 (20.3)	3500 (24.1)	3870 (26.7)	4680 (32.3)	3.5 (2.4)	4.3 (3.0)	4.6 (3.2)	—	0.13	0.25	0.20	—
5	Yellowtail	—	4580 (31.6)	5420 (37.4)	5640 (38.9)	—	6.1 (4.2)	5.4 (3.7)	6.2 (4.3)	—	0.24	0.26	0.27
6	Morrow Point*	4770 (32.9)	5960 (41.1)	6430 (44.3)	6680 (46.1)	4.4 (3.0)	4.9 (3.4)	5.3 (3.7)	4.6 (3.2)	0.22	0.22	0.23	0.20
7	Lower Granite*	1270 (8.8)	2070 (14.3)	2420 (16.7)	2730 (18.8)	2.8 (1.9)	3.9 (2.7)	3.8 (2.6)	3.9 (2.7)	0.19	0.20	—	—
8	Libby	1450 (10.0)	2460 (17.0)	—	3190 (22.0)	3.2 (2.2)	4.0 (2.8)	—	5.5 (3.8)	0.14	0.18	—	—
9	Dworshak*	1200 (8.3)	2030 (14.0)	—	3110 (21.4)	—	3.7 (2.6)	—	3.8 (2.6)	—	—	—	—
10	Ilha Solteira	2320 (16.0)	2755 (19.0)	3045 (21.0)	3190 (22.0)	5.1 (3.5)	5.9 (4.1)	—	—	0.15	0.16	—	—
11	Itaipu	1885 (13.0)	2610 (18.0)	2610 (18.0)	2755 (19.0)	5.5 (3.8)	6.2 (4.3)	6.2 (4.3)	6.5 (4.5)	0.18	0.21	0.22	0.20
12	Peace Site* 1	3060 (21.1)	3939 (27.2)	4506 (31.1)	4666 (32.2)	—	—	—	—	—	—	—	—
13	Theodore Roosevelt Modification	2400 (16.5)	4500 (31.0)	5430 (37.4)	5800 (40.0)	4.5 (3.1)	5.4 (3.7)	—	6.2 (4.3)	0.20	0.21	—	0.21

*Water-reducing agent used.

3.2.6—Concrete that is strong in compression is also strong in tension but this strength relationship is not linear. Tensile strength can be measured by several tests, primarily direct tensile, splitting tensile, and modulus of rupture (flexural) tests. Each of these tests has a different relationship with compressive strength. An expression that relates tensile strength, f_t, to compressive strength, f_c, is

for f_t and f_c in psi

$$f_t = 1.7 f_c^{2/3}$$

for f_t and f_c in MPa

$$f_t = 0.32 f_c^{2/3}$$

Raphael (1984) discussed these and other tensile-compressive strength relationships, and their use in design. Relationships of these types for specific materials can vary significantly from the formulas above, based on aggregate quality and many other factors. Where feasible and necessary, testing should be conducted to confirm these relationships.

3.2.7—The strength of concrete is also influenced by the speed of loading. Values usually reported are for static loads that take appreciable time to develop, e.g. dead load or water load. During earthquakes, however, stresses may be fully developed in a small fraction of a second. It has been found that when loaded at this speed, compressive strength of a concrete for moist specimens may be increased up to 30 percent and tensile strength may be increased up to 50 percent, when compared to values obtained at standard rates of loading (Saucier 1977; Graham 1978; Raphael 1984).

3.3—Elastic properties

3.3.1—Concrete is not a truly elastic material, and the graphic stress-strain relationship for continuously increasing load is generally in the form of a curved line. However, the modulus of elasticity is for practical purposes considered a constant within the range of stresses to which mass concrete is usually subjected.

3.3.2—The moduli of elasticity of concrete representative of various dams are given in Table 3.3.2. These values range from 2.8 to 5.5 x 10^6 psi (1.9 to 3.8 x 10^4 MPa) at 28 days and from 3.8 to 6.8 x 10^6 psi (2.6 to 4.7 x 10^4 MPa) at one year. Usually, concretes having higher strengths have higher values of elastic modulus and show a general correlation of increase in modulus with strength, although modulus of elasticity is not directly proportional to strength, since it is influenced by the modulus of elasticity of the aggregate. In the past, data from concrete modulus of elasticity tests showed relatively high coefficient of variation resulting from attempts to measure small strains on a heterogeneous mixture

Table 3.4.2— Elastic properties of mass concrete

Age at time of loading	Instantaneous and sustained modulus of elasticity,* psi x 10^6 (MPa x 10^4)														
	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 (1.2)	0.83 (0.57)	0.76 (0.52)	1.4 (0.97)	0.54 (0.37)	0.49 (0.34)	2.8 (1.9)	1.5 (1.0)	1.4 (0.97)	1.4 (0.97)	0.75 (0.52)	0.70 (0.48)	1.6 (1.1)	1.0 (0.69)	0.9 (0.62)
7 days	2.3 (1.6)	1.1 (0.76)	1.0 (0.69)	2.1 (1.4)	1.0 (0.69)	0.96 (0.66)	4.2 (2.9)	1.9 (1.3)	1.8 (1.2)	2.0 (1.4)	1.0 (0.69)	0.90 (0.62)	3.2 (2.2)	1.6 (1.1)	1.3 (0.90)
20 days	3.5 (2.4)	1.8 (1.2)	1.6 (1.1)	3.5 (2.4)	1.8 (1.2)	1.6 (1.1)	4.5 (3.1)	2.6 (1.8)	2.4 (1.7)	2.8 (1.9)	1.4 (0.97)	1.3 (0.90)	4.1 (2.8)	2.2 (1.5)	2.0 (1.4)
90 days	4.1 (2.0)	2.5 (1.7)	2.3 (1.6)	4.4 (3.0)	2.7 (1.9)	2.5 (1.7)	5.2 (3.6)	3.2 (2.2)	3.0 (2.1)	3.8 (2.6)	2.2 (1.5)	2.0 (1.4)	5.2 (3.6)	2.9 (2.0)	2.7 (1.9)
1 yr	5.0 (3.4)	2.5 (1.7)	2.3 (1.6)	4.4 (3.0)	2.7 (1.9)	2.5 (1.7)	5.2 (3.6)	3.2 (2.2)	3.0 (2.1)	3.8 (2.6)	2.2 (1.5)	2.0 (1.4)	5.2 (3.6)	2.9 (2.0)	2.7 (1.9)
5 yr	5.3 (3.7)	3.6 (2.5)	3.4 (2.3)				5.9 (4.1)	4.0 (2.8)	3.8 (2.6)	4.9 (3.4)	3.0 (2.1)	2.9 (2.0)	6.4 (4.4)	4.3 (3.0)	4.1 (2.8)
$7^1/_4$ yr				5.6 (3.9)	4.3 (3.0)	4.1 (2.8)									

*All concretes mass mixed, wet screened to $1^1/_2$ in. (37.5 mm) maximum-size aggregate.
E = instantaneous modulus of elasticity at time of loading.
E^1 = sustained modulus after 365 days under load.
E^2 = sustained modulus after 1000 days under load.
Note: The instantaneous modulus of elasticity refers to the "static" or normal load rate (1 to 5 min duration) modulus, not a truly instantaneous modulus measured from "dynamic" or rapid load rate testing.

containing large-size aggregate. Modern electronic devices such as the linear variable differential transformer (LVDT) can measure small length changes with great accuracy. Tensile modulus of elasticity is generally assumed to be identical to the compressive modulus of elasticity.

3.3.3—Poisson's ratio data given in Table 3.3.2 tend to range between the values of 0.16 and 0.20 with generally 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, the cement paste, and relative proportions of the two.

3.3.4—The growth of internal microcracks in concrete under load commences at compressive stresses equal to about 35 to 50 percent of the nominal compressive strength under short term loading. Above this stress, the overall volumetric strain reflects the volume taken up by these internal fissures, and Poisson's ratio and the elastic moduli are no longer constant.

3.3.5—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).

3.4—Creep

3.4.1—Creep of concrete is partially-recoverable plastic deformation that occurs while concrete is under sustained stress. Creep appears to be mainly related to the modulus of elasticity of the concrete. Concretes having high values of modulus of elasticity generally have low values of creep deformation. The cement paste is primarily responsible for concrete creep. With concretes containing the same type of aggregate, the magnitude of creep is closely related to the paste content (Polivka, Pirtz, and Adams 1963) and the water-cementitious material ratio of the concrete. ACI 209R discusses the prediction of creep, shrinkage, and temperature effects in concrete structures.

3.4.2—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-mixed concrete wet screened to $1^1/_2$ in. (37.5 mm) maximum size, are recorded in Table 3.4.2. The instantaneous modulus is measured immediately after the concrete is subjected to load. The sustained modulus represents values after 365 and 1000 days under load. From Table 3.4.2 it can be seen that the sustained values for modulus are approximately one-half that of the instantaneous modulus when load is applied at early ages and is a slightly higher percentage of the instantaneous modulus when the loading age is 90 days or greater. Creep of concrete appears to be approximately directly proportional to the applied stress/strength ratio up to about 40 percent of the ultimate strength of the concrete.

3.5—Volume change

3.5.1—Volume changes are caused by changes in moisture content of the concrete, changes in temperature, chemical reactions, and stresses from applied loads. Excessive volume change is detrimental to concrete. Cracks are formed in restrained concrete as a result of shrinkage or contraction and insufficient tensile strength or strain capacity. Cracking is a weakening factor that may affect the ability of the concrete to withstand its design loads and may also detract from durability and appearance. Volume change data for some mass concretes are given in Table 3.5.1. Various factors influencing cracking of mass concrete are discussed in Carlson, Houghton, and Polivka (1979).

Table 3.5.1— Volume change and permeability of mass concrete

Structure	Autogenous volume change		Drying shrinkage	Permeability,	
	90 days, millionths	1 yr, millionths	1 yr, millionths	*K* ft/s/ft* hydraulic head	m/s/m*
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}
Clen 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	—		

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 the other dams tabulated 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).

*ft/s/ft = ft^3/ft^2-s/ft of hydraulic head; m/s/m = m^3/m^2-s/m of hydraulic head; millionths = in. x 10^{-6}/in. (mm x 10^{-6}/mm), measured in linear length change.

3.5.2—Drying shrinkage ranges from less than 0.02 percent (or 200 millionths) for low-slump lean concrete with good quality aggregates to over 0.10 percent (or 200 millionths) for rich mortars or some concretes containing poor quality aggregates and an excessive amount of water. The principal drying shrinkage of hardened concrete is usually occasioned by the drying and shrinking of the cement gel which is formed by hydration of portland cement. The main factors affecting drying shrinkage are the unit water content and aggregate mineralogy and content. Other factors influence drying shrinkage principally as they influence the total amount of water in mixtures. The addition of pozzolans generally increases drying shrinkage except where the water requirement is significantly reduced, such as with fly ash. Some aggregates, notably graywacke and sandstone, have been known to contribute to extremely high drying shrinkage. ACI 224R and Houghton (1972) discuss the factors involved in drying characteristics of concrete.

3.5.3—Autogenous volume change results from the chemical reactions within the concrete. Unlike drying shrinkage it is unrelated to the amount of water in the mix. The net autogenous volume change of most concretes is a shrinkage of from 0 to 150 millionths. When autogenous expansion occurs it usually takes place within the first 30 days after placing. Concretes containing pozzolans may sometimes have greater autogenous shrinkage than portland cement concrete without pozzolans (Houk, Borge, and Houghton 1969).

3.5.4—The thermal coefficient of expansion of a concrete depends mainly upon the type and amount of coarse aggregate in the concrete. Various mineral aggregates may range in thermal coefficients from below 2 millionths to above 8 millionths per deg F (3 to 14 millionths per deg C). Neat cement pastes will vary from about 6 millionths to 12 millionths per deg F (10 millionths to 21 millionths per deg C) depending on the chemical composition and the degree of hydration. The thermal coefficient of the concrete usually reflects the weighted average of the various constituents. Sometimes coefficient of expansion tests are conducted on concrete that has been wet screened to $1^1/_2$ in. (37.5 mm) maximum size in order to work with smaller-size specimens. However, the disproportionately larger amount of cement paste, which has a higher coefficient, results in values higher than that of the mass concrete. Concrete coefficients of thermal expansion are best determined on specimens containing the full concrete mix. Refer to values in Table 3.7.1.

3.5.5—The portland cement in concrete liberates heat when it hydrates and the internal temperature of the concrete rises during this period (Dusinberre 1945; Wilson 1968). The concrete is relatively elastic during this early stage, and it can be assumed to be at or near zero stress when the maximum temperature is attained. When cooling begins, the concrete is gaining strength and stiffness rapidly. If there is any restraint against free contraction during cooling, tensile strain and stress develop. The tensile stresses developed during the cooling stage are determined by five quantities: (1) thermal differential and rate of temperature change, (2) coefficient of thermal expansion, (3) modulus of elasticity, (4) creep or relaxation, and (5) the degree of restraint. If the tensile stress developed exceeds the tensile strength of the concrete, cracking will occur (Houghton 1972; Houghton 1976; Dusinberre 1945). Principal methods utilized to reduce the potential for thermally induced cracking in concrete are outlined in ACI 224R and Carlson, Houghton, and Polivka (1979). They include reducing the maximum internal temperature which the concrete attains; reducing the rate at which the concrete cools; and increasing the tensile strength of the concrete. Concrete resistance to cracking can be equated to tensile strain ca-

Table 3.7.1—Thermal properties of concrete

Structure	Coarse aggregate type	Inch-pound units: Temperature, F	Inch-pound units: Coefficient of expansion,* millionths/F: $1^1/_2$ in. max	Inch-pound units: Coefficient of expansion,* millionths/F: $4^1/_2$ in. max	Inch-pound units: Thermal conductivity,† $\frac{Btu}{ft \times hr \times F}$	Inch-pound units: Specific heat, $\frac{Btu}{lb \times F}$	Inch-pound units: Density, $\frac{lb}{ft^3}$	Inch-pound units: Diffusivity,** $\frac{ft^2}{hr}$	SI units: Temperature, C	SI units: Coefficient of expansion,* millionths/C: 37.5 mm max	SI units: Coefficient of expansion,* millionths/C: 114 mm max	SI units: Thermal conductivity,† $\frac{kJ}{m \times hr \times C}$	SI units: Specific heat $\frac{kJ}{kg \times C}$	SI units: Density, $\frac{kg}{m^3}$	SI units: Diffusivity,** $\frac{m^2}{hr} \times 10^{-3}$
Hoover	Limestone and granite	50 100 150	5.3	4.8	1.70 1.67 1.65	0.212 0.225 0.251	156.0	0.051 0.047 0.042	10 38 66	9.5	8.6	10.6 10.4 10.3	0.887 0.941 1.050	2500	4.7 4.4 3.9
Grand Coulee	Basalt	50 100 150	4.4	4.6	1.08 1.08 1.09	0.219 0.231 0.257	158.1	0.031 0.029 0.027	10 38 66	7.9	8.3	6.74 6.74 6.78	0.916 0.967 1.075	2534	2.9 2.7 2.5
Friant	Quartzite gran-ite and rhyolite	50 100 150	—	—	1.23 1.23 1.24	0.216 0.230 0.243	153.8	0.037 0.035 0.033	10 38 66	—	—	7.66 7.66 7.70	0.904 0.962 1.017	2465	3.4 3.2 3.1
Shasta	Andesite and slate	50 100 150	—	4.8	1.32 1.31 1.31	0.219 0.233 0.247	156.6	0.039 0.036 0.034	10 38 66	—	8.6	8.20 8.16 8.16	0.916 0.975 1.033	2510	3.6 3.3 3.2
Angostura	Limestone	50 100 150	4.0	—	1.49 1.48 1.46	0.221 0.237 0.252	151.2	0.045 0.041 0.038	10 38 66	7.2	—	9.29 9.20 9.08	0.925 0.992 1.054	2423	4.2 3.8 3.5
Kortes	Granite gabbros and quartz	50 100 150	5.2	4.5	1.61 1.60 1.59	0.208 0.221 0.234	151.8	0.050 0.047 0.044	10 38 66	9.4	8.1	10.0 9.96 9.87	0.870 0.925 0.979	2433	4.6 4.4 4.1
Hungry Horse	Sandstone	50 100 150	6.2	5.7	1.72 1.71 1.69	0.217 0.232 0.247	150.1	0.053 0.049 0.046	10 38 66	9.7	9.4	10.1 10.0 9.87	0.895 0.937 0.983	2425	4.6 4.4 4.2
Monticello	Sandstone metasiltstone quartzite rhyolite	50 100 150	5.2	—	1.57 1.55 1.53	0.225 0.237 0.250	151.3	0.046 0.043 0.040	10 38 66	9.4	—	9.79 9.67 9.54	0.941 0.992 1.046	2454	4.3 4.0 3.7
Anchor	Andesite, latite and limestone	50 100 150	5.6	4.5	1.14 1.14 1.15	0.227 0.242 0.258	149.0	0.034 0.032 0.030	10 38 66	10.1	8.1	7.11 7.11 7.15	0.950 1.013 1.079	2388	3.2 3.0 2.8
Glen Canyon	Limestone, chert and sandstone	50 100 150	—	—	2.13 2.05 1.97	0.217 0.232 0.247	150.2	0.065 0.059 0.053	10 38 66	—	—	13.3 12.8 12.3	0.908 0.971 1.033	2407	6.0 5.5 4.9
Flaming Gorge	Limestone and sandstone	50 100 150	—	—	1.78 1.75 1.73	0.221 0.234 0.248	150.4	0.054 0.050 0.046	10 38 66	—	—	11.1 10.9 10.8	0.925 0.979 1.038	2411	5.0 4.6 4.3
Yellowtail	Limestone and andesite	50 100 150	—	4.3	1.55 1.52 1.48	0.226 0.239 0.252	152.5	0.045 0.042 0.039	10 38 66	—	7.7	9.67 9.46 9.20	0.946 1.000 1.054	2444	4.2 3.9 3.6
Libby	Natural quartz gravel	100	6.5	6.0	2.24	0.220	152	0.067	36	11.7	10.8	13.9	0.920	2435	6.2
Dworshak	Granite gneiss	100	—	5.5	1.35	0.220	154	0.040	36	—	9.9	8.41	0.920	2467	3.9
Ilha Solteira	Quartzite and basalt	100	—	6.9	1.73	0.220	159	0.049	36	—	12.5	10.8	0.920	2552	4.6
Itaipu	Basalt	100	—	4.3	1.06	0.233	158	0.029	36	—	7.8	6.61	0.975	2537	2.7
Theodore Roosevelt Modification	Granite	50 100 150	4.3	—	1.71 1.73 1.70	0.234 0.248 0.260	148.7	0.049 0.047 0.044	10 38 66	7.7	—	10.7 10.9 10.6	0.979 1.037 1.088	2380	4.6 4.4 4.1

*$1^1/_2$ in. (37.5 mm) max and $4^1/_2$ in. (114 mm) max refer to maximum size of aggregate in concrete.

†Procedure for calculating thermal conductivity is described in CRD-44 (U.S. Army Corps of Engineers, 1949).

**Diffusivity is often expressed in ft^2/day (m^2/day) for convenience in calculations.

pacity rather than to strength. When this is done, the average modulus of elasticity (sustained E) can be omitted from the testing and computation requirements (ACI 207.2R; Houghton 1976). Tensile strain capacity may be predicted using compressive strength and the modulus of elasticity (Liu and McDonald 1978). Thermal tensile strain capacity of the concrete is measured directly in tests on concrete made during the design stages of the project. Thermal tensile strain developed in mass concrete increases with the magnitude of the thermal coefficient of expansion, thermal differential and rate of temperature change, and degree of restraint (ACI 207.2R).

3.5.6—Volume changes can also result from chemical reactions, which can be potentially disruptive. These reactions are discussed in 3.9.4.

3.6—Permeability

3.6.1—Concrete has inherently low permeability to water. With properly proportioned mixtures that are compacted by vibration, permeability is not a serious problem. Permeability of concrete increases with increasing water-cementitious material ratios (U.S. Bureau of Reclamation 1981). Therefore, low water-cementitious material ratio and good consolidation and curing are the most important factors in producing concrete with low permeability. Air-entraining and other chemical admixtures permit the same workability with reduced water content and therefore contribute to reduced permeability. Pozzolans usually reduce the permeability of the concrete. Permeability coefficients for some mass concretes are given in Table 3.5.1.

3.7—Thermal properties

3.7.1—Thermal properties of concrete are significant in connection with keeping differential volume change at a minimum in mass concrete, extracting excess heat from the concrete, and dealing with similar operations involving heat transfer. These properties are specific heat, conductivity, and diffusivity. The main factor affecting the thermal properties of a concrete is the mineralogic composition of the aggregate (Rhodes 1978). Since the selection of the aggregate to be used is based on other considerations, little or no control can be exercised over the thermal properties of the concrete. Tests for thermal properties are conducted only for providing constants to be used in behavior studies as described in Chapter 5. Specification requirements for cement, pozzolan, percent sand, and water content are modifying factors but with negligible effect on these properties. Entrained air is an insulator and reduces thermal conductivity, but other considerations which govern the use of entrained air outweigh the significance of its effect on thermal properties. Some rock types, such as granite, can have a rather wide range of thermal properties depending upon their source. Quartz aggregate is particularly noted for its high value of thermal conductivity. Thermal property values for some mass concretes are given in Table 3.7.1. Thermal coefficient of expansion is discussed in Section 3.5.4.

3.8—Shear properties

3.8.1—Although the triaxial shear strength may be determined as one of the basic design parameters, the designer usually is required to use an empirical relationship between the shear and compressive strength of concrete. Shear properties for some concretes containing $1^1/_2$-in. (37.5 mm) maximum-size aggregates are listed in Table 3.8.1. These include 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 standard push-off tests. Past criteria have stated that the coefficient of internal friction can be taken as 1.0 and cohesion as 10 percent of the compressive strength (U.S. Bureau of Reclamation 1976). More recent investigation has concluded that assuming this level of cohesion may be unconservative (McLean & Pierce 1988).

3.8.2—The shear strength relationships reported can be linearly 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 ϕ, which is the slope of the line, represents the coefficient of internal fric-

Table 3.8.1— Shear properties of concrete**

Dam	Age, days	W/C	Compressive strength psi	Compressive strength MPa	Cohesion psi	Cohesion MPa	Tan ø	S_s/S_c§
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
Monti-cello	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
Dwor-shak	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

*W/C+P.

All test specimens 6 x 12 in. (150 x 300 mm) with dry, $1^1/_2$ in. (37.5 mm) maximum-size aggregate except † designates 3 x 6 in. (75 x 150 mm) test specimens sealed to prevent drying with $^3/_4$ in. (19 mm) maximum-size aggregate and ‡ designates 18 x 36 in. (450 x 900 mm) test specimens sealed to prevent drying, with 6 in. (150 mm) maximum-size aggregate.

§Cohesion divided by compressive strength.

**Triaxial tests.

tion. *X* and *Y* are normal and shear stresses, respectively. In many cases, the shear strengths in Table 3.8.1 were higher for specimens of greater age; however, no definite trend is in evidence. 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 test confining pressures used should reflect anticipated conditions in the structure. Whenever possible, direct shear tests on both parent concrete and on jointed concrete should be conducted to determine valid cohesion and coefficient of internal friction values for design.

3.8.3—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 to 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 & Pierce 1988).

3.9—Durability

3.9.1—A durable concrete is one which will withstand the effects of service conditions to which it will be subjected, such as weathering, chemical action, alkali-aggregate reactions, and wear (U.S. Bureau of Reclamation 1981). Laboratory tests can indicate relative durabilities of concretes, but it is not generally possible to directly predict durability in field service from laboratory durability studies.

3.9.2—Disintegration of concrete by weathering is caused mainly by the disruptive action of freezing and thawing 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 good materials, use of entrained air, low water-cementitious material ratio, proper proportioning, placement to provide a watertight structure, and good water curing usually provide a concrete that has excellent resistance to weathering action.

3.9.3—Chemical attack occurs from (1) exposure to acid waters, (2) exposure to sulfate-bearing waters, and (3) leaching by mineral-free waters as explained in ACI 201.2R.

No type of portland cement concrete is 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 hydrated lime and hydrated tricalcium aluminate in cement paste to form calcium sulfate and calcium sulfo-aluminates. 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.

Hydrated lime is one of the products formed when cement and water combine in concrete. This lime is readily dissolved in pure or slightly acid water, which may occur in high mountain streams. Pozzolans, which react with lime liberated by cement hydration, can prevent the tendency of lime to leach from concrete. Surfaces of tunnel linings, retaining walls, piers, and other structures are often disfigured by lime deposits 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.

3.9.4—Alkali-aggregate reaction is the chemical reaction between alkalies (sodium and potassium) from portland cement or other sources and certain constituents of some aggregates, which under certain conditions produces deleterious expansion of the concrete. These reactions include alkali-silica reaction and alkali-carbonate rock reaction, discussed in an Engineer Manual (U.S. Army Corps of Engineers 1994). Where it is necessary to use an aggregate containing reactive constituents, low-alkali cement should be specified. Also, as further insurance against alkali-aggregate reaction, a suitable pozzolan should be specified in sufficient quantity to control deleterious reaction. Fly ash is generally considered less effective in controlling alkali-silica reaction and expansion than are Class N pozzolans.

3.9.5—The principal causes of erosion of concrete surfaces are cavitation and the movement of abrasive material by flowing water. Use of concrete of increased strength and wear resistance 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 210R). The use of aeration in high velocity flows is an effective way to prevent cavitation.

CHAPTER 4—CONSTRUCTION

4.1—Batching

4.1.1—Proper batching of mass concrete requires little that is different from the accurate, consistent, reliable batching that is essential for other classes of concrete. ACI 221R covers the processing, handling, and quality control of aggregate. ACI 304R discusses the measuring, mixing, transporting, and placing of concrete.

4.1.2—The desirability of restricting the temperature rise of mass concrete by limiting the cement content of the mix creates a continuing construction problem to maintain workability in the plastic concrete. Efficient mixes for mass concrete contain unusually low portions of cementing materials, sand, and water. Thus the workability of these mixes for conventional placement is more than normally sensitive to variations in batching. This problem can be lessened by the use of efficient construction methods and modern equipment. 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 employ specialty equipment and efficient construction methods. Consistency in the batching is improved by: (1) finish screening of coarse aggregate at the batching plant, preferably on horizontal vibrating screens without intermediate storage, (2) refinements in batching equipment, such as full-scale springless dials which register all stages of the weighing operation, (3) automatic weighing and cutoff features, (4) interlocks to prevent recharging when some material remains in a scale hopper, (5) a device for instant reading of approximate moisture content of sand, (6) graphic or digital recording of the various weighing and mixing operations, and (7) equipment capable of instant automatic selection and setting of at least 11 different batch ingredients in as many different mix proportions. In large central plant mixers, the large batches commonly used for mass concrete also tend to minimize the effect of variations.

4.1.3—Since greater use is made in mass concrete of such special-purpose ingredients as ice, air-entraining, water-reducing and set-controlling admixtures, and fly ash or other pozzolans, the dependable, accurate batching of these materials has become a very important aspect of the concrete plant. For most efficient use of ice, its temperature must be less than 32 F (0 C) and it must be brittle-hard, dry, and finely broken. For maximum efficiency ice should be batched by weighing from a well-insulated storage bin, with quick discharge into the mixer along with the other ingredients. Pozzolan and ground iron blast-furnace slag are batched the same as cement.

4.1.4—Liquid admixtures are generally batched by volume, although weighing equipment has also been used successfully. Reliable admixture batching equipment is available from some admixture or batch plant manufacturers. Means should be provided for making a visual accuracy check. Provisions should be made for preventing batching of admixture while the discharge valve is open. Interlocks should also be provided that will prevent inadvertent overbatching of the admixture. Particularly with air-entraining and water-reducing admixtures, any irregularities in batching can cause troublesome variation in slump and/or air content. When several liquid admixtures are to be used, they should be batched separately into the mixer. The use of comparatively dilute solutions reduces gumming in the equipment. For continuing good operation, equipment must 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.

4.1.5—Batching tolerances frequently used are shown in Table 4.1.5.

4.2—Mixing

4.2.1—Mixers for mass concrete must 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 mixers may be used for mass concrete containing 3-in. (75-mm) aggregate.

4.2.2—Specifications for mixing time range from a minimum of 1 min for the first cubic yard plus 15 sec for each additional cubic yard (80 sec for first m^3 plus 20 sec for each additional m^3) of mixer capacity (ACI 304R and ASTM C 94) to $1^1/_2$ min for the first 2 yards plus 30 sec for each additional yard ($1^1/_2$ min for the first $1^1/_2$ m^3 plus 40 sec for each additional m^3) 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 times should be lengthened or shortened depending upon the results of mixer performance tests. Criteria for these tests are found in ASTM C 94, Annex, Table A1.1. Mixing time is best controlled by a timing device which prevents release of the discharge mechanism until the mixing time has elapsed.

Table 4.1.5— Typical batching tolerances

	Batch weights			
	greater than 30 percent of scale capacity		less than 30 percent of scale capacity	
	Batching			
Ingredient	Individual	Cumulative	Individual	Cumulative
Cement and other cementitious materials	± 1 percent of specified weight, or ± 1 percent of scale capacity, whichever is greater		not less than required weight nor more than 4 percent over required weight	
Water (by volume or weight), percent	± 1	Not recommended	± 1	Not recommended
Aggregates, percent	± 2	± 1	± 2	± 3 percent of scale capacity or ± 3 percent of required cumulative weight, whichever is less
Admixtures (by volume or weight), percent	± 3*	Not recommended	± 3*	Not recommended

*or ± 1fl oz (30 mL), whichever is greater.

4.2.3—During mixing, the batch must be closely observed to assure the desired slump. The operator and inspector must be alert and attentive. Tuthill (1950) has discussed effective inspection procedures and facilities. Preferably the operator should be stationed in the plant where he can see the batch in the mixer and be able to judge whether its slump is correct. If the slump is low, perhaps due to suddenly drier aggregate, he can immediately compensate with a little more water and maintain the desired slump. Lacking this arrangement to see into the mixer, he should be able to see the batch as it is discharged. From this he can note any change from former batches and make subsequent water adjustments accordingly. A sand moisture meter will assist in arriving at the appropriate quantitative adjustment.

4.2.4—Continuous batching and mixing (pugmill) has been used successfully in roller-compacted concrete for years, and 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.

4.3—Placing

4.3.1—Placing includes preparation of horizontal construction joints, transportation, handling, placement, and consolidation of the concrete (ACI SP-6 1963; ACI 304R; U.S. Bureau of Reclamation 1981; Tuthill 1950; Tuthill 1953).

4.3.2—Efficient and best preparation of horizontal joint surfaces begins with the activities of topping out the lift. 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 cleanup easier. As late as is feasible but prior to placement of the next lift, surface film 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. Strength of bond is accomplished by cement grains, not by protruding coarse aggregate. Joint shear strength is determined both by this bond and by interface friction. The friction contribution is affected by confining pressure and coarse aggregate interlock. Usually removal of only about 0.1 in. (a few millimeters) of inferior material will reveal a satisfactory surface.

4.3.3—The best methods of obtaining such a clean surface are by means of sandblasting (preferably wet sandblasting to avoid dust hazard) or high-pressure water jet of at least 6000 psi (41.4 MPa). Operators must be on guard to avoid harm to other personnel, to wooden surfaces, etc., from water-blasted pieces of surface material, which may be hurled forward with great force and velocity. Sandblasting has the advantage that it will do the job at any age the concrete may be, but requires handling of sandblast sand and equipment and its removal after use. The water-jet method leaves relatively little debris for cleanup and removal, but may not work as efficiently after the concrete is more than one week old. Before and after horizontal construction joint cleanup with sandblasting and high-pressure water blasting are illustrated in Fig. 4.3.3(a) and 4.3.3(b), respectively. Clean joints are essential to good bond and watertightness. “Green cutting,” which is the early removal of the surface mortar with an air-water jet about the time the concrete approaches final set, is also used. However, it may not be possible to preserve the initially clean surface until concrete is placed upon it. The initially acceptable surface may become dull with lime coatings or can become contaminated to such an extent that it may be necessary to use sandblasting or high-pressure water jets to reclean it.

4.3.4—The clean concrete surface should be approaching dryness and be free from surface moisture at the time new concrete is placed on it (U.S. Army Corps of Engineers 1959, 1963, and 1966). Testing has shown superior strength and watertightness of joints that are dry and clean when the overlying concrete is placed; then no water is present to dilute and weaken the cement paste of the plastic concrete at the construction joint. Tests have also shown that the practice of placing mortar on the joint ahead of the concrete is not necessary for either strength or impermeability of the joint (Houghton and Hall 1972). The mortar coat, although widely used in the past, is no longer commonly used in mass concrete work. 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.

4.3.5—Selection of equipment for transporting and placing of 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 mix 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, or 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, since 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 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 carry-

(a) Sandblast treatment

(b) High-pressure water-blast treatment

Fig. 4.3.3(a) and (b)—Before and after horizontal construction joint cleanup ing more than 4 yd^3 (3 m^3). Extra care must be taken to assure 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. However, care must be taken to prevent segregation at each transfer point.

4.3.6—Mass concrete may also be transported in dumping rail cars and 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 must 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. 4.3.7. Additional information on placing concrete with conveyors is contained in ACI 304.4R.

4.3.7—Large building foundations and other very large monolithic concrete structures are mass concrete. Availability and job conditions may preclude the use of preferable aggregates larger than $1^1/_2$ in. (37.5 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 upon its ability to successfully place concrete which has been proportioned for mass concrete considerations as defined in Section 2.7, which emphasizes the reduction of heat evolution. It is important that placing capacity be great enough to avoid cold joints and undesirable exposure to extremes of heat and cold at lift surfaces. This is usually accomplished by utilizing many pieces of placing equipment. Additional information on pumping of concrete is contained in ACI 304.2R.

4.3.8—Mass concrete is best placed in successive layers. The maximum thickness of the layer depends upon the ability of the vibrators to properly consolidate the concrete. Six-in. (150-mm) diameter vibrators produce satisfactory results with 4 to 6-in. (100 to 150-mm) nominal maximum size aggregate (NMSA) and less than $1^1/_2$ 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) NMSA 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.

4.3.9—The layer thickness should be an even fraction of the lift height or of the depth of the block. 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 about 5 ft (1.5 m) between the forward edges of successive layers. Placement of the steps is organized so as to expose a minimum of surface and to lessen warming of the concrete in warm weather and reduce the area affected by rain in 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.

4.3.10—Vibration is the key to the successful placement of mass concrete, particularly when the concrete is low slump and contains large aggregate (Tuthill 1953). Ineffectual equipment is more costly to the builder because of a slower placing rate and the hazard of poor consolidation. Vibration must be systematic and should thoroughly cover and deeply penetrate each layer. Partic-

Fig. 4.3.7—Placement of mass concrete by conveyor belt

ular attention must be paid to ensure full vibration where the perimeters of two discharges join, since 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. 4.3.10. To insure proper consolidation, the vibrators should penetrate the lower layer for several inches (50 to 100 mm) and be held in a vertical position and should remain 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 a cubic yard ($^3/_4$ m^3) of concrete may be as much as one minute (80 sec for 1 m^3). Over-vibration of low slump mass concrete is unlikely. 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 and boards should be laid on the surface in sufficient number to prevent deep footprints. Ample and effective vibration equipment must be available and in use during the placement of mass concrete. Anything less should not be tolerated. Specific recommendations for mass concrete vibration are contained in ACI 309R.

Fig. 4.3.10—Consolidation of low slump mass concrete placed by bucket

4.4—Curing

4.4.1—Mass concrete is best cured with water, which provides additional cooling benefit in warm weather. In cold weather, little curing is needed beyond the moisture provided to prevent the concrete from drying during its initial protection from freezing. However, the concrete should not be saturated when it is exposed to freezing. In above-freezing weather when moisture is likely to be lost from the concrete surfaces, mass concrete should be water cured for at least 14 days or up to twice this time if pozzolan is used as one of the cementitious materials. Except when insulation is required in cold weather, surfaces of horizontal construction joints should be kept moist until the wetting will no longer provide beneficial cooling. Curing should be stopped long enough to assure that the joint surface is free of water but still damp before new concrete is placed. The use of a liquid-membrane curing compound is not the best method of curing mass concrete, but in some instances it is the most practical. If used on construction joints, it must be completely removed by sandblasting or waterblasting to prevent reduction or loss of bond.

4.5—Forms

4.5.1—Forms for mass concrete have the same basic requirement for strength, mortar-tightness, accuracy of position, and generally good surface condition as those described in Hurd (1989). Formwork for mass concrete may differ somewhat from other formwork because of the comparatively low height normally required for each lift. 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 upon the methods used and the care exercised in placing concrete adjacent to the form. For this reason, it is recommended that 100 percent of equivalent hydrostatic pressure plus 25 percent for impact be used for design of mass concrete forms.

4.5.2—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 is necessary to avoid spalling concrete around the anchor bolts in the low-early-strength concrete of the lift being stripped of forms, since 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 20 to 40 in. (0.5 to 1.0 m). To facilitate placement by bucket, widely spaced large-diameter, high-tensile-strength ties are required to permit passage of the concrete buckets.

4.5.3—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. This will generally dress up and improve appearance of formed surfaces. When used at the top and bottom of the forms, this can create an effective and 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. Sharp corners of the block otherwise are often damaged and cannot be effectively repaired. Such chamfers also prevent pinching and spalling of joint edges caused by high surface temperatures.

4.5.4—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 continued. Sloping forms are subject to less outward pressure, but uplift should be considered in their anchorage.

4.5.5—A common forming problem for spillway sections of gravity dams is encountered in the sloping and the curved portions of the crest and the bucket. These are the slopes that range from horizontal to about 1.5 to 1.0 vertical at the transition where regular fixed forms can be used. The curved or sloped surfaces are effectively shaped and the concrete thoroughly consolidated by means of temporary holding forms, rather than using screed guides and strikeoff. With no strikeoff involved, the regular mass concrete face mix is as readily used as one with small aggregate, unless a different concrete mix is required on the spillway face for durability reasons. The desired shape is achieved with strong, solidly anchored ribs between which rows of form panels are placed row-on-row upward as the lift space is filled, and removed starting row-on-row at the bottom when the concrete will no longer bulge out of shape but is still responsive to finishing operations (Tuthill 1967). Considerable time and labor are saved by this method and it enables the concrete to be well consolidated by vibration and very accurately shaped and finished.

4.6—Height of lifts and time intervals between lifts

4.6.1—From the standpoint of construction, the higher the lift the fewer the construction joints; with 7.5-ft (2.3-m) lifts there are only two-thirds as many joints as when 5-ft (1.5-m) lifts are used. With regard to hardened concrete temperature histories in cold weather, the shallower the lift the higher the percentage of the total heat of hydration that will escape before the next lift is placed. In hot weather with lean mixes and precooling, the opposite may be true. When lift 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 lift. Hence, with very deep lifts, the internal temperature reached by the concrete is not significantly influenced by the length of time interval between lifts. In such extreme cases, continuous placing in high lifts may be preferable, especially as a means of minimizing joint cleanup, to prevent cracking, or to permit the use of slipforms, e.g., for massive piers.

In large blocks, such as in dam construction, the loss of heat from a lift surface in cold weather does not justify extended exposure. A long exposure of lift surfaces to changes in ambient temperature may initiate cracking. This can defeat an otherwise successful crack-prevention program. Where thermal-control crack-prevention procedures are being used, 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. This is further discussed in Chapter 5.

4.6.2—Control of temperature rise is a design function. Therefore lift heights and placing frequency should be shown on drawings and in specifications. (Refer to Chapter 5). Influencing factors are size and type of massive structure, concrete properties and cement content, prevailing climate during construction and in service, construction schedule and other specified temperature controls. Heights of lifts range from $2^1/_2$ ft (0.75 m) for multiple lifts just above foundations to 5 ft (1.5 m) and $7^1/_2$ ft (2.3 m) in many gravity dams; and to 10 ft (3 m) or more in thin arch dams, piers, and abutments.

4.6.3—High-lift mass concrete construction was adopted by some authorities, particularly in Canada during the 1950s and 1960s, in an attempt to reduce potential leak paths and minimize cracking in dams built in cold and even subzero weather. The procedure is no longer in common usage. 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 of the concrete and the maximum temperature drop to low stable temperatures are approximately equal. For control of cracking most design criteria restrict this maximum drop to 25 to 35 F (14 to 19 C). Design requirements can be met under these conditions by controlling, through mixture proportioning, the adiabatic rise to these levels (Klein, Pirtz, and Adams 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 $7^1/_2$-ft (2.3-m) or even 10-ft (3-m) lifts have been permitted by specifications on several dam projects in recent years.

4.7—Cooling and temperature control

4.7.1—Currently it is common practice to precool mass concrete before placement. Efficient equipment is now available to produce such concrete at temperatures less than 45 F (7 C) in practically any summer weather. The simple expedient of using finely chipped ice instead of 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. To permit maximum use of ice in place of mixing water, fine aggregate should be drained to a water content of not more than 5 percent. Steel aggregate storage bins and aggregate piles should be shaded as illustrated in Fig. 4.7.1(a). Aggregates can be cooled by evaporation through vacuum, by inundation in cold water, by cold air circulation (Roberts 1951; ACI 305R), or by liquid nitrogen. Fig. 4.7.1(b) shows the cooling of coarse ag-

Fig. 4.7.1(a)—Metal cover over drained fine aggregate stock pile to reduce heat absorption

gregate by spraying and inundation with chilled water immediately prior to placing in the batch plant bins.

To obtain full advantage of the low placing temperature, the concrete should be protected from higher ambient temperature conditions during the first few weeks after placement to reduce temperature rise in the concrete and to reduce the thermal differential tending to crack the surface later when much colder ambient conditions may occur. During placement in warm weather, absorption of heat by cold concrete can be minimized by placing at night, by managing placement so that minimum areas are exposed, and, if placement must be done in the sun, by fog spraying the work area. Much can be done during the curing period to prevent heating and to remove heat from the hardening concrete, including use of steel forms, shading, and water curing.

Embedded pipe cooling can be used to control the rise in concrete temperature in restrained zones near foundations when maximum temperatures cannot be limited by other, less expensive cooling measures. Embedded pipe cooling is also normally required to assure at least the minimum opening of contraction joints needed when in dams grouting of joints is necessary.

Aggregate and concrete precooling, insulation, protection from high ambient temperature, and postcooling considerations and recommendations are provided in ACI 207.4R.

Fig. 4.7.1(b)—Cooling coarse aggregate by chilled water spray and inundation

4.8—Grouting contraction joints

4.8.1—With increasingly effective use of cold concrete as placed, and especially when narrow shrinkage slots are left and later filled with cold concrete, some may question whether contraction joint grouting serves much purpose for high thin-arch dams, since a little downstream cantilever movement will bring the joints into tight contact. Nevertheless, grouting relieves later arch and cantilever stresses by distributing them more evenly and it remains general practice to grout contraction joints in such dams.

4.8.2—In recent decades the transverse contraction joints in most gravity dams have not been grouted. It was considered that an upstream waterstop backed up by a vertical drain would prevent visible leakage; that grout filling was unnecessary because there was no transverse stress; and that money would be saved. However, in recent years the appearance of some transverse cracks, generally parallel to the contraction joints, has prompted reconsideration of the grouting of contraction joints in gravity dams. It has been suggested that intermediate cracks can start on the upstream face and be propagated farther into the dam, and sometimes through it, due to the cold temperature and high pressure of deep reservoir water. Its coldness cools the interior concrete at the crack and further opens it. Transverse cracks should be repaired prior to reservoir filling if at all possible. It has been further suggested that if the transverse joints are filled with grout, a surface crack opening somewhere on the upstream face would have effective resistance against propagation and further opening.

4.8.3—Where there is reason to grout contraction joints, the program of precooling and postcooling should be arranged to provide a joint opening of at least 0.04 in. (1 mm) to assure complete filling with grout even though, under special test conditions, grout may penetrate much narrower openings. The grouting system can be designed in such a way as to allow either just one or two grouting operations (when the width of the opening is near its maximum), or several operations, when the first joint filling has to be performed before the maximum opening is reached and there is no provision for postcooling. The U.S. Bureau of Reclamation (1976) Sections 8-9 and 8-10 has described the grouting systems and grouting operations it uses. Silveira, Carvalho, Paterno, and Kuperman (1982) have described a grouting system which employs packers to permit reuse of the piping system.

The use of embedded instrumentation across the joint is the only way to determine with precision the magnitude of the joint opening (Carlson 1979; Silveira, Carvalho, Paterno, and Kuperman 1982).

CHAPTER 5—BEHAVIOR

5.1—Thermal stresses and cracking*

5.1.1—A most important characteristic of mass concrete that differentiates its behavior from that of structural concrete is its thermal behavior. The generally large size of mass-concrete structures creates the potential for significant temperature differentials between the interior and the outside surface of the structure. The accompanying volume-change differentials and restraint result in tensile strains and stresses that may cause cracking detrimental to the structural design. Because concrete has a low thermal conductivity, heat generated within a massive structure can escape only very slowly unless aided artificially. Heat escapes from a body inversely as the square of its least dimension. In ordinary structural construction most of the heat generated by the hydrating cement is rapidly dissipated and only slight temperature differences develop. For example, a concrete wall 6 in. (150 mm) thick can become thermally stable in about $1^1/_2$ hr. A 5-ft (1.5-m) thick wall would require a week to reach a comparable condition. A 50-ft (15-m) thick wall, which could represent the thickness of an arch dam, would require two years. A 500-ft (152-m) thick dam, such as Hoover, Shasta, or Grand Coulee, would take some 200 years to achieve the same degree of thermal stability. Temperature differentials never become large in thin structures and, therefore, thin structures are relatively free from thermal cracking. In contrast, as thickness increases, the uncontrolled interior temperature rise in mass concrete becomes almost adiabatic and this creates the potential for large temperature differentials which, if not accommodated, can impair structural integrity.

5.1.2—In mass concrete, thermal strains and stresses are developed in two ways: from the dissipation of the heat of cement hydration and from periodic cycles of ambient temperature. Since all cements, as they hydrate, cause concrete to heat up to some degree, it is fortunate that the strength and the corresponding cement requirements for mass concrete are usually much less than those for general concrete work; hence, temperature rise is also less. Some reduction in temperature rise can be achieved by (1) the use of minimal cement contents, (2) the partial substitution of pozzolans for cement, and (3) the use of special types of cement with lower or delayed heats of hydration. When the potential temperature rise of a concrete mixture has been reduced to its practical minimum, the temperature drop that causes tensile stress and cracking can be reduced to zero if the initial temperature of the concrete is set below the final stable temperature of the structure by the amount of the potential temperature rise. Theoretically this is possible; however, it is not generally practical except in hot climates. Economy in construction can be gained if the initial temperature is set slightly above this value so that a slight temperature drop is allowed, such that the tensile stresses built up during this temperature drop are less than the tensile strength of the concrete at that time (or such that the tensile strains are less than the tensile strain capacity of the concrete at that time).

5.1.3—Previous chapters describe methods for reducing the initial temperature of concrete, and the benefits of placing cold concrete. It can be seen that if the maximum temperature of the concrete is appreciably above that of the final stable temperature of the mass, volume changes in massive structures will take place continuously for centuries. Since this is intolerable in some structures that depend on fast construction for economy, this excess heat must be removed artificially. The usual method is by circulating a cooling medium in embedded pipes (see 4.7.1).

5.1.4—The behavior of exposed surfaces of concrete is greatly affected by daily and annual cycles of ambient temperature (ACI 305R). At the surface the temperature of concrete responds almost completely to daily variations in air temperature, while at a depth of 2 ft (0.6 m) from the surface, the concrete is affected by only 10 percent of the daily surface temperature variation. The annual ambient temperature cycle affects the concrete at much greater depths. Ten percent of the annual variation in temperature is effective 25 ft (7.6 m) from the surface. It can be seen that the surface is subjected to rather severe tensile strains and stresses caused by temperature changes. Since the interior reacts so much more slowly than the surface, it is as though the surface were completely restrained by the interior concrete. Thus in a location where the surface temperature varies annually by 100 F (59 C) and the concrete is assumed to have a modulus of

*For additional information see Klein, Pirtz, and Adams 1963; Rawhouser 1945; Waugh and Rhodes 1959; U.S. Bureau of Reclamation 1949; U.S. Bureau of Reclamation 1981; and Ross and Bray 1949.

elasticity of 3.0 x 10^6 psi (2.1 x 10^4 MPa) before cracking, the surfaces could be subjected to stresses about 1000 psi (7 MPa) above and below the average. While concrete can quite easily sustain 1000 psi (7 MPa) in compression, its tensile strength is much lower, and cracking would be inevitable. However, because of the rapid deterioration of the temperature differential with distance from the surface, the variation in stress is likewise dissipated rapidly, with the result that surface cracking due to ambient temperature changes originates in and usually is confined to a relatively shallow region at and near the surface. In a massive structure such as a dam, where a regular and orderly construction schedule is being followed, the surface concrete, although superficially cracked by ambient temperature cycles, can protect the structural integrity of the concrete below it. Where there is an interruption to the orderly construction schedule and time intervals between lifts become overly extended, lift surface cracking may become deep and require treatment to prevent propagation into subsequent placements.

5.1.5—The above statements about the effect of variations in surface temperature on cracking explain why form stripping at times of extreme contrast between internal and ambient temperatures will inevitably result in surface cracking. This phenomenon has been termed “thermal shock” and occurs when forms that act as insulators are removed on an extremely cold day. Modern steel forms that allow the surface temperature of the concrete to more nearly correspond to that of the air reduce this differential temperature somewhat. However, they are open to the objection that the thermal shock may be felt from low temperatures at an early age through the form into the concrete. Either a dead airspace or insulation should be provided to protect concrete surfaces where steel forms are used in cold weather. Insulation requirements and the age for form stripping to avoid cracking the surface depend on the air temperature and the strength of the concrete. Requirements for protection in freezing weather are given in ACI 306R.

5.1.6—Any change in temperature in a partially restrained block will cause a corresponding change in stress (Rawhouser 1945). At any point within a dam, the total thermal stress is the sum of the structural stress produced by the average temperature change within the entire volume and the stress caused by the difference between the average temperature and the point temperature. For example, one percent of the annual surface temperature will be felt at a depth 50 ft (15 m) from the surface, thus producing a volume and stress change throughout the block. In designing an arch dam, the total temperature distribution should be considered.

5.2—Volume change

5.2.1—The tables of Chapter 3 list properties affecting volume change for a number of dams. It will be noted from Table 3.5.1 that the values for drying shrinkage, autogenous volume change, and permeability are results of tests on quite small specimens and, except for the permeability specimens, none contained mass concrete. However, the values given can be used as a guide to the actual behavior of mass concrete in service. First, it can be seen that the permeability of these low-cement-content mixtures is very small, a fraction of a foot per year. As a working guide to the behavior of concrete, it can be considered that concrete gives up water with great reluctance, but accepts it at a free surface fairly easily. Thus, at a surface exposed to air, the surface is quite capable of drying out, while the concrete farther from the surface has lost little, if any, of its moisture content (Carlson 1937).

Previous paragraphs have discussed temperature differential as a cause of surface cracking. Another common cause of surface cracking is drying at the surface. It can be seen from Table 3.5.1 that the concrete exhibiting minimum drying shrinkage has a volume change expressed in single dimension shrinkage of roughly 300 millionths. If one considers a drying surface concrete completely restrained by a fully-saturated interior concrete, it will be seen that tensile stresses in the surface concrete can exceed 1000 psi (7 MPa). Concrete cannot withstand such a tensile stress, and the result is an extensive pattern of surface cracking. Exactly as in the case of thermal cracking at the surface, these cracks will extend inward a short distance and disappear in the region of moisture equilibrium. ACI 209R discusses further the prediction of shrinkage in concrete.

5.2.2—Whenever a flat surface of concrete is being finished as in a dam roadway, a spillway apron surface, or a power plant floor, care must be taken to avoid the conditions causing what is known as “plastic shrinkage cracks.” This cracking occurs under extreme drying conditions, when water evaporates from the upper surface of the unhardened concrete faster than it reaches the surface by water gain. Even as the concrete is setting, wide cracks appear, often as parallel tears, across the entire finished surface. These can be prevented in extreme drying weather by shading the area of finishing operations, by providing barriers against the movement of the air, by fog spraying, by surface sealing, or by any other means available to prevent rapid surface moisture evaporation.

5.3—Heat generation

5.3.1—Since one of the main problems of mass concrete construction is the necessity for controlling the heat entrapped within it as the cement hydrates, a short statement will be given here of the thermal properties and mathematical relationships that enable the engineer to estimate rapidly the degree of temperature control needed for a particular application.

Both the rate and the total adiabatic temperature rise differ among the various types of cement. Fig. 5.3.1 shows adiabatic temperature rise curves for mass concretes containing 376 lb/yd^3 (223 kg/m^3) of various types of cement with a 4-$^1/_2$-in. (114 mm) maximum size aggregate. Values shown are averaged from a number of tests; individual cements of the same type will vary considerably from the average for that type. As might be expected, high-early-strength cement, Type III, is the fastest heat generator and gives the highest adiabatic temperature rise. Type IV, or low-heat cement, is not only the slowest heat generator, but gives the lowest total temperature rise. Since the cement is the active heat producer in a concrete mix, the temperature rise of concretes with cement contents differing from 376 lb/yd^3 (223 kg/m^3) can be estimated closely by multiplying the values

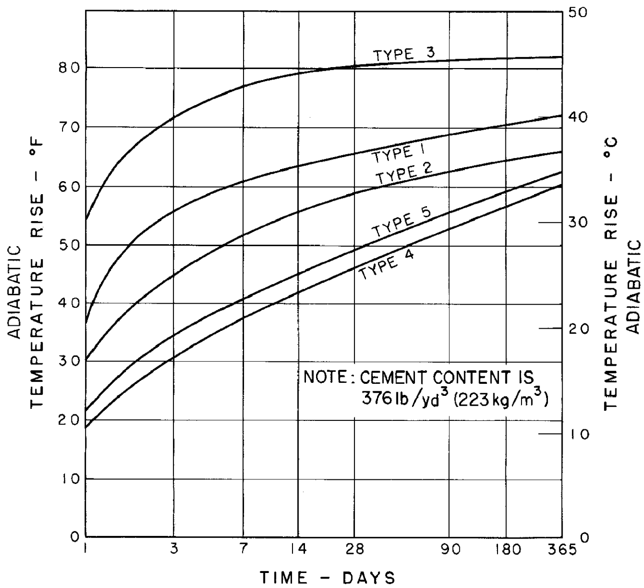


Fig. 5.3.1—Temperature rise of mass concrete

shown on the curves by a factor representing the proportion of cement.

5.3.2—When a portion of the cement is replaced by a pozzolan, the temperature rise curves are greatly modified, particularly in the early ages. While the effects of pozzolans differ greatly, depending on the composition and fineness of the pozzolan and cement used in combination, a rule of thumb that has worked fairly well on preliminary computations has been to assume that pozzolan produces only about 50 percent as much heat as the cement that it replaces.

5.3.3—In general, chemical admixtures affect heat generation of concrete only during the first few hours after mixing and can be neglected in preliminary computations. However, in studies involving millions of cubic yards of concrete, as in a dam, the above remarks should be applied only to preliminary computations, and the adiabatic temperature rise should be determined for the exact mixture to be used in the mass concrete starting at the proposed placing temperature.

5.3.4—The characteristic that determines the relative ability of heat to flow through a particular concrete is its thermal diffusivity which is defined as:

$$h^2 = \frac{K}{C\rho}$$

where

h^2 = diffusivity, ft²/hr (m²/hr)

K = conductivity, Btu/ft·hr·F (kJ/m-hr-C)

C = specific heat, Btu/lb·F (kJ/kg-C)

ρ = density of the concrete, lb/ft³ (kg/m³)

The value of diffusivity is largely affected by the rock type used in the concrete. Table 5.3.4 shows diffusivities for concrete made with different rock types. The higher the value of diffusivity, the more readily heat will move through the concrete. If the rock type is not known, an average value of diffusivity can be taken as 1.00 ft²/day (0.093 m²/day) or 0.042 ft²/hr (3.9 x 10^{-3} m²/hr), although as can be seen from Table 5.3.4, the value of diffusivity can vary substantially from this average value.

5.3.5—Mass concrete can be affected by heat dissipated to or absorbed from its surroundings (Burks 1947). If the external temperature variation can be considered to be expressed as a sine wave, and if, as in a dam, the body of concrete is sufficiently thick so that the internal temperature variation is negligible compared to that of the exposed face, the range of temperature variation any distance in from the surface can be computed from

$$\frac{R_x}{R_o} = e^{-x\sqrt{\pi/h^2\gamma}}$$

where

R_x = temperature range at distance x from surface

R_o = temperature range at the surface (x = 0)

e = base of natural logarithms (= 2.718)

x = distance from surface, ft (m)

h^2 = diffusivity, ft²/hr (m²/hr) as defined in 5.3.4

γ = period of the cycle of temperature variation in days

For concrete with a diffusivity of 1 ft²/day (0.093 m²/day), or 0.042 ft²/hr (3.9 x 10^{-3} m²/hr) the penetration of the daily and the annual temperature cycles is as shown in Fig. 5.3.5.

5.4—Heat dissipation studies

5.4.1—Studies of the dissipation of heat from bodies of mass concrete can be accomplished by the use of charts and graphs, by hand computation, or with finite element computer programs.

When the body to be analyzed can be readily approximated by a known geometrical shape, charts are available for the direct determination of heat losses. For instance, Fig. 5.4.1 can be used to determine the loss of heat in hollow and solid cylinders, slabs with one or two faces exposed, or solid spheres. The application of the values found on these graphs can easily be made to a wide variety of problems such as the cooling of dams or thick slabs of concrete, the cooling of concrete aggregates, artificial cooling of mass concrete by use of embedded pipes, and the cooling of bridge piers. The following five examples are typical concrete cooling problems which can be solved by

Table 5.3.4— Diffusivity and rock type

Coarse aggregate	Diffusivity of concrete, ft²/day (m²/day)	Diffusivity of concrete ft²/hr (m²/hr 10^{-3})
Quartzite	1.39 (0.129)	0.058 (5.4)
Limestone	1.22 (0.113)	0.051 (4.7)
Dolomite	1.20 (0.111)	0.050 (4.6)
Granite	1.03 (0.096)	0.043 (4.0)
Rhyolite	0.84 (0.078)	0.035 (3.2)
Basalt	0.77 (0.072)	0.032 (3.0)

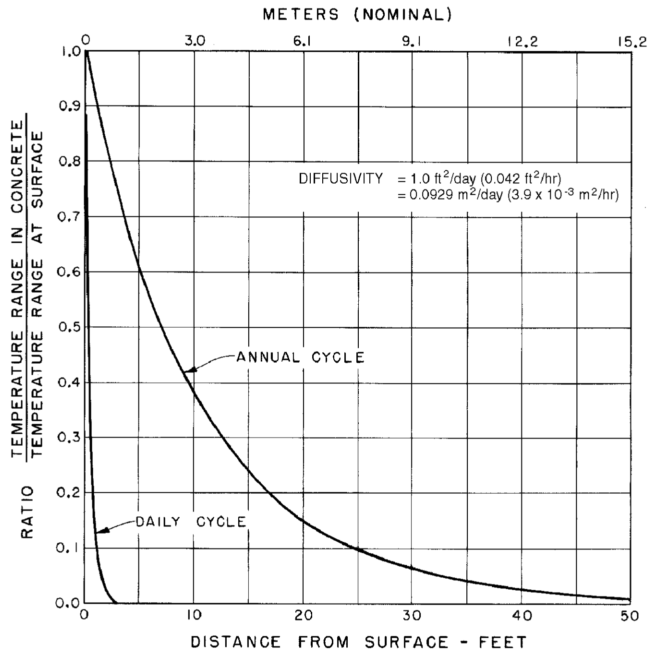


Fig. 5.3.5—Temperature variation with depth

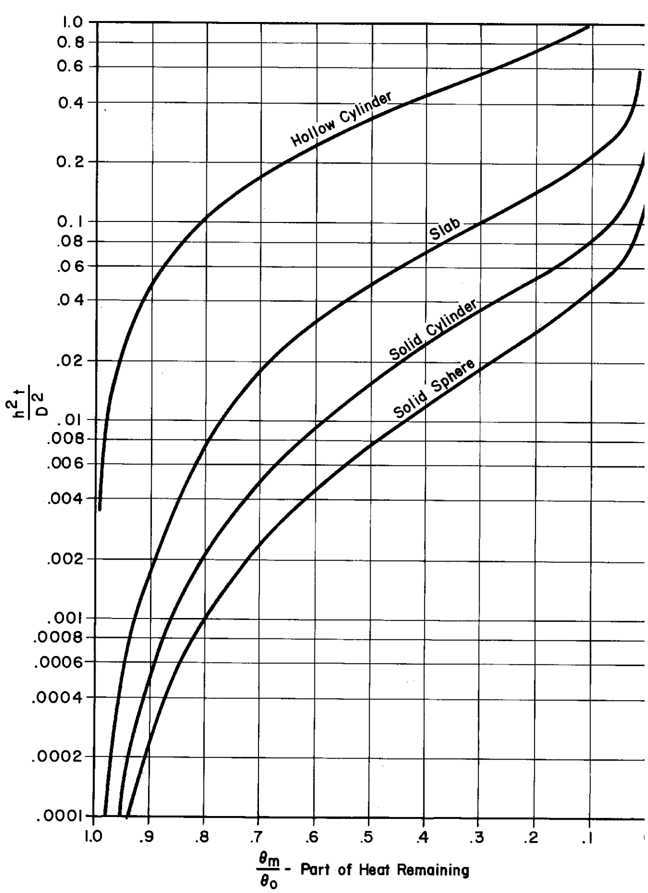


Fig. 5.4.1—Heat loss from solid bodies

use of Fig. 5.4.1. For simplicity of presentation the examples are in inch-pound units only; Appendix A presents the examples worked in SI (metric) units. In the examples below and Fig. 5.4.1, the following notation is followed:

t = time, days
h^2 = diffusivity, ft^2 per day (m^2/day)
D = thickness of concrete section, ft (m)
θ_o = initial temperature difference between concrete and ambient material, F (C)
θ_m = final temperature difference between concrete and ambient material, F (C)

Example 1 (See Appendix A for examples worked in SI units)

At a certain elevation an arch dam is 70 ft thick and has a mean temperature of 100 F. If exposed to air at 65 F, how long will it take to cool to 70 F? Assume h^2 = 1.20 ft^2/day.

Initial temperature difference, θ_o = 100 - 65 = 35 F

Final temperature difference, θ_m = 70 - 65 = 5 F

The portion of the original heat remaining is

$$\frac{\theta_m}{\theta_o} = \frac{5}{35} = 0.142$$

From Fig. 5.4.1, using the slab curve

$$\frac{h^2 t}{D^2} = 0.18$$

Then

$$t = \frac{0.18D^2}{h^2} = \frac{0.18(70)^2}{1.20} = 740 \text{ days}$$

Example 2

A mass concrete bridge pier has a horizontal cross section of 25 x 50 ft, and is at a mean temperature of 80 F. Determine the mean temperature at various times up to 200 days if the pier is exposed to water at 40 F and if the diffusivity is 0.90 ft^2/day. For a prismatic body such as this pier, where heat is moving towards each of four pier faces, the part of original heat remaining may be computed by finding the part remaining in two infinite slabs of respective thickness equal to the two horizontal dimensions of the pier, and multiplying the two quantities so obtained to get the total heat remaining in the pier. For this two-dimensional use, it is better to find for various times the heat losses associated with each direction and then combine them to find the total heat loss of the pier.

Initial temperature difference, θ_o = 80 - 40 = 40 F

For the 25-ft dimension

$$\frac{h^2 t}{D^2} = \frac{0.90t}{(25)^2} = 0.00144t$$

and for the 50-ft dimension

$$\frac{h^2 t}{D^2} = \frac{0.90t}{(50)^2} = 0.00036t$$

Then calculate numerical values of 0.00144t and 0.00036t for times from 10 to 200 days. See Table 5.4.1. These values can be used with Fig. 5.4.1 to obtain the θ_m/θ_o ratios for both 25-ft and 50-ft slabs. The product of these ratios indicates the

Table 5.4.1— Calculations for Example 2

Time, days	$\left(\frac{h^2t}{D^2}\right)_{25}$ = 0.00144t	$\left(\frac{h^2t}{D^2}\right)_{50}$ = 0.00036t	$\left(\frac{\theta_m}{\theta_o}\right)_{25}$ =	$\left(\frac{\theta_m}{\theta_o}\right)_{50}$ =	$\left(\frac{\theta_m}{\theta_o}\right)_{pier}$ =	θ_m	Temperature, F
10	0.0144	0.0036	0.73	0.87	0.64	26	66
20	0.0288	0.0072	0.61	0.80	0.49	20	60
30	0.0432	0.0108	0.53	0.77	0.41	16	56
40	0.0576	0.0144	0.46	0.73	0.34	14	54
60	0.0864	0.0216	0.35	0.67	0.23	9	49
100	0.144	0.036	0.19	0.57	0.11	4	44
200	0.288	0.072	0.05	0.40	0.02	1	41

heat remaining in the pier, and can be used to calculate the final temperature difference θ_m. The values for θ_m are added to the temperature of the surrounding water to obtain mean pier temperatures at various times up to 200 days as shown on Table 5.4.1.

Example 3

Granite aggregate at an initial temperature of 90 F is to be precooled in circulating 35 F water for use in mass concrete. The largest particles can be approximated as 6-in.-diameter spheres. How long must the aggregate be immersed to bring its mean temperature to 40 F?

For granite, h^2 = 1.03 ft^2/day

Initial temperature difference, θ_o= 90 - 35 = 55 F

Final temperature difference, θ_m = 40 - 35 = 5 F

$$\frac{\theta_m}{\theta_o} = \frac{5}{55} = 0.09$$

From Fig. 5.4.1, for θ_m/θ_o = 0.09,

$$\frac{h^2t}{D^2} = 0.050$$

$$t = \frac{(0.050)(6/12)^2}{1.03} = 0.012 \text{ days}$$

or approximately 17 min.

Example 4

A 50-ft diameter circular tunnel is to be plugged with mass concrete with a diffusivity of 1.20 ft^2/day. The maximum mean temperature in the concrete is 110 F, and the surrounding rock is at 65 F.

Without artificial cooling, how long will it take for the temperature in the plug to reach 70 F, assuming the rock remains at 65 F?

Initial temperature difference, θ_o = 110 - 65 = 45 F

Final temperature difference, θ_m = 70 - 65 = 5 F

$$\frac{\theta_m}{\theta_o} = \frac{5}{45} = 0.11$$

From Fig. 5.4.1, for a solid cylinder,

$$\frac{h^2t}{D^2} = 0.080$$

$$t = \frac{(0.080)(50)^2}{1.20} = 170 \text{ days}$$

Example 5

A closure block of concrete initially at 105 F is to be cooled to 45 F to provide a joint opening of 0.025 in. prior to grouting contraction joints. How long will it take to cool the mass by circulating water at 38 F through cooling pipes spaced 4 ft 6 in. horizontally and 5 ft 0 in. vertically. Assume concrete to be made with granite aggregate having a diffusivity of 1.03 ft^2/day.

Cross section handled by each pipe is (4.5)(5.0) = 22 ft^2.

The diameter of an equivalent cylinder can be calculated from 22 = $\pi D^2/4$

$$D^2 = \frac{(4)(22)}{\pi} = 28 \text{ ft}^2$$

$$D = 5.3 \text{ ft}$$

Initial temperature difference, θ_o = 105 - 38 = 67 F

Final temperature difference, θ_m = 45 - 38 = 7 F

$$\frac{\theta_m}{\theta_o} = \frac{7}{67} = 0.10$$

Referring to Fig. 5.4.1 and using the curve for the hollow cylinder (since cooling is from within cross section), for the calculated value of θ_m/θ_o,

$$\frac{h^2t}{D^2} = 1.0$$

$$t = \frac{(1.0)(28)}{1.03} = 27 \text{ days}$$

About the same results can be achieved with greater economy if the natural cold water of the river is used for part of the cooling. Control of the rate of cooling must be exercised to prevent thermal shock, and in many cases postcooling is conducted in two stages.

Assume river water is available at 60 F, cool to 68 F, and then switch to refrigerated water at 38 F. How much time will be taken in each operation, and what is the total cooling time?

For initial cooling, θ_o = 105 – 60 = 45 F and θ_m = 68 – 60 = 8 F

$$\frac{\theta_m}{\theta_o} = \frac{8}{45} = 0.18$$

From Fig. 5.4.1, for a hollow cylinder

$$\frac{h^2 t}{D^2} = 0.75$$

Therefore

$$t = \frac{(0.75)(28)}{1.03} = 20 \text{ days}$$

For final cooling, θ_o = 68 - 38 = 30 F and θ_m = 45 - 38 = 7 F

$$\frac{\theta_m}{\theta_o} = \frac{7}{30} = 0.23$$

$$\frac{h^2 t}{D^2} = 0.67$$

$$t = \frac{(0.67)(28)}{1.03} = 18 \text{ days}$$

Total time is 20 + 18 = 38 days, but of this, the time for using refrigeration has been cut by one-third.

5.4.2—For graphical solutions, Figs. 5.4.2(a), 5.4.2(b) and 5.4.2(c) can be used for the determination of all the characteristics of an artificial cooling system for mass concrete. Fig. 5.4.2(a) can be used for the determination of the actual cooling accomplished in a given number of days with a given pipe spacing and flow of coolant. Fig. 5.4.2(b) gives more detail on the cooling of the mass concrete by determining the temperature at various points along the length of the cooling coil. Fig. 5.4.2(a) can be used to determine the temperature rise of the coolant in the pipe.

Using Fig. 5.4.2(a), one can determine θ_m/θ_o for a given system of 1 in. OD cooling tubes embedded in concrete of known diffusivity. This use is illustrated on the figure.

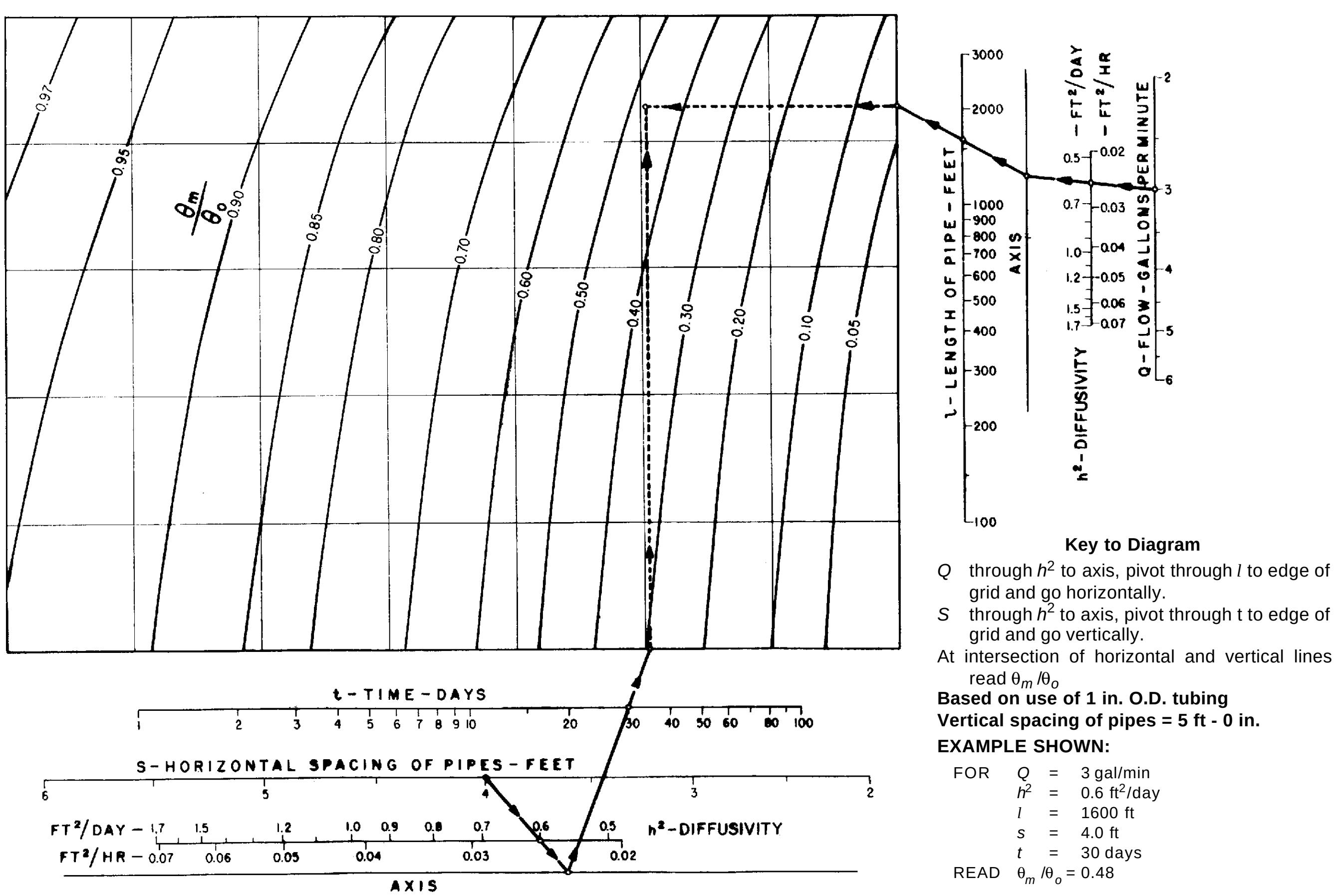
Fig. 5.4.2(a) can also be used to determine how many days of cooling flow will be required to achieve a desired θ_m/θ_o. Using the figure to solve Example 5 of Section 5.4.2, for which it is given that

Q = 5 gal/min,
h^2 = 1.03 ft^2/day,
S = 4.5 ft, and
θ_m/θ_o = (45 - 38) ÷ (105 - 38) = 0.104

and assuming that tube length is 200 ft and cooling water flow in each tube is 5 gal/min, one can read that 35 days will be required to accomplish the required temperature reduction. If tube length is 600 ft, 40 days will be required, according to Fig. 5.4.2(a).

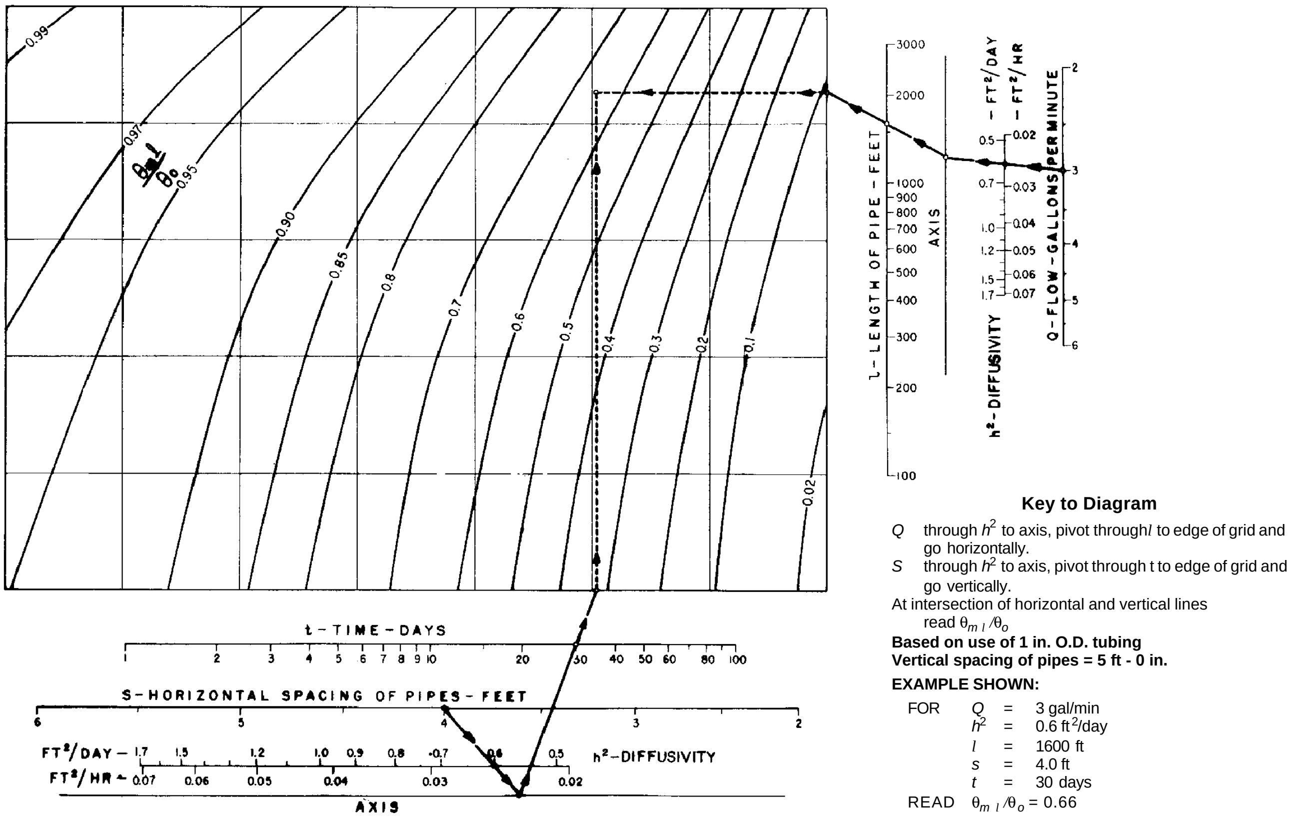
The difference in results between the method using Fig. 5.4.1 and that using Fig. 5.4.2 is due to the fact that the latter takes into account the variation in temperature of the cooling water along the pipe as it extracts heat from the concrete.

5.4.3—All the foregoing methods are only approximations; in the usual case hydration and cooling go on simultaneously. For this more general case in which it is necessary to determine actual temperature gradients, Schmidt's meth-



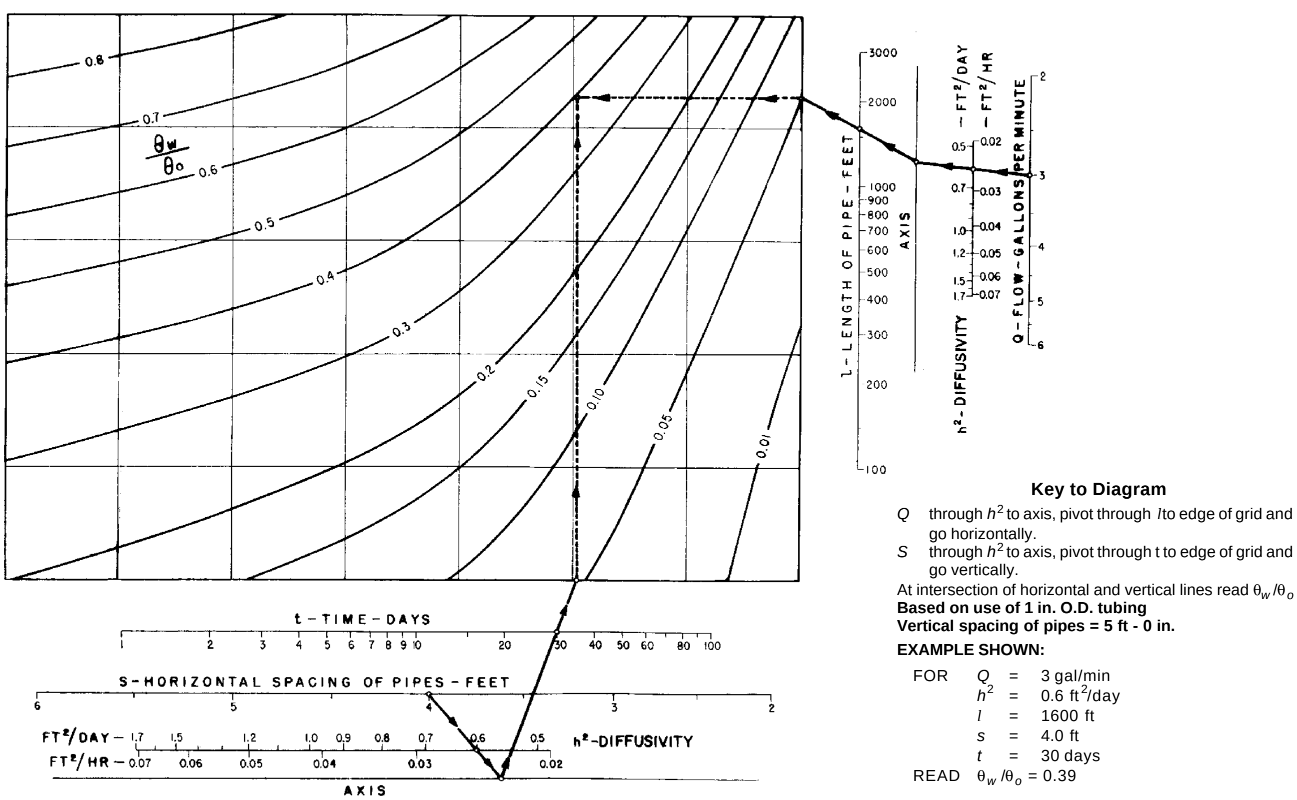
Note: 1.00 mm = 3.28 ft; 1.00 m^3/min = 264 U.S. liquid gal/min; 1.00 m^2/hr = 10.8 ft^2/hr; 1.00 m^2/day = 10.8 ft^2/day

Fig. 5.4.2(a)—Ratio of final mean temperature difference to initial temperature difference θ_m/θ_o, F/F (C/C) *(Rawhouser 1945)*



Note: 1.00 mm = 3.28 ft; 1.00 m³/min = 264 U.S. liquid gal/min; 1.00 m²/hr = 10.8 ft²/hr; 1.00 m²/day = 10.8 ft²/day

Fig. 5.4.2(b)—Ratio of final mean temperature difference at a given length from the inlet to initial temperature difference θ_m/θ_o, F/F (C/C)



Note: 1.00 mm = 3.28 ft; 1.00 m³/min = 264 U.S. liquid gal/min; 1.00 m²/hr = 10.8 ft²/hr; 1.00 m²/day = 10.8 ft²/day

Fig. 5.4.2(c)—Ratio of temperature rise of water in cooling pipes to initial temperature difference θ_m/θ_o, F/F (C/C) od (Rawhouser 1945) has proved of immense value. The concept and application is so simple that it can be performed quite easily with a desk calculator, and yet for complicated cases can easily be programmed for computer application. Without going into its derivation, it can be said that Schmidt's method is based on the theorem that if the body under question is considered to be divided into a number of equal elements, and if a number of physical limitations are satisfied simultaneously, the temperature for a given increment at the end of an interval of time is the average of the temperature of the two neighboring elements at the beginning of that time interval. The necessary physical relationship is

$$\Delta t = \frac{(\Delta x)^2}{2h^2}$$

where Δt is the time interval, Δx is the length of element, and h^2 is the diffusion constant. Units of Δt and Δx must be consistent with units in which h^2 is expressed. Stated mathematically, θ_p, θ_q, and θ_r are the temperatures of three successive elements at time t, then at time t_2

$$\theta_q + \Delta\theta_q = \frac{(\theta_p + \theta_r)}{2}$$

The universal applicability of Schmidt's method is such that it can be extended to cases of two-dimensional and three-dimensional heat flow. For the two-dimensional case the numerical constant 2 is replaced by 4, and the averaging must take into account temperatures on four sides of the given element. For the three-dimensional case, the constant 2 is replaced by the number 6 and the averaging must be carried on for six elements surrounding the cubic element in question. The following example demonstrates the use of Schmidt's method in a practical problem.

Example 6 (See Appendix A for this example worked in SI units).

Determine temperature rise throughout two 6-ft lifts of mass concrete placed at two-day intervals. The concrete contains 376 lb/yd^3 of Type II cement and has a diffusivity of 1.00 ft^2/day. Take the space interval as 1.0 ft.

Then the time interval needed for the temperature at the center of the space to reach a temperature which is the average of the temperatures of the two adjacent elements is

$$\Delta t = \frac{(\Delta x)^2}{2h^2} = \frac{1}{(2)(1.00)} = 0.5 \text{ day}$$

In Table 5.4.3(a) the adiabatic temperature rise (above the temperature of concrete when it was placed) in 0.5-day intervals for a 3-day investigation is taken from Fig. 5.3.1 (except that the temperature rise at 0.5-day age is estimated). The change in temperature $\Delta\theta$ is determined by subtracting the temperature at any time interval from that of the preceding time interval.

In the tabular solution, Table 5.4.3(b), the space interval of 1.0 ft divides each lift into six elements or stations. Boundaries such as rock surface, construction joints, and exposed surfaces must be clearly defined. Note that the adiabatic temperature rise at the rock surface is taken as just one-half of the concrete rise since the rock is not generating heat. At a construction joint the rise is the average of the two lifts, which are generating heat at different rates at any given time. At the exposed surface the adiabatic rise is zero since the heat is dissipated as quickly as it is generated from the concrete below. Note that in the computation above two steps are required to produce the temperature at the end of the half-day period; the first step averages the adjacent temperatures, and the second step adds the adiabatic temperature rise of the concrete.

Normally where there are several stations considered in each lift, the temperature distribution within the lift at any given time can be obtained with sufficient accuracy by calculating only half of the points at any one time, as shown in the tabulated solution. With the use of computers, the calculations of heat and induced-thermal stresses can be easily determined using the finite element method (Wilson 1968; Polivka and Wilson 1976). Thermal gradients may also be determined as part of a wider scope 2-D or 3-D nonlinear, incremental

Table 5.4.3(a)— For Example 6, adiabatic temperature increments read from Table 5.3.1

Time, days	Adiabatic temperature rise above placing temperature θ, F (read from Fig. 5.3.1)	Δθ
0	0	
0.5	20	20
1	31	11
1.5	37	6
2	40	3
2.5	42.5	2.5
3	44.5	2.0

Table 5.4.3(b)— For Example 6, calculated temperature rise in concrete above placing temperature, F

Distance above ground, ft	Time t, days											
	0	0.5	1		1.5		2.0		2.5		3	
	$\Delta\theta_1$ = 20F		$\Delta\theta_1$ = 11F		$\Delta\theta_1$ = 6F		$\Delta\theta_1$ = 3F		$\Delta\theta_2$ = 20F $\Delta\theta_1$ = 2.5F		$\Delta\theta_2$ = 11F $\Delta\theta_1$ = 2F	
12								0	0	0		
11								0			10	21
10								0	0	20		
9								0			20	31
8								0	0	20		
7								0			20.4	31.4
6	0	0			0	0		0	9.5	20.7		
5	0	20	10	21			16	19			25.4	27.4
4	0	20			26	32			27.6	30.1		
3	0	20	20	31			33.2	36.2			32.7	34.7
2	0	20			28.5	34.5			32.8	35.3		
1	0	20	15	26			26.5	29.5			28.2	30.2
0	0	10			15.5	18.5			20.0	21.2		
-1	0	0	5	5			10.5				13.5	
-2	0	0			2.5	2.5			5.8			
-3		0	0				1.2				3.2	
-4					0				0.6			
-5							0				0.3	
-6									0			
											0	

Note that in the computation above two steps are required to produce the temperature at the end of the half-day period: the first step averages the adjacent temperatures, and the second step adds the adiabatic temperature rise of the concrete.

structural analysis. Ordinarily used only for very complex mass concrete structures, this method of analysis can evaluate complex geometry of a structure, nonlinear behavior of concrete, structure interaction with the foundation, fill, or other elements such as a reservoir, the effects of sequential construction, thermal gradients, added insulation, and surface and gravity forces (Corps of Engineers 1994).

5.5—Instrumentation

5.5.1—Factors or quantities that are often monitored in mass concrete dams and other massive structures include structural displacements, deformations, settlement, seepage, piezometric levels in the foundation, and uplift pressures within the structure. A wide variety of instruments can be used in a comprehensive monitoring program. An instrumentation program at a new dam may cost from about 1 to as high as 3 percent of the total construction cost of the dam, depending on the complexity of instrumentation requirements. Instruments installed in mass concrete to date in the United States have been primarily of the unbonded resistance-wire or Carlson-type meter, although a wide variety of instruments is being incorporated in current projects. The U.S. Bureau of Reclamation discussed structural behavior measurement practices (1976), and prepared a concrete dam instrumentation manual (1987). The U.S. Army Corps of Engineers prepared an engineer manual on instrumentation (1980). Some of the instruments available for use are:

Hydrostatic Pressure Measuring Devices—These are generally piezometers, operating either as a closed or open system, or closed system Bourdon-type pressure monitoring systems. Closed system piezometers consist of vibrating-wire units or Carlson-type devices, while open system devices used are commonly called observation wells. A variation of the closed system unit is the well or pipe system, which is capped so that a Bourdon-type gauge may be used for directly reading water pressure. Some similar systems use pressure transducers rather than Bourdon gauges to measure the pressure. Other types of piezometers are available but have not been used in concrete dams. These other types include hydrostatic pressure indicators, hydraulic twin-tube piezometers, pneumatic piezometers, porous-tube piezometers, and slotted-pipe piezometers.

Pressure or Stress Measuring Devices—Four types have been used: Gloetzl cell, Carlson load cell, vibrating-wire gauges, and flat jacks. The Gloetzl cell operates hydraulically to balance (null) a given pressure, while the Carlson load cell uses changing electrical resistance due to wire length changes caused by applied pressure. The vibrating-wire gauge, a variation of the Carlson cell, measures the change in vibration frequency caused by strain in a vibrating wire. The flat jacks use a Bourdon-tube gauge to measure pressures.

Seepage Measurement Devices—Commonly used seepage monitoring devices include quantitative devices that include weirs, flowmeters, Parshall flumes, and calibrated catch containers. Flowmeters and pressure transducer devices are also sometimes used to determine quantity of flow in a pipe or open channel.

Internal Movement Measuring Devices—These are used to obtain measurements of relative movements between the structure and the abutments and/or foundations. The devices consist of essentially horizontal and vertical measurements, using calibrated tapes, single-point and multi-point borehole extensometers, joint meters, plumblines, dial gauge devices, Whittemore gauges, resistance gauges, tilt meters, and inclinometer/deflectometers. Strain meters and "no-stress" strain devices may also be used for measuring internal movements.

Surface Movement Measuring Devices—External vertical and horizontal movements are measured on the surface of structures to determine total movements with respect to a fixed datum located off the structure. Reference points may be monuments or designated points on a dam crest, on the upstream and downstream faces, at the toe of a dam, or on appurtenant structures. Both lateral, or translational, and rotational movements of the dam are of interest. Surface movements are usually observed using conventional level and position surveys. The position surveys may be conducted using triangulation, trilateration, or collimation techniques. Individual measurement devices include levels, theodolites, calibrated survey tapes, EDM (electronic distance measuring) devices, and associated rods, targets, etc.

Vibration Measuring Devices—Various commercially available instruments include the strong motion accelerograph, peak recording accelerograph, and others.

5.5.2—Unbonded resistance-wire or Carlson-type meters include strain meters, stress meters, joint meters, deformation meters, pore pressure cells, and reinforcement meters. In each of these devices, two sets of unbonded steel wires are so arranged that when subjected to the action to be measured, one set increases in tension, while the other decreases. A test set, based upon the Wheatstone Bridge, measures resistance and resistance ratios from which the temperature and the strain and stress can be determined. These instruments embedded in fresh concrete are relatively durable in service, provide a stable zero reading, maintain their calibration, and are constructed so as to be dependable for a long time.

5.5.3—To properly monitor the performance of a mass concrete structure, it is often necessary to collect instrumentation data over extended periods. It is important that the monitoring equipment be as simple, rugged, and durable as possible and be maintained in satisfactory operating condition. The instruments must be rugged enough to be embedded in fresh concrete. When measuring strain, in particular, the instruments must be at least three times the length of the largest particle in the fresh concrete. Since they contain electrical-sensing elements, they must not only be waterproof, but all material must be resistant to the alkalies in concrete. The necessity of maintaining proper operational characteristics creates many problems. Even a simple surface leveling point may be subject to damage by frost action, traffic, and maintenance operations on the crest, or vandalism. Observation wells and most piezometers can be damaged by frost action, caving, corrosion of material used for casing, loss of measuring equipment in the hole, and by vandals dropping rocks into the holes. Unless special precautions are taken, the average life of installations of these types may be significantly reduced. To minimize damage, the tops of measuring points and wells should be capped and locked, and should be as inconspicuous and close to the surrounding surface as possible. Locations of installations should not be immediately adjacent to roads, trails, or water channels, and non-corrosive material should be used wherever possible.

Concrete surfaces may be subjected to excessive stresses and cracking that will make meaningless stress or strain measurements obtained from surface-mounted instrumentation. Reliable measurements of strain and stress must come from electrical measuring instruments embedded far enough from the surface to avoid the effects of daily temperature cycles. Embedded instruments are generally accessed by means of conducting cables leading to convenient reading stations located in dam galleries or at the surface of other mass concrete structures.

If certain types of piezometer tubing are used, there are certain microbes that can live and proliferate within the tubes unless the water in the system is treated with a biological inhibitor. Some antifreeze solutions previously placed in systems develop a floc that results in plugging of the tubes. Also, in certain environments, material in some gauges may corrode and render them useless.

Many devices are removable and many be calibrated on a regular basis. However, most instrumentation is fixed in place and not repairable when damage or malfunctioning is discovered. Fixed devices can generally only be replaced from the surface by devices installed in drilled holes and are, therefore, usually not replaceable. Other devices, such as surface monuments, are replaceable to some extent.

5.5.4—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. For credibility, enough instruments 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.

5.5.5—It is suggested that the reader review Chapter 3 for a reexamination of the scope of laboratory studies that are necessary for a meaningful interpretation of data obtained from an embedded instrument program. Instrumentation should be part of the design and construction of any mass concrete structure wherever it can be foreseen that a future question may arise concerning the safety of the structure. Also, preparations essential for an accurate evaluation of the instrumentation results should have been made through long-term, laboratory-sample studies to determine progressive age relationships for properties of the actual project concrete.

CHAPTER 6—REFERENCES

6.1—Specified and recommended references

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designation. The documents listed were the latest effort at the time this document was revised. Since some of these documents are revised frequently, the user of this document should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Concrete Institute

116R Cement and Concrete Terminology
201.2R Guide to Durable Concrete
207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete
207.4R Cooling and Insulating Systems for Mass Concrete
207.5R Roller Compacted Concrete
209R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
210R Erosion Resistance of Concrete in Hydraulic Structures
211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
212.3R Chemical Admixtures for Concrete
221R Guide for Use of Normal Weight Aggregates in Concrete
224R Control of Cracking in Concrete Structures
226.1R Ground Granulated Blast-Furnace Slag as a Cementitious Constituent in Concrete
226.3R Use of Fly Ash in Concrete
304R Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete
304.2R Placing Concrete by Pumping Methods
304.4R Placing Concrete with Belt Conveyors
305R Hot Weather Concreting
306R Cold Weather Concreting
309R Guide for Consolidation of Concrete

ASTM

C 94 Standard Specification for Ready-Mixed Concrete
C 125 Standard Definitions of Terms Relating to Concrete and Concrete Aggregates
C 150 Standard Specification for Portland Cement
C 260 Standard Specification for Air-Entraining Admixtures for Concrete
C 494 Standard Specification for Chemical Admixtures for Concrete
C 595 Standard Specification for Blended Hydraulic Cements
C 618 Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
C 684 Standard Method of Making, Accelerated Curing, and Testing of Concrete Compression Test Specimens
C 989 Standard Specification for Ground Iron Blast-Furnace Slag for Use in Concrete and Mortars

These publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

ASTM
100 Barr Harbor Drive
West Conshohocken, PA 19428

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APPENDIX—METRIC EXAMPLES

Example A-1

At a certain elevation an arch dam is 21.3 m thick and has a mean temperature of 38 C. If exposed to air at 18 C, how long will it take to cool to 21 C? Assume $h^2 = 0.111$ m^2 per day.

Initial temperature difference, $\theta_o = 38 - 18 = 20$ C
Final temperature difference, $\theta_m = 21 - 18 = 3$ C
The portion of the original heat remaining is

$$\frac{\theta_m}{\theta_o} = \frac{3}{20} = 0.15$$

From Fig. 5.4.1 using the slab curve, the value of h^2t/D^2 corresponding to $\theta_m/\theta_o = 0.15$, is 0.18.

Then

$$t = \frac{0.18D^2}{h^2} = \frac{0.18(21.3)^2}{0.111} = 740 \text{ days}$$

Example A-2

A mass concrete bridge pier has a horizontal cross section of 7.6 x 15.2 m, and is at a mean temperature of 27 C. Determine the mean temperature at various times up to 200 days if the pier is exposed to water at 4 C and if the diffusivity is 0.084 m^2/day. For a prismatic body such as this pier, where heat is moving towards each of four pier faces, the part of original heat remaining may be computed by finding the part remaining in two infinite slabs of respective thickness equal to the two horizontal dimensions of the pier, and multiplying the two quantities so obtained to get the total heat remaining in the pier. For this two-dimensional use, it is better to find for various times the heat losses associated with each direction and then combine them to find the total heat loss of the pier.

Initial temperature difference, $\theta_o = 27 - 4 = 23$ C
For the 7.6 m dimension

$$\frac{h^2t}{D^2} = \frac{0.084t}{(7.6)^2} = 0.00145t$$

and for the 15.2 m dimension

$$\frac{h^2t}{D^2} = \frac{0.084t}{(15.2)^2} = 0.00036t$$

Then calculate numerical values of $0.00145t$ and $0.00036t$ for times from 10 to 200 days. See Table A.5.4.1. These values can be used with Fig. 5.4.1 to obtain the θ_m/θ_o ratios for both 7.6-m and 15.2-m slabs. The product of these ratios indicates the heat remaining in the pier, and can be used to calculate the final temperature difference θ_m. The values for θ_m are added to the temperature of the surrounding water to obtain mean pier temperatures at various times up to 200 days, as shown on Table A.5.4.1.

Example A-3

Granite aggregate at an initial temperature of 32 C is to be precooled in circulating 2 C water for use in mass concrete. The largest particles can be approximated as 150-mm-diameter spheres. How long must the aggregate be immersed to bring its mean temperature to 4 C?

Table A.5.4.1—Example A-2: calculations in SI (metric) units

Time, days	$\left(\frac{h^2t}{D^2}\right)_{7.6} =$ 0.00145t	$\left(\frac{h^2t}{D^2}\right)_{15.2} =$ 0.00036t	$\left(\frac{\theta_m}{\theta_o}\right)_{7.6}$ read from Fig. 5.4.1	$\left(\frac{\theta_m}{\theta_o}\right)_{15.2}$ read from Fig. 5.4.1	$\left(\frac{\theta_m}{\theta_o}\right)_{7.6}$ x $\left(\frac{\theta_m}{\theta_o}\right)_{15.2} = \left(\frac{\theta_m}{\theta_o}\right)_{pier}$	$\left(\frac{\theta_m}{\theta_o}\right)_{pier}$ x $\theta_o = \theta_m$	θ_m + 4 = temperature, C
10	0.0145	0.0036	0.73	0.87	0.64	15	19
20	0.0290	0.0072	0.61	0.80	0.49	11	15
30	0.0435	0.0108	0.53	0.77	0.41	9	13
40	0.0580	0.0144	0.46	0.73	0.34	8	12
60	0.0870	0.0216	0.35	0.67	0.23	5	9
100	0.1450	0.036	0.19	0.57	0.11	3	7
200	0.2900	0.072	0.05	0.40	0.02	0	4

For granite having a diffusivity h^2 of 0.096 m^2/day
Initial temperature difference, θ_o = 32 – 2 = 30 C
Final temperature difference, θ_m = 4 – 2 = 2 C

$$\frac{\theta_m}{\theta_o} = \frac{2}{30} = 0.07$$

From the solid sphere curve of Fig. 5.4.1 the value of h^2t/D^2 corresponding to θ_m/θ_o = 0.07 can be found to be 0.055.

Therefore

$$t = \frac{0.055(0.150)^2}{0.096} = 0.013 \text{ days}$$

or approximately 19 minutes.

Example A-4

A 15.2-m-diameter circular tunnel is to be plugged with mass concrete with a diffusivity of 0.111 m^2/day. The maximum mean temperature in the concrete is 43 C, and the surrounding rock is at 18 C.

Without artificial cooling, how long will it take for the temperature in the plug to reach 21 C, assuming the rock remains at 18 C?

Initial temperature difference, θ_o = 43 - 18 = 25 C
Final temperature difference, θ_m = 21 -18 = 3 C

$$\frac{\theta_m}{\theta_o} = \frac{3}{25} = 0.12$$

From the solid cylinder curve of Fig. 5.4.1, the value of h^2t/D^2 corresponding to Θ_m/Θ_o = 0.12 can be found to be 0.075.

Therefore

$$t = \frac{0.075(15.2)^2}{0.111} = 160 \text{ days}$$

Example A-5

A closure block of concrete initially at 41 C is to be cooled to 7 C to provide a joint opening of 0.64 mm prior to grouting contraction joints. How long will it take to cool the mass by circulating water at 3 C through cooling pipes spaced 1.40 horizontally and 1.50 m vertically. Assume concrete to be made with granite aggregate having a diffusivity h^2 of 0.096 m^2/day.

Cross section handled by each pipe is (1.40)(1.50) = 2.10 m^2

The diameter of an equivalent cylinder can be calculated as $\pi D^2/4$ = 2.10 m^2

Therefore

$$D^2 = \frac{(4)(2.10)}{\pi} = 2.67 \text{ m}^2$$

and

$$D = 1.63 \text{ m}$$

Initial temperature difference, θ_o = 41 - 3 = 38 C
Final temperature difference, θ_m = 7 - 3 = 4 C

$$\frac{\theta_m}{\theta_o} = \frac{4}{38} = 0.11$$

Referring to Fig. 5.4.1 and using the curve for the hollow cylinder (since cooling is from within the cross section), for the calculated value of θ_m/θ_o, h^2t/D^2 can be found to be 1.0.

Therefore

$$t = \frac{1.0(2.67)}{0.096} = 28 \text{ days}$$

About the same results can be achieved with greater economy if the natural cold water of the river is used for part of the cooling. Control of the rate of cooling must be exercised to prevent thermal shock, and in many cases postcooling is conducted in two stages.

Assume river water is available at 16 C, cool to 20 C, and then switch to refrigerated water at 3 C. How much time will be taken in each operation, and what is total cooling time?

For initial cooling, θ_o = 41 - 16 = 25 C and θ_m = 20 – 16 = 4 C

$$\frac{\theta_m}{\theta_o} = \frac{4}{25} = 0.16$$

From Fig. 5.4.1 for a hollow cylinder

$$\frac{h^2 t}{D^2} = 0.84$$

Therefore

$$t = \frac{(0.84)(2.67)}{0.096} = 23 \text{ days}$$

For final cooling, θ_o = 20 - 3 = 17 C and θ_m = 7 - 3 = 4 C

$$\frac{\theta_m}{\theta_o} = \frac{4}{17} = 0.24$$

$$\frac{h^2 t}{D^2} = 0.65$$

$$t = \frac{(0.65)(2.67)}{0.096} = 18$$

Total time is 23 + 18 = 41 days, but of this, the time for using refrigeration has been cut by one third.

Example A-6 (see 5.4.3)

Determine the temperature rise throughout two 1.8-m lifts of mass concrete placed at two-day intervals. The concrete contains 223 kg/m^3 of Type II cement and has a diffusivity of 0.093 m^2/day. Take the space interval as 0.3 m.

Then the time interval needed for the temperature at the center of the space to reach a temperature which is the average of the temperatures of the two adjacent elements is

$$\Delta t = \frac{\Delta x^2}{2h^2} = \frac{(0.3)^2}{2(0.093)} = 0.5 \text{ day}$$

In Table A.5.4.3(a), the adiabatic temperature rise (above the temperature of the concrete when it was placed) in 0.5-day intervals for a three-day investigation is taken from Fig. 5.3.1 (except that the temperature rise at 0.5-day age is estimated). The change in temperature $\Delta\theta$ is determined by subtracting the temperature at any time interval from that of the preceding time interval.

In the tabular solution, Table A.5.4.3(b), the space interval of 0.3 m divides each lift into six elements. Note that the adiabatic temperature rise is taken as just one-half of the concrete rise since the rock is not generating heat. At the construction joint, the rise is the average of the two lifts, which are generating heat at different rates at any given time. At the exposed surface, the adiabatic rise is zero because the heat is dissipated as quickly as it is generated from the concrete below.

Normally where there are several stations considered in each life, the temperature distribution within the lift at any given time can be obtained with sufficient accuracy by calculating only half of the points at any one time, as shown in the tabulated solution, Table A.5.4.3(b).

Table A.5.4.3(a)— For Example A-6, adiabatic temperature increments read from Table 5.3.1

Time, days	Adiabatic temperature rise above concrete placing temperature θ, C	Δθ
0.0	0	
0.5	12	12
1.0	18	6
1.5	22	4
2.0	24	2
2.5	25	1
3.0	26	1

Table A.5.4.3(b)— For Example A-6, calculated temperature rise in concrete above placing temperature, C

Distance above ground, m	Time *t*, days											
	0.0	0.5	1.0		1.5		2.0		2.5		3.0	
	$\Delta\theta_1$ = 12C		$\Delta\theta_1$ = 6C		$\Delta\theta_1$ = 4C		$\Delta\theta_1$ = 2C		$\Delta\theta_2$ = 12C $\Delta\theta_1$ = 1C		$\Delta\theta_2$ = 6C $\Delta\theta_1$ = 1C	
3.6								0	0	0		
3.3								0			6	12
3.0								0	0	12		
2.7								0			12	18
2.4								0	0	12		
2.1								0			12.2	18.2
1.8	0	0			0	0			5.8	12.3		
1.5	0	12	6	12			9.5	11.5			15	16
1.2	0	12			15	19			16.6	17.6		
0.9	0	12	12	18			19.8	21.8			19.4	20.4
0.6	0	12			16.5	20.5			20.3	21.3		
0.3	0	12	9	15			16.8	18.8			17.6	18.6
0.0	0	6			9	13			13	14		
-0.3	0	0	3				7.2				9	
-0.6	0	0			1.5				4			
-0.9		0	0				0.8				2.2	
-1.2					0				0.4			
-1.5							0				0.2	
-1.8									0			

Note that in the computation above two steps are required to produce the temperature at the end of the half-day period; the first step averages the adjacent temperatures, and the second step adds the adiabatic temperature rise of the concrete. Calculations are carried out here to more significant figures than are justified merely to make clear the method.