ACI 207.2R-07

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

Reported by ACI Committee 207

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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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ACI 207.2R-07 supersedes ACI 207.2R-95 and was adopted and published September 2007.

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 practicable width in all structures, the economics 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 lower 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)

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

where

Table 3.1—Thermal properties of concrete*

Structure	Coarse aggregate type	Temperature, °F (°C)	Coefficient of expansion,† millionths/°F (millionths/°C) 1-1/4 in. (37.5 mm) max.	Coefficient of expansion,† millionths/°F (millionths/°C) 4-1/2 in. (114 mm) max.	Thermal conductivity,‡ Btu/ft·h·°F (kJ/m·h·°C)	Specific heat, Btu/lb·°F (kJ/kg·°C)	Density, lb/ft³ (kg/m³)	Diffusivity,§ ft²/h ([m²/h] × 10⁻³)
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, metasiltsone, 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.

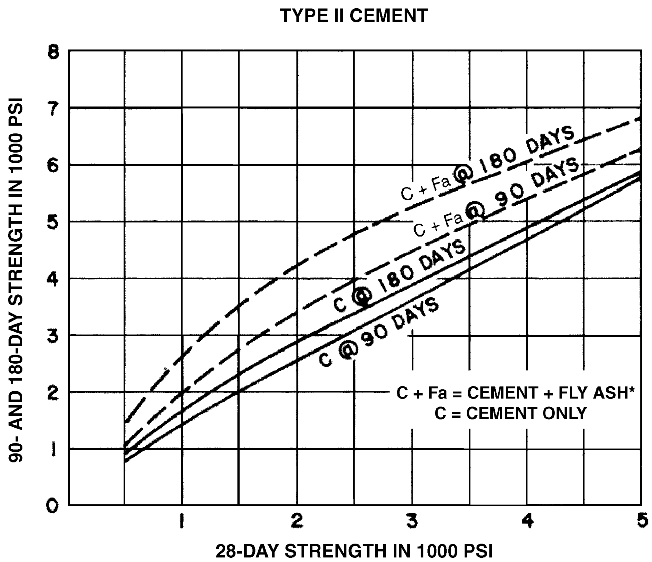
‡Procedure for calculating thermal conductivity is described in CRD-44 (U.S. Army Corps of Engineers 1949).

§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 silicious 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 ft^2/h (0.0060 m^2/h).

3.6—Modulus of elasticity

Before achieving a measurable modulus of elasticity, volume changes occur with no accompanying development 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 $w_c^{1.5} 33\sqrt{f_c'}$ in psi ($w_c^{1.5} 0.043\sqrt{f_c'}$ 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 (58.8 millionths per MPa). At an

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)

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	
	E	*E'*	*E*	*E'*	*E*	*E'*	*E*	*E'*
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)

$$0.244 + (0.400 - 0.244) \log 10/\log 365 = 0.304 \qquad (3\text{-}2)$$

(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

$$\Delta V = [T_f - (T_i + T_{ad}) + T_{env}] \times \mathrm{CTE} \tag{4-1}$$

where

ΔV = volume change of the concrete;
T_f = final stable temperature of the concrete;
T_i = initial placing temperature of the concrete;
T_{ad} = adiabatic temperature rise of the concrete;
T_{env} = temperature change from the heat added or subtracted from the concrete due to environmental conditions; and
CTE = coefficient of thermal expansion.

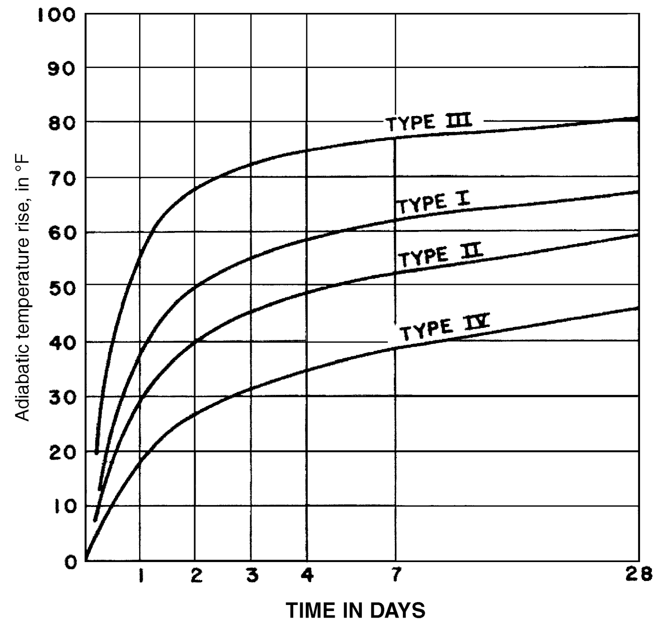
4.1—Heat generation

The rate and magnitude of heat generation of the concrete (T_{ad} in Eq. (4-1)) depends on the amount per unit volume of cement and pozzolan (if any), the compound composition and fineness of cement, and the temperature during hydration of the cement. Design strength requirements, durability, and the characteristics of available aggregates largely dictate the cement content of the mixture to be used for a particular job. Options open to the engineer seeking to limit heat generation include:

1. Use of lower-heat portland cement (ASTM C 1157 Types LH or MH or ASTM C 150 Types IV, V, or II with moderate heat option);
2. Use of blended hydraulic cement (ASTM C 595 Types IS(MH), IP(MH), P(LH), I(PM)(MH), I(SM)(MH), MH, or LH) that exhibit favorable heat of hydration characteristics that may be even more firmly achieved by imposing heat of hydration limit options for the portland cement clinker; and
3. Reduction of the cement content by using a slag or pozzolanic material, either fly ash or a natural pozzolan, to provide a reduction in maximum temperatures produced without sacrificing the long-term strength development. In some instances, advantage can be taken of the cement reduction benefit of a water-reducing admixture. ACI 207.1R, Chapter 2 provides more detailed discussion of cementitious materials commonly used in mass concrete structures.

4.1.1 *Adiabatic temperature rise*—Figure 4.1 shows curves for adiabatic temperature rise versus time for mass concrete placed at 73 °F (22.8 °C) and containing 376 lb/yd^3 (223 kg/m^3) of various types of cement. These curves are typical of cement produced before 1960. The same cement types today may vary widely from those because of increased fineness and strengths. Current ASTM specifications limit the heat of hydration for cement in two ways: 1) by controlling chemical composition, specifically by placing limits on the C_3S and C_3A components; and 2) by physically testing the heat of hydration. These controls should not be specified together.

Heat-of-hydration tests present a fairly accurate picture of the total heat-generating characteristics of cement at 28 days because of the relative insensitivity with age of the total heat-generating capacity of cement at temperatures above 70 °F (21.1°C). At early ages, however, cement is highly sensitive to temperature; therefore, heat-of-solution tests, which are performed under relatively constant temperatures, do not reflect the early-age adiabatic temperature rise. The use of an



Cement type	Fineness ASTM C 115 cm^2/gm	28-day heat of hydration, Calories per gm
I	1790	87
II	1890	76
III	2030	105
IV	1910	60

Fig. 4.1—Temperature rise of mass concrete containing 376 lb/yd^3 (223 kg/m^3) of various types of cement.

isothermal calorimeter for measuring heat of hydration can provide data on the rate of heat output at early ages (Milestone and Rogers 1981). More accurate results for specific cement, mixture proportions, aggregate initial placing temperature, and set of environmental conditions can be determined by adiabatic temperature-rise tests carefully performed in the laboratory under conditions that represent those that will occur in the field.

4.1.2 *Cement fineness*—The fineness of cement affects the rate of heat generation more than it affects the total heat generation, in much the same fashion as placing temperature. The rate of heat generation as affected by cement fineness and placing temperature is shown in Fig. 4.2 and 4.3, respectively. These two figures are based on extrapolation of data from a study of the heat of hydration of cement by Verbeck and Foster (1950).

There are no maximum limitations on cement fineness in ASTM specifications. By varying both fineness and chemical composition of the various types of cement, it is possible to vary widely the rate and total adiabatic temperature rise of the typical types shown in Fig. 4.1. Therefore, it is essential that both the fineness and chemical composition of the cement in question be considered in estimating the temperature rise of massive concrete members.

4.1.3 *Other cementitious materials*—For mass concrete applications and pavement slabs, pozzolans and slags are often used as supplemental cementitious material. In

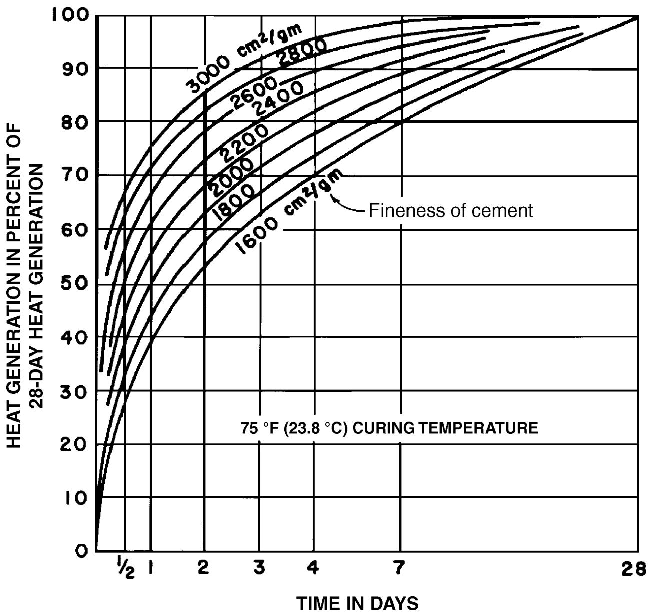


Fig. 4.2—Rate of heat generation as affected by Wagner fineness of cement (ASTM C 115) for cement paste cured at 75 °F (23.8 °C).

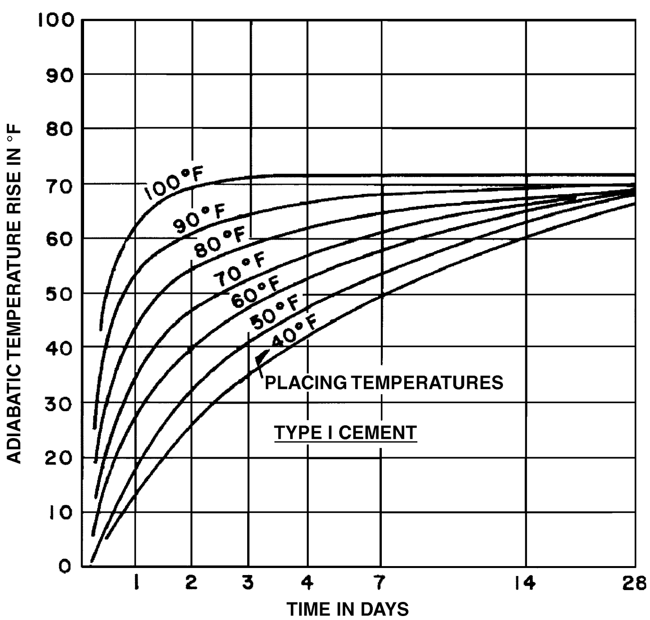


Fig. 4.3—Effect of placing temperature and time on adiabatic temperature rise of mass concrete containing 376 lb/yd³ (223 kg/m³) of Type I cement.

general, the relative contribution of the pozzolan to heat generation increases with age of concrete, fineness of pozzolan compared with cement, and with lower heat-generating cement. The early-age heat contribution of fly ash may conservatively be estimated to range between 15 and 35% of the heat contribution from the same weight of cement. Generally, the low percentages correspond to combined finenesses of fly ash and cement as low as 2/3 to 3/4 that of the cement alone, whereas the higher percentages correspond to fineness equal to or greater than the cement alone.

At early ages (up to 3 days), the temperature rise of mixtures containing ASTM C 618 Class F pozzolan results principally from hydration of the cement, with little if any heat contributed by the pozzolan. At later ages (after 7 days), the pozzolan does participate in the hydration process, and may contribute as much as 50% of the amount of heat that would have been generated by the cement it replaced. ASTM C 618 Class C fly ash generally produces more heat than Classes F or N pozzolans, particularly at early ages.

The benefit of using pozzolan for a portion of the cement is that it will cause a definite reduction in the amount of heat generated by the cementitious materials and will delay the time of peak temperatures by 30 to 60 days. If lifts are placed rapidly in cool environments, as is a frequent case with roller-compacted concrete, the use of pozzolan may result in more heat being trapped inside the mass. This is because the next lift is placed before any significant heat develops that would otherwise escape to the atmosphere. This may be further exaggerated if an effective retarding admixture is used.

Pozzolan can significantly affect the elastic and creep properties of concrete and ultimately the cracking behavior of the concrete structure. The optimal proportions of pozzolan depend on many factors that are often best determined by thorough investigation and material testing. Pozzolan use in roller-compacted concrete is discussed in ACI 207.5R.

4.1.4 *Estimating adiabatic temperature rise*—For a given fineness, the chemical composition of cement has a relatively constant effect on the generation of heat beyond the first 24 hours. Absent specific performance data, the 28-day adiabatic temperature rise (°F) for the four cement types may be approximated by

$$H_a = \frac{1.8 h_g w_c}{0.22(150)(27)} \tag{4-2}$$

where

0.22 = specific heat of concrete in cal/g·°C;
150 = density of concrete in lb/ft^3;
1.8 = conversion factor from Celsius to Fahrenheit;
27 = conversion factor from yd^3 to ft^3;
h_g = 28-day measured heat generation of the cement by heat of hydration in cal/g; and
w_c = weight of cement in pounds per cubic yard of concrete.

For a concrete mixture containing 376 lb of cement per cubic yard of concrete: $H_a = 0.76 h_g$ (°F). For low and medium cement contents, the total quantity of heat generated at any age is directly proportional to the quantity of cement in the concrete mixture.

For high-cement-content concrete mixtures, however, the amount of cement may be sufficiently high to increase the very early age heat to a point where the elevated temperature in turn causes a more rapid rate of heat generation. When fly ash or other pozzolans are used, the total quantity of heat generated is directly proportional to an equivalent cement content C_{eq}, which is the total quantity of cement plus a percentage of total pozzolan content. The contribution of pozzolan to heat generation varies with age of the concrete, the type of pozzolan, and the fineness of the pozzolan compared with the cement. It is best determined by testing the combined portions of pozzolan and cement for fineness and heat of hydration.

As illustrated in Fig. 4.1, high-early-strength cement, Type III, is the fastest heat generator and gives the highest adiabatic temperature rise. Type IV, or low-heat cement, is not only the slowest heat generator, but gives the lowest total temperature rise. Because the cement is the active heat producer in a concrete mixture, the temperature rise of concrete with cement contents differing from 376 lb/yd^3 (223 kg/m^3) can be estimated closely by multiplying the values shown on the curves by a factor representing the proportion of cement.

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

In general, chemical admixtures affect heat generation of concrete only during the first few hours after mixing and can be neglected in preliminary computations. In thermal analyses involving thousands of cubic yards of concrete, as in dams, large foundation mats, and bridge piers, the aforementioned remarks should be applied only to preliminary computations, and the adiabatic temperature rise should be determined for the exact mixture to be used in the mass concrete starting at the proposed placing temperature.

4.1.5 *Effects of member volume and surface area*—Where the least dimension of a concrete unit is large or the surface area to mass ratio is large, heat of hydration can escape readily from the boundary surfaces (forms not insulated), and the maximum temperature rise will not be great. In all instances, however, some internal temperature rise is necessary to create a thermal gradient for conducting the heat to the surface. The rate of heat generation as affected by initial temperature, member size, and environment is difficult to assess because of the complex variables involved. For large concrete members, however, it is advisable to compute their temperature history, taking into account the measured values of heat generation, concrete placement temperatures, and ambient temperature. The problem may be simplified somewhat by assuming that the placing temperature and ambient air temperature are identical. A correction can then be made for the actual difference, considering the size or volume-to-exposed surface ratio (*V/S*) of the member in question. The *V/S* actually represents the average distance through which heat is dissipated from the concrete.

Usually, peak concrete temperatures for most conventional concrete structures may occur at any time during the first week after placement. Figure 4.4 shows the effect of placing temperature and member *V/S* on the age at which peak concrete temperatures occur for concrete containing Type I

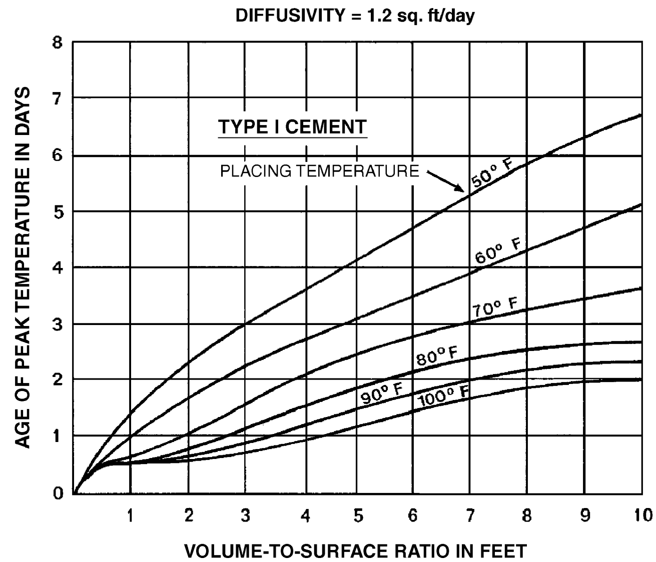


Fig. 4.4—Effect of placing temperature and surface exposure on age at peak temperature for Type I cement in concrete. Air temperature = placing temperature.

cement. Time would be shortened or lengthened for cement with higher or lower heat-generating characteristics.

There can be notable differences between the properties of different cements meeting the industry standard criteria. For general comparative purposes, the early-age heat generation of a Type III cement is approximately equivalent to a Type I cement at a 20 °F (11 °C) higher placing temperature. In a similar fashion, the heat-generating characteristic of Types II and IV cement correspond closely to that of Type I cement at 10 and 20 °F (6 and 11 °C) lower placing temperatures, respectively. Figure 4.4 shows that for *V/S* less than 3 ft (1 m), peak temperature will be reached within 1 day under normal placing temperature (80 °F [27 °C] or higher).

Figure 4.5 gives the approximate maximum temperature rise for concrete members containing 376 lb/yd^3 (223 kg/m^3) of Type I cement for placing temperatures ranging from 50 to 100 °F (10 to 38 °C), assuming ambient air temperatures equal to placing temperatures. Corrections are required for different types and quantities of cementitious materials. A correction for the difference in air and placing temperatures can be made using Fig. 4.6 by estimating the time of peak temperatures from Fig. 4.4. The effect of water-reducing, set-retarding agents on the temperature rise of concrete is usually confined to the first 12 to 16 hours after mixing, during which time these agents have the greatest effect on the chemical reaction. Their presence does not appreciably alter the total heat generated in the concrete after the first 24 hours, and no corrections are applied herein for the use of these agents.

A diffusivity of 1.2 ft^2/day (0.11 m^2/day) has been assumed in the preparation of Fig. 4.4 through 4.6. A concrete of higher or lower diffusivity will decrease or increase, respectively, the *V/S*, and can be accounted for by multiplying the actual *V/S* by 1.2 (0.11) divided by the actual concrete diffusivity.

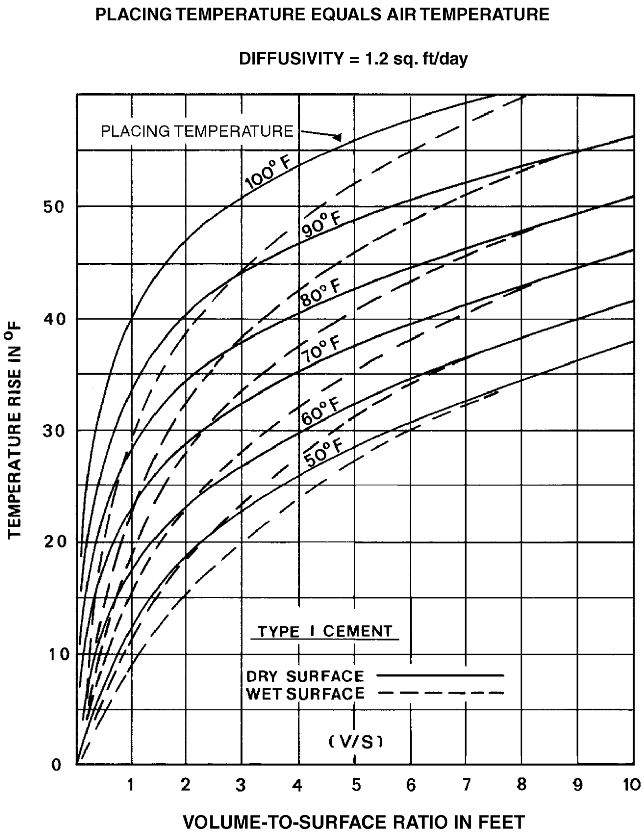


Fig. 4.5—Temperature rise of concrete members containing 376 lb/yd^3 (223 kg/m^3) of cement for different placing temperatures.

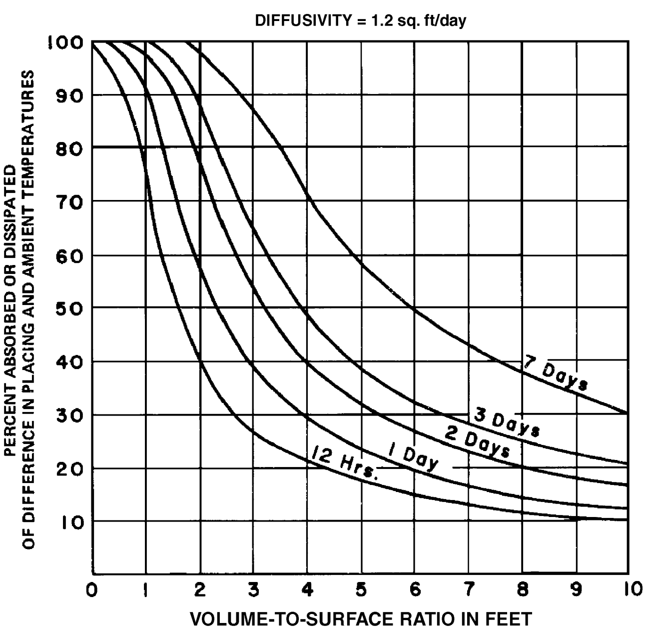


Fig. 4.6—Heat flow between air and concrete for difference between placing temperature and ambient air temperature.

4.1.6 *Other examples*—A large foundation or footing is essentially a wall of large dimensions cast on its side, such that heat is lost principally from a single exposed surface. For this case, Table 4.1 shows the typical maximum temperatures expected, which are not substantially higher than those for a vertically cast wall (Table 4.2). The maximum temperatures, however, occur at later ages and over large portions of the concrete mass. Because a static tension-compression force balance should exist, the compressive unit stress across the center portion is small and essentially uniform, whereas very high tensile stress exists at the exposed sides.

An example of a massive concrete structure that was crack free with modest thermal controls, aided by favorable climate conditions, is a large footing placed in Great Britain (Fitzgibbon 1973). It was 5200 ft^2 (480 m^2) in area and 8.2 ft (2.5 m) in depth, with a cement content of 705 lb/yd^3 (418 kg/m^3), placed as a single unit. A maximum concrete temperature of 150 °F (65 °C) was measured, with side surfaces protected by 3/4 in. (19 mm) plywood forms and top surface by a plastic sheet under a 1 in. (25 mm) layer of sand. Plywood and sand were removed at the age of 7 days, exposing surfaces to the ambient January air temperature and humidity conditions.

Table 4.1—Example temperature rise in slabs-on-ground

	Slab thickness, ft (m)						
	3 (0.9)	5 (1.5)	10 (3.0)	15 (4.6)	20 (6.1)	25 (7.6)	Infinite
Maximum temperature rise in °F/100 lb (°C/kg) cement	6.0 (5.6)	9.3 (8.7)	14.0 (13.1)	16.0 (15.0)	16.8 (15.7)	17.3 (16.2)	17.8 (16.7)

Assuming:
Moderate heat (ASTM C 150 Type II) cement.
Placing temperature equal to exposure temperature.
Two sides exposed.
Thermal diffusivity: 1.0 ft^2/day (0.093 m^2/day).

Table 4.2—Example temperature rise in walls

	Wall thickness, ft (m)						
	1 (0.3)	2 (0.6)	3 (0.9)	4 (1.2)	5 (1.5)	10 (4.0)	Infinite
Maximum temperature rise in °F/100 lb (°C/kg) cement	1.3 (1.2)	4.2 (4.0)	5.2 (4.9)	7.0 (6.6)	8.6 (8.1)	14.7 (12.8)	17.8 (16.7)

Assuming:
Moderate heat (ASTM C 150 Type II) cement.
Placing temperature equal to exposure temperature.
Two sides exposed.
Thermal diffusivity: 1.0 ft^2/day (0.093 m^2/day).

4.2—Moisture contents and drying shrinkage

For tensile stress considerations, the volume change resulting from drying shrinkage is similar to volume change from temperature, except that the loss of moisture from hardened concrete is extremely slow compared with the loss of heat. Therefore, drying shrinkage depends on the length of moisture migration path and often affects the concrete near a surface. When the length of moisture migration or the *V/S* is small, drying shrinkage adds to the stresses induced by external restraint and should be considered in the design of the reinforcement. When the *V/S* is large, the restraint to drying shrinkage is entirely internal, and the result is tension on the surface or an extensive pattern of surface cracks extending only a short distance into the concrete. When surface cracks of this nature do occur, they are small, and reinforcement is not particularly effective in altering the size or spacing of these cracks. Reinforcing bar is also not a solution for surface cracks in fresh concrete that are referred to as plastic cracking (ACI 116R).

Carlson (1937) showed that a 24 in. (610 mm) thick slab would lose approximately 30% of its evaporable water in 24 months of continuous exposure with both faces exposed to 50% relative humidity. If one assumes a total drying shrinkage potential at the exposed faces of 300 millionths, then the average drying shrinkage for a 24 in. (610 mm) slab under this exposure would be 90 millionths in 24 months. Other than in hot, arid climates, concrete is not usually exposed to drying conditions this severe.

Drying shrinkage is affected by the size and type of aggregate used. “In general, concretes low in shrinkage often contain quartz, limestone, dolomite, granite, or feldspar, whereas those high in shrinkage often contain sandstone, slate, basalt, trap rock, or other aggregates which shrink considerably themselves or have low rigidity to the compressive stresses developed by the shrinkage of paste,” (Troxell and Davis 1956). In this discussion, an aggregate low in shrinkage qualities is assumed. Drying shrinkage may vary widely from the values used herein, depending on many factors that are discussed in more detail in ACI 224R.

In determining volume change, it is convenient to express drying shrinkage in terms of equivalent change in concrete temperature T_{DS}. Creep can be expected to significantly reduce the stresses induced by drying shrinkage because of the long period required for full drying shrinkage to develop. Equation (4-3) provides an initial approximation for dry shrinkage expressed as an equivalent temperature drop. The approximation assumes an equivalent drying shrinkage of 150 millionths and an expansion coefficient of 5×10^{-6} per °F (9×10^{-6} per °C) as a basis in establishing the formula for equivalent temperature drop. While the rates of drying and heat dissipation differ, their average path lengths (V/S) are the same. There is, however, a limitation on the length of moisture migration path affecting external restraint and its impact on total volume change. This limit has been assumed as 15 in. (380 mm) maximum in determining equivalent temperature change

$$T_{DS} = \left(30 - \frac{12V}{S}\right)\left(\frac{W_u - 125}{100}\right) \tag{4-3}$$

where

T_{DS} = equivalent temperature change due to drying shrinkage, in °F;

W_u = water content of fresh concrete, lb/yd^3, but not less than 225 lb/yd^3;

V = total volume, in.^{3}; and

S = area of the exposed surface, in.^{2}

4.3—Ambient temperatures

In many structures, the most important temperature considerations are the average air temperatures during and immediately following the placement of concrete, and the minimum average temperature in the concrete that can be expected during the life of the structure. The temperature rise due to hydration may be small, particularly in thin exposed members, regardless of the type or amount of cement used in the mixture, if placing and cooling conditions are right. On the other hand, the same member could have a high temperature rise if placed at high temperature in insulated forms.

As a general rule, when no special precautions are taken, the temperature of the concrete when placed in the forms will be slightly above the ambient air temperature. The final stable temperature in the interior of a massive concrete structure will approximate the average annual air temperature at its geographical location.

Except for tropical climates, deep reservoir impoundments will maintain the concrete in the vicinity of the heel of the dam at the temperature of water at its maximum density, or about 39 °F (4 °C). Thus, the extreme temperature excursion experienced by interior concrete is determined from the initial placing temperature plus the adiabatic temperature rise minus the heat lost to the air and minus the final stable temperature. Lifts of 5 ft (1.6 m) may lose as much as 25% of the heat generated if exposed for enough time (about 5 days) before placing the subsequent lift if the ambient temperature is reasonably (about 20 °F [11 °C]) less than the internal concrete temperature. Lifts greater than 5 ft (1.6 m) and placements with little or no difference between the air temperature and internal concrete temperature will lose little or no heat.

At least of equal importance is the temperature gradient between the interior temperature and the exposed surface temperature. This can create a serious condition when the surface and near-surface temperatures decline at night, with the falling autumn and winter air temperatures, or from cold water filling the reservoir, while the interior concrete temperatures remain high. The decreasing daily air temperatures, augmented by abrupt cold periods of several days' duration that is characteristic of changing seasons, may create tensile strains approaching, if not exceeding, the strain capacity of the concrete. Controlling this problem is usually accomplished with surface insulation.

4.4—Placement temperature

Concrete placement temperatures can be estimated to be approximately equal to or a few degrees higher than the mean monthly ambient temperature. Specifications usually limit the maximum and minimum placing temperatures of concrete for normal and extreme placement conditions.

For hot weather conditions, ACI 305R recommends limiting the initial concrete placement temperature to a maximum of 75 to 100 °F (24 and 38 °C), depending on the placement conditions. The temperature of concrete placed during hot weather may exceed the mean daily ambient air temperature by 5 to 10 °F (3 to 6 °C) unless measures are taken to cool the concrete or the coarse aggregate. Corrections should be made for the difference in air temperature and placing temperature using Fig. 4.6. For example, if during the first 24 hours the ambient temperature is 80 °F (27 °C) and the temperature of the concrete, when placed, is 60 °F (16 °C), a concrete section having a V/S of 2 ft (0.6 m) would absorb 60% of the difference, or 12 °F (7 °C). The

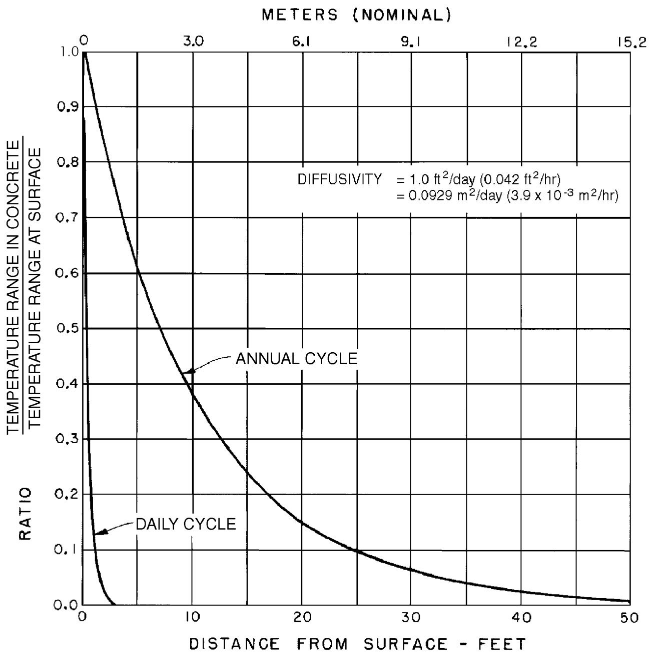


Fig. 4.7—Temperature variation with depth.

maximum placing temperature in summer should be the highest average summer temperature for a given locality, but not more than 100 °F (38 °C).

Minimum concrete temperature recommendations at placing are given in ACI 306R, Table 3.1. These minimums establish the lowest placing temperature to be considered. Placing temperatures for spring and fall can reasonably be considered to be about halfway between the summer and winter placing temperatures.

4.5—Final temperature in service

The minimum expected final temperatures of concrete elements are as varied as their prolonged exposure conditions. The primary concern is for the final or operating exposure conditions because cracks that may form or open during colder construction conditions may be expected to close during operating conditions provided that steel stresses remain in the elastic range during construction conditions. Minimum concrete temperatures can be conservatively taken as the average minimum exposure temperature occurring during a period of approximately 1 week. The mass temperature of earth or rock against concrete walls or slabs forms a heat source, which affects the average temperature of concrete members, depending upon the cooling path or *V*/*S* of the concrete. This heat source can be assumed to affect a constant temperature at some point 8 to 10 ft (2.4 to 3.0 m) from the exposed concrete face.

The minimum temperature of concrete against earth or rock mass, T_{min}, can be approximated by

$$T_{min} = T_A + \frac{2(T_M - T_A)}{3}\sqrt{\frac{V/S}{96}} \qquad \text{(4-4)}$$

where

T_A = average minimum ambient air temperature over a prolonged exposure period of 1 week; and

T_M = temperature of earth or rock mass; approximately 40 to 60 °F, depending on climate.

4.6—Heat dissipation

Studies of the dissipation of heat from bodies of mass concrete can be accomplished by the use of charts and graphs, by hand computation, or with finite-element computer programs. The characteristic that determines the relative ability of heat to flow through a particular concrete is its thermal diffusivity, which was defined in. Eq. (3-1).

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

$$\frac{R_x}{R_o} = e^{-x\sqrt{\pi/h^2\gamma}} \qquad \text{(4-5)}$$

where

R_x = temperature range at distance x from surface;

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

e = base of natural logarithms (= 2.718);

x = distance from surface, ft (m);

h^2 = diffusivity, ft²/h (m²/h) as defined in Section 3.5; and

γ = period of the cycle of temperature variation in days.

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 concrete with a diffusivity of 1 ft²/day (0.093 m²/day), or 0.042 ft²/h (4.9×10^{-3} m²/h), the penetration of the daily and the annual temperature cycles is as shown in Fig. 4.7.

4.6.2 *Heat dissipation from specific shapes*—When the body to be analyzed can be readily approximated by a known geometrical shape, charts are available for the direct determination of heat losses. For instance, Fig. 4.8 can be used to determine the loss of heat in hollow and solid cylinders, slabs with one or two faces exposed, or solid spheres. The application of the values found on these graphs can easily be made to a wide variety of problems such as the cooling of dams or thick slabs of concrete, the cooling of concrete aggregates, artificial cooling of mass concrete by use of embedded pipes, and the cooling of bridge piers. The following five examples are typical concrete cooling problems that can be solved by use of Fig. 4.8. For simplicity of presentation, the examples are in inch-pound units only; Appendix A presents the examples worked in SI (metric) units. In the following examples and Fig. 4.8, the following notation is followed:

t = time, days;

h^2 = diffusivity, ft² per day (m² per day);

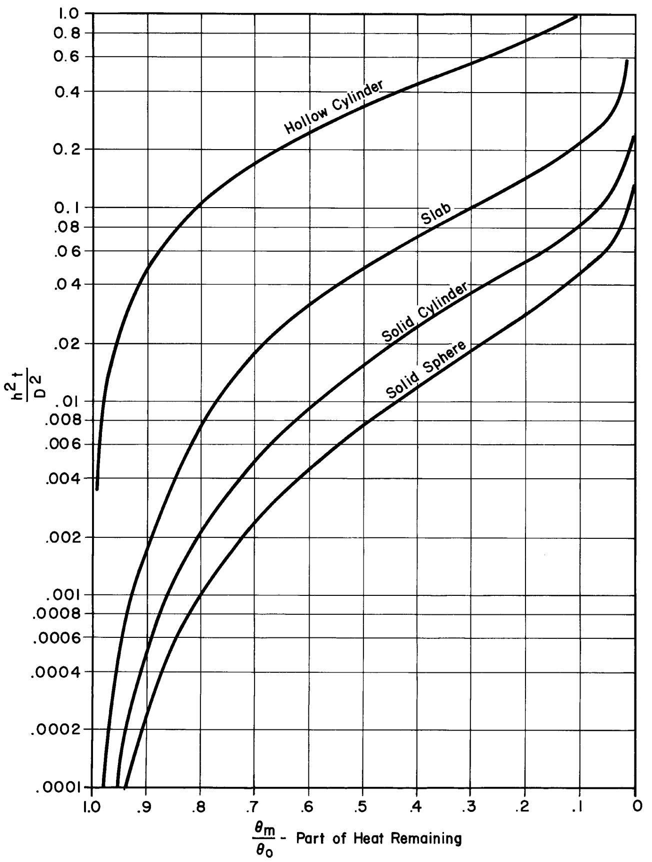


Fig. 4.8—Heat loss from solid bodies.

D = thickness of concrete section, ft (m);
θ_o = initial temperature difference between concrete and ambient material, °F (°C); and
θ_m = final temperature difference between concrete and ambient material, °F (°C).

Example 1—At a certain elevation, an arch dam is 70 ft thick and has a mean temperature of 100 °F. If exposed to air on both faces at 65 °F, how long will it take to cool to 70 °F? Assume h^2 = 1.20 ft^2/day.

Initial temperature difference θ_o = 100 – 65 = 35 °F

Final temperature difference θ_m = 70 – 65 = 5 °F

The portion of the original heat remaining is

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

From Fig. 4.8, using the slab curve, locate 0.14 on the *x*-axis and read the *y*-axis value.

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

Then

$$t = \frac{0.18D^2}{h^2} = \frac{0.18(70)^2}{1.20} = 735 \text{ days}$$

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

Initial temperature difference θ_o = 80 – 40 = 40 °F

For the 25 ft dimension

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

and for the 50 ft dimension

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

Then calculate numerical values of 0.00144*t* and 0.00036*t* for times from 10 to 200 days (refer to Table 4.4). These values can be used with Fig. 4.8 to obtain the θ_m/θ_o ratios for both 25 and 50 ft slabs. The product of these ratios indicates the θ_m heat remaining in the pier, and can be used to calculate the final temperature difference θ_m. The values for θ_m are added to the temperature of the surrounding water to obtain mean pier temperatures at various times up to 200 days, as shown in Table 4.4.

Example 3—Granite aggregate at an initial temperature of 90 °F is to be precooled in circulating 35 °F water for use in mass concrete. The largest particles can be approximated as 6 in. diameter spheres. How long must the aggregate be immersed to bring its mean temperature to 40 °F?

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

Initial temperature difference θ_o = 90 – 35 = 55 °F

Final temperature difference θ_m = 40 – 35 = 5 °F

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

From Fig. 4.8, for θ_m/θ_o = 0.09

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

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

or approximately 17 minutes.

Table 4.4—Calculations for Example 2

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7	Column 8
Time, days	$0.0144t$ (formula)	$0.0036t$ (formula)	$(\theta_m/\theta_o)25$ (lookup)	$(\theta_m/\theta_o)50$ (lookup)	(θ_m/θ_o)pier (Column 4 × Column 5)	(θ_m/θ_o) (Column 6 × 40 °F)	Pier temperature, °F (Column 7 + 40 °F)
10	0.0144	0.0036	0.73	0.87	0.64	25.4	65
20	0.0288	0.0072	0.61	0.80	0.49	19.5	60
30	0.0432	0.0108	0.53	0.77	0.41	16.3	56
40	0.0576	0.0144	0.46	0.73	0.34	13.4	53
60	0.0864	0.0216	0.35	0.67	0.23	7.4	49
100	0.1440	0.0360	0.19	0.57	0.11	4.3	44
200	0.2880	0.0720	0.05	0.40	0.02	0.8	41

Notes: 1) Initial temperature difference = 40 °F; 2) Column 7 = Column 6 × 40 °F (Column 6 × initial temperature difference); and 3) Column 8 = Column 7 + 40 °F) (Column 7 + initial temperature difference).

Example 4—A 50 ft diameter circular tunnel is to be plugged with mass concrete with a diffusivity of 1.20 ft^2/day. The maximum mean temperature in the concrete is 110 °F, and the surrounding rock is 65 °F.

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

Initial temperature difference θ_o = 110 – 65 = 45 °F

Final temperature difference θ_m = 70 – 65 = 5 °F

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

From Fig. 4.8, for a solid cylinder

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

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

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

Cross section handled by each pipe is (4.5)(5.0) = 22 ft^2. The diameter of an equivalent cylinder can be calculated from $22 = \pi D^2/4$ ft^2

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

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

Initial temperature difference θ_o = 105 – 38 = 67 °F

Final temperature difference θ_m = 45 – 38 = 7 °F

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

Referring to Fig. 4.8 and using the curve for the hollow cylinder (because cooling is from within cross section), for the calculated value of θ_m/θ_o

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

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

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

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

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

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

From Fig. 4.8, for a hollow cylinder

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

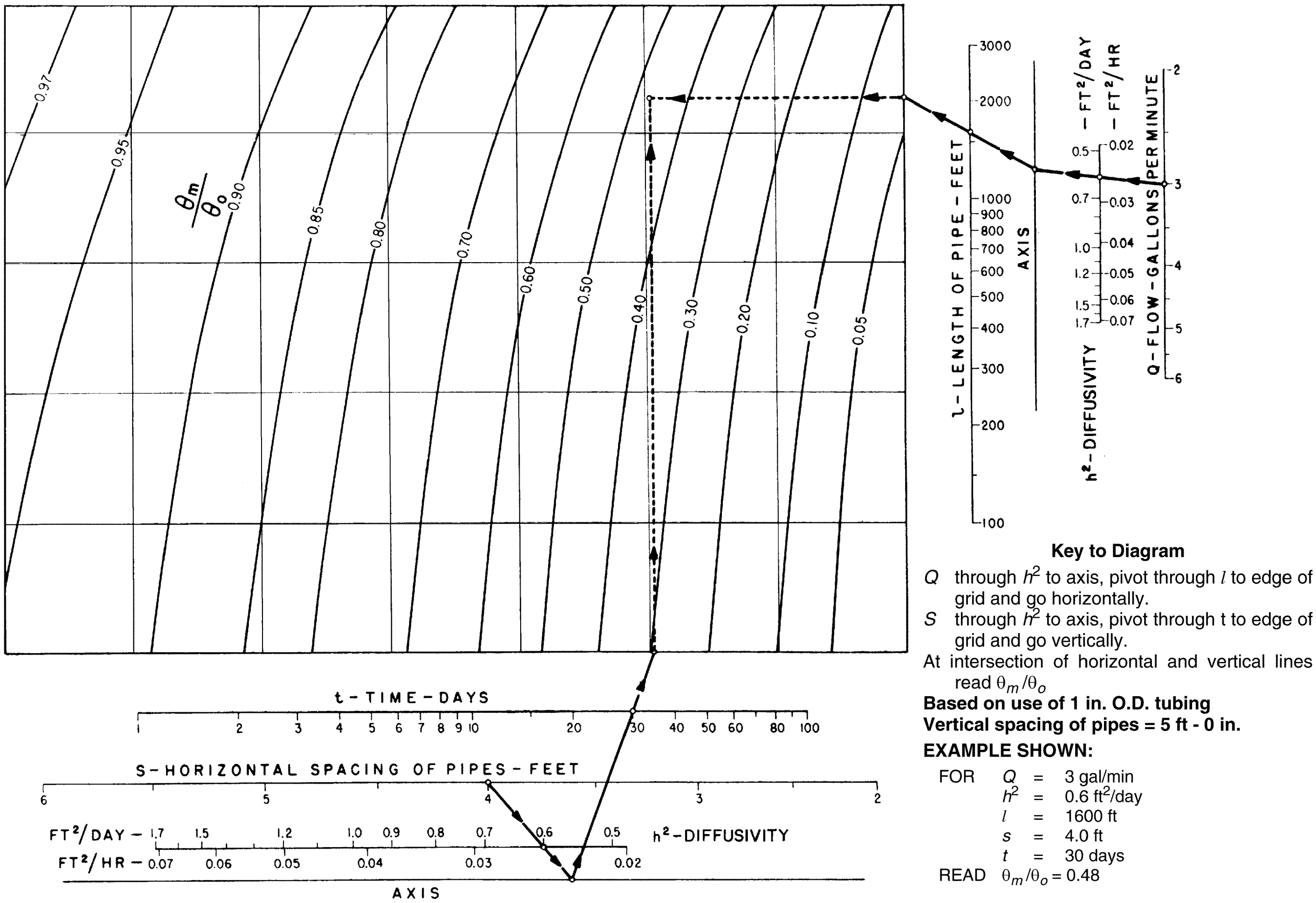
Therefore

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

Total time is 20 + 18 = 38 days, but of this, the time for using refrigeration has been cut by 1/3

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

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



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

Fig. 4.9—Ratio of final mean temperature difference to initial temperature difference θ_m/θ_o, °F/°F (°C/°C).

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

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

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

4.6.3 *Graphical solutions for heat dissipation through mass concrete*—For graphical solutions, Fig. 4.9 through 4.11 can be used for the determination of all the characteristics of an artificial cooling system for mass concrete. Figure 4.9 can be used for the determination of the actual cooling accomplished in a given number of days with a given pipe spacing and flow of coolant. Figure 4.10 gives more detail on the cooling of the mass concrete by determining the temperature at various points along the length of the cooling coil. Figure 4.11 can be used to determine the temperature rise of the coolant in the pipe.

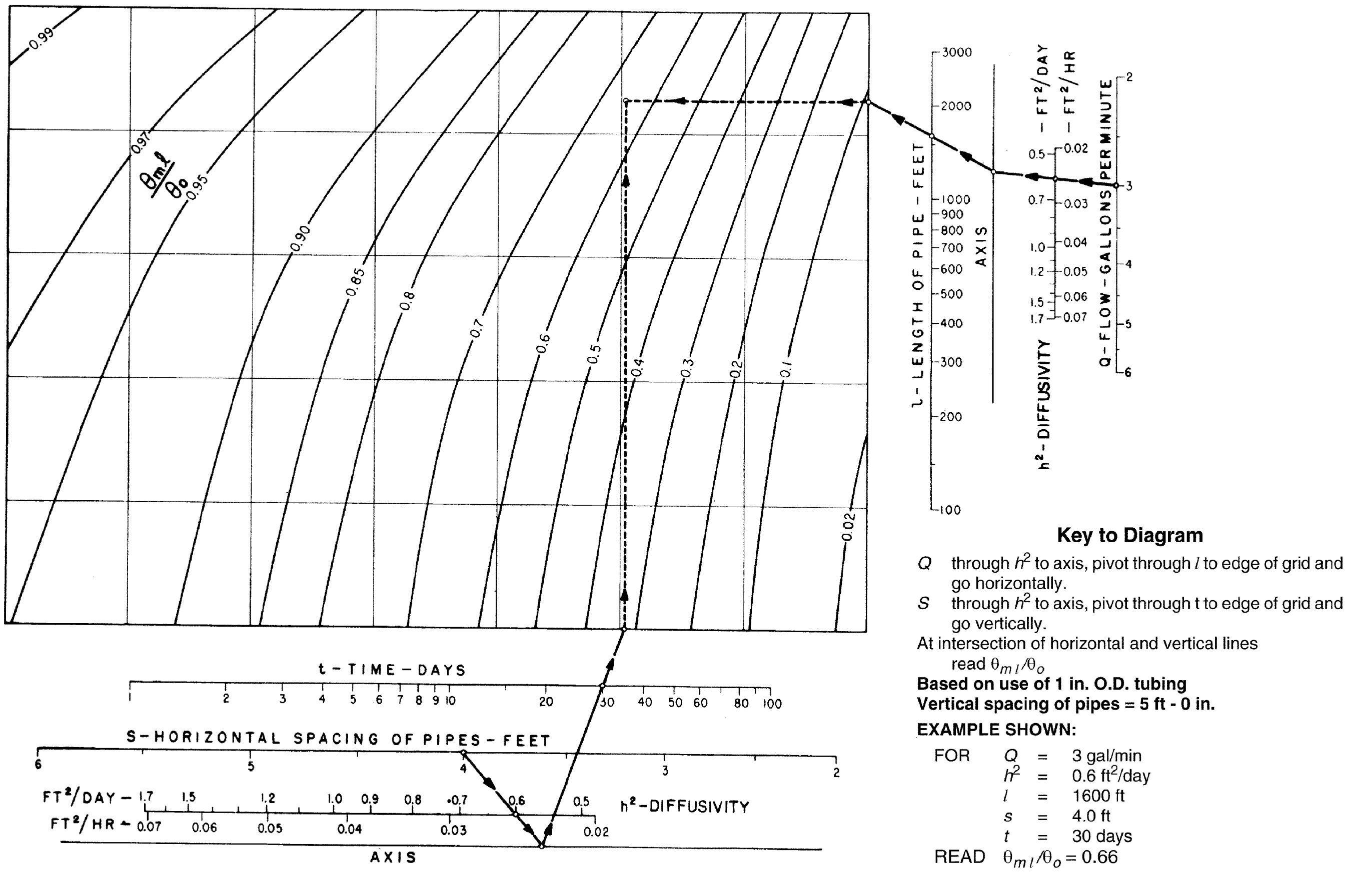
As illustrated in Fig. 4.9, one can determine θ_m/θ_o for a given system of 1 in. (25.4 mm) outside diameter cooling tubes embedded in concrete of known diffusivity. Figure 4.9 can also be used to determine how many days of cooling flow will be required to achieve a desired θ_m/θ_o. Using the figure to solve Example 5 of Section 4.6.2, for which it is given that

Q = 5 gal./minute;
h^2 = 1.03 ft^2/day;
S = 4.5 ft; and
θ_m/θ_o = (45 – 38) ÷ (105 – 38) = 0.104.

and assuming that tube length is 200 ft and cooling water flow in each tube is 5 gal./minute, 35 days will be required to accomplish the required temperature reduction. If tube length is 600 ft, 40 days will be required, according to Fig. 4.9.

The difference in results between the method using Fig. 4.8 and that using Fig. 4.9 is due to the fact that the latter takes into account the variation in temperature of the cooling water along the pipe as it extracts heat from the concrete.

4.6.4 *Schmidt's method for determining temperature gradients*—All the foregoing methods are only approximations; in the typical case, hydration and cooling go on simultaneously. For this general case in which it is necessary to determine actual temperature gradients, Schmidt's method (Rawhouser 1945) has proved to be of immense value. The concept and application is so simple that it can be performed quite easily



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

Fig. 4.10—Ratio of final mean temperature difference at a given length from the inlet to initial temperature difference θ_m/θ_o, °F/°F (°C/°C).

with a desk calculator, and yet for complicated cases can easily be programmed for computer application. Without going into its derivation, it can be said that Schmidt's method is based on the theorem that if the body under question is considered to be divided into a number of equal elements, and if a number of physical limitations are satisfied simultaneously, the temperature for a given increment at the end of an interval of time is the average of the temperature of the two neighboring elements at the beginning of that time interval. The necessary physical relationship is

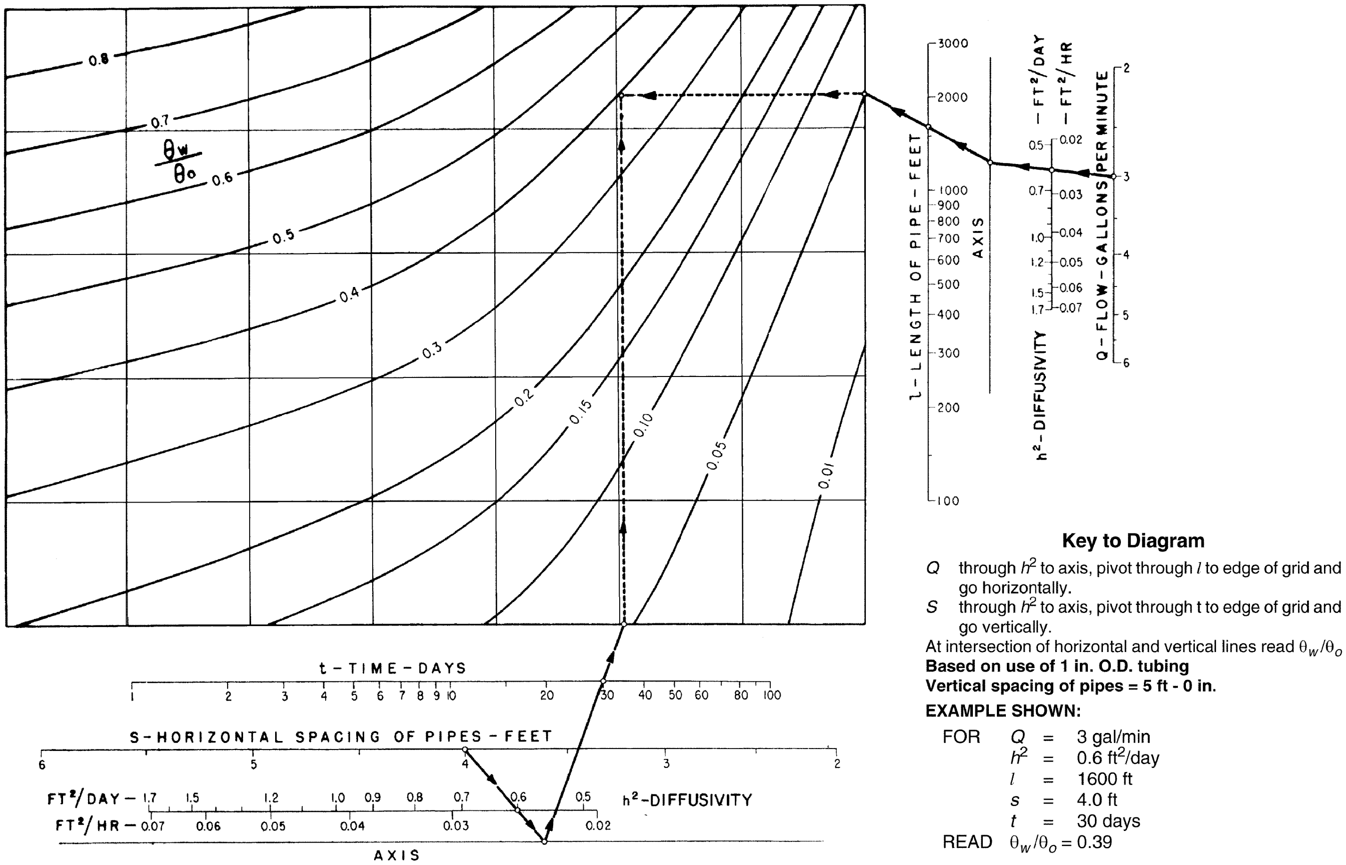
$$\Delta t = \frac{(\Delta x)^2}{2h^2} \qquad (4\text{-}6)$$

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

$$\theta_q + \Delta\theta_q = \frac{(\theta_p + \theta_r)}{2} \qquad (4\text{-}7)$$

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

Normally where there are several stations considered in each lift, the temperature distribution within the lift at any given time can be obtained with sufficient accuracy by calculating only half of the points at any one time, as shown in the tabulated solution. With the use of computers, the calculations of heat- and induced-thermal stresses can be easily determined using the finite-element method. Thermal gradients may also be determined as part of a wider scope two- or three-dimensional nonlinear, incremental structural analysis. Ordinarily used only for very complex mass concrete structures, this method of analysis can evaluate complex geometry of a structure, nonlinear behavior of concrete, structure interaction with the foundation, fill, or other elements such as a reservoir, the effects of sequential



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

Fig. 4.11—Ratio of temperature rise of water in cooling pipes to initial temperature difference θ_m/θ_o, °F/°F (°C/°C).

construction, thermal gradients, added insulation, and surface and gravity forces (U. S. Army Corps of Engineers 1994).

Example 6 (refer to Appendix A for this example worked in SI units)—Determine temperature rise throughout two 6 ft lifts of mass concrete placed at 2-day intervals. The concrete contains 376 lb/yd^3 of Type II cement and has a diffusivity of 1.00 ft^2/day. Take the space interval as 1.0 ft. The time interval needed for the temperature at the center of the space to reach a temperature which is the average of the temperatures of the two adjacent elements, using Eq. (4-6), is

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

In Table 4.5, the adiabatic temperature rise (above the temperature of concrete when it was placed) in half-day intervals for a 3-day investigation is taken from Fig. 4.1 (except that the temperature rise at half-day age is estimated). The change in temperature Δ_θ is determined by subtracting the temperature at any time interval from that of the preceding time interval.

In the tabular solution (Table 4.6) the space interval of 1.0 ft divides each lift into six elements or stations. Boundaries such as rock surface, construction joints, and exposed surfaces should be clearly defined. Note that the adiabatic temperature rise at the rock surface is taken as just one-half of the concrete rise because the rock is not generating heat. At a construction joint, the rise is the average of the two lifts, which are generating heat at different rates at any given time. At the exposed surface, the adiabatic rise is zero because the heat is dissipated as quickly as it is generated from the concrete below. Note that in the aforementioned computation, two steps are required to produce the temperature at the end of the half-day period: the first step averages the adjacent temperatures, and the second step adds the adiabatic temperature rise of the concrete.

Table 4.5—Adiabatic temperature elements for Example 6

Time, days	Adiabatic temperature rise above placing temperature θ, °F (from Fig. 4.1)	Δ_θ
0	0	—
0.5	20	20
1	31	11
1.5	37	6
2	40	3
2.5	42.5	2.5
3	44.5	2.0

4.6.5 *Heat dissipation through reinforced concrete*—Reinforced elements or structures do not generally require the same degree of accuracy in determining peak tempera-

Table 4.6—Calculated temperature rise in concrete above placing temperature using Schmidt's method, °F, for Example 6

Distance above ground, ft	Time t, days											
	0	0.5	1		1.5		2.0		2.5		3	
	$\Delta\theta_1$ = 20 °F	$\Delta\theta_1$ = 20 °F	$\Delta\theta_1$ = 11 °F		$\Delta\theta_1$ = 6 °F		$\Delta\theta_1$ = 3 °F		$\Delta\theta_2$ = 20 °F $\Delta\theta_1$ = 2.5 °F		$\Delta\theta_2$ = 11 °F $\Delta\theta_1$ = 2 °F	
12	—	—	—	—	—	—	—	0	0	0	—	—
11	—	—	—	—	—	—	—	0	—	—	10	21
10	—	—	—	—	—	—	—	0	0	20	—	—
9	—	—	—	—	—	—	—	0	—	—	20	31
8	—	—	—	—	—	—	—	0	0	20	—	—
7	—	—	—	—	—	—	—	0	—	—	20.4	31.4
6	0	0	—	—	0	0	—	0	9.5	20.7	—	—
5	0	20	10	21	—	—	16	19	—	—	25.4	27.4
4	0	20	—	—	26	32	—	—	27.6	30.1	—	—
3	0	20	20	31	—	—	34.2	36.2	—	—	32.7	34.7
2	0	20	—	—	28.5	34.5	—	—	32.8	35.3	—	—
1	0	20	15	26	—	—	26.5	29.5	—	—	28.2	30.2
0	0	10	—	—	15.5	18.5	—	—	20.0	21.2	—	—
–1	0	0	5	5	—	—	10.5	—	—	—	14.5	—
–2	0	0	—	—	2.5	2.5	—	—	5.8	—	—	—
–3	—	0	0	—	—	—	1.2	—	—	—	4.2	—
–4	—	—	—	—	0	—	—	—	0.6	—	—	—
–5	—	—	—	—	—	—	0	—	—	—	0.3	—
–6	—	—	—	—	—	—	—	—	0	—	—	—
Deeper than –6 ft	—	—	—	—	—	—	—	—	—	—	0	—

Notes: $\Delta\theta$ is the adiabatic temperature rise for each time interval and each placement (1 and 2). Distance 0 to –6 ft is foundation; 0 to 6 ft is Lift 1 placed on Day 1; and distance 6 to 12 is Life 2 placed on Day 3. Foundation rock and top of concrete surface adiabatic temperature rise = 0.

tures as unreinforced mass concrete. In unreinforced mass concrete, peak temperatures are determined for the purpose of preventing cracking. In reinforced concrete, cracking is presumed to occur, and the consequences of overestimating or underestimating the net temperature rise is usually minor compared with the overall volume change consideration. Sufficient accuracy is normally obtained by use of charts or graphs, such as Fig. 4.5, to quickly estimate the net temperature rise for concrete members cooling in a constant temperature environment equal to the placing temperature, and by use of Fig. 4.6 to account for the difference in the actual and assumed cooling environment.

Figure 4.5 gives the maximum temperature rise for concrete containing 376 lb of Type I portland cement per cubic yard of concrete (223 kg/m^3) in terms of V/S of the member. The V/S actually represents the average distance through which heat is dissipated from the concrete. This distance will always be less than the minimum distance between faces. In determining the V/S, only the surface area exposed to air or cast against forms should be considered. The insulating effect of formwork should be considered in the calculation of volume of the member. Steel forms are poor insulators; without insulation, they offer little resistance to heat dissipation from the concrete. The thickness of wood forms or insulation in the direction of principal heat flow should be considered in terms of their affecting the rate of heat dissipation (ACI 306R). Each inch of wood has an equivalent insulating value of approximately 20 in. (510 mm) of concrete but can, for convenience, be assumed equivalent to 2 ft (0.6 m) of additional concrete. Any faces farther apart than 20 times the thickness of the member can be ignored as contributing to heat flow. Therefore, for a long retaining wall, the end surfaces are normally ignored.

The V/S can be determined by summing the volume of concrete and including the additional concrete volume assumed to model the effects of insulation. The next step is to sum the area of the surfaces from which heat is assumed to flow. The V/S is the ratio of the two summations. For slabs, V/S should not exceed 3/4 of the slab thickness. While multiple lift slabs are not generally classed as reinforced slabs, V/S should not exceed the height of lift if ample time is provided for cooling lifts.

The temperature rise for other types of cement and for mixtures containing differing quantities of cement or cement plus pozzolan from 376 lb/yd^3 (223 kg/m^3) can be proportioned as per Section 4.1.

Figure 4.6 accounts for the differences in placing temperatures and ambient air temperatures. The V/S for Fig. 4.6 should be identical to those used with Fig. 4.5. In all previous temperature determinations, the placing temperature has been assumed equal to ambient air temperature. This may not be the case if cooling measures have been taken during the hot-weather period or heating measures have been taken during cold weather. When the placing temperature of concrete is lower than the average ambient air temperature, heat will be absorbed by the concrete, and only a proportion of the original temperature difference will be effective in lowering the peak temperature of the concrete. When the placing temperature is higher, the opposite effect is obtained. As an example, for an ambient air temperature of 75 °F (24 °C), the placing temperature of a 4 ft (1.2 m) thick wall 12 ft (3.7 m) high is 60 °F (16 °C) instead of 75 °F (24 °C). The *V/S* would be 4.4 ft (1.3 m), assuming 1 in. (25.4 mm) wooden forms. The age for peak temperature would be 2.3 days from Fig. 4.4. From Fig. 4.6, 50% of the heat difference will be absorbed or 7.5 °F (4.2 °C); therefore, the base temperature or the effective placing temperature for determining temperature rise will be 68 °F (20 °C). In contrast, if no cooling methods are used, the actual placing temperature of the concrete will be 85 °F (29 °C), the age of peak temperature would be 1 day, and the base temperature or effective placing temperature for determining temperature rise will be 81 °F (27 °C).

4.7—Summary and examples

The maximum effective temperature change constitutes the summation of four basic temperature determinations. They are:

1. The effective placing temperature;
2. The final or operating temperature of the concrete;
3. The temperature rise of the concrete due to hydration; and
4. The equivalent temperature change to compensate for drying shrinkage, if any.

Measures for making these determinations have been previously discussed; therefore, the following example problems use most of the calculations required in determining the maximum effective temperature change.

Example 7—Assume: a 2 ft wide retaining wall with rock base and backfill on one side; 20 ft high by 100 ft long placed in two 10 ft lifts, wood forms; summer placing with concrete cooled to 60 °F; concrete mixture designed for a specified strength of 3000 psi or average strength of 3700 psi at 90 days contains 215 lb of Type II cement (adiabatic curve same as Fig. 4.1), 225 lb of fly ash, and 235 lb of water per yd^3. The insulating effect of 1 in. thick wood forms on each face would be to effectively increase the thickness by 2(20)/12 = 4.34 ft (assuming a 1 in. thick wood form facing is equivalent to 20 in. concrete). Determine the temperature change of the concrete that results in thermal contraction.

The basic approach is to determine values for each of the each of the variables in the following expression

$$T_E = (T_{pl} + T_{C+F}) - T_{min}$$

1. Determine the *V/S* for a unit width (1 ft) of one 10 ft lift. Remember that the wood forms provide an additional 20 in. of equivalent concrete on each vertical face that should be considered in the volume calculation.

Volume = 10 × (2 + 20/12 + 20/12) = 53.33 ft^3 (per foot)

Surf area = (10 + 10 + 2) = 22 ft^2

V/S = 53.33/22.0 = 2.42 ft

2. Determine the effective placing temperature T_{pl}:

a. Establish ambient air temperature for summer placement based on locality. Use of the average monthly temperature is good in the absence of more specific information. Assume 75 °F average temperature;

b. From Fig. 4.4, concrete temperature peaks at 2 days. Ignore the fact that Type II may peak slightly later that Type I cement;

c. Using Fig. 4.6, the heat absorbed for *V/S* = 2.4 is approximately 60%; and

d. The net effective placing temperature T_{pl} = 60 + 0.6(75 – 60) = 69 °F.

3. Determine the final exposure temperature T_{min} at 1 week old:

a. Establish minimum exposure temperature for 1-week duration. Ambient temperature data is needed for area. For this example, assume that minimum ambient temperatures are 20 °F;

b. For final exposure conditions, assume that the retaining wall has been backfilled and forms removed. Note that only one 10 ft lift has been placed.

V/S = (10 × 2) ft^3/(10 + 2) ft^2 = 20/12 = 1.67 ft = 20 in.

If no backfill, *V/S* = (10 × 2) ft^3/(10 + 10 + 2) ft^2 = 20/22 = 0.9 ft = 11 in.

c. Using Eq. (4-4) to compute heat dissipation, select an earth temperature appropriate for the region. Assume 60 °F

T_{min} = minimum ambient temperature + 2/3(earth temperature – minimum ambient temperature) × $(V/S/96)^{0.5}$

T_{min} = 20 °F + 2/3(60 – 20) × $(20/96)^{0.5}$ = 32 °F.

4. Determine the temperature rise of the concrete, T_{C+F}:

a. From Fig. 4.5, the temperature rise for Type I cement for dry surface exposure and an effective placing temperature of 69 °F and *V/S* of 2.4 ft = 30 °F;

b. From Fig. 4.1, correct for actual Type II cement peaking at 2 days by ratio of 2-day adiabatic temperatures for each cement type, T_c = (40/50)(30) = 24 °F;

c. Correct for actual cement and fly ash content of actual mixture based on ratio of cement contents assuming that fly ash is equivalent to a weight of cement equal to 1/4 of the weight of fly ash

C_{eq} = 215 + 225/4 = 272 lb

T_{C+F} = 24 °F (272)/(376) = 17.4 °F, say 18 °F

d. Temperature of the concrete at the end of 2 days = 69 + 18 = 87 °F.

5. Determine the equivalent temperature for drying shrinkage. Because *V/S* for final exposure conditions is greater than 15 in., no additional temperature considerations are required for external restraint considerations.

6. Compute the maximum effective temperature change T_E

$T_E = (T_{pl} + T_{C+F}) - T_{min}$

T_E = (69 + 18) – 32 = 87 – 32 = 55 °F

Example 8—Same wall as Example 7, except that no cooling measures were taken and the concrete mixture contains 470 lb/yd^3 of a Type I cement, having a turbidimeter fineness of 2000 cm^2/g and 28-day heat of solution of 94 cal/g. Determine the temperature change of the concrete that results in thermal contraction.

1. Determine the *V/S* for a unit width (1 ft) of one 10 ft lift (same as Example 7).

$V/S = 2.42$

2. Determine the effective placing temperature T_{pl}:

a. With no cooling measures, the placing temperature could be as much as 10 °F above the previously assumed ambient temperature of 75 °F. Assume T_p = 75 °F + 10 °F = 85 °F.

b. From Fig. 4.4, the concrete peaks at 3/4 of a day for 85 °F placing temperature.

c. From Fig. 4.6, the heat dissipated to the environment is 36% of the difference in placing and air temperature:

Difference = 0.36(85 – 75) = 4 °F

T_{pl} = 85 – 4 = 81 °F

3. Determine the final exposure temperature T_{min} at 1 week old. As in Example 7, minimum ambient temperature is 20 °F; therefore, T_{min} remains 32 °F.

4. Determine the temperature rise of the concrete, T_{C+F}:

a. From Fig. 4.5, the temperature rise for 376 lb/yd^3 of Type I cement for dry surface exposure and an effective placing temperature of 81 °F and *V/S* of 2.4 ft = 37 °F.

b. From Fig. 4.1, the adiabatic temperature rise of the Type I cement at 18 hours is 30 °F. Note that this time corresponds to when the concrete reaches peak temperature. This value should be corrected for the various factors because all the figures are based on 376 lb/yd^3 of Type I cement.

c. Correction factor for difference in cement fineness. From Fig. 4.2, the difference in Type I cement fineness for 2000 versus 1800 at 3/4 of a day (18 hours) = 45/38 = 1.18.

d. Correction value for difference in heat of solution of cement. From Eq. (4-2), the temperature difference due to heat of solution

H_a = 0.76(94 – 87) = 5 °F

Note that 87 cal/g is the 28-day heat of hydration for Type I cement with a fineness of 1790 as shown in Fig. 4.1. This value needs to be further corrected for 18 hours. Use the ratio of the 18-hour/28-day adiabatic heat rise for Type I cement from Fig. 4.1.

28-day adiabatic heat rise = 67 °F

18-hour adiabatic heat rise = 30 °F

H_a = 0.76(94 – 87) × 30/67 = 2.4 °F

e. Combining these corrections factors (Lines c and d) results in a corrected 18-hour adiabatic temperature rise for 376 lb/yd^3 or Type I cement of

1.18 × (30 + 2.4) = 38.2 °F

Consequently, the corrected adiabatic temperature rise is 38.2/30 = 1.27 times greater than the adiabatic temperature rise for the cement used in the figures.

f. Correct the effective placing temperature (from 4(a)) by the 1.27 ratio

30 × 1.27 = 47.1

g. Correct for higher actual cement content based on ratio of cement contents

47.1 × (470/376) = 58.9 °F

h. Correct for use of Type II cement based on ratio of 18-hour adiabatic temperature rise for each cement type

58.9 × (25/37) = 39.8 °F, say 40 °F

5. No addition for drying shrinkage.

6. The peak temperature of the concrete at 18 hours: 81 + 40 = 121 °F.

7. The drop in temperature affecting volume change: 121 – 32 = 89 °F.

Comment: The effect of increasing cement content by using no fly ash and implementing no cooling measures results in increasing the peak concrete temperatures from 87 to 121 °F. Upon cooling to a minimum temperature, the temperature change that results in thermal volume change is 55 versus 89 °F, resulting in approximately 60% more volume change.

CHAPTER 5—RESTRAINT

5.1—General

No tensile strain or stress caused by restraint would develop if the length or volume changes associated with decreasing temperature within a concrete mass or element could take place freely. When these potential contractions, either between a massive concrete structure and its foundation, between contiguous structural elements, or internally within a concrete member are prevented (restrained) from occurring, tensile strain and stress will result.

To restrain an action is to check, suppress, curb, limit, or restrict its occurrence to some degree. The degree of restraint K_R is the ratio of actual stress resulting from volume change to the stress that would result if completely restrained. Numerically, the strain is equal to the product of the degree of restraint existing at the point in question and the change in unit length that would occur if the concrete were not restrained.

All concrete elements are restrained to some degree by volume because there is typically some restraint provided either by the supporting elements or by different parts of the element itself. Restrained volume change can induce tensile, compressive, or flexural stresses in the elements, depending on the type of restraint and whether the change in volume is an increase or decrease. We are normally not concerned with restraint conditions that induce compressive stresses in concrete because of the ability of concrete to withstand compression. The primary concern with restraint conditions is that the induced tensile stresses in concrete are the ones that can lead to cracking.

In the following discussion, the types of restraint to be considered are continuous external restraint and internal restraint. Both types are interrelated and usually exist to some degree in all concrete elements.

Concrete placed on an unjointed rigid rock foundation will be essentially restrained at the concrete-rock interface, but the degree of restraint will decrease considerably at locations above the rock, as shown in Fig. 5.1. Yielding foundations will cause less than 100% restraint. Total restraint at the rock plane is mitigated because the concrete temperature rise (and subsequent decline) in the vicinity of the rock foundation is reduced as a result of the flow of heat into the foundation itself. Discussions of restraint and analytical procedures to evaluate its magnitude and effect appear in Tatro and Schrader (1985, 1992) and Gamer and Hammons (1991).

5.2—Continuous external restraint

Continuous restraint exists along the contact surface of concrete and any material against which the concrete has been cast. The degree of restraint depends primarily on the relative dimensions, strength, and modulus of elasticity of the concrete and restraining material.

5.2.1 *Stress distribution*—By definition, the stress at any point in an uncracked concrete member is proportional to the strain in the concrete. The horizontal stress in a member continuously restrained at its base and subject to an otherwise uniform horizontal length change varies from point to point in accordance with the variation in degree of restraint throughout the member. Two restraint factors have been developed to more fully model the restraint conditions on a massive structure: the structural shape restraint factor K_R and the foundation restraint factor K_f.

5.2.1.1 *Structural shape restraint factor* K_R—The distribution of restraint varies with the length-height ratio (L/H) of the member or structural element. The case of concrete placed without time lapses for lifts is shown graphically in Fig. 5.1. This chart has evolved from years of experience in cracking evaluations. Its origins are from test data derivations originally reported by Carlson (1937) and later published by the U.S. Bureau of Reclamation (1965). The restraint factor K_R modifies the degree of restraint and stress in Eq. (5-2) based on the relative dimensions of the concrete structure.

5.2.1.2 *Foundation restraint factor* K_f—The stresses in concrete due to restraint decrease in direct proportion to the decrease in stiffness of the restraining foundation material. The restraint factor K_f to be used in Eq. (5-2) has been approximated by (U.S. Bureau of Reclamation 1965)

$$K_f = \frac{1}{1 + \frac{A_g E_c}{A_F E_F}} \tag{5-1}$$

where

A_g = gross area of concrete cross section;

A_F = area of foundation or other element restraining shortening of element, generally taken as a plane surface at contact;

E_c = modulus of elasticity of concrete; and

E_F = modulus of elasticity of foundation or restraining element.

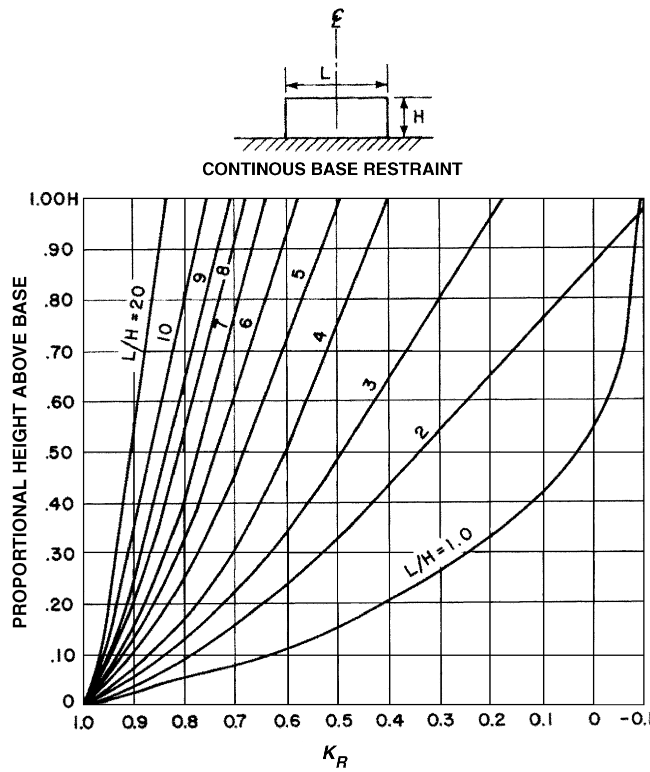


Fig. 5.1—Degree of tensile restraint at center section.

Table 5.1—Foundation restraint factor for foundation rigidity

E_F/E_c	K_f
∞	1.0
2	0.83
1	0.71
0.5	0.56
0.2	0.33
0.1	0.20

In general, for mass concrete on rock, the maximum effective restraining mass area A_F can be assumed at $2.5A_g$, and the values of the multipliers are then shown in Table 5.1.

Using the degree of restraint K_R from Table 5.1 and K_f from Eq (5-1), the tensile stress at any point on the centerline due to a decrease in length can be calculated from

$$f_t = K_R K_f \Delta_c E_c \tag{5-2}$$

where

K_f = degree of foundation restraint expressed as a ratio with 1.0 = 100%;

K_R = degree of structural geometry restraint expressed as a ratio with 1.0 = 100%;

Δ_c = contraction if there were no restraint; and

E_c = sustained modulus of elasticity of the concrete at the time when Δ_c occurred and for the duration involved.

5.2.2 *Cracking pattern*—When stress in the concrete due to restrained volume change reaches the tensile strength of

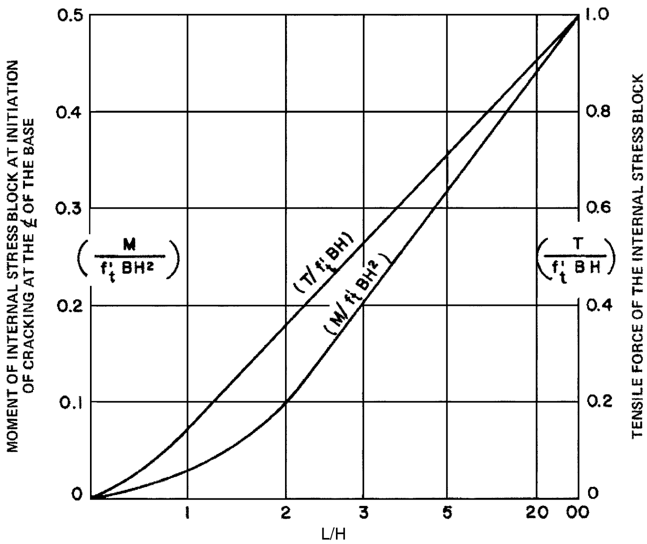


Fig. 5.2—Internal forces at initiation of cracks at restrained base.

the concrete, a crack will form. If a concrete member is subject to a uniform reduction in volume but is restrained at its base or at an edge, cracking will initiate at the base or restrained edge where the restraint is greatest and progress upward or outward until a point is reached where the stress is insufficient to continue the crack. After initial cracking, the tension caused by restraint in the region of the crack is transferred to the uncracked portion of the member, thereby increasing the tensile stresses above the crack. For *L*/*H* greater than about 2.5, Fig. 5.1 indicates that if there is enough tensile stress to initiate a crack, it should propagate to the full block height because of the aforementioned stress-raising feature. Many tests also indicate that, once begun, a crack will extend with less tensile stress than required to initiate it (ACI 224R).

From the preceding discussion, unreinforced walls or slabs, fully restrained at their base and subject to sufficient volume change to produce full-section cracking, will ultimately attain full-section cracks spaced about 1.0 to 2.0 times the height of the block. As each crack forms, the propagation of that crack to the full height of the block will cause a redistribution of base restraint such that each portion of the wall or slab will act as an individual section between cracks. Using Eq. (5-2) and K_R values from Fig. 5.1 to determine the stress distribution at the base centerline, the existing restraining force and moment at initiation of cracking can be determined from the internal stress block for various *L*/*H*, and is shown in Fig. 5.2. Because cracks do not immediately propagate to the full block height throughout the member, a driving force of continuing volume change should be present.

A propagating crack will increase the tensile stress at every section above the crack as it propagates. Once a crack exists, stress is required to propagate the crack into the area of tensile stress that is less than the tensile strength. This depends on several factors, such as the shape of the crack tip, that is, whether it is pointed or rounded. From Fig. 5.3, the maximum restraining force in the stress block, corresponding to maximum base shear, occurs with the volume reduction

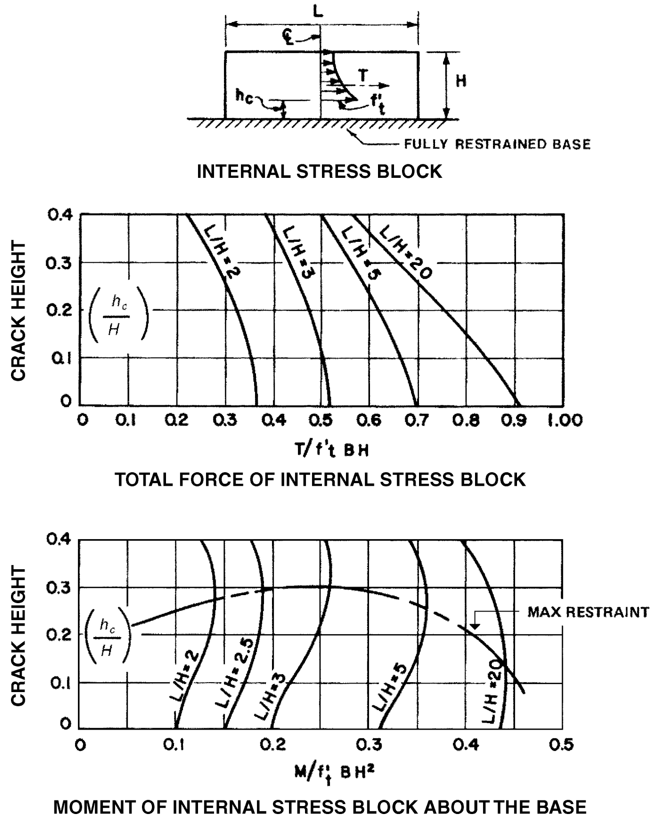


Fig. 5.3—Effect of crack propagation on internal forces.

producing initial cracking. The maximum moment of the internal stress block, corresponding to maximum base restraint, does not occur until the crack propagates to a height of 0.2 to 0.3 times the height of section. At that point, the crack is free to propagate to its full height without a further reduction in volume. From Fig. 5.3, the maximum base restraint at the centerline of a block having an *L*/*H* of 2.5 is approximately $0.18f_t'BH^2$, which rounds up to $0.2f_t'BH^2$. This may be assumed as the minimum base restraint capable of producing full-block cracking. The corresponding spacing of full-block cracking in unreinforced concrete would therefore be approximately 1.25*H*.

5.3—Internal restraint

Internal restraint exists in members with nonuniform volume change on a cross section. This occurs, for example, within walls, slabs, or masses with interior temperatures greater than surface temperatures or with differential drying shrinkage from outside to inside. It also occurs in slabs projecting through the walls of buildings with cold outside edges and warm interiors and in walls with the base or lower portions covered and the upper portions exposed to air.

Internal restraint depends on the differential volume change within a member. Its effects add algebraically to the effects of external restraint, except that their summation will never exceed the effects of 100% external restraint. Therefore, where high external restraint conditions exist, the effects of internal restraint may be negligible.

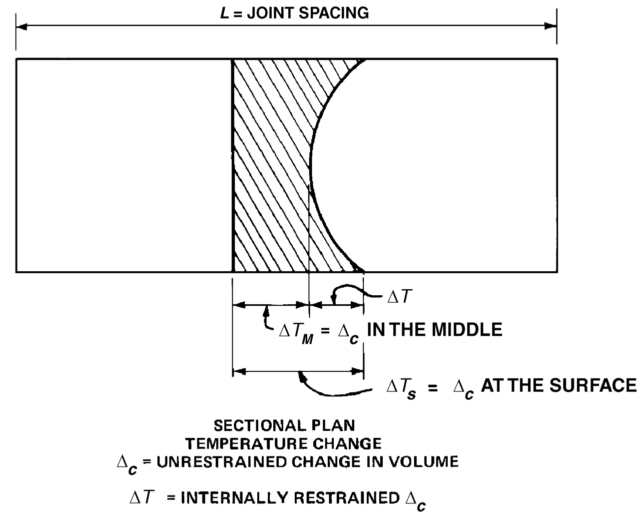
For example, the interior of most concrete structures with a minimum dimension greater than about 2 ft (0.6 m) will be at a temperature above the ambient air temperature at the time forms are removed. At the boundary between the concrete and the forms, the concrete temperature will be below that in the interior but above that of the air. With steel forms, the latter difference may be small, but with insulated steel or wood forms, the difference may be substantial. When the forms are removed in that instance, the concrete is subjected to a sudden steepening of the thermal gradient immediately behind the concrete surface. This sudden thermal shock can cause surface cracking.

Identical circumstances will arise with the approach of the cooler autumn months or the filling of a reservoir with cold runoff. Abrupt and substantial drops in air temperature will cause the near-surface gradient to suddenly steepen, resulting in tensile strains that are nearly 100% restrained. Exposed unformed concrete surfaces are also vulnerable.

These critical conditions are mostly avoided during the second and subsequent cold seasons because much of the heat has been lost from the interior concrete, and the temperature gradient in the vicinity of the surface is much less severe.

5.3.1 *Stress distribution and cracking*—Internal restraint is similar to continuous edge restraint, except that the effective restraining plane is the plane of zero stress in the internal stress block and is dependent on the actual temperature gradient in the concrete (Fig. 5.4). For section stability, the summation of tensile stress induced by the temperature or moisture gradient on a cross section should be balanced by an equal compressive force. This balance line locates the depth d_s of the internal stress block. If the depth of the tensile stress block d_s is large compared with the spacing of joints, L, then the stress induced by volume change will not be significant. As an example, if the annual temperature range at the surface is four times the range in concrete, then a 100 ft (30 m) thick dam would have a 15 ft (4.6 m) deep tensile stress block using the distribution shown in Fig. 4.7. If one assumes a 50 ft (15 m) spacing of joints, the L/d_s ratio would be 3.3, and the degree of restraint at the surface would be 25% using Fig. 5.1 and L/d_s as L/H. In contrast, from the same chart, the daily cycle shows a penetration of only 2 to 2.5 ft (0.6 to 0.8 m) Using 2 ft (0.6 m) as d_s, the degree of restraint at the surface would be approximately 85%, and assuming a concrete tensile strength of 300 psi (2 MPa), a concrete modulus of 3×10^6 psi (2×10^4 MPa), and a coefficient of thermal expansion of 5×10^{-6} in./in./°F (9×10^{-6} mm/mm/°C), cracking would occur at the face with a 24 °F (13 °C) drop in surface temperature. For equal stress, the annual temperature variation should be 82 °F (46 °C). Cracking from the daily temperature cycle is not usually significant in dams and large masses, particularly in moderate climates, because of the limited penetration or significance of such cracks. The 24 °F (13 °C) drop in mean daily temperature corresponds to normal winter temperature fluctuations for moderate climates.

Temperatures on the opposite faces of a wall or slab may not be equal because of a difference in exposure conditions. The variation of temperatures through the slab or wall may be assumed to be parabolic or exponential.



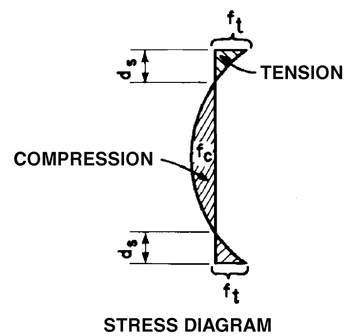


Fig. 5.4—Internal restraint.

Temperature distribution of this sort will curl the slab or wall if unrestrained, or induce bending stresses along the member if its ends are restrained.

The plane of zero stress of the tensile stress block for projecting portions of concrete walls or slabs may be determined by a heat-flow analysis or by trial as just described. The proportion of cold volume to total volume is larger for members of this type than for dams or other large concrete masses. The penetration of the daily temperature cycle may therefore be assumed somewhat more than the 2 to 2.5 ft (0.6 to 0.8 m) penetration previously mentioned for dams. Restraint at the free edge may also be determined for these cases from Fig. 5.1 by setting the depth of the tensile stress block, d_s, as a fixed plane 3 ft (1 m) inside the exterior surface.

CHAPTER 6—CRACK WIDTHS

6.1—General

Large-sized, randomly spaced cracks are usually objectionable in most structures. In reinforced structures, such cracking may indicate that the reinforcement transverse to the crack has yielded. Reinforcement, when used in mass concrete, is intended to restrict the size of cracks that would otherwise occur. Large-sized cracking may be cause for concern, depending on the structure in question and the primary purpose of the reinforcement. Surface-crack widths are important from an aesthetic viewpoint, are easy to measure, and are the subject of most limitations. While the width of a crack at the surface may initially be larger than the crack width at the reinforcement, the difference may be expected to decrease with time. Similarly, fiber reinforcement creates many fine cracks rather than one single large crack; however, fibers may be of limited benefit at the percent of reinforcement normally achieved.

For water-retention elements, very narrow, barely visible cracks (0.002 in. [0.05 mm]) will probably leak, at least initially; however, nonmoving cracks up to 0.005 in. (0.13 mm) may heal in the presence of excess moisture and therefore would not be expected to leak continually. Any leakage may be expected to stain the exposed concrete face or create problems with surface coatings.

Guidance for designing reinforcement to control cracks in concrete structures is provided by ACI 224R.

6.2—Crack control joints

It has been common practice for many years to use expansion and contraction joints to reduce the size and number of uncontrolled cracks. In sidewalk and pavement construction, formed grooves have also been used to create planes of weakness, thereby inducing cracking to coincide with the straight lines of the grooves. This concept has been expanded in the UK as a method of controlling cracks in massive walls and slabs. The practice has also been extended to construction of conventional facing concrete in roller-compacted concrete dams. The British install plastic or metal bond breakers to induce cracks at specific locations. Turton (1977) indicates that a cross-sectional reduction of as little as 10% has proved successful in experiments, but 20% is recommended to ensure full section cracking in practice. The depth of surface grooves is obviously limited by any continuous reinforcement; therefore, some form of void should be cast into massive sections to achieve the needed section reduction. These voids can be formed with plastic pipes or deflatable duct tubes. Alternately, the reduction may be accomplished by using proprietary crack-inducing water barriers that have been designed to act as both bond breakers and water stops. The principal advantage of a crack-control system is that cracking can essentially be hidden by the formed grooves. Also, the crack size (width) loses its significance when there is a water barrier and the reinforcement crossing the crack is principally minimum steel that is not required for structural integrity.

6.3—Limitations

It is desirable to limit the width of cracks in mass concrete structures to the minimum practical size, in keeping with the function of the structure. Reinforced concrete structures are generally designed in accordance with ACI 318 or 350. The reasonable crack widths versus exposure conditions in Table 4.1 of ACI 224R represent a historical viewpoint of tolerable crack width. While they may not represent a current consensus, they do offer guidance to what has been considered acceptable.

Limiting crack width through the use of reinforcement becomes increasingly difficult as member size increases. The most effective means to control thermal cracking in any member is to restrict its peak hydration temperatures. This becomes increasingly important with increasing member size.

CHAPTER 7—REFERENCES

7.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Concrete Institute

116R	Cement and Concrete Terminology
207.1R	Guide to Mass Concrete
207.4R	Cooling and Insulating Systems for Mass Concrete
207.5R	Roller-Compacted Mass Concrete
214R	Evaluation of Strength Test Results of Concrete
224R	Control of Cracking in Concrete Structures
305R	Hot Weather Concreting
306R	Cold Weather Concreting
318	Building Code Requirements for Structural Concrete
350	Environmental Engineering Concrete Structures

ASTM International

C 94/C 94M	Specification for Ready-Mixed Concrete
C 115	Test Method for Fineness of Portland Cement by the Turbidmeter
C 150	Specification for Portland Cement
C 595	Specification for Blended Hydraulic Cement
C 618	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
C 1157	Performance Specification for Hydraulic Cement

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APPENDIX A

A.1—Notation

A = effective tension area of concrete surrounding a group of reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars

A_B = area of a member subject to volume change

A_F = area of foundation or other element restraining shortening of element

A_g = gross area of concrete cross section

B, b = width of cross section

C = weight of portland cement per cubic yard of concrete, lb

C_{eq} = weight of portland cement plus a percentage of the weight of pozzolan per cubic yard of concrete, lb

C_h = specific heat, Btu/lb·°F

C_T = linear thermal coefficient, 5×10^6 per °F for limestone aggregate, 6×10^{-6} per °F for siliceous river gravel aggregate

d_c = thickness of concrete cover measured from the concrete surface at which cracks are being considered to the center of the nearest reinforcing bar

d_s = assumed depth of tensile stress block for internal restraint considerations

E_c = modulus of elasticity of concrete

E_F = modulus of elasticity of foundation or restraining element

E_s = modulus of elasticity of steel

e = eccentricity of a load with respect to the centroid of the section

F_a = weight of fly ash per square yard of concrete, lb

f_c' = specified compressive strength of concrete, psi

f_s = calculated stress in reinforcement, psi

f_t = tensile stress, psi

f_t' = tensile strength of concrete, psi

f_y = design yield stress of steel

H = perpendicular distance from restrained edge to free edge. Where a slab is subject to edge restraint on two opposite edges, H is one-half the distance between edges. For slab-on-ground, H is the slab thickness, ft

H_a = adiabatic temperature rise of the concrete

h^2 = diffusivity in ft^2 per hour

h_c = elemental height of crack above base

h_g = 28-day heat generation of cement by heat of hydration, cal/g

I_c = moment of inertia of gross concrete section subjected to flexure by restraining forces

K = conductivity, Btu/ft/h/°F

K_c = stiffness of vertical restraining element subjected to flexure by restraining forces

K_f = stiffness of floor system being tensioned by restraint

K_R = degree of restraint. Ratio of actual stress resulting from volume change to the stress that would result if completely restrained. In most calculations, it is convenient to use the ratio of the difference in free length change and actual length change to the free length change

k = ratio of depth of compressive area to depth d of flexural member using straight line theory of stress distribution
L = distance between contraction or expansion joints in direction of restraint or overall length of a member undergoing volume change
L' = calculated average distance between cracks
N = number of cracks
n = ratio of modulus of elasticity of steel to that of concrete
S = surface area of concrete member exposed to air
T = tensile force, lb
T_A = average minimum ambient air temperature over prolonged exposure period of 1 week
T_{C+F} = temperature generated by mixture of portland cement and pozzolan
T_c = temperature generated by total quantity of cementitious materials if all were portland cement
T_{DS} = equivalent temperature drop to be used instead of drying shrinkage
T_E = effective temperature change in members including an equivalent temperature change to compensate for drying shrinkage
T_M = temperature of earth or rock mass
T_{min} = minimum temperature of concrete against earth or rock mass, °F
T_{PK} = effective placing temperature after accounting for heat gained from or lost to air, °F
T_p = placing temperature of fresh concrete
T_1 = high temperature in temperature gradient
T_2 = low temperature in temperature gradient
V = volume of concrete member
W_u = water content of fresh concrete, lb/yd^3
W_c = weight of cement per cubic yard of concrete, lb
w = maximum surface crack width, in.
w_c = weight of concrete, lb/ft^3
x = distance between resultant tension force and compression face, in.
Δ_c = contraction of concrete, in./in.

A.2—Metric conversions

1 in.	=	25.4 mm
1 ft	=	0.3048 m
1 in.^{2}	=	645.1 mm^2
1 ft^2	=	0.0929 m^2
1 in.^{3}	=	16.39×10^3 mm^3
1 ft^3	=	0.0283 m^3
1 yd^3	=	0.7646 m^3
1 lb	=	0.4536 kg
1 lb/in.^{2} (psi)	=	6895 Pa
1 kip/in.^{2} (ksi)	=	6.895 MPa
1 lb/ft^2	=	47.88 Pa
1 lb/ft^3 (pcf)	=	16.02 kg/m^3
1 lb/yd^3	=	0.5933 kg/m^3
1 Btu/lb·°F	=	4.87 J/(kg·K)
1 Btu/lb·h·°F	=	1.731 W/m·K
1 in./in./°F	=	1.8 mm/mm/°C
Temperature		
t_C	=	$(t_F - 32)/1.8$
Difference in temperature		
Δt_C	=	$t_F/1.8$

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