

Report on Thermal and Volume Change Effects on Cracking of Mass Concrete

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Report on Thermal and Volume Change Effects on Cracking of Mass Concrete

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This report presents a discussion of the effects of heat generation and volume change on the design and behavior of mass concrete elements and structures. Emphasis is placed on the effects of restraint on cracking and the effects of controlled placing temperatures, concrete strength requirements, and material properties on volume change.

Keywords: adiabatic; cement; concrete cracking; creep; drying shrinkage; foundation; heat of hydration; mass concrete; modulus of elasticity; placing; portland cement; pozzolan; restraint; stress; temperature; tensile strength; thermal expansion; volume change.

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CHAPTER 1—INTRODUCTION

1.1—Scope

This report is primarily concerned with evaluating the thermal behavior of mass concrete structures to control the cracking in members that occurs principally from thermal contraction with restraint. This report presents a detailed discussion of the effects of heat generation and volume changes on the design and behavior of mass concrete elements and structures, a variety of methods to compute heat dissipation and volume changes, and an approach to determine mass and surface gradient stresses. It is written primarily to provide guidance for the selection of concrete materials, mixture requirements, and construction procedures necessary to control the size and spacing of cracks. The quality of concrete for resistance to weathering is not emphasized in recommending reduced cement contents; however, it should be understood that the concrete should be sufficiently durable to resist expected service conditions. This report can be applied to most concrete structures with a potential for unacceptable cracking. Its general application has been to massive concrete members 18 in. (460 mm) or more in thickness; it is also relevant for less massive concrete members.

1.2—Mass concrete versus structural concrete

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 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 large temperature changes in the structure and significant temperature differentials between the interior and the outside surface of the structure. The accompanying volume-change differentials and restraint result in tensile strains and stresses that may cause cracking detrimental to the structural design, the serviceability, or the appearance.

In most structural concrete construction, most of the heat generated by the hydrating cement is rapidly dissipated, and only slight temperature differences develop. For example, a concrete wall 6 in. (150 mm) thick can become thermally stable in approximately 1-1/2 hours. A 5 ft (1.5 m) thick wall would require a week to reach a comparable condition. A 50 ft (15 m) thick wall, which could represent the thickness of an arch dam, would require 2 years. A 500 ft (152 m) thick dam, such as Hoover, Shasta, or Grand Coulee, would take approximately 200 years to achieve the same degree of thermal stability. Temperature differentials never become large in typical structural building elements and, therefore, typical structural building elements are relatively free from thermal cracking. In contrast, as thickness increases, the uncontrolled interior temperature rise in mass concrete becomes almost adiabatic, and this creates the potential for large temperature differentials that, if not accommodated, can impair structural integrity.

There are many concrete placements considered to be structural concrete that could be significantly improved if some of the mass concrete measures presented in this report

were implemented. Measures include consideration of issues such as required concrete strengths, age when strength is required, cement contents, supplemental cementitious materials, temperature controls, and jointing.

1.3—Approaches for crack control

If cementitious materials did not generate heat as the concrete hardens, if the concrete did not undergo volume changes with changes in temperature, and if the concrete did not develop stiffness (high modulus of elasticity), there would be little need for temperature control. In the majority of instances, this heat generation and accompanying temperature rise will occur rapidly before the development of elastic properties and, consequently, little or no stress development during this phase. A continuing rise in temperature for many more days is concurrent with the increase in elastic modulus (rigidity). Even these circumstances would be of little concern if the entire mass of the placement could:

1. Be limited in maximum temperature to a value close to its final, cooled, stable temperature;
2. Be maintained at the same temperature throughout its volume, including exposed surfaces;
3. Be supported without restraint (or supported on foundations expanding and contracting in the same manner as the concrete);
4. Relieve its stress through creep; and
5. Have no stiffness or rigidity.

None of these conditions, of course, can be achieved completely. The first and second conditions (such as temperature controls) can be realized to some extent in most construction. The third condition (such as limited restraint) is the most difficult to obtain, but has been accomplished on a limited scale for extremely critical structures by preheating the previously placed concrete to limit the differential between older concrete and the maximum temperature expected in the covering concrete. The fourth and fifth conditions can be somewhat influenced if there is an option to use lower-strength concrete and aggregates with lower coefficients of thermal expansion and lower modulus. This report provides discussion and explanation about these issues and other issues related to controlling thermal volume changes and subsequent cracking.

All concrete elements and structures are subject to volume change in varying degrees dependent upon the makeup, configuration, and environment of the concrete. Uniform volume change will not produce cracking if the element or structure is relatively free to change volume in all directions. This is rarely the case for massive concrete members because size alone usually causes nonuniform change, and there is often sufficient restraint either internally or externally to produce cracking.

The measures used to control cracking depend, to a large extent, on the economics of the situation and the seriousness of cracking if not controlled. Cracks are objectionable where their size and spacing compromise the strength, stability, serviceability, function, or appearance of the structure.

While cracks should be controlled to the minimum practical width in all structures, the economic of achieving this goal

should be considered. The change in volume can be minimized or controlled by such measures as reducing cement content, replacing part of the cement with pozzolans, precooling, post-cooling, insulating to control the rate of heat absorbed or lost, and by other temperature control measures outlined in ACI 207.4R. Restraint is modified by installing joints to permit controlled contraction or expansion and also by controlling the rate at which volume change takes place. Construction joints may also be used to reduce the number of uncontrolled cracks that may otherwise be expected. By appropriate consideration of the preceding measures, it is usually possible to control cracking or at least to minimize the crack widths.

In the design of reinforced concrete structures, cracking is presumed mitigated through the effective placement of reinforcement. For this reason, the designer does not normally distinguish between tension cracks due to volume change and those due to flexure. Instead of employing many of the previously recommended measures to control volume change, the designer may choose to add sufficient reinforcement to distribute the cracking so that one large crack is replaced by many smaller cracks of acceptably smaller widths. The selection of the necessary amount and spacing of reinforcement to accomplish this depends on the extent of the expected volume change, the spacing or number of cracks that would occur without the reinforcement, and the ability of reinforcement to distribute such cracks.

The degree to which the designer will either reduce volume changes or use reinforcement for control of cracks in a given structure depends largely on the massiveness of the structure itself and on the magnitude of forces restraining volume change. No clear-cut line can be drawn to establish the extent to which measures should be taken to control the change in volume. Design strength requirements, placing restrictions, and the environment itself are sometimes so severe that it is impractical to mitigate cracking solely by measures to minimize volume change. On the other hand, fortunately, the designer normally has a wide range of choices when selecting design strengths and structural dimensions.

In many cases, the cost of increased structural dimensions required by the selection of lower-strength concrete (within the limits of durability requirements) is more than repaid by the savings in reinforcing steel, reduced placing costs, and the savings in material cost of the concrete itself. Recommendations for reinforcement of mass concrete elements are not a part of this report.

CHAPTER 2—THERMAL BEHAVIOR

2.1—General

In mass concrete, thermal strains and stresses develop by a change in the mass concrete volume. The two primary causes of such a volume change are from the generation and dissipation of the heat of cement hydration and from periodic cycles of ambient temperature. Consequently, the measures to reduce mass concrete volume changes include reducing the heat generated by the hydration of the cement and reducing the initial placing temperature of the mixture.

All cements, as they hydrate, cause concrete to heat up to some degree. This temperature rise can be minimized by the

use of minimal cement contents in the mixture, partial substitution of pozzolans for cement, and use of special types of cement with lower or delayed heat of hydration. To minimize the heat generated by the mixture, mass concrete mixtures are designed to minimize the cement content. Typically, the cement requirements for mass concrete mixtures are usually much less than those for general concrete work; hence, temperature rise is also less. The tensile stress and cracking can be reduced to zero if the initial temperature of the concrete is set below the final stable temperature of the structure by the amount of the potential temperature rise. Theoretically, this is possible; however, it is not practical except in hot climates. Economy in construction can be gained if the initial temperature is set slightly above this zero stress initial temperature so that a slight temperature drop is allowed such that the tensile stresses built up during this temperature drop are less than the tensile strength of the concrete at that time (or such that the tensile strains are less than the tensile strain capacity of the concrete at that time).

ACI 207.4R describes methods for reducing the initial temperature of concrete and the benefits of placing cold concrete. If the maximum internal temperature of a large mass concrete structure is above that of the final stable temperature of the mass, volume changes in massive structures will take place continuously for decades. Structures that require more rapid volume change so construction operations can be concluded as soon as possible may require that the internal heat be removed artificially. The usual method is by circulating a cooling medium in embedded pipes.

2.2—Thermal gradients

Volume changes are a direct result of temperature changes in the structure. The temperature changes along a particular path or through a section of a structure are called thermal gradients. Thermal gradients are determined by establishing the time history of temperature for a specific path through a structure. Thermal gradients are categorized as either mass gradients or surface gradients. Mass gradient is the differential temperature between that of a concrete mass and a restraining foundation. The long-term maximum internal temperature change of a large concrete mass as it cools from an internal peak temperature to a stable temperature equal to approximately the annual average temperature is a mass gradient. The properties of the mass concrete, the foundation rock, and the contact between the concrete and the rock along with the geometry of the structure determine how a mass gradient and its consequent volume change result in strains and stresses that can cause cracking.

Surface gradients are the result of cooling of the surface concrete relative to the more stable internal temperature. As this surface “skin” contracts with cooling, tension is created in the skin concrete that results in cracking. In this case, the interior becomes the restraining surface against which the surface concrete reacts. Surface gradient cracking is often limited to shallow depths; however, conditions can develop where surface cracking penetrates deeply into the structure and, when combined with mass gradient volume changes or other load conditions, may compound cracking conditions.

The behavior of exposed surfaces of concrete is greatly affected by daily and annual cycles of ambient temperature (ACI 305R). At the concrete surface, the temperature of the concrete is almost identical to the air temperature. Consequently, the temperature variation of the concrete at the surface is the same as the daily air temperature variation. At a depth of 2 ft (0.6 m) from the surface, the variation in concrete temperature is much less than the air temperature variation, possibly only 10% of the daily surface temperature variation. Interior concrete will exhibit even less temperature variation from air temperature variations that occur over a longer period of time, (for example, seasonal variations in temperature).

Likewise, stresses vary in the same manner. In a location where the surface temperature varies annually by 100 °F (59 °C) and the concrete has a modulus of elasticity of 4.0×10^6 psi (2.8×10^4 MPa) before cracking, the surface could be subjected to stresses approximately 1000 psi (7 MPa) above and below the average. While concrete can quite easily sustain 1000 psi (7 MPa) in compression, its tensile strength is much lower, and cracking would be likely. Because of the rapid deterioration of the temperature differential with distance from the surface, however, the variation in stress is likewise dissipated rapidly, with the result that surface cracking due to ambient temperature changes originates in and is usually confined to a relatively shallow region at and near the surface. In a massive structure, such as a dam, where a uniform construction schedule is followed that minimizes the exposure of concrete monolith surfaces and lift surfaces, the surface concrete, although superficially cracked by ambient temperature cycles, can protect the structural integrity of the concrete below it. Where there is an interruption to the construction schedule and time intervals between lifts become overly extended, lift surface cracking may become deep and require treatment to prevent propagation into subsequent placements.

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

CHAPTER 3—PROPERTIES

3.1—General

This chapter discusses the principal properties of massive concrete that affect the control of cracking and provides guidance to evaluate those properties. The tables of Chapter 3

of ACI 207.1R list properties affecting volume change for a number of dams. Table 3.5 of that report notes that values for drying shrinkage, autogenous volume change, and permeability are results of tests on quite small specimens and, except for the permeability specimens, none contained mass concrete. The values given, however, can be used as a basis for estimating the actual behavior of mass concrete in service.

3.2—Strength requirements

The geometry of massive reinforced concrete sections are often set by criteria totally unrelated to the strength of concrete. Such criteria are often based on:

- Stability requirements where self-weight rather than strength is of primary importance;
- Arbitrary requirements for water tightness per unit of water pressure;
- Stiffness requirements for the support of large pieces of vibrating machinery where the self-weight itself is of primary importance; or
- Shielding requirements, as found in nuclear power plants.

Once these dimensions are established, they are then investigated using an assumed concrete strength to determine the reinforcement requirements to sustain the imposed loadings. In slabs, the design is almost always controlled by flexure. In walls, the reinforcement requirements are usually controlled by flexure or by minimum requirements as load-bearing partitions. Shear rarely controls, except in the case of cantilevered retaining walls or structural frames involving beams and columns.

The geometry and reinforcement of structural concrete is usually determined by structural requirements using 28-day strength concrete of 3000 psi (21 MPa) or more. When the geometry and reinforcement are based on normal code stress limitations for concrete, the spacing of cracks will be primarily influenced by flexure, and the resultant steel stresses induced by volume change will commonly be small in comparison with flexural stresses. Under these conditions, volume control measures do not have the significance that they have for more massive sections with little or no reinforcement. They are, however, not insignificant.

The goal of the designer of a mass concrete and structural concrete structure is to balance the performance requirements of the structure and the material performance of the constituents. More strength is not always better. Excessive strength results in higher heat generation and increases the potential for adverse cracking. Similarly, higher-strength mixtures exhibit more shrinkage and increase the potential for adverse cracking. Cannon et al. (1991) documented several steps designers can take to minimize the adverse effects of excessive strength requirements and excessive cement. Strength-related recommendations include limiting overdesign strength requirements to realistic levels, optimizing concrete mixture proportions, minimizing design strengths as appropriate, and using less restrictive water-to-cementitious material ratios.

One reason for consideration of the effects of low concrete strengths concerns the early loading of massive

sections and the preeminent need in massive concrete to control the heat of hydration of the concrete. If the superimposed loading is not placed until the concrete is 90 or 180 days old, then there is no difficulty using pozzolans in designing low-heat-generating concrete. Such concrete may, however, have significantly lower early strengths for sustaining construction loadings and could present a practical scheduling problem, requiring more time before form stripping and lift joint surface preparation. Commonly, the designer investigates only those construction loads that exceed operational live loads and usually applies a lower load factor for these loads because of their temporary nature.

3.3—Tensile strength

In conventional reinforced concrete design, it is assumed that concrete has no tensile strength. When considering tensile stress due to internal thermal volume change, the actual tensile strength is one of the most important considerations and should be determined to correspond in time to the critical volume change. Because compressive strength is normally specified, it is desirable to relate tensile and compressive strength and take reasonable credit for the tensile contribution of the concrete.

Tensile strength of the concrete will be affected by the type and properties of aggregates used. A restrained concrete made from crushed coarse aggregate will withstand a larger drop in temperature without cracking than comparable concrete made from rounded coarse aggregate. For a given compressive strength, however, the type of aggregate does not appreciably affect tensile strength. The age at which concrete attains its compressive strength does affect the tensile-compressive strength relationship such that the older the concrete, the larger the tensile strength for a given compressive strength.

The most commonly used test to determine the tensile strength of concrete is the splitting tensile test. This test tends to force the failure to occur within a narrow band of the specimen rather than occurring in the weakest section. If the failure does not occur away from the center section, the calculations will indicate a higher-than-actual strength. Without specific actual tensile strength data, the tensile strength for normalweight concrete can be conservatively assumed to be $6.7 \sqrt{f'_c}$ psi ($\sqrt{f'_c} / 1.8$ MPa). Drying has little effect on the relationship.

Direct tensile tests made by attaching steel base plates with epoxy resins indicate approximately 25% lower strengths. Such tests are significantly affected by drying (Raphael 1984).

If the concrete surface has been subjected to drying, a somewhat lower tensile strength than $6.7 \sqrt{f'_c}$ psi ($\sqrt{f'_c} / 1.8$ MPa) should be used to predict cracks initiating at the surface. Where drying shrinkage has relatively little influence on section cracking, a tensile strength of $6 \sqrt{f'_c}$ psi ($0.5 \sqrt{f'_c}$ MPa) appears reasonable.

In the preceding expressions, it is more appropriate to use the probable compressive strength at critical cracking rather than the design strength. Therefore, for structural concrete, it is recommended that the appropriate additional strength (ACI 214R and ASTM C 94/C 94M) be added to the design strength in the design of concrete mixtures to account for

drying shrinkage. The strength of concrete that controls the critical volume change may occur either during the first 7 days following placement or after a period of 3 to 6 months, depending primarily upon peak temperatures. If the cracking potential occurring upon initial cooling exceeds the cracking potential occurring during the seasonal temperature drop, the critical volume change will occur during the first week.

When the critical volume change is seasonal, some allowance should be made for the strength gain beyond 28 days at the time of cracking, particularly where fly ash is used. The strength gain from 28 days to 90 and 180 days of age as a percentage of the 28-day strength varies with the 28-day strength, depending on the cement and the proportions of fly ash or other pozzolans used. For concrete mixtures properly proportioned for maximum strength gain, Fig. 3.1 gives a typical comparison for mixtures with and without fly ash that use Type II cement.

The 7-day strength of concrete normally ranges from 60 to 70% of 28-day strengths for standard cured specimens of Types II and I cement, respectively. Slightly lower strengths may be encountered when fly ash or other pozzolans are used. In-place strengths will vary depending on the massiveness of the cross section and the resulting curing temperatures.

3.4—Creep

Creep is related to a number of factors, including elastic modulus at the time of loading, age, and length of time under load. Although creep plays a large part in relieving thermally induced stresses in massive concrete, it plays a lesser role in thinner concrete sections where temperature changes occur over a relatively short time period. Its primary effect, as noted in Section 3.2, is the relief of drying shrinkage stresses in small elements. Generally, when maximum temperature changes occur over a relatively short time period, creep can only slightly modify temperature stresses.

The stress relief provided by the creep property is used in thermal analysis in several ways. Modulus of elasticity relates the stress-strain relationship during the elastic phase of performance. A modification of this property is the sustained modulus of elasticity. This parameter is the modulus of elasticity reduced by several creep parameters to effectively account for the stress relief provided by creep. Similarly, a property was developed called the tensile strain capacity. This parameter also modifies the property of tensile strain, developed for the elastic phase of loading, and includes the added strain provided by creep. Each of these parameters is discussed in this document.

3.5—Thermal properties of concrete

The thermal properties of concrete are coefficient of expansion, conductivity, specific heat, and diffusivity.

The relationship of diffusivity, conductivity, and specific heat is defined by (ASTM 2006)

$$h^2 = \frac{K}{C_h \times \rho} \quad (3-1)$$

where



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Table 3.1—Thermal properties of concrete*

Structure	Coarse aggregate type	Temperature, °F (°C)	Coefficient of expansion, [†] millionths/°F (millionths/°C)		Thermal conductivity, [‡] Btu/ft·h·°F (kJ/m·h·°C)	Specific heat, Btu/lb·°F (kJ/kg·°C)	Density, lb/ft ³ (kg/m ³)	Diffusivity, [§] ft ² /h ([m ² /h] × 10 ⁻³)
			1-1/4 in. (37.5 mm) max.	4-1/2 in. (114 mm) max.				
Hoover	Limestone and granite	50 (10)	5.3 (9.5)	4.8 (8.6)	1.70 (10.6)	0.212 (0.887)	156.0 (2500)	0.051 (4.7)
		100 (38)			1.67 (10.4)	0.225 (0.941)		0.047 (4.4)
		150 (66)			1.65 (10.3)	0.251 (1.050)		0.042 (3.9)
Grand Coulee	Basalt	50 (10)	4.4 (7.9)	4.6 (8.3)	1.08 (6.74)	0.219 (0.916)	158.1 (2534)	0.031 (2.9)
		100 (38)			1.08 (6.74)	0.231 (0.967)		0.029 (2.7)
		150 (66)			1.09 (6.78)	0.257 (1.075)		0.027 (2.5)
Friant	Quartzite granite and rhyolite	50 (10)	—	—	1.23 (7.66)	0.216 (0.904)	153.8 (2465)	0.037 (3.4)
		100 (38)			1.23 (7.66)	0.230 (0.962)		0.035 (3.2)
		150 (66)			1.24 (7.70)	0.243 (1.017)		0.033 (3.1)
Shasta	Andesite and slate	50 (10)	—	4.8 (8.6)	1.32 (8.20)	0.219 (0.916)	156.6 (2510)	0.039 (3.6)
		100 (38)			1.31 (8.16)	0.233 (0.975)		0.036 (3.3)
		150 (66)			1.31 (8.16)	0.247 (1.033)		0.034 (3.2)
Angostura	Limestone	50 (10)	4.0 (7.2)	—	1.49 (9.29)	0.221 (0.925)	151.2 (2423)	0.045 (4.2)
		100 (38)			1.48 (9.20)	0.237 (0.992)		0.041 (3.8)
		150 (66)			1.46 (9.08)	0.252 (1.054)		0.038 (3.5)
Kortes	Granite gabbros and quartz	50 (10)	5.2 (9.4)	4.5 (8.1)	1.61 (10.0)	0.208 (0.870)	151.8 (2433)	0.050 (4.6)
		100 (38)			1.60 (9.96)	0.221 (0.925)		0.047 (4.4)
		150 (66)			1.59 (9.87)	0.234 (0.979)		0.044 (4.1)
Hungry Horse	Sandstone	50 (10)	6.2 (9.7)	5.7 (9.4)	1.72 (10.1)	0.217 (0.895)	150.1 (2425)	0.053 (4.6)
		100 (38)			1.71 (10.0)	0.232 (0.937)		0.049 (4.4)
		150 (66)			1.69 (9.87)	0.247 (0.983)		0.046 (4.2)
Monticello	Sandstone, metasilstone, quartzite, and rhyolite	50 (10)	5.2 (9.4)	—	1.57 (9.79)	0.225 (0.941)	151.3 (2454)	0.046 (4.3)
		100 (38)			1.55 (9.67)	0.237 (0.992)		0.043 (4.0)
		150 (66)			1.53 (9.54)	0.250 (1.046)		0.040 (3.7)
Anchor	Andesite, latite, and limestone	50 (10)	5.6 (10.1)	4.5 (8.1)	1.14 (7.11)	0.227 (0.950)	149.0 (2388)	0.034 (3.2)
		100 (38)			1.14 (7.11)	0.242 (1.013)		0.032 (3.0)
		150 (66)			1.15 (7.15)	0.258 (1.079)		0.030 (2.8)
Glen Canyon	Limestone, chert, and sandstone	50 (10)	—	—	2.13 (13.3)	0.217 (0.908)	150.2 (2407)	0.065 (6.0)
		100 (38)			2.05 (12.8)	0.232 (0.971)		0.059 (5.5)
		150 (66)			1.97 (12.3)	0.247 (1.033)		0.053 (4.9)
Flaming Gorge	Limestone and sandstone	50 (10)	—	—	1.78 (11.1)	0.221 (0.925)	150.4 (2411)	0.054 (5.0)
		100 (38)			1.75 (10.9)	0.234 (0.979)		0.050 (4.6)
		150 (66)			1.73 (10.8)	0.248 (1.038)		0.046 (4.3)
Yellowtail	Limestone and andesite	50 (10)	—	4.3 (7.7)	1.55 (9.67)	0.226 (0.946)	152.5 (2444)	0.045 (4.2)
		100 (38)			1.52 (9.46)	0.239 (1.000)		0.042 (3.9)
		150 (66)			1.48 (9.20)	0.252 (1.054)		0.039 (3.6)
Dworshak	Granite gneiss	100 (38)	—	5.5 (9.9)	1.35 (8.41)	0.220 (0.920)	154 (2467)	0.040 (3.9)
Ilha Solteira	Quartzite and basalt	100 (38)	—	6.9 (12.5)	1.73 (10.8)	0.220 (0.920)	159 (2552)	0.049 (4.6)
Itaipu	Basalt	100 (38)	—	4.3 (7.8)	1.06 (6.61)	0.233 (0.975)	158 (2537)	0.029 (2.7)
Theodore Roosevelt Modification	Granite	50 (10)	4.3 (7.7)	—	1.71 (10.7)	0.234 (0.979)	148.7 (2380)	0.049 (4.6)
		100 (38)			1.73 (10.9)	0.248 (1.037)		0.047 (4.4)
		150 (66)			1.70 (10.6)	0.260 (1.088)		0.044 (4.1)
Olivenhain	Granodiorite	100 (38)	5.4 (9.7)	—	0.94 (5.86)	0.210 (0.880)	147.4 (2360)	0.030 (2.8)

* (U.S. Army Corps of Engineers 1966; U. S. Bureau of Reclamation 1961; Pacellidede et al. 1982).

[†] 1-1/2 in. (37.5 mm) and 4-1/2 in. (114 mm) max. refer to maximum size of aggregate in concrete.

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

[§] Diffusivity is often expressed in ft²/day (m²/day) for convenience in calculations.

h^2 = diffusivity, ft²/h (m²/h);

K = conductivity, Btu/ft·h·°F (kJ/m·h·°C);

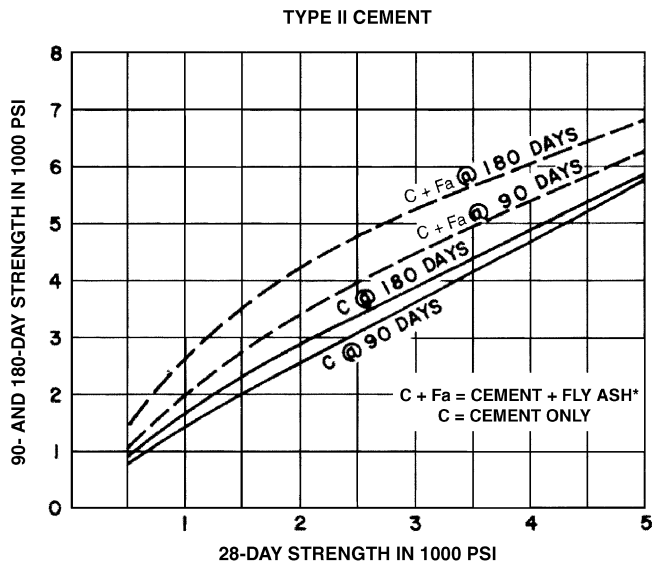
C_h = specific heat, Btu/lb·°F (kJ/kg·°C); and

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

This expression provides a model for how heat is conducted through a solid—specifically, concrete. These thermal properties have a significant effect on temperature that, in turn, results in a change in concrete volume. Such

properties should be determined in the laboratory using job materials in advance of design, if possible. Table 3.1 presents a broad range of thermal properties.

3.5.1 Linear coefficient of expansion—Where laboratory tests are not available, the coefficient of thermal expansion (CTE) should be assumed as 5×10^{-6} in./in./°F (9×10^{-6} mm/mm/°C) for calcareous aggregate, 6×10^{-6} in./in./°F (11×10^{-6} mm/mm/°C) for siliceous aggregate concrete, and



*PROPORTIONED FOR MAXIMUM STRENGTH GAIN

Fig. 3.1—Comparison of 28-, 90-, and 180-day compressive strength.

7×10^{-6} in./in./°F (13×10^{-6} mm/mm/°C) for quartzite aggregate or refer to Table 3.1 or 3.2 for the coefficient of thermal expansion of concrete or aggregates.

3.5.2 Specific heat—The heat capacity per unit of temperature, or specific heat, of normalweight concrete varies only slightly with aggregate characteristics, temperature, and other parameters. Values from 0.20 to 0.25 Btu/lb·°F (KJ/kg·°C) are representative over a wide range of conditions and materials.

3.5.3 Thermal conductivity—Thermal conductivity is a measure of the capability of concrete to conduct heat and may be defined as the rate of heat flow per unit temperature gradient causing that heat movement. Mineralogical characteristics of the aggregate and the moisture content, density, and temperature of the concrete all influence the conductivity. Within the normal concrete temperatures experienced in mass concrete construction, and for the high moisture content existing in concrete at early ages, thermal conductivity values should resemble those shown in Table 3.3.

3.5.4 Thermal diffusivity—Thermal diffusivity is an index of the ease or difficulty with which concrete undergoes temperature change and, numerically, is the thermal conductivity divided by the product of density and specific heat. The value of diffusivity is largely affected by the rock type used in the concrete. Table 3.4 shows diffusivities for concrete made with different rock types. The higher the value of diffusivity, the more readily heat will move through the concrete. For normalweight concrete, where density and specific heat values vary within relatively narrow ranges, thermal diffusivity reflects the conductivity value. High conductivity indicates greater ease in gaining or losing heat. Values for concrete containing quartzite aggregate have been reported up to 0.065 ft²/h (0.0060 m²/h).

3.6—Modulus of elasticity

Before achieving a measurable modulus of elasticity, volume changes occur with no accompanying development

Table 3.2—Typical ranges for common portland-cement concrete (PCC) components

Aggregate	Coefficient of thermal expansion millionths/°F (millionths/°C)
Granite	4 to 5 (7 to 9)
Basalt	3.3 to 4.4 (6 to 8)
Limestone	3.3 (6)
Dolomite	4 to 5.5 (7 to 10)
Sandstone	6.1 to 6.7 (11 to 12)
Quartzite	6.1 to 7.2 (11 to 13)
Marble	2.2 to 4 (4 to 7)
Concrete	4.1 to 7.3 (7.4 to 13)

Table 3.3—Typical thermal conductivity values for concrete selected by type of aggregate (U.S. Bureau of Reclamation 1940)

Aggregate type	Thermal conductivity, Btu-in./h-ft ² ·°F (KJ/kg·°C)
Quartzite	24 (4.5)
Dolomite	22 (4.2)
Limestone	18 to 23 (2.6 to 3.3)
Granite	18 to 19 (2.6 to 2.7)
Rhyolite	15 (2.2)
Basalt	13 to 15 (1.9 to 2.2)

Table 3.4—Diffusivity and rock type

Coarse aggregate	Diffusivity of concrete, ft ² /day (m ² /day)	Diffusivity of concrete, ft ² /h (m ² /h × 10 ⁻³)
Quartzite	1.39 (0.129)	0.058 (5.4)
Limestone	1.22 (0.113)	0.051 (4.7)
Dolomite	1.20 (0.111)	0.050 (4.6)
Granite	1.03 (0.096)	0.043 (4.0)
Rhyolite	0.84 (0.078)	0.035 (4.2)
Basalt	0.77 (0.072)	0.032 (4.0)

of stress. After placement, the concrete will begin to behave elastically. Unless more accurate determinations are made, the elastic modulus in tension and compression for hardened concrete may be assumed equal to $w_c^{1.533} \sqrt{f'_c}$ in psi ($w_c^{1.50} 0.043 \sqrt{f'_c}$ in MPa), which for normalweight concrete reduces to 57,000 $\sqrt{f'_c}$ in psi (4700 $\sqrt{f'_c}$ in MPa). It also should be based on probable strength as discussed in Section 3.4. The modulus of elasticity in mass concrete can depart significantly from these values and should be based on actual test results whenever possible.

Typical instantaneous and sustained (long-term) elastic modulus values for four conventional mass concretes (different coarse aggregates) are given in Table 3.5. The lower modulus of elasticity values after 1 year of sustained loading reflect the increases in strain resulting from the time-dependent characteristic (creep) of the concrete. At intermediate dates, the unit strain increase is directly proportional to the logarithm of the duration of loading. For example, with initial loading at 90 days and basalt aggregate concrete, the initial unit strain is 0.244 millionths per psi (35.7 millionths per MPa). After a 1-year load duration the unit strain value is 0.400 millionths per psi (58.0 millionths per MPa). At an

Table 3.5—Example instantaneous and sustained modulus of elasticity for conventional mass concrete (Harboe 1958)*

Age at time of loading, days	Million psi (GPa)							
	Basalt		Andesite and slate		Sandstone		Sandstone and quartz	
	<i>E</i>	<i>E'</i>	<i>E</i>	<i>E'</i>	<i>E</i>	<i>E'</i>	<i>E</i>	<i>E'</i>
2	1.7 (12)	0.83 (5.7)	1.4 (9.7)	0.54 (4.7)	2.8 (19)	1.5 (10)	1.4 (9.7)	0.63 (4.3)
7	2.3 (16)	1.1 (7.6)	2.1 (14)	1.0 (6.9)	4.2 (29)	1.9 (13)	2.2 (15)	0.94 (6.5)
28	4.5 (24)	1.8 (12)	4.5 (24)	1.8 (12)	4.5 (31)	2.6 (18)	4.6 (25)	1.8 (12)
90	4.1 (28)	2.5 (17)	4.4 (30)	2.7 (19)	5.2 (36)	4.2 (29)	4.2 (29)	2.6 (18)
365	5.0 (34)	4.1 (21)	4.7 (32)	4.5 (24)	5.7 (39)	4.6 (32)	4.6 (32)	4.1 (21)

*Based on ACI 207.1R.

Notes: All concrete mass mixed, wet screened to a 1-1/2 in. (38 mm) maximum size aggregate. *E* = instantaneous modulus of elasticity at time of loading; and *E'* = sustained after 365 days under load.

age of 100 days, or 10 days after initial loading, the unit strain value in millionths per psi is given by Eq. (3-2)

$$0.244 + (0.400 - 0.244) \log 10 / \log 365 = 0.304 \quad (3-2)$$

(in millionths per MPa: $35.7 + (58.8 - 35.7) \log 10 / \log 365$)

The resulting modulus of elasticity is 3.3×10^6 psi (22 GPa).

Elastic properties given in Table 3.5 were influenced by conditions other than aggregate type, and for major work, laboratory-derived creep data based on aggregates and concrete mixtures to be used is probably warranted.

3.7—Strain capacity

Designs based on tensile strain capacity rather than tensile strength are more convenient and simple where criteria are expressed in terms of linear or volumetric changes. Examples include temperature and drying shrinkage phenomena. Ideally, tensile strain capacity is determined for each concrete with tests of large mass beams or prisms containing internal strain gauges, and tested in flexure or direct tension. The tensile strain measured by embedded gauges in large beams will approximate the direct tensile strain capacity. Surface gauges on the tensile face of flexural beams, however, will overestimate the strain capacity and should not be used. Because of its convenience, it is common to use the indirect or splitting tensile strength test to obtain tensile strength properties of concrete, and then convert this to tensile strain capacity by dividing by the modulus of elasticity. The indirect tensile strength, however, should first be converted to the probable direct tensile strength.

The normal tensile strain test beams are 12 x 12 x 64 in. (300 x 300 x 1600 mm), nonreinforced, and tested to failure under third-point loading (U.S. Army Corps of Engineers 1980). Strains of the extreme fiber in tension are measured directly on the test specimen. At the 7-day initial loading age, one specimen is loaded to failure over a period of a few minutes (rapid test). Concurrently, loading of a companion test beam is started, with weekly loading additions, 25 psi/

week (0.17 MPa/week), of a magnitude that will result in beam failure at approximately 90 days (slow test). Upon failure of the slow test beams, a third specimen is sometimes loaded to failure under the rapid test procedure to provide a measure of the change in elastic properties over the duration of the test period.

Tensile strain capacity results aid in establishing concrete crack control procedures. For example, assume that a concrete has a coefficient of thermal expansion of 5.5 millionths/°F (9.9 millionths/°C) 7- and 90-day rapid load tensile strain of 64 and 88 millionths, respectively, and a 7- and 90-day slow load tensile strain of 118 millionths. Sufficient insulation should be used to avoid sudden surface temperature drops greater than $64/5.5 = 11.6$ °F (6.4 °C) at early ages, and $88/5.5 = 16$ °F (8.9 °C) at 3 months or later ages. In the event that embedded pipe cooling is used, the total temperature drop should not exceed $118/5.5 = 21$ °F (12 °C) over the initial 3-month period.

CHAPTER 4—HEAT TRANSFER AND VOLUME CHANGE

The purpose of this chapter is to offer some practical guidance in determining the magnitude of volume change that can be expected in concrete structures or elements. These effects apply to mass concrete structures and reinforced concrete structures. Reinforced concrete structures use cement with higher heat generation, smaller aggregate, more water, and less temperature control than normally used or recommended for mass concrete. In concrete elements, the primary concern is volume changes resulting from thermal and moisture changes. Other volume changes, which are not considered in this document, are alkali-aggregate expansion, autogenous shrinkage, and changes due to expansive cement. Autogenous shrinkage is the volume change due to the chemical process that occurs during hydration.

The volume change that leads to thermal cracking is from the temperature difference between the peak temperature of the concrete attained during early hydration (normally within the first week following placement) and the minimum temperature to which the element will be subjected under service conditions. The initial hydration temperature rise produces little, if any, stress in the concrete. At this early age, the modulus of elasticity of concrete is so small that compressive stresses induced by the rise in temperature are insignificant even in zones of full restraint and, in addition, are relaxed by a high rate of early creep. A slightly conservative and realistic analysis results by assuming a condition of no initial stress.

The following sections discuss the thermal factors of a concrete placement that result in volume change. They are the initial placement temperature of the concrete, the heat generation phase, and the cooling or heat dissipation phase. Practices for cooling materials and the calculation of initial concrete temperatures are presented in ACI 207.4R. Methods to estimate or calculate values for the heat generation and dissipation phases are presented as follows. Ultimately, these steps can be combined to form part of a complete thermal analysis of a structure. The computation of thermal volume change can be summarized in the following expression