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Task 2: Concrete Properties in Nuclear Environment—A Review of Concrete Material Systems for Application to Prestressed Concrete Pressure Vessels

D. J. Naus

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TASK 2: CONCRETE PROPERTIES IN NUCLEAR ENVIRONMENT — A REVIEW OF CONCRETE MATERIAL SYSTEMS FOR APPLICATION TO PRESTRESSED CONCRETE PRESSURE VESSELS

D. J. Naus

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Prepared by the
OAK RIDGE NATIONAL LABORATORY
Oak Ridge, Tennessee 37830
operated by
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D. J. Naus

ABSTRACT

Prestressed concrete pressure vessels (PCPVs) are designed to serve as primary pressure containment structures. The safety of these structures depends on a correct assessment of the loadings and proper design of the vessels to accept these loadings. Proper vessel design requires a knowledge of the component (material) properties. Because concrete is one of the primary constituents of PCPVs, knowledge of its behavior is required to produce optimum PCPV designs.

Concrete material systems are reviewed with respect to constituents, mix design, placing, curing, and strength evaluations, and typical concrete property data are presented. Effects of extreme loadings (elevated temperature, multiaxial, irradiation) on concrete behavior are described. Finally, specialty concrete material systems (high strength, fibrous, polymer, lightweight, refractory) are reviewed.

1. INTRODUCTION

Prestressed concrete pressure vessels (PCPVs) are designed to serve as primary pressure containment structures. The safety of these vessels depends on (1) a correct assessment of the loadings (including overloads) likely to be applied to the vessel as well as the probability of these occurring and (2) the proper design of the vessel to accept these loadings. Proper vessel design thus requires a knowledge of the component (material) properties.

Although the concept of a PCPV is not limited to nuclear applications, most of the applicable codes and standards are nuclear oriented.[1–3] Thus, these codes will be referenced where applicable in the following sections related to materials for PCPV applications.

2. CONCRETE AND SPECIALTY CONCRETES

2.1 Concrete

Concrete is a general term for a class of ceramic materials that vary widely in their properties and applications. The American Concrete Institute (ACI) defines concrete as "a composite material that consists essentially of a binding medium within which are embedded particles or fragments of aggregate; in portland cement concrete the binder is a mixture of portland cement and water."[4] By varying the constituents and their relative proportions in the mixture, concretes of widely differing properties can be obtained.

2.1.1 Materials

Cement. The American Society for Testing and Materials (ASTM) provides for five types of portland cement: (1) Type I — general, (2) Type II — moderate sulphate resistance and heat of hydration, (3) Type III — high early strength, (4) Type IV — low heat of hydration, and (5) Type V — high sulphate resistance.[5] For practical purposes, portland cements may be considered to be composed of four principal compounds: (1) tricalcium silicate ($3CaO \cdot SiO_2$), (2) dicalcium silicate ($2CaO \cdot SiO_2$), (3) tricalcium aluminate ($3CaO \cdot Al_2O_3$), and (4) tetracalcium aluminoferrite ($4CaO \cdot Al_2O_3 \cdot Fe_2O_3$).[6] Generally, the relative percentages of these compounds determine the particular type of portland cement. Type II cement conforming to American National Standards Institute (ANSI)/ASTM C 150-78a (Ref. 5) is generally used for fabrication of PCPVs.

Water. Almost any natural water that is drinkable and has no pronounced taste or odor is satisfactory as mixing water for making concrete. Excessive impurities in mixing water may affect setting time, strength, and volume constancy of the concrete as well as cause corrosion of reinforcement. Water containing less than 2000 ppm of total undissolved solids can generally be used satisfactorily for making concrete. Water of unknown performance is considered acceptable if (1) the 7- and 28-d compressive strengths of mortar cubes [fabricated according to ANSI/ASTM C 109-77 (Ref. 7)] are at least 90% of the values obtained from companion specimens fabricated using water of known quality and (2) the setting time of the mortar [evaluated in accordance with ANSI/ASTM C 191-77 (Ref. 8)] is not adversely affected. Additionally, the mixing water, including that contained as free water in the aggregates, should contain not more than 250 ppm of chorides as Cl^- [determined by ANSI/ASTM D 512-67 (Ref. 9)].

Aggregates. Aggregates generally occupy 60 to 80% of the volume of concrete; thus, their characteristics influence the properties of concrete, its mix proportions, and economy. Aggregates must conform to certain requirements and should consist of clean, hard, strong, and durable particles free of chemicals, coatings of clay, or other fine materials that may affect hydration and bonding of the cement paste. Weak, friable, or laminated aggregate particles are undesirable. The most commonly used aggregates, such as sand, gravel, crushed stone, and air-cooled blast furnace slag, produce normal-weight concrete (2160 to 2560 kg/m^3). Expanded shale, clay, slate, and slag are used as aggregates to produce structural lightweight concretes having unit weights ranging from 1360 to 1840 kg/m^3. Other lightweight materials such as pumice, scoria, perlite, vermiculite, and diatomite are used to produce insulating concretes weighing from 240 to 1440 kg/m^3. Heavyweight materials such as barites, limonite, magnetite, ilmenite, iron, and steel particles are used for producing heavyweight concrete (unit weights up to 6410 kg/m^3). Table 1 (obtained from Ref. 6) provides a summary of aggregate characteristics, their significance, and test method.

Normal-weight aggregates should meet the specifications of ANSI/ASTM C 33-78 (Ref. 10), which limits the permissible amounts of deleterious substances and covers the requirements of gradation, abrasion resistance, and soundness.

Structural lightweight aggregates should meet the requirements of ANSI/ASTM C 330-77 (Ref. 11).

Heavy aggregates required for concretes weighing more than 2560 kg/m^3 should conform to ANSI/ASTM C 637-73 (Ref. 12).

Admixtures. When admixtures are to be used, the type, quantity, and additional limitations should be specified. Recommendations are that admixtures containing more than 1% chloride ions should not be used.[13] Chemical admixtures should conform to the requirements of ANSI/ASTM C

Table 1. Characteristics of aggregates

Characteristic	Significance or importance	Test or practice[a]	Specification requirement
Chemical stability	Strength and durability of all types of structures	C 227 (mortar bar)	Maximum expansion of mortar bar[b]
		C 289 (chemical), C 586 (aggregate prism), C 295 (petrographic)	Aggregates must not be reactive with cement alkalies[b]
		D 1411	Water soluble chlorides
Particle shape and surface texture	Workability of fresh concrete		Maximum percent flat and elongated pieces
Grading	Workability of fresh concrete; economy	C 136	Maximum and minimum percent passing standard sieves
Bulk unit weight	Mix design calculations; classification	C 29	Maximum or minimum unit weight (special concretes)
Specific gravity	Mix design calculations	C 127 (coarse aggregate), C 128 (fine aggregate)	
Absorption and surface moisture	Control of concrete quality	C 70, C 127, C 128	

[a]ASTM designation.

[b]Aggregates not conforming to specification requirements may be used if service records or performance tests indicate they produce concrete having desired properties.

C 494-79 (Ref. 14); fly ash and pozzolanic materials should conform to ANSI/ASTM C 618-78 (Ref. 15); and air-entraining admixtures should conform to ANSI/ASTM C 260-77 (Ref. 16).

2.1.2 Concrete mix design

Concrete properties. Table 2 (obtained from Ref. 17) lists desirable properties for PCPV concretes and justifications for each. Tan[18] lists essentially the same properties, with additions of high specific heat, low heat of hydration, and satisfactory hydrogen content that is materially unaffected by the operating temperature (Tan was concerned with the effects of irradiation and the effects of temperature above the normal ambient on these properties). Table 3 lists concrete properties and appropriate ANSI/ASTM specifications which, according to subsection CB-2231.2, Ref. 1, should be defined in a construction specification; Table 3 also indicates the age and temperature at which the properties will be determined and any environmental or design conditions that will apply. Additionally, if a particular property is not of interest, the construction specification will indicate this.

Concrete mix proportions. Actual concrete mix proportions are selected on the basis of meeting property requirements listed in a construction specification. Proportions of concrete materials are established on the basis of laboratory trial batches to provide (1) conformance with concrete strength requirements, (2) proper consistency to permit the concrete to be worked readily into the forms and around reinforcement under the specific conditions of placement without excessive segregation or bleeding, and (3) resistance to aggressive environments. Because of considerable variation in materials and mix proportions for each application and great variability in properties of concretes used in laboratory investigations and field construction, precise data are required for a specific vessel design, and tests must be performed on the concrete mixture designed for each vessel.[17] Proportions can be established on the basis of ACI 211.1, "Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete."[19] Guidance for evaluation of strength test results is obtained from ACI 214, "Recommended Practice for Evaluation of Compression Test Results of Field Concrete."[19]

Table 2. Preferred concrete properties

Preferred property	Reasons
High compressive strength at normal and elevated temperature	To reduce vessel thickness and increase allowable stresses (other strength properties are to some extent related to compressive strength)
Good mix workability	To ensure good compaction in placing, particularly in areas where high concentrations of reinforcement and prestress ducting exist
High density	To provide good neutron and gamma ray absorption properties
Low elastic and creep deformation under load	To reduce movements and the redistribution of stresses under varying load and temperature cycles; to reduce prestress losses
Low drying shrinkage	To reduce movements and temperature stresses
Low thermal expansion	To reduce movements and temperature stresses
Resistance to thermal shock	To prevent damage to structure under rapid heat application (i.e., adjacent to steam openings)
High thermal conductivity	To minimize the cooling system required to keep the vessel concrete at a permissible temperature
Immunity to radiation damage	To minimize the possible deterioration of concrete in high-irradiated areas

Source: R. D. Browne, "Properties of Concrete in Reactor Vessels," Group C, Paper 13, Conference on Prestressed Concrete Pressure Vessels, Westminster, S.W.I., March 1967.

Table 3. Concrete properties defined in construction specification (Ref. 1, Subsection CB-2231.2)

Property	Specification
Slump	ASTM C 143
Compressive strength	ASTM C 39
Flexural strength	ASTM C 78
Splitting tensile strength	ASTM C 496
Static modulus of elasticity	ASTM C 469
Poisson's ratio	ASTM C 469
Coefficient of thermal conductivity	CRD-C 44
Coefficient of thermal expansion	CRD-C 39
Creep of concrete in compression	ASTM C 512
Shrinkage coefficient (length change of cement-mortar and concrete)	ASTM C 157
Density (specific gravity)	ASTM C 642
Aggregates for radiation shielding concrete	ASTM C 637

2.1.3 Storing, batching, mixing, transporting, placing, consolidating, and curing

Storing, batching, mixing, transporting, placing, consolidating, and curing are done in conformance with procedures described in a construction specification that generally meets the requirements specified in Ref. 19.

Aggregates should be handled, transported, and stockpiled in a manner that ensures a minimum of segregation and contamination by deleterious substances. Cement, admixtures, and other materials adversely affected by moisture before mixing should be stored, handled, and transported in a manner that will prevent the introduction of moisture.

In conformance with ACI 304-73 (Ref. 19), batching of materials should be with aggregates, cements, and powdered admixtures measured by weight and liquids measured by volume or weight. Corrections should be made for free moisture content of aggregates or admixtures. Measurement accuracy should be within the ranges established in ANSI/ASTM C 94-78a (Ref. 20).

Mixing should be done in accordance with the requirements of Ref. 20. The range of mixing capacities and corresponding mixing times for all mixers should be determined by performance of mixer uniformity tests as specified in Ref. 20.

Concrete should be conveyed from the mixer to the place of final placement using methods that will prevent the separation or loss of materials. Techniques for handling and transporting concrete include (1) using chutes, buggies, buckets, small railway cars, and trucks; (2) pumping through pipelines; and (3) pneumatically forcing the concrete material through hoses. Proper techniques for each of these methods can be obtained from Refs. 6 and 19. Concrete should be deposited as near as practicable to its final position to avoid segregation caused by rehandling or flowing. The delivery rate should be such that unconsolidated material is not covered by newly placed concrete and that the newly deposited concrete is not placed on hardened concrete, unless the surface of the concrete has been properly prepared as a construction joint. Once started, concreting should be carried on as a continuous operation until the placing of a particular structural feature is completed. If the concrete is to be placed under conditions where either relatively low (<4°C) or high (>32°C) ambient temperatures exist, special precautions [ACI 306-66 (Ref. 21) and ACI 305-72 (Ref. 22)] should be followed to protect it from the disruptive effects of freezing and excessive shrinkage, respectively.

Concrete should be thoroughly consolidated around reinforcement and embedded fixtures and into the corners of forms, using ACI 309-72 (Ref. 23) as a guide to develop suitable consolidation techniques. When regions are encountered where consolidation may be difficult, recommendations are that mock-ups be prepared and filled with the specified concrete to demonstrate that it can be properly placed and consolidated under field conditions.

Curing concrete provides a satisfactory moisture content (ensures that sufficient water is available for cement hydration) and favorable temperature (low temperature retards hydration, and high temperature can cause cracking on cooling) during hydration of the cementitious materials so that the desired concrete properties are developed. Two methods provide the required moisture: (1) maintaining a moist environment by application of water and (2) preventing loss of mixing water by sealing materials. The second method is generally applicable to PCPVs. Recommendations are that the concrete in PCPVs be maintained above 4°C in a moist condition for at least 7 d after placement and even longer ($\geq$28 d) for high-strength concrete (>41 MPa).[24] Basic principles and commonly accepted methods, procedures, and materials for successful curing are described in Ref. 24.

2.1.4 Construction testing

Quality control requires that concrete and concrete materials (including aggregates, cement, water, and admixtures) be tested at specified intervals. Table 4 (Ref. 25) presents materials, requirements, test methods, and suggested test frequencies for concrete materials and concrete.

2.1.5 Concrete properties

Expanding interest in PCPV operation under high pressures, high temperatures, and hostile environments has necessitated detailed investigations of concrete behavior under specific operating conditions. Many

Table 4. Testing frequencies for concrete materials and concrete

Material	Requirements	Test method	Frequency
Cement	Standard physical and chemical properties	ASTM C 150	Each 1200 tons
Fly ash and pozzolans	Chemical and physical properties in accordance with ASTM C 618	ASTM C 311	Each 200 tons
Aggregate	Gradation	ASTM C 136	Once daily during production[a]
	Moisure content	ASTM C 566	Twice daily during production
	Material finer than No. 200 sieve	ASTM C 117	Daily during production
	Organic impurities	ASTM C 40	Daily during production
	Flat and elongated particles	CRD-C 119	Monthly during production
	Friable particles	ASTM C 142	Monthly during production
	Lightweight particles	ASTM C 123	Monthly during production
	Soft fragments	ASTM 235	Monthly during production
	Specific gravity and absorption	ASTM C 127 or ASTM C 128	Monthly during production
	Los Angeles abrasion	ASTM C 131 or ASTM C 535	Every 6 months
	Potential reactivity	ASTM C 289	Every 6 months
	Soundness	ASTM C 88	Every 6 months
Water and ice	Compliance with CB-2223		
	Effect on compressive strength	ASTM C 109	Monthly
	Setting time	ASTM C 191	Monthly
	Soundness	ASTM C 151	Monthly
	Total solids	ASTM D 1888	Monthly
	Chlorides	ASTM D 512	Monthly
Admixtures	Chemical composition	Infrared spectrophotometry pH and solids content in accordance with ASTM C 494	Composite of each shipment
Concrete	Mixer uniformity	ASTM C 94	Initially and every 6 months
	Sampling method	ASTM C 172	
	Compression cylinders	ASTM C 31	
	Compression strength	ASTM C 39	One set of 2 cylinders from each 100 yd^3 or a minimum of 1 set/d for each class of concrete given in CB-5234.2
	Slump	ASTM C 143	First batch placed each day and every 50 yd^3 placed
	Air content	ASTM C 173 or ASTM C 231	First batch placed each day and every 50 yd^3 placed
	Temperature		First batch placed each day and every 50 yd^3 placed
	Unit weight/yield	ASTM C 138	Daily during production

[a]Twice daily during production if more than 200 yd^3 (153 m^3) is placed.

research programs have been conducted and valuable information has been obtained about PCPV structural behavior in general and concrete properties in particular. However, correlation of this data is sometimes difficult because a variety of concrete mix designs and materials have been used. Additionally, accurate structural representation in mechanical testing is a problem, because mass concrete curing conditions and moisture migration encountered in a PCPV are difficult to simulate using laboratory-size specimens. Difficulties are also encountered in obtaining multiaxial, sealed-specimen elevated temperature, and long-term specimen behavior.

To establish the behavior of a PCPV with acceptable accuracy, it is necessary to have reliable knowledge of the concrete behavior.[26] This accuracy is difficult to achieve because (1) concrete is sensitive to composition and properties of its basic components; (2) its strength is a function of age, curing conditions, and environment (temperature and moisture state); and (3) it exhibits time-dependent deformations. The following sections will present an overview of these effects on concrete behavior; more detailed information may be obtained by consulting Refs. 27 through 46, which are pertinent to the particular application.

Strength. A PCPV is generally in its least stressed condition during operation and its most highly stressed condition during depressurization, when the axial and circumferential prestressing loads are not counterbalanced. The strength of concrete in a PCPV has little effect on the ultimate vessel strength and its operational performance, provided it is of sufficient magnitude to sustain the working loads throughout the design life of the vessel.[47] When limit state design is used, a detailed knowledge of concrete's strength variation with temperature and increasing stress level in the PCPV have to be appreciated more fully.[48] This involves an investigation of concrete behavior in tension, compression, and shear for both multiaxial load conditions and thermal environments likely to be encountered by the PCPV during its design life.

1. In situ strength. Normally, little attention is given to the in situ concrete strength because 28-d moist-cured control specimens are used to indicate the correct strength in a particular structure and very few concrete structures actually fail.[49] Bloem[50] notes two concepts of strength that should be distinguished: (1) strength as a measure of load-carrying capacity in structures and (2) strength as a measure of concrete quality and uniformity. The relation of the latter (determined by standard cylinder tests) to the former (determined on cores from the structure) is extremely variable. Reference 49 lists the main factors affecting the in situ strength of concrete in a vessel: (1) damage caused by early-age temperature cycle, (2) presence of free water in the concrete to allow cement hydration with time, (3) degree of saturation at loading,

(4) void content in the concrete, (5) subsequent thermal damage and healing, and (6) long-term irradiation damage (reactor pressure vessels).

Studies[51–53] have shown that measurements of concrete strengths in structures do not correlate consistently with the standard tests that provide the basis for design and acceptance. Core strengths (Ref. 50) ranged from ~60 to 100% of the control specimen's strength. Field-cured control specimens provided somewhat improved results; they produced results within 10 to 21% of core strengths, depending on the curing conditions. Although core samples tested lower than control specimens, Bloem[5] sees no cause for alarm because design formulas based on compressive strength have a large margin of safety. Furthermore, for mass concrete structures such as PCPVs, core strengths were equal to or greater than the 28-d control specimen strengths.

2. Uniaxial compresssive strength, modulus of elasticity, and Poisson's ratio. Compressive strength of concrete is considered to be its most valuable property because most structures are designed on the assumption that concrete resists compressive stresses only. Values of concrete compressive strength for general purpose construction range from 20.7 to 41 MPa (concretes having compressive strengths in excess of this range will be considered as specialty concretes). The two primary factors affecting concrete strength are the degree of compaction and the water-cement ratio. The effect of coarse aggregate on concrete strength is of secondary importance; its effect varies in magnitude depending on the water-cement ratio of the mix. The rate of strength gain varies according to the type of cement used [Fig. 1 (obtained from Ref. 30)]. Curing significantly influences both the rate of strength gain and the ultimate strength of concrete [Fig. 2 (obtained from Ref. 54)]. Temperature and degree of lateral restraint also influence the concrete strength, but their effects will be considered in later sections. Furthermore, obtaining a valid measure of concrete's load-carrying ability is difficult because the value obtained is influenced by (1) specimen size, (2) relation between specimen coordinate dimensions, (3) loading rate, (4) moisture content, (5) method of specimen preparation, and (6) degree of restraint at the bearing surfaces. An example of the effect of testing conditions is illustrated in Fig. 3 (obtained from Ref. 55), which shows

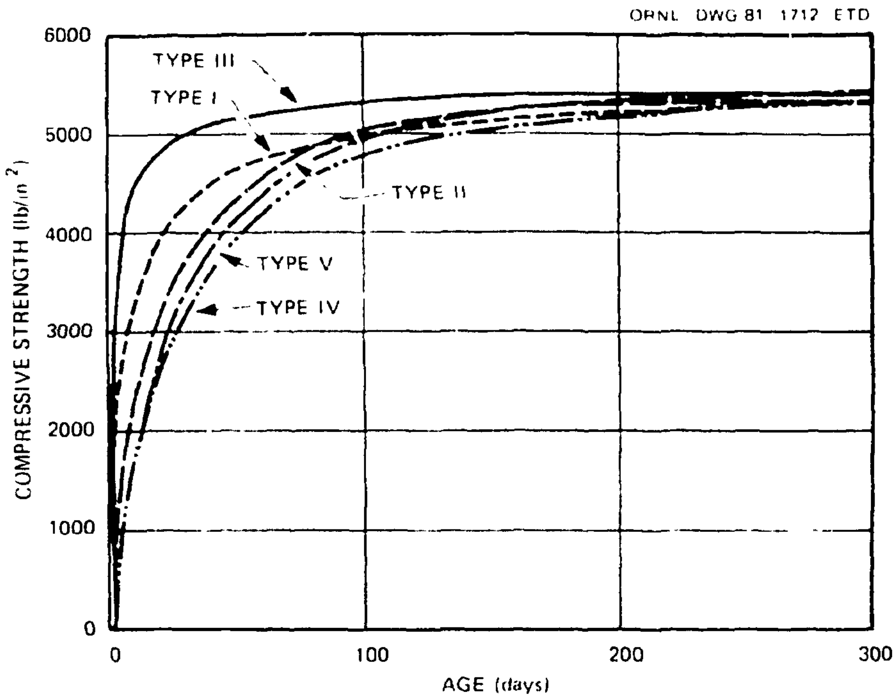

Fig. 1. Rate of development of strength on concrete made with cements of different types. Source: A. M. Neville, *Properties of Concrete*, Pitman, London, 1970.

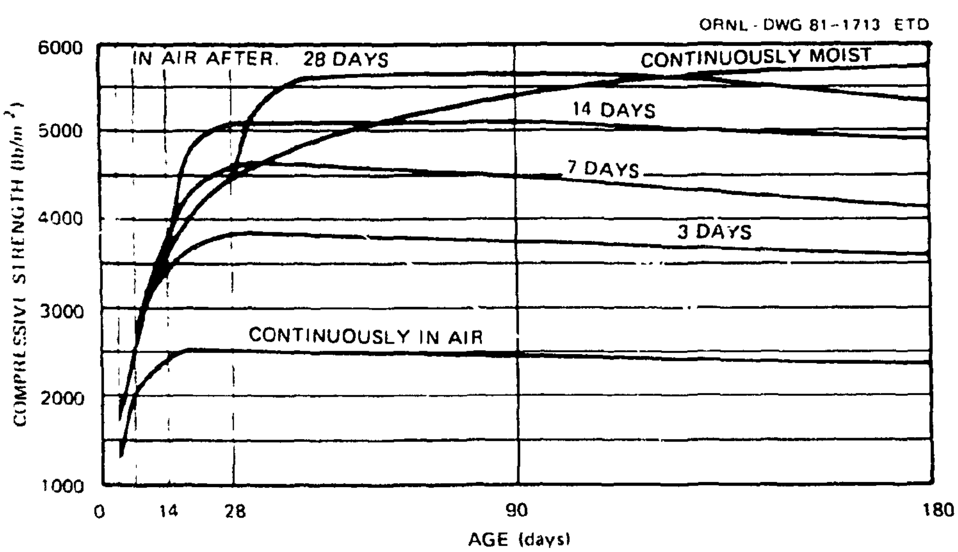

Fig. 2. The influence of moist curing on strength of concrete with a water/cement ratio of 0.50. Source: W. H. Price, "Factors Influencing Concrete Strength," pp. 417–32 in *Proc. J. Am. Concr. Inst.* (February 1951).

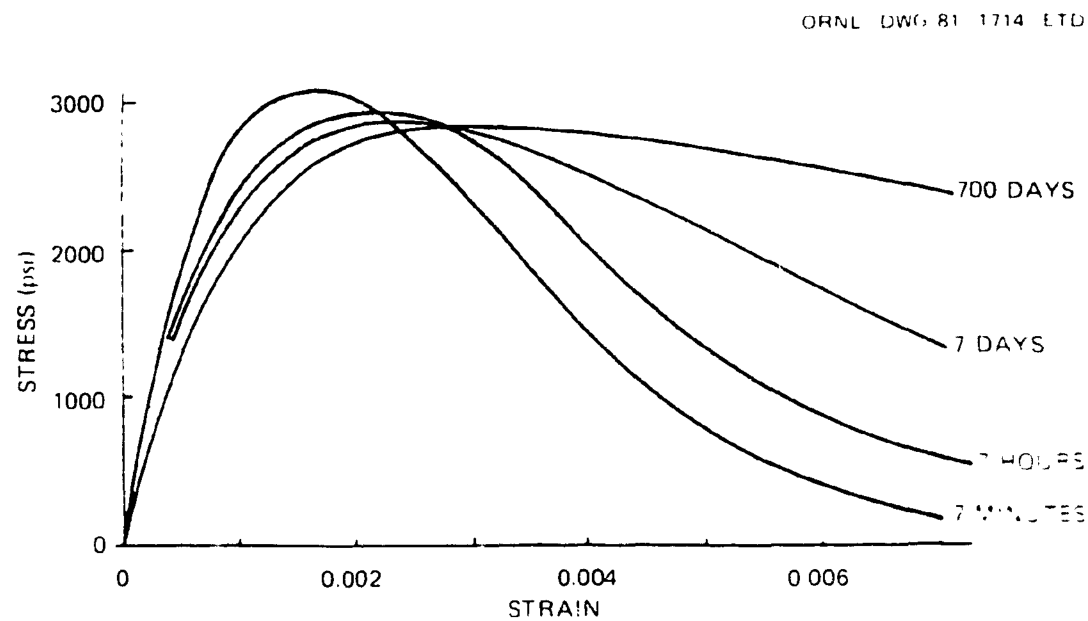

Fig. 3. Effect of strain rate on concrete stress-strain behavior. Source: J. W. Dougill, "Structural Properties of Concrete: A Review," Lecture X, Prestressed Concrete Nuclear Reactor Structures (March 19[illegible]).

the effect of strain rate on concrete tested in uniaxial compression. As indicated, the peak stress did not change appreciably with strain rate, but the strain at peak stress and toughness of material did. Note also that only significantly stiff testing machines can provide a true representation of concrete behavior.

Concrete's modulus of elasticity — a measure of its stiffness or resistance to deformation — is used extensively in the analysis of reinforced concrete structures to determine the stresses developed in simple elements and the stresses, moments, and deflections in more complicated structures. Because concrete's stress-strain curve is nonlinear, the modulus of elasticity is determined by either the initial tangent modulus, secant modulus, or tangent modulus method. Factors that influence the strength of concrete also influence the modulus of elasticity but to a lesser degree. Principal variables affecting modulus include (1) richness of mix (richer the mix, the greater the modulus increase with age); (2) water-cement ratio (higher values reduce modulus); (3) age (modulus increases rapidly during first few months and shows continual increase up to ~3 years); (4) kind and gradation of aggregate (stiffer aggregates produce higher modulus concretes and the modulus increases with aggregate fineness modulus as long as mix is workable); (5) and moisture content at time of test (wet specimens produce higher modulus values than dry specimens). The effect of temperature significantly affects modulus values and will be discussed later.

Poisson's ratio is not considered in many concrete designs; however, it is needed for conducting structural analyses of flat slabs, arch dams, tunnels, tanks, and other statically indeterminate members. Poisson's ratio for concrete can vary from 0.11 to 0.32 but is generally in the range from 0.15 to 0.20. Available data do not indicate a consistent trend for variation of Poisson's ratio with age, strength, or other concrete properties. However, some test results indicate that the ratio increases with the age of concrete up to one to two years and is lower for higher strength concretes.[30,56] Some available data indicate that Poisson's ratio decreases with increasing temperature.[57]

Requirements for obtaining concrete samples for strength evaluations of concrete in PCPV structures are listed in ANSI/ASTM C 172-71 (Ref. 58). Control cylinders should be molded and cured in accordance with ANSI/ASTM C 31-69 (Ref. 59) and tested in accordance with ANSI/ASTM C 39-72 (Ref. 60) and C 469-65 (Ref. 61) to obtain mechanical properties. The frequency for obtaining samples is listed in Table 4. Each strength test result should be the average of two or more cylinders made from the same sample test at 28 d or at the age specified in design documents. The concrete strength level would be considered acceptable if the averages of all sets of three consecutive strength tests exceeded the specified strength or if no individual strength result fell below the specified strength by more than 3.5 MPa. If the cylinder strength test falls below the acceptance criteria, the load-bearing capacity can be demonstrated by testing cores drilled from the structure in the questionable area using procedures described in ANSI/ASTM C 42-77 (Ref. 62). Concrete in the area from which the core specimen was obtained is structurally adequate when (1) the average of three strength values is equal to at least 85% of the specified strength and (2) no single core is less than 75% of specified strength.

3. Tensile strength. The tensile strength of concrete is important because it determines the ability of concrete to resist cracking. Concrete's tensile strength is closely related to its compressive strength because as the latter increases the former also increases but at a slower rate.[30] Concrete's tensile strength generally ranges from 7 to 11% of its compressive strength.[63] Direct measurement of concrete's tensile strength is seldom made because of difficulties in gripping the specimen to apply loads. Thus, this test is not used for concrete quality control and, in fact, no standard test exists. An indication of concrete's tensile strength, however, can be obtained from two tests: (1) splitting tension and (2) flexure.

The splitting tensile test is an indirect test for the tensile strength of concrete. In the test, a conventional 152-mm-diam by 305-mm concrete cylinder (which has been cast and cured in the same manner as a compression test cylinder) is loaded in compression through plywood bearing strips 3.2 mm thick[63] placed along two axial lines that are diametrically opposite on the specimens. Failure occurs in tension along the vertical diameter when concrete's compressive strength is at least three times its tensile strength. The splitting tensile strength is then computed from

$$\sigma = 2P/(\pi d \ell) ,$$

where

σ = splitting tensile strength (psi),
P = maximum applied load (lb),
ℓ = length of cylinder (in.),
d = diameter of cylinder (in.).

Splitting tensile strength is ~15% higher than that determined by direct tension tests and ranges from 8 to 14% of the compressive strength.

Flexure strength of concrete is expressed in terms of "modulus of rupture," which is the maximum tensile (or compressive) stress at rupture computed from

$$\sigma = Mc/I ,$$

where

σ = stress in fiber farthest from neutral axis (psi),

M = bending moment at the section (in.-lb),

I = moment of inertia of cross section (in.^{4}),

c = distance from neutral axis to farthest fiber (in.).[64,65]

Because the flexure formula was derived for linear elastic conditions, it is a fictitious value but is convenient for comparison purposes. The modulus of rupture is 60 to 100% higher than the direct tensile strength, 11 to 23% of the compressive strength, and 100 to 133% of the splitting tensile strength.[56] Figure 4 (obtained from Ref. 66) presents relations between modulus of rupture, compressive strength, and direct tensile strength for concrete strengths up to 62 MPa.

4. Shear strength. Shear is the action of two equal and opposite parallel forces applied in planes a short distance apart. Shear stresses

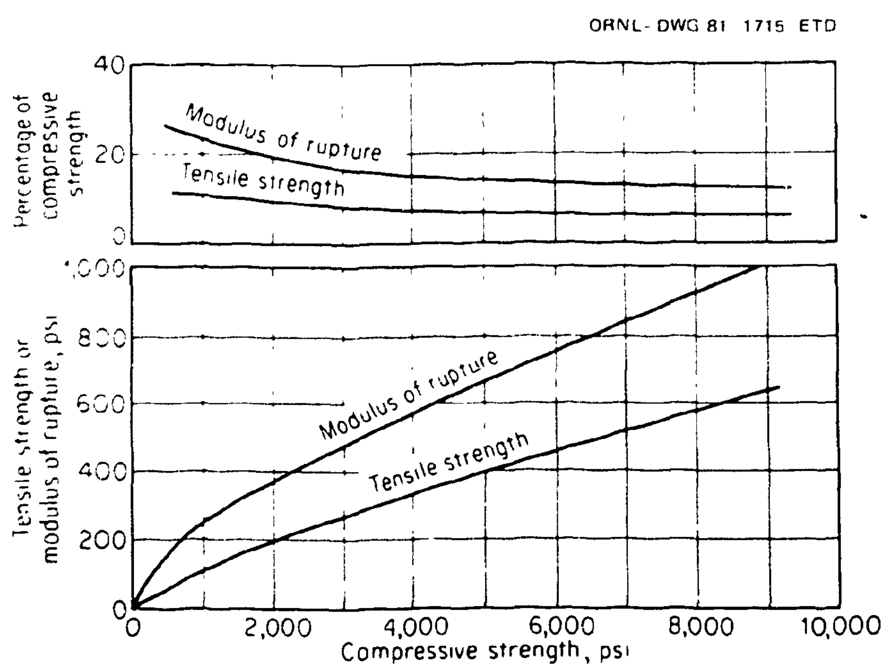

Fig. 4. Relations between compressive strength, direct tensile strength, and modulus of rupture. Source: H. F. Gonnerman and E. C. Shuman, "Compression, Flexure, and Tension Tests of Plain Concrete," *ASTM Proc.* 28(II), 527–73 (1928).

cannot exist without accompanying tensile and compressive stresses. Pure shear can be applied only through torsion of a cylindrical specimen. Because concrete is weaker in tension than shear, failure in torsion invariably occurs in diagonal tension. Tests to determine shearing strength directly are inconclusive because of the effects of bending, friction, cutting, or lateral restraint imposed by the test apparatus. Some investigators have concluded that the shear strength is 20 to 30% greater than the tensile strength (≈12% of the compressive strength), while others have determined the shear strength to be several times the tensile strength (50 to 90% of the compressive strength).[56]

Although no standard test method is available for directly measuring the shear strength of concrete, investigators[67] have used an S-shaped parallelepiped specimen (Fig. 5) to determine relative concrete shear strength data. The specimen is a modification of one tested to investigate shear transfer in reinforced concrete.[68] An indication of the shear strength is obtained by loading the specimen to failure in compression and dividing the ultimate load that the specimen can withstand by the area of the predesignated shear plane. Although the specimen does have a predesignated shear plane, specimen failure includes effects caused by tensile loadings; the results obtained therefore should be used only for comparisons and not as design values.

5. <u>Concrete-rebar bond strength</u>. Bond arises primarily from friction and adhesion between concrete and steel and may be affected by the relative magnitude of concrete shrinkage. It is a function of (1) the concrete properties (cement type, admixtures, water-cement ratio), (2) the mechanical properties of the steel (size and spacing of lugs), and (3) rebar position within the concrete member (bond is greater for vertical bars than for horizontal bars). Permissible bond stresses are generally specified as percentages of concrete's compressive strength, although [Fig 6 (obtained from Ref. 54)] a consistent relation between bond strength and compressive strength does not exist.[69-71]

The bond strength between concrete and rebar is evaluated according to procedures described in ANSI/ASTM C 234-71 (Ref. 72). As noted in the specification for the test method, the method is not intended either for

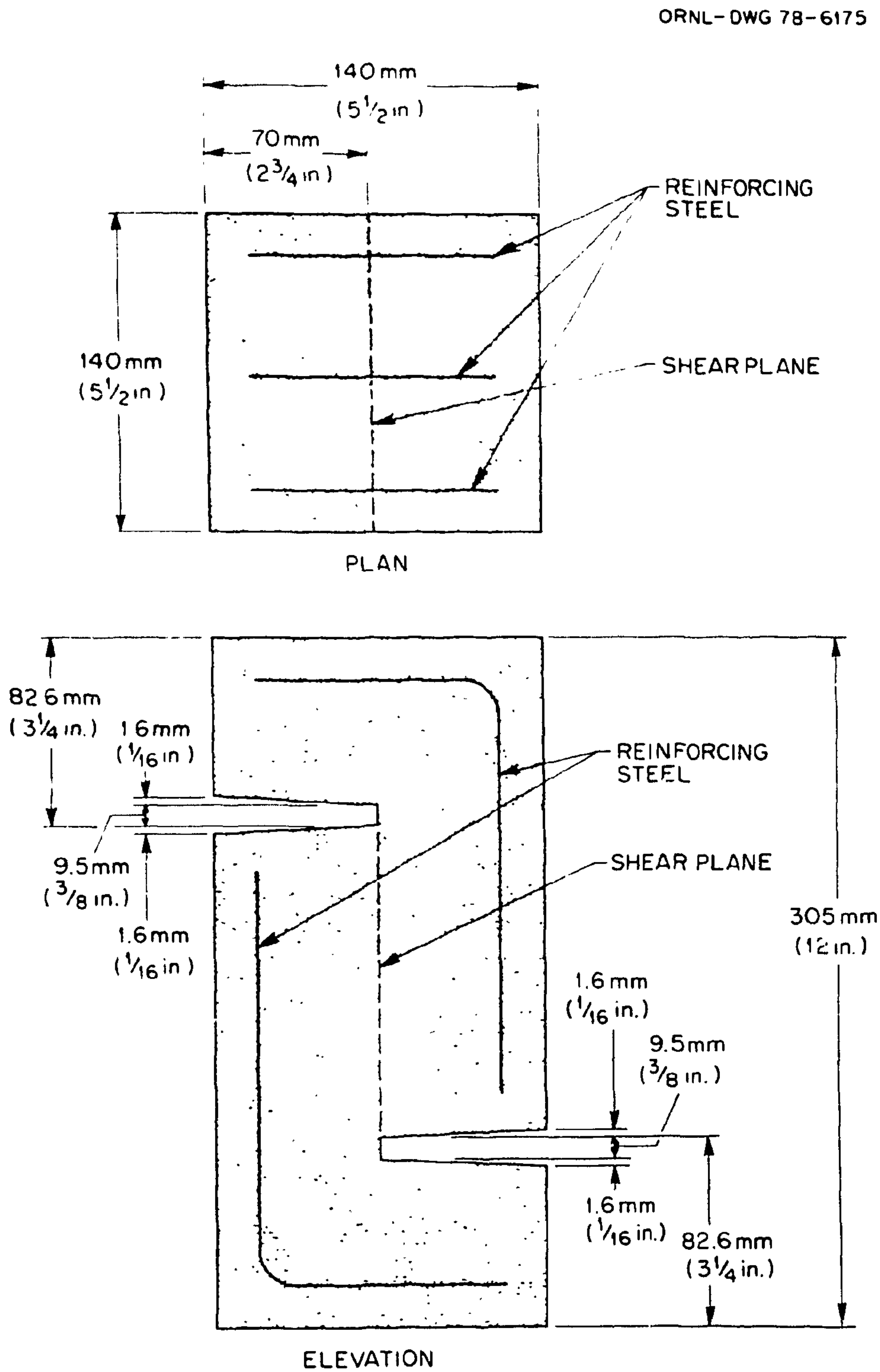

Fig. 5. S-shaped parallelepiped shear strength test specimen. Source: C. B. Oland et al., *Final Report of Comprehensive Testing Program for Concrete at Elevated Temperatures*, ORNL/BRP-80-5 (1980).

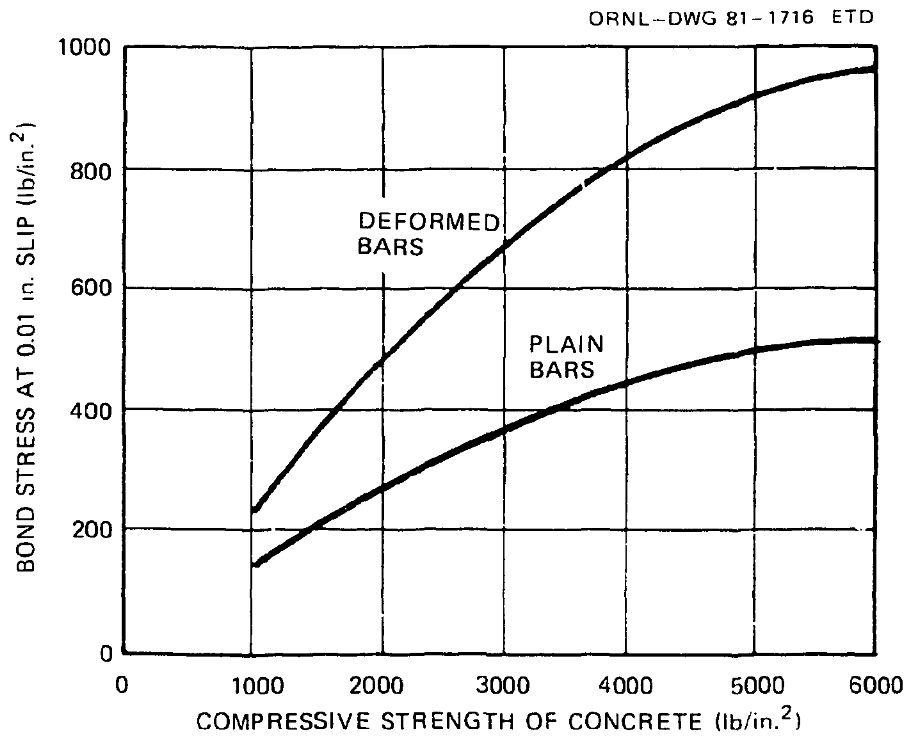

Fig. 6. Influence of strength of concrete on bond determined by pull-out tests.

use in tests in which the principal variable is the size or type of reinforcing bar or for establishing bond values for structural design purposes. Values obtained vary with the concrete, size and brand of bar, and amount of slip used as a measure of failure. The general order of magnitude of bond stress on deformed bars at free-end initial slip is 0.69 to 2.1 MPa and bond stress at the loaded end for a slip of 0.25 mm is 2.1 to 5.5 MPa.[69]

Thermal properties. Thermal properties of concrete are important both in the planning of mass concrete construction (thermal volume changes) and the dissipation of heat buildup during operation.

As water is added to cement, an exothermic chemical reaction takes place. If the heat is generated at a faster rate than it can be dissipated, a temperature rise occurs. Factors affecting the amount and rate of heat generated during this reaction are the cement type, temperature at placement, water-cement ratio, and cement content. In mass concrete structures where there can be significant heat buildup, cracking can occur upon cooling because the exterior of the structure will cool faster than the interior. However, by using low (Type IV) or moderate (Type II) heat of hydration cements and following the procedures recommended in Ref. 73, this problem can be minimized.

Temperature variations produce expansions or contractions of concrete structures. If movement of a structure is restrained, significant internal stresses can develop, thus leading to cracking, warping, or even destruction. The coefficient of thermal expansion α is used as a measure of the volume change of a material subjected to a temperature differential.

Dissipation of heat is important to a PCPV because it affects the development of thermal gradients and the resulting thermal stresses. The basic quantitites involved are (1) the coefficient of thermal conductivity k, (2) the thermal diffusivity a, and (3) the specific heat c. These quantities are related by the term

$$a = k/c\rho \ ,$$

where ρ is the material density.

1. Coefficient of thermal expansion. The coefficient of thermal expansion represents the volume change of a material due to temperature changes and is expressed as a change in length per degree of temperature change. The coefficient is important as a measure of the structural movement and thermal stresses resulting from a temperature change. Concrete's thermal expansion is a complicated phenomenon because of the interaction of its two main components — cement paste and aggregate — which each have their own coefficients of thermal expansion. Because the aggregate generally constitutes a major proportion of the mix, it primarily influences the resultant coefficient of thermal expansion. Figure 7 (obtained from Ref. 74) presents thermal coefficient of expansion values for neat cements, mortars, and concretes. As illustrated, values of the coefficient for concrete range from ~2.2×10^{-6} to 3.9×10^{-6} mm/mm/°C with 3.1 per degree Centigrade being a typical value. The coefficient is influenced by the moisture condition (applies to paste component) and has minimum

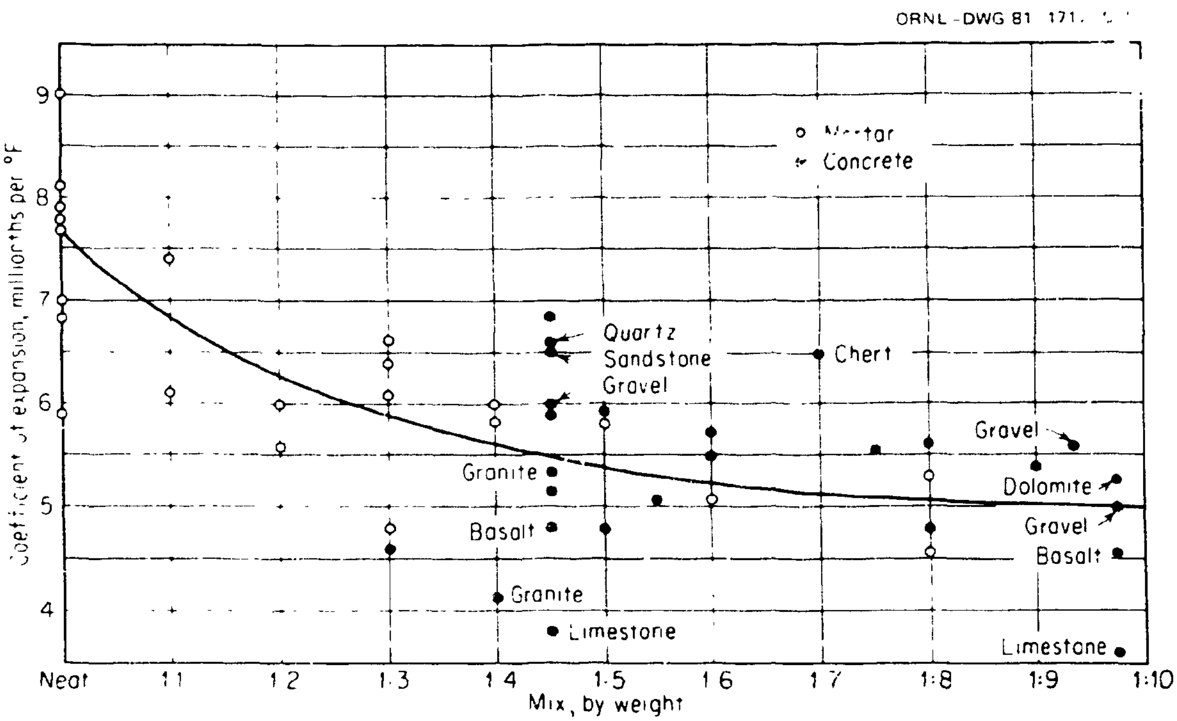

Fig. 7. Coefficients of thermal expansion of neat cements, mortars and concretes. Source: U.S. Bureau of Reclamation, *Concrete Manual*, 7th ed., Denver, Colorado, 1963.

values for the two extremes: dry and saturated (Fig. 8).[75] The coefficient of linear expansion also apparently increases with increasing temperature (Fig. 8); however, the effects of specimen moisture condition at test initiation (i.e., the number of thermal cycles that have been applied to the specimen) also has to be taken into consideration in determining the net specimen length change with temperature.[76]

2. Thermal conductivity. Thermal conductivity is a measure of the ability of the material to conduct heat and is measured in British thermal units per hour per square foot of area of body when the temperature difference is 1°F per foot of body thickness. For PCPVs, concrete with a high thermal conductivity is generally desirable but not always used to allow a rapid dissipation of heat flux so thermal gradients through the thickness will be minimal. The thermal conductivity of concrete depends on its composition and degree of saturation. Table 5 lists typical values of thermal conductivity for several concretes fabricated using a wide variety of aggregates.[77] Because the conductivity of water is approximately

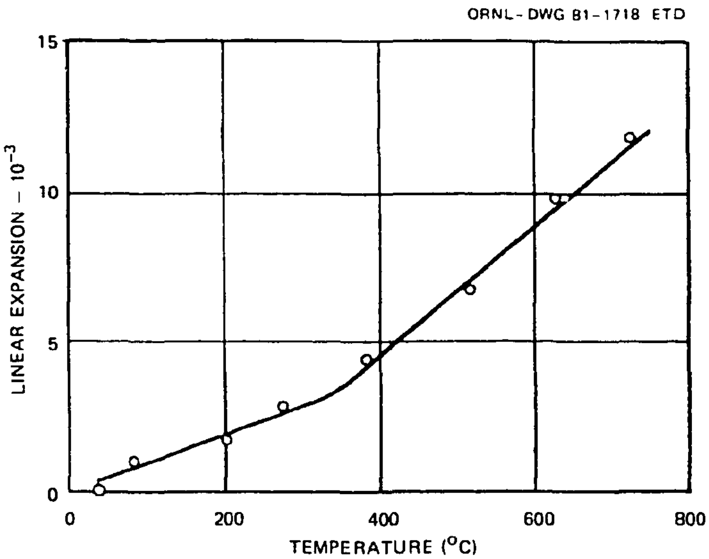

Fig. 8. Linear expansion of concrete on heating. Source: R. Philleo, "Some Physical Properties of Concrete at High Temperature," *Proc. J. Am. Concr. Inst.* 54(10) (April 1958).

Table 5. Typical values of thermal conductivity of concrete

Type aggregate	Concrete density [lb/ft^3 (kg/m^3)]	Conductivity [Btu/ft^{2}h°F/ft (MW/cm°C)]
Barite	227 (3640)	0.80 (13.9)
Igneous	159 (2550)	0.83 (14.4)
Dolomite	160 (2560)	2.13 (37)
Limestone or gravel	150 (2400)	0.75–1.00 (13–17)
Lightweight No. 1	30 (481)[a]	0.08 (1.4)
Lightweight No. 2	110 (1760)[a]	0.35 (6.1)
Pumice		0.02 (0.3)
Foam	20 (320)	0.04 (0.7)
Cinder		0.43 (7.4)
Expanded clay		0.23–0.36 (4–6.2)
Perlite or vermiculite		0.04–0.07 (0.7–1.2)

[a]Oven dried.

Source: L. J. Mitchell, *Thermal Properties*, ASTM Special Technical Publication 169, 129–35 (October 1962).

half that of cement paste, the lower the mix-water content, the higher the conductivity of the hardened concrete.[30] The effect of temperature on conductivity is relatively insignificant, and the change for an increase in temperature range is likely to be positive for low-conductivity concrete and negative for high-conductivity concrete.[56,77] Typically the thermal conductivity of concrete ranges from 1.4 to 3.6 W/m·K (0.8 to 2.1 Btu ft^{-2} h^{-1} $°F^{-1}$ ft^{-1}) for normal saturated concrete between 10 and 66°C (50 and 150°F). Because thermal diffusivity is easier to measure, concrete's thermal conductivity is generally calculated from the diffusivity.

3. Thermal diffusivity. Thermal diffusivity is a measure of the rate at which heat will diffuse through a material in all directions due to a temperature change and is thus an index of the facility with which the material will transfer heat due to a temperature change. Thermal diffusivity is important to PCPVs for the same reasons cited for thermal conductivity and is influenced by the same factors. Aggregates with increasing values of diffusivity include basalt, rhyolite, granite, limestone, dolerite, and quartzite.[78] Typical values for concrete diffusivity range from 0.002 to 0.006 m^2/h (0.02 to 0.06 ft^2/h). References 79 through 81 present procedures for determining concrete's thermal diffusivity.

4. Specific heat. Specific heat is the amount of heat required to change the temperature of 1 lb (0.45 kg) of material 1°F (0.56°C) and thus represents the heat capacity of the material. Specific heat is only slightly affected by the mineralogical character of the aggregate but increases considerably as the moisture content of the concrete increases. Specific heat also varies with the temperature.[81] Typical values for concrete's specific heat range from 837 to 1172 J/kg·K (0.20 to 0.28 Btu/lb·°F). References 82 and 83 describe test procedures for obtaining specific heat values for concrete.

Shrinkage and creep. Shrinkage of concrete is important because of its effect on movement of the structure and its tendency to induce cracking. Shrinkage occurs as a result of two effects: (1) drying or (2) autogenous volume change. Drying shrinkage results from the loss of absorbed water and is generally the more predominant of the two effects. Autogenous shrinkage is more prevalent in mass concrete structures where the total moisture content remains relatively constant; it results from continued cement hydration reducing the free-water content (products of hydration occupy less volume than the sum of the separate volumes of the original components). Several factors affect concrete drying shrinkage: (1) cement and water contents (shrinkage of cement paste varies directly with the water-cement ratio;[84] (2) composition and fineness of cement (Table 6);[84] (3) type and gradation of aggregate (shrinkage inversely proportional to size and amount of coarse aggregate — sandstone, slate, basalt, and trap rock produce concretes having greater shrinkage than quartz, limestone, dolomite, granite, and feldspar aggregate concretes); (4) admixtures (those that increase water requirement increase shrinkage, and those that reduce water requirement decrease shrinkage); (5) moisture and temperature conditions; and (6) amount and distribution of reinforcement (reinforcement inhibits shrinkage and can lead to cracking, Table 7).[85] The effects of several variables influencing autogenous shrinkage are summarized in Fig. 9 (Ref. 56). In mass structures such as PCPVs where the concrete is maintained well below 100°C, Browne contends that shrinkage will not be a significant factor in design over the 30- to 40-year vessel life unless severe temperature gradients exist.[49]

Creep can be defined as the increase in strain in a structural member with time due to a sustained stress (Fig. 10). Because creep affects strains, deflections, and stress redistribution, it is important with

Table 6. Shrinkage of neat cements in air at age of one year

Cement	Shrinkage [millionths (10^{-6} in./in.)]
Type I, normal	2150
Type III, high-early-strength	2335
Type IV, low-heat	2870
Portland-pozzolan	3150

Source: R. W. Carlson, "Drying Shrinkage of Concrete as Affected by Many Factors," *ASTM Proc.* 38(II), 419–37 (1938).

Table 7. Effect of reinforcement on volume change

Reinforcement (%)	Contraction [millionths (10^{-6} in./in.)]	Shrinkage stress [psi (MPa)]	
		Steel, compression	Concrete, tension
0.00	640	0 (0)	0 (0)
0.55	540	19,000 (131)	100 (0.7)
1.23	420	15,500 (107)	200 (1.4)
2.18	290	12,000 (83)	250 (1.7)

Source: T. Matsumato, "Study of the Effect of Moisture Content Upon the Expansion and Contraction of Plain and Reinforced Concrete," *Univ. Ill. Eng. Exp. Sta. Bull. No. 126* (Dec. 5, 1921).

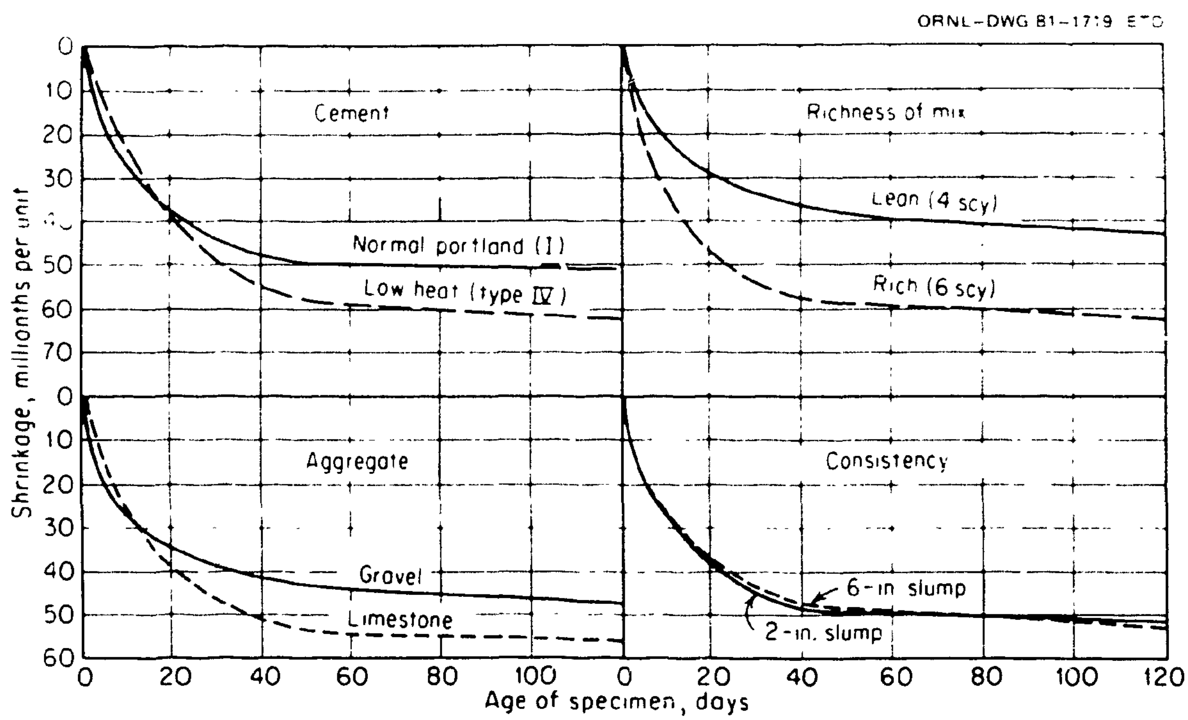

Fig. 9. Effect of several factors on autogenous shrinkage of concrete. Source: G. E. Troxell et al., *Composition and Properties of Concrete*, 2d ed., McGraw-Hill, New York, 1968.

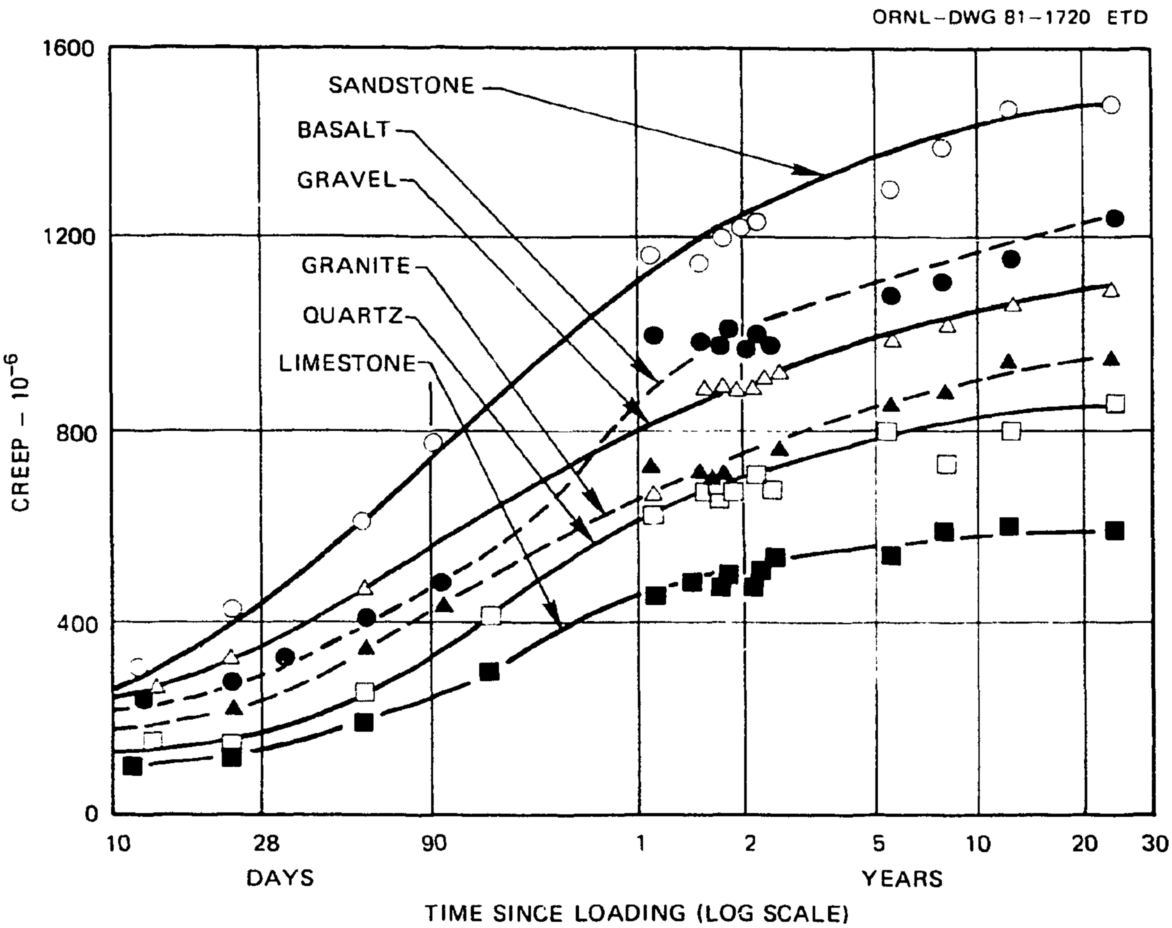

Fig. 10. Creep of concretes with different aggregates (aggregate-cement ratio 5.67, water-cement ratio 0.59, stress 56 kg/cm^2). Source: G. E. Troxell et al., "Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete," *ASTM Proc.* 48, 1101–120 (1958).

respect to structural analyses. Creep may also be viewed from another standpoint: if a loaded specimen is restrained from movement (constant net strain), creep will manifest itself as a progressive decrease in stress with time (stress relaxation). Although creep is generally considered only for specimens loaded in compression, creep of concrete in tension also occurs and is on the same order of magnitude as creep in compression.[86] Also, upon release of the sustained load, an initial elastic recovery of strain occurs followed by creep recovery that can continue for several days. The magnitude of creep recovery is greater for concrete specimens that were loaded later in their cure cycle, and it is inversely proportional to the period of sustained stress.

Several theories for the creep mechanism have been proposed: viscous flow of the cement-water paste, closure of internal voids, crystalline flow of aggregate, and seepage into internal voids of colloidal (adsorbed) water formed by cement hydration.[56] Some investigators[87] divide creep in two types: (1) basic creep under conditions of hygrometric equilibrium caused by molecular diffusion of the gel and absorbed water, causing a partially viscous (irrecoverable) and partially delayed elastic (partially recoverable) behavior; and (2) drying creep caused by a mechanism similar to that involved in free shrinkage due to desiccation.

Several physical and environmental parameters affect creep deformations of normal-weight concrete. Physical parameters inherent to the particular concrete mix include: (1) cement type (degree of hydration); (2) cement paste proportions and content (creep proportional to volume fraction of cement paste in mix); (3) aggregate properties and volume fraction: (a) aggregate restrains creep, (b) mineral character effects are presented in Fig. 10, and (c) creep tends to be inversely proportional to maximum aggregate size for uniformly graded mixes;[56,88,89] (4) strength and stage of hydration: (a) creep decreases with degree of cement hydration of a mix, and (b) generally the amount of creep is inversely proportional to the concrete strength; (5) moisture conditions of storing: (a) creep is generally inversely proportional to the relative humidity of the medium surrounding the concrete (Fig. 11);[30] and (6) size of mass (the larger the mass, the lower the creep). Mechanical parameters include: (1) state of stresses: (a) under uniaxial compressive stress for stress-strength ratio <0.4, creep is proportional to applied stress; (b) at high stress-strength levels ($\geq$0.85), creep can lead to failure; (c) creep under multiaxial compression is less than under uniaxial compression of the same magnitude in the given direction (Fig. 12); (d) creep occurs under hydrostatic compression;[27,90,91] (2) age at loading: (a) specific creep decreases for increased loading age; and (3) temperature: (a) creep follows the same general pattern as creep at room temperature — being an exponential function of time under load and a relatively linear function of stress up to a stress-strength ratio of ~0.4; (b) sealed specimens exhibit

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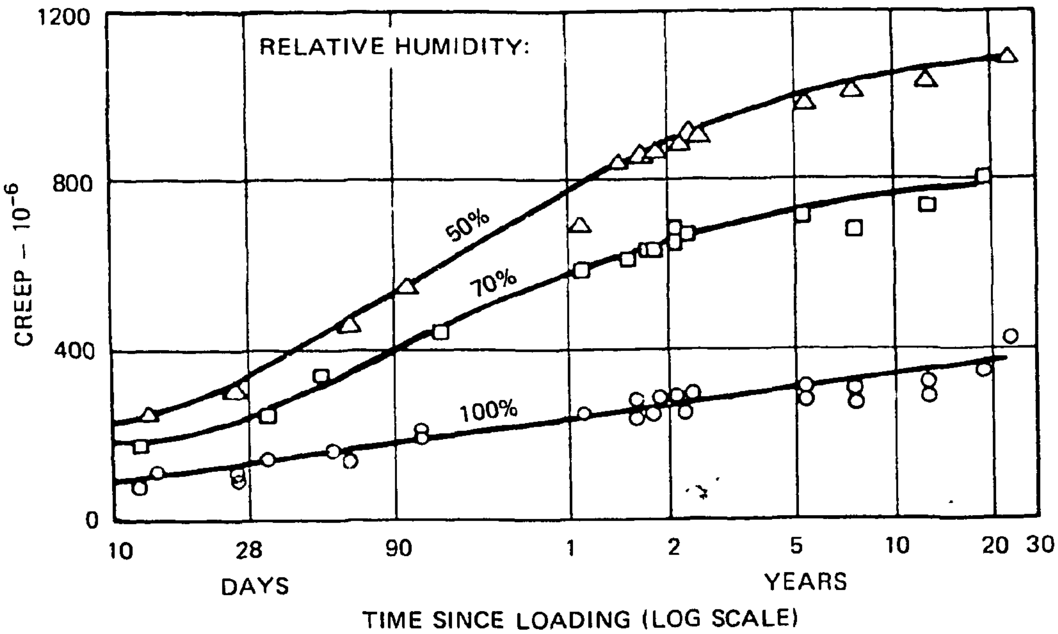

Fig. 11. Creep of concrete stored at different relative humidities. Source: A. M. Neville, *Properties of Concrete*, Pitman, London, 1970.

less creep than unsealed; (c) creep definitely increases with temperature up to at least 50°C and probably increases with temperature up to 150°C (Fig. 13); and (d) the degree of creep recovery appears to be more dependent on stress level than temperature.[92,93]

2.1.6 Effect of elevated temperatures

1. Significance and current practice. 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. Relative to allowable concrete temperatures, current designs for PCPVs are conservative because of a lack of understanding of concrete behavior at elevated temperatures. Table 8 presents current American Society of Mechanical Engineers (ASME) code limits for various locations in a PCRV for the appropriate conditions (normal operation and abnormal environment).[1] As noted in Table 8, the temperature in the concrete should not exceed 65°C

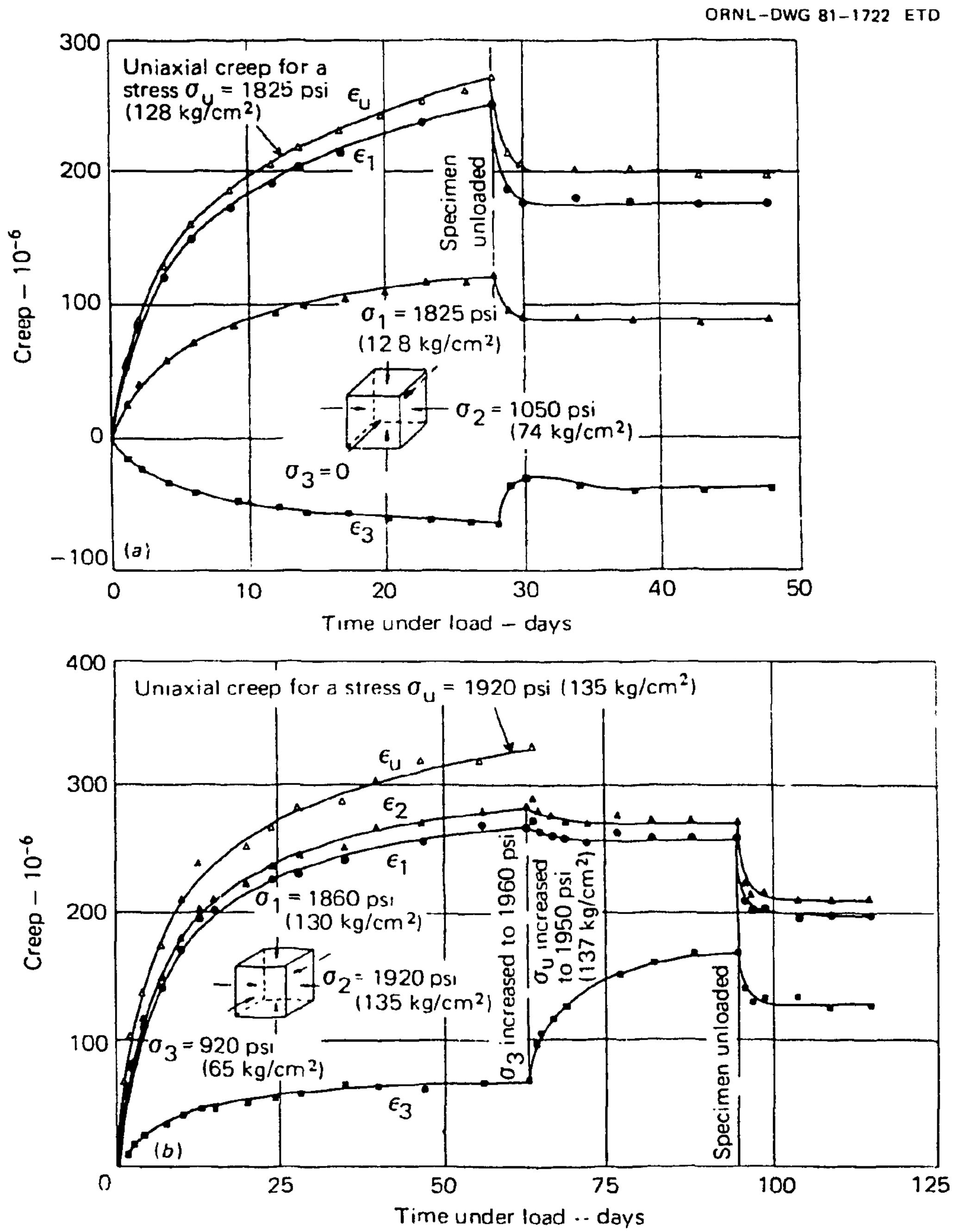

Fig. 12. Typical creep-time curves under multiaxial compression: (a) biaxial; (b) triaxial. Source: A. M. Neville, *Creep of Concrete: Plain, Reinforced and Prestressed*, North Holland Publishing Co., Amsterdam, 1970.

Table 8. 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 (1975).

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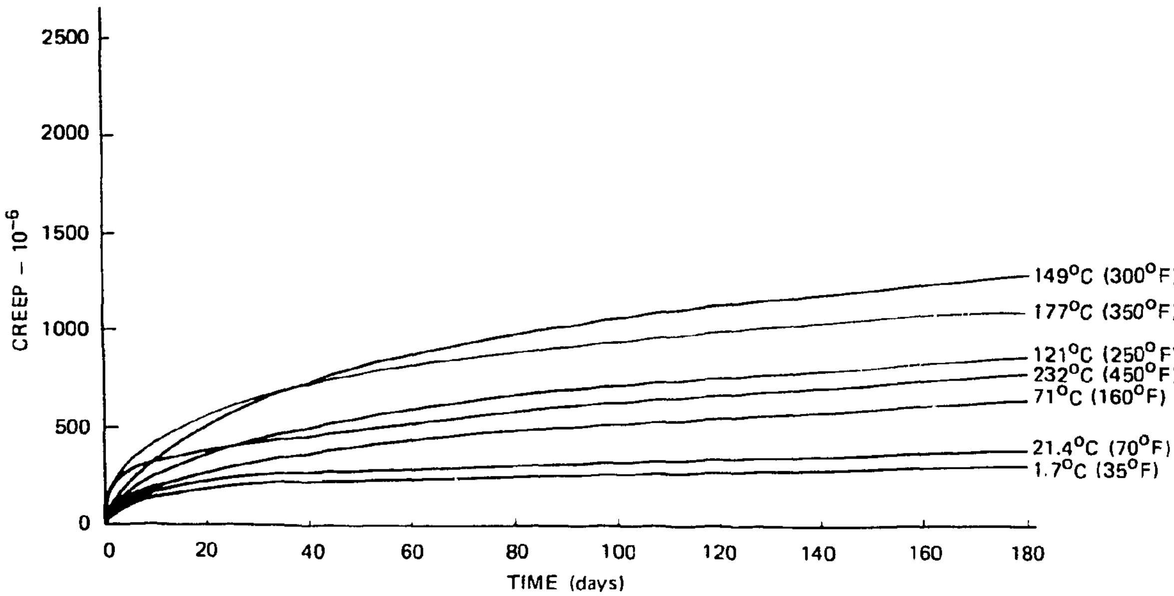

Fig. 13. Creep of sealed concrete at various temperatures.

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[3] for PCRVs limits temperatures in active parts of the concrete to 90°C; the British specification[2] 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. Consequently, permissible temperatures for the concrete in PCRVs for gas-cooled reactors has generally been in the range of 45 to 80°C (Ref. 94).

2. Review of available data. It has long been established that the compressive strength and the modulus of elasticity of structural concrete generally decrease with exposure to elevated temperatures. Quantitative interpretation of the data is difficult, however, because (1) samples were tested either hot or cold, (2) moisture migration was either free

or restricted, (3) concrete was either loaded or unloaded while heated, (4) concrete constituents and proportions varied from mix to mix, (5) test specimen size was not consistent, (6) specimens were tested at different degrees of hydration, and (7) heat-soak duration varied from test to test. To better evaluate the data obtained during elevated temperature testing of concrete, Ref. 95 recommends taking into account the following factors: (1) concrete strength class; (2) test specimen size; (3) thermal compatibility of aggregates and cement-paste matrix; (4) cement and concrete composition; (5) level of temperature; (6) degree of hydration; (7) moisture content; (8) moisture gradients, rate of drying and wetting; (9) temperature gradient, rate of heating or cooling; (10) duration of temperature exposure; (11) loading during temperature exposure; (12) temperature activated transformations in microstructure and chemical composition of cement; (13) state of specimens on testing — hot or cold; (14) strength testing procedure; and (15) reference strength selected — wet, moist, or dry strength.

A review of methods used by various investigations for elevated temperature testing of concrete indicates that, generally, the tests can be categorized according to cold or hot testing. In cold testing, specimens are gradually heated to a specified temperature, permitted to thermally stabilize at that temperature for a prescribed period of time, permitted to slowly cool to ambient, and then tested to determine mechanical properties. In hot testing, specimens are gradually heated to a specified temperature, permitted to thermally stabilize at that temperature for a prescribed period of time, and then tested at temperature to determine mechanical properties. During testing, specimens are maintained in either an open environment where water vapor can escape or a closed environment where the moisture is contained. The closed environment represents conditions for mass concrete where moisture does not have ready access to the atmosphere, and the open environment represents conditions where the element is either vented or has free atmospheric communication. During heating and cooling, the specimens may be either loaded or unloaded.

References 96 through 108 present results obtained from elevated temperature testing of concrete. Figures 14 and 15 present a summary of published results on the residual compressive strength of concrete exposed

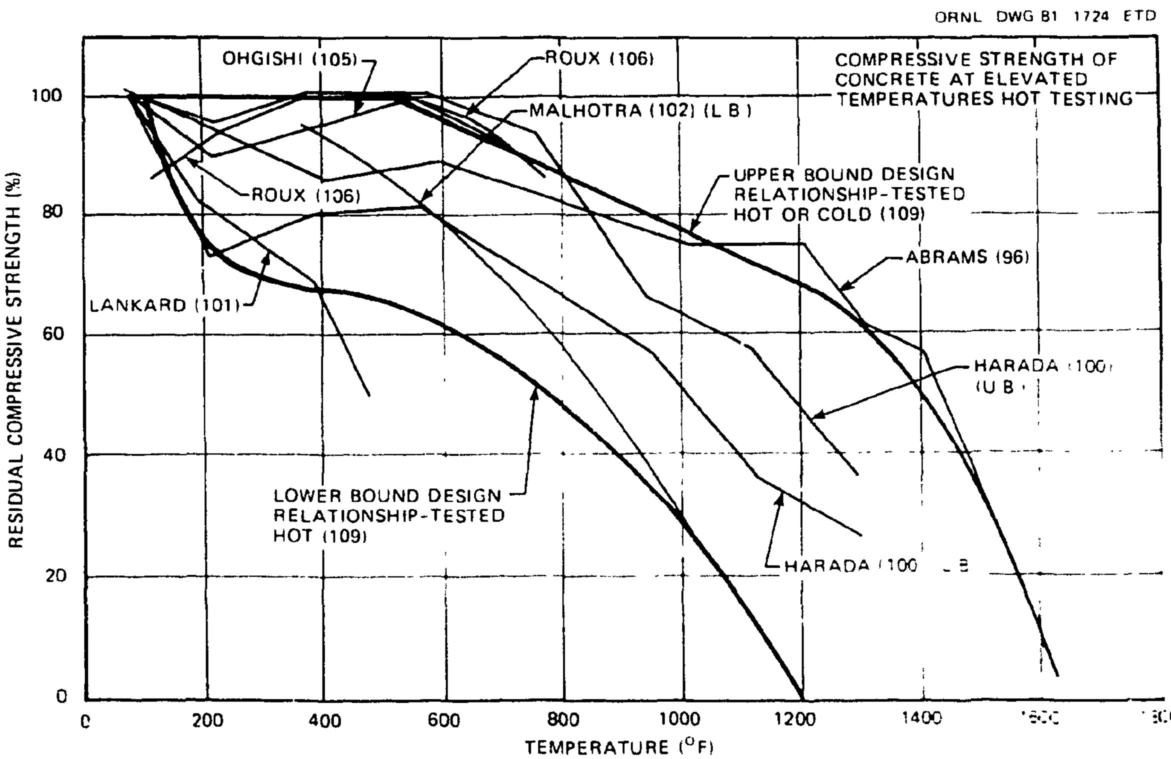

Fig. 14. Effect of temperature exposure on compressive strength of concrete-hot testing. Source: G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperatures," *Civ. Eng. Nucl. Power*, Vol. 1, ASCE National Convention, Boston, Mass. (April 1979).

to elevated temperatures for hot testing and cold testing, respectively. The effect of elevated temperature on concrete's residual modulus of elasticity is presented in Fig. 16 for both hot and cold testing conditions. Reference 109, which presents results of an overview of the effects of elevated temperatures on concrete properties, lists the following general observations on the effects of elevated temperature on concretes properties.

1. Specimens lose more strength if moisture is not permitted to escape while heating than do those where the moisture escapes.[99,101,106]
2. Specimens heated and then permitted to cool before testing lose more strength than those tested while hot (Figs. 14 and 15).
3. Concrete specimens loaded during heating lose less strength than unloaded specimens.[96,102]

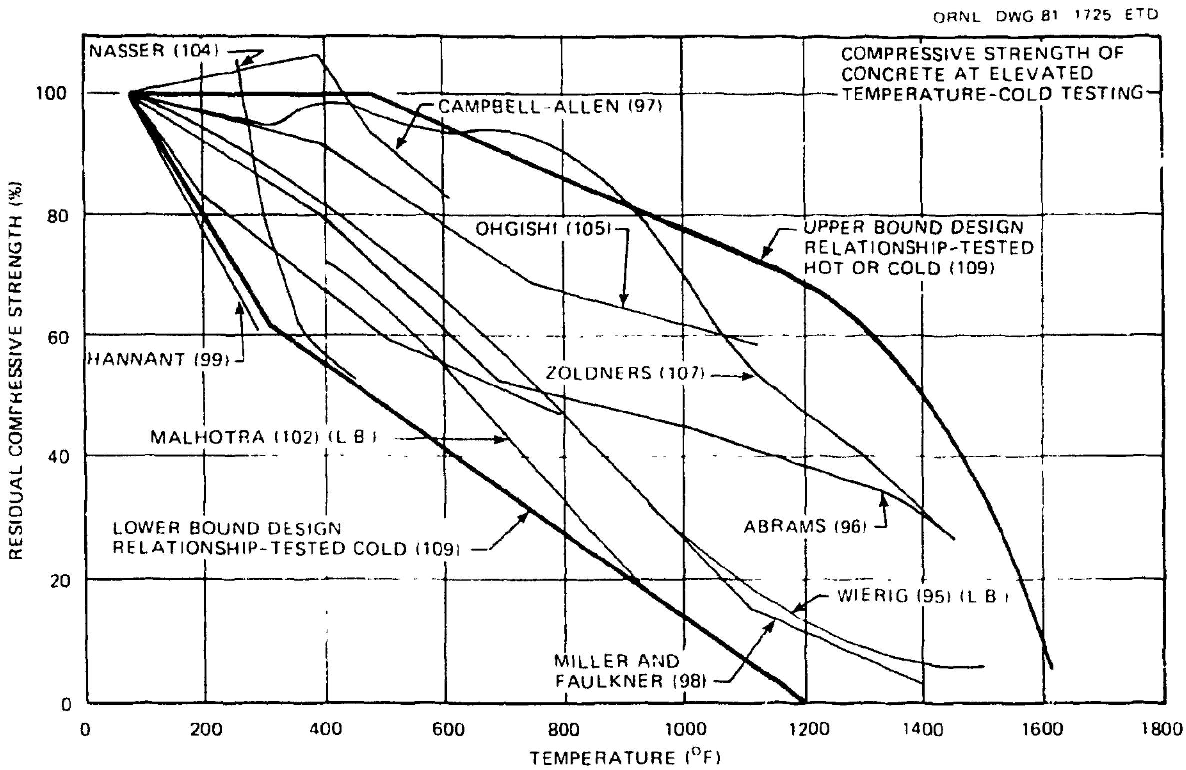

Fig. 15. Effect of temperature exposure on the compressive strengths of concrete-cold testing. Source: G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperatures," *Civ. Eng. Nucl. Power*, Vol. 1, ASCE National Convention, Boston, Mass. (April 1979).

4. The longer the duration of heating before testing, the larger the loss in strength. This loss of strength, however, stabilizes after a period of long isothermal exposure.
5. The decrease in modulus of elasticity caused by elevated temperature exposure is more pronounced than the decrease in compressive strength (Figs. 14 through 16).
6. Mix proportions and aggregate type influence the strength of heated concrete as follows: (a) low cement-aggregate ratio mixes lose less strength as a result of heating than richer mixes;[102,105] (b) concrete made with limestone aggregate degrades less because of heating than concrete made with siliceous aggregate.[98,100,105]
7. The water-cement ratio has a limited effect on strength degradation of heated concrete.[102]

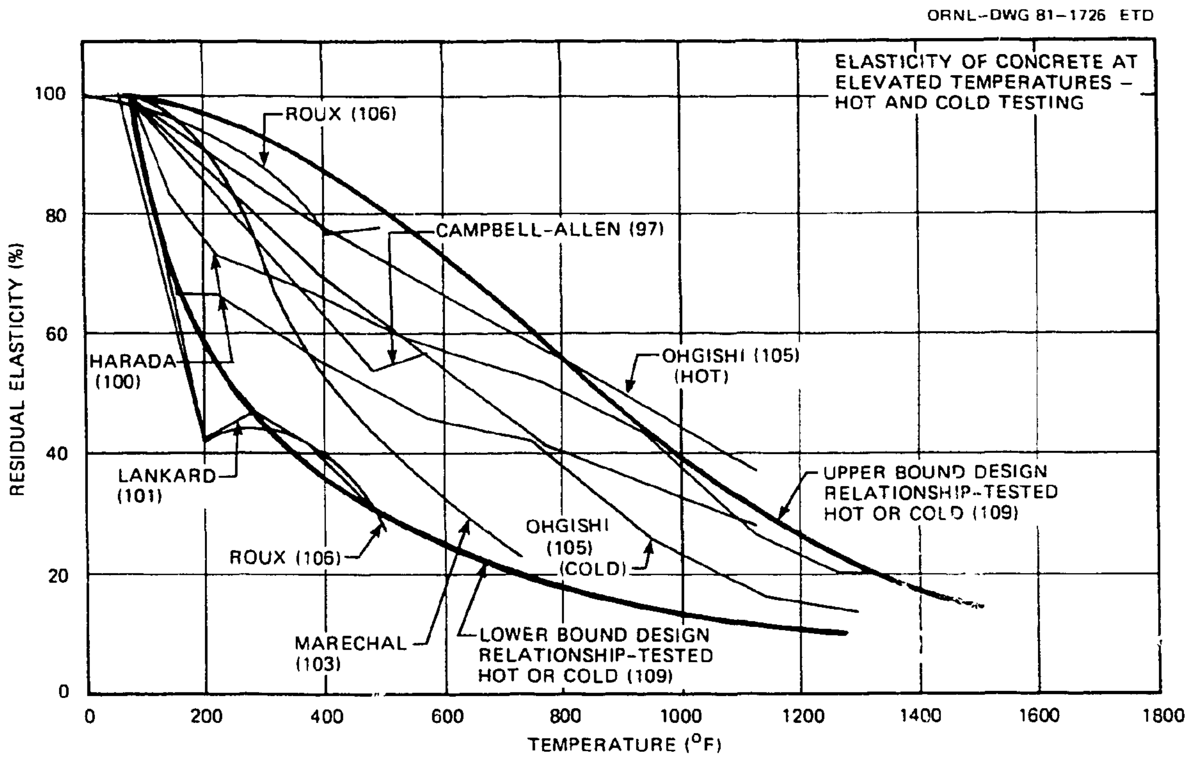

Fig. 16. Effect of temperature exposure on the modulus of elasticity of concrete-hot and cold testing. Source: G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperatures," *Civ. Eng. Nucl. Power*, Vol. 1, ASCE National Convention, Boston, Mass. (April 1979).

8. Small test specimens generally incur greater strength losses than larger ones.
9. Specimens subjected to several cycles of heating and cooling lose more strength than those not subjected to thermal cycling.[97]
10. The strength of concrete before testing has little effect on the percentage of strength retained at elevated temperatures.[96]

3. Stress-strain relationships. Evaluation of structures for small strain conditions involves elastic analysis procedures for which knowledge of the concrete modulus of elasticity and strength is sufficient. When large strains are involved (such as those that can occur when a structure is subjected to elevated temperatures), elastic-plastic analysis procedures are required that involve the use of load-deformation or stress-

strain relations developed for concrete at the elevated temperature level of interest.

A generalized compressive stress-strain curve for concrete (Fig. 17)[110] consists of four regions: (1) 0 to 30% ultimate stress where crack extension is confined to the aggregate matrix interface and the curve is relatively linear, (2) 30 to 50% ultimate stress where bond crack growth increases, (3) 50 to 75% ultimate stress where matrix cracking bridges between bond cracks, and (4) 75 to 100% ultimate stress where cracks reach a critical size and grow spontaneously. After reaching ultimate stress, the cracking increases with continued loading, causing the stress-strain curve to descend until failure occurs. The stress-strain curve for concrete at elevated temperatures is similar to that at normal temperature except the maximum stress is attained at a much higher strain.

A number of relationships have been proposed by various authors to describe concrete's stress-strain behavior.[109,111-113] These expressions

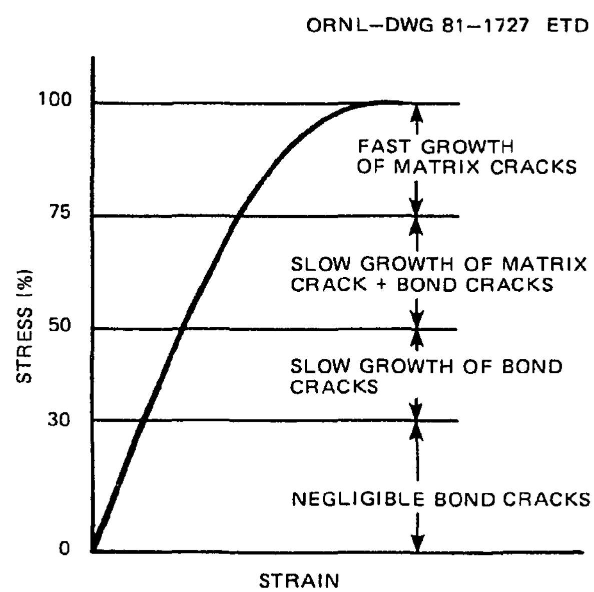

Fig. 17. Generalized stress-strain curve for concrete. Source: J. Glucklich, "The Effect of Microcracking on Time-Dependent Deformations and Long-Term Strength of Concrete," *Proceedings First International Conference on the Structure of Concrete and Its Behaviour Under Load*, Cement and Concrete Association, London, 1968.

generally provide good agreement with the ascending portion of the stress-strain curve but differ significantly beyond the point of maximum stress. According to Ref. 113, the stress-strain relationships at elevated temperatures may be derived from the room-temperature relationships if the variation of maximum stress and corresponding strain with temperature is known. Examples of the effect of elevated temperature on concrete's stress-strain behavior are shown in Fig. 18 for a mass concrete mix containing flyash and in Fig. 19 for a quartz aggregate concrete.[114,115] Figure 20 presents the effects of thermal cycling on the stress-strain behavior of a limestone aggregate concrete that was tested and sealed to simulate mass concrete conditions.[116]

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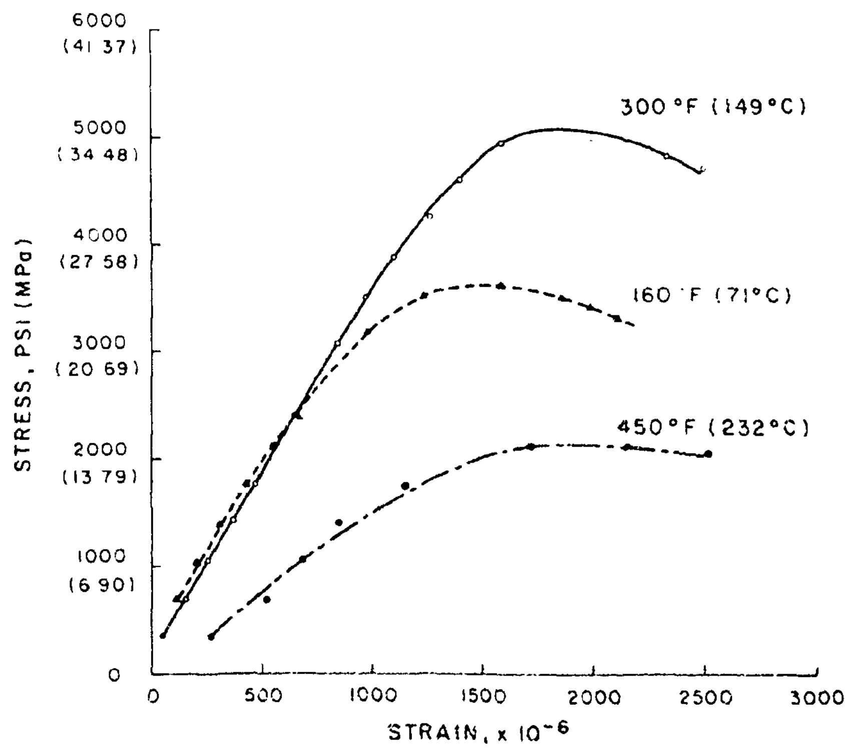

Fig. 18. Effect of elevated temperature on stress-strain behavior of a mass concrete mix containing fly ash. Source: K. W. Nasser and H. M. Marzouk, "Properties of Mass Concrete Containing Fly Ash at High Temperatures," pp. 537–50 in *Proc. J. Am. Concr. Inst.* (April 1979).

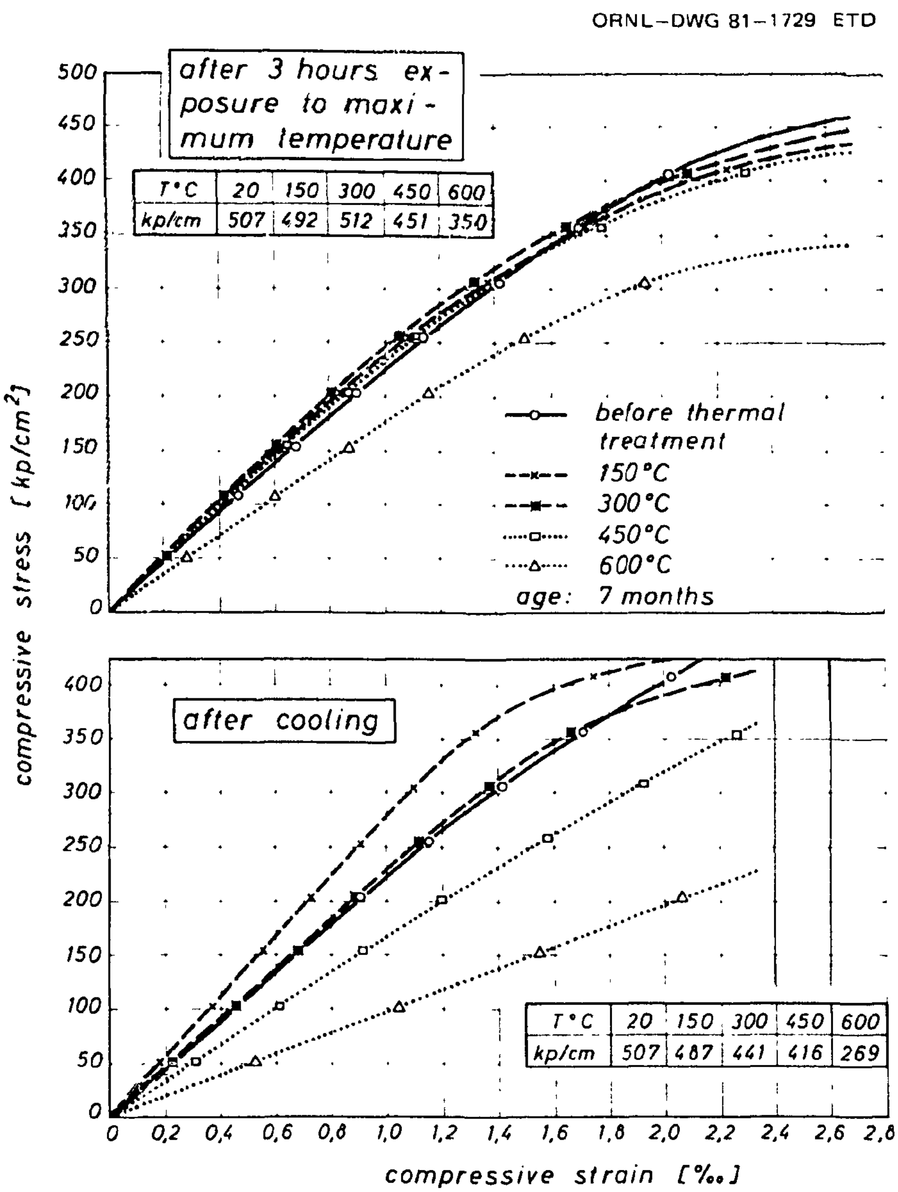

Fig. 19. Effect of elevated temperature on stress-strain behavior of a quartz aggregate concrete. Source: H. Weigler and R. Fischer, "Influence of High Temperatures on Strength and Deformations of Concrete," Paper SP 34-26 in Special Publication SP-34, Vol. I—III, American Concrete Institute, Detroit (1972).

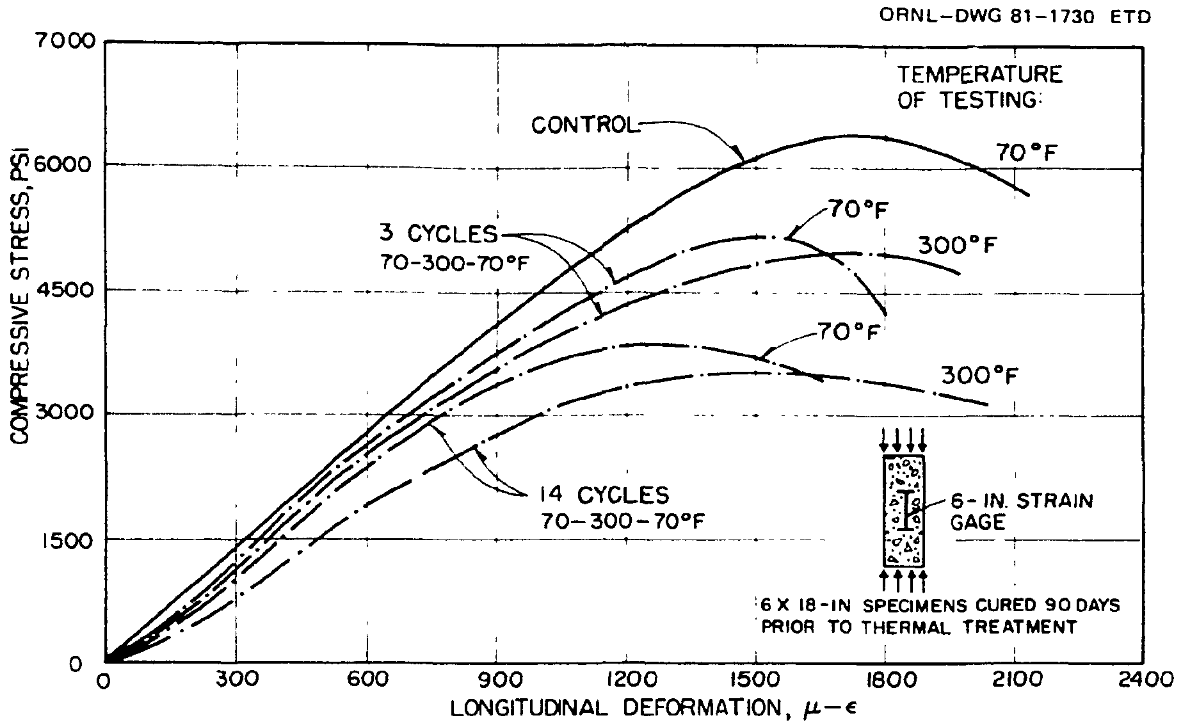

Fig. 20. Effect of thermal cycling on stress-strain behavior of sealed limestone concrete specimens. Source: *THTR-Spannbetonbehälter Bericht Nr. 1 Nachweise für den Gebrouchszustand.*

2.1.7 Multiaxial loading conditions

1. Current practice for multiaxial loading considerations. Once a PCPV has all its prestressing applied, the bulk concrete will always be in a multiaxial state of stress. Regulatory documents of the United States,[1] United Kingdom,[2] France,[3] and Germany[116] permit consideration of multiaxial stress states by permitting use of stresses that are somewhat higher than would be allowable for uniaxial loading conditions. Guidelines provided in each of these codes, however, differ somewhat in detail.

The ASME code[1] uses an enhancement factor C that is applied to the permissible uniaxial compressive stress to account for multiaxial loadings. For this factor to be applicable, however, certain conditions must be met: (1) in a triaxial compression field, the smallest principal stress f_{c3} must be at least 15% of the cylinder strength at the time of test f_{cua} and (2) the maximum principal stress f_{c1} must be less than 300% of the cylinder strength at the time of test. If these conditions are met, Fig. 21 may be used to determine a value for C that generally is conservative. Under triaxial compression loadings where the smallest principal stress is less than 15% of the cylinder strength (essentially a biaxial loading condition) and for triaxial states of stress where biaxial compression is combined with tension, the designer must justify using $C \geq 1$.

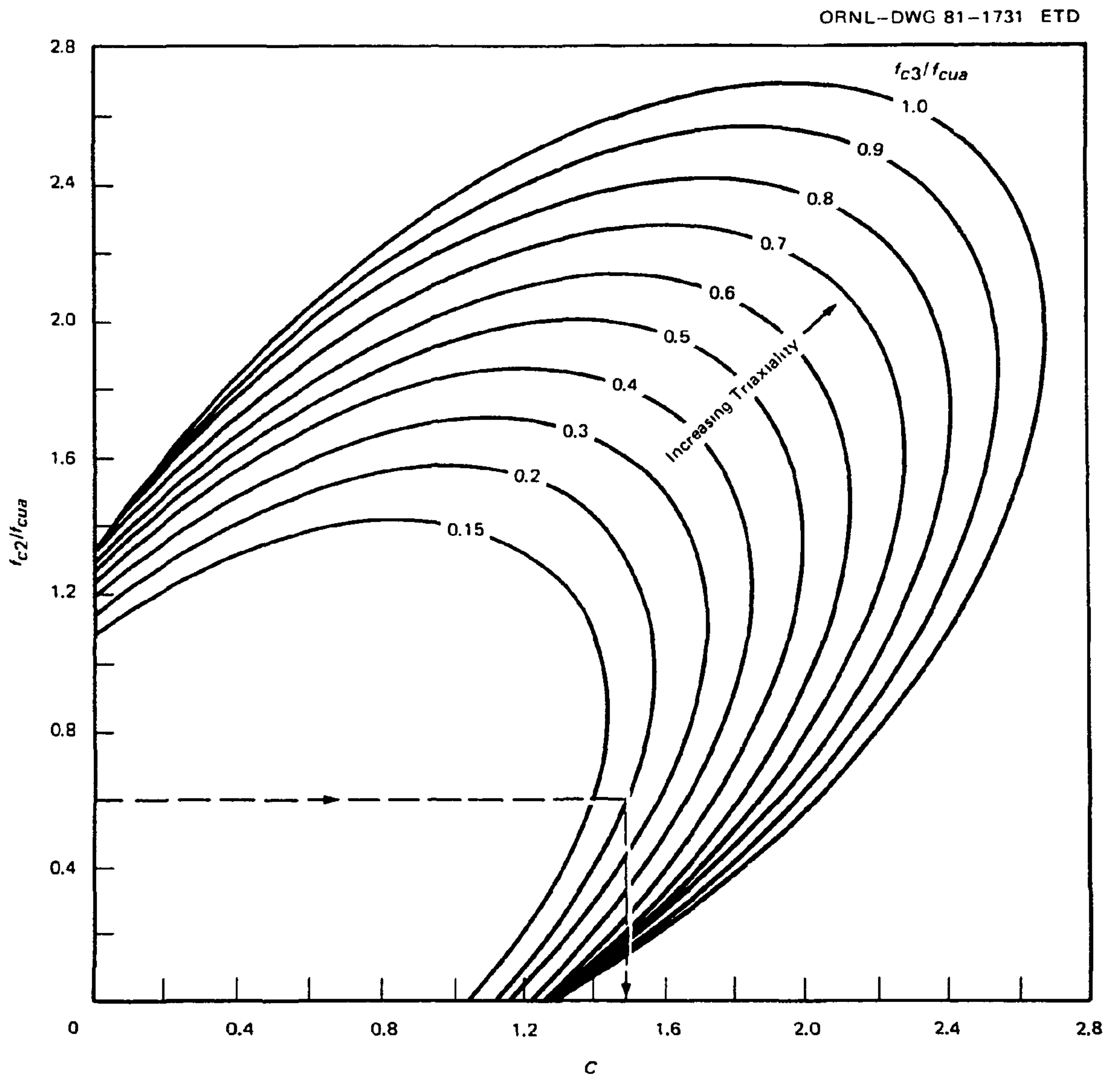

Fig. 21. Enhancement factors C for triaxial compression loadings. Source: "Code for Concrete Reactor Vessels and Containments," Nuclear Power Plant Components, *ASME Boiler and Pressure Vessel Code*, Section III, Division 2 (1975).

The British code[2] permits compressive stresses to exceed the permissible uniaxial value if certain conditions are met: (1) difference between maximum and minimum principal stresses must not exceed one-third the 28-d cube strength U_W or 0.4 U_W for construction loadings, (2) the sum of the three principal stresses must be less than three times the 28-d cube strength, and (3) the smallest of the three principal stresses must be equal to or greater than one-eighth the 28-d cube strength. If these conditions are met, Fig. 22 may be used to check the admissibility of a multiaxial compression condition existing at a point in a structure. As

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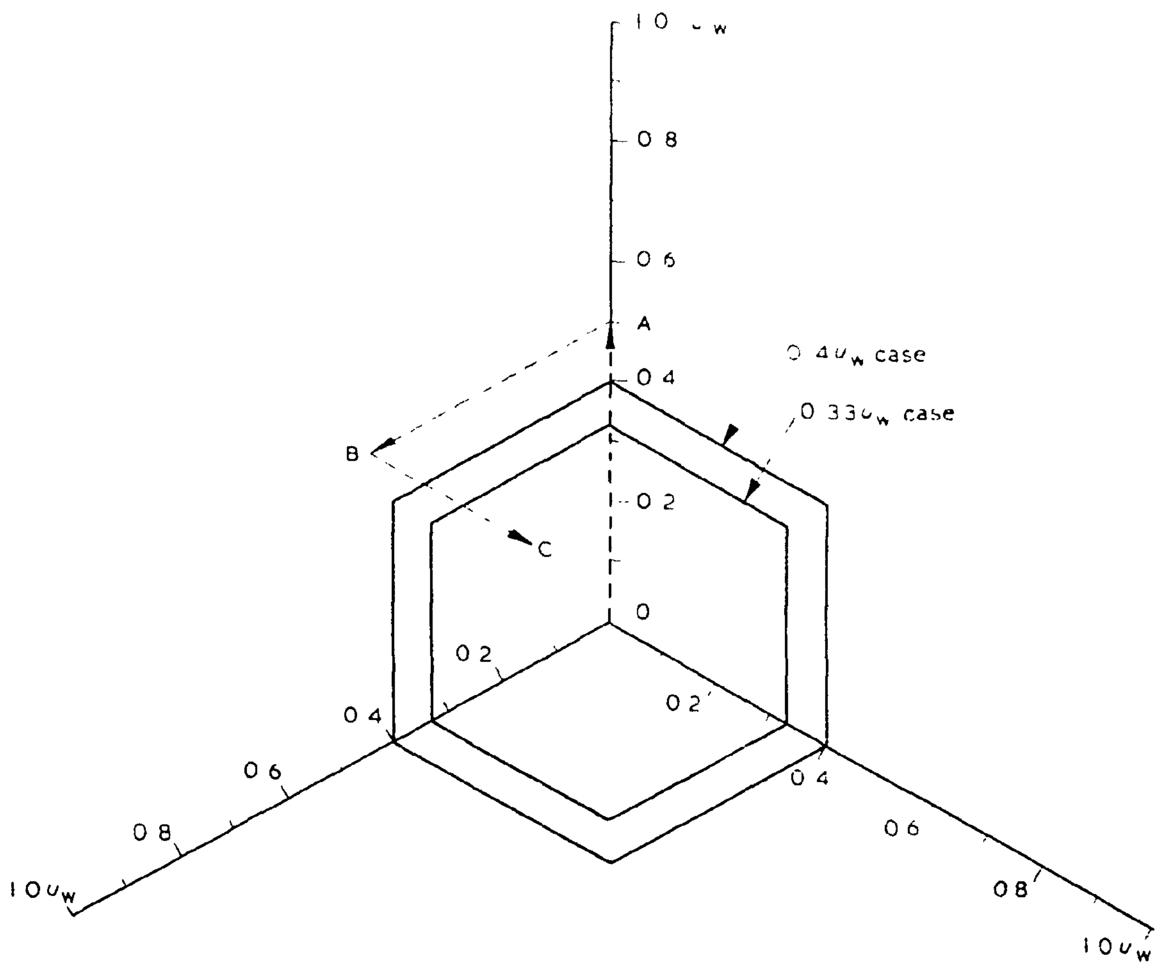

Fig. 22. Design limits for multiaxial compression. Source: British Standards Institution, *Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors*, BS 4975 (July 1973).

noted in Ref. 2, which presents details for use of the figure, the design zone is conservative because of uncertainty in some of the data used to construct the failure surface.

French regulations require that the Mohr's circle drawn for the principal stresses lie within a specified permissible envelope derived by applying a safety factor of 3 to the Mohr-Caquot failure envelope.[3] Since issuance of the above regulation, research conducted at Société d'Etudes de Caissons Nucléaires, Electricite de France, and Commissariat à l'Energie Atomique indicates that stress limitations can be obtained using the lemniscoid model as a basis.[117,118] Parameters of the lemniscoidal surface are not adapted to rupture conditions but to the following criteria: (1) in uniaxial compression, the stress must not exceed one-third the concrete strength; (2) for biaxial loading, the maximum stress must not exceed one-half the uniaxial concrete strength; and (3) for triaxial loading, the maximum principal stress must not exceed twice the uniaxial concrete strength. Figure 23 presents the lemniscate criterion for triaxial compression for a 600-bar uniaxial concrete strength.[118]

The German code[116] permits stresses in the concrete to exceed those permissible for uniaxial conditions when the following conditions are met: (1) all three principal stresses are compressive, and the smallest must be at least 10% of the greatest; (2) the maximum principal stress must not exceed 150% of the uniaxial cylinder strength β_c at 90 d; (3) combination of principal stresses must provide a minimum factor of safety (generally in the range 1.3 to 2.1) against material failure; and (4) if the smallest principal stress is tensile or <10% of the greatest compressive stress, the compressive stress must not exceed the biaxial strength of the concrete divided by the specified safety factors, and, where the stress is tensile, the equivalent tensile force must be carried by reinforcement. The safety factor is assessed using a figure such as presented in Fig. 24 (Ref. 119).

2. Review of available data. Numerous investigations have been conducted relative to the multiaxial behavior and strength of concrete.[28-31,34,36,42-46,120-141] These studies show a great diversity of results that can be attributed to two principal factors: (1) variation of the materials used and (2) variation in the test methods. Differences

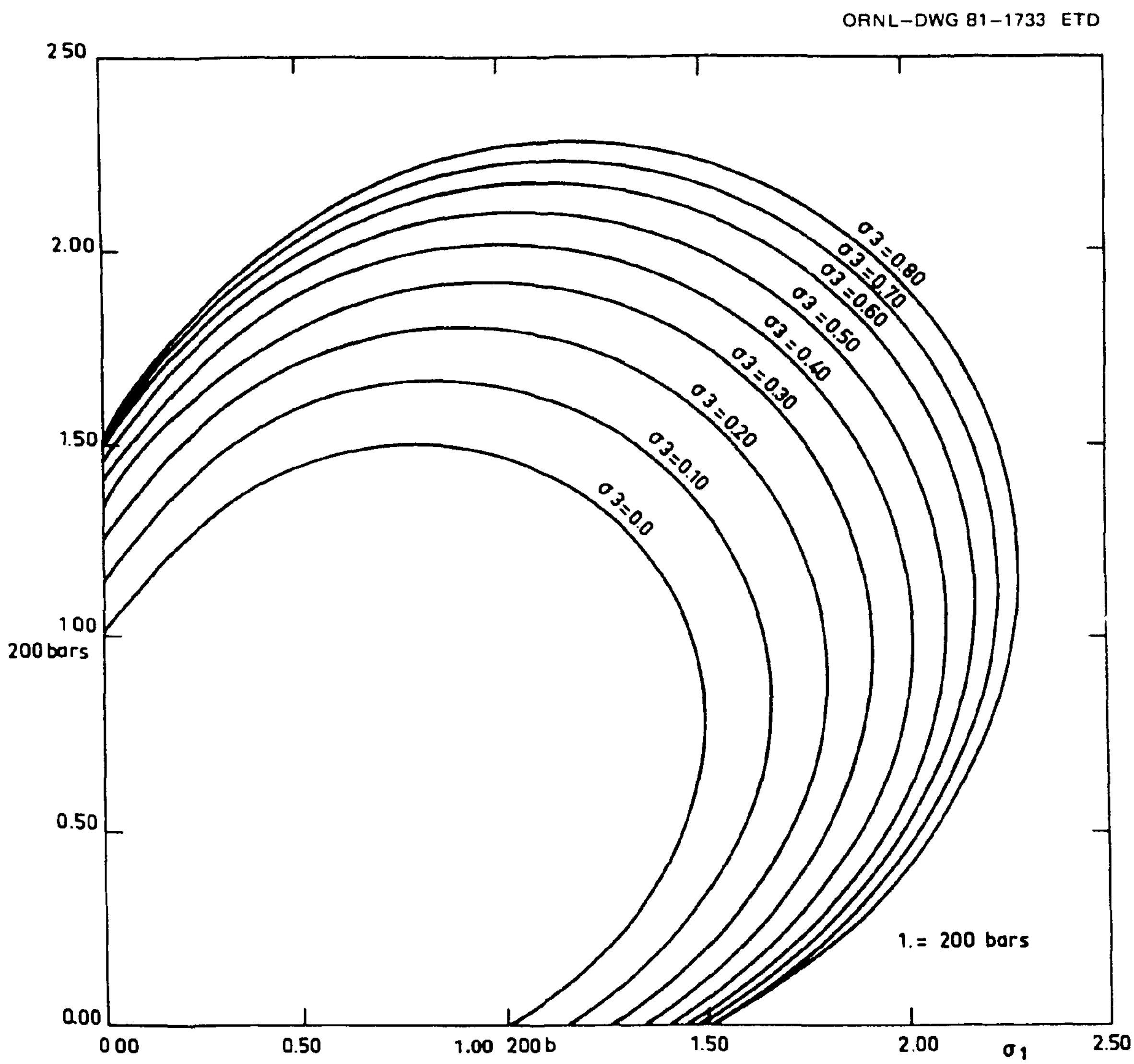

Fig. 23. Lemniscate criterion for triaxial compression of a 600-bar concrete. Source: D. Costes, "Stress Criteria for Nuclear Vessel Concrete," Paper 138/75, Experience in Design, Construction, and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors, University of York, Great Britain (Sept. 8–12, 1975).

among the test methods are predominantly a function of specimen boundary conditions as determined by different loading systems (Fig. 25)[131] and instrumentation techniques. Relative to boundary conditions, at one extreme is a perfectly flexible interface where (1) a stress-controlled boundary condition exists, (2) the specimen is in a state of uniform

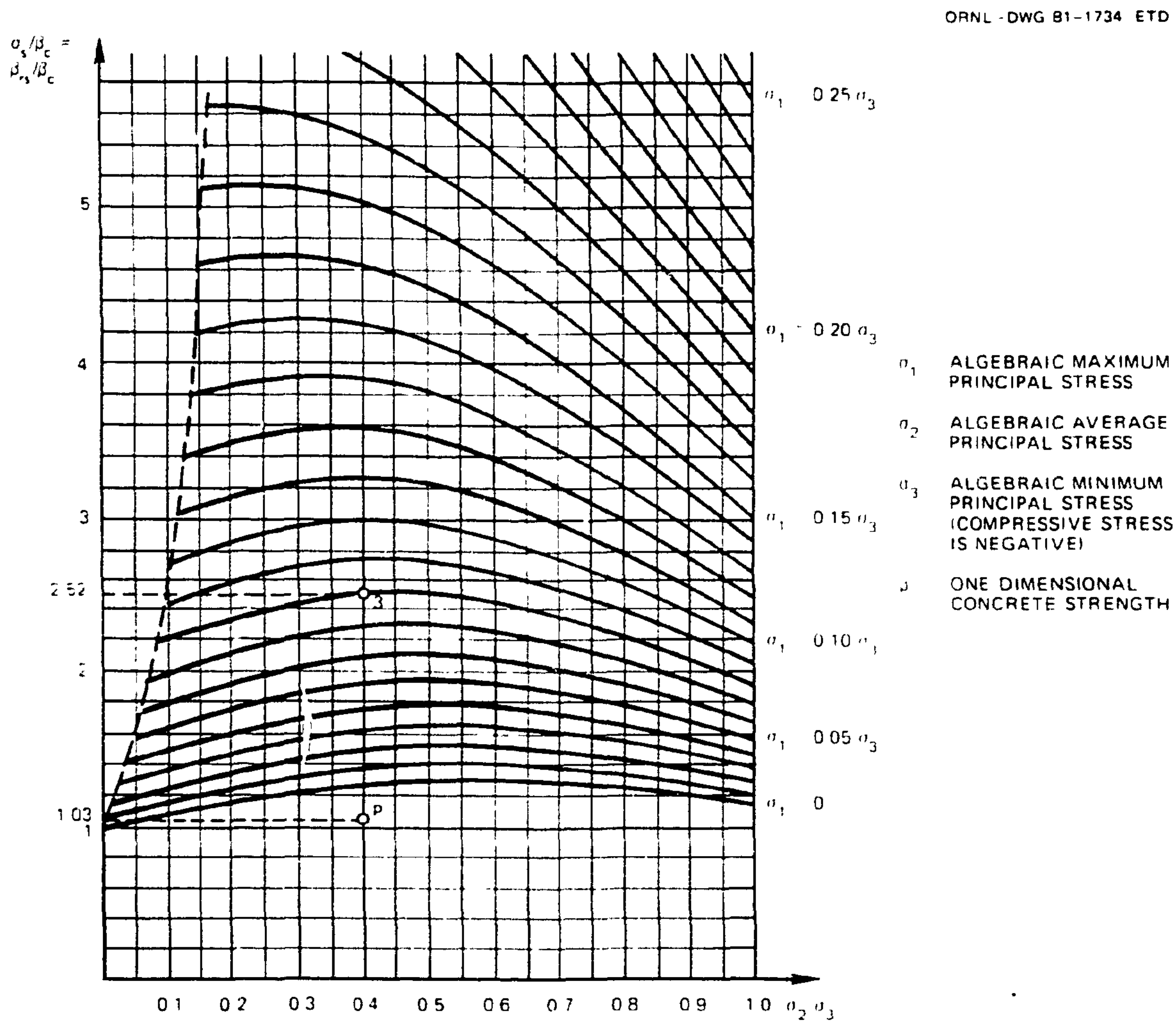

Fig. 24. Safety factors for concrete under multiaxial loading. Source: German Standardization Committee, "Reactor Pressure Vessels Made of Prestressed Concrete. A Study of the Development of a Standard" (DNA) (June 1972).

stress throughout, and (3) failure will be brittle and occur at the weakest link. At the other extreme are rough or unlubricated thick steel plates that transmit specified displacements which may cause (1) nonuniform stresses in the specimen, (2) stress redistribution at local areas of distress, apparently higher strength, and (3) a more ductile failure mode. An example of the effect of the boundary conditions (frictional restraint) on the biaxial strength of concrete is presented in

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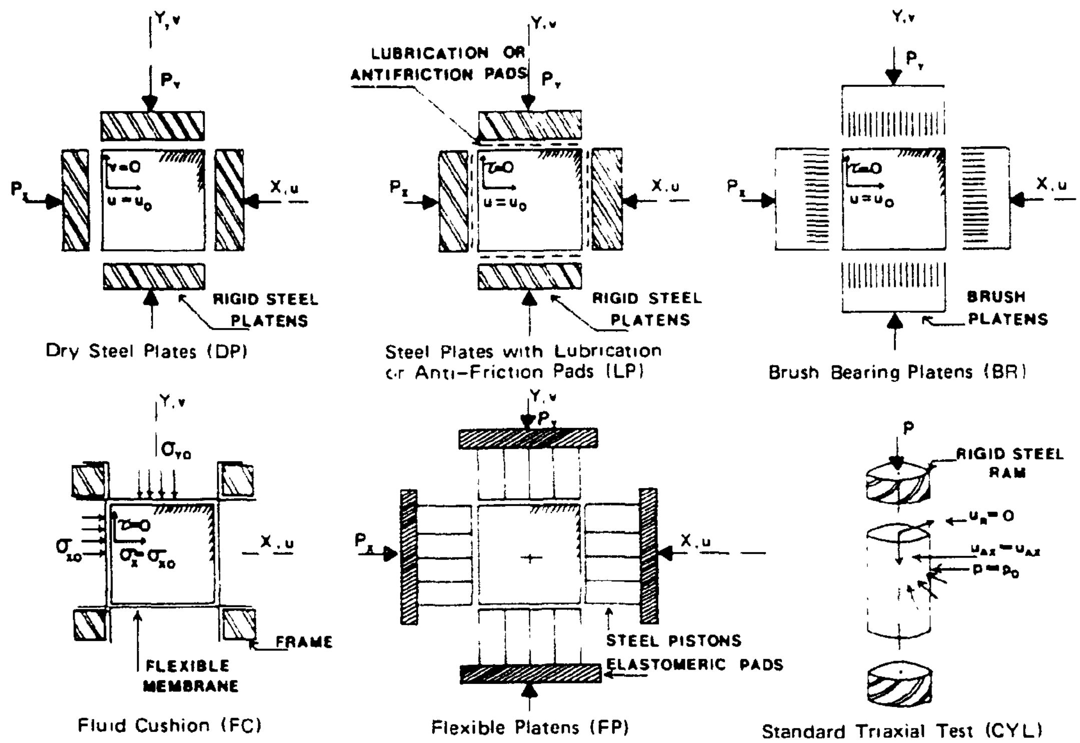

Fig. 25. Boundary conditions for concrete multiaxial testing. Source: H. K. Gerstle et al., "Strength of Concrete Under Multiaxial Stress States," Paper SP 55-5 in Publication SP-55, American Concrete Institute, Detroit (1978).

Table 9.[142]] Several instrumentation techniques involving three basic approaches have been used to make concrete strain measurements: (1) direct surface strain measurements (bonded-wire resistance strain gages, mechanical gages with inductive transducers), (2) length change measurements (mechanical gages with inductive transducers, clip gages containing bonded-wire resistance gages), and (3) deformation measurements (relative platen displacements, brush distortions, gap changes between specimen and fixed reference).

The present state of the art with respect to concrete multiaxial testing is contained in Ref. 131, which presents results of an international program involving researchers from the United States, United Kingdom, Federal Republic of Germany, and Italy. The program was conducted to provide insight into the significance of systematic effects on

Table 9. Effect of frictional restraint on biaxial strength of concrete

Loading conditions	Biaxial strength (σ_c)
Cylinders using wire winding	0.51–0.62
Cylinders using wire winding over 0.015-in. rubber with grease	0.63
Cylinders using wire winding over MGA jacket	1.20–1.50
Cylinders using wire winding over 0.020-in. steel jacket	1.85–2.00
Small slab elements using stiff platens	1.20–1.50
Cylinders using hydraulic pressure	0.20–0.40
Slabs using brush platen	1.15
Slabs and cubes using stiff platen	1.20–1.50

Source: F. K. Garas, "Strength of Concrete Under Different States of Stress," Paper SP 34-18 in Special Publication SP-34, Vol. I-III, Detroit (1972).

concrete multiaxial test results. All participants in the program used specimens from the same concrete and mortar mix that were (1) mixed, cast, and cured at one laboratory; (2) shipped under controlled conditions to the other laboratories; and (3) tested at an identical age. The testing program was divided into two parts: (1) deformations and strength of concrete under biaxial loading, applied monotonically and proportionally, with stress ratios σ_2/σ_1 = 0/3, 1/3, 2/3, 3/3; and (2) deformations and strength of concrete under triaxial loading with stress increased hydrostatically up to one of several specified octahedral shear planes ranging from 75 to 200% of the uniaxial strength and then deviated within that plane along one of three stress paths — triaxial compression, constant intermediate principal stress, or triaxial extension to failure. Figure 26 presents results obtained for strength of concrete under biaxial loading, and Fig. 27 presents triaxial failure envelopes within the octahedral plane σ_0 = 35 kg/cm^2. The following conclusions were derived from

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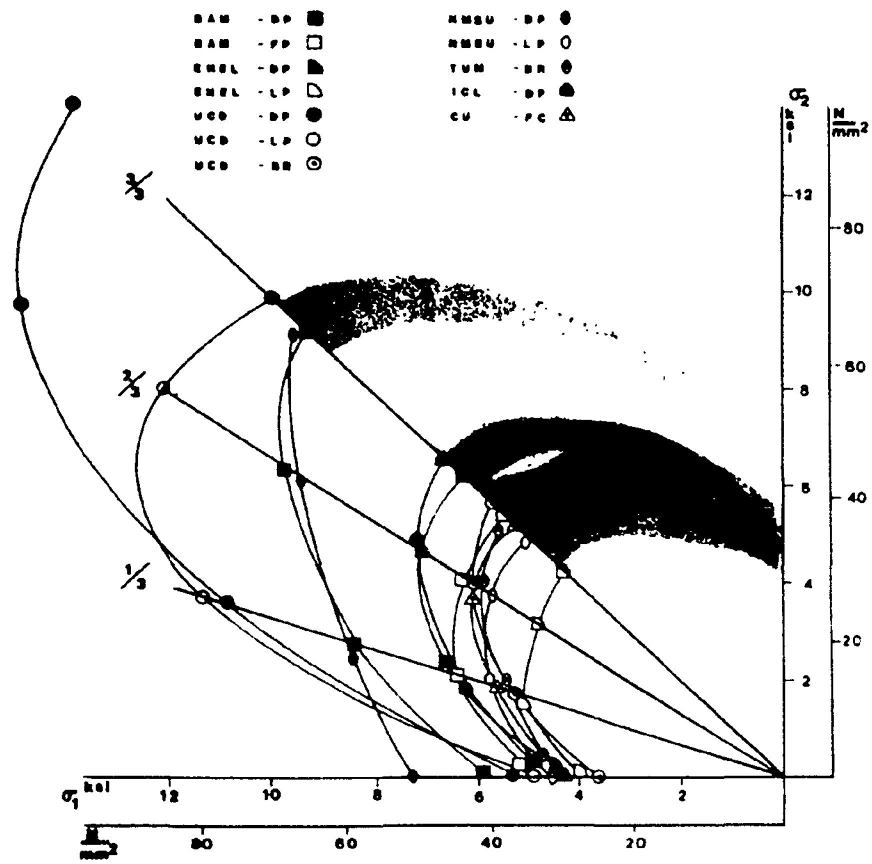

Fig. 26. Strength of concrete under biaxial loading. Source: H. K. Gerstle et al., "Strength of Concrete Under Multiaxial Stress States," Paper SP 55-5 in Publication SP-55, American Concrete Institute, Detroit (1978).

the investigations: (1) a systematic relationship exists between the degree of constraint of the loading system and the uniaxial and multiaxial strengths (differences in strength found in previous investigations at least partially can be ascribed to differences in loading systems) and (2) decomposition of the failure stress state into hydrostatic and deviatoric portions appears to offer a systematic approach to the development of failure criteria for concrete and mortar under multiaxial stresses (establishment of a common failure criterion for uniaxial, biaxial, and triaxial loading conditions that is path-independent seems possible on the basis of test results).

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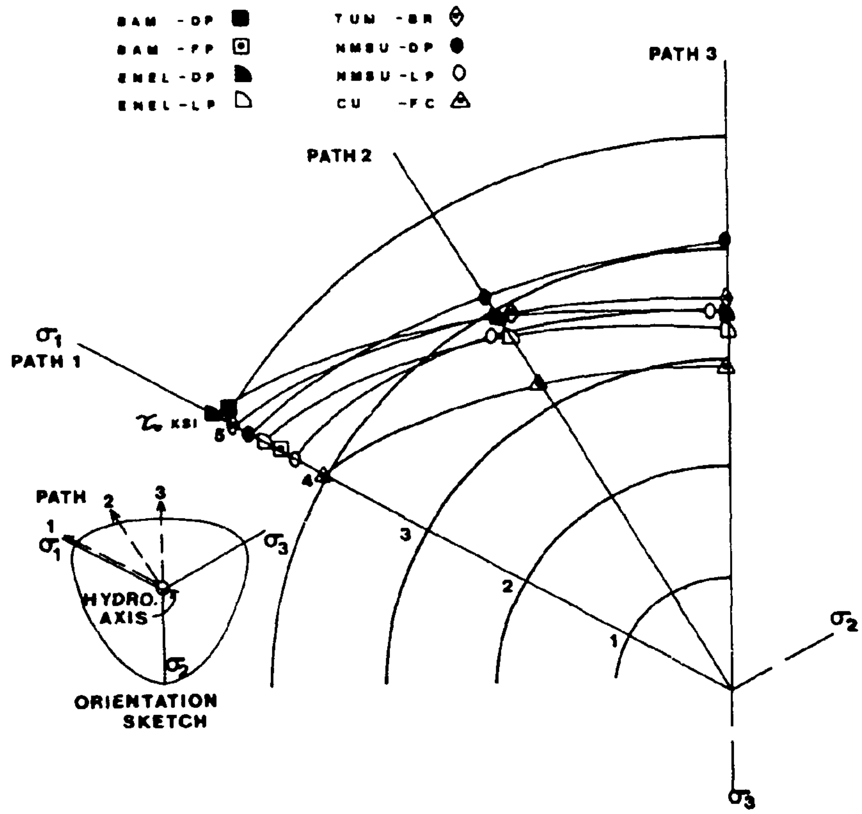


Fig. 27. Triaxial failure envelopes within the octahedral plane σ_0 = 5 ksi = 35 kg/cm^2. Source: H. K. Gerstle et al., "Strength of Concrete Under Multiaxial Stress States," Paper SP 55-5 in Publication SP-55, American Concrete Institute, Detroit (1978).

2.1.8 Irradiation effects

In addition to serving as primary pressure-retaining structures, PCPVs for nuclear applications also serve as biological shields. Concrete has traditionally been used as a shielding material because it (1) attenuates radiation with reasonable thickness requirements, (2) has sufficient mechanical strength, (3) can be constructed in virtually any size and shape at reasonable cost, and (4) requires minimal maintenance. However, irradiation can affect the concrete whether it is in the form of either fast and thermal neutrons emitted by the reactor core or gamma rays produced as a result of capture of neutrons by members (particularly steel) in contact with the concrete. Fast neutrons are mainly responsible for the considerable growth (caused by atomic displacements) that has been measured in the aggregate. Gamma rays produce radiolysis of water in the cement paste that can affect its creep and shrinkage behavior to a limited extent and also result in evolution of gas.

Operation of a reactor over its 30- to 40-year life expectancy may subject the concrete to considerable fast and thermal neutron fluxes. Reference 143 lists the following values for estimates of the maximum radiation to which a PCRV may be exposed after 30 years of service:

thermal neutrons	6×10^{19} neutrons/cm^2
fast neutrons	2×10^{18} to 3×10^{18} neutrons/cm^2
gamma radiation	10^{11} rads

Section III, Division 2 of the ASME Boiler and Pressure Vessel Code[1] gives an allowable radiation exposure level of 10×10^{20} nvt. The British code[2] states that the maximum permissible neutron dose is controlled by the effects of irradiation on the concrete properties, and the effects are considered to be insignificant for doses up to 0.5×10^{18} neutrons/cm^2. Note, however, that these criteria are based on a very limited amount of data; it is not possible to quantify the extent to which irradiation will change the properties of concrete because this is dependent on many factors, such as variation of material properties, material state of testing, neutron energy spectrum, and neutron dose rate.

Several reports have been written on the effects of irradiation on concrete properties.[143-171] The apparent availability of data on irradiation effects on concrete properties is misleading because of technical and experimental difficulties in conducting meaningful tests. Additionally, available data are generally not comparable because (1) different materials were used, (2) mix proportions varied, (3) specimen size was inconsistent, (4) temperatures varied, and (5) cooling and drying conditions were used. Reference 134 presents an excellent summary of experimental data that is available on irradiation effects on concrete properties. Data indicated that (1) for some concretes, neutron radiation of more than 1×10^{19} neutrons/cm^2 may cause some reduction in compressive strength (Fig. 28) and tensile strength (Fig. 29); (2) the decrease of tensile strength caused by neutron radiation is more pronounced than the

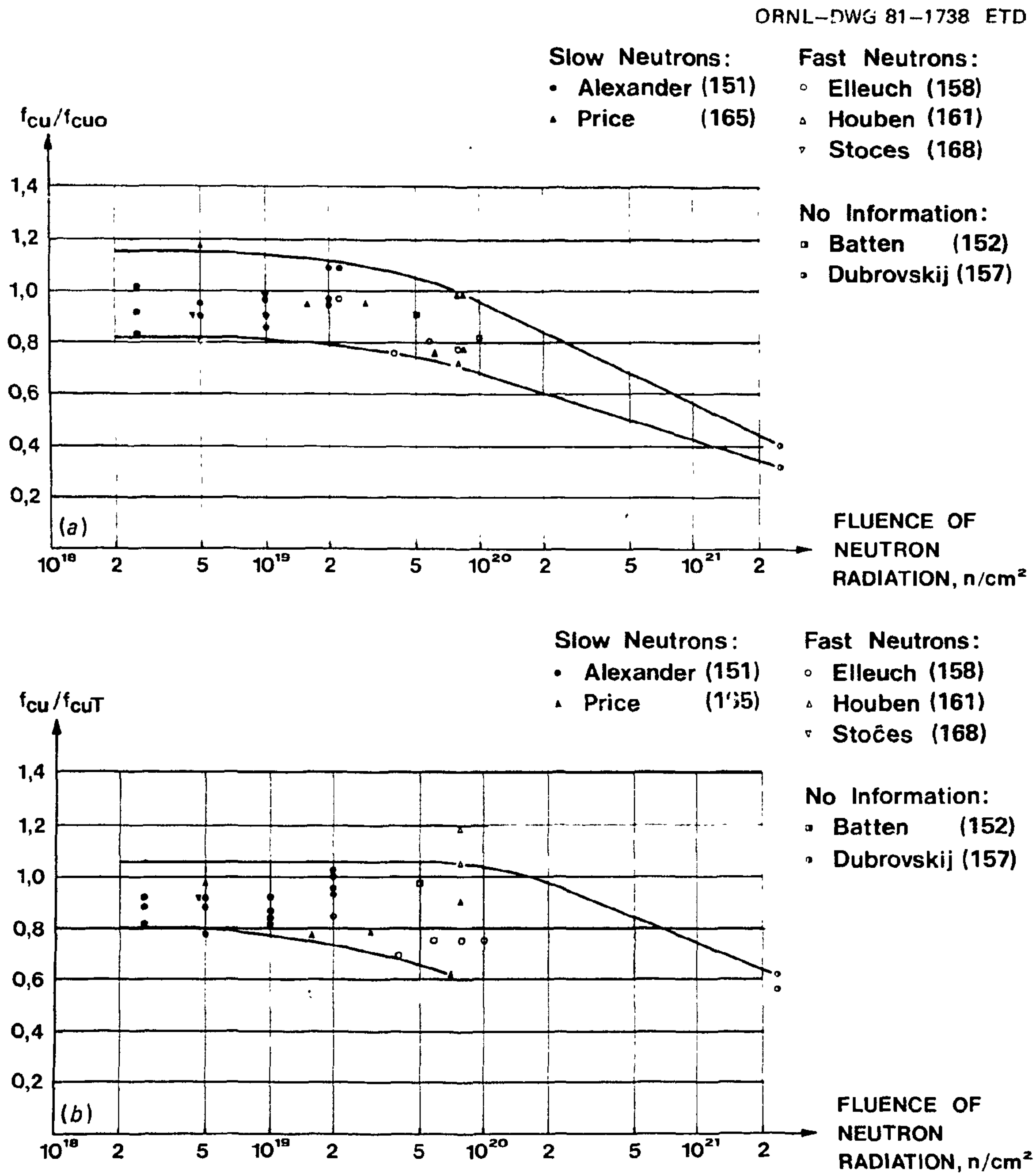
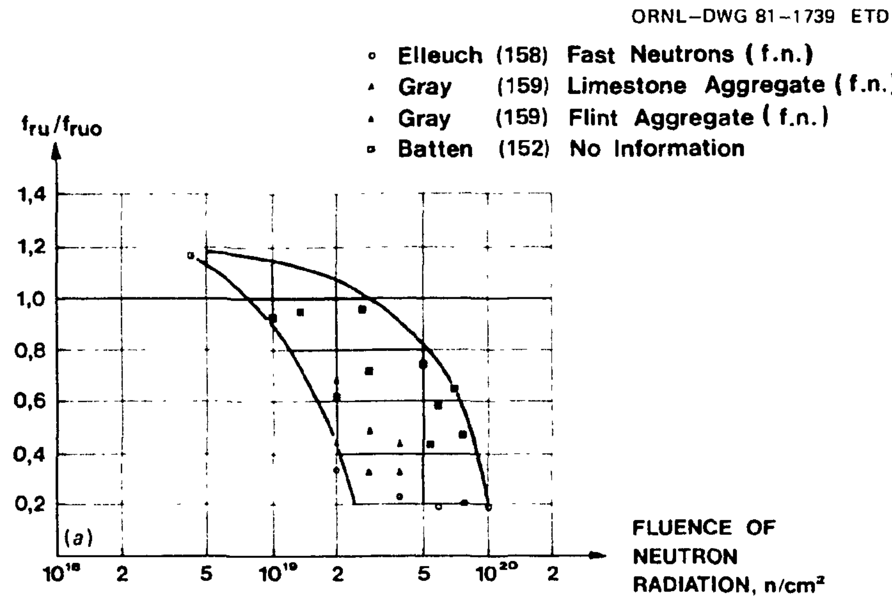


Fig. 28. Compressive strength of concrete exposed to neutron radiation relative to untreated concrete: thermal effects on strength (a) not included, (b) included. Source: P. Bertacchi and R. Bellotti, "Experimental Research on Deformation and Failure of Concrete Under Triaxial Loads," *Proceedings Rilem Symposium on the Deformation and Rupture of Solids Subject to Multiaxial Stresses*, Vol. 1 (1972).



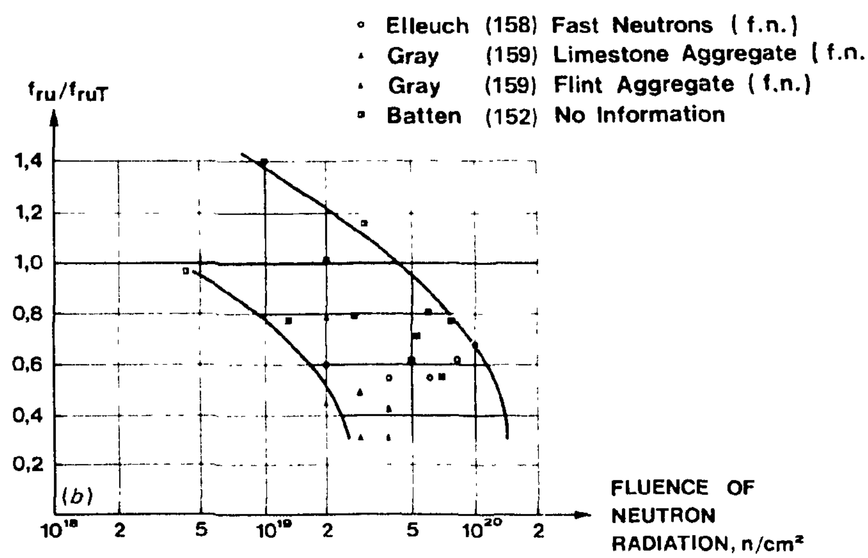


Fig. 29. Tensile strength of concrete exposed to neutron radiation relative to untreated concrete: thermal effects on strength (a) not included, (b) included. Source: P. Bertacchi and R. Bellotti, "Experimental Research on Deformation and Failure of Concrete Under Triaxial Loads," *Proceedings Rilem Symposium on the Deformation and Rupture of Solids Subject to Multiaxial Stresses*, Vol. 1 (1972).

decrease of compressive strength; (3) resistance of concrete to neutron radiation apparently depends on the type of neutrons (slow or fast) involved, but the effect is not clarified; (4) resistance of concrete to neutron radiation depends on mix proportions, type of cement, and type of aggregate (Fig. 30); (5) the effect of gamma radiation on concrete's mechanical properties requires clarification; (6) the deterioration of concrete properties associated with a temperature rise resulting from irradiation is relatively minor; (7) coefficients of thermal expansion and conductivity of irradiated concrete differ little from those of temperature-exposed concrete; (8) when exposed to neutron irradiation, the modulus of elasticity of concrete decreases with increasing neutron fluence (Fig. 31); (9) creep of concrete is not affected by low-level radiation exposure, but for high levels of exposure creep probably would increase with exposure because of the effects of irradiation on the concrete's tensile and compressive strengths; (10) for some concretes, neutron radiation with a fluence of more than 1×10^{19} neutrons/cm^2 can cause

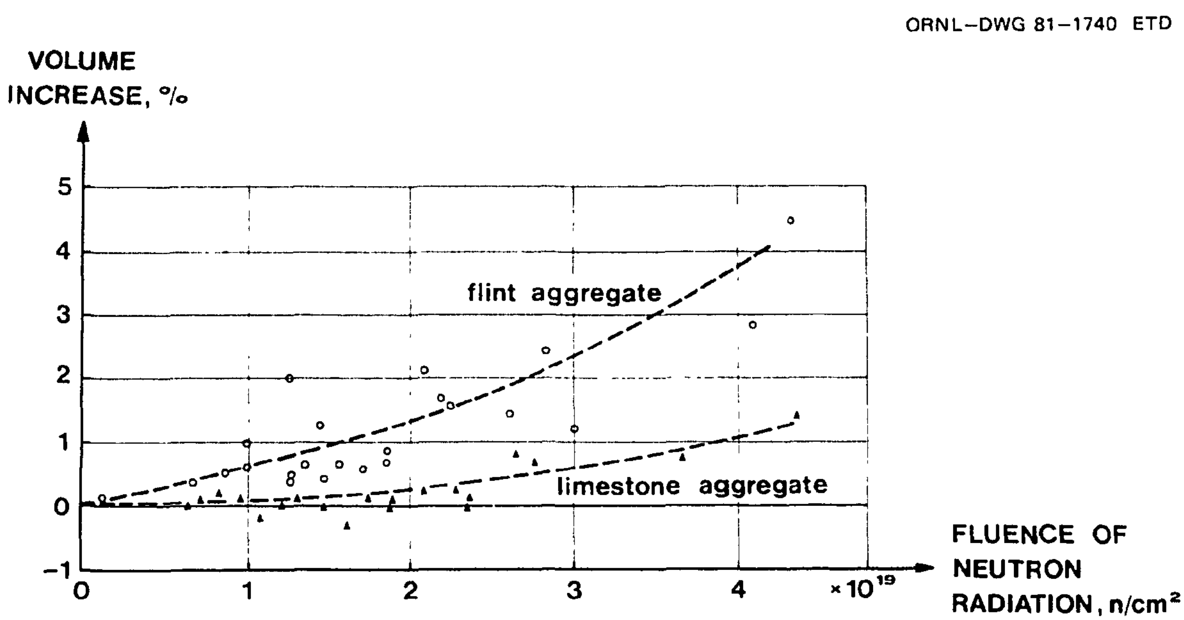


Fig. 30. Effect of fast neutron exposure on volume change of flint aggregate and limestone aggregate concretes. Source: P. Bertacchi and R. Bellotti, "Experimental Research on Deformation and Failure of Concrete Under Triaxial Loads," *Proceedings Rilem Symposium on the Deformation and Rupture of Solids Subject to Multiaxial Stresses*, Vol. 1 (1972).

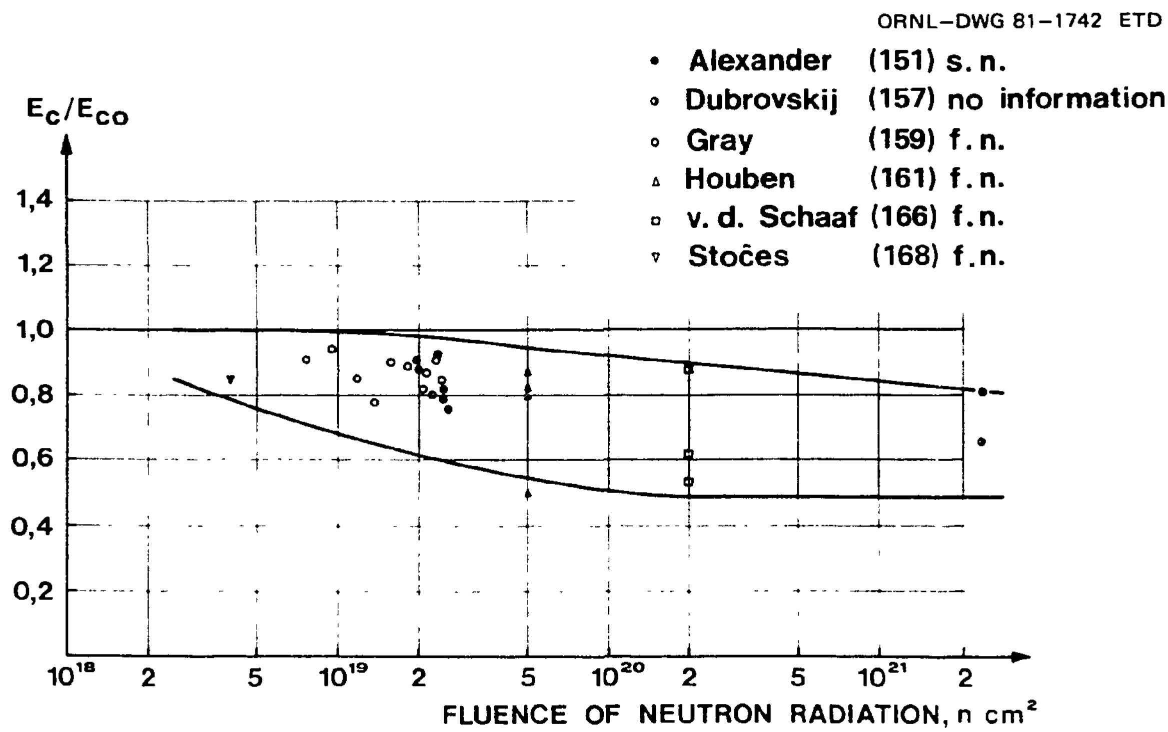


Fig. 31. Modulus of elasticity of concrete exposed to neutron radiation relative to untreated concrete: thermal effects on modulus not included. Source: P. Bertacchi and R. Bellotti, "Experimental Research on Deformation and Failure of Concrete Under Triaxial Loads," *Proceedings Rilem Symposium on the Deformation and Rupture of Solids Subject to Multiaxial Stresses*, Vol. 1 (1972).

a marked increase in volume; (11) generally, concrete's irradiation resistance increases as the irradiation resistance of the aggregate increases; and (12) irradiation has little effect on shielding properties of concrete beyond moisture loss caused by a temperature increase.

2.2 Specialty Concretes for Application to PCPVs

Concrete containment structures have traditionally been massive, thick-walled, flat-headed, essentially right circular cylinders fabricated from conventional reinforced-concrete material systems. Properties of the system (e.g., low tensile and shear stre·.gths, decreasing strength and modulus of elasticity with temperature, and creep) have led to designs that have produced massive, costly containment structures.

Recently, several innovative concrete material systems have been developed to the stage that they may be potentially cost- and performance-effective for application to PCPVs. Included in these material systems are high-strength concretes, fiber-reinforced concrete, polymer concrete, lightweight-insulating concrete, and high-temperature concrete.

2.2.1 High-strength concrete

1. Definition. High-strength concrete is a relative term, but for purposes of definition it will be considered as concrete with compressive strengths in excess of 41.4 MPa (6000 psi) for normal-weight aggregates and 27.6 MPa (4000 psi) for lightweight aggregates.[172] Concrete with compressive strengths in excess of 172.4 MPa (25,000 psi) are attainable but require special fabrication procedures such as drying the concrete's capillary pores and filling with a solid polymeric material. The practical and economical strength limit of ready-mixed concrete appears to be on the order of 75.8 MPa (11,000 psi) for normal-weight aggregate concrete and 55.2 MPa (8,000 psi) for lightweight aggregate concrete.[173]

2. Requirements for producing high-strength concretes. Optimization of the following factors is required to achieve high-strength concrete: (1) characteristics of the cementing medium; (2) characteristics of the aggregate; (3) proportions of the paste; (4) paste-aggregate interaction; (5) mixing, consolidating, and curing; and (6) testing procedures. Compressive strength results obtained from specimens cast from a series of trial mixes that have a constant slump between 76 and 102 mm, cement contents from 7 to 10 sacks/yd^3, and different cement brands can be used to identify an optimum performing cement brand and cement content. (Reference 174 indicates that the choice of brand and type of cement is probably the most important factor in the selection of materials for high-strength concrete.) Sands with a fineness modulus of ~3.0 appear to provide the best workability and highest compressive strengths. The strength of concrete for rich mixes tends to increase with a decrease in maximum aggregate size[175] [Fig. 32 (Ref. 176)]; a 12.7-mm maximum aggregate size performs the best in many cases.[173] The bond between the cement paste and aggregate increases as the particles become rougher and more angular.

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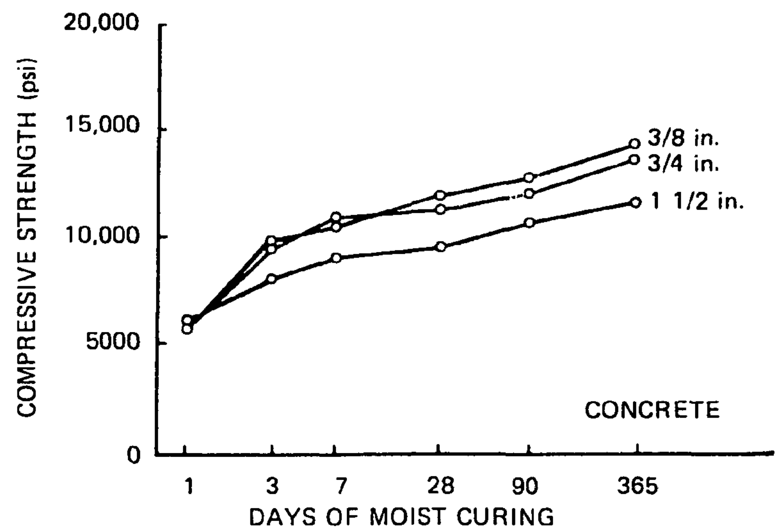

Fig. 32. Effect of maximum particle size of aggregate on compressive strength. Source: S. Walker and D. L. Bloem, "Effects of Aggregate Size on Properties of Concrete," *Proc. J. Am Concr. Inst.* 32(3), 283–98, Detroit (September 1960).

Crushed rocks such as trap-rock (basalt or diabase), limestone, quartzite, or granite are suitable for high-strength concretes (Fig. 33).[176] Water-reducing retarders, both conventional and extended range [conforming to the requirements of ANSI/ASTM C 494-79 (Ref. 14)], should be used to reduce the water-cement ratio, but this should not be done without first evaluating them under construction conditions (temperature, humidity) for compatibility with other materials. (Reference 177 recommends addition of water reducers after all cement has come in contact with initial mixing water.) Pozzolanic materials such as flyash that conform to ANSI/ASTM C 618-78 (Ref. 15) and have an ignition loss <3% should be used to provide increased strength at later ages and to reduce cement requirements and the associated heat of hydration. To obtain high-strength concretes, it is necessary to use the lowest possible water-cement ratio (0.30 to 0.40) and high cement factors (>7 sacks/yd^3) when proportioning the mix. (After material selection, the water-cement ratio is the most important factor affecting high-strength concrete producibility.) Ready-mix plants, which would be required for producing concrete for mass construction, provide sufficient control and efficiency for producing high-strength concrete

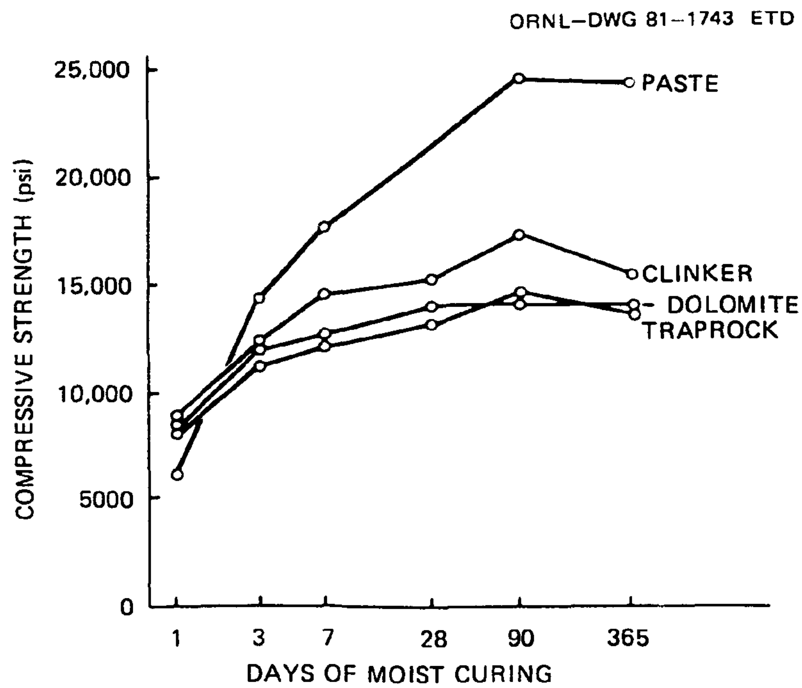

Fig. 33. Effect of coarse aggregate type on compressive strength. Source: S. Walker and D. L. Bloem, "Effects of Aggregate Size on Properties of Concrete," *Proc. J. Am. Concr. Inst.* 32(3), 283–98, Detroit (September 1960).

mixes, but the ability to control the production operation is greatly dependent on the personnel. On delivery from the central mix facility, the concrete should be rapidly discharged and completely consolidated using high-frequency vibrators to obtain high strength (Fig. 34).[178] Curing techniques, such as the use of membrane-curing compounds and possibly the use of aggregates in a saturated condition, should be used to ensure that the internal relative humidity of the concrete does not drop below 80% to halt hydration. Control specimens should be fabricated, cured, and tested using recommended ANSI/ASTM procedures.[58–61,63–65] Finally, a comprehensive quality control program is required at both the central mix facility and site to guarantee consistent production and placement of high-strength concrete. Table 10 presents a mix design used to produce a nominal 75.8-MPa (11,000-psi) concrete.[174]

3. Properties of high-strength concretes. In the design of concrete structures, it is important to know the material's stress-strain behavior

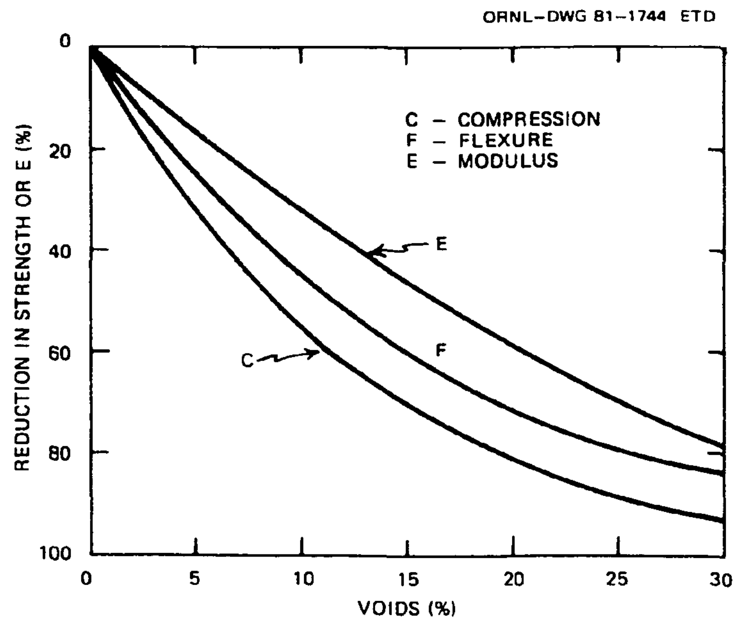

Fig. 34. Effect of voids content on concrete strength. Source: W. F. Perenchio, *An Evaluation of Some of the Factors Involved in Producing Very High-Strength Concrete*, Research and Development Bulletin RD014, Portland Cement Association, Skokie, Ill. (1973).

Table 10. Mix designs for a nominal 75.8 MPa (11,000 psi) concrete

Material	Quantity/yd^3	
	Water Tower Place	River Plaza
Cement, lb (kg)	846 (384)	850 (386)
Fine aggregrate, lb (kg)	1025 (465)	1040 (472)
5/8-in. stone, lb (kg)	1800 (816)	
1/2-in. stone, lb (kg)		1730 (785)
Water, lb (kg)	300 (136)	330 (150)
Pozzolith 100XR, fl. oz. (ml)	25.4 (751)	43.0 (1270)
Fly ash, lb (kg)	100 (45)	100 (45)
Slump, in. (mm)	4-1/2 (114)	4-1/2 (114)
Air content, %		1.5
Unit weight, lb/ft^3 (kg/m^3)	151.9 (2433)	148.8 (2384)

Source: Chicago Committee on High-Rise Buildings, *High-Strength Concrete in Chicago High-Rise Buildings*, Report No. 5, Chicago, Ill. (February 1977).

(including the descending portion of the curve), the modulus of elasticity, thermal properties, adiabatic temperature rise, drying shrinkage, and creep and bond to steel. However, only a limited amount of information is available for high-strength concretes.

The effects of strength on the stress-strain behavior of normal-weight aggregate concretes having compressive strengths from ~41.4 to 96.6 MPa (6,000 to 14,000 psi) and lightweight aggregate concretes having compressive strengths from ~27.6 to 82.7 MPa (4,000 to 12,000 psi) are presented in Fig. 35 (Ref. 179) and in Fig. 36 for concrete strengths to 93.1 MPa (13,500 psi).[180] Moduli-of-elasticity values obtained from these

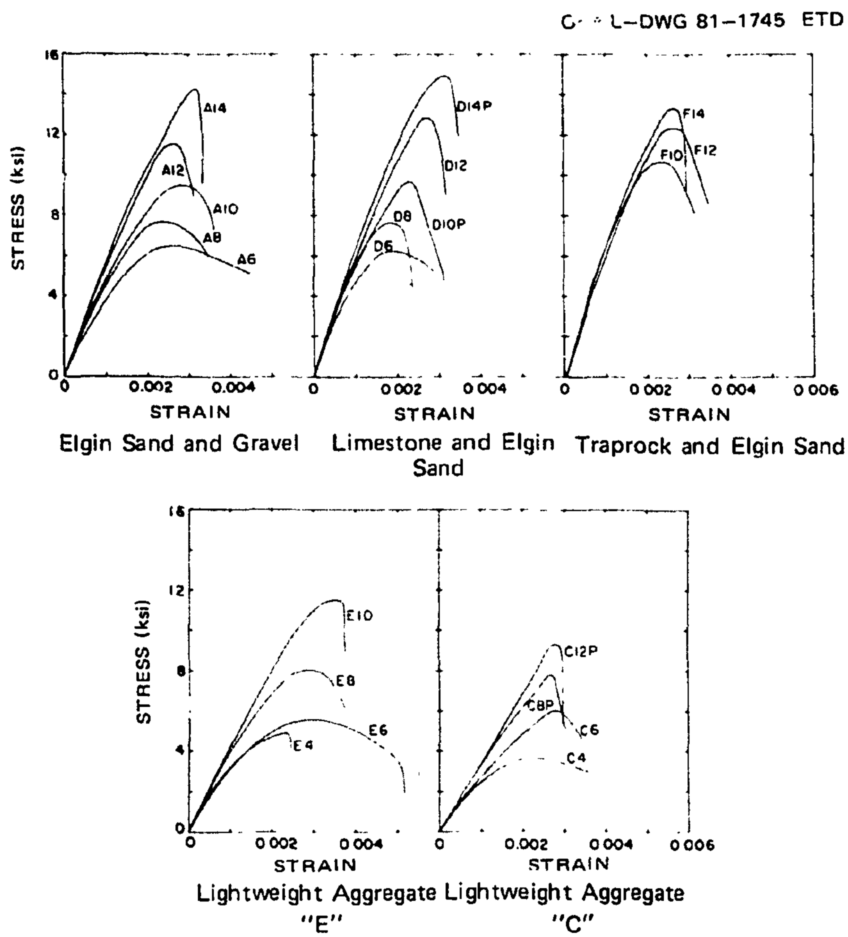

Fig. 35. Stress-strain curves for normal-weight and lightweight aggregate high-strength concretes. Source: V. H. Dodson and E. Farkas, "Delayed Addition of Set Retarding Admixtures to Portland Cement Concrete," *ASTM Proc.* 64, 816–29.

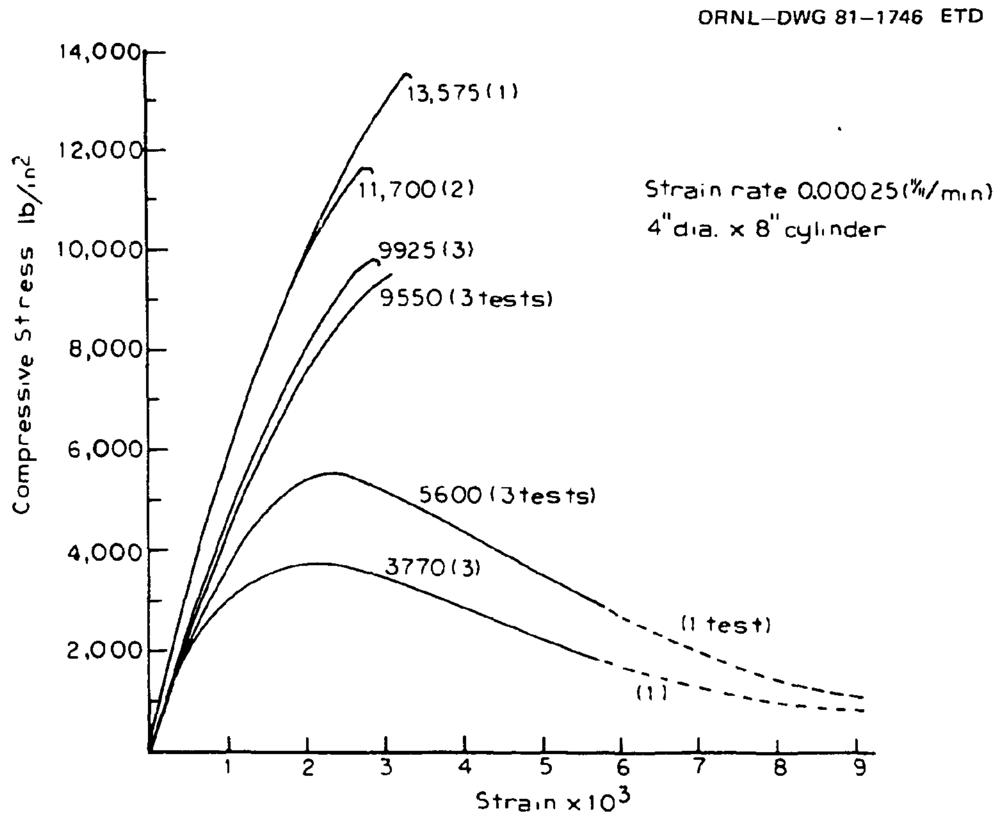

Fig. 36. High-strength concrete stress-strain curves. Source: P. H. Kaar et al., "Stress-Strain Characteristics of High-Strength Concrete," Paper SP 55-7 in Publication SP-55, American Concrete Institute, Detroit (1978).

tests showed a tendency to increase as the compressive strength increased. The concrete ductility has a tendency to decrease as the strength increases, and the maximum strain at failure in compression is lower at higher concrete strengths. (The maximum ultimate strain for design at the extreme compression fiber may be less than the 0.003 used for normal-strength concrete design.[173])

Thermal properties (conductivity, diffusivity, specific heat, and coefficient of expansion) are within the common range for conventional-strength concretes.[181-184] Conductivity and diffusivity are both affected by the type of aggregate, moisture condition, and unit weight; however, water-cement ratio and strength appear to have little effect. Diffusivity of concrete normally varies between 5.2 and 20.6 × 10^{-7} m^2/s (0.02 and 0.08 ft^2/h) (Ref. 173). Specific heat ranges from 837 to 1170 J/kg·K

(0.02 to 0.28 Btu/lb-°F) and is affected by temperature variation and concrete moisture content.[173] Concrete's thermal expansion and contraction is affected by aggregate type, richness of mix, water-cement ratio, temperature range, concrete age, and degree of concrete saturation; it ranges from 9×10^{-6} to 12.6×10^{-6}/°C (5×10^{-6} to 7×10^{-6}/°F) for siliceous aggregate concretes and 6.3×10^{-6} to 9×10^{-6}/°C (3.5×10^{-6} to 5×10^{-6}/°F) for limestone or calcareous aggregate concretes.[173] Heat generation of concrete depends on the heat of hydration of the cement, water-cement ratio, and cement content; the heat rise of high-strength concretes will generally be 11 to 15°F per 100 lb of cement per yd^3 (Ref. 181). (Recorded temperatures in 4-ft^2 9000-psi columns reached a maximum of 140 to 150°F between 18 and 20 h after casting.[174])

Creep and shrinkage of concrete are related phenomena and are controlled by similar parameters: water-cement ratio, aggregate characteristics and content, age of concrete at first exposure to load or drying, size and shape of member, steel reinforcement content, and environmental exposure. Limited data are available on the creep response of high-strength concrete. Reference 185, however, presents some general observations: (1) because creep is proportional to water-cement ratio and high-strength concretes have lower water-cement ratio, they should have less creep, and (2) high-aggregate contents reduce creep at working stress levels.

Bond of concrete to steel increases with an increase in compressive strength of concrete. The bond strengths developed by high-strength concretes will therefore be above the allowable average bond stress (Ref. 186) for many of the reinforcing bar sizes.[187,188]

4. Research requirements. Because the use of high-strength concretes for structural applications is a relatively recent development, a number of research requirements exist: (1) obtain information on mechanical characteristics of concrete and determine how they are related to the properties of the matrix, aggregates, and interface; (2) develop theoretical models to predict composite behavior based on constituent behavior (micro, macro, continuum, and fracture mechanics); (3) determine mechanical characteristics (under monotonically increasing and cyclic loading) of confined high-strength and lightweight concrete and formulate

models to predict constitutive behavior; (4) improve knowledge about mechanical behavior of structural elements made with high-strength concrete subjected to static as well as earthquake-type loadings; (5) examine the applicability of current code methods (e.g., Ref. 186) for predicting limiting amount of tensile steel, load-moment diagram, shear strength, minimum hoop reinforcement, moment redistribution for continuous beams, and serviceability (deflection and cracking); and (6) analyze relationships between different kinds of cementitious materials, products, and rate of heat evolution.[172]

2.2.2 Fibrous concrete

1. Definition. Fiber-reinforced concrete is made of hydraulic cements containing fine or fine and coarse aggregates and discontinuous discrete fibers.[189] Table 11 presents typical fibers and properties of the fibers used to produce fibrous concrete. As noted, several types of fibers are available; however, steel fibers are used primarily to produce fibrous concrete. Available in a variety of lengths, diameters, and configurations, steel fibers are generally produced from carbon steel wire or a melt extraction process. Alloy combinations may be used where high temperatures and corrosion are factors.

2. Batching, mixing, and placement. Dispersion of the fibers throughout the mix is very important. Two possible problem areas experienced during mixing of fibrous concrete are balling and segregation of the fibers. Factors affecting balling or fiber segregation are: (1) fiber-aspect ratio (ratio of length to equivalent diameter); (2) volume percentage of fiber; (3) coarse aggregate size, gradation, and quantity; (4) water-cement ratio; and (5) method of mixing. Increases in aspect ratio, volume percentage, and size and quantity of coarse aggregate intensify balling tendencies. Generally, an aspect ratio of $\leq$100, steel fiber contents of $\leq$2% by volume,* and use of a maximum aggregate of $\leq$9.5 mm (3/8 in.) produce good results. Because plasticity of the mix is

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*Specially shaped fibers that have hooked ends and longer lengths have recently come on the market; they can be proportioned into mixes at ~60% the rate for straight fibers and still provide equal or improved properties.[190]

Table 11. Typical properties of fibers used to produce fibrous concrete

Type of fiber	Tensile strength [ksi (MPa)][a]	Young's modulus [10^3 ksi (GPa)]	Ultimate elongation (%)	Specific gravity
Acrylic	30–60 (207–414)	0.3 (2.1)	25–45	1.1
Asbestos	80–140 (552–1034)	12–20 (83–138)	~0.6	3.2
Cotton	60–100 (414–689)	0.7 (4.8)	3–10	1.5
Glass	150–550 (1034–3792)	10 (69)	1.5–3.5	2.5
Nylon (high tenacity)	110–120 (758–827)	0.6 (4.1)	16–20	1.1
Polyester (high tenacity)	105–125 (724–862)	1.2 (8.3)	11–13	1.4
Polyethylene	~100 (~689)	0.02–0.06 (0.1–0.4)	~10	0.95
Polypropylene	80–110 (552–758)	0.5 (3.4)	~25	0.90
Rayon (high tenacity)	60–90 (414–621)	1.0 (6.9)	10–25	1.5
Rock wool (Scandinavian)	70–110 (483–758)	10–17 (69–117)	~0.6	2.7
Steel	40–600 (276–4137)	29 (200)	0.5–35	7.8

[a] 1 ksi = 70.31 kgf/cm^2.

Source: ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete," ACI SP-44 *Fiber Reinforced Concrete*, American Concrete Institute, Detroit (1974).

important for ensuring proper fiber distribution, experience has shown that good results are achieved for water-cement ratios between 0.4 and 0.6 and cement contents between 6 and 10 sacks/yd^3 (Ref. 189). Use of conventional admixtures for air entrainment, water reduction, and shrinkage control, as well as the use of flyash to reduce cement content, is recommended. Proportioning of a fibrous concrete mix is similar to that used for conventional concrete mixes, except that fine aggregate contents for fibrous concrete typically are 45 to 55% of the total aggregate content. Reference 191 presents a mix design method for fibrous concrete, and Tables 12 and 13 (Ref. 189) present typical mix proportions for fibrous concrete mixes with and without flyash.

Several procedures are available for providing laboratory or quantity steel fiber mixes using batch plant or ready-mixed concrete trucks: (1) blend fiber and aggregate before charging the mixer, followed by standard mixing procedures; (2) blend fine and coarse aggregates in mixer, add fibers at mixing speed, add cement and water simultaneously, or add cement followed by water and additives; (3) add fibers to previously charged aggregates and water, add cement and remaining water; and (4) add all blended dry ingredients to mixer that has previously been charged with water. Information on mixing large batches of synthetic fiber fibrous concrete is limited. In the laboratory, the fibers can be added in small amounts to a rotating drum mixer that has been charged with the cement, aggregates, and water. For large batches, satisfactory mixes can be achieved by: (1) dispersing the fibers throughout the fine and coarse aggregates in a weight hopper, charging the mixer by conveyor belt, and adding the cement and water last; and (2) following conventional batch plant or truck ready-mixed concrete procedures by adding the glass fibers last by dumping them directly or blowing them into the truck. Properties of fibrous concrete (air content, unit weight, strength, and modulus of elasticity) are determined according to Ref. 192.

Transporting and placing fibrous concrete can be accomplished with most conventional equipment of good design and in clean condition. Recommendations are that transit trucks not be loaded to capacity because the fibrous concrete mix is relatively stiff, and it requires slightly more

Table 12. Typical mix proportions for fibrous concrete mixes with fly ash

Cement, lb/yd^3[a] (kg/m^3)	490 (291)
Fly ash, lb/yd^3 (kg/m^3)	225 (133)
W/C ratio	0.54
Percentage of sand to aggregate	50
Maximum size coarse aggregate, in. (mm)[b]	3/8 (9.5)
Steel fiber content (0.010 × 0.022 × 1.0 in.) (0.25 × 0.36 × 25.4 mm), % by volume	1.5
Air-entraining agent	Manufacturer's recommendations
Water-reducing agent	Manufacturer's recommendations
Slump, in. (mm)	5 to 6 (127 to 152)

[a] 1 lb/yd^3 = 0.5933 kg/m^3.

[b] 1 in. = 2.54 cm.

Source: ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete," ACI SP-44 *Fiber Reinforced Concrete*, American Concrete Institute, Detroit (1974).

Table 13. Typical mix proportions of fibrous concrete mixes without fly ash

Cement, lb/yd^3[a] (kg/m^3)	550–950 (326–564)
W/C ratio	0.4–0.6
Percentage of sand to aggregate	50–100
Maximum aggregate, in.[b] (mm)	3/8 (9.5)
Air content, %	6–9
Fiber content, % by volume of mix[c]	0.5–2.5

[a] 1 lb/yd^3 = 0.5933 kg/m^3.

[b] 1 in. = 2.54 cm.

[c] Steel: 1% = 132 lb/yd^3 (78 kg/m^3), glass: 1% = 42 lb/yd^3 (25 kg/m^3), nylon: 1% = 19 lb/yd^3 (11 kg/m^3).

Source: ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete," ACI SP-44 *Fiber Reinforced Concrete*, American Concrete Institute, Detroit (1974).

power to rotate a drum full of fibrous concrete than one with conventional concrete.[191] Vibrators can be used to facilitate discharge from chutes or buckets because the fibrous concrete may not flow under its own weight. Fibrous concrete is also amenable to pumping or shotcreting for placement. Localized redistribution of fibrous concrete is best accomplished with forks or rakes. Compaction is best accomplished by external vibration of the forms and exposed surface to prevent fiber segregation, but properly controlled internal vibration can be used. Conventional screed methods can be used as well as metal trowels, tube floats, and rotating power floats for finishing.

3. Effect of fiber addition on material properties. Addition of dispersed fibers in a concrete mix imparts a number of material property improvements. Added in amounts up to 4% by volume, steel fibers increased the first crack flexural strength of concrete up to 2.5 times more than the conventional plain concrete mix and also slightly increased the compressive strength.[193] Dynamic strengths of fibrous concrete are 5 to 10 times greater than those for plain concrete.[194] Thermal conductivity of steel fiber-reinforced mortar with 0.5 to 1.5% fiber by volume at atmospheric pressure increased slightly with fiber content.[195] Toughness (area under load-deformation curve) can be increased by up to an order of magnitude with the addition of fiber reinforcement based on the area under the curve to maximum stress[196] and up to 40 times based on total area under the curve.[197,198] Splitting tensile strength of fibrous concrete is up to 70% greater than that for plain concrete.[198,199] Impact resistance of fibrous concrete is up to an order of magnitude greater than for plain concrete.[196] Fibrous concrete provides improved resistance to spallation and can reduce rebar congestion in regions of stress concentration such as at a PCPV penetration. Also, fibrous concrete has computational advantages because the fiber (by being dispersed in a concrete mix) provides an integral reinforcement in a monolithic construction, thus permitting analysis on a uniform basis using techniques such as finite elements.

2.2.3 Polymer concrete

1. Definition. Polymers in concrete combine hydraulic cement with polymer technologies to form new and improved construction material systems. Generally, three types of concrete materials use polymers to form composite materials.[200] Polymer-impregnated concrete (PIC) is a hydrated portland cement concrete impregnated with a monomer and subsequently polymerized in situ. Polymer—portland cement concrete (PPCC) is a premixed material in which either a monomer or polymer is added to a fresh concrete mixture in a liquid, powdery, or dispersed phase and subsequently allowed to cure (and, if needed, polymerized in place). Polymer concrete (PC) is a composite material formed by polymerizing a monomer and aggregate mixture, with the monomer acting as the binder. Table 14 presents some physical properties of common monomers used in PIC and PC, and Table 15

Table 14. Physical properties of common monomers used in PIC and PC

Monomer	Viscosity (cP)[a]	Density (g/cm³)[b]	Vapor pressure (mm Hg)[c]	Boiling point (°C)	Solubility in water (%)
Acrylonitrile	0.34[d]	0.81	85[e]	77	7.4[d]
Diallyl phthalate	12.0[e]		2.54[f]	300	Insoluble
Methyl methacrylate	0.57[d]	0.94	35[e]	100	1.5[g]
Monochlorostyrene	1.04[d]	1.11	0.68[e]	180	0.0064[d]
Styrene	0.76[e]	0.91	2.9[e]	135	0.070[d]
Tert-butylstyrene	1.46[d]	0.88	1.0[h]	218	0.0005[d]
Vinyl acetate	0.43[e]	0.93	115[d]	73	2.5[e]
Vinyl chloride	0.28[i]	0.91	1660[e]	−13.9	Slight
Vinylidene chloride		1.21	599[d]	32	Insoluble

[a] Centipoise = mPa·s.
[b] g/cm^3 = 1000 kg/m^3.
[c] mm Hg (at 0°C) = 133 Pa.
[d] 25°C.
[e] 20°C.
[f] 150°C.
[g] 30°C.
[h] 46°C.
[i] −20°C.

Source: ACI Committee 548, "State-of-the-Art Report," *Polymers in Concrete*, American Concrete Institute, Detroit (1977).

Table 15. Properties of some polymers used in PIC, PPCC, and PC

	PS[a]	PMMA[b]	PVAc[c]	PVC[d]
		Physical properties		
Specific gravity	1.05	1.18	1.19	1.38
Glass transition temperature, °C	93	100	70	80
Decomposition temperature, °C	250	260	200	110
Water adsorption, %	0.03–0.05	0.3–0.4	3–6	
		Mechanical properties		
Compressive strength, ksi (MPa)	11.5–16 (79–110)	11–19 (76–131)		10 (69)
Tensile strength, ksi (MPa)	5–12 (34–83)	8–11 (55–76)	2–4 (14–28)	
Flexural strength, ksi (MPa)	8.7–14 (60–97)	12–17 (83–117)		
Modulus of elasticity, ksi (MPa)	400–500 (2757–3447)	400–500 (2757–3447)	40 (276)	30–60 (207–414)
Poisson's ratio	0.33	0.33		0.38
Impact strength, ft-lb/in. (J/m)	0.25–0.4 (13.4–21.4)	0.4–0.5 (21.4–26.7)		

[a]Polystyrene.

[b]Polymethyl methacrylate.

[c]Polyvinyl acetate.

[d]Polyvinyl chloride.

Source: ACI Committee 548, "State-of-the-Art Report," *Polymers in Concrete*, American Concrete Institute, Detroit (1977).

presents properties of some polymers used in PIC, PPCC, and PC.[200] Polymerization is generally initiated using one of the following techniques: (1) thermal catalytic (compounds are added to the mix that generate free radicals on heating), (2) promoted catalytic (decomposition of organic peroxide catalysts is initiated by promoters or accelerators instead of temperature), and (3) radiation (ionizing radiation such as gamma rays emitted by cobalt-60 produce free radicals).

2. Process technology for polymers in concrete materials. The basic technique for producing PIC consists of fabrication of a precast concrete specimen, oven drying, saturation with a monomer, and in situ polymerization. Fabrication of the specimen is done according to standard concrete procedures, preferably using a good quality dense concrete that has been properly cured. The specimen is then dried at a temperature of ~150°C (302°F) to remove free water. [Drying temperatures of up to 400°C (750°F) have been used successfully for bridge deck sections to obtain deep monomer penetration.[201]] To obtain full penetration of concrete sections up to 305 mm (12 in.) thick, the following procedures can be used following drying:[202] (1) evacuate at 101 kPa (~30 in. Hg) and maintain for ~30 min; (2) introduce monomer under vacuum, subsequently pressurize to 68.9 kPa (~10 psig), and pressure-soak for ~60 min; (3) remove excess monomer; (4) remove and place section under water or, if section is too large, backfill impregnator with water; (5) polymerize monomer in situ; and (6) remove water and clean section. Techniques are also available for partial impregnation of concrete;[203] they include complete immersion in monomer (70 to 80% of voids become filled) or application of a monomer to just one surface. When high vapor pressure monomers are used, evaporation and drainage losses of the monomer should be minimized by: (1) wrapping the specimen in polyethylene sheet or aluminum foil after monomer saturation, (2) specimen encapsulation in a form during impregnation and polymerization, (3) polymerization while specimen is immersed in water, or (4) impregnation followed by a prepolymer dip before wrapping.[200] Manufacturer's recommendations should be followed for storing and handling monomers used for PIC.

The PPCC process technology is similar to that of conventional portland cement concrete because organic materials in either a powdery or dispersed form are added during the mix process. Premix polymerization (polymer latex) and water-soluble polymers are used in the production of PPCC. Typical properties of polymer latexes for PPCC are presented in Table 16 (Ref. 200). The latexes (1) are more effective in rich concrete mixes having cement contents ≥ 5 sacks/yd^3, (2) entrain air in the mix (antifoaming agents frequently required), and (3) act as water reducers. Curing of latex PPCC is generally done by moist curing the concrete for 1 to 3 d followed by dry curing at ambient conditions. Water-soluble polymers are added to the mix in a water solution; they differ from the latexes because they are thermosetting. Curing consists of standard moist curing followed by curing at elevated temperatures.

Epoxy, polyester and furan resins, and methyl methacrylate and styrene monomer systems are predominantly used as the binders in PC. The choice of resin system is dependent on cost, durability, adhesion to aggregate, handling properties, and ease of curing. Aggregate type and composition do not significantly influence the strength properties of the mix,[204] but aggregate gradation and maximum particle size are important because they determine the amount of relatively high-cost resin required to coat the particles and fill the voids. Conventional mixing equipment may be used for PC, but some of the resin systems, such as the polyesters and epoxies, present cleaning problems (that can ordinarily be handled by solvents). Because some of the monomers are volatile, irritating, or toxic, they should be handled using recognized safety procedures and mixed in a closed system or in a well-ventilated area. PC is cast in conventional molds to which a layer of mold release (silicone gels, automobile wax, or paraffins) has been applied. Consolidation is performed by external vibration, rodding, mechanical pressure, or application of a vacuum. Curing is performed by one of the three previously stated polymerization methods.

3. Properties of polymer in concrete materials. The ability of different monomers to provide high-strength PIC depends on (1) efficiency of monomer impregnation, (2) conversion of monomer to polymer, (3) formation of continuous polymer phase, and (4) mechanical properties of monomer.

Table 16. Typical properties of polymer latexes for PPCC

Polymer type	Polyvinyl acetate	Styrene butadiene	Acrylic	*Poly (vinylidiene chloride)*-PVC Copolymer (saran)	Neoprene
Percent solids	50	48	46	50	42
Stabilizer type	Nonionic	Nonionic	Nonionic	Nonionic	Anionic
Specific gravity at 25°C	1.09	1.01	1.05	1.23	1.10
Weight/gal, pounds at 25°C[a]	9.2	8.4	8.8	10.25	9.3
pH	2.5	10.5	9.5	2.0	9.0
Particle size, Å[b]	NA[c]	2000	NA	1400	NA
Surface tension, dynes/cm^2 at 25°C[d]	NA	32	40	33	40
Shelf life	NA	>2 years	Excellent	6 months	NA
Freeze-thaw stability (−15 to 25°C)	NA	5 cycles	5 cycles	None	NA
Viscosity, cP[e] at 20°C	17	24	~250	~15	10

[a]lb/gal = 119.8 kg/m^3.

[b]Å = 0.1 nanometre.

[c]NA = not available.

[d]dyne/cm^2 = 0.1 pascal.

[e]Centipoise = mPa·s.

Source: ACI Committee 548, "State-of-the-Art Report," *Polymers in Concrete*, American Concrete Institute, Detroit (1977).

Table 17 presents some typical mechanical properties of PIC.[202] Additionally, PIC exhibits reduced creep, improved resistance to aggresssive environments, and similar thermal expansion values to conventional concrete. At temperatures above the glass transition temperature of the polymer, the PIC loses most of its strength that is attributable to the polymer impregnation. However, polymers (such as 60-40 styrene-trimethylolpropane trimethacrylate) that have glass transition temperatures up to 214°C (417°F) are available.

The PPCC contains large amounts of polymer compared with PIC. Thus, the more predominant polymer phase can impart desirable properties to the concrete. Polymer latexes provide higher tensile and flexure strengths, increased strain at failure, improved durability, and superior adhesion.[205] The latex-modified concretes, however, may lose strength if immersed in water for extended periods of time and rapidly lose mechanical properties at temperatures higher than their glass transition temperature. Postmix polymerized vinyl monomers generally impart small, if any, compressive strength increases to concrete. Some systems (polystyrene) impart improved durability. Water-soluble epoxies provide improved flexural strengths.[206]

The PC differs from portland cement concrete, PIC, and PPCC because only organic polymer materials are used as the binder or matrix of the concrete. The properties of PC are dependent on the properties of the polymer binder and the amount of polymer in PC. The response of PC to mechanical stress is complex, being both temperature dependent and time dependent as well as being a function of the molecular structure.[207] The shape of the PC compressive stress-strain curve is dependent on the polymer binders; thus, it can be varied to suit a particular application. Table 18 presents representative properties for several PC systems.[200] Additionally, PC exhibits shrinkage during hardening, high thermal expansion coefficients compared with portland cement concretes, high chemical resistance, and creep behavior that is significantly affected by the temperature and humidity of the environment. An indication of the effect of temperature on PC is given in Table 19, which presents the dependence of several mechanical properties of a methyl methacrylate-trimethylolpropane trimethacrylate PC on temperature.[202]

Table 17. Typical mechanical properties of PIC

[Concrete dried at 221°F (105°C) overnight, radiation polymerization]

Polymer	Polymer loading (wt %)	Strength [psi (MPa)][a]			Modulus of elasticity [10^6 psi (GPa)]	
		Compressive	Tensile	Flexural	Uniaxial test	Beam test
Unimpregnated	0	4,950 (34)	335 (2.3)	630 (4)	2.7 (19)	3.0 (21)
MMA	4.6–6.7	20,250 (140)	1,630 (11)	2,640 (18)	6.3 (43)	6.2 (43)
MMA + 10% TMPTMA	5.5–7.6	21,590 (149)	1,510 (10)	2,220 (15)	6.1 (42)	6.1 (42)
Styrene	4.2–6.0	14,140 (97)	1,100 (8)	2,300 (16)	6.3 (43)	6.3 (43)
Acrylonitrile	3.2–6.0	14,140 (97)	1,040 (7)	1,470 (10)	5.9 (41)	4.5 (31)
Chlorostyrene	4.9–6.9	16,090 (111)	1,120 (8)	2,380 (16)	5.6 (39)	6.3 (43)
10% polyester + 90% styrene	6.3–7.4	20,500 (141)	1,500 (10)	3,300 (23)	6.5 (45)	6.4 (44)
Vinyl chloride[b]	3.0–5.0	10,240 (71)	675 (5)		4.2 (29)	
Vinylidene chloride[b]	1.5–2.8	6,650 (46)	370 (3)		3.0 (21)	
t-butyl styrene[b]	5.3–6.0	18,150 (125)	1,445 (10)		6.4 (44)	
60% styrene + 40% MPTMA[b]	5.9–7.3	17,140 (118)	910 (6)		6.3 (43)	

[a] psi = 7 kPa.

[b] Dried at 302°F (150°C) overnight.

Source: L. E. Kukacka et al., *Concrete Polymer Materials — Fifth Topical Report*, Brookhaven National Laboratory and U.S. Bureau of Reclamation, BNL No. 50390 and USBR No. REC-ERC-73-12 (December 1973).

Table 18. Typical mechanical properties of PC

Polymer	Polymer: aggregate ratio	Density $(g/cm^3)^a$	Strength [psi (MPa)]^b			Modulus of elasticity [10^6 psi (GPa)]
			Compressive	Tensile	Flexural	
Polyester	1:10	52–2.34	15,650–17,800 (108–123)	1,700–2,000 (12–14)	5,000–5,700 (34–39)	4–5 (28–34)
Polyester	1:9	2.33	9,800 (68)		2,400 (17)	4 (28)
Polyester-styrene	1:4		11,650 (80)			
Epoxy + 40% dibutyl phthalate	1:1^c	1.65	7,100 (49)	18,500 (128)		0.3 (2)
Epoxy + poly-aminoamide	1:9	2.28	9,250 (64)		3,250 (22)	4.5 (31)
Epoxy-polyamide	1:9	9–2.1	12,800–14,200 (88–98)		4,250–5,000 (29–34)	
Epoxy-furan	1:1^c	1.7	8,500–10,000 (59–69)	1,000–1,150 (7–8)	14 (0.1)	
MMA-TMPTMA	1:15	2.40	19,600 (135)	1,430 (10)	3,100 (21)	5 (34)

$^a 10^{-3}$ $g/cm^3 = mg/cm^3$.

^{b}psi = 7 kPa.

^{c}Polymer mortar.

Source: ACI Committee 548, "State-of-the-Art Report," *Polymers in Concrete*, American Concrete Institute, Detroit (1977).

Table 19. Properties of methyl methacrylate-trimethylolpropane trimethylacrylate PC at various temperatures

Test	Temperature (°C)	Result
Tensile splitting strength	–26	1,510 psi[a] (10.4 MPa)
	21	1,430 psi (9.9 MPa)
	88	1,370 psi (9.4 MPa)
Compressive strength stress	–26	24,800 psi (171 MPa)
	21	19,600 psi (135 MPa)
	49	15,800 psi (109 MPa)
	88	14,100 psi (97 MPa)
Modulus of elasticity	–26	6.11×10^6 psi (42 GPa)
	21	5.28×10^6 psi (36 GPa)
	88	4.44×10^6 psi (31 GPa)
Poisson's ratio	–26	0.24
	21	0.23
	88	0.22
Elastic limit stress	–26	14,000 psi (97 MPa)
	21	7,500 psi (52 MPa)
	88	4,800 psi (33 MPa)
Ultimate compressive strain	–26	5,360 μin./in.[b]
	21	7,080 μin./in.
	88	8,000 μin./in.
Coefficient of expansion	–20–21	5.30×10^{-6} in./in./°F (9.54×10^{-6} cm/cm/°C)
	21–60	7.53×10^{-6} in./in./°F (13.6×10^{-6} cm/cm/°C)

[a]psi = 7 kPa.

[b]Min/in. = μm/m.

Source: L. E. Kukacka et al., *Concrete Polymer Materials — Fifth Topical Report*, Brookhaven National Laboratory and U.S. Bureau of Reclamation, BNL No. 50390 and USBR No. REC-ERC-73-12 (December 1973).

2.2.4 High-temperature concrete

1. Elevated temperature cements. Type II portland cement, meeting the requirements of ANSI/ASTM C 150-78a (Ref. 5), finds limited use in concretes for elevated temperature applications. 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 (800°F). (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 (1200°F); hydrothermal, calcium aluminate, or tricalcium aluminate cements are required for such applications.

Hydrothermal (nonportland) cements have recently been developed for lining oil wells,[208] 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 (10,000 to 20,000 psi), (2) excellent adhesion to metals except for aluminum, (3) good resistance to aggressive environments, (4) low permeability, and (5) material system costs comparable with those of special portland cements (such as Type III high early strength cement). Large placement of the material system is apparently not the same problem as it is for polymer-concrete materials because the exothermic heat buildup is very low. Available property data indicate that hydrothermal cement systems are quite promising as structural materials. However, the available data are limited to those supplied by the manufacturer; more detailed data need to be developed by an independent investigator.

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%; Al_2O_3, [illegible] to 44%; SiO_2, 3 to 11%; and Fe_2O_3, 4 to 12%. The principal products of hydration at room temperature are calcium-aluminate hydrates and some colloidal alumina.[209,210] Table 20 (Ref. 211) gives the chemical composition of typical commercial calcium-aluminate cement. 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 (3270°F) 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.[30] 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

Table 20. Chemical composition of calcium aluminate cements

Class brand	Low purity		Intermediate purity		High purity	
	A	B	A	B	A	B
Al_2O_3	41.0	40.0	55.0	51.0	79.0	73.0
CaO	37.0	37.0	37.0	40.0	18.0	25.0
SiO_2	9.0	3.0	5.0	6.0	0.1	0.7
Fe_2O_3	11.0	18.1	1.5	0.6	0.3	0.3

Source: W. T. Bakker, "Properties of Refractory Concretes," *Refractory Concrete*, SP 57-2, American Concrete Institute, Detroit (1978).

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. Table 21 (Ref. 212) presents the effect of various additives on calcium-aluminate cements. The use of calcium-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.

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 (1065°F). Accordingly, crushed stone and gravel-based aggregates suitable for use are limited to diabase traprock, olivine, pyrophyllite, 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 (1830 to 2100°F). 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_2O_3 (Ref. 211). Refractory aggregates such as crushed firebrick (30 to 45% Al_2O_3) are stable to temperatures of 1300°C (2300°F). For temperatures up to 1600°C (2900°F), aggregates such as fused alumina or carborundum can be used; for temperatures up to 1800°C (3270°F) 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 (500°F). Table 22 (Ref. 211) presents examples of typical aggregates for dense refractory concretes.

3. High-temperature (refractory) concrete mixes. Refractory concrete is defined (Ref. 211) as a granular refractory material that, when

Table 21. Fractors affecting calcium aluminate cement

Additive or admixture	Effect
Accelerators	Generally, all alkalies and alkaline compounds Dilute sodium, potassium and calcium hydroxide (hydrated lime) Sodium and potassium carbonate Sodium and potassium silicate Sodium and calcium sulfate (gypsum or anhydrite, >0.5—1.0%) Plaster of paris (hemihydrate) Dilute sulfuric acid Lithium salts Prehydrated and ground calcium aluminate cement Portland cement Triethanol amine
Retarders	Generally, acids and acidic compounds Generally, air-entraining and water-reducing agents Dilute hydrochloric (muriatic) and acetic acid Sodium, potassium, magnesium, and barium chloride Calcium chloride (deleterious to calcium aluminate cement concrete) Aluminum chloride hydrate Boric acid and borax Sodium and calcium sulfate (gypsum or anhydrite, <0.25%) Magnesium and barium hydroxide Sea water Lead salts Phosphates Isopropylalcohol, glycols, glycerine, starch, flour, sugar, casein, cellulose products, ligno-sulfonates Salts of hydroxy-carboxylic acids (citric, gluconic, tartaric)
Water-reducing agents	Glúconic acid Sodium gluconate Sodium citrate
Plasticizers	Bentonite (2—3%) Raw fire clay (5—15%) Low alkali fly ash Fine ground granulated slag Air-entraining agents

Source: F. E. Linck, Turnaround Maintenance, Houston, Texas, personal communication to D. J. Naus, Oak Ridge National Laboratory (Oct. 6, 1980).

Table 22. 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," *Refractory Concrete*, SP 57-2, American Concrete Institute, Detroit (1978).

mixed with water, will harden at room temperature to support its own weight sufficiently. 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 (Table 23).[213]

The effect of water content on the physical properties is critical. The amount of water necessary for a given material will depend on a number of factors: material proportions, ambient temperatures, water temperature, type and speed of the mixer, and the size and shape to be cast. Excess water can seriously degrade the strength of dense refractory concrete

Table 23. Classification of refractory concretes

Classes	Test requirements	
	Permanent linear shrinkage ≯1.5% when fired for 5 h at temperature [°F (°C)]	Maximum bulk density after drying at 104–110°C[a] [lb/ft^3 (kg/m^3)]
Alumina-silica-base castable refractories[b]		
A	2000 (1095)	
B	2300 (1260)	
C	2500 (1370)	
D	2700 (1480)	
E	2900 (1595)	
F	3100 (1705)	
G	3200 (1760)	
Insulating castable refractories[c]		
N	1700 (925)	55 (881)
O	1900 (1040)	65 (1041)
P	2100 (1150)	75 (1201)
Q	2300 (1260)	95 (1522)
V	3200 (1760)	105 (1682)

[a]220–230°F.

[b]Regular dense concretes.

[c]Insulating concretes.

Source: "Standard Classification of Castable Refractories," Part 17 of *Annual Book of ASTM Standards*, ANSI/ASTM C 401-77 (1980).

(Fig. 37).[214] 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 (Fig. 38).[212] 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. Troweling requires a product that exhibits plasticity and adherence properties generally achieved through the addition of clay, bentonite, and other plasticizers to the mix

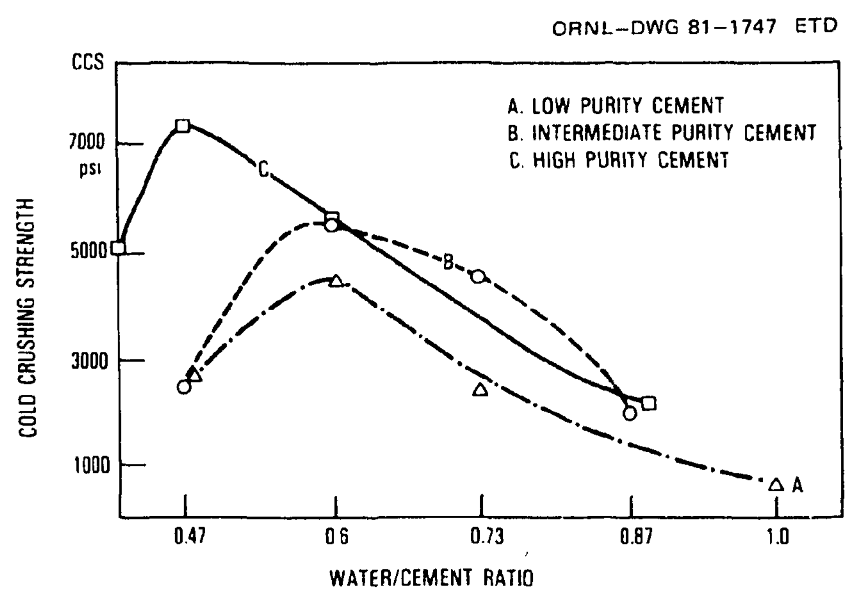

Fig. 37. Effect of water-cement ratio on dried strength of dense refractory concrete. Source: A. V. Briebach, "A Review of Refractory Hydraulic Cements," *J. Brit. Cer. Soc.* 71(7), 153–58.

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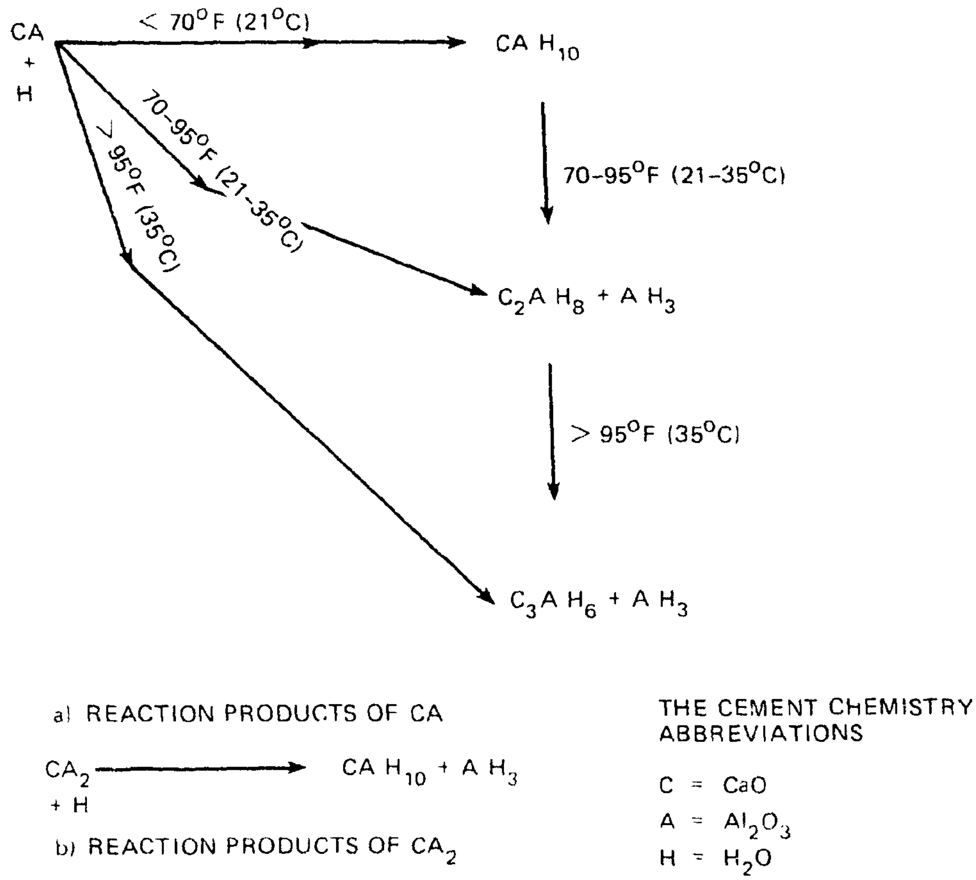

Fig. 38. Hydration reaction products of calcium aluminates. Source: F. E. Linck, Turnaround Maintenance, Houston, Texas, personal communication to D. J. Naus, Oak Ridge National Laboratory (Oct. 6, 1980).

(these materials, however, increase the water requirements leading to reduced strength). Casting is the most common method of application, and use of the lowest possible water-cement ratio is desired to produce optimum properties with respect to rheology and strength. Shotcreting has become an increasingly popular method for installing refractory concretes[215] for two reasons: (1) installation cost is low and (2) quality of installed concrete is superior to cast concrete because a lower water-cement ratio can be used. However, the concrete mix must be specially designed for shotcreting, and experienced operators must be used to cut down rebound losses.

4. Properties of high-temperature concretes. 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 and dehydration† of the concrete and again when the glassy bond is formed at high temperatures.[216] Most normal-weight high-temperature concretes will have <0.5% permanent linear shrinkage after firing at 1090°C (2000°F).[212] Figure 39 presents length change as a function of temperature of a typical high-temperature concrete.[212]

*Generally values are ~5 × 10^{-6} cm/cm/°C (3 × 10^{-6} in./in./°F) but may be as high as 9 × 10^{-6} cm/cm/°C (5 × 10^{-6} in./in./°F).

†Shrinkage during dehydration is generally 0.1 to 0.3% (Ref. 211), ANSI/ASTM C 401-79 (Ref. 213), limits linear shrinkage to 1.5% at all temperatures below the service limit to minimize cracking on heatup or cooldown (0.5 to 1.0% where cracking is detrimental, such as in petrochemical plants), permanent expansion can occur at high temperatures.

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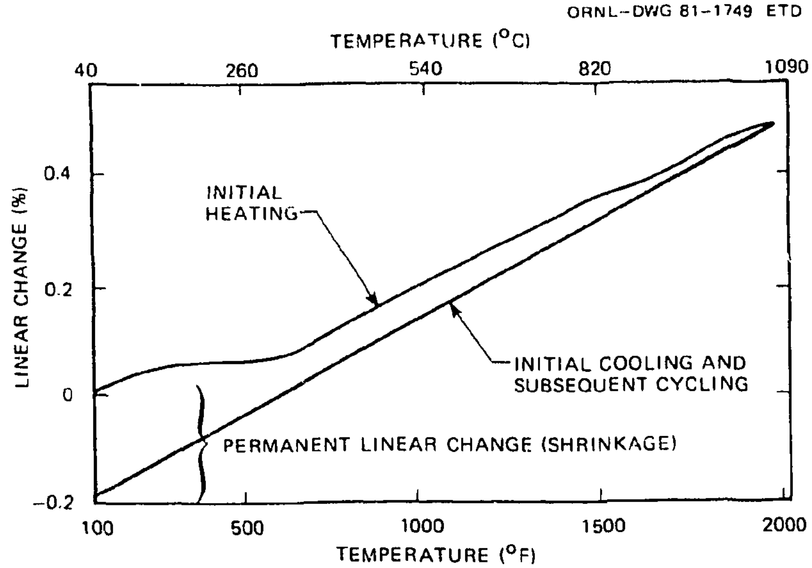

Fig. 39. Length change as a function of temperature of a typical high-temperature concrete. Source: F. E. Linck, Turnaround Maintenance, Houston, Texas, personal communication to D. J. Naus, Oak Ridge National Laboratory (Oct. 6, 1980).

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 to 540°C (220 to 1000°F). Further heating from 540 to 1090°C (1000 to 2000°F) usually has only a slight effect on strength. At about 1090°C (2000°F), initial liquid formation occurs, and the hot strength decreases considerably. Specimens heated above 1090°C (2000°F) and tested after cooling show a marked increase in cold strength because the liquids formed vitrify to produce high cold strengths. Room-temperature compressive strengths of dense refractory concretes generally range between 13.8 and 55.2 MPa (2000 to 8000 psi).[211] The effect of elevated temperature on modulus of elasticity is relatively minor when compared with normal portland concrete systems. [The modulus of elasticity of refractories is determined according to ANSI/ASTM C 885-79 (Ref. 217).] Figure 40 (Ref. 216) presents typical modulus-of-elasticity curves as a function of temperature for refractory

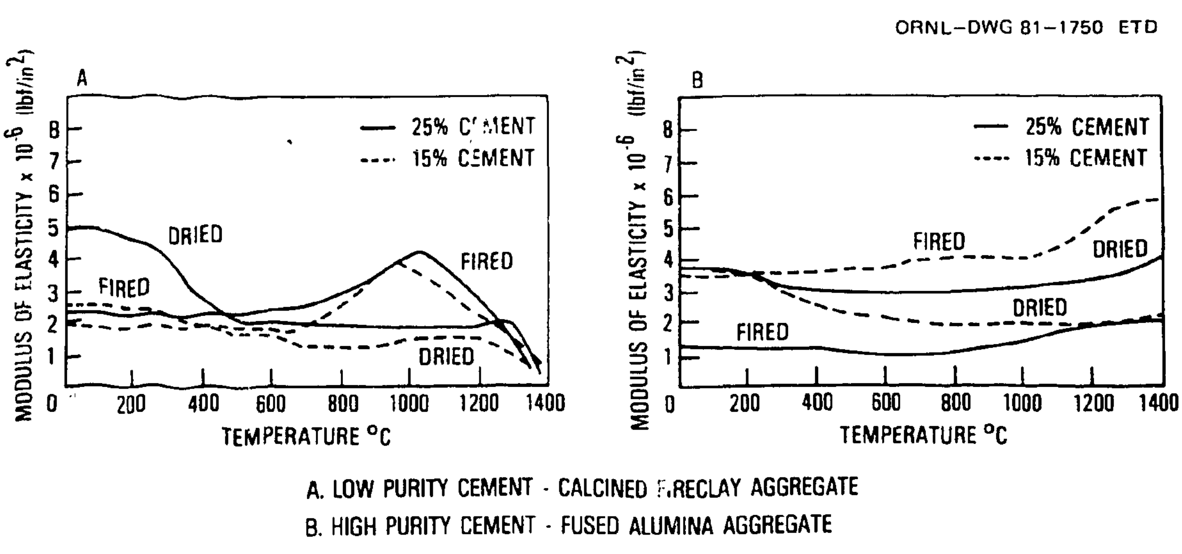

Fig. 40. Typical modulus-of-elasticity curves for refractory concretes containing low- and high-purity cements. Source: J. M. McCullough and G. R. Rigby, "Mechanical Properties of Refractory Castables," *J. Brit. Cer. Soc.* 71(7), 233.

concretes containing low- and high-purity cements. Although data are limited, the modulus of elasticity tends to vary with strength, and values range from 6.9 to 55 GPa (1×10^6 to 8×10^6 psi). Room-temperature modulus-of-rupture tests are conducted according to ANSI/ASTM C 268-79 (Ref. 218), and elevated temperature tests are conducted according to ANSI/ASTM C 583-76 (Ref. 219). Generally, the modulus of rupture of dense refractory concretes varies from about 4.8 to 10.3 MPa (700 to 1500 psi) after drying at 104°C (220°F).[211] Figure 41 (Ref. 215) presents typical hot and cold modulus-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. Figure 42 (Ref. 220) presents the effect of temperature on the stress-strain behavior of alumina-silicate bricks (85% alumina) tested in three-point bending.

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 Figs. 43 (Ref. 221) and 44 (Ref. 212). On first heatup 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 liner.[222] Typical k factors range from about 72 W-cm m^{-2} $°C^{-1}$ (5 Btu-in. ft^{-2} h^{-1} $°F^{-1}$) for 1920 kg/m^3 (120 lb/ft^3) material to about 144 W-cm m^{-2} $°C^{-1}$ (10 Btu-in. ft^{-2} h^{-1} $°F^{-1}$) for 2560 kg/m^3 (160 lb/ft^3) material.[212]

The specific heat of refractory concrete depends on its chemical composition and increases with temperature. Typical values range from 837 J kg^{-1} $°C^{-1}$ (0.20 Btu lb^{-1} $°F^{-1}$) at 40°C (100°F) to 1210 J kg^{-1} $°C^{-1}$ (0.29 Btu lb^{-1} $°F^{-1}$) at 1370°C (2500°F).[212]

Total creep of refractory concretes does not vary much with temperature. The materials generally deform plastically at relatively low loads [~0.2 MPa (25 psi)] at temperatures greater than 1090°C (2000°F) and at high loads [3.4 to 13.8 MPa (500 to 2000 psi)] for temperatures as low as 316 to 538°C (600 to 1000°F).[223] Creep evidently proceeds by the cement deforming until contacts between aggregate particles are established.[223]

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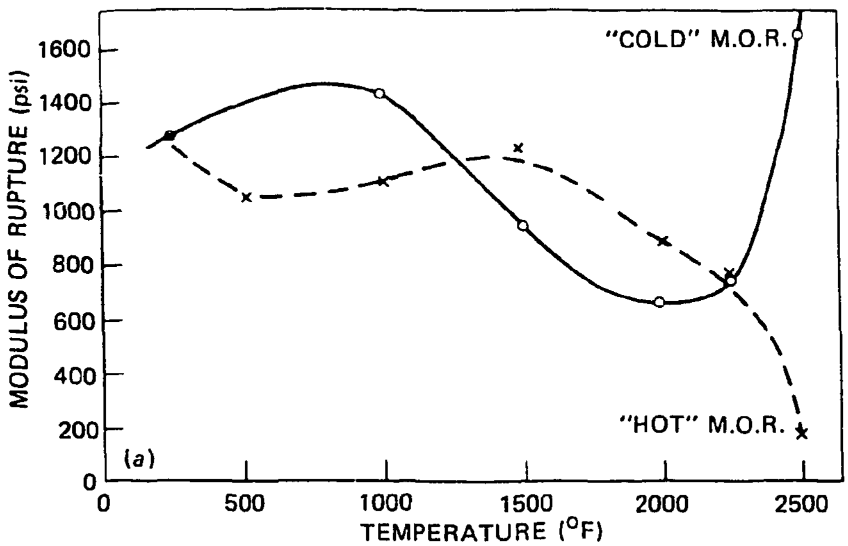


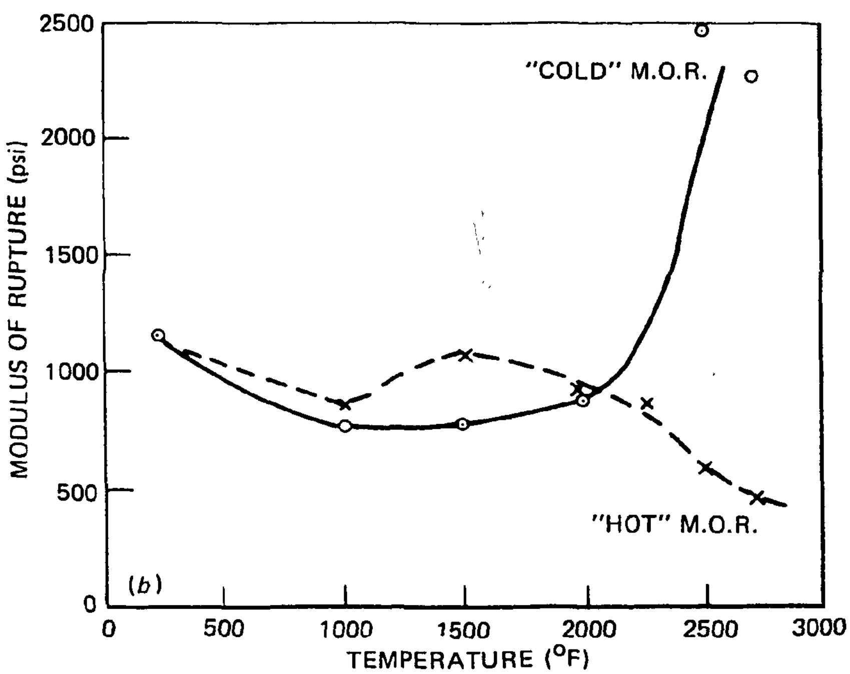


Fig. 41. Typical hot and cold modulus-of-rupture results for a 40–50% Al_2O_3 castable using (a) intermediate-purity cement and (b) high-purity cement. Source: W. T. Bakker et al., "Blast Furnace Gunning in the USA," *Proceedings International Feuerfest Colloquium*, Aachen, Germany (Oct. 27–29, 1971).

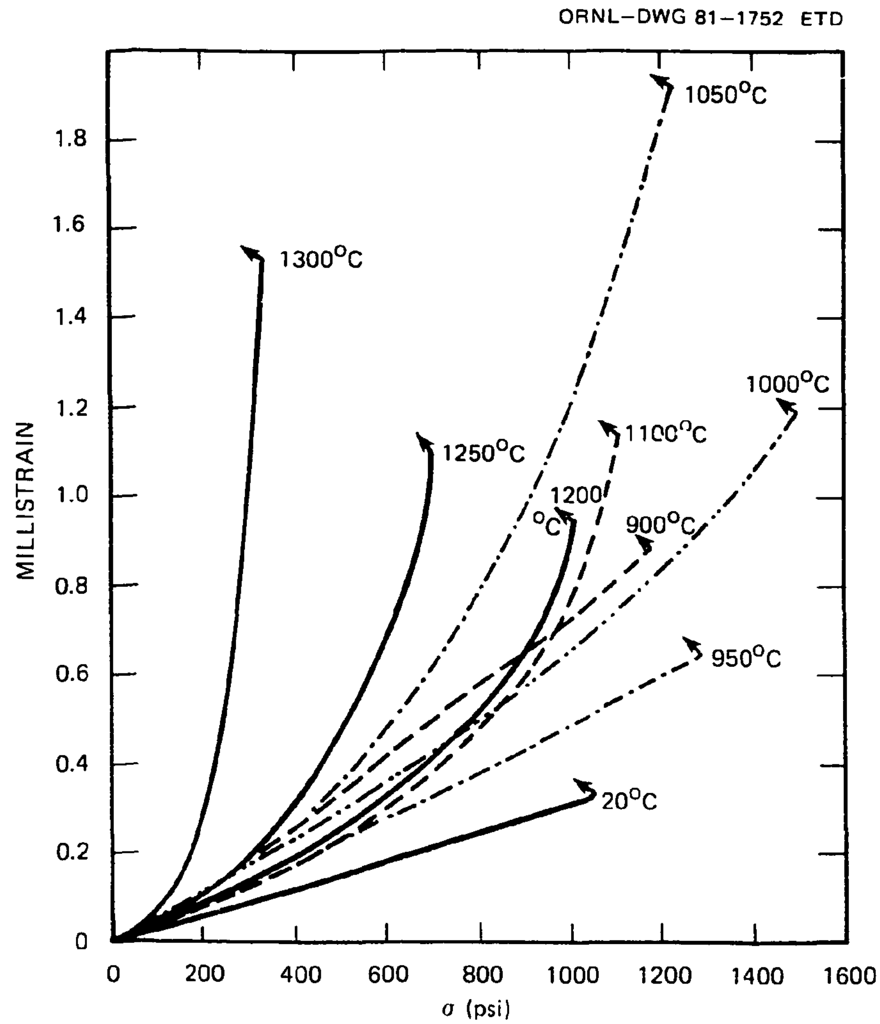
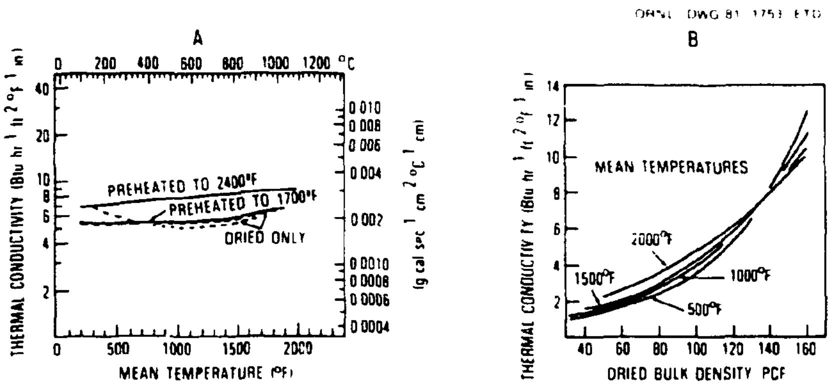


Fig. 42. Effect of temperature on stress-strain behavior of alumina-silicate bricks. Source: G. C. Padgett et al., "Stress/Strain Behavior of Refractory Materials at High Temperatures," Research Paper 608, The British Ceramic Association.

Figure 45 presents creep of a 1442 kg/m^3 (90 lb/ft^3) refractory concrete at 538°C (1000°F) and 13.8 MPa (2000 psi).[223]

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 (1500°F) and measured at room temperature is very low — typically



A. AS A FUNCTION OF TEMPERATURE
B. AS A FUNCTION OF BULK DENSITY
(ALUMINA-SILICA BASE CONCRETES ONLY)

Fig. 43. Thermal conductivity of refractory concretes as a function of temperature and dried bulk density. Source: E. Ruh and A. Renky, "Thermal Conductivity of Castable Refractories," *J. Am. Cer. Soc.* 46(2) (1963).

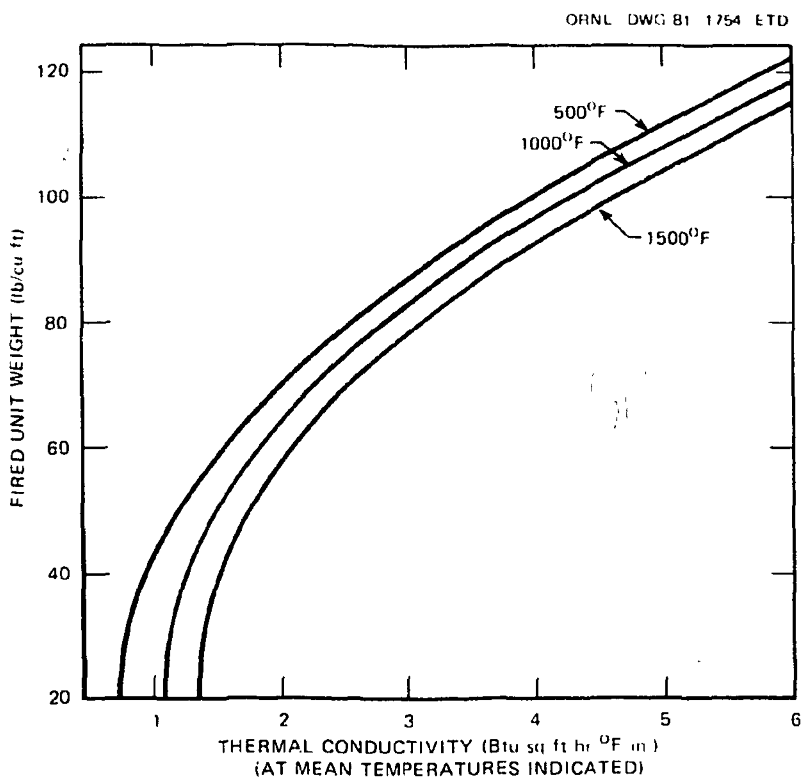


Fig. 44. Thermal conductivity of lumnite concretes as a function of fired unit weight. Source: F. E. Linck, Turnaround Maintenance, Houston, Texas, personal communication to D. J. Naus, Oak Ridge National Laboratory (Oct. 6, 1980).

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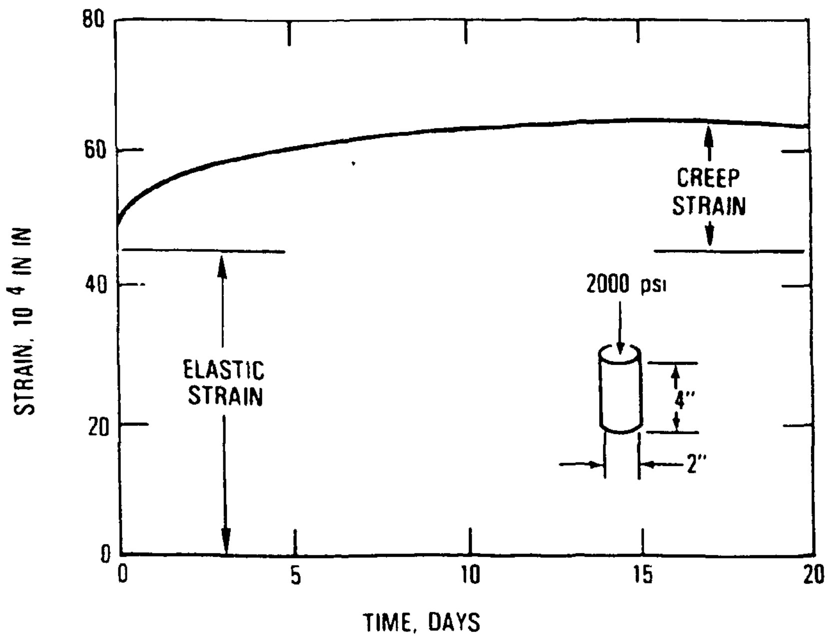

Fig. 45. Creep of a refractory concrete material. Source: J. F. Wygant and M. S. Crowley, "Designing Monolithic Refractory Vessel Linings," *Am. Cer. Soc. Bull.* 43(3) (1964).

15 millidarcys but can be as high as 1000 millidarcys. When fired to between 820 and 1090°C (1500 and 2000°F), the room-temperature permeability may increase by a factor of 2 to 3 (Ref. 212).

Abrasion is defined as the wearing away of a surface by rubbing action; erosion is the deterioration brought about by the action of fluids, gases, and solids in motion. These two concepts are closely related and at times used interchangeably. The erosion process begins with the wearing away of the weakest matrix constituent (usually the binder), leaving the coarse or hard aggregate to eventually fall away. A hard aggregate, a high modulus of rupture, and high compressive strength at the hot face are necessary for good abrasion and erosion resistance.

5. Effect of chemicals on refractory concretes. Corrosive substances may present a problem for refractory concrete materials. Corrosive liquids, such as slags, can have adverse effects leading to rapid wear beause the calcium-aluminate bond in the concrete reacts at higher temperatures with the aluminum silicates in the refractory grain to form

low-melting calcium-aluminum silicates (very high alumina concretes show good resistance to basic slags but are less stable in the presence of acid slags).[211] Refractory concretes can be corroded by acid condensates on the cold face of linings if the cold face temperature lies below the dew point. When the concrete contains free iron or iron oxides, it can disintegrate in CO-rich atmospheres.[224] Volatile alkali compounds in the gas atmosphere can cause disintegration of the refractory because of formation of low-density alkali-alumina silicates.[215] Hydrogen attack on refractory concretes occurs mainly through the reduction of silica and silicates to the volatile SiO (Ref. 225). Steam adversely affects the strength of very high alumina concretes through strength degradacion by dehydration of the AH(Boehmite) bond phase and formation of the high-pressure $C_4A_3H_3$ phase.[226] [Calcined flint—high purity alumina cement concretes show no strength degradation through formation of a CAS_2 (anorthite) bond phase.]

2.2.5 Lightweight and insulating concretes

1. Definitions. Lightweight concrete material systems that are suitable for structural and/or insulating applications can be generally classified into four areas: (1) low-density concretes, (2) structural lightweight concretes, (3) moderate strength concretes, and (4) refractory insulating concretes. Low-density concrete is made with (aggregate type) or without (cellular type) aggregate additions to portland cement, water, and air to form a hardened material that will have an oven-dry unit weight of 800 kg/m^3 (50 lb/ft^3) or less and compressive strengths from ~0.7 to 6.9 MPa (100 to 1000 psi).[227] Structural lightweight concretes (1) have full structural efficiency; (2) are generally made with expanded shales, clays, slates, slags, and pelletized fly ash; (3) have unit weights from 1350 to 1900 kg/m^3 (85 to 120 lb/ft^3); and (4) have compressive strengths from 17.5 to >42 MPa (2500 to >6000 psi).[228] Moderate-strength concretes include cellular and aggregate concretes with oven-dry unit weights between 800 and 1350 kg/m^3 (50 and 85 lb/ft^3) but have compressive strength <17.5 MPa (2500 psi).[229] Figure 46 (Ref. 228) presents a spectrum of the lightweight aggregate concrete material systems. Lightweight refractory-

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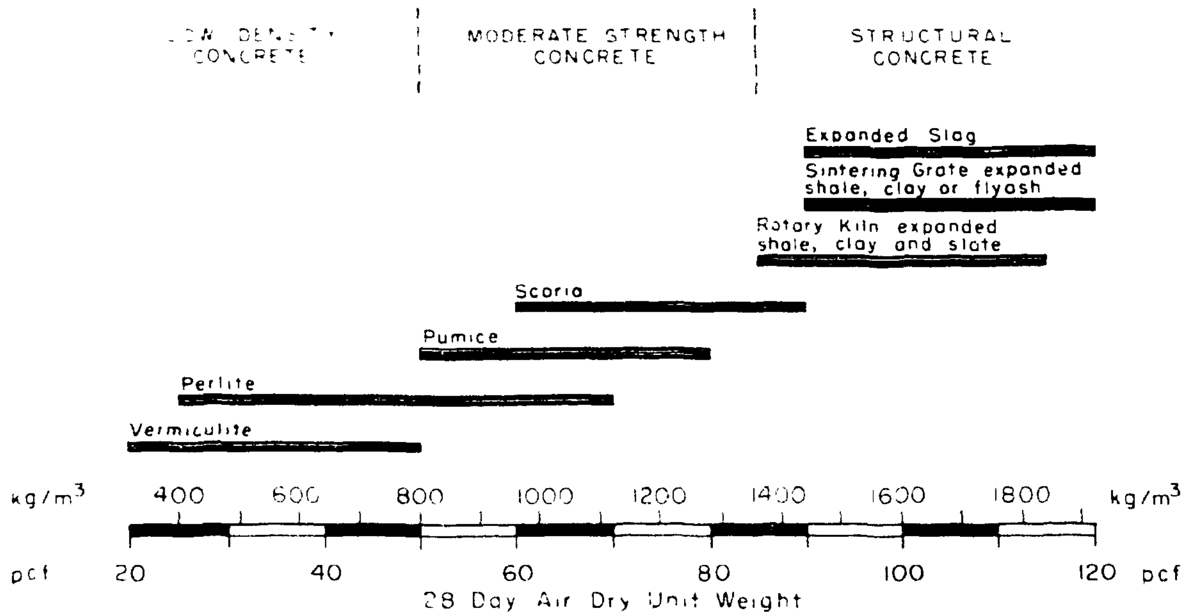

Fig. 46. Spectrum of lightweight aggregate concretes. Source: ACI Committee 213, "Guide for Structural Lightweight Aggregate Concrete," *Proc. J. Am. Concr. Inst.* 64(8), 433–69, Detroit (August 1967).

insulating concretes use a calcium-aluminate cement binder (in lieu of a portland cement binder), have unit weights between 881 and 1680 kg/m^3 (55 and 105 lb/ft^3), and service temperature limits between 927 and 1760°C (1700 and 3200°F). Of the four classifications of lightweight concretes, the first three have minimal application to PCPVs.

2. Low-density concretes. Low-density concretes are primarily used for fills, thermal insulation and roof decks. The concrete is produced by (1) using lightweight aggregate fillers such as perlite (siliceous rock), vermiculite (micaceous material that has been exfoliated), or expanded, calcined, or sintered natural or artificial materials; or (2) introducing foam into a cement slurry or a cement-aggregate slurry. Batching, mixing, placing, and curing of low-density concrete should be done according to procedures recommended in Ref. 227. Compressive strengths are low [<0.7 MPa (<100 psi)] and vary with the oven-dry unit weight (Table 24).[227] Shrinkage of low-density concrete is not usually important when it is used for fills or thermal insulation; however, for structural applications,

Table 24. Compressive strength ranges for low-density concrete

Oven-dry unit weight [pcf (kg/m^3)]	Usual range of compressive strength at 28 d [psi (kgf/cm^3)]
20–25 (320–400)	100–125 (7.0–8.8)
25–30 (400–480)	125–225 (8.8–15.8)
30–35 (480–560)	225–350 (15.8–24.6)
35–40 (560–640)	350–450 (24.6–31.6)
40–50 (640–800)	450–750 (31.6–52.7)

Source: ACI Committee 523, "Guide for Cast-in-Place Low Density Concrete," *Proc. J. Am. Concr. Inst.* 64(9), 529–37, Detroit (September 1967).

shrinkage must be considered. Typical shrinkage ranges (%) after 6 months at 50% relative humidity for perlite, vermiculite, and cellular concretes are presented in Table 25 (Ref. 227). Tables 26 through 28 present typical values for thermal conductivity, modulus of elasticity, and coefficient of thermal expansion for low-density concretes.[227]

Table 25. Shrinkage values for low-density concrete after 180 d at 50% relative humidity

Concrete type	Usual range of shrinkage at 180 d (%)
Perlite	0.10–0.30
Vermiculite	0.20–0.45
Cellular	0.20–0.60

Source: ACI Committee 523, "Guide for Cast-in-Place Low Density Concrete," *Proc. J. Am. Concr. Inst.* 64(9), 529–37, Detroit (September 1967).

Table 26. Thermal conductivity values for low-density concrete

Oven-dry unit weight [pcf (kg/m^3)]	Thermal conductivity (k factor)[a] [$Btu \cdot in./h \cdot ft^2 \cdot °F$ ($MW/cm \cdot °C$)]
20 (320)	0.70 (1.01)
30 (480)	0.90 (1.30)
40 (640)	1.15 (1.66)
50 (800)	1.40 (2.02)

[a]Representative values for dry materials from ASHRAE Guide and Data Book. They are intended as design (not specification) values for materials in normal use. For the conductivity of a specific concrete, the user may obtain the value supplied by the producer or secure the results of tests.

Source: ACI Committee 523, "Guide for Cast-in-Place Low Density Concrete," *Proc. J. Am. Concr. Inst.* 64(9), 529–37, Detroit (September 1967).

Table 27. Modulus of elasticity values for low-density concrete

Concrete type	Oven-dry unit weight [pcf (kg/m^3)]	Compressive strength [psi (kgf/cm^2)]	Modulus of elasticity [10^3 psi (10^3 kgf/cm^2)]
Perlite	20–40 (320–640)	80–450 (5.6–32)	70–250 (4.9–18)
Vermiculite	15–40 (240–640)	70–400 (4.9–28)	40–140 (2.8–10)
Cellular (S)[a]	25–35 (400–560)	130–250 (9.1–18)	20–100 (1.4–7)
Cellular (N)[b]	15–40 (240–640)	70–450 (4.9–32)	10–240 (0.7–17)

[a]Cement-sand ratio = 1.

[b]Neat cement.

Source: ACI Committee 523, "Guide for Cast-in-Place Low Density Concrete," *Proc. J. Am. Concr. Inst.* 64(9), 529–37, Detroit (September 1967).

Table 28. Coefficient of thermal expansion values for low-density concrete

Concrete type	Coefficient ($\times 10^{-6}$) of thermal expansion [°F (°C)]
Perlite	4.3–6.1 (7.7–11.0)
Vermiculite	4.6–7.9 (8.3–14.2)
Cellular	5.0–7.0 (9.0–12.6)

Source: ACI Committee 523, "Guide for Cast-in-Place Low Density Concrete," *Proc. J. Am. Concr. Inst.* 64(9), 529–37, Detroit (September 1967).

3. Structural lightweight concrete. When used to replace normal-weight concrete in a structure, structural lightweight concretes can effect reduced costs because of reduced dead loads, lower volume of concrete and reinforcing steel, and lower handling and forming costs. Aggregates used are lightweight because of the cellular structures of individual aggregate particles that were produced during formation of the particles at high temperatures, generally 1100°C (2000°F) or higher. The particles may be naturally occurring (pumice, scoria) or processed (rotary kiln-expanded shales, clays, and slates; sintered shales, clays, and pelletized or extended fly ash; and expanded slags). Batching, mixing, placing, and curing of structural lightweight concretes should be done according to procedures recommended in Refs. 228 and 230. Figure 47 presents (1) ranges of cement contents required to produce indicated compressive strengths, (2) ranges in total water content to obtain slumps between 2.5 and 10 cm (1 and 4 in.), and (3) typical properties of lightweight concretes as a function of compressive strength with respect to 28-d air-dry unit weight, modulus of elasticity, creep (normal or steam curing), and drying shrinkage (normal and steam curing).[228] Similarly, Fig. 48 presents typical properties of lightweight concretes as a function of compressive strength with respect to splitting tensile strength (moist cure

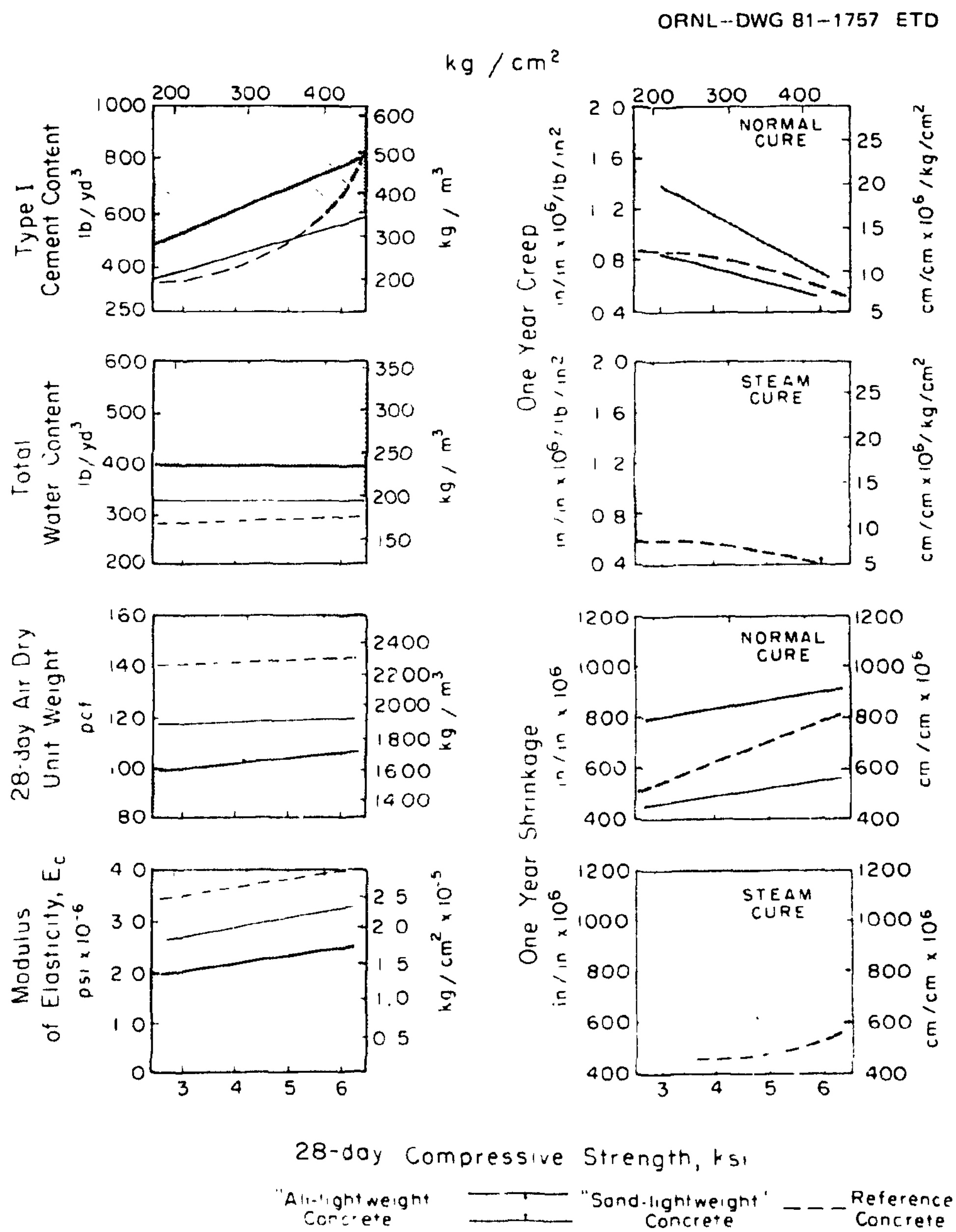

Fig. 47. Properties of lightweight concrete.

or air dried), modulus of rupture (steam or normal cure), durability factors (freezing and thawing), bond strengths, ultimate strain, and stress-block factors.[228] Also, for lightweight aggregate concretes, Poisson's ratio varies between 0.17 and 0.23, and thermal expansion ranges from 7×10^{-6} to 11×10^{-6} cm/cm/°C (4×10^{-6} to 6×10^{-6} in./in./°F). Figure 49

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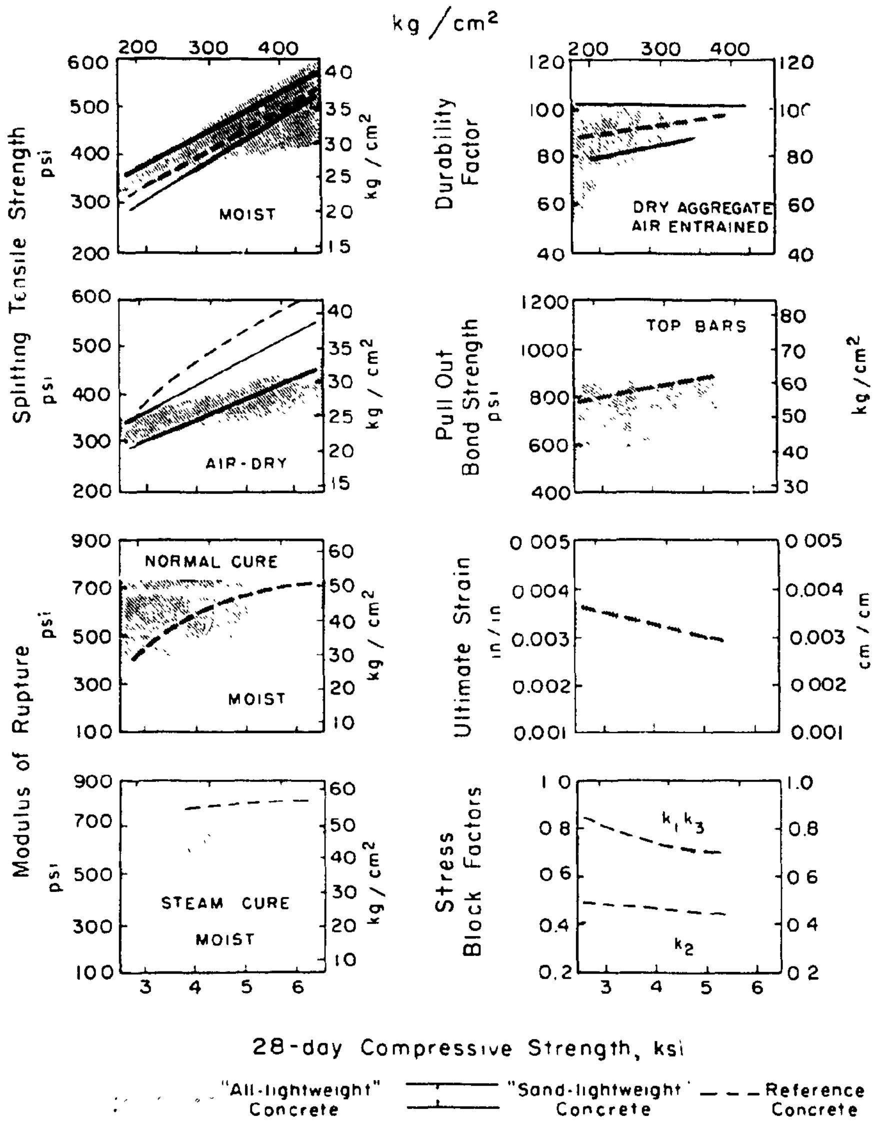

Fig. 48. Properties of lightweight concrete.

presents the thermal conductivity data.[228] Design using structural lightweight concretes should be in accordance with recommendations in Ref. 228.

4. Moderate-strength concretes. Moderate-strength concretes are produced as either (1) cellular concretes that contain uniformly distributed stable air or gas cells and natural or manufactured sand aggregate or

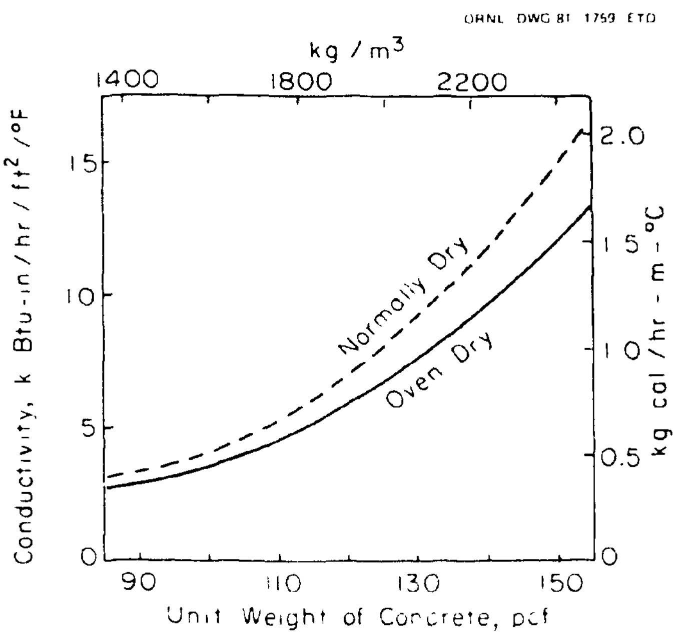

Fig. 49. Relation between unit weight of lightweight aggregate concrete and thermal conductivity. Source: ACI Committee 213, "Guide for Structural Lightweight Aggregate Concrete," *Proc. J. Am. Concr. Inst.* 64(8), 433–69, Detroit (August 1967).

(2) aggregate concretes that are made with lightweight aggregates (expanded clay, shale, slate, slag, sintered fly ash, perlite, vermiculite, pumice, scoria, or tuff) conforming to ANSI/ASTM C 332-77a (Ref. 231). Batching, mixing, placing, and curing should be done in accordance with procedures recommended in Refs. 229 and 230. Figures 50 (Ref. 232), 51 (Ref. 229), and 52 (Ref. 229), and Table 29 (Ref. 229) present typical results from cellular concrete's strength, modulus of elasticity, coefficient of thermal expansion, and thermal conductivity. Properties for lightweight aggregate concrete[229] include: (1) compressive strengths ranging from 10 to 12 MPa (1400 to 1700 psi) [1120 kg/m^3 (70 lb/ft^3)] to 14 to 17.5 MPa (2000 to 2500 psi) [1440 kg/m^3 (90 lb/ft^3)] for expanded shale and clay aggregate concretes; (2) >17.5 MPa (>2500 psi) [1280 to 1440 kg/m^3 (80 to 90 lb/ft^3)] for expanded shale aggregate concrete; (3) modulus of elasticity related to unit weight (Fig. 51); and (4) thermal conductivity values as a function of density and mix moisture content. [Typical values for lightweight aggregate concretes are presented

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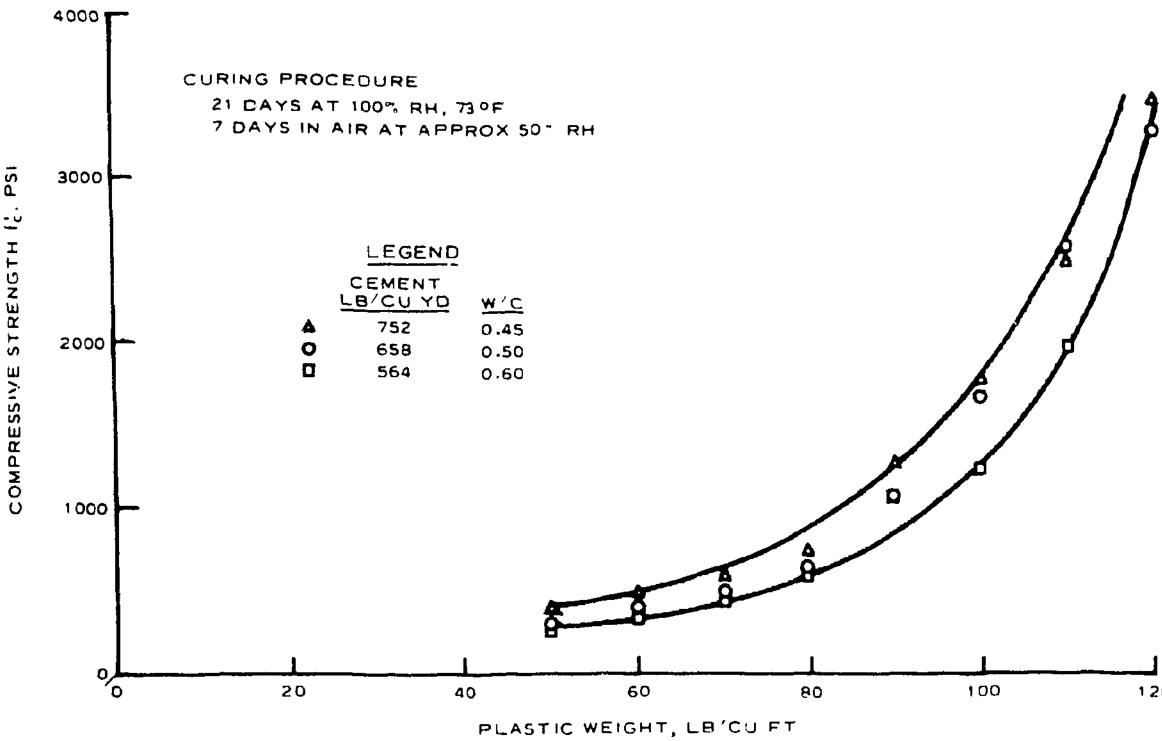

Fig. 50. Compressive strength as a function of wet unit weight. Source: "Standard Specification for Lightweight Aggregates for Insulating Concretes," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 332-77a (1979).

in Fig. 53 (Ref. 233).] Moderate-strength lightweight concretes are used as fills, cast-in-place elements, and precast elements.

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 (55 to 105 lb/ft^3)] and service temperature [927 to 1760°C (1700 to 3200°F)], as shown in Table 23 (Ref. 213).

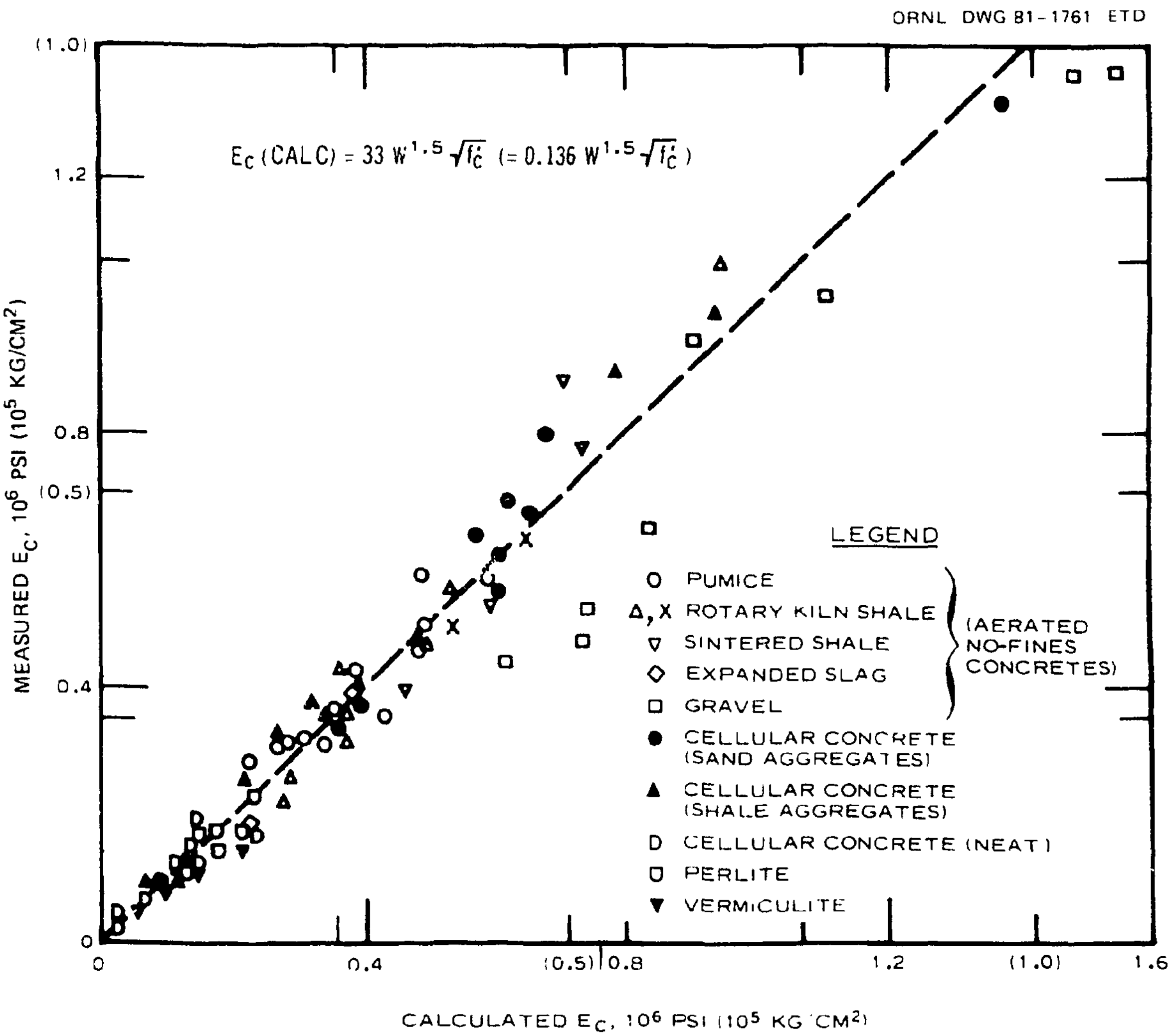

Fig. 51. Measured vs calculated modulus of elasticity. Source: ACI Committee 523, "Guide for Cellular Concretes Above 50 pcf, and for Aggregate Concretes Above 50 pcf with Compressive Strengths Less Than 2500 psi," *Proc. J. Am. Concr. Inst.* 72(2), Detroit (February 1975).

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 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

Table 29. Thermal conductivity of lightweight aggregate concrete as a function of oven-dry density

Oven-dry density [pcf (kg/m^3)]	Thermal conductivity (k factor) for oven-dry concrete [$Btu \cdot in./h \cdot ft^2 \cdot {}^\circ F$ ($MW/cm \cdot {}^\circ C$)]	Adjustment factor[a,b] (times k factor) for normal air dry concrete
50 (800)	1.40 (2.02)	1.31
60 (960)	1.70 (2.45)	1.25
70 (1120)	2.10 (3.03)	1.22
80 (1280)	2.50 (3.61)	1.20
90 (1440)	3.00 (4.33)	1.17
100 (1600)	3.60 (5.19)	1.16
110 (1760)	4.25 (6.13)	1.15
120 (1920)	5.20 (7.50)	1.13
140 (2240)	9.00 (12.99)	1.11

[a]Their intended use is as typical values (not specification) for insulating concretes in normal use by designers who require such values in preliminary designs. For the conductivity of a specific concrete, the user may obtain the value supplied by the producer or secure the results of tests.

[b]Adjustment factor times k factor will result in an adjusted k factor applicable to normal air-dry (not oven-dry) concrete. A constant 5% moisture by volume or 3.12 pcf (50 kg/m^3) is assumed at all densities.

Source: ACI Committee 523, "Guide for Cellular Concretes Above 50 pcf, and for Aggregate Concretes Above 50 pcf with Compressive Strengths Less Than 2500 psi," *Proc. J. Am. Concr. Inst.* 72(2), Detroit (February 1975).

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

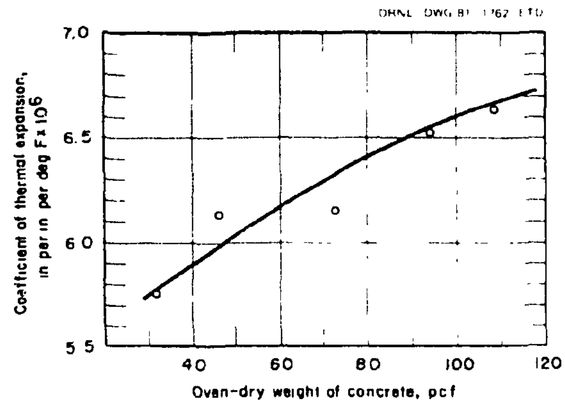

Fig. 52. Coefficient of thermal expansion vs oven-dry weight for a cellular concrete made with siliceous fine aggregate. Source: ACI Committee 523, "Guide for Cellular Concretes Above 50 pcf, and for Aggregate Concretes Above 50 pcf with Compressive Strengths Less Than 2500 psi," *Proc. J. Am. Concr. Inst.* 72(2), Detroit (February 1975).

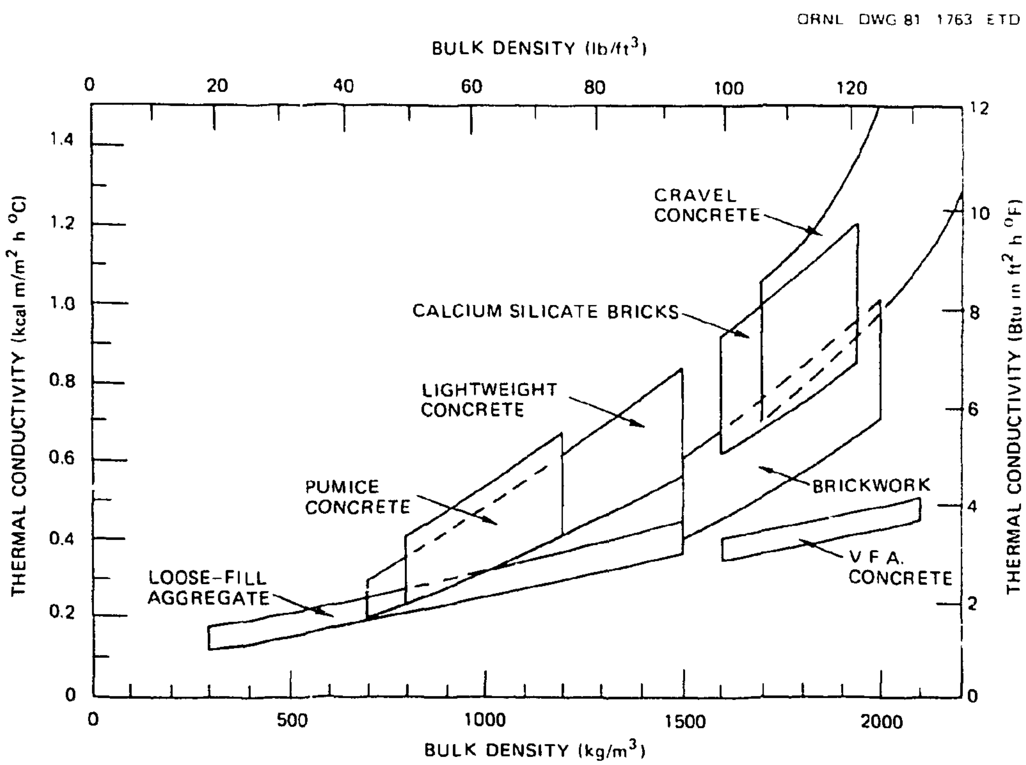

Fig. 53. Thermal conductivity as a function of bulk density for lightweight aggregate concretes. Source: K. Ott, "The Technology of Structural Lightweight Concrete Made with Vitreous Fine Aggregates," Paper 6, Session B, Vol. 1, *Proceedings of the First International Congress on Lightweight Concrete*, Cement and Concrete Association, London (May 27–29, 1968).

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.[234,235] Table 30 (Ref. 211) presents some typical lightweight aggregate materials used in refractory-insulating concretes, and Table 31 (Ref. 212) presents maximum service temperatures of selected aggregates mixed with calcium-aluminate cements under optimum conditions.

Field mixes with insulating concretes are made with presoaked aggregates. Because specified mix proportions are based on dry materials, the actual batch mixes require corrections. This is done by checking the volume after presoaking a premeasured amount of dry aggregate; dry expanded shale will have an absorption of 7 to 15% that will require 74 to 158 L of water per cubic meter of aggregate for saturation (15 to 32 gal/yd^3). Recommended procedures for batching, mixing, and transporting lightweight aggregate refractory-insulating concretes should be followed.[228,230] Refractory concretes may be installed by troweling, casting, or shotcreting.

Table 30. Some typical lightweight aggregate materials used in refractory insulating concretes

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," *Refractory Concrete*, SP 57-2, American Concrete Institute, Detroit (1978).

Table 31. 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 31 (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, personal communication to D. J. Naus, Oak Ridge National Laboratory (Oct. 6, 1980).

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. Generally, the modulus of rupture varies between 0.3 and 2.8 MPa (50 and 400 psi), depending on the material's weight and cement content. (Table 32 presents hot and cold modulus-of-rupture values

Table 32. Hot and cold modulus of rupture values for an expanded clay insulating refractory concrete

Temperature [°F (°C)]	Modulus of rupture values [psi (MPa)]	
	At temperature	At room temperature after firing
230 (110)	350 (2.4)	350 (2.4)
1000 (538)	300 (2.1)	*a*
1500 (816)	250 (1.7)	250 (1.7)
2000 (1093)	210 (1.4)	225 (1.6)
2500 (1371)	240 (1.7)	470 (3.2)
2700 (1482)	90 (0.6	800 (5.5)

[a] Not determined.

Source: W. T. Bakker, "Properties of Refractory Concretes," *Refractory Concrete*, SP 57-2, American Concrete Institute, Detroit (1978).

for an expanded fire clay aggregate mix.[211]) Cold compressive strengths vary between 1.4 and 3.4 MPa (200 and 500 psi) for materials having densities up to 800 kg/m^3 (50 lb/ft^3) and between 6.9 and 17.2 MPa (1000 and 2500 psi) for materials having a density of 1200 to 1600 kg/m^3 (75 to 100 lb/ft^3).[211]

3. CONCRETE MATERIAL SYSTEMS — SUMMARY AND CONCLUSIONS

3.1 Summary

Concrete material systems were reviewed with respect to constituents, mix design, placing, curing, and strength evaluations. Although somewhat limited for extreme environmental conditions, typical concrete property data are presented for both normal and extreme environments (elevated temperature, multiaxial, irradiation). Data shortcomings were identified with respect to interpretation of concrete properties under (1) elevated temperatures (samples tested hot or cold, moisture migration state either free or restricted, specimens loaded or unloaded during heating, constituents and mix proportions varied, specimen size inconsistent, degree of specimen hydration varied, heatup rate and heat-soak duration varied, and limited basalt and serpentine aggregate concrete data available); (2) multiaxial loadings (different materials used, specimen preparation techniques varied, platen-specimen interface influenced results, instrumentation techniques varied and may not have been of sufficient accuracy, and elevated temperature data are extremely limited); and (3) irradiation (technical and experimental difficulties encountered in conducting tests, different materials used, mix proportions varied, specimen size varied, temperatures differed, and cooling and dry conditions). Specialty concrete material systems (high-strength concrete, fibrous concrete, polymer concrete, elevated temperature concrete, lightweight concrete, and refractory concrete) were examined and property data presented.

3.2 Conclusions

Based on an overview of concrete material systems for application to PCPVs, the following conclusions can be derived:

1. Portland cement concrete property data for normal operating conditions are adequate (some additional multiaxial property data are desirable).
2. Portland cement concrete property data for elevated temperature conditions require development, especially with respect to basalt and serpentine aggregate concretes.

3. Multiaxial concrete property data, especially at elevated temperatures, are required for development of improved constitutive relations for concrete.
4. Additional data on the effects of irradiation on concrete properties are required.
5. High-strength concretes (>41.4 MPa) have application to PCPVs, but they need to be investigated to define their mechanical behavior more clearly, develop fracture criteria, and assess current ACI Code applicability.
6. Fibrous concrete exhibits significant potential to effect improved PCPV performance at reduced costs as a result of property enhancements provided by the fiber reinforcement.
7. Polymer concrete and lightweight concrete material systems apparently have limited application to PCPVs because of high costs and poor hot strengths of the former and low strengths of the latter.
8. Refractory concretes offer potential as thermal barrier (hot liner) materials for PCPVs.

REFERENCES

1. "Code for Concrete Reactor Vessels and Containments," Nuclear Power Plant Components, *ASME Boiler and Pressure Vessel Code*, Section III, Division 2 (1975).

2. British Standards Institution, *Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors*, BS 4975 (July 1973).

3. "Nuclear Reactor Vessels of Prestressed Concrete with Metal Reinforcements," *J. Offic. Republ. Fr.*, 6119–128 (June 1970).

4. American Concrete Institute Committee 116, *Cement and Concrete Terminology*, ACI SP-19 (1967).

5. "Standard Specification for Portland Cement," Part 13 of *Annual Book of ASTM Standards*, ANSI/ASTM C 150-78a (1979).

6. Portland Cement Assoc., *Design and Control of Concrete Mixtures*, Skokie, Ill. (1968).

7. "Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)," Part 13 of *Annual Book of ASTM Standards*, ANSI/ASTM C 109-77 (1979).

8. "Standard Test Method for Time of Setting of Hydraulic Cement by Vicant Needle," Part 13 of *Annual Book of ASTM Standards*, ANSI/ASTM C 191-77 (1979).

9. "Standard Test Method for Chloride Ion in Industrial Water and Industrial Waste Water," Part 31 of *Annual Book of ASTM Standards*, ANSI/ASTM D 512-67 (1979).

10. "Standard Specification for Concrete Aggregates," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 33-78 (1979).

11. "Standard Specification for Lightweight Aggregates for Structural Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 330-77 (1979).

12. "Standard Specification for Aggregates for Radiation-Shielding Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 637-73 (1979).

13. "Code for Concrete Reactor Vessels and Containments," Nuclear Power Plant Components, *ASME Boiler and Pressure Vessel Code*, Section III, Division 2, Subsection CB-2224.1, p. 50 (1975).

14. "Standard Specification for Chemical Admixtures for Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 494-79 (1979).

15. "Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 618-78 (1979).

16. "Standard Specification for Air-Entraining Admixtures for Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 260-77 (1979).

17. R. D. Browne, "Properties of Concrete in Reactor Vessels," Group C, Paper 13, Conference on Prestressed Concrete Pressure Vessels, Westminster, S.W.I., March 1967.

18. C. P. Tan, *Prestressed Concrete in Nuclear Pressure Vessels. A Critical Review of Current Literature*, ORNL-4227 (May 1968).

19. American Concrete Institute, Part I of "Materials and Properties of Concrete, Construction Practices and Inspection, Pavements and Slabs," *ACI Manual of Concrete Practice*, 1976.

20. "Standard Specification for Ready-Mixed Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 94-78a (1979).

21. American Concrete Institute, "Recommended Practice for Cold Weather Concreting," *ACI Manual of Concrete Practice*, ACI 306-66, 1976.

22. American Concrete Institute, "Recommended Practice for Hot Weather Concreting," *ACI Manual of Concrete Practice*, ACI 308-71, 1976.

23. American Concrete Institute, "Recommended Practice for Consolidation of Concrete," *ACI Manual of Concrete Practice*, ACI 309-72, 1976.

24. American Concrete Institute, "Recommended Practice for Curing Concrete," *ACI Manual of Concrete Practice*, ACI 308-71, 1976.

25. "Code for Concrete Reactor Vessels and Containments," Nuclear Power Plant Components, *ASME Boiler and Pressure Vessel Code*, Section III, Division 2, Table CB-5200-1, p. 121 (1975).

26. Fédération Internationale de la Précontrainte, *The Design and Construction of Prestressed Concrete Reactor Vessels*, FIP/3/3 (March 1978).

27. A. M. Neville, *Creep of Concrete: Plain, Reinforced and Prestressed*, North Holland Publishing Co., Amsterdam, 1970.

28. *Proceedings of Conference on Prestressed Concrete Pressure Vessels at Church House, Westminster, S.W.I.*, Inst. Civ. Eng., London (1968).

29. "Experience in the Design, Construction and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors," *Proceedings*, Conference at University of York, Inst. Mech. Eng., London (1976).

30. A. M. Neville, *Properties of Concrete*, Pitman, London, 1970.

31. American Concrete Institute, *Concrete for Nuclear Reactors*, Special Publication SP-34, Vol. I-III, Detroit (1972).

32. American Concrete Institute, "Shrinkage and Creep of Concrete 1966–1970 (Annotated)," Bibliography No. 10, Detroit (1972).

33. American Concrete Institute, *Behavior of Concrete Under Temperature Extremes*, Special Publication SP-39, Detroit (1973).

34. ISMES, *Seminar on Concrete Structures Subjected to Triaxial Stresses*, Bergamo, Italy, May 1974.

35. American Concrete Institute, *Temperature and Concrete*, Special Publication SP-25, Detroit (1971).

36. American Concrete Institute, *Douglas McHenry International Symposium on Concrete and Concrete Structures*, Publication SP-55, Detroit (1978).

37. R. Ausangee et al., "Properties of Concrete and Creep Under Temperature," Paper 9 presented at the Commission of the European Communities Second Information Meeting on Prestressed Concrete Reactor Pressure Vessels and Their Thermal Isolation Meeting, Brussels (November 1969).

38. G. J. Komendant et al., *Study of Concrete Properties for Prestressed Concrete Reactor Vessels. Final Report — Part I*, UC SESM 75-2, University of California, Berkeley (March 1975).

39. G. J. Komendant et al., *Study of Concrete Properties for Prestressed Concrete Reactor Vessels. Final Report — Part II, Creep and Strength Characteristics of Concrete at Elevated Temperatures*, UC SESM 76-3, University of California, Berkeley (April 1976).

40. F. S. Ople et al., *Proposed Study on Properties of Concrete for Prestressed Concrete Reactor Vessel Usage*, GA-8305, Gulf General Atomic, San Diego, California (April 1968).

41. R. K. Nanstad, *A Review of Concrete Properties for Prestressed Concrete Pressure Vessels*, ORNL/TM-5497 (October 1976).

42. "Part H. Prestressed Concrete Pressure Vessels," *Proc. First Int. Conf. Struct. Mech. Reactor Technol.*, Berlin, September, 1971.

43. "Part H. Prestressed Concrete Pressure Vessels," *Proc. Second Int. Conf. Struct. Mech. Reactor Technol.*, Berlin, September 1973.

44. "Part H. Prestressed Concrete Pressure Vessels," *Proc. Third Int. Conf. Struct. Mech. Reactor Technol.*, London, September 1975.

45. "Part H. Prestressed Concrete Pressure Vessels," *Proc. Fourth Int. Conf. Struct. Mech. Reactor Technol.*, San Francisco, August 1977.

46. "Part H. Prestressed Concrete Pressure Vessels," *Proc. Fifth Int. Conf. Struct. Mech. Reactor Technol.*, Berlin, August 1979.

47. D. Langon et al., "The Design of Pod Boiler Pressure Vessels with Particular Reference to Hartlepool Nuclear Power Station," Paper H 4/4 in *Proc. First Int. Conf. Struct. Mech.*, Berlin, September 1971.

48. British Standards Institution, *Unified Code of Practice for the Structural Use of Concrete*, Draft 69/23C07 (September 1969).

49. R. D. Browne and R. Blundell, "The Behavior of Concrete in Prestressed Concrete Pressure Vessels," Paper H 1/1 in *Proc. First Int. Conf. Struct. Mech.*, Berlin, September 1971.

50. D. L. Bloem, "Concrete Strength in Structures," *Proc. J. Am. Concr. Inst.* 65(3) (March 1968).

51. N. Petersons, "Strength of Concrete in Finished Structures," *Transactions No. 232*, Swedish Cement and Concrete Research Institute at the Royal Institute of Technology, Stockholm (1964).

52. D. L. Bloem, "Concrete Strength Measurement — Cores vs Cylinders," *ASTM Proc.* 65, 668–96 (1965).

53. R. H. Campbell and R. E. Tobin, "Core and Cylinder Strengths of Natural and Lightweight Concrete, *Proc. J. Am. Concr. Inst.* 64(4) (April 1967).

54. W. H. Price, "Factors Influencing Concrete Strength," pp. 417–32 in *Proc. J. Am. Concr. Inst.* (February 1951).

55. J. W. Dougill, "Structural Properties of Concrete: A Review," Lecture X, Prestressed Concrete Nuclear Reactor Structures (March 1968).

56. G. E. Troxell et al., *Composition and Properties of Concrete*, 2d ed., McGraw-Hill, New York, 1968.

57. J. C. Marechal, "Variations in the Modulus of Elasticity and Poisson's Ratio with Temperature," Paper SP 34-27 in Special Publication SP-34, Vol. I-III, American Concrete Institute, Detroit (1972).

58. "Standard Method of Sampling Fresh Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 172-71 (1979).

59. "Standard Method for Making and Curing Concrete Test Specimens in the Field," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 31-69 (1979).

60. "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 39-72 (1979).

61. "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 469-65 (1979).

62. "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 42-77 (1979).

63. "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 496-71 (1979).

64. "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 78-75 (1979).

65. "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading)," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 293-79 (1979).

66. H. F. Gonnerman and E. C. Shuman, "Compression, Flexure, and Tension Tests of Plain Concrete," *ASTM Proc.* 28(II), 527–73 (1928).

67. C. B. Oland et al., *Final Report of Comprehensive Testing Program for Concrete at Elevated Temperatures*, ORNL/BRP-80-5 (1980).

68. J. A. Hofbeck et al., "Shear Transfer in Reinforced Concrete," *Proc. J. Am. Concr. Inst.* 66(2) (February 1969).

69. A. P. Clark, "Bond of Concrete Reinforcing Bars," pp. 17–24 in *Proc. J. Am. Concr. Inst.* (1951).

70. R. E. Davis et al., "Some Factors Influencing the Bond Between Concrete and Reinforcing Steel," *ASTM Proc.* 38(II), 394–409 (1938).

71. C. A. Menzel, "Some Factors Influencing Results of Pull-Out Bond Tests," pp. 517–44 in *Proc. J. Am. Concr. Inst.* (1939).

72. "Standard Test Method for Comparing Concretes on the Basis of the Bond Developed with Reinforcing Steel," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 234-71 (1979).

73. "Mass Concrete for Dams and Other Massive Structures," *Proc. J. Am. Concr. Inst.* 67(4) (April 1970).

74. U.S. Bureau of Reclamation, *Concrete Manual*, 7th ed., Denver, Colorado, 1963.

75. R. Philleo, "Some Physical Properties of Concrete at High Temperature," *Proc. J. Am. Concr. Inst.* 54(10) (April 1958).

76. H. C. Hirth et al., "Final Report on Thermal Properties of Concrete at Elevated Temperature," private communication, University of California, Berkeley (July 1980).

77. L. J. Mitchell, *Thermal Properties*, ASTM Special Technical Publication 169. 129–35 (October 1962).

78. S. L. Meyers, "How Temperature and Moisture Changes May Affect the Durability of Concrete," *Rock Products*, 153–57 (August 1951).

79. U.S. Army Corps of Engineers, "U.S. Corps of Engineers Standard CRD-C36-49," *Handbook for Concrete and Cement*, Vicksburg, Mississippi, 1949.

80. T. Z. Harmathy, "Variable-State Methods of Measuring the Thermal Properties of Solids," *J. Appl. Phys.* 34(4) (April 1964).

81. U.S. Bureau of Reclamation, "Thermal Properties of Concrete," Part VII of *Boulder Canyon Project Final Report*, Bulletin 7, Denver (1940).

82. U.S. Bureau of Reclamation, *Materials Laboratory Procedures Manual*, Denver (1951).

83. U.S. Army Corps of Engineers, "U.S. Corps of Engineers Standard CRD-C124-51," *Handbook for Concrete and Cement*, Vicksburg, Mississippi, 1949.

84. R. W. Carlson, "Drying Shrinkage of Concrete as Affected by Many Factors," *ASTM Proc.* 38(II), 419–37 (1938).

85. T. Matsumato, "Study of the Effect of Moisture Content Upon the Expansion and Contraction of Plain and Reinforced Concrete," *Univ. Ill. Eng. Exp. Sta. Bull. No. 126* (Dec. 5, 1921).

86. U.S. Bureau of Reclamation, *A 10-Year Study of Creep Properties of Concrete*, Concrete Laboratory Report No. SP-38, Denver, Colorado (July 28, 1953).

87. I. Ali and C. E. Kesler, *Creep in Concrete With and Without Exchange of Moisture With the Environment*, Department of Theoretical and Applied Mechanics, University of Illinois, Theoretical and Applied Mechanics Report 641 (1963).

88. A. M. Neville, "Discussion on the Influence of Sand Concentrations on Deformations of Mortar Bars Under Low Stresses," *J. Am. Concr. Inst.* 59(2), 931–34 (June 1962).

89. G. E. Troxell et al., "Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete," *ASTM Proc.* 48, 1101–120 (1958).

90. A. M. Neville, "The Influence of Cement on Creep of Concrete and Mortar," *J. Prestressed Concr. Inst.* 2, 12–18 (March 1958).

91. K. S. Gopalakrishnan et al., "Creep Poisson's Ratio of Concrete Under Multiaxial Compression," *Proc. J. Am. Concr. Inst.* 66, 1008–1020 (1969).

92. K. W. Nasser and R. P. Lohtia, "Creep of Mass Concrete at High Temperatures," *Proc. J. Am. Concr. Inst.* 68(4), 276–81 (April 1971).

93. H. G. Geymayer, "Effect of Temperature on Creep of Concrete: A Literature Review," Paper SP 34-31 in Special Publication SP-34, Vol. I-III, American Concrete Institute, Detroit (1972).

94. W. Fürste et al., "Prestressed Concrete Reactor Vessels for Nuclear Reactor Plants Compared to Thick-Walled and Multilayer Steel Vessels," Second International Conference on Pressure Vessel Technology (Part 1), San Antonio, Texas (October 1973).

95. S. E. Pihlajavaara, "An Analysis of the Factors Exerting Effect on Strength and Other Properties of Concrete at Elevated Temperature," Paper SP 34-19 in Special Publication SP-34, Vol. I-III, American Concrete Institute, Detroit (1972).

96. M. S. Abrams, "Compressive Strength of Concrete at Temperatures to 1600°F," *SP-25 Temperature and Concrete*, American Concrete Institute, 33–58 (1971).

97. D. A. Campbell-Allen et al., "An Investigation of the Effect of Elevated Temperatures on Concrete for Reactor Vessels," *Nucl. Struct. Eng.* 2, 382–88, Amsterdam (1965).

98. Portland Cement Association, "Effect of Long Exposure of Concrete to High Temperatures," *Concrete Information*, ST 32-3-53.

99. D. J. Hannant, "The Effects of Heat on Concrete Strength," *Eng.* 197(5105), 302, London (February 1963).

100. T. Harada et al., "Strength, Elasticity and Thermal Properties of Concrete Subjected to Elevated Temperatures, Paper SP 34-21 in Special Publication SP-34, Vol. I-III, American Concrete Institute, Detroit (1972).

101. D. T. Lankard et al., "Effects of Moisture Content on the Structural Properties of Portland Cement Concrete Exposed to Temperatures Up to 500°F," *SP-25 Temperature and Concrete*, American Concrete Institute Special Publication, 59–102 (1971).

102. H. L. Malhotra, "The Effect of Temperature on the Compressive Strength of Concrete," *Mag. Concr. Res.* 8(2), 382–88 (1965).

103. J. C. Marechal, "Variations in the Modulus of Elasticity and Poisson's Ratio with Temperature," Paper SP 34-27 in Special Publication SP-34, Vol. I-III, American Concrete Institute, Detroit (1972).

104. K. W. Nasser and R. P. Lohtia, "Mass Concrete Properties at High Temperatures," *J. Am. Concr. Inst.* 68(3), 180–81 (March 1971).

105. S. Ohgishi et al., "On Properties of Magnetite and Serpentine Concrete at Elevated Temperatures for Nuclear Reactor Shielding," Paper SP 34-57 in Special Publication SP-34, Vol. I-III, American Concrete Institute, Detroit (1972).

106. F. J. Roux, *Concrete at Elevated Temperatures*, Doctoral Thesis, University of Capetown, South Africa (August 1974).

107. N. G. Zoldners, "Effect of High Temperatures on Concrete Incorporating Different Aggregates," *ASTM Proc.* 60, 1087–108 (1960).

108. American Concrete Institute, *Behaviour of Concrete Under Temperature Extremes*, Publication SP-39 (1973).

109. G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperatures," *Civ. Eng. Nucl. Power*, Vol. 1, ASCE National Convention, Boston, Mass. (April 1979).

110. J. Glucklich, "The Effect of Microcracking on Time-Dependent Deformations and Long-Term Strength of Concrete," *Proceedings First International Conference on the Structure of Concrete and Its Behaviour Under Load*, Cement and Concrete Association, London, 1968.

111. E. Hognestad, "A Study of Combined Bending and Axial Load in Reinforced Concrete Members," *University of Illinois Engineering Experiment Station, Bull.* No. 399, Vol. 49 (November 1951).

112. D. C. Kent and R. Park, "Flexural Members With Confined Concrete," *ASCE Proc.* 97(ST 7), 1969–990 (July 1971).

113. R. Baldwin and M. A. North, "A Stress-Strain Relationship for Concrete at High Tempratures," *Mag. Concr. Res.* 25(85) (December 1973).

114. K. W. Nasser and H. M. Marzouk, "Properties of Mass Concrete Containing Fly Ash at High Temperatures," pp. 537–50 in *Proc. J. Am. Concr. Inst.* (April 1979).

115. H. Weigler and R. Fischer, "Influence of High Temperatures on Strength and Deformations of Concrete," Paper SP 34-26 in Special Publication SP-34, Vol. I-III, American Concrete Institute, Detroit (1972).

116. *THTR-Spannbetonbehälter Bericht Nr. 1 Nachweise für den Gebrouchszustand.*

117. J. P. Magnas and A. Audibert, "Critères de Résistance du Béton sous Sollicitations Multiaxiales," *Annales de l'ITBTP*, Suppl. du No. 287 (November 1971).

118. D. Costes, "Stress Criteria for Nuclear Vessel Concrete," Paper 138/75, Experience in Design, Construction, and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors, University of York, Great Britain (Sept. 8–12, 1975).

119. German Standardization Committee, "Reactor Pressure Vessels Made of Prestressed Concrete. A Study of the Development of a Standard" (DNA) (June 1972).

120. D. Linse, "Experimental Apparatus for Determining the Triaxial Strength of Concrete: First Results of Experiments," *Cem. Concr. Res.* 3, 445–57 (1973), ORNL-TR-2878.

121. R. H. Atkinson and H. Y. Ko, "A Fluid Cushion, Multiaxial Cell for Testing Cubical Rock Specimens," *Int. J. Rock Mech. Min. Sci.* 10, 351–61 (1973).

122. E. Andrenes, Response of Mortar to Biaxial Compression, M.S. thesis, University of Colorado, 1974.

123. M. A. Taylor and B. K. Patel, "The Influence of Path Dependency and Moisture Conditions on the Biaxial Compression Envelope for Normal Weight Concrete," *J. Am. Concr. Inst.* 71(12), 627 (December 1974).

124. H. B. Kupfer, "Nonlinear Behavior of Concrete in Biaxial Stress," *Beton-Stahlbetonbau* 11, 269–74 (1973), ORNL-TR-2909.

125. T. N. W. Akroyd, "Concrete Under Triaxial Stress," *Mag. Concr. Res.* 13(39), 111–18 (November 1961).

126. J. Isenberg, "Strength of Concrete Under Combined Stress," *Civ. Eng. Public Works Rev.* (October 1965).

127. F. E. Richart, A. Brandtzaeg, and R. L. Brown, "A Study of the Failure of Concrete Under Combined Compressive Stresses," *Eng. Exp. Sta. Bull.* No. 185, University of Illinois-Urbana (Nov. 20, 1928).

128. J. Chinn and R. M. Zimmerman, *Behavior of Plain Concrete Under Various High Triaxial Compression Loading Conditions*, WL TR 64-163, Air Force Weapons Laboratory (August 1965).

129. C. D. Goode and M. A. Helmy, "The Strength of Concrete Under Combined Shear and Direct Stress," *Mag. Concr. Res.* 19(59), 105 (June 1967).

130. N. J. Gardner, "Triaxial Behavior of Concrete," *J. Am. Concr. Inst.* 66(2), 136 (February 1969).

131. H. K. Gerstle et al., "Strength of Concrete Under Multiaxial Stress States," Paper SP 55-5 in Publication SP-55, American Concrete Institute, Detroit (1978).

132. G. Schickert and H. Winkler, "Results of Test Concerning Strength and Strain of Concrete Subjected to Multiaxial Compressive Stresses," *Deutscher Auschuss fuer Stahlbeton*, Heft No. 277, Berlin (1977).

133. H. Winkler, "Pruefanlage fuer Dreizxiale Festigkeitsuntersuchungen an Betonkoerpern," *Materialpruefund* 18(4) (1976).

134. P. Bertacchi and R. Bellotti, "Experimental Research on Deformation and Failure of Concrete Under Triaxial Loads," *Proceedings Rilem Symposium on the Deformation and Rupture of Solids Subject to Multiaxial Stresses*, Vol. 1 (1972).

135. G. W. D. Vile, "Combined Stress Testing Machine for Concrete," *The Engineer*, London (July 9, 1965).

136. J. B. Newman, "Apparatus for Testing Concrete Under Multiaxial States of Stress," *Mag. Concr. Res.* 26(89) (December 1974).

137. D. Linge, "Loesung versuchstechnischer Fragen bei der Ermittlung des Festigkeits- und Verformungsverhaltens von Beton unter dreiachsiger Belastung," *Deutscher Ausschuss fuer Stahlbeton*, Heft No. 292, Berlin (1978).

138. R. M. Zimmerman and L. A. Traina, "Strength and Deformation Response of Concrete Under Multiaxial Loadings -- Cooperative Project," Final Report EN676-12097, Eng. Exp. Sta., New Mexico State University (May 1977).

139. M. A. Taylor et al., "Biaxial Compressive Behavior of Fiber-Reinforced Mortar," *Proc. J. Am. Concr. Inst.* 72(9), 496—501 (September 1975).

140. H. Y. Ko and S. Sture, "Three-Dimensional Mechanical Characterization of Anisotropic Composites," *J. Compos. Mater.*, Vol. 8 (1974).

141. H. B. Kupfer and K. H. Gerstle, "Behavior of Concrete Under Biaxial Stresses," *Trans. ASCE, J. Eng. Mech.* No. EM4 (August 1973).

142. F. K. Garas, "Strength of Concrete Under Different States of Stress," Paper SP 34-18 in Special Publication SP-34, Vol. I-III, Detroit (1972).

143. H. K. Hilsdorf et al., "The Effects of Nuclear Radiation on the Mechanical Properties of Concrete," Paper 55-10 in Publication SP-55, American Concrete Institute, Detroit (1978).

144. B. T. Kelley et al., "The Effects of Reactor Radiation on Concrete," *Proceedings of the Second Conference on PCPVs and Their Thermal Insulation, Brussels*, pp. 237–66 (November 1969).

145. D. C. McDowall, *The Effect of Gamma Irradiation on the Creep Properties of Concrete*, Euratom Report No. EUR 4751.

146. R. G. Clark, *Radiation Damage to Concrete*, Hanford Atomic Products Operation HW-56195 (Mar. 31, 1958).

147. T. V. Blosser et al., *A Study of the Nuclear and Physical Properties of the ORNL Graphite Reactor Shield*, ORNL-2195 (August 1958).

148. M. F. Elleuch et al., "Effects of Neutron Radiation on Special Concretes and Their Components, Paper 34-51 in Special Publication SP-34, Vol. I-III, Detroit (1972).

149. H. S. Davis, "N-Reactor Shielding," Paper 34-52 in Special Publication SP-34, Vol. I-III, Detroit (1972).

150. S. Granata and A. Montagnini, "Studies on Behavior of Concretes Under Irradiation," Paper SP 34-53 in Special Publication SP-34, Vol. I-III, Detroit (1972).

151. S. C. Alexander, *Effects of Irradiation on Concrete, Final Results, Atomic Energy Research Establishment*, Harwell (1963).

152. A. W. Ch. Batten, *Effect of Irradiation on the Strength of Concrete, Atomic Energy Research Establishment*, Harwell (1960).

153. G. Christiani et al., *Behavior of Plastic Mortar Samples Under Temperature and Radiation Conditions, Commission of European Communities*, Luxembourg (1972).

154. E. Crispino et al., "Behavior of Concrete in the Presence of Thermal Stresses and Radiation," Second Information Meeting on Prestressed Concrete Reactor Vessels and Their Heat Insulation, Brussels (1969).

155. E. Crispino et al., "Irradiation Effects on Reactor Structural Materials," Fourth United Nations International Conference on Peaceful Uses of Atomic Energy (1971).

156. Danish Atomic Energy Commission, *Radiation Damage in Concrete — Measurements on Miniature Specimens of Cement Mortar*, Communication Research Establishment Risö; Reactor Engineering Department (1971).

157. V. B. Dubrovskij et al., "The Effect of Neutron Irradiation on Certain Properties of Refractory Concretes," *At. Energ.* 21, 108–12 (1966).

158. M. R. Elleuch, F. Dubois, and J. Rappenau, "Behavior of Special Shielding Concretes and of Their Constituents Under Neutron Irradiation," Fourth United Nations International Conference on the Peaceful Uses of Atomic Energy, 7 pp. (1971).

159. B. S. Gray, "The Effect of Reactor Radiation on Cements and Concrete," Commission of the European Communities, Luxembourg, pp. 17–39 (1972).

160. T. C. Hansen and L. Erikson, "Temperature Change Effect on Behavior of Cement Paste, Mortar and Concrete Under Load," *Proc. Am. Concr. Inst.* 63(4), 489–504 (April 1966).

161. J. A. Houben, "DeBestraling van Mortelproefstukken" (Radiation of Mortar Specimens), Commission of the European Communities, Brussels, pp. 170–83 (1969).

162. B. T. Kelly and I. Davidson, "Irradiation Effects on Concrete," Group C, Paper 16, Conference on Prestressed Concrete Pressure Vessels at Church House, S.W.I., Inst. of Cir. Eng., London (1968).

163. W. Meyer and H. K. Al-Dujaili, "Radiation Effects on Concrete and Carbonate Compounds," Second Cairo Solid State Conference, Cairo (1973).

164. A. Pedersen, "Radiation Damage in Concrete — Measurements on Miniature Specimen of Cement Mortar," Commission of the European Communities, Luxembourg, pp. 5–16 (1971).

165. B. T. Price, C. C. Horton, and K. T. Spinney, "Radiation Shielding," *International Series of Monograph on Nuclear Energy*, Pergamon Press, New York, pp. 276–78, 1975.

166. C. R. Van der Schaaf, "Effect of Heating and Radiation on Some Properties of Mortar and Concrete Specimens with Different Compositions," Commission of the European Communities, Brussels, pp. 42–71 (1967).

167. J. F. Sommers, "Gamma Radiation Damage of Structural Concrete Immersed in Water," *Health Physics*, 16, 503–508, Pergamon Press, 1969.

168. B. Stôces et al., "The Effect of Radiation on the Mechanical Properties of Concrete," Ceskoslovenska Akademie, Translated from the Czech; Oak Ridge National Laboratory, Purchase Order: 34B-83481, Letter Release No.: T81, STS No.: 14087.

169. R. L. Walker and M. Grotenhuis, *A Summary of Shielding Constants for Concrete*, Argonne National Laboratory, ANL-6443 (November 1961).

170. M. F. Eleuch and F. Dubois, "Les Experiences d'Irradiation des Betons et de leurs Constituants," Commission of the European Communities, pp. 71–96 (1972).

171. S. Granata and A. Montagnini, "Behavior of Plastic Mortar Samples Under Temperature and Radiation Conditions," Information Exchange Meeting Concerning the Results of Concrete Irradiation Programmes, 18 pp. (1971).

172. "High-Strength Concrete," Proceedings of a Workshop Held at University of Illinois at Chicago Circle (Dec. 2–4, 1979).

173. S. Freedman, "High-Strength Concrete," *Mod. Concr.* (October 1970–February 1971).

174. Chicago Committee on High-Rise Buildings, *High-Strength Concrete in Chicago High-Rise Buildings*, Report No. 5, Chicago, Ill. (February 1977).

175. S. Walker and D. L. Bloem, "Effects of Aggregate Size on Properties of Concrete," *Proc. J. Am. Concr. Inst.* 32(3), 283–98, Detroit (September 1960).

176. W. F. Perenchio, *An Evaluation of Some of the Factors Involved in Producing Very High-Strength Concrete*, Research and Development Bulletin RD014, Portland Cement Association, Skokie, Ill. (1973).

177. V. H. Dodson and E. Farkas, "Delayed Addition of Set Retarding Admixtures to Portland Cement Concrete," *ASTM Proc.* 64, 816–29.

178. M. F. Kaplan, "Effects of Incomplete Consolidation on Compressive and Flexural Strength, Ultrasonic Pulse Velocity, and Dynamic Modulus of Elasticity of Concrete," *Proc. J. Am. Concr. Inst.* 31(9), 853–67 (March 1960).

179. P. H. Kaar et al., "Stress-Strain Characteristics of High-Strength Concrete," Paper SP 55-7 in Publication SP-55, American Concrete Institute, Detroit (1978).

180. A. H. Nilson and F. O. Slate, *Structural Properties of Very High-Strength Concrete*, Second Progress Report, Department of Structural Engineering, Cornell University (January 1979).

181. K. L. Saucier et al., *High-Compressive Strength Concrete*, TR-65-16, U.S. Air Force Weapons Laboratory (September 1965).

182. H. W. Brewer, *General Relation of Heat Flow Factors to the Unit Weight of Concrete*, Research and Development Bulletin D114, Portland Cement Association, Skokie, Ill. (1967).

183. R. E. Davis, "A Summary of Investigations of Volume Changes in Cements, Mortars and Concretes Produced by Causes Other Than Stress," *ASTM Proc.* 30(I) (1930).

184. D. G. R. Bonnell and F. C. Harper, "The Thermal Expansion of Concrete," Technical Paper No. 7, *National Building Studies*, Department of Scientific and Industrial Research, Building Research Station, London (1961).

185. ACI Committee 209, "Effects of Concrete Constituents, Environment, and Stress on Creep and Shrinkage of Concrete," Paper SP 27-1, Designing for Effects of Creep, Shrinkage, Temperature in Concrete Structures, American Concrete Institute, Detroit (1971).

186. ACI Committee 318, *ACI Standard, Building Code Requirements for Reinforced Concrete*, ACI 318-71, American Concrete Institute, Detroit (November 1971).

187. P. Klieger, *Early High-Strength Concrete for Prestressing*, Research Bulletin 91, Portland Cement Association, Skokie, Ill. (1958).

188. J. J. Shiedler, *Lightweight Aggregate Concrete for Structural Use*, Development Bulletin D17, Portland Cement Association, Skokie, Ill. (1957).

189. ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete," ACI SP-44 *Fiber Reinforced Concrete*, American Concrete Institute, Detroit (1974).

190. "Dramix Steel Fibers, A New Concrete Reinforcement," Bekaert Steel Wire Corporation, Niles, Ill.

191. E. K. Schrader, "Mix Design Procedure for Fibrous Concrete," Symposium on Fiber Reinforced Concrete Design and Applications, GAI Consultants, Pittsburgh, Pa. (Nov. 28–30, 1979).

192. ACI Committee 544, "Measurement of Properties of Fiber Reinforced Concrete," *Proc. J. Am. Concr. Inst.* 75(7), 283–89 (July 1978).

193. G. R. Williamson, *Fibrous Reinforcement for Portland Cement Concrete*, Technical Report No. 2-40, U.S. Army Corps of Engineers, Ohio River Division Laboratories, Cincinnati (May 1965).

194. G. R. Williamson, *Response of Fibrous-Reinforced Concrete to Explosive Loading*, Technical Report No. 2-48, U.S. Army Corps of Engineers, Ohio River Division Laboratories, Cincinnati (January 1966).

195. K. C. Morel, *The Thermal Conductivity of Steel Fiber Reinforced Portland Cement Mortar*, M.S. Thesis, Clarkson College of Technology, Potsdam, N.Y. (September 1961).

196. C. D. Johnston, "Steel Fiber Reinforced Mortar and Concrete: A Review of Mechanical Properties," Paper SP 44-8, SP-44, *Fiber Reinforced Concrete*, American Concrete Institute, Detroit (1974).

197. S. P. Shah and R. V. Rangan, "Fiber Reinforced Concrete Properties," *Proc. J. Am. Concr. Inst.* 68(2), 126–35, Detroit (February 1971).

198. G. W. Blood, *Properties of Fiber Reinforced Concrete*, M.S. Thesis, The University of Calgary (July 1970).

199. W. Chen and J. L. Carlson, "Stress-Strain Properties of Random Wire Reinforced Concrete," *Proc. J. Am. Concr. Inst.* 68(12), 933–36, Detroit (December 1971).

200. ACI Committee 548, "State-of-the-Art Report," *Polymers in Concrete*, American Concrete Institute, Detroit (1977).

201. H. C. Mehta et al., "Polymer-Impregnated Concrete: Field Studies," *Transp. Eng. J.*, ASCE, 101(TE1) (1975).

202. L. E. Kukacka et al., *Concrete Polymer Materials — Fifth Topical Report*, Brookhaven National Laboratory and U.S. Bureau of Reclamation, BNL No. 50390 and USBR No. REC-ERC-73-12 (December 1973).

203. B. Sopler et al., "A Study of Partially Impregnated Polymerized Concrete Specimens," SP-44, *Polymers in Concrete*, American Concrete Institute, Detroit (1973).

204. D. R. Bloss et al., *Development and Evaluation of High-Strength Polyester Synthetic Concrete*, U.S. Army Construction Engineering Research Laboratory, Technical Report M-2, Champaign, Ill. (1970).

205. P. C. Keijer, *Improvements of Concretes and Mortars by Adding Resins*, General Report on Topic 1a, RILEM International Symposium on Experimental Research on New Developments Brought by Synthetic Resins in Building Technologies, Materials and Structures, 1(3) (May/June 1968).

206. W. O. Nutt, "An Evaluation of Polymer-Cement Composite," *Composites* 1(4) (June 1970).

207. T. Alfrey and E. F. Gurnee, *Organic Polymers*, Prentice-Hall, Englewood Cliffs, N.J., 1967.

208. Southwest Research Institute, Department of Materials Science, San Antonio, Tex. (1977).

209. F. M. Lea, *The Chemistry of Cement and Concrete*, 2d ed., St. Martin's, N.Y., 1956.

210. H. F. W. Taylor, *The Chemistry of Cements*, Vol. 1, Academic, N.Y., 1964.

211. W. T. Bakker, "Properties of Refractory Concretes," *Refractory Concrete*, SP 57-2, American Concrete Institute, Detroit (1978).

212. F. E. Linck, Turnaround Maintenance, Houston, Texas, personal communication to D. J. Naus, Oak Ridge National Laboratory (Oct. 6, 1980).

213. "Standard Classification of Castable Refractories," Part 17 of *Annual Book of ASTM Standards*, ANSI/ASTM C 401-77 (1980).

214. A. V. Briebach, "A Review of Refractory Hydraulic Cements," *J. Brit. Cer. Soc.* 71(7), 153–58.

215. W. T. Bakker et al., "Blast Furnace Gunning in the USA," *Proceedings International Feuerfest Colloquium*, Aachen, Germany (Oct. [illegible]–29, 1971).

216. J. M. McCullough and G. R. Rigby, "Mechanical Properties of Refractory Castables," *J. Brit. Cer. Soc.* 71(7), 233.

217. "Standard Test Method for Young's Modulus of Refractory Shapes by Sonic Resonance," Part 17 of *Annual Book of ASTM Standards*, ANSI/ASTM C 885-78 (1980).

218. "Standard Test Method for Modulus of Rupture of Castable Refractories," Part 17 of *Annual Book of ASTM Standards*, ANSI/ASTM C 268-70 (1980).

219. "Standard Test Method for Modulus of Rupture of Refractory Materials at Elevated Temperatures," Part 17 of *Annual Book of ASTM Standards*, ANSI/ASTM C 583-76 (1980).

220. G. C. Padgett et al., "Stress/Strain Behavior of Refractory Materials at High Temperatures," Research Paper 608, The British Ceramic Association.

221. E. Ruh and A. Renky, "Thermal Conductivity of Castable Refractories," *J. Am. Cer. Soc.* 46(2) (1963).

222. J. F. Wygant and M. S. Crowley, "Effects of High Conductivity Gases on Thermal Conductivity of Insulating Refractory Concrete," *J. Am. Cer. Soc.* 41(5) (1958).

223. J. F. Wygant and M. S. Crowley, "Designing Monolithic Refractory Vessel Linings," *Am. Cer. Soc. Bull.* 43(3) (1964).

224. W. H. Gitzen et al., "Carbon Monoxide Disentegration of Calcium Aluminate Cements in Refractory Castables," *Am. Cer. Soc. Bull.* 43(7) (1964).

225. M. S. Crowley, "Hydrogen-Silica Reactions in Refractories," *Am. Cer. Soc. Bull.* 48(7) (1967).

226. E. R. Fuller et al., "Effect of Hydrothermal Environment on Strength and Erosion Resistance of Castable Refractories," *Proceedings MFPC*, Columbus, Ohio (1976).

227. ACI Committee 523, "Guide for Cast-in-Place Low Density Concrete," *Proc. J. Am. Concr. Inst.* 64(9), 529—37, Detroit (September 1967).

228. ACI Committee 213, "Guide for Structural Lightweight Aggregate Concrete," *Proc. J. Am. Concr. Inst.* 64(8), 433—69, Detroit (August 1967).

229. ACI Committee 523, "Guide for Cellular Concretes Above 50 pcf, and for Aggregate Concretes Above 50 pcf with Compressive Strengths Less Than 2500 psi," *Proc. J. Am. Concr. Inst.* 72(2), Detroit (February 1975).

230. ACI Committee 211, "Recommended Practice for Selecting Proportions for Structural Lightweight Concrete," ACI 211.2-69 in *ACI Manual of Concrete Practice*, 1976.

231. "Standard Specification for Lightweight Aggregates for Insulating Concretes," Part 14 of *Annual Book of ASTM Standards*, ANSI/ASTM C 332-77a (1979).

232. F. C. McCormick, "Rationale Proportioning of Preformed Foam Cellular Concrete," *Proc. J. Am. Concr. Inst.* 64(2), 104—10 (February 1967).

233. K. Ott, "The Technology of Structural Lightweight Concrete Made with Vitreous Fine Aggregates," Paper 6, Session B, Vol. 1, *Proceedings of the First International Congress on Lightweight Concrete*, Cement and Concrete Association, London (May 27—29, 1968).

234. D. R. Lankard, "Steel Fiber Reinforced Refractory Concrete," Paper SP 57-11, American Concrete Institute, Detroit (1978).

235. Y. Ichikawa and L. A. Nelson, "Fiber Reinforced Composite Refractory Insulation System," Paper SP 57-12, American Concrete Institute, Detroit (1978).