Guide to Mass Concrete

Reported by ACI Committee 207

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Mass concrete is any volume of concrete with dimensions large enough to require that measures be taken to cope with the 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. This document contains a history of the development of mass concrete practice and discussion of materials and concrete mixture proportioning, properties, construction methods, and equipment. It covers traditionally placed and consolidated mass concrete and does not cover roller-compacted concrete.

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CHAPTER 1—INTRODUCTION AND HISTORICAL DEVELOPMENTS

1.1—Scope

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 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. Because 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. Significant tensile stresses and strains may result from the restrained volume change associated with a decline in temperature as heat of hydration is dissipated. Measures should be taken where cracking due to thermal behavior may cause a loss of structural integrity and monolithic action, excessive seepage and shortening of the service life of the structure, or be aesthetically objectionable. Many of the principles in mass concrete practice can also be applied to general concrete work, whereby economic and other benefits may be realized.

This document contains a history of the development of mass concrete practice and a discussion of materials and concrete mixture proportioning, properties, construction methods, and equipment. This document 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 has also 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; however, thin arch dams are exceptions. Massive structures, such as gravity dams, resist loads primarily by 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

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

Covered herein are the principal steps from those very small beginnings to the present. In large dam construction, there is now exact and automatic proportioning and mixing of materials. Concrete in 12 yd^3 (9 m^3) buckets can be placed by conventional methods at the rate of 10,000 yd^3/day (7650 m^3/day) at a temperature of less than 50 °F (10 °C) as placed, even during extremely hot weather. Grand Coulee Dam still holds the 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). 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. Lean mixtures are now made workable by means of air entrainment 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 and in controlling the time of setting. Placing rates for no-slump concrete, by 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.

1.2.1 *Before 1900*—Before 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 the benefit of washing to remove objectionable dirt and fines. Concrete mixtures varied widely in cement content and in sand-coarse aggregate ratio. Mixing was usually done by hand and proportioning by shovel, wheelbarrow, box, or cart. The effect of the water-cement ratio (*w/c*) was unknown, and generally no attempt was made to control the volume of mixing water. There was no measure of consistency except by visual observation of the newly mixed concrete.

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

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

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

1.2.2 *1900 to 1930*—After the turn of the century, construction of all types of concrete dams was greatly accelerated. More and higher dams for irrigation, power, and water supply were built. Concrete placement by means of towers and chutes became common. In the United States, the portland cement industry became well established, and cement was rarely imported from Europe. ASTM specifications for portland cement underwent little change during the first 30 years of the century, aside from a modest increase in fineness requirement determined by sieve analysis. Except for the limits on magnesia and loss on ignition, there were no chemical requirements. Character and grading of aggregates were given more attention during this period. Very substantial progress was made in the development of methods of proportioning concrete. The water-cement strength relationship was established by Abrams and his associates from investigations before 1918, when Portland Cement Association (PCA) Bulletin 1 appeared (Abrams 1918). Nevertheless, little attention was paid to the quantity of mixing water. Placing methods using towers and flat-sloped chutes dominated, resulting in the use of excessively wet mixtures for at least 12 years after the importance of the *w/c* had been established.

Generally, portland cements were employed without admixtures. There were exceptions, such as the sand-cements used by the U.S. Reclamation Service (now the U.S. Bureau of Reclamation [USBR]) in the construction of the Elephant Butte Dam in New Mexico and the Arrowrock Dam in Idaho. At the time of its completion in 1915, the Arrowrock Dam, a gravity-arch dam, was the highest dam in the world at 350 ft (107 m). The dam was constructed with lean interior concrete and a richer exterior face concrete. The mixture for interior concrete contained approximately 376 lb/yd^3 (223 kg/m^3) of a blended, pulverized granite-cement combination. The cement mixture was produced at the site by intergrinding approximately equal parts of portland cement and pulverized granite so that no less than 90% passed the No. 200 (75 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 used as a 20% replacement by mass for portland cement.

During this 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). The slump test had come into use as a means of measuring consistency. The testing of 6 x 12 in. (150 x 300 mm) and 8 x 16 in. (200 x 400 mm) job cylinders became common practice in the United States. European countries generally adopted the 8 x 8 in. (200 x 200 mm) cube for testing the strength at various ages. Mixers of 3 yd^3 (2.3 m^3) capacity were commonly used 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 mixture containing 564 lb/yd^3 (335 kg/m^3) of cement for the entire concrete mass. The construction practice of using an interior mixture containing 376 lb/yd^3 (223 kg/m^3) and an exterior face mixture containing 564 lb/yd^3 (335 kg/m^3) was developed 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 the Theodore Roosevelt Dam built during the years of 1905 to 1911 in Arizona. This dam consists of a rubble masonry structure faced with rough stone blocks laid in portland cement mortar made with a cement manufactured in a plant near the dam site. For this structure, the average cement content has been calculated to be approximately 282 lb/yd^3 (167 kg/m^3). For the interior of the mass, rough quarried stones were embedded in a 1:2.5 mortar containing approximately 846 lb/yd^3 (502 kg/m^3) of cement. In each layer, the voids between the closely spaced stones were filled with a concrete containing 564 lb/yd^3 (335 kg/m^3) of cement, into which rock fragments were manually placed. These conditions account for the very low average cement content. Construction was slow, and Roosevelt Dam represents perhaps the last of the large dams built in the United States by this method of construction.

1.2.3 *1930 to 1970*—This was an era of rapid development in mass concrete construction for dams. The use of the tower and chute method declined during this period and was used only on small projects. Concrete was typically placed using large buckets with cranes, cableways, railroad systems, or a combination of these. On the larger and more closely controlled construction projects, the aggregates were carefully processed, ingredients were proportioned by weight, and the mixing water was 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 used large spud vibrators for consolidation.

A study of the records and actual inspection of a considerable number of dams shows that there were differences in condition that could not be explained. Of two structures that appeared to be of similar quality subjected to the same environment, one might exhibit excessive cracking while the other, after a similar period of service, would be in near-perfect condition. The 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 that 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, the Hoover Dam in Nevada was in the early stages of planning. Because of the unprecedented size of the Hoover Dam, investigations much more elaborate than any previously undertaken were carried out to determine the effects of factors, such as composition and fineness of cement, cement factor, temperature of curing, and maximum size of aggregate, on the heat of hydration of cement, compressive strength, and other properties of mortars and concrete.

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

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

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

The chemical admixtures that function to reduce water in concrete mixtures, control setting, and enhance strength of concrete began to be seriously recognized in the 1950s 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 been used in most mass concrete.

Around 1945, it became standard practice to use intentionally 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 to concrete pavements and reinforced concrete in general. Air-entraining admixtures introduced at the mixer have been used 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.

1.2.4 *1970 to present*—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 walls, and some foundation works, particularly where reinforcement is required.

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

1.2.5 *Cement content*—During the late 1920s and early 1930s, it was practically an unwritten law that no mass concrete for large dams should contain less than 376 lb/yd^3 (223 kg/m^3) of cement. Some authorities of that period believed 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 (TVA) in 1936, was 376 lb/yd^3 (223 kg/m^3). The degree of cracking was excessive. The compressive strength of the wet-screened 6 x 12 in. (150 x 300 mm) job cylinders at 1 year of age was 7000 psi (48.3 MPa). Similarly, core specimens 18 x 36 in. (460 x 910 mm) drilled from the first stage concrete containing 376 lb/yd^3 (223 kg/m^3) of cement at Grand Coulee Dam tested in excess of 8000 psi (55 MPa) at the age of 2 years. Judged by composition, the cement was 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, which began a trend toward reducing the cement content, which is still continuing. Since that time, the Type II cement content of the interior mass concrete has been approximately 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 USACE in 1954. In arch-type high dams where stresses are moderately high, the cement content of the mass mixture is usually in the range of 300 to 450 lb/yd^3 (180 to 270 kg/m^3), with the higher cement content being used in the thinner and more highly stressed dams of this type.

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

- *Arch dams*—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; 373 lb/yd^3 (221 kg/m^3) of cement in Morrow Point Dam in Colorado, completed in 1968; and 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*—226 lb/yd^3 (134 kg/m^3) of Type II cement in Detroit Dam in Oregon, completed in 1952; 194 lb/yd^3 (115 kg/m^3) of Type II cement and fly ash in Libby Dam in Montana, completed in 1972; and 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

The practice of precooling concrete materials before mixing to achieve a lower maximum temperature of interior mass concrete during the hydration period began in the early 1940s, and has been extensively used in the construction of large dams.

The first practice of precooling appears to have occurred during the construction of Norfork Dam from 1941 to 1945 by the USACE. The plan was to introduce crushed ice into the mixing water during the warmer months. By so doing, the temperature of freshly mixed mass concrete could be reduced by approximately 10 °F (5.6 °C). Not only has crushed ice been used in the mixing water, but coarse aggregates have been precooled either by cold air or cold water before 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 use precooling for large dams in regions where the summer temperatures are high to ensure that the temperature of concrete, as it is placed, does not exceed approximately 50 °F (10 °C).

On some large dams, including Hoover (Boulder) Dam, a combination of precooling and postcooling refrigeration by embedded pipe has been used (USBR 1949). A good example of this practice is Glen Canyon Dam, where the ambient temperatures can be greater than 100 °F (38 °C) during the summer months. The temperature of the precooled fresh concrete did not exceed 50 °F (10 °C). Both refrigerated aggregate and crushed ice were used to achieve this low 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 to control cracking is also done for control of peak temperatures.

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 1 year old. Under mass curing conditions, with the cement and pozzolans customarily employed, the gain in concrete strength between 28 days and 1 year is generally large. ACI 232.2R reports that the gain can range from 30 to more than 100%, depending on the quantities and proportioning of cementitious materials 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 at 1 or 2 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). The aggregate larger than 1-1/2 in. (37.5 mm) is removed from the concrete by wet-screening. Strength requirements of the wet-screened concrete are correlated with the specified full-mixture strength by laboratory tests.

CHAPTER 2—MATERIALS AND MIXTURE PROPORTIONING

2.1—General

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 mixture 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 describes materials that have been successfully used in mass concrete construction and the factors influencing their selection and proportioning. The recommendations contained herein may need to be adjusted for special uses, such as for massive precast beam segments, tremie placements, and roller-compacted concrete. Guidance in proportioning mass concrete can also be found in ACI 211.1, particularly

Appendix 5, which details procedures for mass concrete proportioning.

2.2—Cements

ACI 207.2R and 207.4R contain additional information on cement types and effects on heat generation. The following types of hydraulic cement are suitable for use in mass concrete construction:

- *Portland cement*—Types I, II, IV, and V, as covered by ASTM C 150;
- *Blended cement*—Types P, IP, S, IS, I(PM), and I(SM), as covered by ASTM C 595; and
- *Hydraulic cement*—Types GU, MS, HS, MH, and LH, as covered by ASTM C 1157.

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.

Type I and GU cements are suitable for use in general construction. They are not recommended for use alone in mass concrete without other measures that help to control temperature problems because of their substantially higher heat of hydration.

Type II (moderate heat) and MH cements are suitable for mass concrete construction because they have a moderate heat of hydration, which is important to the control of cracking. Type II must be specified with the moderate heat option as most Type II and MS cements are designed for moderate sulfate resistance and do not have moderate heat properties. Specifications for Type II portland cement require that it contain no more than 8% tricalcium aluminate (C_3A), the compound that contributes substantially to early heat development in concrete. Optional specifications for Type II cement place a limit of 58% or less on the sum of C_3A and C_3S or a limit on the heat of hydration to 70 cal/g (290 kJ/kg) at 7 days. When one of the optional requirements is specified, the 28-day strength requirement for cement paste under ASTM C 150 is reduced due to the slower rate of strength gain of this cement.

Types IV and LH, low-heat cements, may be used where it is desired to produce low heat development in massive structures. They have not been used in recent years because they have 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%, the C_3S to 35%, and place a minimum on the C_2S of 40%. At the option of the purchaser, the heat of hydration may be limited to 60 cal/g (250 kJ/kg) at 7 days and 70 cal/g (290 kJ/kg) at 28 days. Type IV cement is generally not available in the United States.

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

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% by weight of the portland-pozzolan cement, with Type P generally having the higher pozzolan content.

Type I(PM) pozzolan-modified portland cement contains less than 15% pozzolan, and its properties are close to those of Type I cement. A heat of hydration limit of 70 cal/g (290 kJ/kg) at 7 days is an optional requirement for Types IP and 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).

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% by weight of the portland blast-furnace slag cement. This cement has sometimes been used with a pozzolan. Type S slag cement is a finely divided material consisting of a uniform blend of granulated blast-furnace slag and hydrated lime in which the slag constituent is at least 70% 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% slag, and its properties are close to those of Type I cement. Optional heat-of-hydration requirements can be applied to Types IS and I(SM), similar to those applied to Types IP, I(PM), and P.

Low-alkali cements are defined by ASTM C 150 as portland cements containing not more than 0.60% alkalies calculated as the percentage of Na_2O plus 0.658 times the percentage of K_2O. These cements can 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 ensure control of the alkali-aggregate reaction.

2.3—Pozzolans and ground slag

A pozzolan is generally defined as a siliceous or siliceous-and-aluminous material that 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 (Class F or C). There are some pozzolans, such as the Class C fly ash, that contain significant amounts of compounds like those of portland cement. The Class C fly ashes likewise have cementitious properties by themselves that 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 should be in an amorphous state, such as glass or opal. Crystalline siliceous materials, such as quartz, do not combine readily with lime at a normal temperature unless they are ground into a very fine powder. The use of fly ash in concrete is discussed in ACI 232.2R, and the use of ground-granulated blast-furnace slag is discussed in ACI 233R.

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, should be activated to form an amorphous state by calcining at temperatures in the range of 1200 to 1800 °F (650 to 980 °C).

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 approximately 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. This may, however, reduce its lubricating qualities and increase the water requirement of the concrete. 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 is less helpful in lowering heat generation in the concrete.

Pozzolans in mass concrete may be used to reduce portland cement factors for better economy, lower internal heat generation, improve workability, and 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. 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 with 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 to temporarily forego the use of the pozzolan and otherwise accommodate the increased internal heat generated by the use of straight portland cement. If there is a dangerous potential from alkali-aggregate reaction, however, 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, the fineness of the pozzolan, and the amount employed. For some pozzolans, the reduction in expansion may exceed 90%. 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.

Results of some experiments reported by 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 it is more economical 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 1-year compressive strength of wet-screened 6 x 12 in. (150 x 300 mm) cylinders of this concrete was approximately 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 so that the concrete, as it reaches the forms, will be approximately 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 have 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.

Finely ground-granulated iron blast-furnace slag, commonly referred to as slag cement, may also be used as a separate ingredient with portland cement as a 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% finely ground slag of total cementitious material may be needed with an active slag to produce a cement-slag combination that will have a heat of hydration 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. Finely ground slag, however, is usually used in much higher percentages than pozzolan to achieve similar properties.

2.4—Chemical admixtures

Chemical admixtures can provide important benefits to mass concrete in its plastic state by increasing workability, reducing water content, or both. Also, chemical admixtures can be used for retarding initial setting, modifying the rate or capacity for bleeding, reducing segregation, and reducing rate of slump loss. 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 or erosion resistance. A full coverage of admixtures is contained in ACI 212.3R. The chemical admixtures that are important to mass concrete are classified as air-entraining, water-reducing, or set-controlling.

Air-entraining admixtures are materials that 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. Air entrainment also facilitates the placing and handling of mass concrete. Each 1% of entrained air permits a reduction in mixing water from 2 to 4%, with some improvement in workability and no loss in slump. 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.

Entrained air will generally reduce the strength of most concrete. 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 even slightly increase it. Among the factors that influence the amount of entrained air in concrete for a given amount of agent are: grading and particle shape of the aggregate, richness of the mixture, presence of other admixtures, mixing time, slump, and temperature of the concrete. For a given quantity of air-entraining admixture, air content increases with increases in slump 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 carbon, an increased dosage of air-entraining admixture is required. Most specifications for mass concrete 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 approximately 5%, although in some cases as high as 8%. Requirements for air-entraining admixtures are contained in ASTM C 260.

Water-reducing and set-controlling admixtures generally consist of one or more of the following: lignosulfonic acid, hydroxylated carboxylic acid, polymeric carbohydrates, and naphthalene or melamine types of high-range water reducers.

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.

Water-reducing admixtures are used to reduce the mixing water to increase strength and reduce shrinkage of the concrete, increase the workability of the concrete, or produce the same strength with less cement. 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. Admixtures from the aforementioned first three families of materials generally will reduce the water requirement up to approximately 10%, retard initial set at least 1 hour (but not reduce slump loss), and appreciably increase the strength. When a retarder is used, the strength after 12 hours is generally comparable to that of concrete containing no admixture. Depending on the richness of the concrete, composition of the cement, temperature, and other factors, the 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 *w/c*; the chemicals have a favorable effect on the hydration of the cement. Admixtures of the carboxylic acid family augment bleeding. High-range water-reducing admixtures have not been used 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. Continued admixture development has resulted in very stable admixtures that maintain consistent and long-term performance. This has resulted in a wide range of water-reducing admixtures to be used in most mass concrete mixtures today. Requirements for chemical admixtures are contained in ASTM C 494.

2.5—Aggregates

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

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 using normal proportions of the ingredients. Deleterious substances are usually limited to the percentages by weight given in Table 2.1. For exposed concrete in the zone of fluctuating water lever for bridge piers, dams, and other hydraulic structures, the maximum allowable percentage of the deleterious substance should be 50% lower than that given in Table 2.1 for face concrete in the zone of fluctuating water levels. It can be 50% higher for concrete constantly immersed in water and for concrete in the interior of massive dams.

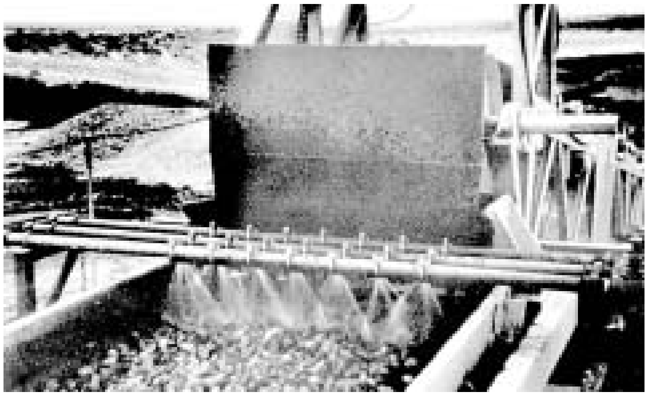

Fig. 2.1—Coarse aggregate rewashing.

The grading of fine aggregate strongly influences the workability of concrete. A good grading of sand for mass concrete should be within the limits shown in Table 2.2. Laboratory investigation, however, 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.

Coarse aggregate is defined as gravel, crushed gravel, crushed rock, or a mixture of these, nominally larger than the No. 4 (4.75 mm) and smaller than the 6 in. (150 mm) 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 should be offset by greater effort to reduce the cement requirement and concrete placing temperatures. The maximum size of coarse aggregate should not exceed 1/4 of the least dimension of the structure nor 2/3 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 practical size of coarse aggregate.

Coarse aggregate should consist of hard, dense, durable, uncoated particles. Rock that is friable or tends to degrade during processing, transporting, or in storage should be avoided. Rock having an absorption greater than 3% 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% 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 generally not exceed the values outlined in Table 2.3.

Table 2.1—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 important	0.5
All other concrete	1.0

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

Table 2.2—Fine aggregate for mass concrete*

Sieve designation	Percentage retained, individual by weight
3/8 in. (9.50 mm)	0
No. 4 (4.75 mm)	0 to 5
No. 8 (2.36 mm)	5 to 15
No. 16 (1.18 mm)	10 to 25
No. 30 (600 μm)	10 to 30
No. 50 (300 μm)	15 to 35
No. 100 (150 μm)	12 to 20
Pan fraction	3 to 7

*U.S. Bureau of Reclamation (1981).

Table 2.3—Maximum allowable percentages of deleterious substance in coarse aggregate (by weight)

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

Figure 2.1 shows a coarse-aggregate rewashing screen at the batch plant where dust and coatings accumulating from stockpiling and handling can be removed to ensure aggregate cleanliness.

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. It has been demonstrated (Fig. 2.2), however, 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 et al. 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 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

Table 2.4—Grading requirements for coarse aggregate

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

Table 2.5—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 to 3 in. (150 to 75 mm)	Coarse 3 to 1-1/2 in. (75 to 37.5 mm)	Medium 1-1/2 to 3/4 in. (37.5 to 19 mm)	Fine: 3/4 to 3/8 in. (19 to 9.5 mm)	Fine: 3/8 in. to No. 4 (9.5 to 4.75 mm)
6 (150)	20 to 30	20 to 32	20 to 30	12 to 20	8 to 15
3 (75)		20 to 40	20 to 40	15 to 25	10 to 15
1-1/2 (37.5)			40 to 55	30 to 35	15 to 25
3/4 (19)				30 to 70	20 to 45

*U.S. Bureau of Reclamation (1981).

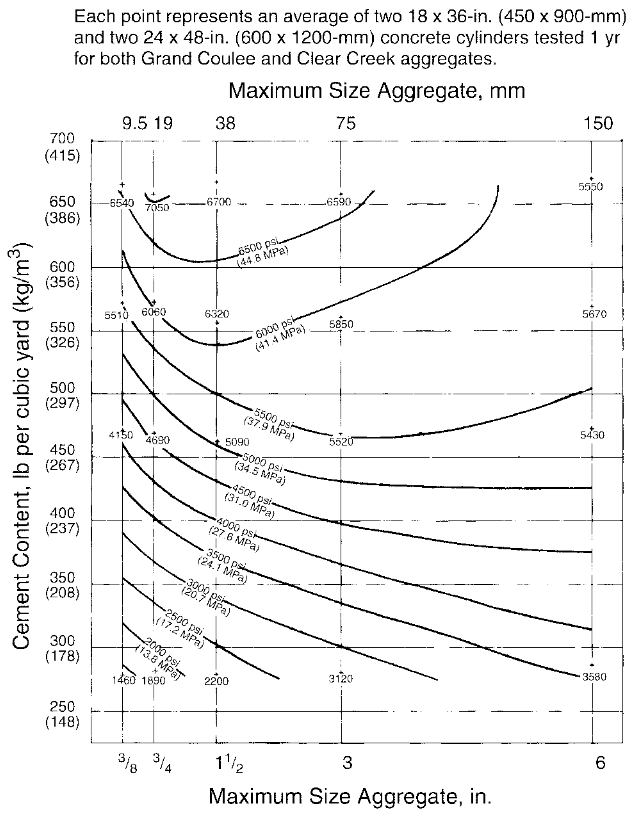

Fig. 2.2—Effect of aggregate size and cement content on compressive strength at 1 year (adapted from Higginson et al. [1963]).

volume change. They also cause voids to form underneath due to bleeding water and air accumulation during placing of concrete. Although larger sizes have occasionally been used, an aggregate size of 6 in. (150 mm) has normally been adopted as the maximum practical size.

The particle shape of aggregates has some effect on workability and, consequently, on water requirement. Rounded particles, such as those that occur in deposits of stream-worn sand and gravel, provide the 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% in each size group. A flat particle is defined as having a width-thickness ratio greater than 3, while an elongated particle is defined as having a length-width ratio greater than 3.

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 2.4 (Tuthill 1980).

Sizes are satisfactorily graded when 1/3 to 1/2 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% 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.

Experience has shown that a rather wide range of material percentage in each size group may be used as listed in Table 2.5. Workability is frequently improved by reducing the proportion of cobbles called for by the theoretical 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 used, the amount of the finest size of coarse aggregate can be reduced because that is the size of the accumulated undersize of the larger sizes. Finish screening at the batching plant, however, 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% of the cobbles, 3% of the intermediate sizes, and 2% of the fine coarse aggregates. Undersize is defined as that passing a test screen having openings 5/6 of the nominal minimum size of the aggregate fraction. Undersize larger than this 5/6 fraction has no measurable effect on the concrete (Tuthill 1943).

In some parts of the world, gap-graded aggregate is used in mass concrete. In these gradings, the material in one or more sieve sizes is missing. Continuous gradings are normally used in the United States. Gap-graded aggregate can be used economically where the material is naturally gap-graded; however, comparisons that can be made between concrete 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

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 amounts 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 (Test method CRD C 400 from USACE [1963b]). If the average of the results of these tests on specimens containing the water being evaluated is less than 90% 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 of 5000 ppm or more, tests for strength and volume stability (length change) may also be advisable to ensure durable concrete.

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

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

Selection of *w/c* or *w/cm* will establish the strength, durability, and permeability of the concrete. There should also 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 approximately 10% less than for a concrete containing angular aggregates. Trial mixtures using the required *w/cm* and the observed water requirement for the job materials will demonstrate the cementitious material content that may be safely used to provide the required workability (Portland Cement Association 1979; Ginzburg et al. 1966).

The first step in arriving at the actual batch weights is to select the maximum aggregate size for each part of the work. Criteria for this selection are given in Section 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 to 2 in. (38 to 50 mm). In tests for slump, aggregate larger than 1-1/2 in. (38 mm) should be removed by promptly screening the wet concrete. For 6 in. (150 mm) maximum-size aggregate, water contents for air-entrained, minimum-slump concrete may vary from approximately 120 to 150 lb/yd^3 (71 to 89 kg/m^3) for natural aggregates, and from 140 to 190 lb/yd^3 (83 to 113 kg/m^3) for crushed aggregates. Corresponding water requirements for 3 in. (76 mm) maximum-size aggregate are approximately 20% higher. For strengths above 4000 psi (28 MPa) at 1 year, however, the 3 in. (75 mm) maximum-size aggregate may be more efficient (Fig. 2.2).

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

When a pozzolan is included in the concrete as a part of the cementitious material, the mixture proportioning procedure does not change. Attention should be given to the following matters:

- The water requirement may change;
- Early-age strength may become critical; and
- 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 many months or years old. Therefore, mass concrete containing pozzolan is usually designed on the basis of 90-day to 1-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 should be great enough to prevent pullout of the form anchors. Specially designed form anchors may be required to allow safe, rapid turnaround times for the forms, especially when large amounts of pozzolan are used or when the concrete is lean and precooled.

2.8—Temperature control

The four elements of an effective temperature control program, any or all of which may be used for a particular mass concrete project, are:

- Cementitious material content control, where the choice of type and amount of cementitious materials can lessen the heat-generating potential of the concrete;
- Precooling, where cooling of ingredients achieves a lower concrete temperature as placed in the structure;
- Postcooling, where removing heat from the concrete with embedded cooling coils limits the temperature rise in the structure; and
- Construction management, where efforts are made to protect the structure from excessive temperature differentials by knowledge 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 can include the prudent selection of a low-heat-generating cement system including:

- The use of pozzolans;
- The careful production control of aggregate gradings and the use of large-size aggregates in efficient mixtures 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;
- The use of appropriate block dimensions for placement;
- The coordination of 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;
- The evaporative cooling of surfaces through water curing;
- The dissipation of heat from the hardened concrete by circulating cold water through embedded piping; and
- The insulation of 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 mixture 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 because the water requirement decreases as the temperature drops. Specified placing temperatures should be established by temperature studies to determine what is required to satisfy the design. A detailed discussion of thermal issues is contained in ACI 207.2R. Guidance in cooling systems for mass concrete can be found in ACI 207.4R.

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 that makes it possible to limit the heat of hydration to 70 cal/g (290 kJ/kg) at 7 days. The 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 concrete will not exceed 35 °F (19 °C).

CHAPTER 3—PROPERTIES

3.1—General

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 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, such 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. Increasing utilization has been made of finite-element computer

Table 3.1—Typical concrete mixture 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/cm*	Entrained air, %	Density, lb/yd^3 (kg/m^3)	WRA used
1	Hoover (U.S.)	1936	Arch gravity	IV	380 (225)	—	0	931 (552)	2679 (1589)	Limestone and granite	9.0 (225)	220 (130)	0.58	0	155.9 (2498)	No
2	Norris (U.S.)	1936	Straight gravity	II	338 (200)	—	0	1264 (750)	2508 (1487)	Dolomite	6.0 (150)	227 (135)	0.67	0	156.0 (2499)	No
3	Bonneville (U.S.)	1938	Gravity	Portland pozzolan	329 (195)	—	0	1094 (649)	2551 (1513)	Basalt	6.0 (150)	251 (149)	0.76	0	156.4 (2505)	No
4	Bartlett (U.S.)	1939	Multiple arch	IV	466 (276)	—	0	1202 (713)	2269 (1346)	Quartzite and granite	3.0 (75)	270 (160)	0.58	0	154.8 (2480)	No
5	Grand Coulee (U.S.)	1942	Straight gravity	II and IV	377 (224)	—	0	982 (582)	2568 (1523)	Basalt	6.0 (150)	226 (134)	0.60	0	153.8 (2464)	No
6	Kentucky (U.S.)	1944	Straight gravity	II	338 (200)	—	0	967 (573)	2614 (1550)	Limestone	6.0 (150)	213 (126)	0.63	0	153.2 (2454)	No
7	Shasta (U.S.)	1945	Curved gravity	IV	370 (219)	—	0	906 (537)	2721 (1614)	Andesite and slate	6.0 (150)	206 (122)	0.56	0	155.7 (2494)	No
8	Hungry Horse (U.S.)	1952	Arch gravity	II	188 (111)	Fly ash	90 (53)	842 (499)	2820 (1672)	Sandstone	6.0 (150)	130 (77)	0.47	3.0	150.7 (2415)	No
9	Detroit (U.S.)	1953	Straight gravity	II and IV	226 (134)	—	0	1000 (593)	2690 (1595)	Diorite	6.0 (150)	191 (113)	0.85	5.5	152.1 (2437)	No
10	Monticello (U.S.)	1957	Arch	II LA	212 (126)	Calcinated diatomaceous clay	70 (42)	770 (457)	2960 (1756)	Graywacke sandstone quartzite	6.0 (150)	161 (96)	0.57	2.7	154.6 (2477)	No
11	Flaming Gorge (U.S.)	1962	Arch gravity	II	188 (111)	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 (U.S.)	1963	Arch gravity	II	188 (111)	Pumicite	94 (56)	777 (461)	2784 (1651)	Limestone, chaledonic chert, and sandstone	6.0 (150)	153 (91)	0.54	3.5	148.0 (2371)	No
		1963	Arch gravity	II	188 (111)	Pumicite	90 (53)	800 (474)	2802 (1662)		6.0 (150)	140 (83)	0.50	3.5	148.9 (2385)	Yes
13	Yellowtail (U.S.)	1965	Arch gravity	II	197 (117)	Fly ash	85 (50)	890 (526)	2817 (1670)	Limestone and andesite	6.0 (150)	139 (82)	0.49	3.0	152.9 (2449)	No
14	Morrow Point (U.S.)	1967	Thin arch	II	373 (221)	—	0	634 (376)	2851 (1691)	Andesite, tuff, and basalt	4.5 (114)	156 (93)	0.42	4.3	148.7 (2382)	Yes
15	Dworshak (U.S.)	1972	Gravity	II	211 (125)	Fly ash	71 (42)	740 (439)	2983 (1770)	Crushed granite gneiss	6.0 (150)	164 (97)	0.59	3.5	154.4 (2473)	No
16	Libby (U.S.)	1972	Gravity	II	148 (88)	Fly ash	49 (29)	903 (536)	2878 (1708)	Natural quartzite gravel	6.0 (150)	133 (79)	0.68	3.5	152.3 (2439)	No
17	Lower Granite (U.S.)	1973	Gravity	II	145 (86)	Milled volcanic cinders	49 (29)	769 (456)	3096 (1837)	Natural basaltic gravel	6.0 (150)	138 (82)	0.71	3.5	155.4 (2490)	Yes
18	Pueblo (U.S.)	1974	Buttress	II LA	226 (134)	—	75 (44)	952 (565)	2589 (1535)	Granite, shist, limestone, dolomite	3.5 (89)	168 (100)	0.56	3.5	148.5 (2379)	Yes
19	Crystal (U.S.)	1976	Thin arch	II LA	390 (231)	—	0	829 (492)	2740 (1625)	Shist, altered volcanics	3.0 (75)	183 (109)	0.47	3.5	153.4 (2457)	Yes
20	Richard B. Russell (U.S.)	1982	Gravity	II	226 (134)	Fly ash	59 (35)	822 (488)	2958 (1755)	Crushed granite	6.0 (150)	173 (103)	0.57	3.4	157.0 (2515)	Yes
				II	173 (103)	Fly ash	73 (43)	864 (513)	2935 (1741)		6.0 (150)	177 (105)	0.67	3.4	156.0 (2499)	Yes
21	Rossens (Switzerland)	1948	Arch	I	421 (250)	—	0	—	—	Limestone	3.1 (79)	225 (133)	0.53	0	—	No
22	Pieve di Cadore (Italy)	1949	Arch gravity	Ferric-pozzolanic	253 (150)	Natural	84 (50)	1180 (700)	2089 (1239)	Limestone	4.7 (120)	213 (126)	0.63	2.0	159.9 (2560)	Yes
23	Francisco Madero (Mexico)	1949	Round-head buttress	IV	372 (221)	—	0	893 (530)	2381 (1412)	Rhyolite and basalt	6.0 (150)	223 (132)	0.60	—	—	—
24	Chastang (France)	1951	Arch gravity	250/315	379 (225)	—	0	759 (450)	2765 (1640)	Granite	9.8 (250)	169 (100)	0.45	—	150.8 (2415)	—
25	Salmonde (Portugal)	1953	Thin arch	II	421 (250)	—	0	739 (438)	2621 (1554)	Granite	7.9 (200)	225 (133)	0.54	0	148.4 (2376)	—
26	Warragamba (Australia)	1960	Straight gravity	II	330 (196)	—	0	848 (503)	2845 (1687)	Porphyry and granite	6.0 (150)	175 (104)	0.53	0	154.2 (2469)	No
27	Krasnoiarsk (U.S.S.R.)	About 1970	Straight gravity	IV and portland blast furnace	388 (230)	—	0	—	—	Granite	3.9 (100)	213 (126)	0.55	—	—	Yes
28	Ilha Solteira (Brazil)	1974	Gravity	II	138 (82)	Calcinated clay	46 (27)	788 (468)	3190 (1893)	Quartzite gravel, crushed basalt	6.0 (150)	138 (82)	0.75	3.5	159.3 (2552)	No
29	Itaipu (Brazil-Paraguay)	1982	Hollow gravity buttress	II	182 (108)	Fly ash	22 (13)	981 (582)	3096 (1837)	Crushed basalt	6.0 (150)	143 (85) 170 (101)	0.70	4.0	158.4 (2537)	No
30	Peace Site 1 (Canada)	1979	Gravity	I	158 (94)	Fly ash	105 (63)	967 (575)	2610 (1549)	Quartzite limestone sandstone	3 (75)	144 (85)	0.67	3.6	148.5 (2379)	Yes
31	Theodore Roosevelt modification (U.S.)	1995	Arch gravity	II LA	216 (128)	Fly ash	54 (32)	954 (566)	2672 (1585)	Granite	4.0 (100)	144 (85)	0.53	4.0	149.7 (2397)	Yes

*Pounds mass.

Table 3.2—Cement and water contents 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/cm*	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)	Pophyry	3.9 (98)	0.59	4920 (33.9)	12.2 (0.141)
Cabrilo	Portugal	370 (220)	195 (116)	Granite	5.9 (150)	0.53	4150 (28.6)	11.2 (0.130)
Salamonde	Portugal	420 (249)	225 (133)	Granite	7.9 (200)	0.54	4250 (29.3)	10.1 (0.118)
Castelo Bode	Portugal	370 (220)	180 (107)	Quartzite	7.9 (200)	0.49	3800 (26.2)	10.3 (0.119)
Rossens	Switzerland	420 (249)	225 (133)	Glacial mixture	2.5 (64)	0.54	5990 (41.3)	14.3 (0.166)
Mauvoisin	Switzerland	319 (189)	162 (96)	Gneiss	3.8 (96)	0.51	4960 (34.2)	15.5 (0.181)
Zervreila	Switzerland	336 (199)	212 (126)	Gneiss	3.8 (96)	0.63	3850 (26.5)	10.5 (0.133)
Hungry Horse	U.S.	188-90 (111-53)	130 (77)	Sandstone	6.0 (150)	0.47	3100 (21.4)	11.2 (0.130)
Glen Canyon	U.S.	118-94 (111-56)	153 (99)	Limestone	6.0 (150)	0.54	3810 (26.3)	13.5 (0.160)
Lower Granite	U.S.	145-49 (86-29)	138 (82)	Basalt	6.0 (150)	0.71	2070 (14.3)	10.7 (0.124)
Libby	U.S.	148-49 (88-29)	133 (79)	Quartzite	6.0 (150)	0.68	2460 (17.0)	12.5 (0.145)
Dworshak	U.S.	211-71 (125-42)	164 (97)	Granite	6.0 (150)	0.58	3050 (21.0)	10.8 (0.126)
Dworshak	U.S.	198-67 (117-40)	164 (97)	Gneiss	6.0 (150)	0.62	2530 (17.4)	9.5 (0.111)
Dworshak	U.S.	168-72 (100-43)	166 (98)	Gneiss	6.0 (150)	0.69	2030 (14.0)	8.5 (0.098)
Dworshak	U.S.	174-46 (130-27)	165 (98)	Gneiss	6.0 (150)	0.75	1920 (13.2)	8.7 (0.084)
Pueblo	U.S.	226-75 (134-44)	168 (100)	Granite, limestone, dolomite	3.5 (89)	0.56	3000* (20.7)	10.0 (0.116)
Crystal	U.S.	390 (231)	183 (109)	Shist and altered volcanics	3.0 (75)	0.47	4000† (27.6)	10.3 (0.119)
Flaming Gorge	U.S.	188-94 (111-56)	149 (88)	Limestone and sandstone	6.0 (150)	0.53	3500 (24.1)	12.4 (0.144)
Krasnoiarsk	U.S.S.R.	388 (230)	213 (126)	Granite	3.9 (100)	0.55	3280 (22.6)	8.5 (0.098)
Ilha Solteira	Brazil	138-46 (82-27)	132 (82)	Quartzite gravel, crushed basalt	6.0 (150)	0.75	3045 (21.0)	16.5 (0.193)
Itaipu	Brazil	182-22 (108-13)	143 (85)	Crushed basalt	6.0 (150)	0.70	2610 (18.0)	12.8 (0.149)
Theodore Roosevelt modification	U.S.	270 (160)	144 (85)	Granite	4.0 (100)	0.53	4500 (31.0)	16.7 (0.194)

*Strength at 80 days.
†Strength at 1 year.

programs for thermal analysis (Polivka and Wilson 1976; USACE 1994a). Determination of tensile strain capacity has also led to a better understanding of the potential for cracking under rapid and slow loading conditions (Houghton 1976).

The specific properties of concrete that 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 should be conducted to ensure a safe structure at lowest cost. In addition, the moisture condition of the specimens and structure, and the loading rate required, should 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.

3.1.1—A compilation of concrete proportion data on representative dams is given in Table 3.1 (Price and Higginson 1963; Ginzburg et al. 1966; ICOLD 1964; Harboe 1961; USBR 1958; Houghton and Hall 1972; Houghton 1970; Houghton 1969). Reference will be made to concrete mixtures described in Table 3.1 and in discussions of properties reported in Tables 3.2 through 3.6.

3.2—Strength

The *w/cm*, to a large extent, governs the quality of the hardened portland-cement binder. Strength, permeability, and most other desirable properties of concrete are improved by lowering the *w/cm*. A study of compressive strength data given in Table 3.2 shows a considerable variation from the direct relationship between the *w/cm* and strength. Factors, totally or partially independent of the *w/cm*, that affect the strength are: composition and fineness of cement, amount and type of pozzolan, surface texture and shape of the aggregate, the mineralogical makeup and strength of the aggregate, aggregate grading, and the improvement of strength by the

Table 3.3—Compressive strength and elastic properties of mass concrete

						Elasticity properties							
		Compressive strength, psi (MPa)				Modulus of elasticity, $E \times 10^6$, psi ($E \times 10^4$, MPa)				Poisson's ratio			
		Age, days				Age, days				Age, days			
No.	Dam	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	—
4	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	—
5	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	—
6	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
7	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
8	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	—	—
9	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	—	—
10	Dworshak*	1200 (8.3)	2030 (14.0)	—	3110 (21.4)	—	3.7 (2.6)	—	3.8 (2.6)	—	—	—	—
11	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	—	—
12	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
13	Peace site*	3060 (21.1)	3939 (27.2)	4506 (31.1)	4666 (32.2)	—	—	—	—	—	—	—	—
14	Theodore Roosevelt modification	2400 (16.5)	4500 (31.0)	5430 (37.4)	5800 (40.0)	4.5 (3.1)	5.4 (3.7)	—	6.2 (4.3)	0.20	0.21	—	0.21

*Water-reducing agent used.

aforementioned admixtures that are attributable to a reduction in the *w/cm*.

High strengths (greater than 5000 psi [34.5 MPa]) are usually not required in mass concrete, except in thin arch dams. Concrete proportioning should determine the minimum cement content required to meet the average compressive strength, as defined by ACI 116R, to give greatest economy and minimum temperature rise. Cement requirements for adequate workability and durability, rather than strength, frequently govern the portland cement content.

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 1 year, and sometimes up to 2 years. Job control cylinders should be tested at an earlier age to be useful in exercising control and maintaining consistency during construction. Job control test specimens are usually 6 x 12 in. (150 x 300 mm) cylinders containing concrete that has been wet-screened to remove aggregate larger than 1-1/2 in. (37.5 mm) maximum size. Correlation tests should 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 up to 36 x 72 in. (900 x 1800 mm) will usually be 80 to 90% of the strength of 6 x 12 in. (150 x 300 mm) cylinders tested at the same age (USBR 2001). Accounting for the continued strength development beyond 28 days, particularly where pozzolans are used, the correlation factors at 1 year may range from 1.15 to 3.0 times the strength of the wet-screened control specimens tested at 28 days.

Accelerated curing procedures set forth in ASTM C 684 yield compression test results in 24 to 48 hours that can provide an indication of potential concrete strength. The use of these procedures, however, 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 a dubious relationship to the actual future strength in the concrete structure, it can be helpful during construction.

There are several complex factors involved in relating results of strength tests on small samples to the probable strength of mass concrete structures that are 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 USBR, and the USACE set the maximum allowable compressive stress for usual loading combinations at 1/3 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 (USBR 1976; USACE 1990).

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 provided in ACI 318:

for f_t and f_c, in psi

$$f_t = 7.5 f_c^{1/2} \tag{3-1a}$$

Table 3.4—Elastic properties of mass concrete

Age of time of loading	Instantaneous and sustained modulus of elasticity,* psi × 10^6 (MPa × 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 year	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 years	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 years				5.6 (3.9)	4.3 (3.0)	4.1 (2.8)									

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

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

for f_t and f_c in MPa

$$f_t = 0.6 f_c^{1/2} \tag{3-1b}$$

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

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, such as dead load or water load. During earthquakes, however, stresses may be fully developed in a small fraction of a second. When loaded at this speed, compressive strength of a concrete for moist specimens may be increased up to 30%, and tensile strength may be increased up to 50%, when compared with values obtained at standard rates of loading (Saucier 1977; Graham 1978; Raphael 1984; Harris et al. 2000).

3.3—Elastic properties

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. The modulus of elasticity, however, is, for practical purposes, considered a constant within the range of stresses to which mass concrete is usually subjected.

The moduli of elasticity of concrete representative of various dams are given in Table 3.3. These values range from 2.8 to 5.5 × 10^6 psi (1.9 to 3.8 × 10^4 MPa) at 28 days and from 3.8 to 6.8 × 10^6 psi (2.6 to 4.7 × 10^4 MPa) at 1 year. Usually, concrete with higher strengths has higher values of elastic modulus and shows a general correlation of increase in modulus with strength. Modulus of elasticity is not directly proportional to strength; however, it is influenced by the modulus of elasticity of the aggregate. In the past, data from concrete modulus of elasticity tests showed relatively high coefficients of variation resulting from attempts to measure small strains on a heterogeneous mixture containing large-size aggregate. Modern electronic devices such as the linear variable differential transformer (LVDT) can measure small length changes with great accuracy. The tensile modulus of elasticity is generally assumed to be identical to the compressive modulus of elasticity.

The Poisson's ratio data given in Table 3.3 tend to range between the values of 0.16 and 0.20, with small increases with increasing time of cure. Extreme values may vary from 0.11 to 0.27. Poisson's ratio, like modulus of elasticity, is influenced by the aggregate, cement paste, and relative proportions of the two.

The growth of internal microcracks in concrete under load starts at compressive stresses equal to approximately 35 to 50% 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 the Poisson's ratio and elastic moduli are no longer constant.

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

Creep of concrete is time-dependent deformation due to a sustained load. Creep appears to be mainly related to the modulus of elasticity of the concrete. Concrete with high values of modulus of elasticity generally have low values of creep deformation. The cement paste is primarily responsible for concrete creep. With concrete containing the same type of aggregate, the magnitude of creep is closely related to the paste content (Polivka et al. 1963) and the *w/cm* of the concrete. ACI 209R discusses the prediction of creep, shrinkage, and temperature effects in concrete structures.

One method of expressing the effect of creep is as the sustained modulus of elasticity of the concrete in which the stress is divided by the total deformation for the time under the load. The instantaneous and sustained modulus of elasticity values obtained on 6 in. (150 mm) diameter cylinders made with mass concrete that is wet screened to remove 1-1/2 in. (37.5 mm) maximum size are recorded in Table 3.4. The instantaneous modulus is measured immediately after the concrete is subjected to loading. The sustained modulus represents values after 365 and 1000 days under loading. Table 3.4 shows that the sustained values for the modulus of elasticity are approximately 1/2 that of the instantaneous modulus when load is applied at early ages, and is a slightly higher percentage of the instantaneous modulus of elasticity when the loading age is 90 days or greater. Creep of concrete appears to be approximately directly proportional to the applied stress-strength ratio, up to approximately 40% of the ultimate strength of the concrete.

3.5—Volume change

Volume change is caused by changes in the 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 concrete are given in Table 3.5. Various factors influencing cracking of mass concrete are discussed in ACI 207.2R and USACE (1997).

Drying shrinkage ranges from less than 0.02% (or 200 millionths) for low-slump lean concrete with good-quality aggregates to over 0.10% (or 1000 millionths) for rich mortars, or concrete containing poor-quality aggregates and an excessive amount of water (Neville 1996). Drying shrinkage is caused by the loss of moisture from the cement paste constituent, which can shrink as much as 1%. Fortunately, aggregate provides internal restraint that reduces the magnitude of this volume change to about 0.06% (ACI 224.1R). The amount of drying shrinkage is influenced mainly by the volume and type of aggregate and the water content of the mixture. 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.

Autogenous volume change is a change in volume produced by continued hydration of cement, exclusive of effects of applied load and change in either thermal condition or moisture content. Unlike drying shrinkage, it is unrelated to the amount of water in the mixture. The net autogenous volume change of most concrete is a shrinkage of 0 to 150 millionths. When autogenous expansion occurs, it usually takes place within the first 30 days after placing. Concrete containing pozzolans may sometimes have greater autogenous shrinkage than portland-cement concrete without pozzolans (Houk et al. 1969).

The thermal coefficient of expansion of a concrete depends mainly on the type and amount of coarse aggregate in the concrete. Various mineral aggregates may range in thermal coefficients from less than 2 to more than 8 millionths per °F (3 to 14 millionths per °C). Neat cement pastes will vary from about 6 to 12 millionths per °F (10 millionths to 21 millionths per °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 to work with smaller-size specimens. 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 mixture. Refer to ACI 207.2R for thermal properties of concrete. 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

Table 3.5—Volume change and permeability of mass concrete

Structure	Autogenous volume change, 90 days, millionths	Autogenous volume change, 1 year, millionths	Drying shrinkage, 1 year, millionths	Permeability, K ft/s/ft*, Hydraulic head	Permeability, 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}
Glen Canyon	–32	–61	–459	5.74×10^{-12}	5.33×10^{-13}
Flaming Gorge	—	—	–496	3.52×10^{-11}	3.27×10^{-12}
Yellowtail	–12	–38	–345	6.25×10^{-12}	5.81×10^{-13}
Dworshak	+10	–8	–510	6.02×10^{-12}	5.59×10^{-13}
Libby	+3	+12	–480	1.49×10^{-11}	1.38×10^{-12}
Lower Granite	+4	+4	—	—	—

*ft/s/ft = ft³/ft² – s/ft of hydraulic head; m/s/m = m³/m² – s/m of hydraulic head; millionths = in. × 10^{-6}/in. (mm × 10^{-6}/mm), measured in linear length change.

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

Table 3.6—Shear properties of concrete (triaxial tests)

Dam	Age, days	w/cm	Compressive strength psi	Compressive strength MPa	Cohesion psi	Cohesion MPa	tanφ	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
Monticello	28	0.62	2800	19.3	610	4.2	0.93	0.218
	40	0.92	4120	28.4	950	6.6	0.85	0.231
Shasta	28	0.50	5740	39.6	1140	7.9	1.05	0.199
	28	0.60	4920	33.9	1060	7.3	0.95	0.215
	90	0.50	5450	37.6	1090	7.5	1.05	0.200
	90	0.50	6590	45.4	1360	9.4	1.01	0.206
	90	0.60	5000	34.5	1040	7.2	1.00	0.208
	245	0.50	6120	42.2	1230	8.5	1.04	0.201
Dworshak	180†	0.59	4150	28.6	1490	10.3	0.44	0.359
	180†	0.63	3220	22.2	1080	7.4	0.46	0.335
	180†	0.70	2420	16.7	950	6.6	0.43	0.393
	200‡	0.59	2920	20.1	720	5.0	0.84	0.247

*Cohesion divided by compressive strength.
Notes: 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 18 x 36 in. (450 x 900 mm) test specimens sealed to prevent drying, and "‡" designates 18 x 36 in. (450 x 900 mm) test specimens sealed to prevent drying, with 6 in. (150 mm) maximum-size aggregate.
§Triaxial tests.

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, or the tensile strain developed exceeds the tensile strain capacity of the concrete, cracking will occur (Houghton 1972; Houghton 1976; Dusinberre 1945). Principal methods used to reduce the potential for thermally induced cracking in concrete are outlined in ACI 224R and Carlson et al. (1979). Such methods include reducing the maximum internal temperature that the concrete attains, reducing the rate at which the concrete cools, and increasing the tensile strength of the concrete. Concrete's resistance to cracking can be equated to tensile strain capacity 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).

Volume changes can also result from chemical reactions, which can be potentially disruptive.

3.6—Permeability

Concrete has an 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 w/cm. Therefore, low w/cm 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.

3.7—Thermal properties

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, along with restraint, result in tensile strains and stresses that may cause cracking that is detrimental to the structure. Thermal properties that influence this behavior in mass concrete are specific heat, conductivity, and diffusivity. The primary factor affecting the thermal properties of a concrete, however, is the mineralogical composition of the aggregate (Rhodes 1978). Requirements for cement, pozzolan, percent sand, and water content are modifying factors, but offer a negligible effect on thermal properties. Entrained air is an insulator and reduces thermal conductivity, but other considerations that govern the use of entrained air outweigh the significance of its effect on thermal properties. Thermal property values for some mass concrete, an extensive discussion on thermal properties and behavior, and example computations are provided in ACI 207.2R.

3.8—Shear properties

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 concrete containing 1-1/2 in. (37.5 mm) maximum size aggregates are listed in Table 3.6. 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 biaxial shear strength (USBR 1992). Past criteria have stated that the coefficient of internal friction can be taken as 1.0 and cohesion as 10% of the compressive strength (USBR 1976). More recent investigation has concluded that assuming this level of cohesion may be unconservative (McLean and Pierce 1988).

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; $\tan\phi$, which is the slope of the line, represents the coefficient of internal friction. X and Y are normal and shear stresses, respectively. In many cases, the shear strengths in Table 3.6 were higher for older specimens; however, no definite trend is evident (Harboe 1961). The ratio of triaxial shear strength to compressive strength varies from 0.19 to 0.39 for the various concretes shown. When shear strength is used for design, the 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.

Bonded horizontal construction joints may have shear strength comparable to that of the parent concrete. Unbonded joints typically have lower cohesion, but the same coefficient of internal friction, when compared with the parent concrete. If no tests are conducted, the coefficient of internal friction can be taken at 1.0 and the cohesion as 0 for unbonded joints. For bonded joints, the coefficient of internal friction can be taken as 1.0, while the cohesion may approach that of the parent concrete (McLean and Pierce 1988).

3.9—Durability

A durable concrete is one that has the ability to resist weathering action, chemical attack, abrasion, and other conditions of service (ACI 116R). Laboratory tests can indicate relative durabilities of concrete, but it is usually not possible to directly predict durability in field service from laboratory durability studies.

Disintegration of concrete by weathering is mainly caused 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 high-quality materials, use of entrained air, low *w/cm*, proper mixture proportioning, proper placement techniques to provide a watertight structure, and good water curing usually provide a concrete that has excellent resistance to weathering action.

Chemical attack occurs from exposure to acid waters, exposure to sulfate-bearing waters, and 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 sulfoaluminates. These reactions are accompanied by considerable expansion and disruption of the concrete. Concrete containing cement low in tricalcium aluminate (ASTM Types II, IV, and V) is more resistant to attack by sulfates.

Hydrated lime is one of the products formed when cement and water combine in concrete. This lime is readily dissolved in pure or slightly acidic water that may occur in high mountain streams. Pozzolans that 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.

Alkali-aggregate reaction is the chemical reaction between alkalies (sodium and potassium) from portland cement or other sources and certain constituents of some aggregates that, under certain conditions, produces deleterious expansion of the concrete. These reactions include alkali-silica reaction and alkali-carbonate rock reaction (ACI 221.1R) (USACE 1994b; Farny and Kosmatka 1997). 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.

The principal causes of erosion of concrete surfaces are cavitation and the movement of abrasive material by flowing water. Use of increased-strength and wear-resistant concrete offers some relief, but the best solution lies in the prevention, elimination, or reduction of the causes by proper design, construction, and operation of the concrete structure (ACI 210R). The use of aeration in high-velocity flows is an effective way to prevent cavitation.

CHAPTER 4—CONSTRUCTION

4.1—Batching

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 presents information on selection and use of aggregates in concrete. ACI 304R presents information on the handling, measuring, and batching of all the materials used in making concrete.

The desirability of restricting the temperature rise of mass concrete by limiting the cement content of the mixture creates a continuing construction problem to maintain workability in the plastic concrete. Efficient mixtures for mass concrete contain unusually low portions of cementitious materials, sand, and water. Thus, the workability of these mixtures 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 use specialty equipment and

Table 4.1—Typical batching tolerances

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

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

efficient construction methods. Consistency in the batching can be improved by the following measures:

- Finish screening of coarse aggregate at the batching plant, preferably on horizontal vibrating screens without intermediate storage;
- Calibration of the scale range that is appropriate for the range of batch weights to be used;
- Automatic weighing and material flow cutoff features;
- Interlocks to prevent recharging when some material remains in a scale hopper;
- A device for instant reading of approximate moisture content of sand; and
- Equipment capable of instant automatic selection and setting of numerous batch ingredients in many different mixture proportions. In large central plant mixers, the large batches commonly used for mass concrete also tend to minimize the effect of variations.

Because 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 the most efficient use of ice, it should be less than 32 °F (0 °C), and be brittle-hard, dry, and finely broken. For maximum efficiency, 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.

Liquid admixtures are generally batched by volume, although weighing equipment has also been used successfully. Reliable admixture batching equipment is available from admixture or batch plant manufacturers. Means should be provided for making a visual accuracy check. Provisions should be made for preventing batching of admixture while the discharge valve is open. Interlocks should also be provided that will prevent inadvertent over-batching of the admixture. Particularly with air-entraining and water-reducing admixtures, any irregularities in batching can cause troublesome variations in slump, air content, or both. When several liquid admixtures are to be used, they should be batched separately into the mixer. The use of comparatively dilute solutions reduces gumming in the equipment. For continuing good operation, equipment should be maintained and kept clean. Timed-flow systems should not be used. Also, it is important to provide winter protection for storage tanks and related delivery lines where necessary. Table 4.1 shows batching tolerances frequently used.

4.2—Mixing

Mixers for mass concrete should be capable of discharging low-slump concrete quickly and with consistent distribution of large aggregate throughout the batch. This is best accomplished with large, tilting mixers in stationary central plants. The most common capacity of the mixer drum is 4 yd^3 (3 m^3), but good results have been achieved with mixers as small as 2 yd^3 (1.5 m^3) and as large as 12 yd^3 (9 m^3). Truck mixers are not suited to the mixing and discharging of low-slump, large-aggregate concrete. Turbine-type mixers may be used for mass concrete containing 3 in. (75 mm) aggregate.

Specifications for mixing time range from a minimum of 1 min for the first cubic yard plus 15 s for each additional cubic yard (80 s for first cubic meter plus 20 s for each additional cubic meter) of mixer capacity (ACI 304R; ASTM C 94) to 1-1/2 min for the first 2 yd^3 plus 30 s for each additional cubic yard (1-1/2 min for the first 1-1/2 m^3 plus 40 s for each additional cubic meter) of capacity (USBR 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 on the results of mixer performance tests. Criteria for these tests are found in ASTM C 94. Mixing time is best controlled by a timing device that prevents release of the discharge mechanism until the mixing time has elapsed.

During mixing, the batch should be closely observed to ensure the desired slump. Amperage meters can also be used to assist visual observations. The operator and inspector should be alert and attentive. ACI 311.5R provides recommendations for plant inspection and process quality control testing. Tuthill (1950) discussed effective inspection procedures and facilities. Preferably, the operator should be stationed in the plant where he or she 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, the operator can immediately compensate with a little more water and maintain the desired slump. Lacking this arrangement to see into the mixer, the operator should be able to see the batch as it is discharged, note any change from former batches, and make subsequent water adjustments accordingly. A sand moisture meter provides a quick method to compare the moisture content of sand entering the mixer with the sand tested in stockpile. A significant difference in moisture content will alert the operator that variations in sand moisture may be occurring and initiate additional testing or a change in batching procedure.

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

Placing includes preparation of horizontal construction joints, transportation, handling, placement, and consolidation of the concrete (ACI 304R; USBR 2001).

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 before 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 approximately 0.1 in. (a few millimeters) of inferior material will reveal a satisfactory surface.

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 should be on guard to avoid harm to other personnel and to wooden surfaces from water-blasted pieces of surface material that may be hurled forward with great force and velocity. Sandblasting has the advantage of being able to clean concrete of any age, but it 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 it may not work as efficiently after the concrete is more than 1 week old. Before and after horizontal construction joint cleanup with sandblasting and high-pressure water blasting are illustrated in Fig. 4.1(a) and (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 at about the time the concrete approaches final set, is also used. It may not, however, be possible to preserve the initially clean surface until concrete is placed on 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.

(a) Sandblast treatment

(b) High-pressure water-blast treatment

Fig. 4.1—Before and after horizontal construction joint cleanup.

The clean concrete surface should be approaching dryness and be free from surface moisture at the time new concrete is placed on it (USACE 1959, 1963a; Tynes and McClease 1973; Neeley and Poole 1996; Neeley et al. 1998). Testing has shown superior strength and watertightness of joints that are dry and clean when the overlying concrete is placed. In this condition, 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 permeability of the joint (Houghton and Hall 1972). The mortar coat, although widely used in the past, is no longer commonly used in mass concrete 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.

Fig. 4.2—Placement of mass concrete by conveyor belt.

Selection of equipment for transporting and placing mass concrete is strongly influenced by the maximum size of the aggregate. Concrete for mass placements, such as in dams, often contains cobbles, which are defined as coarse-aggregate particles larger than 3 in. (75 mm) and smaller than 12 in. (300 mm). The tendency of cobbles to segregate from the mixture as a result of their greater inertia when in motion may dictate the use of large, 2 to 12 yd^3 (1.5 to 9 m^3) capacity buckets. Railcars, trucks, cableways, cranes, or some combination of these, may be used to deliver the buckets to the point of placement. For concrete containing coarse aggregate 3 in. (75 mm) and larger, a bucket size of 4 to 8 yd^3 (3 to 6 m^3) is preferable because smaller buckets do not discharge as readily, and each delivery is too small to work well with a high-production placement scheme. On the other hand, the 12 yd^3 (9 m^3) bucket puts such a large pile in one place that much of the crew's time is devoted to vibrating for spreading instead of for consolidation. To preclude these piles being larger than 4 yd^3 (3 m^3), one agency requires controllable discharge gates in buckets carrying more than 4 yd^3 (3 m^3). Extra care should be taken to ensure ample vibration deep in the center of these piles and at points of contact with concrete previously placed. Mass concrete of proper mixture proportions and low slump does not separate by settlement during such transportation over the short distances usually involved. Care should be taken, however, to prevent segregation at each transfer point.

Mass concrete may also be transported in dumping rail cars and trucks and placed by use of conveyors. Cranes equipped with telescoping conveyors, termed "creter cranes," are widely used for the placement of modern mass concrete (Fig. 4.2). Placing mass concrete with conveyors has been most successful and economical when the aggregate size is 4 in. (100 mm) or less. The point of discharge from conveyors should be managed so that concrete is discharged onto fresh concrete and immediately vibrated to prevent stacking. Placement of mass concrete by conveyor is shown in Fig. 4.2. Additional information on placing concrete with conveyors is contained in ACI 304.4R.

Large building foundations and other very large monolithic concrete structures are considered 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 on its ability to successfully place concrete that has been proportioned for mass concrete considerations as defined in Section 2.7, which emphasizes the reduction of heat evolution. Placing capacity should be great enough to avoid cold joints and undesirable exposure to extremes of heat and cold at lift surfaces. This is usually accomplished by using many pieces of placing equipment. Additional information on pumping of concrete is contained in ACI 304.2R.

Mass concrete is best placed in successive layers. The maximum thickness of the layer depends on the ability of the vibrators to properly consolidate the concrete. Six in. (150 mm) diameter vibrators produce satisfactory results with 4 to 6 in. (100 to 150 mm) nominal maximum-size aggregate 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) nominal maximum-size aggregate and less than 2 in. (50 mm) slump placed in 12 to 15 in. (300 to 380 mm) layers with smaller buckets. Shallower layers, rather than deeper layers, give better assurance of satisfactory consolidation and freedom from rock pockets at joint lines, corners, and other form faces, as well as within the block itself.

The layer thickness should be an even fraction of the lift height or of the depth of the block. The layers are carried forward in a stair-step fashion in the block by means of successive discharges, so there will be a setback of approximately 5 ft (1.5 m) between the forward edges of successive layers. Placement of the steps is organized to expose a minimum of surface to lessen warming of the concrete in warm weather and reduce the area affected by rain 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.

Vibration is the key to the successful placement of mass concrete, particularly when the concrete is low slump and contains large aggregate (Tuthill 1953). Ineffective equipment is more costly to the builder because of a slower placing rate and the hazard of poor consolidation. Vibration should be systematic and should thoroughly cover and deeply penetrate each layer. Particular attention should be paid to ensure full vibration where the perimeters of two discharges join because the outer edge of the first batch is not vibrated until the next batch is placed against it. The two discharges can then be vibrated monolithically together without causing either edge to flow downward. Proper vibration of large aggregate mass concrete is shown in Fig. 4.3. To ensure proper consolidation, the vibrators should penetrate the lower layer for 2 to 4 in. (50 to 100 mm) and be maintained in a vertical position at each penetration during vibration. To prevent imperfections along lift lines and layer lines at form faces, these areas should be systematically deeply revibrated as each layer advances from the starting form, along each of the side forms, to the other end form. Any visible clusters of separated coarse aggregate should be scattered on the new concrete before covering with additional concrete. Vibration is unlikely to fill and solidify unseparated aggregate clusters with mortar. During consolidation, the vibrators should remain at each penetration point until large air bubbles have ceased to rise and escape from the concrete. The average time for one vibrator to fully consolidate a 1 yd^3 (3/4 m^3) of concrete may be as much as 1 min (80 s for 1 m^3). Overvibration 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 should be available and in use during the placement of mass concrete. Specific recommendations for mass concrete vibration are contained in ACI 309R.

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

Fig. 4.3—Consolidation of low-slump mass concrete placed by bucket.

4.4—Curing

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 ensure that the joint surface is free of water but still damp and clean before new concrete is placed. The use of a liquid-membrane curing compound is not the best method of curing mass concrete, but is applicable where moist curing is not practical, such as in below-freezing conditions or where the application of water may damage prepared foundations or impede work. If used on construction joints, it should be completely removed by sandblasting or water-blasting to prevent reduction or loss of bond.

4.5—Forms

Forms for mass concrete have the same basic requirements for strength, mortar-tightness, accuracy of position, and generally good surface condition as those described in Hurd (1995). 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 on the methods used and the care exercised in placing concrete adjacent to the form. For this reason, it is recommended that 100% of equivalent hydrostatic pressure plus 25% for impact be used for design of mass concrete forms.

Form ties connected to standard anchors in the previous lift and braces have long been used. Many large jobs are now equipped with forms supported by cantilevered strongbacks anchored firmly into the lift below. Additional support of cantilevered forms may be provided by form ties, particularly when the concrete is low in early strength. Cantilevered forms are raised by hydraulic, air, or electric jacking systems. Care should be taken to avoid spalling concrete around the anchor bolts in the low-early-strength concrete of the lift being stripped of forms because these bolts will be used to provide horizontal restraint in the next form setup. High-lift, mass concrete formwork is comparable to that used for standard structural concrete work except that ties may be 20 to 40 ft (6 to 12 m) long across the lift rather than 20 to 40 in. (0.5 to 1.0 m). To facilitate placement by bucket, widely spaced large-diameter, high-tensile-strength ties should be used to permit passage of the concrete buckets.

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

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

A common forming problem for spillway sections of gravity dams is encountered in the sloping and curved portions of the crest and bucket. These slopes range from horizontal to approximately 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 is thoroughly consolidated by means of temporary holding forms rather than using screed guides and strikeoff. With no strikeoff involved, the regular mass concrete face mixture is as readily used as one with small aggregate, unless a different concrete mixture 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. The rows of form panels are 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

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 past experience of hardened concrete temperature 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 mixtures 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 the time interval between lifts. In such extreme cases, continuous placing in high lifts may be preferable, especially as a means of minimizing joint cleanup, preventing cracking, or permitting the use of slipforms, such as for massive piers. In large blocks, such as in dam construction, the loss of heat from a 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.

Control of temperature rise is a design function; therefore, lift heights and placing frequency should be shown on drawings and in specifications. 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. Lift heights range from 2-1/2 ft (0.75 m) for multiple lifts just above foundations to 5 and 7-1/2 ft (1.5 and 2.3 m) in many gravity dams, and to 10 ft (3 m) or more in thin arch dams, piers, and abutments.

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 et al. 1963). With precooled 50 °F (10 °C) mass concrete of low cement content in a warm climate, ambient heat removes the advantage of shallower lifts and is the reason 7-1/2 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

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%. Steel aggregate storage bins and aggregate piles should be shaded as illustrated in Fig. 4.4. Aggregates can be cooled by evaporation through vacuum, inundation in cold water, cold air circulation (ACI 207.4R; ACI 305R), or liquid nitrogen. Figure 4.5 shows the cooling of coarse aggregate by spraying and inundation with chilled water immediately before 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, managing placement so that minimum areas are exposed, and, if placement will be 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 ensure at least the minimum opening of contraction joints needed when grouting joints in dams is necessary. Aggregate and concrete precooling, insulation, protection from high ambient temperature, and postcooling considerations and recommendations are provided in ACI 207.4R.

4.8—Instrumentation

The specific goals of data collection, transmittal, processing, review, and action procedures are to provide accurate and timely evaluation of data for potential remedial action relating to the

Fig. 4.4—Metal cover over drained fine aggregate stockpile to reduce heat absorption.

Fig. 4.5—Cooling coarse aggregate by chilled water spray and inundation.

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.

Instrumentation should be part of the design and construction of any mass concrete structure wherever a future question may arise concerning the safety of the structure. Also, preparations essential for an accurate evaluation of the instrumentation results should be made through long-term, laboratory-sample studies to determine progressive age relationships for properties of the actual project concrete (refer to Chapter 3).

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. Instruments installed in mass concrete in the United States have been primarily of the unbonded resistance-wire or Carlson-type meter and vibrating wire, although a wide variety of instruments is being incorporated in current projects. Some of the instruments available for use are: hydrostatic pressure measuring devices, pressure or stress measuring devices, seepage measurement devices, internal movement measuring devices, surface movement measuring devices, and vibration measuring devices. Instrumentation systems should include provisions for automated collection of instrumentation data, remote access to data, and, where applicable, real-time monitoring of structural performance. Several manuals on instrumentation of concrete dams are available (USACE 1980, 1985; USSD 2002; U.S. Bureau of Reclamation 1987).

Hydrostatic pressure measuring devices—These are generally piezometers, operating either as closed or open systems, or closed-system Bourdon-type pressure monitoring systems. Closed-system piezometers consist of vibrating-wire units or Carlson-type devices, whereas 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, flow meters, 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, foundations, or both. The devices consist of essentially horizontal and vertical measurements using calibrated tapes, single-point and multipoint borehole extensometers, joint meters, plumb lines, 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 surfaces 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, on 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, electronic distance measuring (EDM) devices, and associated rods or targets.

Vibration measuring devices—Various commercially available instruments include the strong motion accelerograph and the peak-recording accelerograph.

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 arranged so that when subjected to the action to be measured, one set increases in tension, while the other decreases. A test set, based on 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.

To properly monitor the performance of a mass concrete structure, it is often necessary to collect instrumentation data over extended periods. The monitoring equipment should be as simple, rugged, and durable as possible and be maintained in satisfactory operating condition. The instruments should be rugged enough to be embedded in fresh concrete. When measuring strain, the instruments should be at least three times the length of the largest particle in the fresh concrete. Because they contain electrical-sensing elements, they should not only be waterproof, but all material should 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, 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 noncorrosive material should be used wherever possible.

Concrete surfaces may be subjected to excessive stresses and cracking that will make stress or strain measurements obtained from surface-mounted instrumentation meaningless. Reliable measurements of strain and stress should 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 may be calibrated on a regular basis; however, most instrumentation is fixed in place and is 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.

4.9—Grouting contraction joints

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 because 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.

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. In recent years, however, the appearance of some transverse cracks, generally parallel to the contraction joints, has prompted reconsideration of the grouting of contraction joints in gravity dams. 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 before reservoir filling if at all possible. If the transverse joints are filled with grout, a surface crack opening somewhere on the upstream face may have effective resistance against propagation and further opening.

Where there is reason to grout contraction joints, a program of precooling and postcooling should be arranged to provide a joint opening of at least 0.04 in. (1 mm) to ensure 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. Warner (2004) describes the grouting systems and grouting operations for grouting contraction joints. Silveira et al. (1982) describe a grouting system that employs packers to permit reuse of the piping system. The use of embedded instrumentation across the joint is the only way to accurately determine the magnitude of the joint opening (Carlson 1979; Silveira et al. 1982).

CHAPTER 5—REFERENCES

5.1—Referenced standards and reports

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 Mass Concrete
207.4R	Cooling and Insulating Systems for Mass Concrete
207.5R	Roller-Compacted Mass Concrete
209R	Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
210R	Erosion 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 and Heavyweight Aggregates in Concrete
221.1R	State-of-the-Art Report on Alkali-Aggregate Reactivity
224R	Control of Cracking in Concrete Structures
224.1R	Causes, Evaluation and Repair of Cracks in Concrete Structures
232.2R	Use of Fly Ash in Concrete
233R	Slag Cement in Concrete and Mortar
304R	Guide 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
309R	Guide for Consolidation of Concrete
311.5	Guide for Concrete Plant Inspection and Testing of Ready-Mixed Concrete
318/318R	Building Code Requirements for Structural Concrete and Commentary

ASTM International

C 94	Standard Specification for Ready-Mixed Concrete
C 125	Standard Terminology 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 Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
C 684 Standard Method for Making, Accelerated Curing, and Testing for Concrete Compression Test Specimens
C 989 Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
C 1157 Standard Performance Specification for Hydraulic Cement

These publications may be obtained from the following organizations:

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

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428
www.astm.org

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