Chapter 7

Engineering Properties of Structural Lightweight Concrete

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CHAPTER 7

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CHAPTER 7 ENGINEERING PROPERTIES OF STRUCTURAL LIGHTWEIGHT CONCRETE

7.0 Introduction

Comprehensive reports detailing the properties of Lightweight Concrete and lightweight aggregates have been published by Shideler (1957), Reichard (1964), Holm (1983 and 2006), Carlson (1956), and Valore (1956, 1980). The first three reports dealt with structural-grade concretes, Carlson reported on lightweight aggregate for concrete masonry units, and Valore covered both structural and insulating concretes.

7.1 Compressive Strength

While all structural lightweight aggregates are capable of producing concretes with compressive strengths in excess of 35 MPa (5000 psi), a limited number of lightweight aggregates can be used in concretes that develop cylinder strengths from 48 to greater than 69 MPa (7000 to greater than10,000 psi).

Compressive strengths of 21 to 35 MPa (3000 to 5000 psi) are common for cast-in-place structural lightweight concretes; higher strengths are presently being specified for precast bridge members and offshore applications.

Maximum Strength Ceiling

Concrete will demonstrate a strength "ceiling" where further additions of cementitious materials will not significantly raise the maximum attainable strength. At this point, the strength of the coarse aggregate particle or the quality of the transition zone will determine the limiting strength. After reaching the strength ceiling, normalweight concrete will demonstrate a small positive slope for the strength/binder relationship, while for a strong lightweight concrete, the slope will be significantly less. In concretes containing highly expanded lightweight aggregate, there will be essentially no increase in strength. Figure 7.1 demonstrates that the compressive strength ceiling for the particular 3/4 in. (20.0-mm) maximum size lightweight aggregate tested was about 8,000 psi (55 MPa) at a concrete age of 75 days (Holm 1980a). When the maximum size of this aggregate was reduced to 3/8 in. (10 mm), the concrete strength ceiling significantly increased to more than 10,000 psi (69 MPa). Nevertheless, mixtures incorporating fly ash demonstrated higher concrete strength ceilings at later ages than mixtures without fly ash, as a result of the substantial strength gain of the matrix.

Analyzing strength as a function of the quantity of cementitious binder, however (as shown in Fig. 7.2), reveals that mixtures incorporating binder quantities exceeding an optimum volume are not cost effective (Holm and Bremner 1994). The schematic curves shown are for illustrative purposes only. In some areas it is not unusual to observe an overlap of strength/binder relationship when concretes containing a strong lightweight aggregate are compared with concretes containing a midrange normalweight aggregate.

Strength ceilings of lightweight aggregate will vary considerably depending on the quarry and manufacturing plants. This variation is due to the raw material and the structural characteristics of the pore system developed during the firing process. The aggregate producer's goal is to manufacture a high-quality uniform structural-grade lightweight aggregate that has well-distributed pores of moderate size (5 to 300 μm) surrounded by a strong, relatively crack-free vitreous ceramic matrix. The size, shape, and distribution of the vesicular pores will determine the lightweight aggregate particle compressive and tensile strength. In general, greater pore volume will correlate with lower strength. As shown in the scanning electron micrograph in Figure 6.6 in Chapter 6, a well-developed pore distribution system can be seen in concrete from an exposed mature bridge deck (Holm 1983).

Because the bond of lightweight aggregate to the surrounding matrix is greater than the particle strength, the failure surface is through both aggregate and matrix. Since the tensile strength of a very strong normalweight aggregate particle greatly exceeds the matrix tensile strength, in general the failure surface will pass around the stone coarse aggregate and through the weaker contact zone.

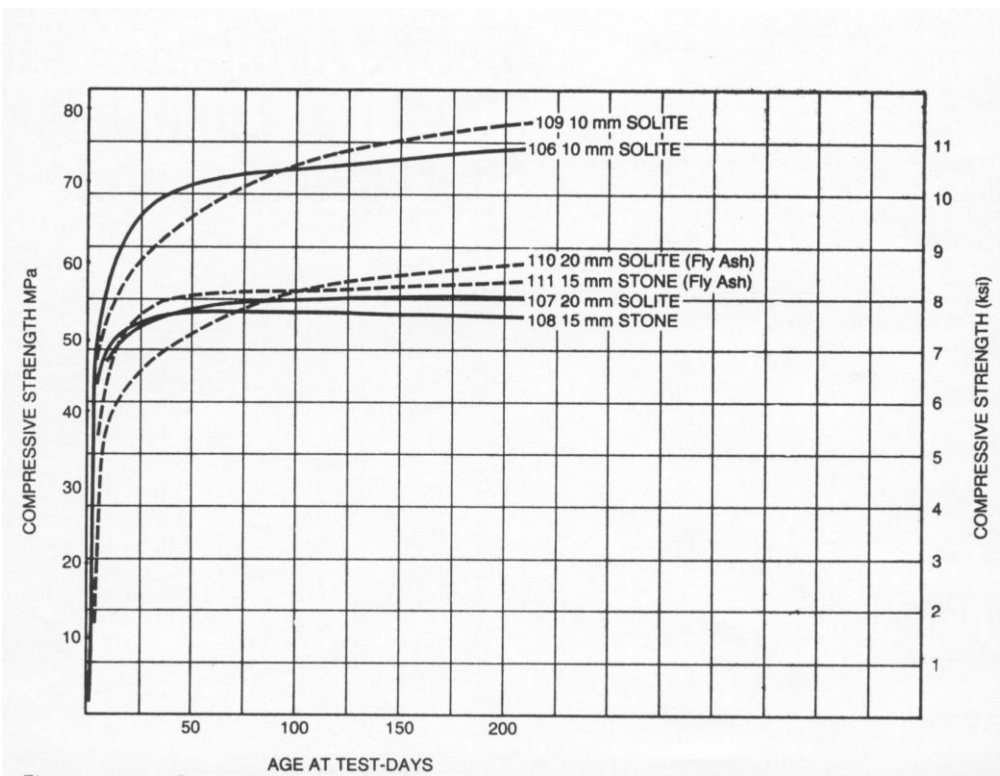

Figure 7.1. ***Compressive strength versus age of lightweight concrete (from Holm 1980a)***

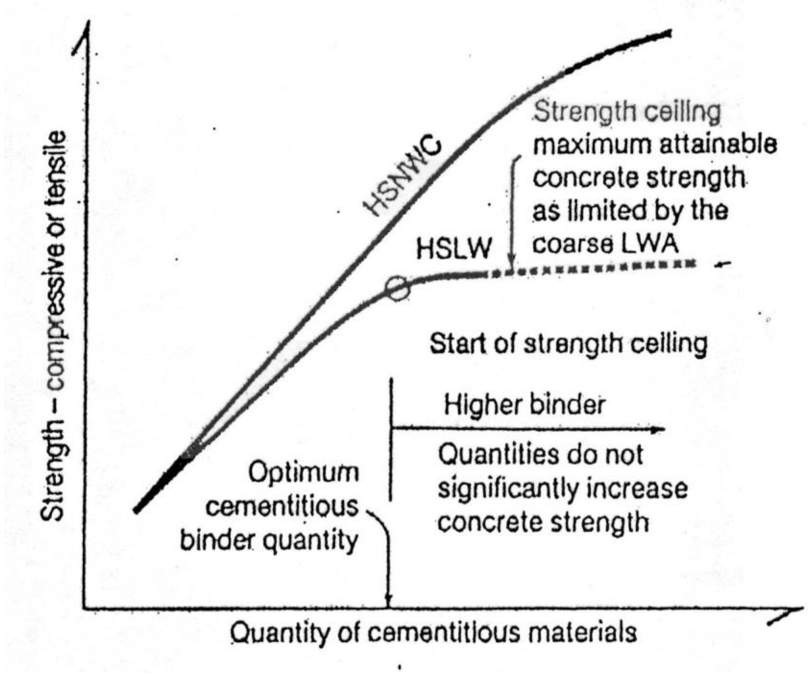

Figure 7.2. ***Strength versus concrete binder content (from Holm 1994)***

7.2 Tensile and Shear Strength

Shear, torsion, anchorage, bond strength, and crack resistance are related to tensile strength, which, in turn, is dependent upon tensile strength of the coarse aggregate and mortar phases and the degree to which the two phases are securely bonded. Traditionally, tensile strength has been defined as a function of compressive strength, but this is known to be only a first approximation that does not reflect aggregate particle strength, surface characteristics, nor the concrete's moisture content and distribution. The splitting tensile strength test as determined by ASTM C 496, is used throughout North America as a simple and practical method of determining the structural design parameters (ACI 318). Splitting tests are conducted by applying diametrically opposite compressive line loads to a concrete cylinder laid horizontally in a testing machine. A minimum lightweight concrete tensile splitting strength of 290 psi (2.0 MPa) is a requirement for structural lightweight aggregates conforming to the requirements of ASTM C 330.

Tests by Hanson (1961) have shown that diagonal shear strengths of lightweight concrete beams and slabs correlate closely with the concrete tensile splitting strengths. As tensile splitting results vary for different combinations of materials, the specifier should consult with the aggregate suppliers for laboratory-developed splitting tensile strength data. Specific mixture tensile strength test data should be developed before beginning projects in which early-age tensile-related handling forces develop, as in precast or tilt-up members. Lightweight concrete shear and tensile strengths may be assumed to vary from approximately 75 percent for concretes with fine and coarse lightweight aggregate to 85 percent of the tensile strength of normalweight concrete for lightweight concrete containing only coarse lightweight aggregate.

Tensile strength tests on structural lightweight concrete specimens that undergo some drying correlate better with the behavior of concrete in actual structures than specimens that have been continuously moist-cured. Moisture loss progressing slowly into the interior of concrete members will result in the development of outer envelop tensile stresses that balance the compressive stresses in the still-moist interior zones. ASTM C 496 requires 7-day moist and 21-day laboratory air-drying at 23 ºC (73 ºF) and 50 percent relative humidity prior to conducting splitting tests.

Tensile strength of high strength lightweight concrete

Visual examination of splitting tensile test specimens of dry mature specimens of high strength lightweight concrete clearly shows visible signs of high moisture contents on the split surface, demonstrating that well-compacted mixtures with high binder content, and particularly those incorporating mineral admixtures (silica fume, fly ash), are virtually impermeable and will release moisture very slowly. High-strength specimens drying in laboratory air for over several months were still visibly moist over 90 percent of the split diameter (Holm and Bremner 1994). The commonly observed reductions in splitting strength on air-dried commercial-strength lightweight concrete is caused by differential drying moisture gradients. The reduction is significantly delayed and diminished in high strength lightweight concrete containing high cementitious content.

At strength level of 2,400 to 5,080 psi (20 to 35 MPa), the relatively similar tensile strength and elastic rigidity of the two components (lightweight aggregate and matrix) will minimize stress concentrations and microcracking. At higher strengths, however, strong normalweight coarse aggregates will remain intact after matrix failure and provide a measure of post-elastic strain capacity and a greater resistance to splitting. Because of the lower post-elastic capacity of lightweight concrete, it is prudent to limit the maximum strength levels too the ACI 318 requirements which govern shear, tension, torsion, development lengths, and seismic parameters. Concrete compressive strengths greater than 6,000 psi (41 MPa) require compressive testing programs conducted on concretes containing specific combinations of aggregates and demonstrate adequate performance at these higher strength levels.

Tensile strength of high strength specified density concrete

Hoff et al. (1995) reported that the tensile splitting strength of the specified density concrete used in the Hibernia offshore platform slightly exceeded results for the normalweight concrete. This behavior is project specific and should not be anticipated with all specified density concrete mixtures. In general, the splitting ratio as defined by $\sqrt{f'_c} / f_{ct}$ will be reduced as compressive strengths are increased, which is particularly true when the concrete is air-dried.

7.3 ELASTIC PROPERTIES

Modulus of Elasticity Approximations from ACI 318 Building Code

The modulus of elasticity of concrete is a function of the modulus of each constituent (cementitious matrix, lightweight and normal-density aggregates) and their relative proportions in the mixture. The elastic modulus of normalweight concrete is higher than lightweight concrete because the moduli of the normalweight aggregate particles are greater than the moduli of lightweight aggregate particles. For practical design conditions, the modulus of elasticity of concretes with densities between 90 to 155 lb/ft^3 (1,440 and 2,500 kg/m^3) and within strength ranges up to 5,000 psi (35 MPa) can be represented by the following formula (Pauw 1960), ACI 318:

$$E = 33\omega^{1.5}\sqrt{f'_c}$$

or

$$E = 0.043\omega^{1.5}\sqrt{f'_c}$$

where

E = denotes the secant modulus in psi (MPa)

ω = density in lb/ft^3 (kg/m^3)

f'_c = compressive strength in psi (MPa) of a 6-by 12-in. (152-by 305 mm) cylinder

This or any other formula should be considered as only a first approximation, as the modulus is significantly affected ($\pm$ 25 percent) by moisture, aggregate type, and other variables. The formula clearly overestimates the modulus for high strength lightweight concrete where limiting values are determined by the modulus of the lightweight aggregate. When design conditions require accurate elastic modulus data, laboratory tests should be conducted on specific concretes proposed for the project in accordance with the procedure of ASTM C 469.

In general, all structural lightweight aggregates have a comparable chemical composition and are manufactured in a similar way and at similar temperatures. Lightweight aggregate achieve low density by formation of a porous structure in which the pores are generally spherical and enveloped in a vitreous matrix. With such similarities, the variability in stiffness of the aggregate would be principally due to the lightweight aggregate density.

As with normalweight concrete, increasing matrix stiffness is directly related to matrix strength which, in turn, affects concrete strength. When large percentages of cementitious materials are used, the lightweight concrete strength ceiling may be reached, causing the ACI 318 equation to overestimate the stiffness of the concrete. One factor affecting stiffness of normalweight concrete is the variation of aggregate modulus of elasticity within a particular density range. At the same specific gravity, LaRue (1946) found that the modulus of elasticity of natural aggregates could vary by a factor of as much as 3.

Although the ACI 318 formula has provided satisfactory results in estimating the elastic modulus of normalweight concrete and lightweight concrete in the usual commercial-strength range from 3,000 to 5,000 psi (20 to 35 MPa), it has not been adequately calibrated to predict the modulus of high-strength concretes. Practical modification of the formula was first provided by ACI 213 to more reasonably estimate the elastic modulus (E_c) of high strength lightweight concrete as

$$E_c = C \bullet \omega^{1.5} \sqrt{f'_c}$$

where

C	=	31 for 5,000 psi (C = 0.040 for 35 MPa)
C	=	29 for 6,000 psi (C = 0.038 for 41 MPa)
ω	=	density (lb/ft^3 or kg/m^3)
f'_c	=	compressive strength (psi or MPa)

When designs are controlled by elastic properties (e.g., deflections, buckling, etc.), the specific value of E_c should be measured on the proposed concrete mixture in accordance with the procedure of ASTM C 469.

Elastic Compatibility

A particulate composite is by its very definition heterogeneous, and concrete is perhaps the most heterogeneous of composites – with size of inclusions varying from large aggregate down to un-hydrated cement grains, and containing voids the size of entrained and entrapped air bubbles down to the gel pores in the cement paste.

Concrete can be considered as a two-phase composite composed of coarse particles enveloped in a continuous mortar matrix. This latter phase includes all the other concrete constituents, including fine aggregate, mineral admixtures, cement, water, and voids from all sources. This division is visible to the naked eye and may be used to explain important aspects of the strength and durability of concrete and is schematically shown in Figure 7.3.

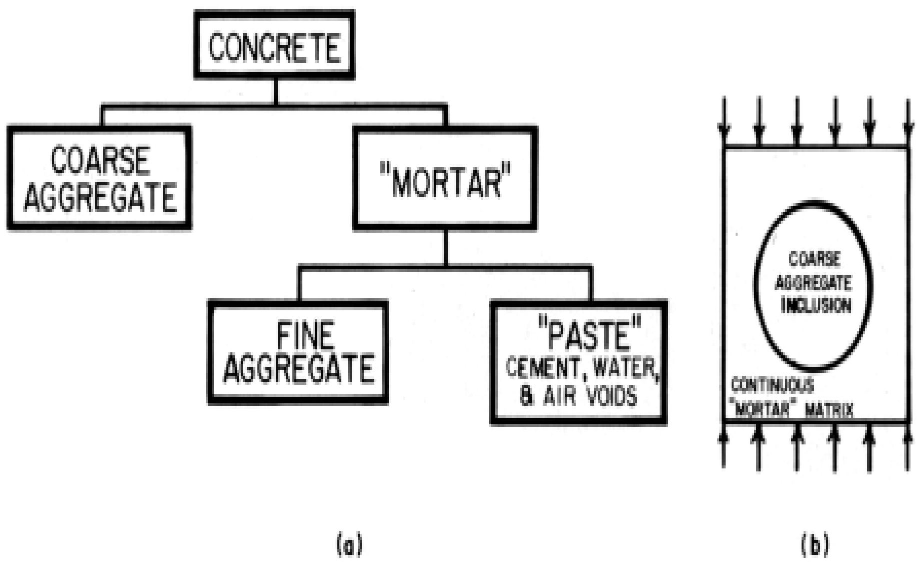

Figure 7.3. ***Cement, water, air voids, and fine aggregate combine in (a) to form the continuous mortar matrix that surrounds the coarse aggregate inclusion in (b) to produce concrete (from Bremner and Holm 1986)***

With normalweight aggregate there is an elastic mismatch between coarse aggregate particles and the surrounding mortar matrix, which creates stress concentrations when the composite is subjected to an applied stress. These stress concentrations are superimposed on a system already subjected to internal stresses arising from dissimilar coefficients of thermal expansion of the constituents and from the aggregate restraint of matrix volume changes. The latter can be caused by drying shrinkage, thermal shrinkage during cooling from hydration temperatures, or changes that result from continued hydration of the cement past. These inherent stresses are essentially self-induced and may be of a magnitude to induce extensive microcracking before any superimposed stress is applied.

Natural aggregates have an extremely wide range of elastic moduli resulting from large differences of mineralogy, porosity, flaws, laminations, grain size, and bonding. It is not uncommon for a fine-grained diabase rock to have an elastic modulus greater than 13 x 10^6 psi (90 GPa) while poorly bonded, highly porous natural aggregates have been known to have values lower than 3 x 10^6 psi (20GPa). Aggregate description by name of rock is insufficiently precise, as demonstrated in one rock mechanics text that reported a range of elastic modulus of 3 to 10 x 10^6 psi (20 to 69 GPa) for one rock type (Stagg and Zienkiewicz 1968).

Figure 7.4 is adapted from Stagg and Zienkiewicz (1968) and illustrates compressive strength and stiffness characteristics reported for several rock types and compares these wide ranges with the modulus of elasticity of concrete as calculated by ACI 318 Code:

$$E_c = 33\omega^{1.5}\sqrt{f_c'} \ (\omega = \text{density in lb/ft}^3 \text{ and } f_c' \text{ in psi}), \text{ or}$$
$$E_c = 0.043\omega^{1.5}\sqrt{f_c'} \ (\omega = \text{density in kg/m}^3 \text{ and } f_c' \text{ in MPa})$$

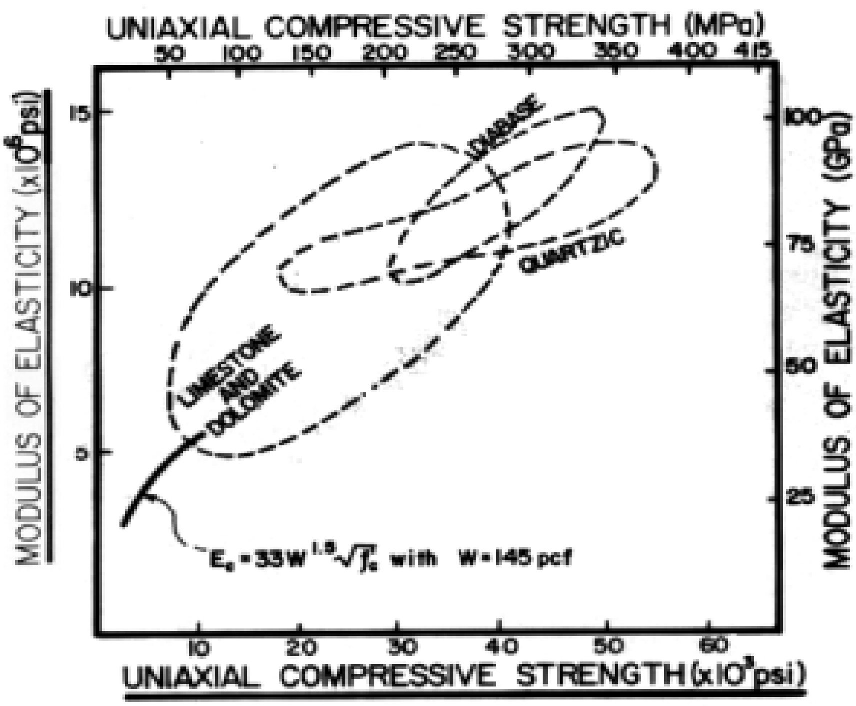

Figure 7.4. ***Range of stiffness of concrete caused by variability in the stiffness of the aggregate (E_C added after Staff and Zienkiewicz 1968)***

The ratio of the coarse aggregate modulus to that of the concrete composite can be shown to be as much as 3, signaling a further difference between the two interacting phases (mortar and coarse aggregate) of as much as 5 to 1 (Bremner 1981). That the strength-making potential of the stone or gravel is normally not fully developed is evident from visual examination of fracture surfaces of concrete cylinders after compression testing. The nature of the fracture surfaces of concretes is strongly influenced by the degree of heterogeneity between the two phases and the extent to which they are securely bonded together. Shah and Chandra (1968) reported on the profound influence exerted by the contact zone in compressive strength tests on concretes in which aggregate surface area was modified by coatings. The degree of heterogeneity and the behavior of the contact zone between the two phases are the principal reasons for the departure of some concretes from estimates of strength based upon the w/c ratio. As has been suggested, undue preoccupation with the matrix w/c ratio may lead to faulty estimates of compressive strength and even greater misunderstanding of concrete's behavior from durability, permeability, and tensile-type loading conditions (Bremner and Holm 1986).

Obviously, the characteristics of the normalweight aggregate will have a major effect on elastic compatibility. The interaction between the absolute volume percentage of coarse aggregate ($\pm$35 percent) and the mortar phase ($\pm$65 percent) will result in a concrete with a modulus intermediate between the two fractions. As shown in Fig. 7.5, at typical commercial strength levels, the elastic mismatch within lightweight concrete is considerably reduced due to the limited range of elastic properties of typical lightweight particles.

Muller-Rochholz (1979) measured the elastic modulus of individual particles of lightweight aggregate and normalweight aggregate using ultrasonic pulse-velocity techniques. His report concluded that the modulus of elasticity of structural lightweight aggregate exceeded values of the cementitious paste fraction. This explains that instances in which lightweight concrete strength exceeded that of companion normalweight concrete at equal binder content were understandable in light of the relative stress homogeneity.

The modulus of elasticity of an individual particle of lightweight aggregate may be estimated by the formula $E_c = 0.008\ p^2$(MPa), where p is the dry particle density (Muller-Rochholz 1979). Typical North American structural lightweight aggregates having dry-particle relative densities of 1.2 to 1.5 (1,200 to 1,500 kg/m³) would result in a particle modulus of elasticity from 1.7 to 2.6 x 10^6 psi (11.5 to 18 GPa). At these densities the modulus of elasticity of individual particles of lightweight aggregate approaches that measured on the mortar fraction of air-entrained commercial-strength lightweight concrete (Bremner and Holm 1986).

The elastic modulus of air-entrained and non-air-entrained mortars is shown as a function of compressive strength in Figure 7.5. The modulus of typical individual particles of coarse lightweight aggregate, as well as a range of values of modulus for stone aggregates, is shown. These results were obtained by testing concretes and equivalent mortars with the same composition found in concrete, with the exception that the coarse aggregate had been fractioned out. Mortar mixtures were produced to cover the typical ranges of cement contents at the same time as companion structural lightweight concretes were cast with all other mixture constituents kept the same (Bremner and Holm 1986).

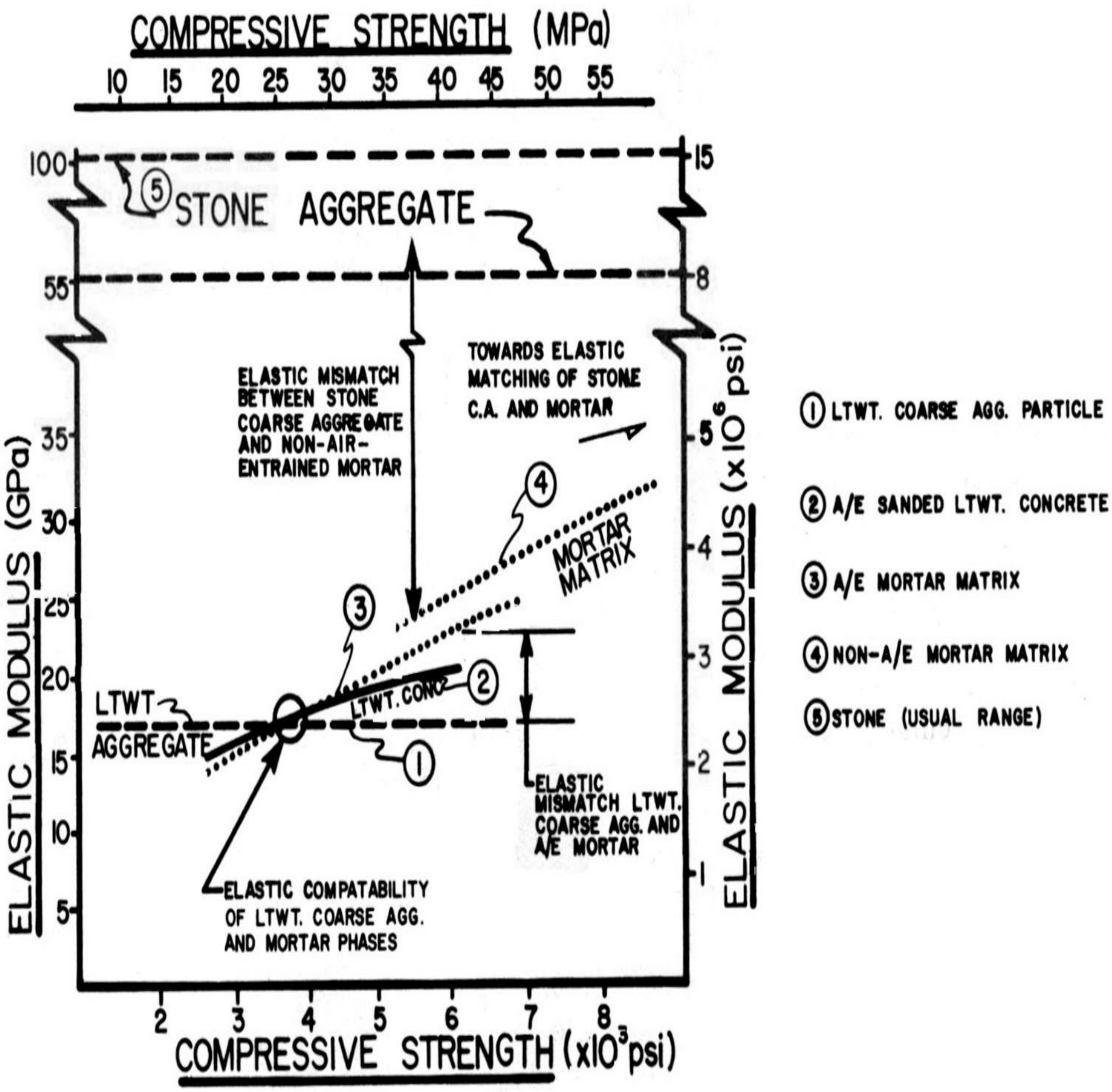

Figure 7.5. ***Elastic mismatch in low- and normal-density Concrete (from Bremner and Holm 1986)***

Sanded lightweight concrete with a compressive strength of approximately 4,000 psi (28 MPa) using typical North American structural lightweight aggregate and natural sand have values of $L_{ightweight\ aggregate\ particle}$ / $M_{ortar\ matrix}$ approaching unity. From a stress concentration point of view, this combination of constituents would act as a homogeneous material, resulting in concrete with minimum stress-induced microcracking. Thus, at ordinary commercial strengths, the elastic match of the two components will be close for air-entrained lightweight concrete made with high-quality lightweight aggregate. In contrast, matching of the elastic properties of ordinary concrete using a high-modulus normalweight aggregate such as a diabase will be possible only with the ultrahigh-quality matrix fractions incorporating superplasticizers high-range water reducing admixtures and supplementary cementitious materials.

Air entrainment in concrete significantly reduces the stiffness of the mortar fraction and, as shown in Figure 7.5, results in a convergence of elastic properties of the two phases of sanded structural lightweight concrete while increasing the degree of elastic mismatch in normalweight concrete. This fact, combined with the slight reduction in mixing water caused by air entrainment, explains why the strength penalty caused by air entrainment is often less significant for lightweight concrete than for concretes using highly rigid normalweight aggregate.

Elastic Compatibility of High-Strength Lightweight Concrete

Combining ultrahigh-strength, low-air content mortar matrix fractions with coarse lightweight aggregate will produce an elastic mismatch resulting in fracture that starts with transverse splitting of the structural lightweight aggregate particles. Splitting action stemming from lateral strains is indirectly responsible for the strength ceiling of structural lightweight concrete observed when improvements in mortar matrix quality result in little or no increase in compressive strength.

In general, for concretes using high-quality normalweight aggregate, elastic compatibility between the two fractions will occur only at extremely high compressive strengths. Ultrahigh-strength mortar fractions developed by high range water reducer admixtures and mineral admixtures will increase the possibility of achieving elastic compatibility at higher compressive strengths when normalweight aggregates are used.

While elastic mismatching plays an important role in the compressive strength capabilities of the concrete composite, the influence on other properties such as tensile and shrinkage cracking, and particularly the effect on in-service permeability and durability due to microcracking, is far more significant.

Poisson's ratio

Testing programs investigating the elastic properties of high strength lightweight concrete have reported an average Poisson's ratio of 0.20, with only slight variations due to age, strength level, curing environment, or aggregates used. Hoff et al. (1995) reported similar values for Poisson's ratio for specified density concrete and normalweight concrete.

Maximum Strain Capacity

Several methods of determining the complete stress-strain curve of lightweight concrete have been attempted. At Lehigh University, the concrete cylinders were loaded by a beam in flexure (Holm 1980b). The approach at the University of Illinois, however, was to load a concrete cylinder completely enclosed within a steel tube of suitable elastic properties (Wang, Shah, and Naamen 1978). Despite formidable testing difficulties, both methods secured meaningful data.

The failure of high strength lightweight concrete will release a greater amount of energy stored in the loading frame than will an equal-strength concrete composed of stiffer normalweight aggregate. As energy stored in the test frame is proportional to the applied load moving through a deformation that is inversely proportional to the modulus of elasticity, it is not unusual for the failure of high strength lightweight concrete cylinder to release almost 50 percent more energy stored in the frame. To avoid shock damage to the testing equipment, it is recommended that a lower percentage of maximum usable machine capacity be used when testing high strength lightweight concrete and that suitable precautions be taken by testing technicians as well (Holm 1980b).

Seismic Ductility

The ductility of concrete structural frames should be analyzed as a composite system – that is, as reinforced concrete. Studies by Ahmad and Batts (1991) and Ahmad and Barker (1991) indicate, for the materials tested, that the ACI rectangular stress block is adequate for strength predictions of high strength lightweight concrete beams, and the recommendation of 0.003 as the maximum usable concrete strain is an acceptable lower bound for high strength lightweight concrete members with strengths not exceeding 11,000 psi (76.5 MPa) and reinforcement ratios less than 54 percent of balanced ratio P_b. Moreno (1986) found that while lightweight concrete exhibited a rapidly descending portion of the stress-strain curve, it was possible to obtain a flat descending curve with reinforced lightweight concrete members that were provided with a sufficient amount of confining reinforcement slightly greater than that with normalweight concrete. Additional confining steel is recommended to compensate for the lower post-elastic strain behavior of lightweight concrete. This report also included study results that showed that it was economically feasible to obtain the desired ductility by increasing the amount of steel confinement.

Rabbat et al. (1986) came to similar conclusions when analyzing the seismic behavior of lightweight concrete and normalweight concrete columns. This report focused on how properly detailed reinforced concrete column-beam assemblages could provide ductility and maintain strength when subjected to inelastic deformations from moment reversals. These investigations concluded that properly detailed columns made with lightweight concrete performed as well under moment reversals as normalweight concrete columns.

7.4 Bond Strength, Development Length and Bearing Strength

Field performance has demonstrated satisfactory performance of lightweight concrete with strength levels of 2,900 to 5,080 psi (20 to 35 MPa) with respect to bond and development length. Because of the lower particle strength, lightweight concrete have lower bond-splitting capacities and a lower post-elastic strain capacity than normalweight concrete. Usual North American design practice (ACI 318) is to require longer embedment lengths for reinforcement in lightweight concrete than for normalweight concrete. Unless tensile splitting strengths are specified, ACI 318 requires the development lengths for lightweight concrete to be increased by a factor of 1.3 over the lengths required for normalweight concrete With closely spaced and larger diameter prestressing strands that can cause high splitting forces, this increase may no longer be conservative. A conservative design approach or a pre-project testing program may be advisable for some structures for example, short-span bridge decks, or combinations of highly reinforced thin members using high-strength lightweight concrete. Continual research on development length requirements for prestressing strands in high strength lightweight concrete and specified density concrete is clearly warranted. (Peterman 2000 , Ramirez 1999 , Nasser 2002 &Meyer 2002)

Because of the lower tensile strength, the bearing strength of lightweight concrete at anchorage zones is less than that of Normalweight concrete of equal compressive strength. Test results of an investigation reported by Roberts-Wollman et.al. (2006) indicate that the bearing strengths predicted by formulas in ACI 318 are un-conservative for lightweight concrete at a compressive strength of approximately 5000 psi (35 MPa).

The ACI 318 predictions are also un-conservative for higher strengths, (8000 psi) for both lightweight and Normalweight concrete Reported below are several of the conclusions derived from this testing program:

- For concretes with similar compressive strengths, lightweight concrete specimens exhibited significantly lower bearing strengths than Normalweight concrete specimens;
- For both Normalweight and lightweight concrete, higher-strength concretes exhibited lower bearing strengths, relative to their compressive strengths, than lower-strength concretes;
- The current ACI equation for bearing strength can over-predict strength for lightweight and high-strength concretes for conditions with A_2/A_1 ratios less than approximately 7;

- A lightweight modification factor of 0.7 used with the current ACI bearing strength equation provides a best fit to the data, however, a considerable number of the tests failed at loads below those predicted by this equation. With a ϕ factor of 0.65, all failure loads are at or above the design strength. However, this leaves little room for strength reductions due to other factors.

7.5 Drying Shrinkage

Drying shrinkage is an important property that can affect the extent of cracking, prestress loss, effective tensile strength, and warping. It should be recognized that large-size concrete member, or these in high ambient relative humidity, might undergo substantially less shrinkage than that exhibited by small laboratory specimens stored at 50% relative humidity.

As with normalweight concretes, shrinkage of structural lightweight concrete is principally determined by

a. Shrinkage characteristics of the cement paste.
b. Internal restraint provided by the aggregate.
c. Relative absolute volume fractions occupied by the cement paste and the aggregate.
d. Humidity and temperature.

Aggregate characteristics influences cement paste quantities (the shrinking fraction) necessary to produce a required strength at a given slump. Particle strength, shape, and grading influence water demand and directly determine the fractional volume and quality of the cement paste necessary to meet specified strength levels. Once that interaction had been established, the rigidity of the aggregate restrains shrinkage of the cement paste.

When structural lightweight concrete is proportioned with similar cementitious amounts to those required for normal aggregate concretes, the drying shrinkage of lightweight concrete is generally, but not always, slightly greater than that of normalweight concrete due to the lower aggregate stiffness. The time rate of shrinkage strain development in structural lightweight concrete is lower, and the time required to reach a plateau of equilibrium is longer when the as-batched, lightweight aggregate absorbed moisture is high. Maximum shrinkage strains of high strength lightweight concrete may be approximately 15 percent greater than high-strength, normalweight concretes containing similar cement paste content.

ASTM C 330 limits drying shrinkage of structural lightweight concrete to less than 0.07 percent after 28 days of drying in a curing cabinet maintained at 100 ºF (37.8 ºC) at a relative humidity of 32 percent. Concrete mixtures used in the test specimens are prepared with a cement content of 564 lb/yd³ (335 kg/m³) with water contents necessary to produce a slump of 2 to 4 in. (50 to 100 mm) and air content of 6 $\pm$ 1 percent. Specimens are removed from the molds at 1 day's age, and moist-cured for 7 days age, at which time the accelerated drying is initiated.

High-strength lightweight concrete

Figure 7.6 shows a typical shrinkage versus time curve, and ultimate shrinkage from one extensive testing program that incorporated both high strength lightweight concrete and high strength normalweight concrete (Holm 1980a). Shrinkage of the 3/8" (9.5-mm) maximum-size high strength lightweight concrete mixture lagged behind early values of the high strength normal density concrete mixtures, equaled them at 90 to 130 days, and reached an ultimate value at 1 year, approximately 14 percent higher than the reference high strength normal density concrete. Shrinkage values of mixtures incorporating cement containing interground fly ash averaged somewhat greater than their high-strength non-fly ash counterparts.

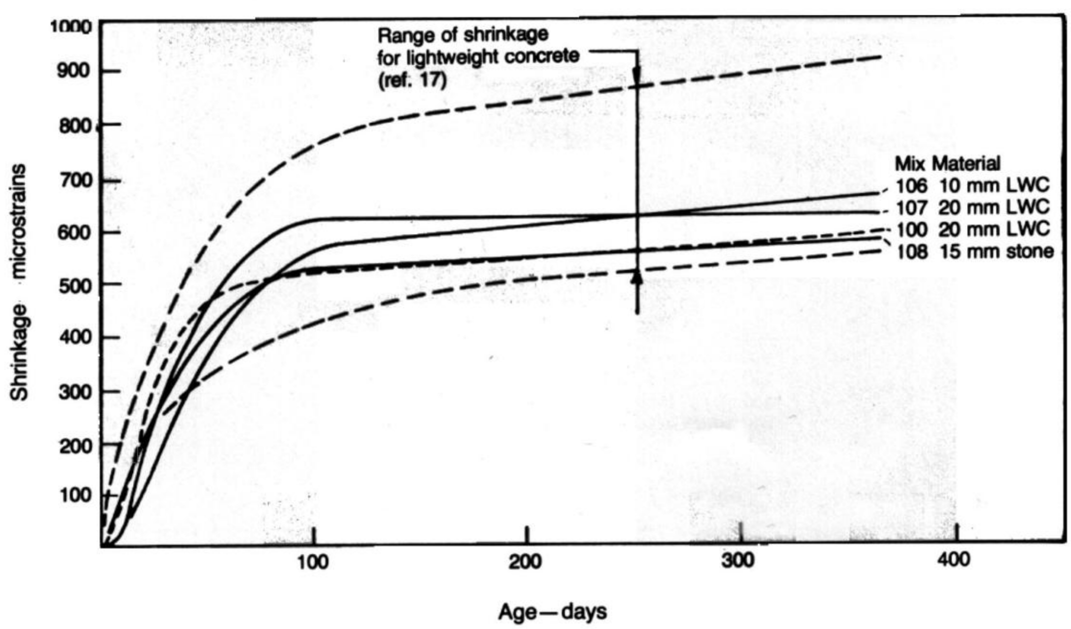

Figure 7.6. ***Shrinkage of high-strength, low- and normal-density concrete (after Holm 1980a)***

Shrinkage and density data were measured on 4- by 4- by 12-in. (102- by 102- by 305-mm) concrete prisms fabricated at the same time and from the same mixture as the compressive strength cylinders. Curing was provided by damp cloth for 7 days, after which the specimens were stripped from the molds. After stripping at one day, brass wafers were attached to the bar surface at a 10-in (254-mm) gage distance. Mechanical measurements were made with a Whittemore gage. Reference readings were established 7 days after fabrication, after which specimens were allowed to dry in laboratory air, 70 ºF (21 ºC) and 50 ± 5 percent relative humidity, with no further curing. Shrinkage and mass readings were taken weekly for 3 months, then monthly with results shown to 1 year.

Shrinkage measured on prisms exposed to similar curing conditions (77 days moist) reported by Hoff (1992) were of similar shape and magnitude to the lightweight concrete shown in Figure 7.6 reported by Holm (1980a). Specimens cured with 1 day of steam and 6 days moist curing prior to exposure in laboratory air had shrinkage strains approximately 20 percent less than the standard 7-day moist-cured specimens.

7.6 CREEP

Creep is the increase in strain of concrete under a sustained stress. Creep properties of concrete may be either beneficial or detrimental, depending on the structural conditions. Concentrations of stress, either compressive or tensile, may be reduced by stress transfer through creep, or creep may lead to excessive lone-time deflection, prestess loss, or loss of camber. The effects of creep along with those of drying shrinkage should be considered and, if necessary, taken into account in structural designs.

Time-related increases in concrete strain due to sustained stress can be measured according to procedures of ASTM C 512. Creep and shrinkage characteristics on any type of concrete are principally influenced by aggregate characteristics, water and cement content (paste volume fraction), age at time of loading, type of curing, and applied stress-to-strength ratio

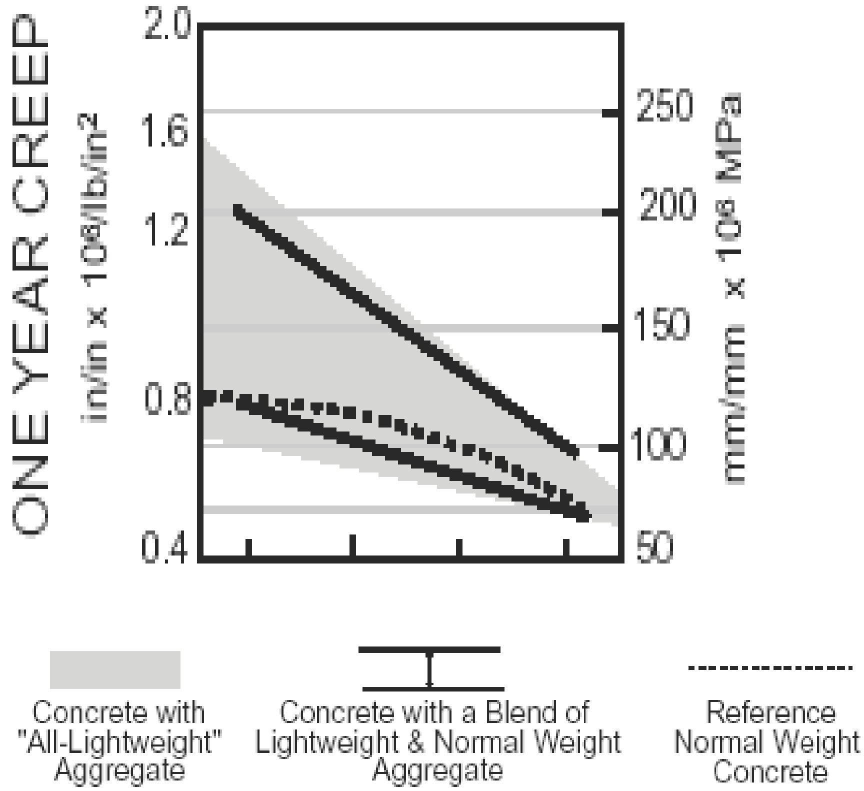

Figure 7.7. ***Creep of normally cured concrete (From ACI 213-8)***

Structural lightweight concrete

As shown in Figure 7.7, ACI 213R provides wide envelopes of 1-year specific creep values for lightweight, normally cured concretes. Test results for higher strength, steam-cured, sanded lightweight concrete have a range of values that narrows significantly and closely envelopes the performance of the normalweight reference concrete. These values are principally base upon the results of the comprehensive testing program of Shideler (1957). Long-term investigations by Troxell, Raphael, and Davis (1958) on normalweight concrete, report similar wide envelopes of results for different natural aggregate types. Therefore, comparisons with reference concretes should be based upon data specific to the concretes considered.

Additional large-scale creep testing programs have been reported by Holm (1983); Pfeifer (1968); and Valore (1973), who provided a comprehensive report that also includes European data on structural as well as insulating-grade lightweight concrete.

Creep of High-strength lightweight concrete

Rogers (1957) reported that the 1-year creep strains measured on several North Carolina and Virginia high strength lightweight concretes were similar to those measured on companion normalweight concrete. Greater creep strains were reported by Reichard (1964) and Shideler (1957) on high strength lightweight concrete containing both fine and coarse lightweight aggregate, compared with reference high strength normalweight concrete. These higher creep strains could be anticipated due to the significantly larger cementitious volume required because of the angular particle shape of the lightweight aggregate fines used in those testing programs.

The *Prestressed Concrete Institute Design Handbook* recommends a higher value of creep strain and an equal value of shrinkage when comparing lightweight concrete to normalweight concrete. It provides recommendations for increasing prestress losses when using lightweight concrete 30,000 to 55,000 psi (207 to 379 MPa) compared with a range of 25,000 to 40,000 psi (172 to 345 MPa) for normalweight concrete. However, it is advisable to obtain accurate design coefficients for long-span high strength lightweight concrete structures by conducting pre-bid laboratory tests in accordance with the procedures of ASTM C 512.

7.7 THERMAL EXPANSION

Accurate physical property input data are essential when considering the thermal response of restrained members in exposed structures. Such cases include bridge decks, exposed exterior columns of multistory cast-in-place concrete frames, as well as massive offshore concrete structures constructed in temperate zones and then towed to harsh Arctic marine environments. The coefficient of thermal expansion of concrete is principally determined by the expansion characteristics of the aggregates as aggregates compose approximately 70 percent of the total volume of concrete. To a lesser degree expansion is determined by the volumetric proportions and the moisture conditions of the concrete,

“Determination (Price and Cordon 1949) of linear thermal expansion coefficients made on lightweight concrete indicate values are 4 to 5 x 10^6 in./in/ºF (7 to 11 x 10^6 mm/mm/ºC), depending on the amount of natural sand used”, ACI 213R-03.

High-strength lightweight concrete

Hoff (1992) reported the coefficients of thermal expansion of various high-strength lightweight concrete measured by differential dilatometric procedures. After being cured at three pretest moisture conditions, the specimens were then exposed to 14 days of fog during prior to examination. The pretest moisture conditions were:

a. 0 percent relative humidity, oven-dried to a constant mass of 221 ± 5 ºF (105 ± 2.8 ºC).

b. 50 percent relative humidity, 50 ± 5 percent RH at 73 ± 3 ºF (22.8 ± 1.7 ºC).

c. 100 percent relative humidity, submerged at a temperature of 73 ± 3 ºF (22.8 ± 1.7 ºC).

The results of this testing program are summarized in Table 7.1 (Hoff 1992).

Table 7.1
Coefficient of Thermal Expansion of High-Strength Lightweight Concrete

Mixture	Moisture Condition, Relative Humidity, %	Specimen Size, Mm (in.)	Coefficient of Thermal Expansion Between 21 ºF (70 ºC) and -30 ºF (-22 ºC) [microstrain / ºC (/ ºF)]
LWC1	100	13 x 75 (0.5 x 3)	6.1 (3.4)
LWC1	50	13 x 75 (0.5 x 3)	7.7 (4.3)
LWC1	0	13 x 75 (0.5 x 3)	6.3 (3.5)
LWC1	50	152 x 305 (6 x 12)	7.4 (4.1)
LWC3	100	152 x 305 (6 x 12)	12.8 (7.1)
LWC3	50	152 x 305 (6 x 12)	11.0 (6.1)
LWC3	0	152 x 305 (6 x 12)	5.8 (3.2)
LWC4	100	152 x 305 (6 x 12)	9.0 (5.0)
LWC4	50	152 x 305 (6 x 12)	8.1 (4.5)
LWC4	0	152 x 305 (6 x 12)	7.0 (3.9)
HSLWC	100	152 x 305 (6 x 12)	12.8 (7.1)
HSLWC	50	152 x 305 (6 x 12)	7.0 (3.9)
HSLWC	0	152 x 305 (6 x 12)	7.0 (3.9)

Note: 1 microstrain = m x 10^{-6}

High-strength specified-density concrete

There is limited data available of the measurement of the coefficients of thermal expansion of specified-density concrete, however the coefficient should be intermediate to that of lightweight and normalweight concrete, and as mentioned earlier, would be highly dependent on the coefficient expansion of the various aggregates used.

7.8 FATIGUE

The first recorded North American comparison of the fatigue behavior between lightweight and normalweight concrete was reported by Gray and McLaughlin (1961). These investigators concluded that

a. The fatigue properties of lightweight concrete are not significantly different over large variations in strength level of the concrete.
b. The fatigue properties of lightweight concrete are not significantly different from the fatigue properties of normalweight concrete.

This work was followed by Ramakrishnan, Bremner, and Malhotra (1992) who found that, under wet conditions, the fatigue endurance limit was the same for lightweight and normalweight concrete.
Because of the significance of oscillating stresses that would be developed by wave action on offshore structures, and due to the necessity for these marine structures to use a lightweight concrete for buoyancy considerations, a considerable amount of research has been commissioned to determine the fatigue resistance of high strength lightweight concrete and to compare these results with the characteristics of normalweight concrete. Hoff (1994) reviewed much of the North American and European data and concluded that, despite the lack of a full understanding of failure mechanisms, "under fatigue loading, high strength lightweight concrete performs as well as high strength normalweight concrete and, in many instances, provides longer fatigue life." It is, however, the long-term service performance of real structures that provides improved confidence in material behavior rather than the extrapolation of conclusions obtained from laboratory investigations.

The long-term field performance of lightweight concrete bridge members constructed in Florida in 1964 (Figure 4.8) was evaluated in an in-depth investigation conducted in 1992 (Brown, Larsen, and Holm 1995). Comprehensive field measurements of service load strains and deflections taken in 1968 and 1992 were compared with the theoretical bridge responses predicted by a finite element model that is part of the Florida Department of Transportation bridge rating system (Brown and Davis 1993). The original 1968 loadings and measurements of the bridge were duplicated in 1992 and compared with calculated deflections, as shown in Figure 4.9. Maximum deflection for one particular beam due to a midpoint load was 0.28 in. (71. mm), measured at 60.5 ft (18.4 m) from the unrestrained end of the span. This compares very well with the original deflection, which was recorded to be 0.26 in. (6.6 mm) measured at 50.5 ft (15.4 m). Rolling load deflections measured in 1968 and 1992 were also comparable, but slightly less in magnitude than the static loads.

Strain measurements across the bridge profile were also duplicated, and these compared very closely for most locations in areas of significant strain. Highest strains of 85 and 72 microstrains were recorded for the exterior beam at 50.6 and 60.5 ft (15.4 and 18.4 m) when loaded with a truck in the appropriate lane. Again, comparison of the 1994 and 1968 data shows bridge behavior to be essentially similar, with the profiles closely matched.

It appears that dynamic testing of the flexural characteristics of the 28-year-old long-span lightweight concrete bridge at the time of test corroborates the conclusions of fatigue investigations conducted on small specimens tested under controlled conditions in several laboratories (Hoff 1994, Gjerde 1982, Gray and McLaughlin 1961). In these investigations, it was generally observed that the lightweight concrete performed as well as and, in most cases, somewhat better than companion normalweight control specimens. Several investigators have stated that improved performance was due to the elastic compatibility of the lightweight aggregate particles to that of the surrounding cementitious matrix. In lightweight concrete, the elastic modulus of the constituent phase (coarse aggregate and the enveloping mortar phase) is relatively similar, while with normalweight concrete the elastic modulus of most normalweight aggregates may be as much as 3 to 5 times greater than their enveloping matrix (Bremner and Holm 1986). With lightweight concrete, elastic similarity of the two phases of a composite system results in a profound reduction of stress concentrations that can lead to microcracking in the concrete composite.

Additionally, because of the pozzolanic reactivity of the surface of the vesicular aggregate that has been fired at temperatures above 2,012 ºF (1,100 ºC) (Khokrin 1973), the quality and integrity of the contact zone of lightweight concrete is considerably improved. As the onset of microcracking is most often initiated at the weak link interface between the dense aggregate and the enveloping matrix, it follows that lightweight concrete will develop a lower incidence of microcracking (Holm, Bremner, and Newman 1984).

Figure 7.8. ***Barge-mounted frame placed lightweight concrete beams (to the right is an old truss bridge; both bridges will carry U.S. 19 traffic) (Brown, Larsen and Holm 1995)***

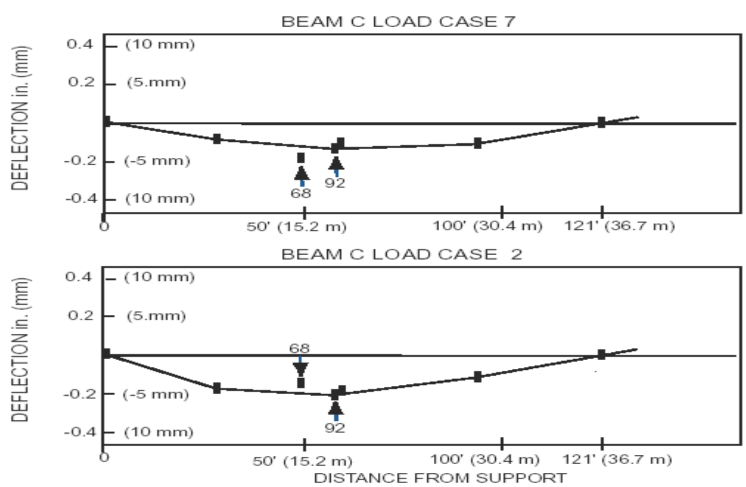

Figure 7.9. ***Florida DOT-predicted deflections compared with 1968 and 1992 measurements (Brown, Larsen and Holm 1995)***

7.9 Fire Resistance

As a typical example of prescribed fire ratings for structures, the information tabulated below is extracted from the Government of the District of Columbia approvals under their Research File Numbers RE 63-82 (PM) and RE 66-16 (PM) and a letter of approval. The data shown (Table 7.2) is based upon various fire tests conducted by the Underwriters' Laboratory and the Portland Cement Association. Comparisons with normalweight concrete are shown in Fig. 6.14 of Chapter 6.

Table 7.2 Fire Resistance Ratings of Typical Sanded Lightweight Concrete

Fire Resistance Ratings	Minimum Reinforced Concrete Slab Thickness	Minimum Cover on Reinforcement		
		Conventional Reinforcement Slabs	Post Tensioned Designs	
			Beams, Girders, Joists	Slabs
3/4 hour	2-1/2 inches	3/4 inch	1 inch	3/4 inch
1 hour	3 inches	3/4 inch	1 inch	3/4 inch
1-1/2 hour	3-1/2 inches	3/4 inches	1 inch	3/4 inch
2 hour	3-3/4 inches	3/4 inches	1 inch	3/4 inch
3 hour	4-1/2 inches	3/4 inches	1-1/2 inches	1 inch
4 hour	5 inches	3/4 inch	2 inches	1 inch

The results shown above are influenced by the following physical properties:

- Rotary kiln expanded aggregates (ESCS) have already demonstrated their stability at high temperatures during their production…in effect they have a prior experience of "load testing".
- Low Thermal Conductivity – Low "Conductivity" value resists the transfer of heat through floors and effectively insulated embedded reinforcement against the loss of strength at high temperatures.
- Resistance to Thermal Shock – Similar modulii of elasticity of paste and lightweight aggregate minimize microscopic discontinuities.
- Low Coefficient of Linear Thermal Expansion – The significantly lower coefficient of linear thermal expansion combines with the lower modulus of elasticity to reduce thermal movements that cause stresses in restrained structural components.

Proof of the effectiveness of the listed physical properties have been dramatically demonstrated in actual fires and in numerous refractory applications.

7.10 Behavior of Lightweight Concrete at Cryogenic Temperatures

The paper "Behavior of Prestressed Lightweight Concrete Subjected to High-Intensity Cyclic Stress at Cryogenic Temperatures", by Berner et al (1986) was prompted by the need for analysis of major structures designed for the storage or transport of liquefied natural gas (LNG) and liquefied petroleum gas (LPG). These structures could be exposed to severe loading conditions as those encountered in offshore and seismic areas.

Because of safety concerns reinforced and prestressed concrete offer the following advantages in cryogenic applications:

- Ability to withstand cryogenic thermal shock.
- High resistance to impact loads.
- Slow thermal response time to fire or cryogenic shock.
- Good structural characteristics regarding fatigue, crack propagation and buckling.

Lightweight concrete was selected for their study because of advantages offered for cryogenic structures including:

- Lightweight (important in floating structures)
- Low modulus of elasticity (E)
- Low coefficient of thermal expansion (α) (Thermal stresses are directly related to the elastic modulus (E) and the coefficient of thermal expansion (α).
- Low thermal conductivity (reduces the area of the structure affected by a release of cryogenic liquid)
- High-strength, prestressed lightweight concrete offers excellent durability and energy absorbtion characteristics.

After an extensive structural/thermal testing program the authors reported several conclusions, two of which directly apply to lightweight concrete:

- "The cryogenic temperatures will not extend far into the surrounding structure from the thermally shocked region even when the lightweight concrete is heavily reinforced with steel."
- "Properly designed prestressed lightweight concrete elements can retain their integrity against those cracks when subjected to thermal shock and high intensity cyclic loading and remain structurally sound with only relatively minor reductions in stiffness.