3 EFFECTS OF ELEVATED TEMPERATURE ON HIGH-STRENGTH CONCRETE MATERIALS

Early research on the effects of elevated temperature on concrete material properties and performance in large measure was in support of the development of prestressed concrete pressure vessels for nuclear power plant designs. Starting in the 1980s concretes with increasing compressive strengths started to become commercially available and primarily utilized in the construction of high-rise buildings. Use of HSC* offered economic advantages because concrete column size could be reduced, thus permitting more usable space. It also had application in the construction of prestressed girders for bridge construction and other specialized applications in which high performance (e.g., low permeability) is required. Today concretes having compressive strengths up to 140 MPa and above can be produced, with strengths of 172 MPa and above attainable through use of special fabrication procedures.[129]

HSC is produced primarily through use of a relatively low water/cementitious ratio and incorporates silica fume. Because this leads to a reduced permeability relative to normal weight concretes, there has been a concern that the HSC may be more susceptible to explosive spalling under fire conditions due to the buildup of pore pressure in the cement paste. Because one of the primary applications of HSC has been to general civil engineering building structures, most recent research on elevated-temperature effects on concrete materials has shifted to an evaluation of the HSC materials under fire conditions. Because some of the newer generation reactor designs may incorporate HSC, or high-performance concretes, a limited discussion is provided below.

A comprehensive review of the experimental and analytical studies on the performance of concrete when exposed to short-term, rapid heating, such as occurs in fires, has been prepared.[130] Although the report does present some information on normal-strength concretes (NSC), the emphasis of the report is on concretes having initial high compressive strengths. Basic conclusions provided in this report follow:

- The material properties of HSC vary differently with temperature than those of NSC. The differences are more pronounced in the temperature range from 25°C to about 400°C, where higher strength concretes have higher rates of strength loss than lower strength concretes. These differences become less significant at temperatures above 400°C. Compressive strengths of HSC at 800°C decrease to about 30% of the original room temperature strengths.
- For unstressed and stressed tests of HSC, the variations of compressive strength with temperature are characterized by three stages: (1) an initial stage of strength loss (25°C to approximately 100°C), (2) a stage of stabilized strength and recovery (100°C to approximately 400°C), and (3) a stage above 400°C characterized by a monotonic decrease in strength with increase in temperature. HSC has a higher rate of compressive strength loss in the temperature range between 100°C and 400°C compared to NSC.
- For unstressed residual strength tests of HSC, the compressive strength vs temperature relationships are characterized by two stages: (1) an initial stage of minor strength gain or loss (25°C to 200°C), and (2) a stage above 200°C in which the strength decreases with increasing temperature.
- The strength recovery stage of higher strength concretes occurs at higher temperatures than lower strength concretes. Compressive strengths of HSC obtained from the stressed tests are higher than those obtained from the unstressed and unstressed residual strength tests in the temperature range of 25°C to 400°C. The application of preload reduces strength loss in this range of temperature. Varying

*High-strength concrete is defined as concrete having a compressive strength in excess of 41.4 MPa for normal-weight concretes and 27.6 MPa for lightweight concretes.[129]

the preload levels from 25 to 55% of the original compressive strength, however, does not cause significant difference in compressive strengths of HSC at elevated temperatures.

- HSC mixtures with silica fume have higher strength loss with increasing temperatures than HSC mixtures without silica fume.
- The difference between the compressive strength vs temperature relationships of normal weight and lightweight aggregate concrete appears to be insignificant based on the limited amount of existing test data.
- The tensile strength vs temperature relationships decrease similarly and almost linearly with temperature for HSC and NSC. HSC retains approximately 50% of its original tensile strength at 500°C, and NSC retains an average of 45% of its original tensile strength at this same temperature.
- Explosive spalling failure occurs more in HSC than in NSC specimens. The reported temperature range when explosive spalling occurs is from 300°C to 650°C. Factors that influence spalling include original compressive strength, moisture content of concrete, concrete density, heating rate, and specimen dimensions and shape.
- Concrete with dense pastes due to the addition of silica fume are more susceptible to explosive spalling. Likewise, HSC made with lightweight aggregate appears to be more prone to explosive spalling than HSC made of normal weight aggregate concretes. HSC specimens heated at higher heating rates and larger specimens are more prone to spalling than specimens heated at lower rates and of smaller size.
- The failure of HSC is more brittle than NSC at temperatures up to 300°C. With further increase in temperature, specimens exhibit a more gradual failure mode.
- A temperature of 300°C marks the beginning of a higher rate of decrease in modulus of elasticity for all concretes. Lightweight aggregate concretes retain higher proportions of the original modulus of elasticity at high temperature than normal weight aggregate concretes. The difference is more pronounced for unstressed residual strength tests than for unstressed tests.

4 CONCRETE MATERIALS FOR ELEVATED-TEMPERATURE SERVICE

4.1 Elevated-Temperature Cements

NSCs using Type II Portland cement have somewhat limited use for high-temperature applications for the reasons cited previously. Refractory concretes, using Portland cement as the binder, perform poorly when thermally cycled in the presence of moisture, especially when cycled to temperatures above ~430°C. (Adding a fine siliceous material to react with the calcium hydroxide formed during hydration is helpful in alleviating this problem.) Portland cement binders are rarely used for applications above 650°C; hydrothermal, calcium aluminate, or tricalcium aluminate cements are required for such applications.

Hydrothermal (non-Portland) cements have been developed for lining oil wells,[131] but they are also potentially suitable for other applications in which heat may be deleterious to normal concrete materials. The materials are basically polymer silicates whose cure initiates at an activation temperature dependent on material formulation. After curing, the cements are capable of withstanding service temperatures of up to 538°C (up to 1093°C in certain formulations) without alteration of physical or mechanical properties. Additionally, the material system can be formulated to obtain (1) compressive strengths of 68.9 to 137.9 MPa, (2) excellent adhesion to metals except for a1uminum, (3) good resistance to aggressive environments, (4) low permeability, and (5) material system costs comparable with those of special Portland cements. However, the available data generally are limited to those supplied by the manufacturer.

Aluminous or high-alumina cement is a hydraulic cement used to make concrete in much the same manner as normal Portland cement. Calcium aluminate cement is made by grinding a compound formed by fusion or sintering of (1) high-iron bauxite and limestone (low purity), (2) low-iron bauxite with limestone (intermediate purity), or (3) aluminum hydroxide and hydrated lime (high purity). Although composition varies, chemical analyses of representative cements shows the principal oxides to be as follows: CaO, 35 to 44%; $A1_20_3$, 33 to 44%; SiO_2, 3 to 11%; and Fe_20_3, 4 to 12%. The principal products of hydration at room temperature are calcium-aluminate hydrates and some colloidal a1umina.[132,133] The high-alumina cements (1) exhibit rapid strength gains (up to 96.5 MPa in 24 h and 124.1 MPa in 28 d for a water-cement ratio of 0.5), (2) are resistant to aggressive environments, (3) may be used as refractory materials at temperatures up to 1800°C when special white calcium-aluminate cement is used with fused-alumina aggregate, and (4) exhibit creep similar to that of normal concretes loaded to the same stress/strength ratio.[32] However, the high-alumina cements (1) cost several times more than normal Portland cements, (2) must be protected against water loss during curing, (3) lose strength on exposure to hot moist environments unless a rich mix has been used, (4) are generally not compatible with many additives, (5) develop heat on curing ~2.5 times that of normal Portland cement (which may develop cracking and strength reductions in thick sections), (6) may lose workability rapidly after mixing, and (7) can contribute to accelerated steel corrosion. High-purity calcium-aluminate cements are used if high strengths are desired because they have superior resistance to CO attack, provide good workability without requiring water-reducing agents, and provide a high degree of refractoriness. Plasticizer additions generally reduce the strength of calcium-aluminate concrete mixes. The use of ca1cium aluminate cements for structural and load-bearing purposes is cautioned because of the complex chemical phenomenon known as conversion, which depends on time, temperature, and the presence of water. Conversion can cause a significant decrease in strength and an increase in permeability. The effects of conversion can be controlled in nonrefractory applications by employing mix designs and installation practices that enable the use of sufficiently low water-cement ratios.

4.2 High-Temperature Aggregates

Many common coarse aggregates are unsuitable for high-temperature service because they contain quartz, which exhibits a large volume change at ~575°C. Accordingly, crushed stone and gravel-based aggregates suitable for use are limited to diabase traprock, olivine, pyrophylite, emery, and the expanded aluminosilicates (shales, clays, and slates). The latter can be used up to temperatures in the range of 1000 to 1150°C. In principle, all refractory grains may be used as aggregates, but in practice, most aggregates for refractory concretes contain mainly alumina and silica in various forms. The most widely used aggregates are probably calcined flint or kaolin containing 42 to 45% Al_20_3 (Ref. 134). Refractory aggregates such as crushed firebrick (30 to 45% Al_20_3) are stable to temperatures of 1300°C. For temperatures up to 1600°C, aggregates such as fused alumina or carborundum can be used; for temperatures up to 1800°C, special white calcium-aluminate cement and a fused-alumina aggregate are required. Sand, gravel, and traprock aggregates are generally used in calcium-aluminate cement mixes for temperatures below 260°C. Table 3 presents examples of typical aggregates for dense refractory concretes.[134]

4.3 High-Temperature (Refractory) Concrete Mixes

Refractory concrete is defined as a granular refractory material that, when mixed with water, will harden at room temperature to support its own weight sufficiently.[134] Generally a calcium-aluminate cement is used as the binder; however, sodium silicates and certain phosphates have also been used. Refractory concretes are classified according to strength and service limit criteria.[135]

The effect of water content on physical properties is critical. The amount of water necessary for a given material will depend on a number of factors: material proportions, ambient temperature, water temperature, type and speed of mixer, and size and shape of member to be cast. Excess water can seriously degrade the strength of dense refractory concrete, Fig. 98.[136] Mixing and curing temperature can affect the type of hydrates formed in set concrete. A castable develops its full hydraulic bond because of chemical reactions between calcium-aluminate cement and water. To get maximum benefits from these chemical reactions, it is preferable to form the stable C_3AH_6 during the initial curing period. The relative amount of C_3AH_6 formed vs metastable CAH_{10} and C_2AH_8 can be directly related to the temperature at which the chemical reactions take place.[137] Hydration of calcium-aluminate cements is an exothermic reaction. The specific heat of these cements is the same as Portland cement, 0.20 cal/g. Conversion of high-alumina cement hydrates, which occurs if the cement is allowed to develop excessive heat, does not present the same problem in refractory concretes that it does in high-alumina cement concretes used for structural purposes. Three principal techniques may be used for installing refractory concretes: troweling, casting, and shotcreting.

4.4 Properties of High-Temperature Concrete

Properties of refractory concretes are both time and temperature dependent. Initial heating of a high-temperature concrete causes physical and chemical changes (largely associated with eliminating combined water) and slight volume changes (usually shrinkage). Volume change produces two independent effects: (1) reversible thermal expansion* and (2) permanent change occurring during setting

*Generally values are ~5×10^{-6} cm/cm/°C, but can be as high as 9×10^{-6} cm/cm/°C.

Table 3 Aggregates (%) Used in Dense Refractory Concretes

Typical chemical composition	Calcined fireclay	Calcined Alabama bauxite		Calcined S. A. bauxite	High purity alumina sintered or fused	Chrome ore (Phillipine)
SiO_2	45–55	34.9	25.9	7.0	0.06	5.5
Al_2O_3	40–50	60.6	70.1	87.5	99.5	31.0
Fe_2O_3	0.5–1.5	1.3	1.1	2.00	0.06	15.5
TiO_2	1.0–2.0	2.5	2.9	3.25	Trace	
CaO	0.1–0.2	0.07	0.05	Trace	Trace	0.5
MgO	0.05–0.1	0.12	0.03	Trace	Trace	16.0
Cr_2O_3						31.5
Alkalies	0.5–1.5	0.11	0.13	Trace	0.07	
Pyrometric cone equivalent	30–34	37–38	38–39	38+	Not determined	Not determined
Bulk specific gravity	2.4–2.6	2.7–2.8	2.85–3.0	3.1	3.4–3.6[a] 3.7–3.9[b]	3.9
Open-porosity	3–10	3–7	4–10	12–20	5.0[a] 0–3[b]	

[a]Sintered.
[b]Fused.

Source: W.T. Bakker, "Properties of Refractory Concretes," Paper SP 57-2 in Refractory Concrete, American Concrete Institute, Farmington Hills, Michigan, 1978.

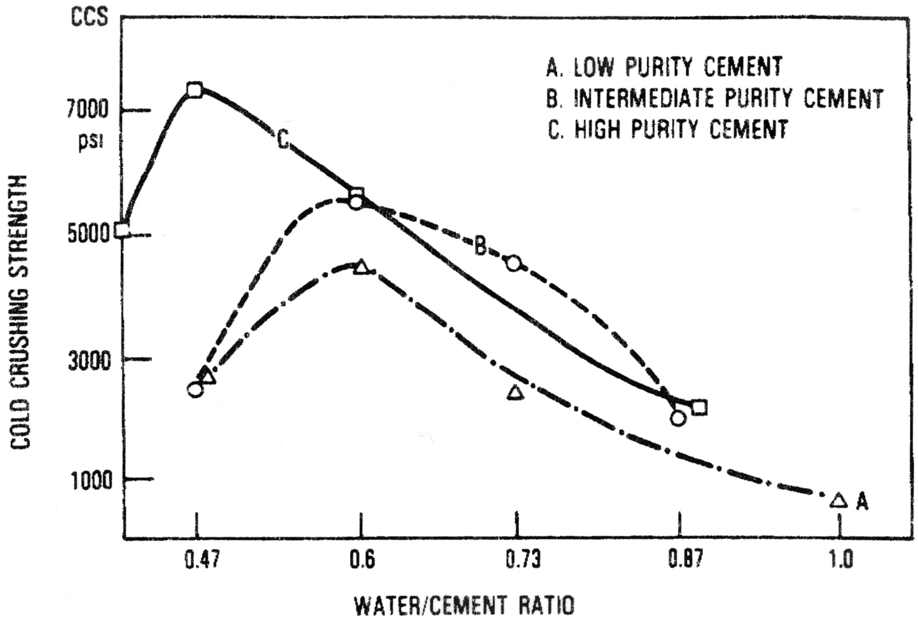

Figure 98 Effect of water/cement ratio on dried strength of dense refractory concrete. *Source*: A. V. Briebach, "A Review of Refractory Hydraulic Cement," *J. Brit. Cera, Soc.* **71**(7), 153–58.

and dehydration of the concrete and again when the glassy bond is formed at high temperatures.[138] Most normal weight high-temperature concretes will have <0.5% permanent linear shrinkage after firing at 1090°C (Ref. 137). Figure 99 presents length change as a function of temperature of a typical high-temperature concrete.[137]

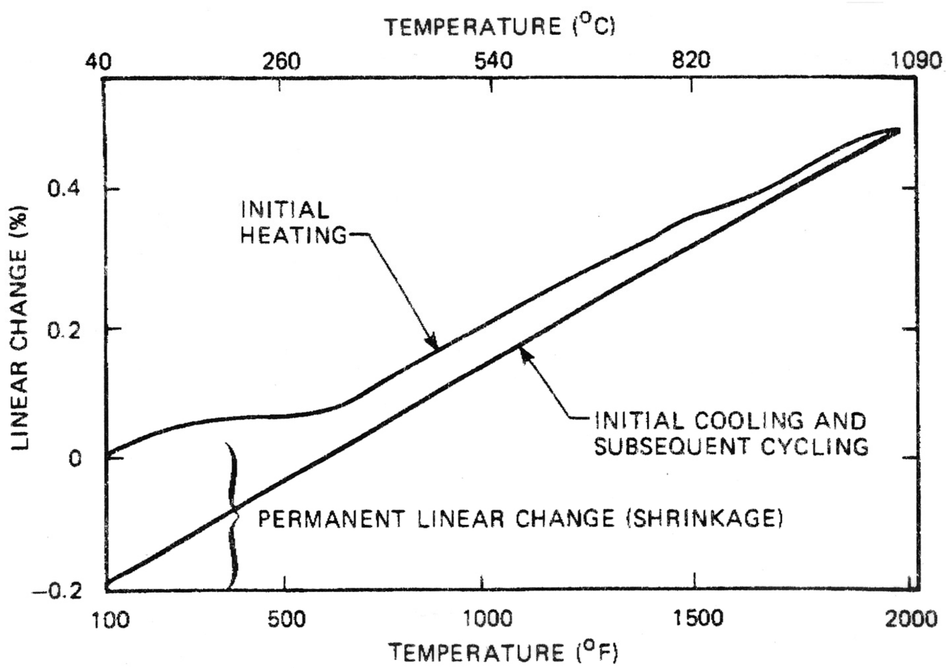

Figure 99 Length change as a function of temperature of a typical high-temperature concrete. *Source*: F. E. Linck, *Turnaround Maintenance*, Houston, Texas, October 6, 1980.

Strength properties are generally measured through compression and modulus-of-rupture tests. Generally, measurements are conducted at room temperature, probably because of difficulties in determining strains at elevated temperatures. Most high-temperature concretes have a marked decrease (25 to 50%) in strength when heated from 105°C to 540°C. Further heating from 540°C to 1090°C usually has only a slight effect on strength. At about 1090°C, initial liquid formation occurs, and the hot strength decreases considerably. Specimens heated above 1090°C and tested after cooling show a marked increase in cold strength because the liquids formed during heating vitrify on cooling to produce high cold strengths. Room-temperature compressive strengths of dense refractory concretes generally range between 13.8 and 55.2 MPa (Ref. 134). The effect of elevated temperature on modulus of elasticity is relatively minor when compared with normal Portland concrete systems. Figure 100 presents typical modulus-of-elasticity curves as a function of temperature for refractory concretes containing low- and high-purity cements.[138] Although data are limited, the modulus of elasticity tends to vary with strength, and values range from 6.9 to 55 GPa. Generally, the modulus of rupture of dense refractory concretes varies from about 4.8 to 10.3 MPa after drying at 104°C (Ref. 134). Figure 101 presents typical hot and cold modu1us-of-rupture results as a function of temperature for 40 to 50% Al_2O_3 castables using a high-purity and intermediate-purity cement binder.[139] Figure 102 presents the effect of temperature on the stress-strain behavior of alumina-silicate bricks (85% alumina) tested in three-point bending.[140]

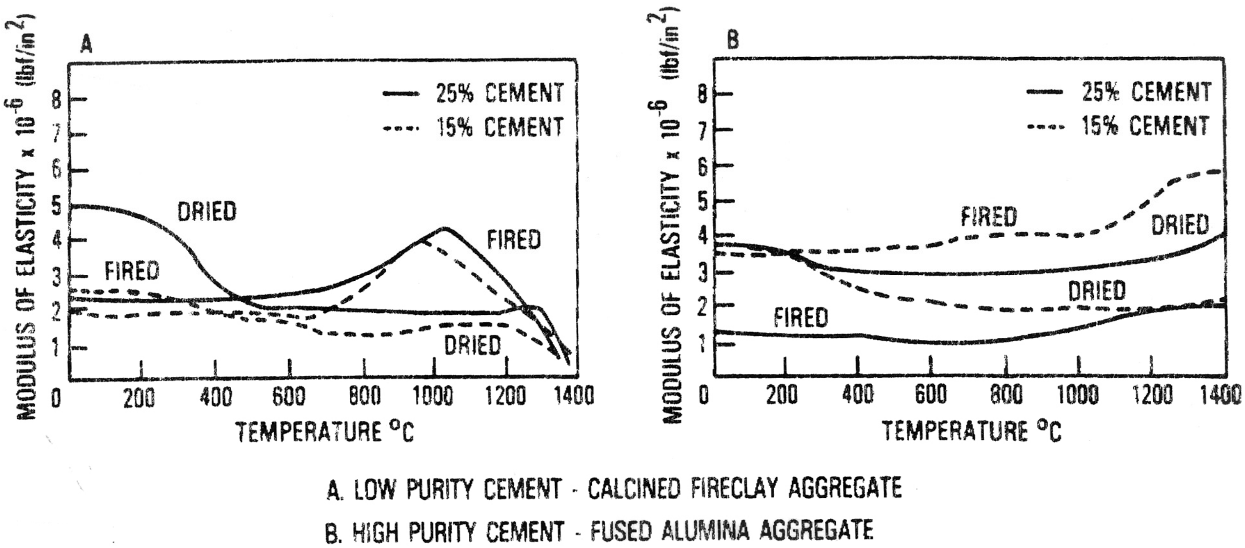

Figure 100 Typical modulus of elasticity curves for refractory concretes containing low- and high-purity cements. *Source*: J. M. McCullough and G. R. Rigby, "Mechanical Properties of Refractory Castables," *J. Brit. Cera. Soc*. **71**(7), 233.

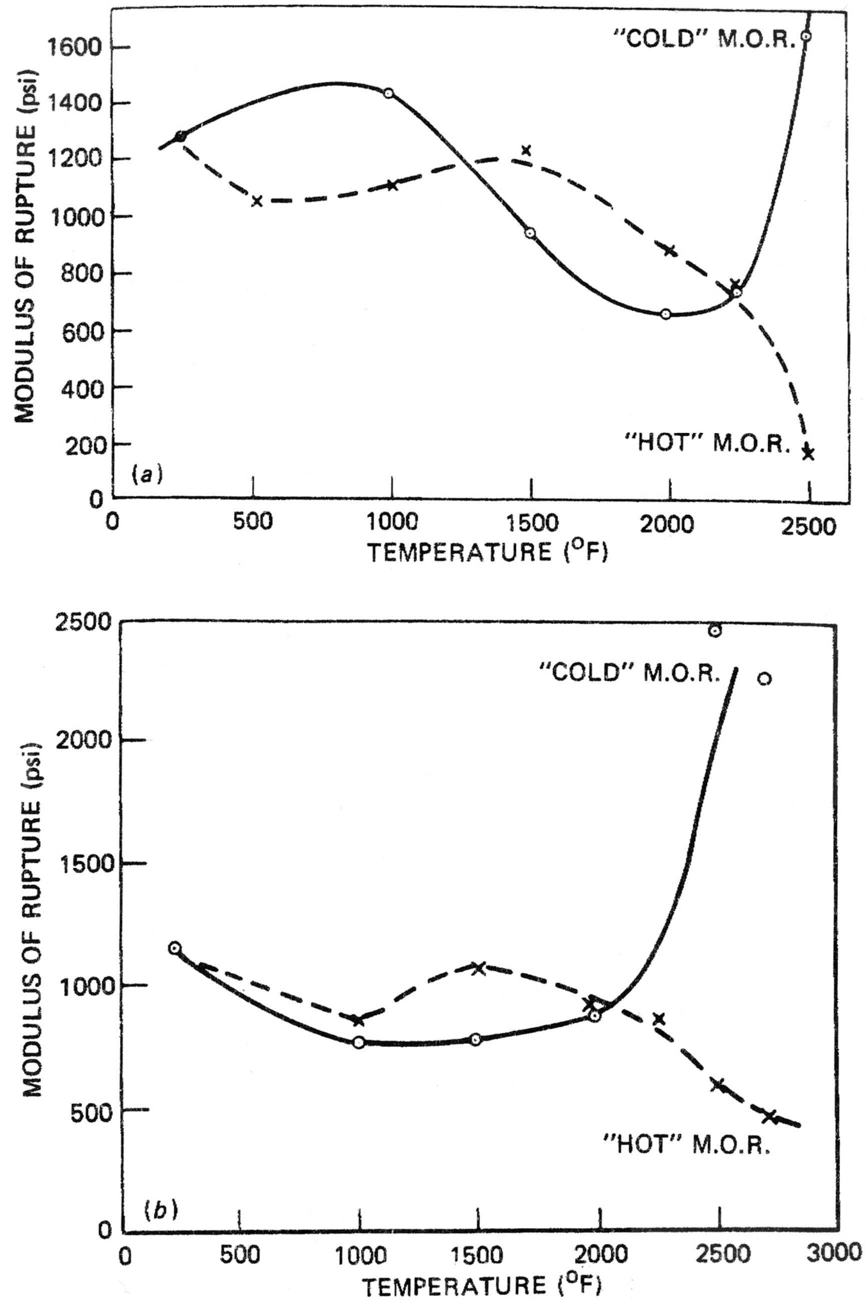

Figure 101 **Typical hot and cold modulus of rupture results for a 40–50% $A1_2O_3$ castable using (a) intermediate-purity cement and (b) high-purity cement.** *Source*: W. T. Bakker et al., "Blast Furnace Gunning in the USA," *Proceedings International Feuerafest Colloquium, Aachen, Germany, October 27–29, 1971*.

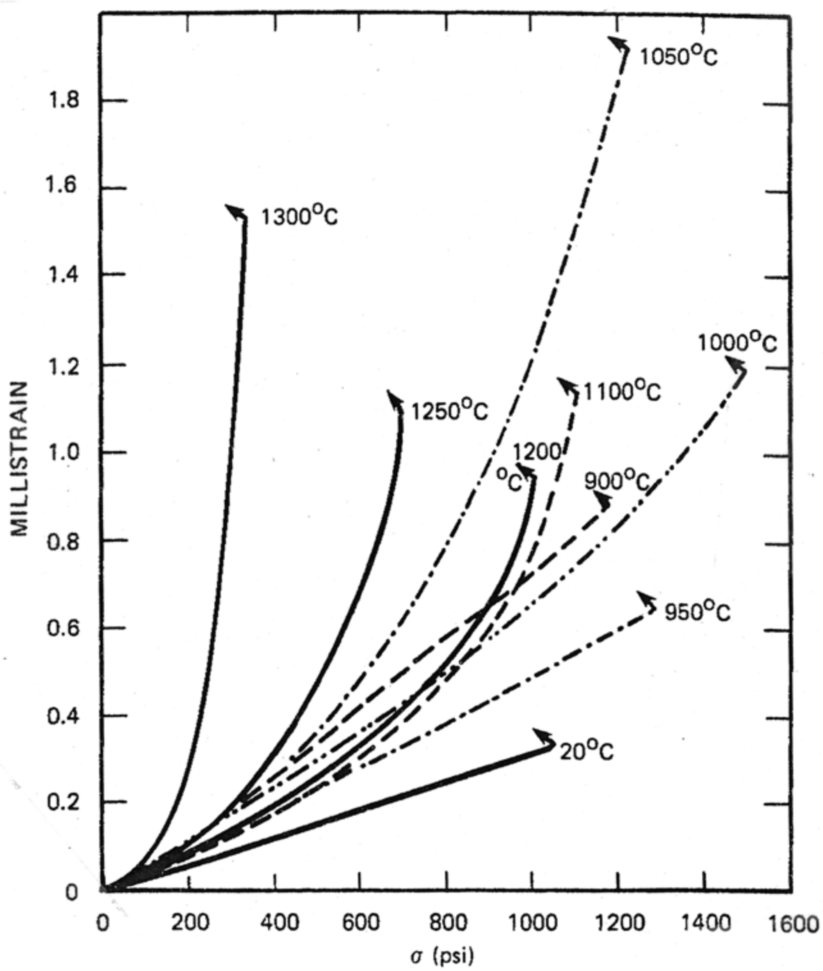

Figure 102 **Effect of temperature on stress-strain behavior of alumina-silicate bricks.** *Source*: G. C. Padgett et al., "Stress/Strain Behavior of Refractory Materials at High Temperatures," Research Paper 608, The British Ceramic Association.

For normal-weight concretes, thermal conductivity tends to increase with density and temperature (some high-alumina concretes may show a decrease with temperature) as shown in Fig. 103.[141] On first heat-up of refractories, generally a drop in thermal conductivity occurs as a result of binder dehydration; however, in actual applications as liner materials, the concrete at the cold face never gets dehydrated, so the thermal conductivity curve before dehydration is used for design. The presence of high thermal conductivity gases will significantly increase the overall thermal conductivity of the refractory 1iner.[142] Typical k factors range from about 72 W-cm m^{-2} $°C^{-1}$ for 1920 kg/m^3 material to about 144 W-cm m^{-2} $°C^{-1}$ for 2560 kg/m^3 material.[137] The specific heat of refractory concrete depends on its chemical composition and increases with temperature. Typical values range from 837 J kg^{-1} $°C^{-1}$ at 40°C to 1210 J kg^{-1} $°C^{-1}$ at 1370°C (Ref. 137). Total creep of refractory concretes does not vary much with temperature. The materials generally deform plastically at relatively low loads [~0.2 MPa] at temperatures greater than 1090°C and at high loads [3.4 to 13.8 MPa] for temperatures as low as 316 to 538°C.[143] Creep evidently proceeds by the cement deforming until contacts between aggregate particles are estab1ished.[143]

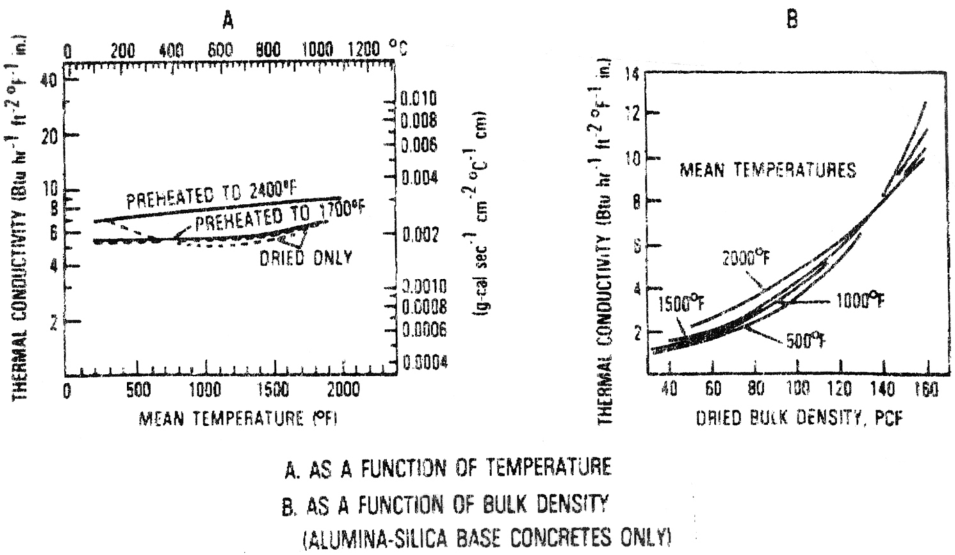

Figure 103 **Thermal conductivities of refractory concrete as a function of temperature and dried bulk density.** *Source*: E. Ruh and A. Renky, "Thermal Conductivity of Castable Refractories," *J. Am. Cera. Soc.* **46**(2), 1963.

The apparent porosity of most normal-weight high-temperature concretes is ~30 to 35% but may be as low as 20% for unfired samples because of closed pores and combined water. Permeability of concretes fired to 820°C and measured at room temperature is very low—typically 15 millidarcys, but can be as high as 1000 millidarcys. When fired to between 820°C and 1090°C, the room-temperature permeability may increase by a factor of 2 to 3 (Ref. 137).

4.5 Refractory-Insulating Concretes

Refractory-insulating concretes generally utilize calcium-aluminate cements as binders. When designed for heat retention purposes, the insulating concretes should not be subjected to impact, heavy loads, abrasion, erosion, or other physical abuse. Normally both the strength and the resistance to destructive forces decline as the bulk density decreases. However, a number of special refractory castables are available (high strength or extra strength) that have better-than-average load-bearing capabilities and can withstand abrasion or erosion much better than standard types. The lightweight refractory concretes are classified by bulk density (880 to 1680 kg/m^3) and service temperature (927°C to 1760°C).[135]

Lightweight aggregate refractory-insulating concretes require the same care in selection, aggregate gradation, and mix design as any other concrete mix. Differences in gradation and fines material content between specific aggregate types can produce variations in cement/aggregate volume, water requirements, and workability or plasticity characteristics. These variations can subsequently affect the porosity, strength, unit weight, and linear length change of the concrete. Fillers that generally consist of common refractory grains such as calcined kaolin, calcined bauxite, or kyanite (reduces high-temperature shrinkage) may be used to achieve proper grain sizing and desirable physical properties. Small amounts of finely ground plastic clay are sometimes added to a given mix to increase the workability or plasticity during placement; however, shrinkage of the concrete may increase proportionally with the clay additions, and setting time and strength may also be adversely affected. Also, short, randomly oriented fibers (stainless steel, fiberglass, tungsten, niobium, molybdenum) can be added to refractory concretes to provide improved properties relative to tensile strength, impact resistance, thermal shock resistance, and thermal stress resistance.[144,145] Table 4[134] presents some typical lightweight aggregate materials used in refractory-insulating concretes, and Table 5[137] presents maximum service temperatures of selected aggregates mixed with calcium-aluminate cements under optimum conditions.

Properties of refractory concretes are time and temperature dependent. Porosities are higher than regular refractory concretes (on the order of up to 50%) because of the highly porous nature of the filler materials. Heat capacity is proportional to density; thus, it is low for these materials. Hot modulus-of-rupture values obtained for an expanded-clay insulating refractory concrete range from 0.6 MPa at 1482°C to 2.4 MPa at 110°C (Ref. 134). Cold compressive strengths vary between 1.4 and 3.4 MPa for materials having densities up to 800 kg/m^3 and between 6.9 and 17.2 MPa for materials having a density of 1200 to 1600 kg/m^3 (Ref. 134).

Table 4 Some Typical Lightweight Aggregate Materials Used in Refractory Concrete

Generic name	Perlite	Expanded shale (haydite)	Expanded fireclay	Alumina bubbles
Typical chemical composition, %				
Al_2O_3	19.5	24.0	27.1	99.0
SiO_2	70.0	63.0	64.3	0.8
Fe_2O_3	0.8	5.5	2.1	0.15
TiO_2	0.1	1.5	2.0	Trace
Alkaline earths	0.3	4.0	0.8	Trace
Alkalies	8.2	2.0	3.3	0.5
Bulk density, lb/ft^3 (kg/m^3)	9–11 (144–176)	55–60 (881–961)	28–32 (449–513)	34–38 (545–609)
Pyrometric cone equivalent	8–11	Not determined	27	>38
°F (°C)	2300–2450 (1260–1343)		2980 (1638)	>3400 (>1871)

Source: W. T. Bakker, "Properties of Refractory Concretes," Paper SP 57-2 in *Refractory Concrete,* American Concrete Institute, Farmington Hills, Michigan, 1978.

Table 5 Maximum Service Temperatures of Selected Aggregates Mixed with Calcium Aluminate Cements Under Optimum Conditions

Aggregate	Remarks	Maximum temperature [°F (°C)]
Alumina, bubble	Refractory, insulating	3300 (1820)
Alumina, fused	Refractory, abrasion resistant	3400 (1870)
Alumina, tabular	Refractory, abrasion resistant	3400 (1870)
Bauxite, calcined		3000 (1650)
Chrome-magnesite		3000 (1650)
Chromite	Slag resistant, high thermal conductivity, heavy	3000 (1650)
Corundum		3270 (1800)
Diatomaceous earth, calcined	Insulating	1830 (1000)
Dolomitic limestone (gravel)	Abrasion and corrosion resistant	930 (500)
Emery		2010 (1100)
Fireclay, expanded	Insulating, abrasion and corrosion resistant	2980 (1640)
Fireclay brick, crushed	Abrasion and corrosion resistant	2910 (1600)
Fireclay brick, crushed insulating	Insulating (maximum temperature depends on Al_2O_3 content)	2730 (1500)
Flint fireclay, calcined		3000 (1650)
Fly ash, expanded	Insulating (depends on composition)	2190 (1200)
Kaolin, calcined	Abrasion and corrosion resistant	3000 (1650)
Kyanite, calcined		3000 (1650)
Limestone (gravel)	Abrasion and corrosion resistant	1290 (700)
Mullite		3000 (1650)
Olivine		2500 (1370)
Perlite	Insulating	2450 (1340)
Pumice, expanded	Insulating	2000 (1090)

Table 5 (continued)

Aggregate	Remarks	Maximum temperature [°F (°C)]
Pyrophyllite[a]		2370 (1300)
Sand	Abrasion and corrosion resistant (silica content less than 90% not recommended)	570 (300)
Shale, expanded	Insulating, abrasion and corrosion resistant	2190 (1200)
Silicon carbide	High thermal conductivity	3090 (1700)
Sillimanite		2910 (1600)
Slag, blast furnace (air cooled)	Abrasion resistant	1000 (540)
Slag, blast furnace (granulated)	Insulating, abrasion and corrosion resistant	2190 (1200)
Slate, expanded	Insulating, abrasion and corrosion resistant	2190 (1200)
Trap rock, diabase	Abrasion and corrosion resistant (basic igneous rock—minimal quartz)	1830 (1000)
Vermiculite	Insulating	2010 (1100)

[a]The properties of pyrophyllite vary considerably, depending on the source and type. Note that both calcined and uncalcined pyrophyllite can be used; however, uncalcined pyrophyllite may undergo significant volume change on heating.

Source: F. E. Linck, "Turnaround Maintenance," Houston, Texas, October 6, 1980.

5 ELEVATED TEMPERATUE DESIGN CONSIDERATIONS

5.1 Significance and Current Practice

As noted previously, thermal gradients are important to concrete structures because they affect the concrete's compressive strength and stiffness. The compressive strength influences the load-carrying capacity, and the stiffness (modulus of elasticity) affects the structural deformations and loads that develop at restraints. Table 6 presents current ASME Code limits for various locations in a prestressed concrete reactor vessel (PCRV) for the appropriate conditions (normal operation and abnormal environment).[1] As noted in this table, the temperature in the concrete should not exceed 65°C at the liner-concrete interface and in the bulk concrete. Between cooling tubes (near the liner), 93°C is given as the maximum allowable. The French specification for PCRVs[146] limits temperatures in active parts of the concrete to 90°C; the British specification[147] states that if the normal operating temperature of any section of the vessel structure is such that the failure strength of the concrete at that temperature is significantly less than at ambient temperature, this will be taken into account. The British specification further notes that most concrete mixes subjected to temperatures above 100°C will suffer a reduction in compressive strength, and concrete with certain aggregates, particularly limestone, may suffer significant losses below that temperature. Figure 104 presents the BS8110 design curves for strength reduction with temperature of unsealed (a) dense concrete and (b) a lightweight aggregate concrete. Permissible temperatures for the concrete in PCRVs for gas-cooled reactors has generally been limited to the range of 45 to 80°C (Ref. 148).

5.2 Design Criteria

General Design Criteria 1. "Quality Standards and Records;" 2, "Design Bases for Protection Against Natural Phenomena;" and 4, "Environmental and Missile Design Bases," of Appendix A, "General Design Criteria for Nuclear Plants," to *10 CFR 50*, "Licensing of Production and Utilization Facilities," require, in part, that structures, systems, and components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the safety functions to be performed and that they be designed to withstand the effects of postulated accidents and environmental conditions associated with normal operating conditions.

Initially, existing building codes such as American Concrete Institute (ACI) Standard 318-71, "Building Code Requirements for Reinforced Concrete (ANSI A89.1-1972)" were used in the nuclear industry as the basis for the design of concrete structural members. However, because the existing building codes did not cover the entire spectrum of design requirements and because they were not always considered adequate, the U.S. Nuclear Regulatory Commission (USNRC) developed its own criteria for the design of Category I structures.* In particular, definitions of load combinations for both operating and accident conditions were provided, as well as a list of tornado-borne missiles and a description of the characteristics of tornados for different regions of the United States.

Using ACI 318-71 as a basis, with modifications to accommodate the unique performance requirements of nuclear plants, ACI Committee 349 developed and published in October 1976 ACI 349-76, "Code Requirements for Nuclear Safety Related Structures." The procedures and requirements described in this

*Category I structures are those essential to the function of the safety class systems and components, or that house, support, or protect safety class systems or components, and whose failure could lead to loss of function of the safety class system and components housed, supported, or protected.

Table 6 Condition Categories and Temperature Limits for Concrete and Prestressing Systems for PCRVs

Load category	Area	Temperature limits [°F (°C)]
Construction	Bulk concrete	130 (54)
Normal	Liner	
	Effective at liner-concrete interface	150 (66)
	Between cooling tubes	200 (93)
	Bulk concrete	150 (66)
	Bulk concrete with nuclear heating	160 (71)
	Local hot spots	250 (121)
	Distribution asymmetry	50 (10)
	At prestresing tendons	150 (66)[a]
	Liner interface transients (twice daily) range	100–150 (38–66)
Abnormal and severe environmental	Liner	
	Effective at liner-concrete interface	200 (93)
	Between cooling tubes	270 (132)
	Bulk concrete	200 (93)
	Local hot spots	375 (191)
	Distribution asymmetry	100 (38)
	At prestressing tendons	175 (79)
	Liner interface transients range	100–200 (38–93)
Extreme environmental	Liner	
	Effective at liner-concrete interface	300 (149)
	Between cooling tubes	400 (204)
	Bulk concrete	310 (154)
	Local hot spots	500 (260)
	Distribution asymmetry	100 (38)
	At prestressing tendons	300 (149)
	Liner interface transients range	100–200 (38–93)
Failure	Bulk concrete	
	Unpressurized condition	400 (204)
	Pressurized condition	600 (316)

[a]Higher temperatures may be permitted as long as effects on material behavior (e.g., relaxation) are accounted for in design.

Source: "Code for Concrete Reactor Vessels and Containments," Nuclear Power Plant Components, *ASME Boiler and Pressure Vessel Code*, Section III, Division 2, American Society of Mechanical Engineers, New York, New York, July 2003.

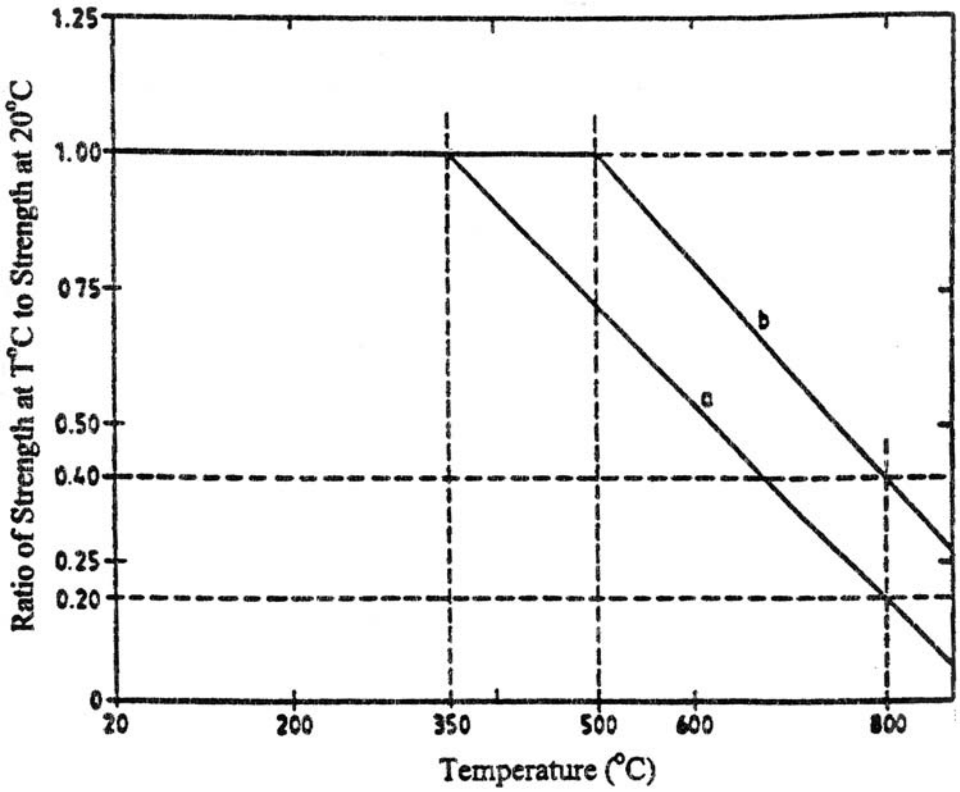

Figure 104 BS8110 design curves for strength variation with the temperature of (a) dense concrete and (b) lightweight concrete. *Source*: "Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors," BS 4975, British Standards Institution, United Kingdom, July 1973.

document are generally acceptable to the NRC staff and provide an adequate basis for complying with the general design criteria for structures other than reactor vessels and containments.[1] Conditions for applying the load requirements in ACI 349 are presented in U.S. Nuclear Regulatory Guide 1.142, and additional information on the design of seismic Category I structures that are required to remain functional if the Safe Shutdown Earthquake (SSE) occurs are contained in Ref. 149. Reference 150 presents a good comparison between ACI 318 and ACI 349.

Requirements for the design of concrete reactor vessels and containments are presented in ACI 359-77, ASME Section III—Division 2, "Code for Concrete Reactor Vessel and Containments."[2] Supplemental load combination criteria are presented in Sect. 3.8.1 of the *NRC Standard Review Plan*.[151]

National and international standards are available that provide guidance for computing concrete strength at elevated temperature: (1) Comité Europeen de Normalisation (Eurocode 2—Part 1-2, "Structural Fire Design," (2) Eurocode 4—Part 1-2, "General Rules for Structural Fire Design" and Comites Euro-International du Beton (CEB model code) Bulletin D'Information No. 208, "Fire Design of Concrete Structures," and National Building Code of Finland's RakMK. Figure 105 presents a comparison of unstressed NCS results with the CEB and Eurocode Design curves.[130]

[1] ACI 349-76 is endorsed by U.S. Nuclear Regulatory Guide 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)."
[2] ACI 359-77 is endorsed by U.S. Nuclear Regulatory Guide 1.136, "Material for Concrete Containments."

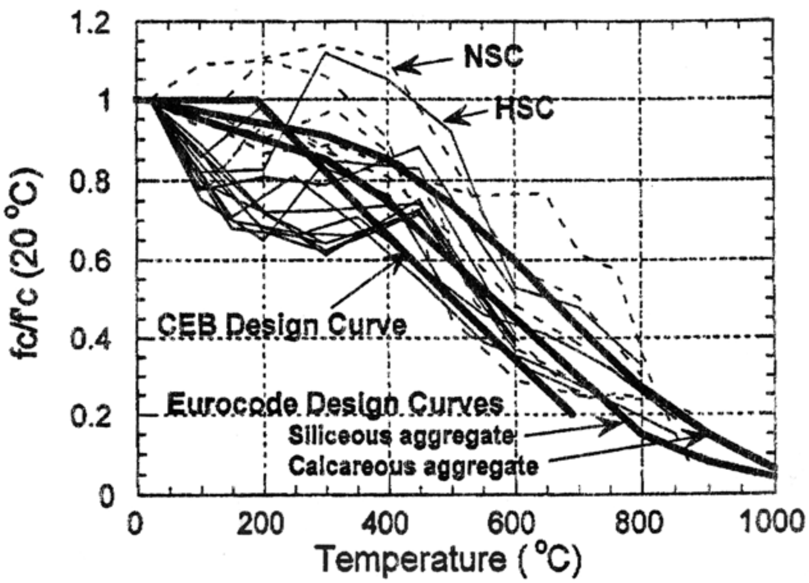

Figure 105 Comparison of unstressed NSC and HSC results with the CEB and Eurocode Design curves. *Source*: L. T. Phan, *Fire Performance of High-Strength Concrete: A Report of the State-of-the-Art*, NISTIR 5934, National Institute of Standards and Technology, Gaithersburg, Maryland, 1996.

5.3 Design of Reinforced Concrete Members Subjected to Elevated Temperature and Mechanical Loadings

The behavior of reinforced concrete sections at elevated temperature is of interest in the design of nuclear plant facilities for abnormal and severe environmental [bulk concrete temperature (≤93°C), local concrete hot spots (191°C)] as well as extreme environmental [bulk concrete temperature (≤149°C), local concrete hot spots (260°C)] load categories.[1] Also, it is important to predict the response of reinforced concrete components under hypothetical accident conditions for reactor designs in which concrete temperatures may exceed 600°C (e.g., a large sodium spill in the inerted and air-filled equipment cells of a liquid-metal fast breeder reactor).[152] The nonlinearities in material properties, the variation of properties with temperature, tensile cracking, and creep effects in the case of sustained capacity affect the buildup of thermal forces[2] and the deformation capacity or ductility of structural members. When severe temperatures are considered, a realistic behavior of concrete is essential in order to avoid undue and impractical design conservatisms. Examples of design procedures or considerations for elevated-temperature exposure are presented below.

American Concrete Institute (ACI) Committee 349, "Concrete Nuclear Structures," presents a design-oriented approach for considering thermal loads on reinforced concrete structures. Although the approach

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[1]Table CB-3430-1 of Ref. 1. The low design temperatures in the Code reflect uncertainty in the concrete behavior at high temperature. Higher temperatures than those listed may be permitted in the concrete if tests are provided to evaluate the reduction in strength, and this reduction is applied to the design allowables. Also, evidence shall be provided which verifies that the increased temperatures do not cause deterioration of concrete either with or without load. Generally, to comply with the Code, structures that operate under or may see temperatures above the limiting values of the Code are provided with insulation and/or a cooling system.

[2]A characteristic of thermal forces in concrete members is that the forces tend to be self-relieving (i.e., high tensile forces produce cracks to lower the loads caused by temperature changes). Due to the cracks and concrete creep, the thermal stresses are nonlinear and dependent on the effect of member stiffness, which in turn varies with the magnitude of loading.

is intended to conform to the general provisions of Appendix A of ACI 349-76, it is not restricted to nuclear power plant structures. For frame structures, the thermal load is assumed to be represented by temperatures that vary linearly through the thickness of the member. A rationale is described for determining the extent of member cracking that can be assumed for purposes of obtaining the cracked structure thermal forces and moments. Stiffness coefficients and carry-over factors (carry-over loading from one section to another of a structural member) are presented in graphical form as a function of the extent of member cracking along its length and the reinforcement ratio. Fixed-end thermal moments for cracked members are expressed in terms of these factors for (1) a temperature gradient across the depth of the member and (2) end displacements due to a uniform temperature change along the axes of adjacent members. For the axisymmetric shells, the structure is considered to be uncracked for all mechanical loads and for part of the thermal loads. The thermal load is assumed to be represented by a temperature that is distributed linearly through the wall of the structure [i.e., linear temperature distribution is separated into a gradient ΔT and into a uniform temperature change $T_m - T_B$ where T_m is the mean temperature and T_B the base (stress-free) temperature]. Normalized cracked section thermal moments are presented in graphical form as a function of the reinforcement ratio and the internal axial forces and moments acting on the section. The moments have been normalized with respect to cross-sectional dimensions and the temperature gradient across the section. Examples are presented for both design of a frame and of an axisymmetric shell under mechanical and thermal loadings.

Kar[153] presents a method for analysis of two types of concrete members having uniform capacities along their lengths and subjected to a differential temperature from face to face: (1) members subjected to bending moments only, and (2) members having combined bending and axial faces. Kar’s analysis is based on the hypothesis that in certain cases, the temperature load may tend to be self-limiting as a result of the formation of cracks in the member; and as the failure load condition or ultimate capacity is approached, the maximum temperature load that can occur is equal to that which is possible in the effective portion on the member-section at this load condition. (Thermal stresses are not considered as completely self-limiting, but are considered together with stresses due to other causes.) The stiffness of the member at ultimate load is used in the analysis. The effective moment of inertia is that for a cracked section with the depth of the cracked section being larger than the depth to the neutral axis. Also, the effective moment of inertia for the full member is larger than the moment of inertia of the cracked section. Recommendations made for the effective moment of inertia of concrete members reduce the necessity of iterative calculations such as required in ACI 349, “Code Requirements for Nuclear Safety Related Concrete Structures.” A step-by-step approach is presented for the analysis and design of a member subjected to bending moment only and a member subjected to combined bending and axial forces.

Gurfinke1[154] used both elastic and inelastic analyses to investigate thermal effects on reinforced concrete sections subjected to axial load and bending (i.e., the incremental bending induced in the wall of an unattached containment by a given thermal differential). Results of the investigation indicated that (1) for a given section, the moment induced by a given thermal differential depends an the external axial load and bending moment to which the section is subjected; (2) upper and lower limits exist for elastically determined thermal moments in any section under a given thermal differential depending on whether the moment of inertia of the uncracked or cracked section is used; (3) at service load conditions, elastic analysis renders thermal effects that are smaller than those obtained using inelastic analysis; and (4) inelastic analysis should be used to determine thermal effects in walls of containment structures since ultimate conditions can only be determined using inelastic analysis.

Mentes et al.[155] note that thermal analysis methods can be classified into three categories, depending on the criteria used to calculate effective stiffness: (1) reduced flexural stiffness, (2) average flexural stiffness, and (3) variable flexural stiffness. The reduced flexural method yields thermal moments that are on the unsafe side, and the combined thermal and nonthermal moments calculated may be safe or unsafe depending on the loading. Average flexural stiffness methods assume an average stiffness between the uncracked and cracked section values to account for the effects of stress level and tension stiffening and may require considerable iterative solutions. Variable flexural stiffness methods assume that members have reduced rigidities in the cracked sections and full uncracked rigidity along the rest of the member, but the methods present difficulties in including the effects of stress level and tension stiffening. These methods do, however, point out that to obtain a realistic evaluation of thermal moments in reinforced concrete members, it is necessary to consider proper stiffnesses and load interaction. The procedure for design of beams and frames is as follows: (1) assume all members uncracked, determine the mechanical load moments and amount of reinforcement required under mechanical load conditions; (2) increase the amount of reinforcement sufficiently to account for both mechanical and thermal load effects; (3) calculate the effective moment of inertia for all members; (4) calculate the equivalent fixed-end moments at all joints due to thermal loads, and add fixed-end moments due to mechanical loads; (5) using the effective moment of inertia, determine factors for distribution of fixed-end moments in the structure; (6) update the effective moment of inertia, using an equation provided, and combine moment values, then, if required, repeat this determination until satisfactory agreement between effective moment of inertia values is obtained; and (7) check that serviceability requirements are met. A flexurally restrained (but freely extendable) beam subjected to thermal load in absence of mechanical load and a portal frame subjected to combined mechanical and thermal loads are analyzed using this procedure and the results compared to those obtained using the aforementioned criteria that have been utilized to calculate effective stiffness.

Freskakis[156] examined the behavior of a reinforced concrete section (Fig. 106) by means of moment-curvature-axial force relationships that account for both the effect of temperature on the material properties and the mechanical effects induced by the tendency for thermal expansion. Of particular interest was the load carrying capacity, the thermal forces, and the deformation capacity. The effects on these properties due to variation in strength with temperature, the temperature level and its distribution across the section, the amount of reinforcing steel (0.75, 1.0, and 2.0%), and limiting values of compressive strain (0.003 and 0.004) were considered. Figures 17 and 23 presents upper and lower bound

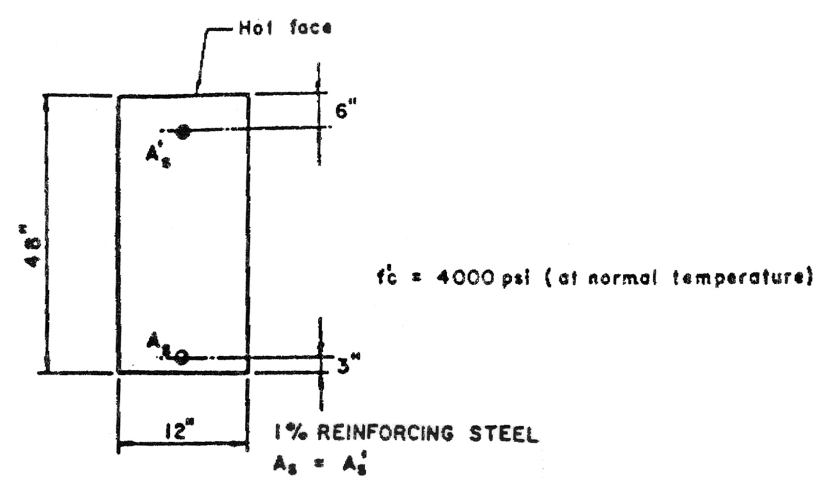

Figure 106 **Reinforced concrete section examined by Freskakis.** *Source*: G. N. Freskakis, “Behavior of Reinforced Concrete at Elevated Temperature,” Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power*, Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

relationships for the concrete modulus of elasticity and residual strength, respectively, as a function of temperature. Stress-strain relationships for the concrete based on the lower and upper bound relations in these figures are presented in Figs. 107 and 108, respectively. Stress-strain relationships at various temperatures used for the Grade 60 reinforcing bars are presented in Fig 109. Thermal gradients considered in the study that had temperatures either of 149, 260, or 427°C at the face of the concrete section are presented in Fig. 110. The three thermal gradients in Fig. 110 represent short (Type I), intermediate (Type II), and long (Type III) duration thermal exposures. Results from this study for selected parameters are presented in Figs. 111 to 119. From these results, it was concluded that (1) the effect of elevated temperature is to decrease the section capacity when axial forces are present, and the net carrying capacity decreases significantly with the buildup of thermal forces; (2) in terms of bending and axial force capacity, reinforced concrete sections can be designed to sustain severe temperature gradients with levels of temperature much higher than allowed by present codes; (3) comparison of net moment capacity results based on the upper and lower bound strength and modulus of elasticity relationships indicates that either upper or lower bound relations may govern the design depending on the level and distribution of temperature across the section and the axial compression (e.g., upper bound should be considered for temperatures above 260°C and where heat of exposure is of short duration); (4) addition of reinforcing steel improves the net capacity of the sections, but the steel must be properly located in the section or a significant reduction in capacity can result; (5) where strain limits are imposed, increases in the temperature level result in significant reductions in rotational ductility; (6) for temperatures up to about 205°C, the use of a limiting concrete strain of 0.003 does not result in a significant loss in member strength, but at higher temperatures a significant reduction of strength occurs in the presence of compressive forces; and (7) the concrete tensile strength is important in the calculation of thermal forces at low elevated temperatures, but is insignificant where severe temperatures are present.

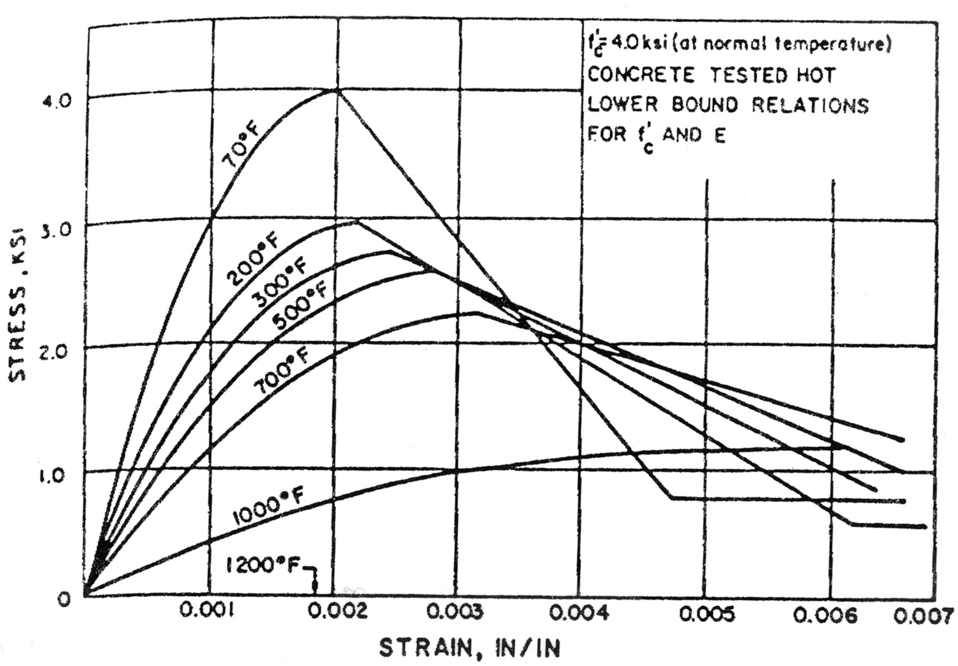

Figure 107 Stress-strain relationships for concrete used by Freskakis (lower bound). *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power*, Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

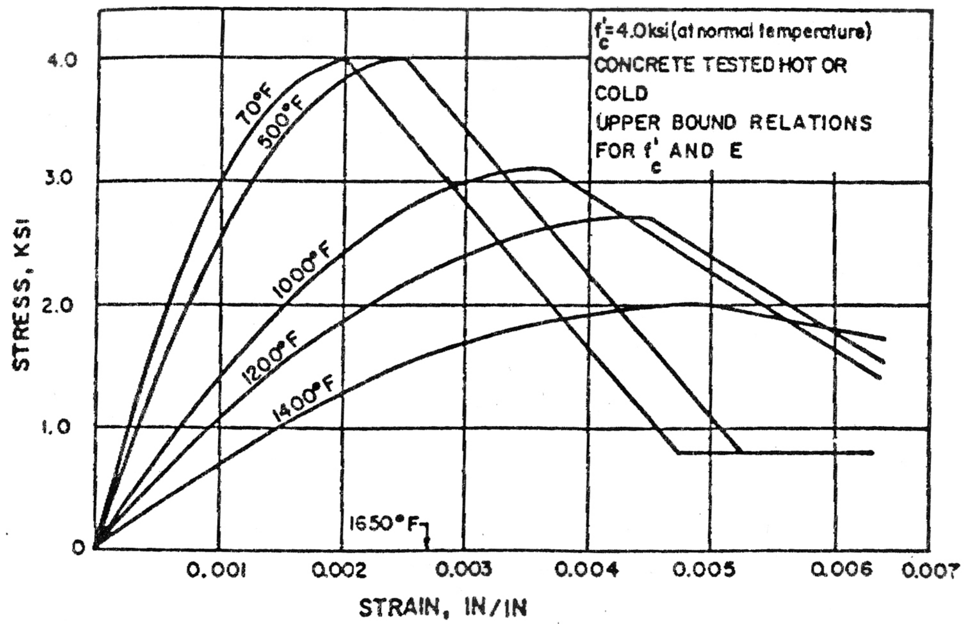

Figure 108 **Stress-strain relationships used by Freskakis (upper bound).** *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power*, Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

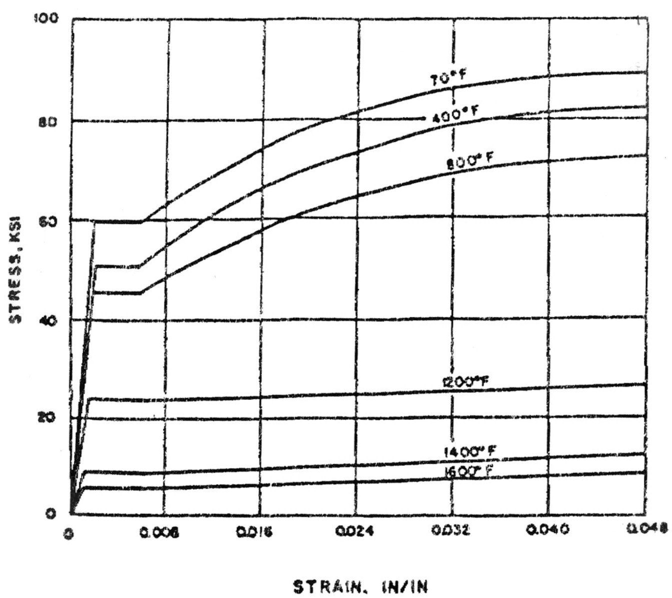

Figure 109 **Stress-strain relationships for rebars used by Freskakis.** *Source*: G. N. Freskakis. "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power*, Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

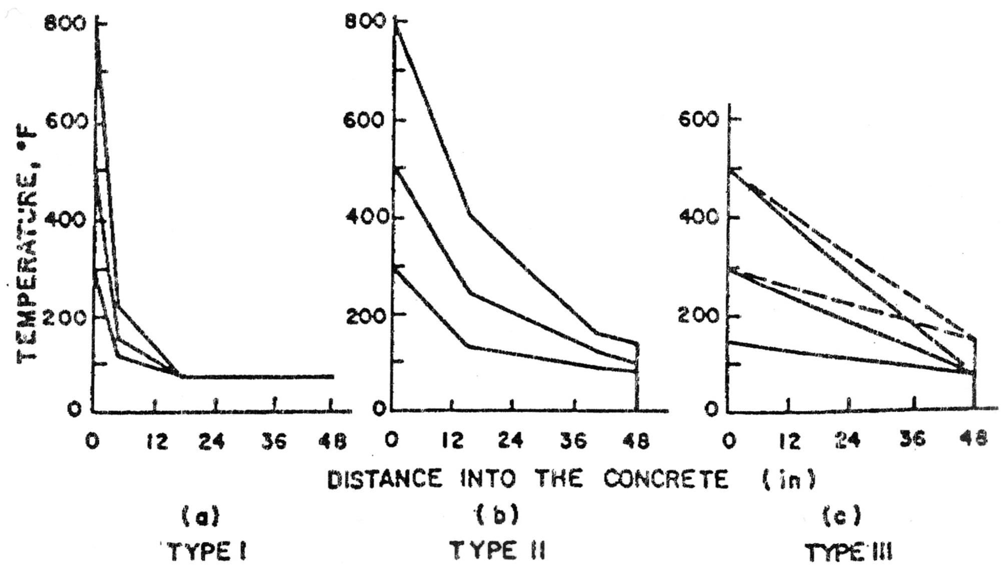

Figure 110 Thermal gradients investigated by Freskakis. *Source*: G. M. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power,* Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

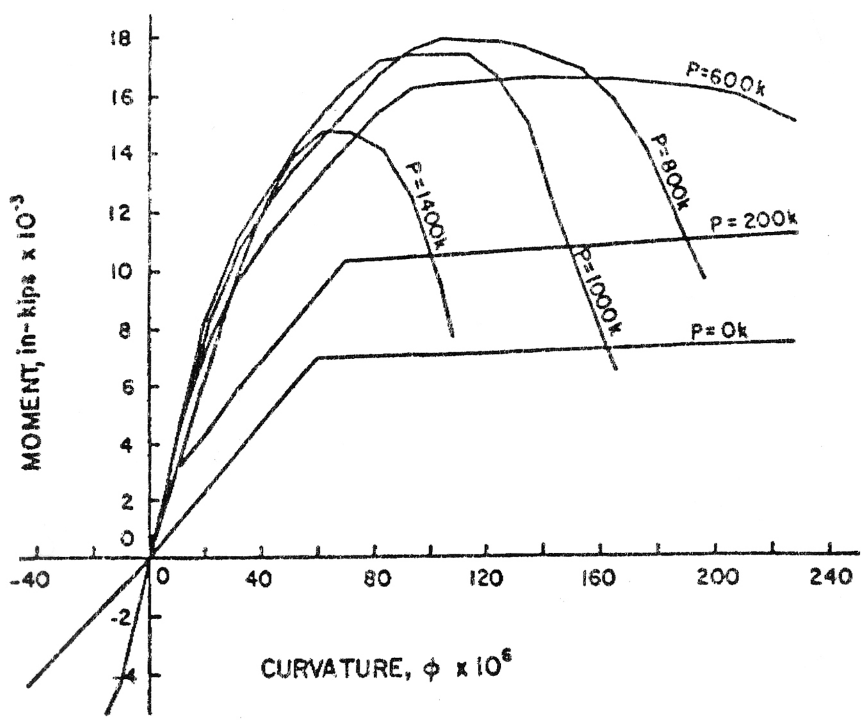

Figure 111 M-ϕ-P relationships: normal temperature. *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power,* Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

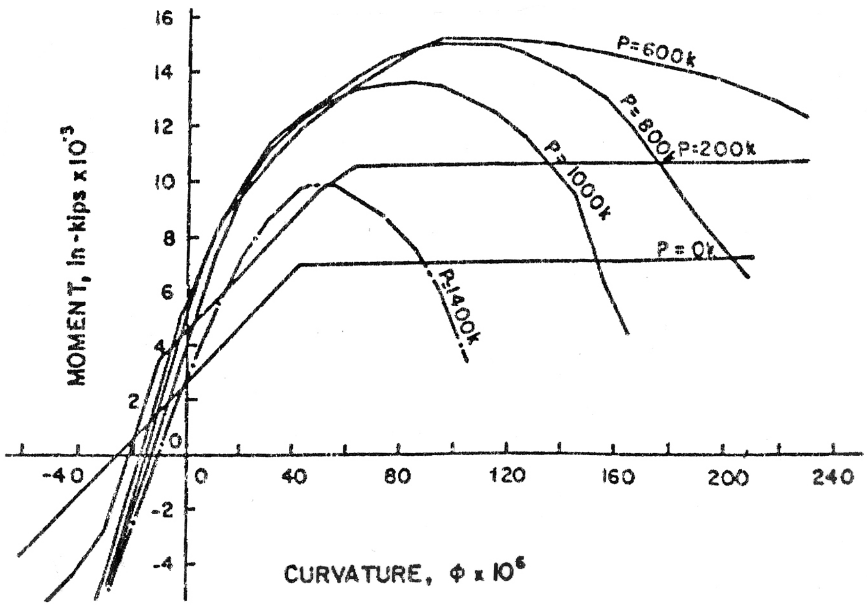

Figure 112 M-ϕ-P relationships: T_1 = 300°F (based on lower bound relations). *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power*, Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September.

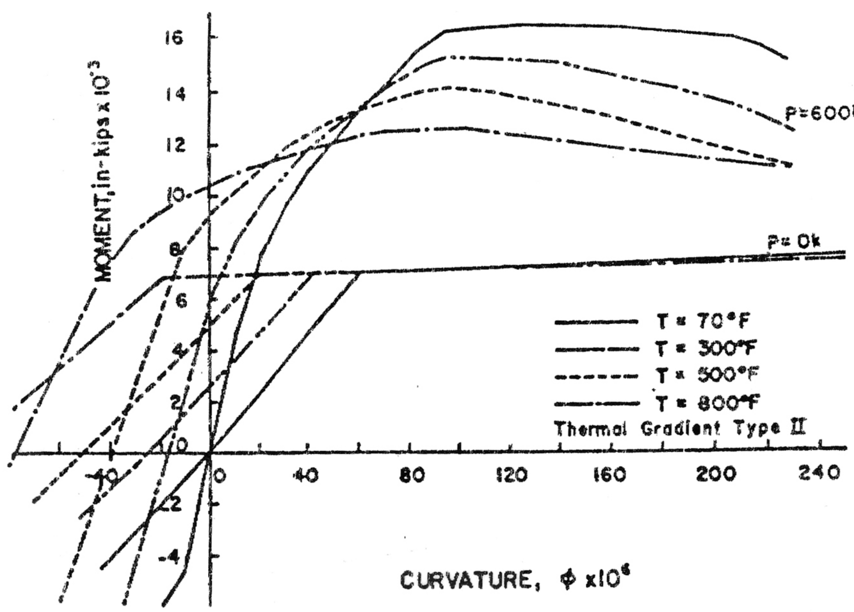

Figure 113 Effect of temperature level on behavior (based on lower bound relations). *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng, and Nuclear Power*, Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

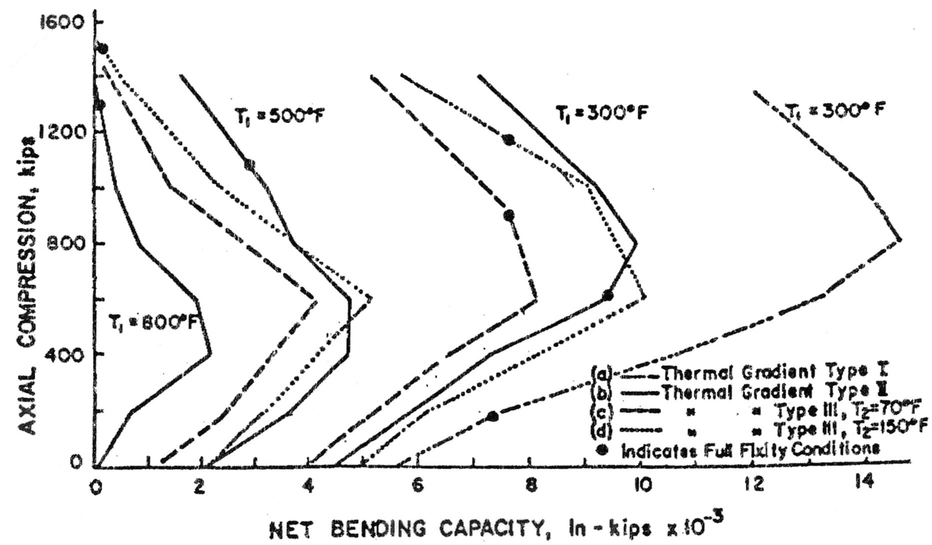

Figure 114 **Effect of temperature distribution on net bending capacity.** *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power,* Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September.

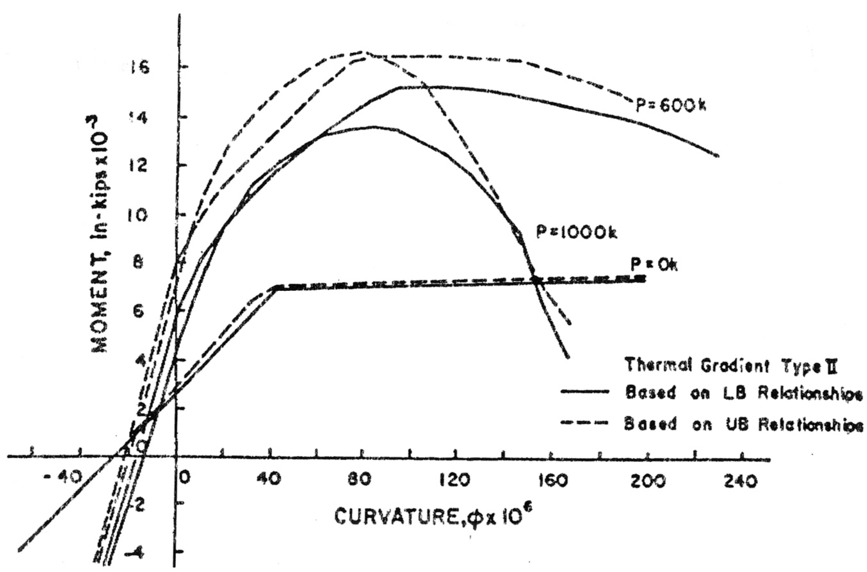

Figure 115 **N-ϕ-P diagrams based on upper and lower bound strength relations.** *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power,* Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

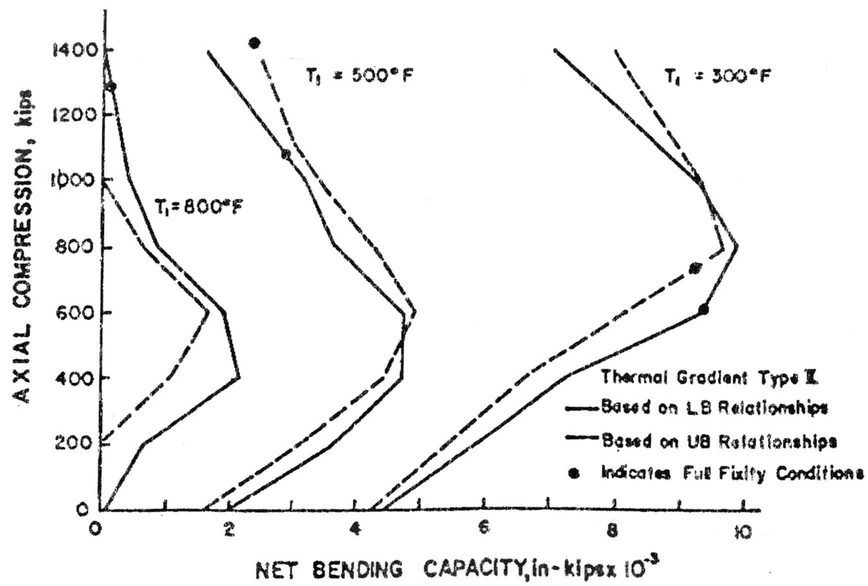

Figure 116 Net bending capacity based on upper and lower bound strength relations. *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power,* Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

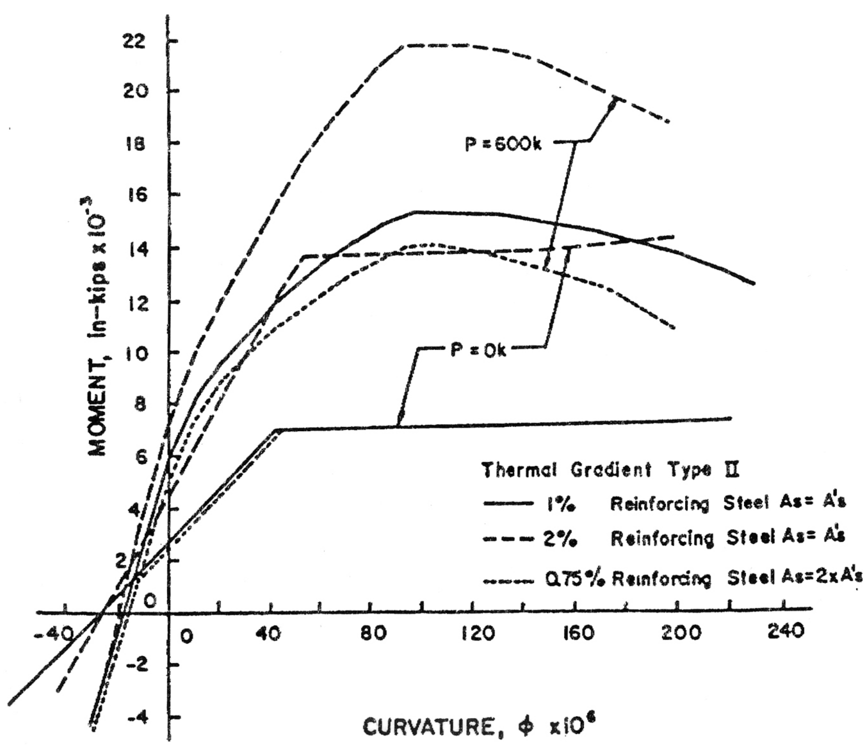

Figure 117 Effect of reinforcing steel on behavior (based on lower bound relations). *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power,* Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

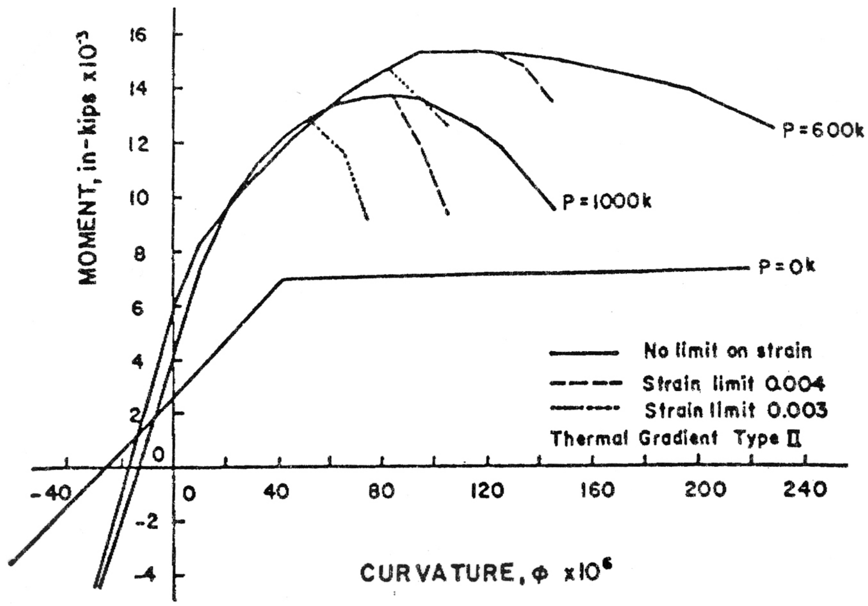

Figure 118 **Effect of strain limits on behavior: T_1 = 300°F (based on lower bound relations).** *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng, and Nuclear Power,* Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

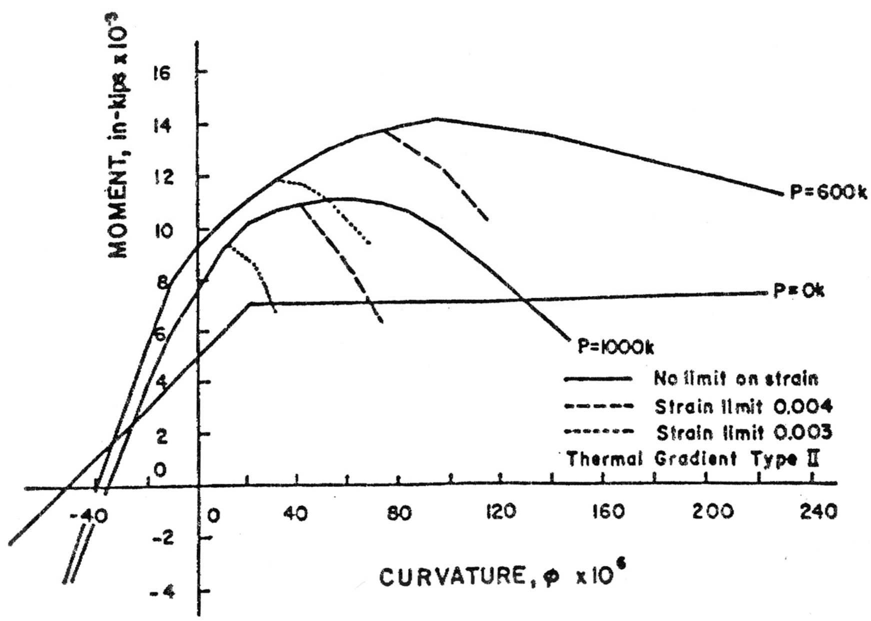

Figure 119 **Effect of strain limits on behavior: T_1 = 500°F (based on lower bound relation).** *Source*: G. N. Freskakis, "Behavior of Reinforced Concrete at Elevated Temperature," Paper 3-4, *Second ASCE Conf. on Civ. Eng. and Nuclear Power*, Vol. 1, Paper 3-5, pp. 3-5-1 to 3-5-21, Knoxville, Tennessee, September 15–17, 1980.

Information has been developed relative to prediction of the response of reinforced concrete structures to variations in temperature, humidity, and load, as well as modeling of the structures, their loading history, environmental factors, and material behavior.[157] Cracking, excessive deformation, spalling, and even partial collapse of reinforced concrete structures frequently result from variations in environmental and loading conditions not considered in design. Such distress should be minimized through incorporation of an analysis of these effects of complex environmental and loading histories into design practice. A general scheme for such a design process is illustrated by flow chart in Fig. 120, where the sequence of evaluation and analysis is indicated. Basically, it includes (1) modeling the structure, (2) modeling environmental and loading history, (3) deciding whether an environmental response analysis is required, (4) executing an environmental response analysis (when necessary), otherwise proceed directly to structural response analysis, (5) analyzing the structural response to critical loading combinations, including environmental effects when necessary, and (6) verifying that structural response compiles with performance requirements. The simplified models for selected environments and for material behavior described in the reference permit practical engineering solutions. The details of the processes using these simplified models are described below. Basic modeling aspects of the approach follow:

1. Modeling the Structure: Simplified structural analysis and determination of thermal and shrinkage distribution throughout a concrete structure, large structures are sub-structured, substructures are divided into members, members are divided into segments, and segments are divided into elements representing concrete and steel components.

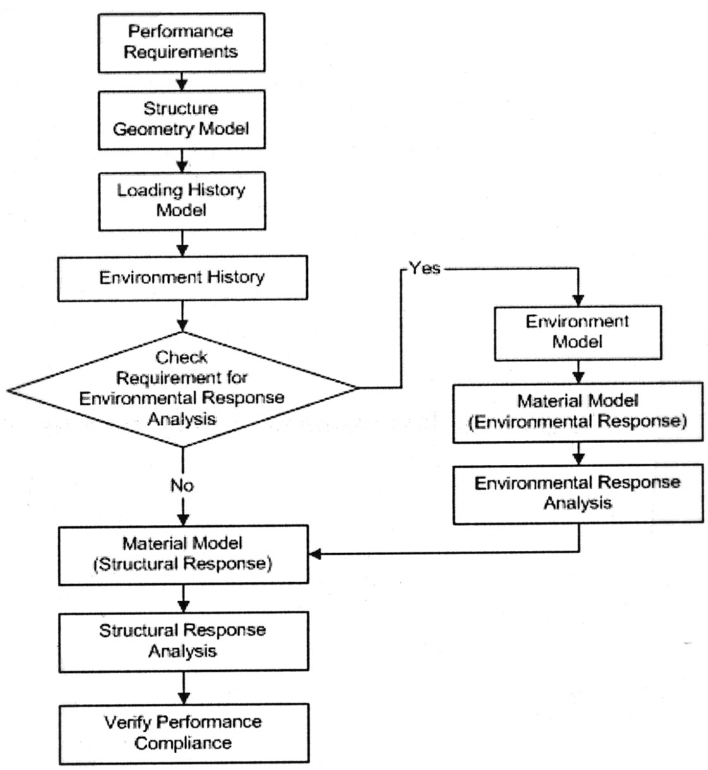

Figure 120 **Design process flow diagram.** *Source*: B. Bresler and R. H. Iding, "Effects of Normal and Extreme Environment on Reinforced Concrete Structures," Paper SP 55-11 in *Douglas McHenry Symposium on Concrete and Concrete Structures*, SP 55, American Concrete Institute, Farmington Hills, Michigan, April 1970.

2. Loading History: Important variation in loading during the life of a structure includes the following two stages: (a) Construction stage: The influence of early load history on structural response is particularly important when large loading increments develop while concrete is not fully mature. (b) Service life stage: Load increments are due to furnishing and occupancy loads in building, traffic load on bridges, etc.
3. Modeling of the Environment: Physical environmental effects can be divided into two categories: (a) normal service conditions (atmospheric environment-structural response to changes in environment depends on variations in ambient air temperature, relative humidity, wind, and sky cover); and (b) extreme conditions (fire environment modeling with important parameters in establishing models for fire environment being fire load, ventilation, compartment geometry, and surface characteristics).
4. Modeling Material Behavior: In the structure response analysis, two types of strains are calculated: (1) free strains associated with changes in the environment in the absence of any restraint or stress, which includes thermal and shrinkage strains, and (2) stress-related strains (instantaneous and time-dependent) including degradation of material such as cracking or crushing of concrete including instantaneous strains, steel and concrete response in tension and compression, effect of temperature on instantaneous strain, creep strain, and effect of temperature on creep strain.
5. Modeling Free Volume Changes: The first step in analyzing environmental response is to determine unrestrained free volume change due to moisture movement and temperature change. Issues to be considered include thermal volume change (e.g., boundary conditions and internal heat generation), and free shrinkage volume changes (i.e., boundary condition, shrinkage diffusivity, and surface layer).

Once all modeling assumptions have been made, it is possible to proceed with the structural analysis. A modification of direct stiffness method of matrix structural theory is used. The approach described above was applied to four case studies.[157] In the first two studies, analytical results were correlated with experimental results of studies on shrinkage in specimens of varying size. In the third study, the method was applied to the special problem in concrete construction related to response under permafrost condition. In the fourth study, the response of reinforced concrete frame to a hypothetical thermal excursion (compartment fire) was analyzed. In the first two case studies, the agreement between calculated and observed shrinkage was very good. In the third case study, the comparison of experimental and analytical results was less satisfactory; however, discrepancies could be attributed to inadequate understanding of concrete behavior at early ages. The behavior of the frame exposed to the elevated temperature (fire) definitely points to the need for better design of certain columns and connections in reinforced concrete frames exposed to certain types of fires. The overview of the methodology and the results point out a number of areas where more information is needed to predict structural response more reliably [i.e., experimental studies of plain concrete and reinforcing steel properties under combined variable environment (temperature, humidity) with variable stress history (short-term cycling as well as long-term sustained and variable loads)]. Additional development of the models is dependent on the development of structural features in which data for model validation is developed from structures under representative loadings and environments.

5.4 Analysis Methods

There are a number of sophisticated methods for analyzing the short-term elastic and long-term time-independent loading of complex structures.[158] These methods are sufficiently developed for three-dimensional, elastic, short-term deformations, but for determination of long-term time-dependent behavior and prediction of ultimate strength and behavior in the load range approaching failure conditions, methods such as the finite-element or finite-differences, may require refinements. Difficulties are encountered due to limited knowledge on (1) nonhomogeneity of structural concrete; (2) nonlinear stress—strain relationship for concrete and lack of suitable constitutive relationships under combined stress conditions; (3) continuously changing topology due to crack propagation under increasing load; (4) lack of suitable failure criteria for concrete under combined stress conditions; (5) creep and shrinkage of concrete; and (6) effect of anchorage or pullout, dowell and interlock forces in cracked and uncracked sections. The performance of cement-based materials under elevated temperatures is very complicated and difficult to characterize. Current constitutive models are generally calibrated from isothermal conditions in which the transitional effects in variable temperature, humidity, chemical and mechanical loading are neglected.[159] Analyses are further complicated if the structures are subjected to loads far beyond those for which they were designed (i.e., dynamic and/or elevated-temperature loading).[*]

As noted in the previous section, procedures for design of reinforced concrete members for elevated-temperature exposure often involve simplifying assumptions, the validity of which becomes questionable for extreme environmental conditions. Accurate analytical modeling is possible only if the phenomenon modeled is well understood. For example, uncertainties may exist with respect to the strength and deformation of concrete due to the stochastic nature of its material properties.[160,161] Statistical approaches, however, have been developed for inclusion of these effects into structural analyses (i.e., Latin hypercube sampling technique[162–164] for prediction of effects of creep and shrinkage in concrete and Bayesian statistical approach[165] to predict the long-term creep and shrinkage of concrete. Also, computer codes such as DYNAPCON[166,167] and TEMP-STRESS[168–171] have been developed at Argonne National Laboratory to predict the response of concrete structures subjected to loads beyond those for which they were designed. The TEMP-STRESS code involves not only thermal stress considerations but also the prediction of temperature and moisture distribution within the structure. Use of TEMP-STRESS (as well as other elevated-temperature codes) is restricted, however, because of limited representative structural features data that can be used to verify and calibrate the code, particularly with respect to the various types of failure at high temperature. Also, many of these codes have been developed only for two- dimensional modeling problems of concrete. Three-dimensional computer codes capable of handling reinforced concrete structural problems at elevated temperature require further development. A listing of needs that are required to develop and/or refine computer codes for simulating concrete behavior at elevated temperature is available.[172]

[*]Response of structures to overload conditions requires a nonlinear constitutive model to describe material behavior, and the mode of failure exhibited by concrete introduces localized violation of the compatibility requirements of continuum mechanics.