Guide for Structural Lightweight-Aggregate Concrete

Reported by ACI Committee 213

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Guide for Structural Lightweight-Aggregate Concrete

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SYNOPSIS

The guide summarizes the present state of technology, presents and interprets the data on lightweight-aggregate concrete from many laboratory studies and the accumulated experience resulting from its successful use, and reviews performance of structural lightweight aggregate concrete in service.

This guide includes a definition of lightweight-aggregate concrete for structural purposes and discusses, in a condensed fashion, the production methods for and inherent properties of structural lightweight aggregates. Current practices for proportioning, mixing, transporting, and placing; properties of hardened concrete; and the design of structural concrete with reference to ACI 318 are all discussed.

Keywords: abrasion resistance; aggregate; bond; contact zone; durability; fire resistance; internal curing; lightweight aggregate; lightweight concrete; mixture proportion; shear; shrinkage; specified density concrete; strength; thermal conductivity.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

The objectives of this guide are to provide information and guidelines for designing and using lightweight concrete. By using such guidelines and construction practices, the structures can be designed and performance predicted with the same confidence and reliability as normalweight concrete and other building materials.

This guide covers the unique characteristics and performance of structural lightweight-aggregate (LWA) concrete. General historical information is provided along with detailed information on LWA and proportioning, mixing, and placing of concrete containing these aggregates. The physical properties of the structural LWA, along with design information and applications, are also included.

Structural lightweight concrete has many and varied applications, including multistory building frames and floors, curtain walls, shell roofs, folded plates, bridge decks and girders, prestressed or precast elements of all types, and marine structures. In many cases, the architectural expression of form, combined with functional design, is achieved more readily with structural lightweight concrete than with any other medium. Many architects, engineers, and contractors recognize the inherent economies and advantages offered by this material, as evidenced by the many impressive lightweight concrete structures found throughout the world.

Because much of the properties and performance of lightweight concrete are dependent on the type of LWA used, the ready mix supplier, LWA producer, or both, might be an important source of specific information for attaining the project objectives.

1.2—Scope

1.2.1 *Historical background*—The first known use of lightweight concrete dates back over 2000 years. There are several lightweight concrete structures in the Mediterranean region, but the three most notable structures were built during the early Roman Empire and include the Port of Cosa, the Pantheon Dome, and the Coliseum.

Built in approximately 273 BC, the Port of Cosa used lightweight concrete made from natural volcanic materials. These early builders learned that expanded aggregates were better suited for marine facilities than the locally available beach sand and gravel. They traveled 25 mi. (40 km) to the northeast to quarry volcanic aggregates at the Volcine complex for use in the harbor at Cosa (Bremner et al. 1994). This harbor on the west coast of Italy consists of a series of four piers (~13 ft [4 m] cubes) extending into the sea. For two millennia the piers have withstood the forces of nature with only surface abrasion. They became obsolete only because of siltation of the harbor.

Built circa 126 AD, the Pantheon incorporates concrete varying in density from bottom to top of the dome. Roman engineers had sufficient confidence in lightweight concrete to build a dome with a diameter of 142 ft (43 m), which was not exceeded for almost two millennia. The structure is in excellent condition and is still being used today for spiritual purposes (Bremner et al. 1994).

The dome contains intricate recesses formed with wooden formwork to reduce the dead load and the imprint of the grain of the wood can still be seen. The excellent cast surfaces that are visible to the observer clearly show that these early builders had successfully mastered the art of casting concrete made with LWA. The Roman writer, architect, and engineer, Vitruvius, who took special interest in building construction, commented on several unusual features of the Pantheon. The fact that he did not single-out lightweight concrete for comment could imply that these early builders were fully familiar with this material (Morgan 1960).

Built in 75 to 80 AD, the Coliseum is a gigantic amphitheater with a seating capacity of 50,000 spectators. The foundations were cast with lightweight concrete using crushed volcanic lava. The walls were made using porous, crushed-brick aggregate. Vaults and spaces between the walls were constructed using porous-tufa cut stone.

1.2.2 *Development of manufacturing process*—After the fall of the Roman Empire, lightweight concrete use was limited until the 20th century when a new type of manufactured expanded shale LWA became available for commercial use.

The rotary kiln process was developed in 1918 and is used to produce expanded shale, clay, and slate. LWAs are manufactured by heating small particles of shale, clay, or slate in a rotary kiln. A particle size was discovered that, with limited crushing, produced an aggregate grading suitable for making lightweight concrete (Expanded Shale, Clay and Slate Institute 1971). When clay bricks are manufactured, it is important to heat the preformed clay slowly so that evolved gases have an opportunity to diffuse out of the clay. If they are heated too rapidly, a bloater is formed that does not meet the dimensional uniformity essential for a successfully fired brick. These rejected bricks were recognized by Hayde as an ideal material for making a special concrete. When reduced to appropriate aggregate size and grading, these bloated bricks could be used to produce a lightweight concrete with mechanical properties similar to regular concrete.

Commercial production of expanded slag (that is, expanded shale, clay, or slate) began in 1928, and in 1948, the first structural-quality sintered-shale LWA was produced using shale in eastern Pennsylvania.

One of the earliest uses of reinforced lightweight concrete was in the construction of ships and barges in the early 1900s. The U.S. Emergency Fleet Building Corporation found that for concrete to be effective in ship construction, the concrete would need a maximum density of about 110 lb/ft^3 (1760 kg/m^3) and a compressive strength of approximately 4000 psi (28 MPa) (Expanded Shale, Clay, and Slate Institute 1960). Concrete was obtained with a compressive strength of approximately 5000 psi (34 MPa) and a unit weight of 110 lb/ft^3 (1760 kg/m^3) or less using rotary-kiln-produced expanded shale and clay aggregate.

1.2.3 *Early modern uses*—Considerable impetus was given to the development of lightweight concrete in the late 1940s when a survey was conducted on the potential use of lightweight concrete for home construction. This led to an extensive study of concrete made with LWAs. Sponsored by the Housing and Home Finance Agency (1949), parallel studies were conducted simultaneously in the laboratories of the National Bureau of Standards (Kluge et al. 1949) and the U.S. Bureau of Reclamation (Price and Cordon 1949) to determine properties of concrete made with a broad range of LWA types. These studies and earlier works focused attention on the potential structural use of some LWA concrete and initiated a renewed interest in lightweight members for building frames, bridge decks, and precast products in the early 1950s. Following the collapse of the original Tacoma Narrows Bridge in Washington, the replacement suspension structure design used lightweight concrete in the deck to incorporate additional roadway lanes without the necessity of replacing the original piers.

During the 1950s, many multistory structures were designed with lightweight concrete from the foundations up, taking advantage of reduced dead weight. Examples include the 42-story Prudential Life Building in Chicago, which used lightweight concrete floors, and the 18-story Statler Hilton Hotel in Dallas, designed with a lightweight concrete frame and flat plate floors.

These structural applications stimulated more concentrated research into the properties of lightweight concrete. In energy-related floating structures, such as an oil drilling rig, great efficiencies are achieved when a lightweight material is used. A reduction of 25 percent in mass in reinforced normalweight concrete will result in a 50 percent reduction in load when submerged. Because of this, the oil and gas industry recognized that lightweight concrete could be used to good advantage in its floating structures and structures built in a graving dock, and then floated to the production site and bottom-founded. To provide the technical data necessary to construct huge offshore concrete structures, a consortium of oil companies and contractors was formed to evaluate which LWA candidates were suitable for making high-strength lightweight concrete that would meet their design requirements. The evaluations began in the early 1980s with results available in 1992. As a result of this research, design information became readily available and has enabled lightweight concrete to be used for new and novel applications where high strength and high durability are desirable (Hoff 1992).

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A = fractional solid volume (without pores) of the vitreous material of an individual particle
B = subsequent fractional volume of pore (within the particle)
BD = bulk density, lb/ft^3 (kg/m^3)
C = fractional volume of particles
C_f = cement factor or the mass of cement per cubic foot (cubic meter)
CS = chemical shrinkage of the binder per unit mass of the binder at 100 percent reaction (typically 0.07 mL/g cement)
c = heat capacity, Btu/(lb · °F) (kJ/(kg · K))
D = fractional volume of interstitial voids (between particles)
E = calculated equilibrium density, lb/ft^3 (kg/m^3)
E_c = modulus of elasticity, ksi (GPa)
E_{cd} = dynamic modulus of elasticity of the particle, ksi (GPa)
f_c = concrete compressive strength, psi (MPa)
f_{ct} = average splitting tensile strength, ksi (MPa)
f_c' = compressive strength
k = thermal conductivity, Btu/(hr · ft · °F) (W/(m · K))
M_{LWA} = mass of the lightweight aggregate
p = dry mean particle density, lb/ft^3 (k/m^3)
R = thermal resistance
RD = relative density, lb/ft^3 (kg/m^3)
U = thermal transmittance, Btu/(hr · ft^2 · °F) (W/(m^2 · K))
S = saturation level of the LWA
V = volume of concrete produced, ft^3 (m^3)
W_c = oven-dry density of concrete, lb/ft^3 (kg/m^3)
W_{ct} = weight of cement in batch, lb (kg)
W_{dc} = weight of coarse aggregate in batch, lb (kg)
W_{df} = weight of dry fine aggregate in batch, lb (kg)
w_c = unit weight of normal concrete or equilibrium density of lightweight concrete, lb/ft^3 (kg/m^3)
w_m = densities in moist conditions, lb/ft^3 (k/m^3)
w_{od} = densities in oven-dry conditions, lb/ft^3 (k/m^3)
α_{max} = expected maximum degree of reaction for the binder ranging from 0 to 1
Φ_{LWA} = measured absorption capacity of the lightweight aggregate

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," http://www.concrete.org/Tools/ConcreteTerminology.aspx. Definitions provided here compliment that resource.

all-lightweight concrete—concrete in which both the coarse- and fine-aggregate components are lightweight aggregates.

contact zone—transitional layer of material connecting aggregate particles with the enveloping continuous mortar matrix.

fresh density—mass-per-unit volume of concrete in fresh state, before setting.

high-strength lightweight concrete—structural lightweight concrete with a 28-day compressive strength of 6000 psi (40 MPa) or greater.

insulating aggregate—nonstructural aggregate meeting the requirements of ASTM C332; includes Group I aggregate, perlite with a bulk density between 7.5 and 12 lb/ft^3 (120 and 192 kg/m^3), expanded vermiculite with a bulk density between 5.5 and 10 lb/ft^3 (88 and 160 kg/m^3), and Group II aggregate that meets the requirements of ASTM C330/C330M and ASTM C331/C331M.

internally stored water—water internally held by the lightweight aggregate that is not readily available at mixing and, therefore, does not affect water-cementitious material ratio (*w/cm*).

masonry-lightweight aggregate (MLWA)—aggregate meeting the requirements of ASTM C331/C331M with bulk density less than 70 lb/ft^3 (1120 kg/m^3) for fine aggregate and less than 55 lb/ft^3 (880 kg/m^3) for coarse aggregate.

net water—total water less amount of water absorbed by the aggregate.

oven-dry density—density reached by structural lightweight concrete after being placed in a drying oven at 230 ± 9°F (110 ± 5°C) for a period of time sufficient to reach constant density, as defined in ASTM C567/C567M.

specified density concrete—structural concrete having a specified equilibrium density between 50 to 140 lb/ft^3 (800 to 2240 kg/m^3) or greater than 155 lb/ft^3 (2480 kg/m^3).

structural lightweight aggregate—structural aggregate meeting the requirements of ASTM C330/C330M with bulk density less than 70 lb/ft^3 (1120 kg/m^3) for fine aggregate and less than 55 lb/ft^3 (880 kg/m^3) for coarse aggregate.

CHAPTER 3—STRUCTURAL LIGHTWEIGHT AGGREGATES

3.1—Internal structure of lightweight aggregates

Lightweight aggregates (LWAs) have a low-particle relative density because of the cellular pore system. The cellular structure within the particles is normally developed by heating certain raw materials to incipient fusion; at this temperature, gases evolve within the pyroplastic mass, causing expansion that is retained upon cooling. Strong, durable LWAs contain a uniformly distributed system of pores that have a size range of approximately 5 to 300 µm, developed in a continuous, relatively crack-free, high-strength vitreous phase. Pores close to the surface are readily permeable and fill with water within a few hours to days of exposure to moisture. Interior pores, however, fill extremely slowly, with many months of submersion required to approach saturation. A small fraction of interior pores are essentially noninterconnected and remain unfilled after years of immersion.

3.2—Production of lightweight aggregates

Lightweight aggregates (LWAs) are produced in several ways. Some are produced in manufacturing plants from raw materials, including suitable shales, clays, slates, fly ashes, or blast-furnace slags. Naturally occurring LWAs are mined from volcanic deposits that include pumice and scoria. Pyroprocessing methods include the rotary kiln and sintering processes. The rotary kiln method uses a long, slowly rotating, slightly inclined cylinder lined with refractory materials similar to cement kilns. In the sintering process, a bed of raw materials and fuel is carried by a traveling grate under an ignition hood. After the bed passes the firing hood, the molten slag is agitated with controlled amounts of air or water. No single description of raw material processing is all-inclusive. Local LWA manufacturers can be consulted for physical and mechanical properties of their LWAs and the concrete made with them.

3.3—Aggregate properties

Each of the properties of lightweight aggregate (LWA) may have some bearing on properties of the fresh and hardened concrete. It is important to recognize that properties of lightweight concrete, in common with those of normalweight concrete, are greatly influenced by the cementitious matrix. A list of specific properties of aggregates that may affect the properties of the concrete are included in 3.3.1 through 3.3.8.

3.3.1 *Particle shape and surface texture*—Lightweight aggregates from different sources or produced by different methods may differ considerably in particle shape and texture. Shape may be cubical and reasonably regular, essentially rounded, or angular and irregular. Surface textures may range from relatively smooth with small exposed pores to irregular with small to large exposed pores. Particle shape and surface texture of both fine and coarse aggregates influence proportioning of mixtures in such factors as workability, pumpability, fine-to-coarse aggregate ratio, binder content, and water requirement. These effects are analogous to those obtained with normalweight aggregates with such diverse particle shapes as exhibited by rounded gravel, crushed limestone, traprock, or manufactured sand.

3.3.2 *Specific gravity*—Due to their cellular structure, the specific gravity (relative density divided by density of water) of LWA particles is lower than that of normalweight aggregates. The specific gravity of LWA varies with particle size, being highest for the fine particles and lowest for the coarse particles, with the magnitude of the differences depending on the processing methods. The practical range of coarse LWA specific gravity corrected to the dry condition is from approximately 1/3 to 2/3 that for normalweight aggregates. Particle densities below this range may require more cement to achieve the required strength and may thereby fail to meet the density requirements of the concrete.

3.3.3 *Bulk density*—The bulk density of LWA is significantly lower, due to the cellular structure, than that of normalweight aggregates. For the same grading and particle shape, the bulk density of an aggregate is essentially proportional to particle relative densities. Aggregates of the same particle density, however, may have markedly different bulk densities because of different percentages of voids in the dry-loose or dry-rodded volumes of aggregates of different particle shapes. The situation is analogous to that of rounded gravel and crushed stone, where differences may be as much as 10 lb/ft^3 (160 kg/m^3) for the same particle density and grading in the dry-rodded condition. Rounded and angular LWAs of the same particle density may differ by 5 lb/ft^3 (80 kg/m^3) or more in the dry-loose condition, but the same mass of either will occupy the same volume in concrete. This should be considered in assessing the workability when using different aggregates. Table 3.3.3 summarizes the maximum densities for the LWAs listed in ASTM C330/C330M and C331/C331M.

Table 3.3.3—Bulk-density requirements of ASTM C330/C330M and C331/C331M for dry, loose LWAs

Aggregate size and group	Maximum density, lb/ft^3 (kg/m^3)
ASTM C330/C330M and C331/C331M	
Fine aggregate	70 (1120)
Coarse aggregate	55 (880)
Combined fine and coarse aggregate	65 (1040)

3.3.4 *Strength of lightweight aggregates*—The strength of aggregate particles varies with type and source and is measurable only in a qualitative way. Some particles may be strong and hard while others are weak and friable. For manufactured LWAs, there is usually no reliable correlation between aggregate intrinsic strength and concrete strength for concrete compressive strengths up to approximately 5000 psi (35 MPa). For concrete made with natural LWAs, which are usually lower in strength, there could be a direct relationship between compressive strength and the amount and strength of the aggregate as shown by Videla and Lopez (2000, 2002).

3.3.4.1 *Strength ceiling*—Strength ceiling is the upper limit in compressive and tensile strength above which paste strength is no longer the controlling factor in concrete strength. A mixture is near its strength ceiling when similar mixtures containing the same aggregates and with lower water-cement ratio (w/c) have only slight increases in strengths. At the point of diminishing returns, the mixture no longer produces a correlating increase in strength with the increase of cement content. The strength ceiling for some LWAs could be quite high, approaching that of some normalweight aggregates.

The strength ceiling is influenced predominantly by the coarse aggregate. The strength ceiling can be increased appreciably by reducing the maximum size of the coarse aggregate for most LWAs. This effect is more apparent for the weaker and more friable aggregates. In one case, strength attained in the laboratory, for concrete containing 3/4 in. (19 mm) maximum size of a specific LWA, was 5000 psi (35 MPa). The strength, however, was increased to 6100 and 7600 psi (42 and 52 MPa) when the maximum size of the aggregate was reduced to 1/2 and 3/8 in. (13 and 10 mm), respectively, without changing the cement content. Consequently, concrete unit weight increased by 3 and 5 lb/ft^3 (48 and 80 kg/m^3) when the maximum size of the aggregate was reduced to 1/2 and 3/8 in. (13 and 10 mm), respectively.

Meyer et al. (2003) reported that for a given LWA, the tensile strength may not increase in a manner comparable to the increase in compressive strength. Increases in tensile strength occur at a lower rate relative to increases in compressive strength. This becomes more pronounced as compressive strength exceeds 5000 psi (35 MPa).

3.3.5 *Total porosity*—Proportioning concrete mixtures and making field adjustments of lightweight concrete require a comprehensive understanding of porosity, absorption, and the degree of saturation of lightweight-aggregate particles. The degree of saturation, which is the fractional part of the pores filled with water, can be evaluated from pychnometer measurements. These measurements determine the relative density at various levels of absorption, thus permitting proportioning by the absolute volume procedure. Pores are defined in this guide as the air space inside an individual aggregate particle and voids are defined as the interstitial space between aggregate particles. Total porosity that is found within and between the particles is determined from measured values of particle relative density and bulk density. The following example shows how to determine total porosity:

a) Bulk density (BD), dry, loose 48 lb/ft^3 (770 kg/m^3), BD = 0.77 (ACI 211.2; ASTM C138/C138M)

b) Dry-particle relative density (RD) 87 lb/ft^3 (1400 kg/m^3), RD = 1.4 (ACI 211.2; ASTM C138/C138M)

c) Relative density of the solid particle material without pores 162 lb/ft^3 (2600 kg/m^3), RD = 2.6 (ACI 211.1; ASTM C138/C138M)

- Note that the relative particle density of the solids, which, in this example, is ceramic material without pores (162 lb/ft^3 [2600 kg/m^3] RD = 2.6) was the average value determined by the following procedure.
- Small samples of three different expanded aggregates were ground separately in a jar ball mill for 24 hours. After each sample was reduced, it was tested in accordance with ASTM C188 to determine the relative density of the ground LWA.
- According to Weber and Reinhardt (1995), a small percentage of pores in expanded aggregates are less than 10 μm and exist unbroken within the less than 200 sieve (75 μm) sized particles. The relative densities of the solid vitrified portion of the expanded aggregate are typically in excess of 162 lb/ft^3 (2600 kg/m^3). The true particle porosity may be slightly greater than that determined by the following calculations. When very small pores are encapsulated by a strong, relatively crack-free vitreous structure, the pores are not active in any moisture dynamics.
- Then the total porosity (pores and voids), as graphically shown in Figure 3.3.5, equals

$$0.45 \text{ (voids)} + (0.46 \text{ (pores)} \times 0.55 \text{ (particles)} = 0.70$$

where

A = the fractional solid volume (without pores) of the vitreous material of an individual particle, equals 1.4/2.6 = 0.54;

B = the subsequent fractional volume of pore (within the particle), equals 1.00 – 0.54 = 0.46;

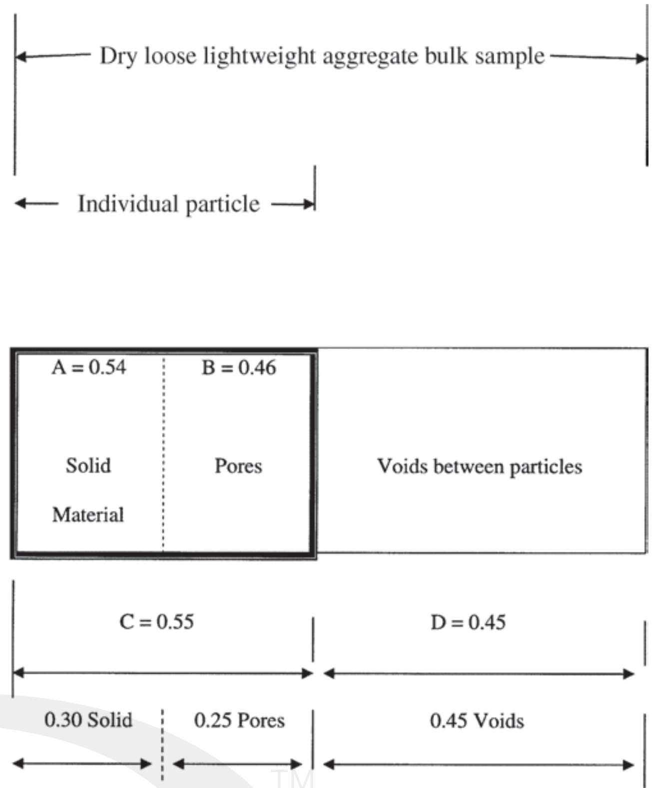


Fig. 3.3.5—Representation of solids, pores, and voids in LWA.

C = for this example, the fractional volume of particles equals 0.77/1.4 = 0.55; and

D = the fractional volume of interstitial voids (between particles) = 1.00 – 0.55 = 0.45.

3.3.6 *Grading*—Grading requirements for LWAs deviate from those of normalweight aggregates (ASTM C33/C33M) by requiring a larger mass of the LWAs to pass through the finer sieve sizes. This modification in grading (ASTM C330/C330M) recognizes the increase in density with decreasing particle size of LWA. The modification yields the same volumetric distribution of aggregates retained on a series of sieves for both lightweight and normalweight aggregates.

LWA is normally stocked in several standard sizes such as coarse, intermediate, and fine aggregate. By combining size fractions or replacing part or all of the fine fraction with a normalweight sand, a wide range of concrete densities can be obtained. For example, replacing normalweight sand with LWA fines will typically decrease the equilibrium concrete density approximately 5 to 10 lb/ft^3 (80 to 160 kg/m^3).

With the advent of modern concrete technology, however, it is seldom necessary to significantly increase cement content to obtain the reduced *w/cm* required to obtain the specified strength. Instead, this can be obtained using water-reducing or high-range water-reducing admixtures.

3.3.7 *Moisture content and absorption*—LWAs, due to their cellular structure, are capable of absorbing more water than normalweight aggregates. Based on a standard ASTM C127 absorption test, LWAs absorb from 5 to 25

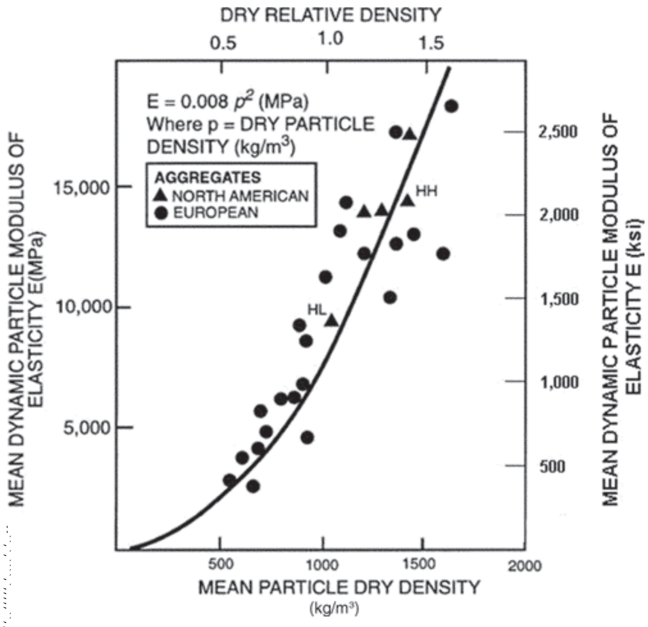


Fig. 3.3.8—Relationship between mean particle density and mean dynamic modulus of elasticity for particles of LWAs (Bremner and Holm 1986).

percent or more by mass of dry aggregate after soaking for 24 hours, depending on the aggregate pore system.

In contrast, most normalweight aggregates will absorb less than 2 percent moisture. The moisture content in a normalweight aggregate stockpile, however, may be as high as 5 percent or more. The important difference is that the moisture content with LWAs is absorbed into the interior of the particles, as well as on the surface, while in normalweight aggregates it is largely surface moisture. This difference becomes important as discussed in the following sections on mixture proportioning, batching, and control of concrete.

Depending on the aggregate pore characteristics, the rate of absorption in LWAs is another factor that has a bearing on mixture proportioning, handling, and properties of concrete. The water, which is internally absorbed in the LWA, is not immediately available to the cement and should not be counted as mixing water or considered in the *w*/*cm* calculations. ACI 211.2 provides details on water content on proportioning.

3.3.8 *Modulus of elasticity of LWA particles*—The modulus of elasticity of concrete is a function of the moduli of its constituents. Concrete may be considered a three-phase material (aggregates, cement paste, and interfacial transition zone); however, as explained in 6.3, lightweight aggregate concrete may be considered a two-phase material consisting of coarse-aggregate inclusions within a continuous mortar fraction that includes cement paste, entrained air, and fine aggregate. For this reason, it is relevant to consider the LWA modulus of elasticity and its influence in the modulus of elasticity on concrete. One approximation to assess LWA modulus of elasticity is to use dynamic measurements on aggregates alone, which have shown a relationship corresponding to the function $E = 0.008p^2$ (Bremner and Holm 1986), where E is the dynamic modulus of elasticity of the particle, in psi (MPa), and p is the dry mean particle density in lb/ft^3 (k/m^3) (Fig. 3.3.8).

Dynamic moduli for typical expanded aggregates have a range of 1.45 to 2.3 × 10^6 psi (10 to 16 GPa), whereas the range for strong normalweight aggregates is approximately 4.35 to 14.5 × 10^6 psi (30 to 100 GPa) (Muller-Rochholz 1979).

CHAPTER 4—SPECIFYING, PROPORTIONING, MIXING, AND HANDLING

4.1—Scope

The proportioning of lightweight concrete mixtures is determined by technical and economical combinations of the constituents that typically include portland cement, aggregate, water, chemical admixtures, or mineral admixtures; therefore, the required properties are developed in both the fresh and hardened concrete. A prerequisite to the selection of mixture proportions is knowledge of the properties of the constituent materials and their compliance with pertinent ASTM specifications.

Based on knowledge of the properties of the constituents and their interrelated effects on the concrete, lightweight concrete can be proportioned to have the properties specified for the finished structure.

4.2—Specifying lightweight concrete

General considerations when specifying lightweight concrete include that:

a) The average strength requirements for lightweight concrete do not differ from those for normalweight concrete for the same degree of field control

b) Tests for splitting tensile strength of concrete are not intended for control or acceptance of the strength of concrete in the field (ACI 318)

c) The analysis of the load-carrying capacity of a lightweight concrete structure, either by cores or load tests, be the same as for normalweight concrete

d) Maximum fresh density be determined before beginning construction of the project

e) Equilibrium density be calculated in accordance with ASTM C567/C567M

4.3—Materials

Lightweight concrete is composed of cement, aggregates, water, and chemical and mineral admixtures similar to normalweight concrete. Admixtures are added to entrain air, reduce mixing water requirements, and modify the setting time or other property of the concrete. Laboratory tests should be conducted on all the ingredients, and trial batches of the concrete mixture proportions be performed with the actual materials proposed for use.

4.3.1 *Cementitious and pozzolanic materials*—These materials should meet ASTM C150/C150M, C595/C595M, C618, C1157/C1157M, or C1240.

4.3.2 *Lightweight aggregates*—For structural concrete, lightweight aggregate (LWA) should meet the requirements of ASTM C330/C330M. Because of differences in particle

Table 4.3.2—Example of mixture design for 3500 psi (24 MPa) lightweight aggregate concrete

	Quantities, lb/yd^3 (kg/m^3)
Cement	510 (303)
Fly ash	130 (77)
Coarse LWA	875 (520)
Normalweight fine aggregate	1350 (800)

strength, the cement contents necessary to produce a specific concrete strength will vary with aggregates from different sources. This is particularly significant for concrete strengths above 5000 psi (35 MPa). An example of an average mixture design of a 3500 psi (24 MPa) compressive strength lightweight concrete is shown by Walter P. Moore and Associates (2012). The use of trial batches is always recommended if using LWA for the first time.

4.3.3 *Normalweight aggregates*—Normalweight aggregates used in lightweight concrete should conform to the provisions of ASTM C33/C33M.

4.3.4 *Admixtures*—Admixtures should conform to appropriate ASTM specifications, and guidance for use of admixtures may be obtained from ACI 232.2R, 233R, 234R, ASTM C260/C260M, ASTM C494/494M, and ASTM C1017/C1017M.

4.4—Mixture proportioning criteria

Chapter 5 includes a broad range of values for many physical properties of lightweight concrete. Specific values depend on the properties of the particular aggregates being used and on other conditions. In proportioning a lightweight-concrete mixture, the architect/engineer is concerned with obtaining predictable values of specific properties for a particular application.

Specifications for lightweight concrete usually require minimum permissible values for compressive and tensile strength, maximum values for slump, and both minimum and maximum values for air content. In addition, a limitation is always placed on the maximum value for fresh and equilibrium density.

From a construction standpoint, the workability of fresh lightweight concrete should also be proportioned for optimum performance. Some properties are interdependent so improvement in one property, such as workability, might affect other properties like density or strength. The final criterion to be met is overall performance in the structure as specified by the architect/engineer.

4.4.1 *Specified properties*

4.4.1.1 *Compressive strength*—Compressive strength is further discussed in Chapter 5. The various types of lightweight aggregate (LWA) available will not always produce similar compressive strengths for concrete of a given cement content, water-cementitious materials ratio (w/cm), and slump.

Compressive strength of structural concrete is specified according to the design requirements of a structure. Normally, strengths specified will range from 3000 to 5000 psi (21 to 35 MPa) and less frequently up to 7000 psi (48 MPa). Although some LWAs are capable of producing consistently strengths over 4000 psi (27.6 MPa), do not expect concrete made with every LWA classified as structural to consistently attain strengths over 4000 psi (27.6 MPa).

4.4.1.2 *Density*—Density is an important consideration in the proportioning of lightweight-concrete mixtures for structural members. While this property depends primarily on aggregate density and the proportions of lightweight and normalweight aggregate, it is also influenced by the cement, water, and air contents. Within limits, concrete density is maintained by adjusting proportions of lightweight and normalweight aggregates. For example, if the cement content is increased to provide higher compressive strength, the density of the concrete might be increased. If several different sources of LWA are available, the optimum balance of cost and concrete performance could require a detailed investigation. Only by comparing concrete of the same compressive strength and equilibrium density can the fundamental differences of concrete made with different aggregates be properly evaluated. In some areas, only a single source of LWA is economically available. In this case, it is only needed to determine the density of concrete that satisfies the economic factors and specified physical properties of the structure.

4.4.1.3 *Modulus of elasticity*—Although values for E_c are not always specified, this information is usually available for concrete made with specific LWAs. This property is further discussed in detail in Chapters 5 and 7.

4.4.1.4 *Slump*—Slump should be consistent with the ability to satisfactorily place, consolidate, and finish the concrete. Slump should be measured at the point of placement.

4.4.1.5 *Entrained-air content*—Air entrainment in lightweight concrete, like normalweight concrete, is required for resistance to freezing and thawing, as shown in ACI 201.2R, Table 1.3.1. In concrete made with some LWAs, it is also an effective means of improving workability. Because entrained air reduces the mixing water requirement, bleeding, and segregation, while maintaining the same slump, it is normal practice to use air entrainment in lightweight concrete regardless of its exposure to freezing and thawing.

The total air content requirement by nominal maximum aggregate size and exposure condition provided in Table 4.4.1.5a for concrete exposed to freezing and thawing apply to lightweight concrete. Recommended total air content by nominal aggregate size for air-entrained lightweight concrete not exposed to freezing and thawing are given in Table 4.4.1.5b.

Attempts to reduce lightweight concrete costs by using a large proportion of normalweight aggregate with a higher than recommended air content are typically counterproductive. This practice is usually self-defeating because compressive strengths are incrementally reduced as the entrained air content is increased beyond the ranges recommended. Although the percentages of entrained air required for workability and resistance to freezing and thawing reduce the concrete density, it is not recommended that air contents above the limits given in Table 4.4.1.5b for lightweight concrete that is not exposed to freezing and thawing be used. It is also recommended that air content be above the limits

Table 4.4.1.5a—Total air content for concrete exposed to cycles of freezing and thawing (Table 4.4.1, ACI 318-11)

Nominal maximum aggregate size, in. (mm)*	Air content, percent	
	Exposure Class F1	Exposure Classes F2 and F3
3/8 (10)	6	7.5
1/2 (13)	5.5	7
3/4 (19)	5	6
1 (25)	4.5	6
1-1/2 (38)	4.5	5.5
2 (51)†	4	5
3 (75)†	3.5	4.5

*Refer to ASTM C33/C33M for tolerance on oversize for various nominal maximum size designations.

†Air content applies to total mixture. When testing concretes, however, aggregate particles larger than 1-1/2 in. (38 mm) are removed by sieving and air content is measured on the sieved fraction (tolerance on air content as delivered applies to this value). Air content of total mixture is computed from value measures on the sieved fraction passing the 1-1/2 in. (38 mm) sieve in accordance with ASTM C231/C231M.

given in Table 4.4.1.5a for lightweight concrete exposed to freezing and thawing be used. Nonstructural or insulating concrete may use higher air contents to lower density.

4.4.1.6 *Other properties*—Other properties such as shrinkage, creep, or thermal insulation, which are not commonly specified, might also be of interest in some specific projects. These properties are further discussed in Chapters 5 and 6.

4.4.2 *Workability*—Workability is an important property of freshly mixed lightweight concrete. The slump test is the most widely used method to measure workability. Similar to normalweight concrete, properly proportioned, lightweight concrete mixtures will have acceptable placement characteristics.

4.4.3 *Heat of hydration*—The heat generated during hydration mainly depends on the type of cement and its fineness, the cement dosage, and the *w*/*cm*. The use of other cementitious materials such as fly ash, silica fume, blast furnace granulated slag, and natural pozzolans have a great influence over the amount and rate of heat released in lightweight concrete as it is in normalweight concrete. Solids in LWA have a specific heat similar to normalweight aggregates; however, because of the presence of voids that can be empty, partially saturated or fully saturated with water the effective heat mass and thermal conductivity might vary widely. This means that for the same amount of heat of hydration generated by the cementitious materials, lightweight concrete might reach higher peak temperature and might reach it more rapidly than normalweight concrete.

4.4.4 *Water-cementitious materials ratio*—The *w*/*cm* can be determined for lightweight concrete proportioned using the specific gravity factor as described in ACI 211.2. When LWAs are adequately prewetted, there will be a minimal amount of water absorbed during mixing and placing. This allows the net *w*/*cm* to be computed with accuracy similar to that associated with normalweight concrete. The time required to reach adequate prewetting and the method of wetting used will vary with each aggregate. The thermal and

Table 4.4.1.5b—Recommended air content for air-entrained lightweight concrete not exposed to freezing (Exposure Class F0*)

Maximum size of aggregate	Air content percent by volume
3/4 in. (19.0 mm)	4.5 ± 1.5 percent
1/2 in. (12.7 mm)	5.0 ± 1.5 percent
3/8 in. (9.5 mm)	5.5 ± 1.5 percent

*See more details in ACI 211.2.

vacuum saturation method may provide adequate prewetting quickly. The sprinkling or soaking method may take several days to reach an adequate prewetted condition. Therefore, it is essential to determine the prewetting method and length of time required. The percent moisture content achieved at an adequately prewetted condition is normally equal or greater than the moisture condition achieved after 24-hour submersion.

4.5—Proportioning and adjusting mixtures

Proportions for concrete should be selected to make the most economical use of available materials to produce concrete of the required physical properties. Basic relationships have been established that provide guidance in developing optimum combinations of materials. Final proportions, however, should be based on field data or established by laboratory trial mixtures that are then adjusted to provide practical field batches, in accordance with ACI 211.2.

The principles and procedures for proportioning normalweight concrete, such as the absolute volume method, may be applied in many cases to lightweight concrete.

4.5.1 *Absolute volume method*—When using the absolute volume method, the volume of fresh concrete produced by any combination of materials is considered equal to the sum of the absolute volumes of cementitious materials, aggregate, net water, and entrained air. Proportioning by this method requires the determination of water absorption and the particle relative density factor of the separate sizes of aggregates in an as-batched moisture condition. The principle involved is that the mortar volume consists of the total of the volumes of cement, fine aggregate, net water, and entrained air. This mortar volume should be sufficient to fill the voids in a volume of rodded coarse aggregate plus sufficient additional volume to provide satisfactory workability. This recommended practice is set forth in ACI 211.1 and represents the most widely used method of proportioning for normalweight concrete mixtures.

The density factor method trial mixture basis is described with examples in ACI 211.2. Displaced volumes are calculated for the cement, air, and net water. The remaining volume is then assigned to the coarse and fine aggregates. This factor may be used in calculations as though it were the apparent particle relative density and should be determined at the moisture content of the aggregate being batched.

4.5.2 *Volumetric method*—The volumetric method, which is described with examples in ACI 211.2, consists of making a trial mixture using estimated volumes of cementitious materials, coarse and fine aggregates, and sufficient added water to produce the required slump. The resultant mixture is observed for workability and finishability characteristics. Tests are made for slump, air content, and fresh density. Calculations are made for yield, which is the total batch mass divided by the fresh density, and for actual quantities of materials per unit volume of concrete. Necessary adjustments are calculated and further trial mixtures made until satisfactory proportions are attained. Information on the dry-loose bulk densities of aggregates, the moisture contents of the aggregates, the optimum ratio of coarse-to-fine aggregates, and an estimate of the required cementitious material to provide the strength desired can be provided by the aggregate supplier.

4.6—Aggregate preparation for mixing

ASTM C94/C94M applies to lightweight concrete as it does to normalweight concrete. Aggregates with relatively low or high water absorption need to be handled according to the procedures that have been proven successful before. The absorptive nature of lightweight aggregate (LWA) requires prewetting to maintain as uniform a moisture content as possible before adding other ingredients to the concrete. The proportioned volume of the concrete is then maintained, and slump loss during transport is minimized or eliminated.

Conditioning of the LWA can be accomplished by any of the following:

a) *Atmospheric*—The length of time required to adequately prewet a LWA is dependent on its absorption characteristics. Uniform prewetting can be accomplished by several methods, including sprinkling, using a soaker hose, and water submersion. These processes can be applied at the aggregate plant, the concrete batch plant, or both.

b) *Thermal*—Achieved by wetting hot LWA. Because of possible thermal shock, the temperature of aggregate when water is introduced should be carefully established for each specific aggregate and it is typically completed at the manufacturer's facility.

c) *Vacuum*—Dry aggregate is introduced into a vessel from which the air can be evacuated. The vessel is then filled with water and returned to atmospheric pressure.

Prewetting minimizes the mixing water absorbed by the aggregate, thereby minimizing the slump loss during pumping. This additional moisture also increases the density of the LWA, which in turn increases the density of the fresh concrete. Increased density due to prewetting will eventually be lost to the atmosphere during the drying process and provides for extended internal curing.

4.7—Placing and finishing

There is little or no difference in the techniques required for placing lightweight concrete from those used in properly placing normalweight concrete. ACI 304R discusses in detail the proper and improper methods of placing concrete. The most important consideration in handling and placing concrete is to avoid segregation of the coarse aggregate from the mortar matrix. The main difference with normalweight concrete is that it is the lower-density coarse aggregate that segregates to the surface instead of sinking to the bottom. The basic principles required for a good lightweight concrete placement are:

a) A workable mixture using minimum water content

b) Equipment capable of expeditiously handling and placing the concrete

c) Proper consolidation

d) Good workmanship

Placing a finishing should be done properly as described in ACI 309R. A well-proportioned lightweight concrete mixture can generally be placed, screeded, and floated with less effort than that required for normalweight concrete. Overvibration or overworking of lightweight concrete should be avoided. Overmanipulation only serves to drive the heavier mortar away from the surface where it is required for finishing and to bring the lower-density coarse aggregate to the surface. Upward movement of coarse lightweight aggregate (LWA) could also occur in mixtures where the slump exceeds the recommendations provided in this chapter.

4.7.1 *Pumping lightweight concrete*

4.7.1.1 *General considerations*—Unless the LWA is satisfactorily prewetted, they may adsorb mixing water that can cause difficulty during the pumping of the concrete. For this reason, it is important to adequately precondition the aggregate by prewetting before batching the concrete (Refer to 4.4.2 and 4.6).

4.7.1.2 *Proportioning pump mixtures*—When considering pumping lightweight concrete, some adjustments may be necessary to achieve the desired characteristics. The architect/engineer and contractor should be familiar with any mixture adjustments required before the decision is made as to the method of placement. Pumping lightweight concrete is extensively covered by the Expanded Shale, Clay and Slate Institute (1996).

When the project requirements call for pumping, the following general guidelines apply. These guidelines are based on the use of lightweight coarse aggregate and normalweight fine aggregate.

a) Prewet LWA to a recommended moisture content.

b) Maintain a 564 lb/yd^3 (335 kg/m^3) minimum cementitious content.

c) Use selected liquid and mineral admixtures that will aid in pumping.

d) To facilitate pumping, adjustments in the standard mixture proportion may result in a slight reduction in the volume of coarse aggregate, with a corresponding increase in the volume of fine aggregate.

e) Cementitious content should be sufficient to accommodate a 4 to 6 in. (100 to 150 mm) slump at the point of placement.

f) Use a well-graded natural sand with a good particle shape and a fineness modulus range of 2.2 to 2.7.

g) Use a properly combined coarse- and fine-aggregate grading. The grading should be made by absolute volume rather than by mass to account for differences in relative density of the various particle sizes.

Sometimes it is advisable to plan on various mixture designs as the height of a structure or distance from the pump to the point of discharge changes. Final evaluation of the concrete shall be made at the discharge end of the pumping system (ACI 304.2R).

4.7.1.3 *Pump and pump system*—The following are key items pertinent to the pump and pumping system.

a) Use the largest diameter line available, with a recommended minimum of 5 in. (125 mm) diameter without reducers

b) All lines should be clean, the same size, and buttered with grout at the start

c) Avoid rapid size reduction from the pump to line

d) Reduce the operating pressure by

i. Slowing down the rate of placement

ii. Using as much steel line and as little rubber line as possible

iii. Limiting the number of bends

iv. Making sure the lines are gasketted and braced by a thrust block at turns

Observers present should include representatives of the contractor, ready-mixed concrete producer, architect/engineer, pumping service, testing agency, and aggregate supplier. In the pump trial, the height and length to the delivery point of the concrete to be moved should be taken into account. Because most test locations will not allow the concrete to be pumped vertically as high as it would be during the project, the following rules of thumb can be applied for the horizontal runs with steel lines (American Concrete Pumping Association 2011):

1.0 ft (0.3 m) vertical = 4.0 ft (1.2 m) horizontal

1.0 ft (0.3 m) rubber hose = 3.0 ft (0.9 m) of steel

90-degree bend = 9.0 ft (2.7 m) of steel

4.7.2 *Finishing horizontal surfaces*—Satisfactory horizontal surfaces are achieved with properly proportioned quality materials, skilled supervision, and good workmanship. The quality of the finishing will be in direct proportion to the efforts expended to ensure that proper principles are observed throughout the finishing process. Finishing techniques for lightweight concrete floors are described in ACI 302.1R.

For lightweight concrete used as part of a fire-rated assembly, consult the listing agency for concrete properties specified for that assembly. Many UL Fire-Resistance Directory-rated assemblies require specific concrete properties, including entrained air content (Underwriters Laboratories Inc., 2014).

4.7.2.1 *Slump*—Slump is an important factor in achieving a good floor surface with lightweight concrete and generally should be limited to a maximum of 5 in. (125 mm) at the point of placement. A lower slump of approximately 3 in. (75 mm) imparts sufficient workability and also maintains cohesiveness and body, thereby preventing the lower-density coarse particles from working to the surface. This is the reverse of normalweight concrete where segregation results in an excess of mortar at the surface. Because the slump test is affected by the weight of concrete, a 3 in. (75 mm) slump lightweight concrete is usually more workable than its normalweight counterpart.

4.7.2.2 *Surface preparation*—Surface preparation before troweling is best accomplished with magnesium or aluminum screeds and floats, which minimize surface tearing and pullouts.

4.8—Curing

Curing of lightweight aggregate (LWA) concrete can occur in a traditional manner in which moisture is applied to the surface of the concrete, and by internal curing that occurs by the release of water absorbed within the pores of LWA. Ultimate performance of the concrete will be influenced by the extent of curing provided. External curing of the concrete should begin upon completion of the finishing operation. ACI 302.1R and 308.1 contain information on proper external curing of concrete floor slabs.

4.9—Laboratory and field control

Changes in absorbed moisture or relative density of lightweight aggregate (LWA), which result from variations in initial moisture content or grading and variations in entrained-air content, suggest that frequent checks of the fresh concrete should be made at the job site to ensure consistent quality (ACI 211.1). Sampling should be in accordance with ASTM C172/C172M. Tests normally required include ASTM C567/C567M, ASTM C143/C143M, ASTM C173/C173M, and ASTM C31/C31M.

At the job start, the fresh properties, density, air content, and slump of the first batch or two should be determined to verify that the concrete conforms to the laboratory mixture. Small adjustments may then be made as necessary. In general, when variations in fresh density exceed 3 lb/ft^3 (50 kg/m^3), an adjustment in batch weights may be required to meet specifications. The air content of lightweight concrete should not vary more than ±1-1/2 percentage points from the specified value to avoid adverse effects on concrete density, compressive strength, workability, and durability.

CHAPTER 5—PHYSICAL AND MECHANICAL PROPERTIES OF STRUCTURAL LIGHTWEIGHT AGGREGATE CONCRETE

5.1—Scope

This chapter presents a summary of the properties of lightweight concrete. Information is based on laboratory studies and the records of a large number of existing structures that have provided satisfactory service for more than eight decades.

5.2—Compressive strength

Compressive strength levels commonly required by the construction industry for design strengths of cast-in-place, precast, or prestressed concrete are economically obtained with lightweight concrete (Shideler 1957; Hanson 1964; Holm 1980a; Trumble and Santizo 1992. Design strengths of 3000 to 5000 psi (21 to 35 MPa) are common. In precast and prestressing plants, design strengths above 5000 psi (35 MPa) are usual. In several civil structures, such as the Heidrun Platform and Norwegian bridges, concrete cube strengths of 8700 psi (60 MPa) have been specified (FIP 2000). Experiences in North America with high design strengths of

10,000 psi (69 MPa) for prestressed lightweight concrete girders have been reported (Meyer and Kahn 2004; Kahn and Lopez 2005). As discussed in Chapter 3, all aggregates have strength ceilings, and with LWAs, the strength ceiling generally can be increased by reducing the maximum size of the coarse aggregate. As with normalweight concrete, water-reducing plasticizing and mineral admixtures are frequently used with lightweight concrete mixtures to increase workability and facilitate placing and finishing.

5.3—Density of lightweight concrete

5.3.1 *Fresh density*—The fresh density of lightweight concrete is a function of mixture proportions, air contents, water demand, particle relative density, and absorbed moisture content of the lightweight aggregate (LWA). Decrease in the density of exposed concrete is due to moisture loss that, in turn, is a function of ambient conditions and surface area/volume ratio of the member. The architect/engineer should consult with the aggregate supplier before specifying a maximum fresh density as limits of acceptability at time of placement.

Fresh density is specified for field control using a unit weight bucket. Measurements on 6 x 12 in. (150 x 300 mm) cylinders will average 2.5 lb/yd^3 (40 kg/m^3) higher than field measurements on 0.5 ft^3 (0.014 m^3) unit weight bucket (ASTM C567/C567M).

Although there are numerous structural applications of lightweight concrete using both coarse and fine LWAs, the customary commercial practice in North America is to design concrete with natural sand fine aggregates. Long-span bridges using concrete with three-way blends (coarse and fine LWAs and small amounts of natural sand) have provided long-term durability and structural efficiency (density/strength ratios) (Holm and Bremner 1994). Earlier research reports (Kluge et al.1949; Price and Cordon 1949; Reichard 1964; Shideler 1957) compared concrete containing both fine and coarse LWAs with reference normalweight concrete. Later studies (Hanson 1964; Pfeifer 1967) supplemented the early findings with data based on lightweight concrete in which the fine aggregate was a natural sand. There are also several reported projects where lightweight fines and normalweight coarse aggregate have been used with precast elevated floors on steel decks and post-tension applications.

5.3.2 *Equilibrium density*—Self loads used for design should be based on equilibrium density that, for most conditions and members, is achieved after 90 days of air drying. Extensive North American studies demonstrated that, despite wide initial variations of aggregate moisture content, equilibrium density was found to be between 3 and 5 lb/ft^3 (50 and 83 kg/m^3) above oven-dry density (Fig. 5.3.2) for lightweight concrete. European recommendations for in-service density are similar (FIP 1983). Concrete containing high cementitious contents, and particularly those containing efficient pozzolans, will tend to dry less, developing equilibrium densities more similar to fresh densities and in longer drying periods.

For precast members or when critical load conditions occur prior to the equilibrium density being achieved, the

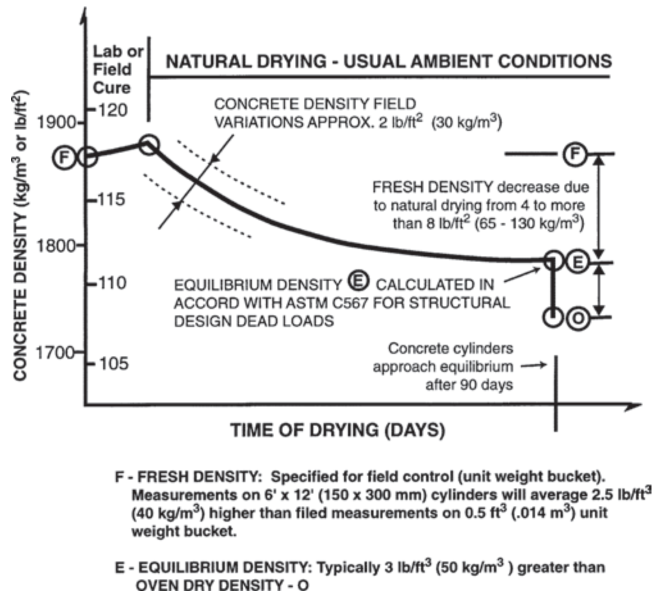


Fig. 5.3.2—Concrete density versus time of drying for structural lightweight concrete (Holm 1994).

fresh density should be used to compute self loads for design calculations.

When weights and moisture contents of all the constituents of the concrete are known, a calculated equilibrium density can be determined according to ASTM C567/C567M from Eq. (5.3.2a) and (5.3.2b)

$$O = (W_{df} + W_{dc} + 1.2W_{ct})/V \qquad (5.3.2a)$$

$$E = O + 3\ \text{lb/ft}^3\ (O + 50\ \text{kg/m}^3) \qquad (5.3.2b)$$

where 1.2 is the factor to account for water of hydration.

5.4—Tensile strength

5.4.1 *Splitting tensile strength*—The splitting tensile strength of concrete cylinders, determined according to ASTM C496/C496M and ACI 318, are not intended for control or acceptance of the strength of concrete in the field.

5.4.1.1 *Moist-cured concrete*—Fig. 5.4.1.1 indicates a narrow range of this property for continuously moist-cured lightweight concrete. The splitting tensile strength of the normalweight reference concrete is nearly intermediate within these ranges.

5.4.1.2 *Air-dried concrete*—The tensile strength of lightweight concrete that undergoes drying is more relevant in respect to the shear strength behavior of structural concrete. During drying of the concrete, moisture loss progresses at a slow rate into the interior of concrete members, resulting in the development of tensile stresses at the exterior faces and balancing compressive stresses in the still-moist interior zones. Thus, after drying, the tensile resistance of lightweight concrete to external loading will be reduced from that indicated by continuously moist-cured concrete (Hanson 1961; Pfeifer 1967). Figure 5.4.1.2 indicates this reduced strength for concrete that has been moist-cured for 7 days followed by 21 days storage at 50 percent relative humidity (ASTM

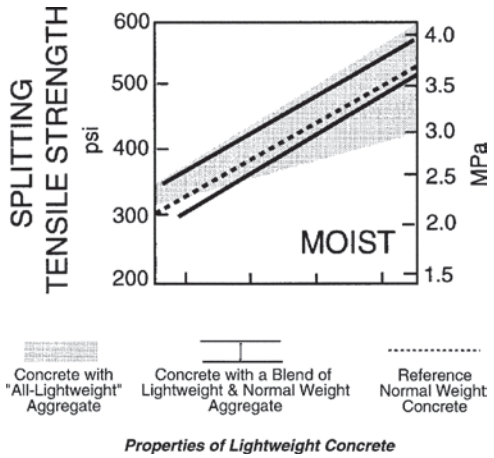


Fig. 5.4.1.1—Splitting tensile strength: moist-cured concrete.

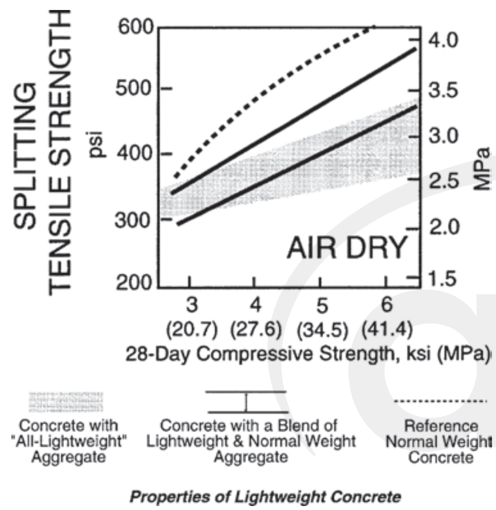


Fig. 5.4.1.2—Splitting tensile strength: air-dried concrete.

C330/C330M). The splitting tensile strength of lightweight concrete varies from approximately 70 to 100 percent that of the normalweight reference concrete when comparisons are made at equal compressive strength.

Replacement of the lightweight fines by sand generally increases the splitting tensile strength of lightweight concrete subjected to drying (Pfeifer 1967; Ivey and Bluth 1966). In some cases, this increase is nonlinear with respect to the sand content so that, for some aggregates, partial sand replacement is as beneficial as complete replacement.

Tests have shown that the diagonal tensile strengths of beams and slabs correlate closely with the splitting tensile strength of lightweight concrete (Hanson 1961).

Tensile strength tests on lightweight concrete specimens that undergo some drying correlate well with the behavior of concrete in actual structures. Moisture loss progressing slowly into the interior of concrete members will result in the development of outer envelope tensile stresses that balance the compressive stresses in the still-moist interior zones. ASTM C496/C496M requires a 7-day moist and 21-day laboratory air drying at 73.4°F (23°C) at 50 percent relative humidity before conducting splitting tests. Lightweight concrete splitting tensile strengths vary from approximately 75 to 100 percent of normalweight concrete of equal compressive strength. Replacing lightweight fine aggregate with normalweight fine aggregate will normally increase tensile strength. Further, natural drying will increase tensile-splitting strengths.

5.4.2 *Modulus of rupture*—The modulus of rupture (ASTM C78/C78M) is also a measure of the tensile strength of concrete. Figure 5.4.2a and 5.4.2b indicate ranges for normally cured and steam-cured concrete, respectively, when tested in the moist condition. For prism specimens, a nonuniform moisture distribution will reduce the modulus of rupture, but the moisture distribution within the structural member is not known and is unlikely to be completely saturated or dry. Studies have indicated that modulus of rupture tests of concrete undergoing drying are extremely sensitive to the transient moisture content and, under these conditions, may not furnish reliable results that are satisfactorily reproducible (Hanson 1961).

The values of the modulus of rupture determined from tests on high-strength lightweight concrete yield inconsistent correlation with code requirements. While Huffington (2000) reported that the tensile splitting and modulus of rupture test results generally met AASHTO requirements for high-strength lightweight concrete, Nassar (2002) found that the modulus of rupture of high-strength lightweight concrete was approximately 60 to 85 percent of code requirements of $\lambda \times 7.5\sqrt{f_c'}$, where λ for sand lightweight concrete is recommended to be 0.85. Meyer (2002), however, found that no additional reduction was required for high-strength lightweight concrete mixtures. It is recommended to perform additional testing for the specific mixtures, for example, LWA, to verify the 0.85 factor for high-strength lightweight concrete.

5.5—Modulus of elasticity

The modulus of elasticity of concrete depends on the relative amounts of paste and aggregate and the modulus of each constituent (LaRue 1946; Pauw 1960). Normalweight concrete has a higher E_c than lightweight concrete because the moduli of sand, stone, and gravel are greater than the moduli of lightweight aggregate (LWA). Figure 5.5 gives the range of modulus of elasticity values for lightweight concrete. Generally, the modulus of elasticity for lightweight concrete is considered to vary between 50 and 75 percent of the modulus of sand and gravel concrete of the same strength. Variations in LWA grading usually have little effect on modulus of elasticity if the relative volumes of cement paste and aggregate remain fairly constant.

While there is agreement that the modulus of elasticity of concrete depends on its density and compressive strength as expressed by the equation for E_c given in ACI 318, some research has shown that such formulas may not adequately represent the relationship between density and compressive strength and the modulus of elasticity.

Cook (2007) has developed a new formula for modulus of elasticity that provides a better estimate of lightweight

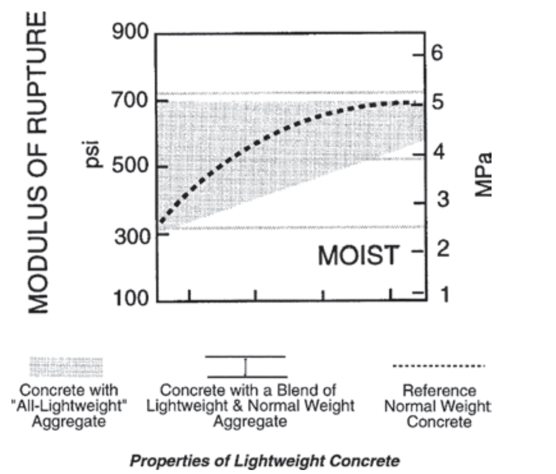


Fig. 5.4.2a—Modulus of rupture: normally cured concrete.

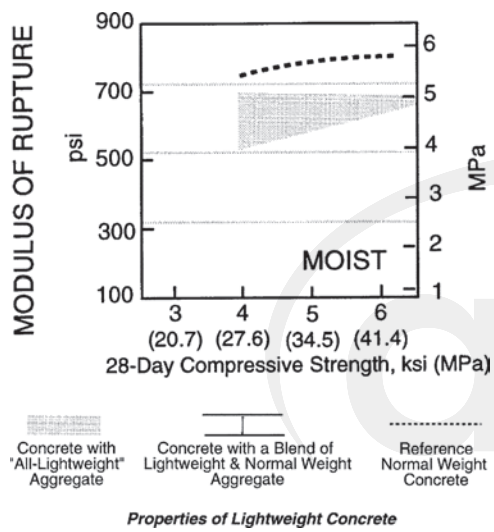


Fig. 5.4.2b—Modulus of rupture: steam-cured concrete.

concrete and high-strength concrete than the following equation.

$$E_c = w_c^{2.687} \cdot (f_c')^{0.24} \tag{5.5}$$

The formula has proven that it may be used for values of w_c between 100 and 155 lb/ft^3 (1600 and 2480 kg/m^3) and strength levels of 1000 to 23,000 psi (7 to 158 MPa). Concretes in service may deviate from this formula; thus, when an accurate evaluation of E_c is required for a particular concrete, a laboratory test in accordance with the methods of ASTM C469/C469M should be carried out.

5.6—Poisson's ratio

Tests to determine Poisson's ratio of lightweight concrete by resonance methods showed that it varied only slightly with age, strength, or aggregate used, and that the values varied between 0.16 and 0.25 with the average being 0.21 (Reichard 1964). Tests to determine Poisson's ratio by the static method for lightweight and normalweight concrete

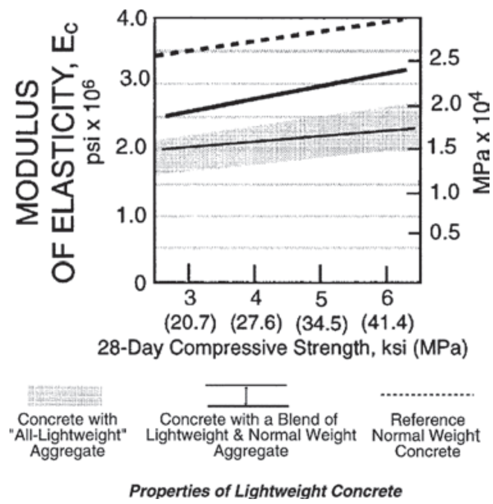


Fig. 5.5—Modulus of elasticity.

gave values that varied between 0.15 and 0.25 and averaged 0.2.

While this property varies slightly with age, test conditions, and physical properties of the concrete, a value of 0.20 may usually be assumed for practical design purposes. An accurate evaluation can be obtained for a particular concrete by testing according to ASTM C469/C469M.

5.7—Ultimate strain

Figure 5.7 gives a range of values for ultimate compressive strain for concrete containing both coarse and fine lightweight aggregate (LWA) and for normalweight concrete. These data were obtained from unreinforced specimens eccentrically loaded to simulate the behavior of the compression side of a reinforced beam in flexure (Hognestad et al. 1955). The diagram indicates that the ultimate compressive strain of most lightweight concrete and of the reference normalweight concrete might be somewhat greater than the value of 0.003 assumed for design purposes.

5.8—Creep

Creep is the increase in strain of concrete under a sustained stress. The effects of creep properties of concrete may be either beneficial or detrimental, depending on the structural conditions. Creep exercises a beneficial effect by relieving undesirable stresses due to shrinkage and to other imposed deformations like extreme initial temperatures, or settlement of supports and yielding of restraints. The long-term reliability of structures, however, may be adversely affected as creep may lead to prestress loss, loss of camber, or excessive long-time deflection, and to unfavorable stress redistribution consequent to delayed change of the structural system in case of sequential construction. In nonhomogeneous and hybrid structures, creep-induced stress redistributions transfer stresses from the parts of the structure creeping more to the parts creeping less, or from concrete to steel elements.

The time-dependent behavior of concrete (creep and shrinkage) can also have an influence on the ultimate limit state of buckling in case of slender structures such as columns, walls, arches, and thin shells with a low degree of restraint, and whenever second-order effects are important. In these cases, the increase of deflections with time reduces the safety margins with respect to instability, which may lead to long-term buckling collapse. Influence on the safety margins with respect to the ultimate limit state of strength depends on the ductile behavior of the structure and it can become a concern in cases where the ultimate limit state is governed by nonplastic failure of concrete.

The effects of creep, along with those of drying shrinkage, should be considered in structural designs. A guide for their evaluation can be found in ACI 209.1R and ACI 209.2R. 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 long-time deflection, prestress loss, or loss of camber. The effects of creep, along with those of drying shrinkage, should be considered in structural designs.

5.8.1 *Factors influencing creep*—Creep and drying shrinkage are closely related phenomena that are affected by many factors, such as type and content of aggregate, type of cement, grading of aggregate, water content of the mixture, moisture content of aggregate at time of mixture, amount of entrained air, age at initial loading, magnitude of applied stress, method of curing, size of specimen or structure, relative humidity of surrounding air, and period of sustained loading.

5.8.2 *Normally cured concrete*—Figure 5.8.2 shows the range in values of specific creep (creep per psi [MPa] of sustained stress) for normally cured concrete, as measured in the laboratory (ASTM C512/C512M) when under constant loads for 1 year. These diagrams were prepared with the aid of two common assumptions: superposition of creep effects is valid (that is, creep is proportional to stress within working stress ranges); and shrinkage strains, as measured on nonloaded specimens, may be directly separated from creep strains. The band for lightweight concrete containing normalweight sand is considerably more narrow than that for concrete containing both fine and coarse lightweight aggregate (LWA). Figure 5.8.2 suggests that an extremely effective method of reducing creep of lightweight concrete is to use a higher-strength concrete. A strength increase from 3000 to 5000 psi (21 to 35 MPa) significantly reduces creep.

5.8.3 *Steam-cured concrete*—Several investigations have shown that creep may be significantly reduced by low-pressure curing and greatly reduced by high-pressure steam curing. Figure 5.8.3 shows that the reduction for low-pressure steamed concrete may be from 25 to 40 percent of the creep of similar concrete subjected only to moist curing.

5.8.4 *Internal curing effect*—One study (Lopez 2005) has shown that internally stored water in the LWA might reduce creep as a result of a reduced concrete permeability and the increase in internal relative humidity, both factors that make creep mechanisms difficult to develop (refer to Chapter 9).

5.9—Shrinkage

Shrinkage, either autogenous or drying, is an important property that can affect the extent of cracking, prestress loss, effective tensile strength, and warping. Note that large-size concrete members, or those in high ambient relative humidity, might undergo substantially less shrinkage than

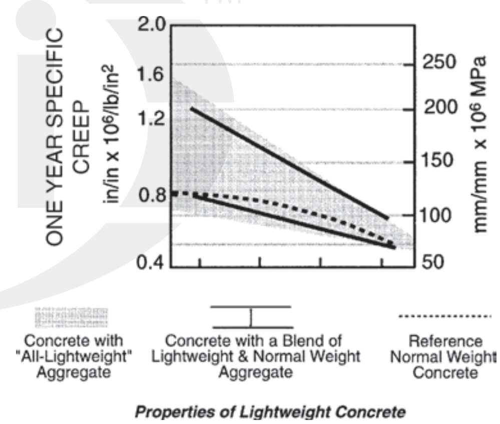


Fig. 5.8.2—Creep: normally cured concrete.

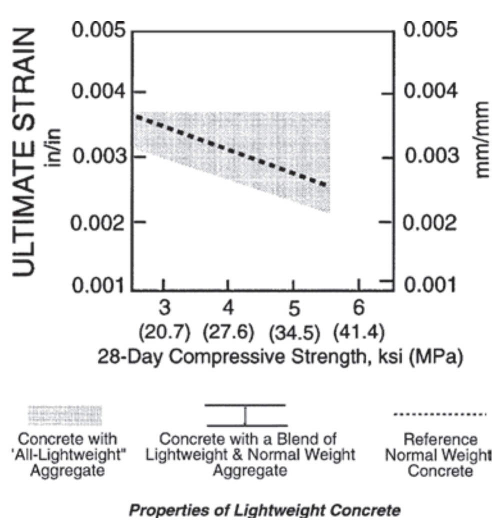


Fig. 5.7—Ultimate strain.

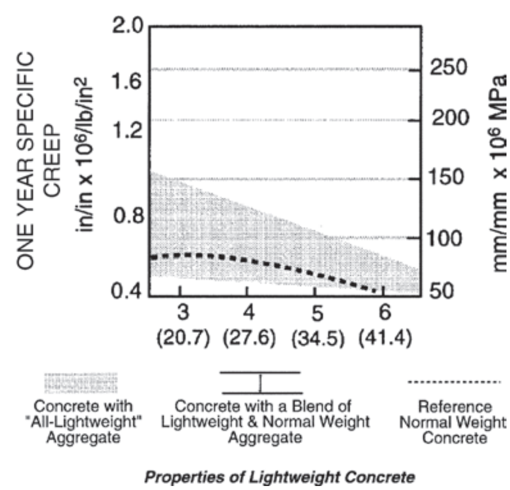


Fig. 5.8.3—Creep: steam-cured concrete.

that exhibited by small laboratory specimens stored at 50 percent relative humidity.

5.9.1 *Normally cured concrete*—Figure 5.9.1 indicates wide ranges of shrinkage values after 1 year of drying for lightweight concrete with normalweight sand. Noting the position within these ranges of the reference concrete, it appears that low-strength lightweight concrete generally has greater drying shrinkage than the reference concrete. At higher strengths, however, some lightweight concrete exhibits lower shrinkage. Partial or full replacement of the lightweight fine aggregate by natural sand usually reduces shrinkage for concrete made with most LWA.

5.9.2 *Atmospheric steam-cured concrete*—Figure 5.9.2 demonstrates the reduction of drying shrinkage obtained through steam curing. This reduction may vary from 10 to 40 percent. The lower portion of this range is not greatly different from that for the reference normalweight concrete.

5.10—Bond strength

ACI 318 includes a factor for development length of 1.3 to reflect the lower tensile strength of lightweight-aggregate concrete and allows that factor to be taken as $6.7\sqrt{f_c'}/f_{ct} \geq 1.0$ if the average splitting strength f_{ct} of the lightweight-aggregate concrete is specified. In general, design provisions require longer development lengths for lightweight-aggregate concrete.

Due to the lower strength of the aggregate, lightweight concrete should be expected to have lower tensile strength, fracture energy, and local bearing capacity than normalweight concrete with the same compressive strength. As a result, the bond strength of bars cast in lightweight concrete, with or without transverse reinforcement, is lower than that in normalweight concrete, with that difference tending to increase at higher strength levels (Fig. 5.10) (Shideler 1957).

ACI Committee 408 (1966) emphasized the paucity of experimental data on the bond strength of reinforced concrete elements made with lightweight-aggregate concrete.

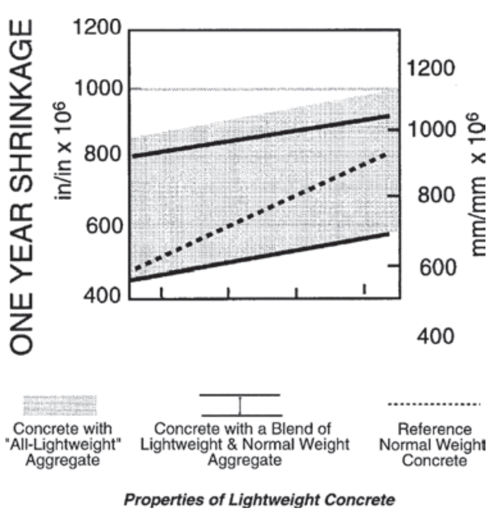


Fig. 5.9.1—Drying shrinkage: normally cured concrete.

Research (Lyse 1934; Petersen 1948; Shideler 1957; Berg 1981; Martin 1982; Clarke and Birjandi 1993) concluded that the bond strength of steel in lightweight-aggregate concrete was comparable to that of normalweight concrete. In contrast, there are studies that indicate significant differences between bond strengths in lightweight and normalweight aggregate concrete (Robins and Standish 1982; Mor 1992) where lightweight concrete showed bond strength between 45 and 88 percent of that observed in normalweight concrete.

Overall, the data indicate that the use of lightweight concrete can result in bond strengths that range from nearly equal to 65 percent to similar or even higher values than those obtained with normalweight concrete. For special structures such as long-span bridges with very high strengths and major offshore platforms, a testing program based on the materials selected to the project is recommended.

5.11—Thermal expansion

Determinations (Price and Cordon 1949) of linear thermal expansion coefficients made on lightweight concrete indicate values are 4 to 5 × 10^{-6} in./in./°F (7 to 11 × 10^{-6} mm/mm/°C), depending on the amount of natural sand used.

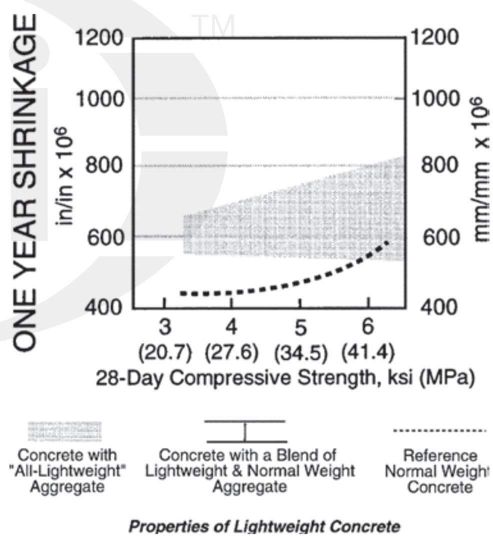


Fig. 5.9.2—Drying shrinkage: steam-cured concrete.

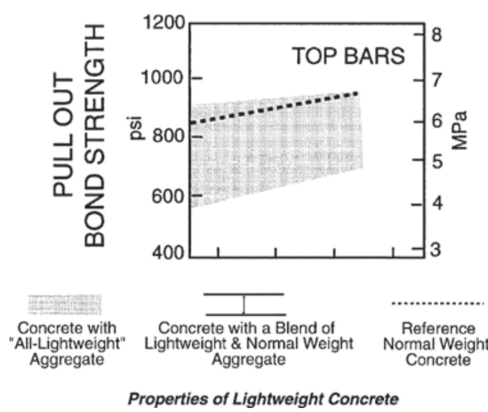


Fig. 5.10—Bond strength: pullout tests.

5.12—Heat flow properties

5.12.1 *Thermal conductivity*—The value of thermal conductivity k is a specific property of a material rather than of a construction and is a measure of the rate at which heat (energy) passes perpendicularly through a unit area of homogeneous material of unit thickness for a temperature gradient of 1 degree (where k = Btu/hour ft^2 [°F/in.] [SI units, k = W/m · °C])

Thermal resistivity is the resistance per unit of thickness and is equal to l/k.

Thermal conductivity has been determined for concrete ranging in oven-dry density from less than 20 lb/ft^3 to over 200 lb/ft^3 (320 to 3200 kg/m^3). Conductivity values are generally obtained from guarded hot-plate specimens (ASTM C177) tested in an oven-dry condition.

When conductivity values for concrete having a wide range of densities are plotted against oven-dry density, best-fit curves show a general dependence of k on density, as shown in Fig. 5.12.1. Different investigators have found different relationships, which are accounted for by differences in aggregate mineralogical type and microstructure, as well as in grading. Differences due to cement content, as well as matrix density and pore structure, also result in changes of the conductivity. Some differences in test methods and specimen sizes also exist.

Valore (1980) plotted over 400 published test results of density w against the logarithm of conductivity k and suggested the following equation

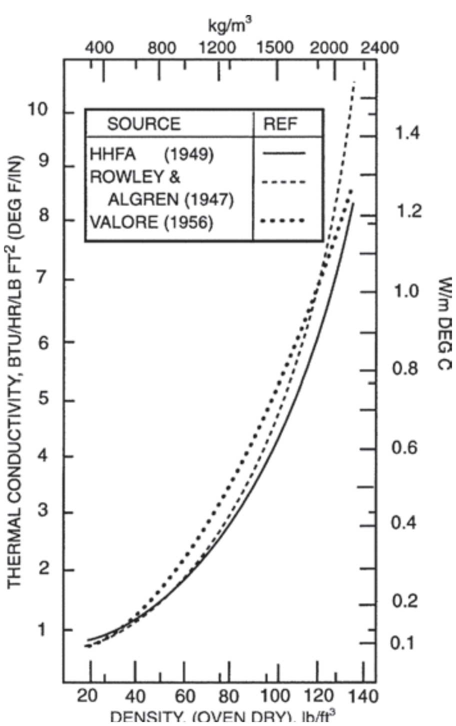


Fig. 5.12.1—Relation of average thermal conductivity k values of concrete in oven-dry condition to density (Valore 1980).

$$k = 0.5e^{0.02 \cdot W_C}, \text{Btu/h} \cdot \text{ft}^2\ {}^\circ\text{F} \quad (k = 0.072e^{0.00125 \cdot W_C}, \text{W/m}\,{}^\circ\text{C}) \tag{5.12.1}$$

An accurate k value for a given concrete, based on ASTM C177, is preferable to a calculated value. For usual construction, however, the formula provides a good base for estimating k for concrete in the oven-dry condition and, in addition, may easily be revised for air-dry conditions.

5.12.1.1 *Effect of moisture on thermal conductivity of concrete*—Increasing the free moisture content of hardened concrete causes an increase in thermal conductivity. Valore (1980) stated that k increases by 6 percent for each 1 percent increment in free or evaporable moisture by weight in relation to oven-dry density. The corrected conductivity may be calculated as follows

$$k(\text{corrected}) = k(\text{oven-dry}) \times \left(1 + 6\frac{(w_m - w_o)}{w_o}\right) \tag{5.12.1.1}$$

where w_m and w_o are densities in moist and oven-dry conditions, respectively.

5.12.1.1.1 *Recommended moisture factor correction for thermal conductivity*—A 6 to 9 percent increase in k per 1 percent increase of moisture content, by weight, is recommended for lightweight-aggregate concrete of all types and normalweight concrete, respectively. These factors are applicable where exposure conditions or other factors produce moisture contents known to depart appreciably from recommended standard moisture contents of 2 percent for ordinary concrete and 4 percent by volume for lightweight concrete.

A simple constant factor can be used for masonry units and concrete under conditions of normal protected exposure. The k values, when corrected for equilibrium moisture in normal protected exposure, are increased by 20 percent over standard values for oven-dry concrete. This results in modification of Valore's (1980) equation, as shown in Fig. 5.12.1.3

$$k = 0.6e^{0.02 \cdot W_C}, \text{Btu/h} \cdot \text{ft}^2\ {}^\circ\text{F} \quad (k = 0.086e^{0.00125 \cdot W_C}, \text{W/m}\,{}^\circ\text{C}) \tag{5.12.1.3}$$

where e = 2.71828.

5.12.1.2 *Equilibrium moisture content of concrete*—Concrete in a structure is not in an oven-dry condition; it is in equilibrium with the relative humidity of the environment. Because k values shown are for oven-dry concrete, it is necessary to know the moisture content for concrete in equilibrium with its normal environment in service and then apply a moisture correction factor for estimating k under anticipated service conditions.

The relative humidity within masonry units in a wall will vary with type of occupancy, geographical location, exposure, and with the seasons, and it is normally assumed to be 50 percent. Also, it is normally assumed that exterior surfaces of single-wythe walls are protected by a breathing-type paint, stucco, or surface-bonding fibered-cement

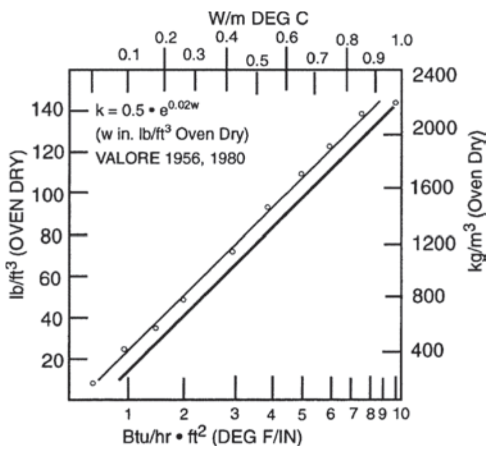


Fig. 5.12.1.3—Relation of average k *values of concrete to dry density (Valore 1980).*

plaster. For single-wythe walls, such protection is necessary to minimize rain penetration. For cavity walls, the average moisture content of both wythes, even with the exterior wythe unpainted, will be approximately equal to that of the protected single-wythe wall.

Data from various sources for normalweight and lightweight concrete and for low-density insulation concrete have been summarized by Valore (1956, 1980). Average long-term moisture contents for concrete are in good agreement with data given herein for concrete masonry units.

Under certain conditions, condensation within a wall can cause high moisture content. This should be considered in selecting an appropriate conductivity value.

5.12.1.3 *Cement paste as insulating material*—The oven-dry density of mature portland cement paste ranges from 100 lb/ft^3 (1600 kg/m^3) for a *w/cm* of 0.4, to 67 lb/ft^3 (1075 kg/m^3) for a *w/cm* of 0.8. This range for *w/cm* encompasses structural concrete. Other data on moist-cured neat cement cellular concrete (aerated cement paste) permit the development of *k*-density relationships for oven-dried, air-dried, and moist pastes (Valore 1980). The latter work shows that neat cement cellular concrete and autoclaved cellular concrete follow a common *k*-density curve.

5.12.2 *Thermal transmittance*—*U*-value (Λ), or thermal transmittance, is a measure of the rate of heat flow through a building construction or the amount of heat transmitted through 1 ft^2 (1 m^2) of the element per second when the temperature of environment differs by 1°F (1 K) from one side to another of a wall or roof. It is expressed as follows:

$$U = \text{Btu/h} \cdot \text{ft}^2 \cdot {}^\circ\text{F}\ (U = \text{W/m}^2 \cdot \text{K}) \quad (5.12.2a)$$

The *U*-value of a wall or roof consisting of homogeneous slabs of material of uniform thickness is calculated as the reciprocal of the sum of the thermal resistance of individual components of the construction

$$U = \frac{1}{R_1 + R_2 + R_3 + \ldots R_n} \quad (5.12.2b)$$

Where R_1, R_2, and higher are resistances of the individual components and include standard constant R values for air spaces and interior and exterior surface resistances. R is expressed as R = hour · ft^2 · °F/Btu (R = m^2 · K/W). Thermal resistances of individual solid layers of a wall are obtained by dividing the thickness of each layer by the thermal conductivity k for each particular material. If there are different areas with varying rates of heat flow, the mean thermal transmittance of the wall or roof may be calculated as the weighted addition of each area with its correspondent transmittance.

Because of the low heat storage capacity of lightweight structures, the higher thermal resistance required is accomplished by using lightweight concrete.

5.12.3 *Heat capacity/specific heat*—When prewetted, LWAs have a higher heat capacity (c) than normalweight aggregates, mainly caused by the presence of water within the aggregate pores. Lopez (2005) estimated that the heat capacity of a rotary kiln expanded slate is 1.37 times the capacity of granite with ranges between 0.19 and 0.26 Btu/lb°F (0.80 and 1.10 kJ/kgK). This produces a concrete with higher heat capacity 0.21 Btu/lb°F (0.86 kJ/kgK) than normalweight concrete 0.20 Btu/lb°F (0.84 kJ/kgK) (FIP 1983). Researchers propose that normalweight concrete heat capacity is approximately 0.24 Btu/lb°F (1.00 kJ/kgK), immediately after casting, while those of expanded shale and expanded clay concrete are 0.30 and 0.32 Btu/lb°F (1.25 and 1.35 kJ/kgK), respectively, due the higher water content of LWAs (FIP 1983).

5.12.4 *Thermal diffusivity*—Usually represented by α, thermal diffusivity indicates local temperature change and how it spreads though the material. Thermal diffusivity depends on the concrete thermal conductivity k (λ), concrete unit weight W_c, and concrete heat capacity c

$$\alpha = \frac{k}{W_C \cdot c}\ (\text{ft}^2/\text{s or m}^2/\text{s}) \quad (5.12.4)$$

Due to higher heat capacity, lower density, and lower thermal conductivity of lightweight concrete, the thermal diffusivity is lower than that of normalweight concrete. For example, concrete with a fresh unit weight of 145 lb/ft^3 (2300 kg/m^3) has a thermal diffusivity of $0.71 \cdot 10^{-5}$ ft^2/s ($0.066 \cdot 10^{-5}$ m^2/s), whereas concrete with a fresh unit weight of 85 lb/ft^3 (1400 kg/m^3) has thermal diffusivity of $0.37 \cdot 10^{-5}$ ft^2/s ($0.034 \cdot 10^{-5}$ m^2/s). A local temperature change, therefore, spreads more slowly in lightweight concrete than in normalweight concrete and, conversely, with high cement contents, lightweight concrete tends to reach higher temperatures (Lopez 2005).

5.13—Fire endurance

Lightweight concrete is more fire resistant than ordinary normalweight concrete because of its lower thermal conductivity, lower coefficient of thermal expansion, and fire stability of the aggregate (ACI 216.1; Abrams and Gustaferro 1968; Abrams 1971; Carlson 1962; Selvaggio and Carlson 1964; Expanded Shale, Clay and Slate Institute 1980; Precast/Prestressed Concrete Institute 2009).

Research on fire endurance comparing lightweight-aggregate concrete with normalweight concrete is shown in Fig. 5.13. For fire ratings, the reinforcing steel cover requirements for lightweight concrete may be slightly lower than those for normalweight concrete (Carlson et al. 1962).

While there is more than 50 years of experience and a multitude of fire tests conducted on lightweight concrete of strength levels appropriate for commercial construction—3000 to 5000 psi (21 to 35 MPa)—the availability of data on high-strength lightweight concrete has, until recently, been limited.

Tests by Bilodeau et al. (1995, 1998) have reported that, because of the extremely low permeability generally associated with high-strength concrete, there is significantly reduced resistance to damage due to spalling. Because of the higher moisture contents of concrete containing lightweight aggregate (LWA) with high, as-batched absorbed water contents, there is increased risk of spalling. With the use of high-strength lightweight concrete on offshore platforms, where intense hydrocarbon fires could develop, there was an obvious need to find a remedy for this potentially serious problem.

Several reports have documented the beneficial influence of adding small quantities of polypropylene fibers to high-strength concrete as demonstrated by exposure to fire testing that was more intense than the exposure conditions (time-temperature criteria) specified by ASTM E119. The fibers melt and provide conduits for release of the pressure developed by the conversion of moisture to steam. Jensen et al. (1995) reported the results of tests conducted at the Norwegian Fire Research Laboratories. These studies included the determination of mechanical properties at high temperature, the improvement of spalling resistance through material design, and the verification of fire resistance and residual strength of structural elements exposed to fire. The addition of 0.1 to 0.2 percent polypropylene fibers in the lightweight concrete mixture provided a significant reduction of spalling. Fire tests on beams confirmed previous findings that greater spalling (exposed reinforcement) occurred on reinforced and prestressed lightweight concrete beams than occurred on normalweight concrete beams. Reduced or no spalling, however, occurred on lightweight concrete beams with polypropylene fibers. Also, no spalling was observed on lightweight concrete beams with passive fire protection (a special cement-based mortar with expanded polystyrene balls that did not contain fibers).

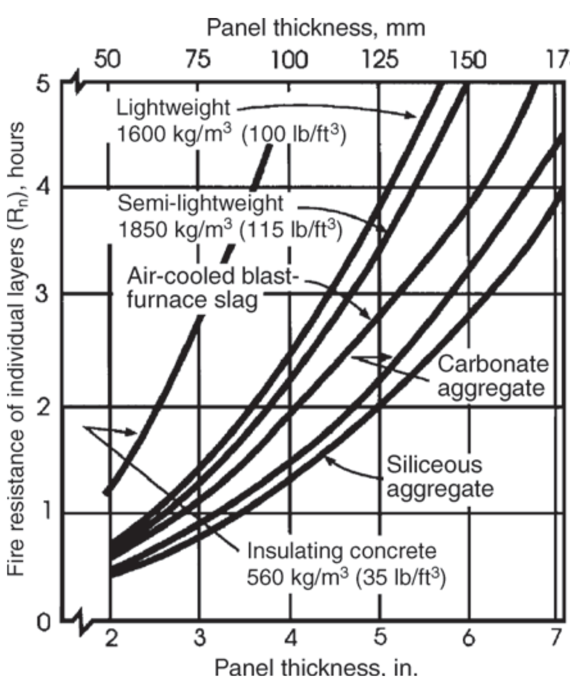


Fig. 5.13—Fire endurance (heat transmission) of concrete slabs as a function of thickness for naturally dried specimens (ACI 216.1).

5.14—Energy absorption and blast resistance

Lightweight concrete has the ability to absorb or damp the energy imparted by projectiles and fragments caused by fire arms and explosive blasts. As the density of the concrete is reduced, its energy absorption capacity is increased. This principle has been demonstrated by the use of all-lightweight-aggregate concrete in the construction of prefabricated telecom equipment buildings and munitions storage facilities.

A unique requirement of the wall panels for telecom equipment buildings is the ability to stop a 30.06 rifle round from a distance of 30 ft (9 m). The bullet must not pass through the panel, or cause excessive damage to the surface. Tests (Speck and Burg 1999) conducted on lightweight concrete with unit weight ranging from 72 to 82 lb/ft^3 (1150 to 1315 kg/m^3) and 28-day compressive strength ranging from 2900 to 4450 psi (20 to 30 MPa) showed excellent performance, indicating that bullets did not pass through the panel. The lightweight concrete with an exposed aggregate surface absorbed the energy of the bullet impact with no spalling, and the bullet hole had a similar appearance to the exposed aggregate, making it difficult to find the point of impact.

Lightweight concrete is now being used to construct munitions storage facilities at naval installations around the world. Lightweight concrete is specified for its ability to absorb fragments created by exploding ordnance and reduce the damage to undetonated ordnance impacting the magazine walls (Speck and Burg 1999). As a result of constructing a prototype magazine using commercially available, expanded clay lightweight aggregate (LWA), and conducting a certification test on the structure, the protected radius (the area where no other buildings can be located) around each magazine can be reduced by 60 to 80 percent, compared with previous requirements. This represents significant cost savings, especially for military bases located in densely populated areas.

CHAPTER 6—DURABILITY OF STRUCTURAL LIGHTWEIGHT-AGGREGATE CONCRETE

6.1—General

Numerous accelerated freezing-and-thawing testing programs have been conducted on lightweight concrete in North America and Europe. These programs researched the influence of entrained-air volume, cement content, aggregate moisture content, specimen drying times, and the testing environment. The results showed similar conclusions in that air-entrained lightweight concrete proportioned with a high-quality binder provides satisfactory durability results when tested under usual laboratory freezing-and-thawing programs. Observations of the resistance to deterioration in the presence of deicing salts on mature bridges indicate similar performance between lightweight and normalweight concrete. Comprehensive investigations into the long-term weathering performance of bridge decks and marine structures exposed for many years to severe environments support the findings of laboratory investigations and suggest that properly proportioned and placed lightweight concrete performs equal to or better than normalweight concrete (Holm 1994).

Core samples taken from hulls of 80-year-old lightweight concrete ships, as well as 40- to 50-year-old lightweight concrete bridges, have shown that concrete having a dense contact zone at the aggregate/matrix interface and has low levels of microcracking throughout the mortar matrix. The explanation for this demonstrated record of high resistance to weathering and corrosion is due to several physical and chemical mechanisms, including superior resistance to microcracking developed by significantly higher aggregate/matrix adhesion and the reduction of internal stresses due to elastic matching of coarse aggregate and matrix phases (Holm et al. 1984). High ultimate strain capacity is also provided by lightweight concrete as it has a high strength/modulus ratio. The strain at which the disruptive dilation of concrete starts is higher for lightweight concrete than for equal-strength normalweight concrete. A well-dispersed pore system provided by the surface of the lightweight fine aggregates may also assist the air-entrainment system and serve an absorption function by reducing concentration levels of deleterious materials in the matrix phase (Holm 1980b).

Internal curing provided by the internally stored water helps enhance hydration and reduces permeability of the contact zone. Although it is widely recognized that the ASTM test method for resistance of concrete to rapid freezing and thawing (ASTM C666/C666M) provides a useful comparative testing procedure, there remains an inadequate correlation between accelerated laboratory test results and the observed behavior of mature concrete exposed to natural freezing and thawing. When freezing-and-thawing tests are conducted, ASTM C330/C330M requires a modification to the procedures of ASTM C666/C666M:

Unless otherwise specified, remove the lightweight concrete specimens from moist curing at an age of 14 days and allow to air-dry for another 14 days exposed to a relative humidity of 50 ± 5 percent and a temperature of 73.5 + 3.5°F (23 ± 2°C). Then submerge the specimens in water for 24 hours before the freezing-and-thawing test.

Durability characteristics of any concrete, both normalweight and lightweight, are primarily determined by the protective qualities of the cement paste matrix. Therefore, it is imperative that the concrete matrix provides low permeability to protect steel reinforcing from corrosion, which is clearly the dominant form of structural deterioration observed in current construction.

The matrix protective quality of nonstructural, insulating concrete proportioned for thermal resistance by using high-air and low-cement contents will be significantly reduced. Very low density, nonstructural concrete will not provide resistance to the intrusion of chlorides and carbonation, comparable to the long-term satisfactory performance demonstrated with high-quality, lightweight concrete.

6.2—Absorption

Lightweight concrete planned for exposed applications will, of necessity, be of high quality. Testing programs by Bremner et al. (1992) have revealed that high-quality lightweight concrete specimens absorbed very little water and, thus, maintained their low density. In addition, it was reported that the permeability of lightweight concrete was extremely low and generally equal to or significantly lower than that reported for the normalweight concrete specimens. The low permeability was attributed to the influence of the high-integrity contact zone possessed by lightweight concrete.

In investigations of high-strength lightweight concrete in the Arctic, Hoff (1992) reported that specimens that had a period of drying followed by water immersion at atmospheric pressure did not refill all the void space caused by drying. Pressurization caused an additional density increase of approximately 2.5 lb/ft^3 (40 kg/m^3). Before the introduction of test specimens into the seawater, all concrete lost mass during the drying phase of curing, although concrete with a compressive strength of 9000 psi (62 MPa) lost little due to its very dense matrix.

6.3—Contact zone/interface

6.3.1 *Influence of contact zone on durability*—The contact zone is the transition layer of material connecting the coarse-aggregate particle with the enveloping continuous mortar matrix. Analysis of this linkage layer requires consideration of more than the adhesion developed at the interface (contact zone) and should include the transitional layer that forms between the two phases. Collapse of the structural integrity of a conglomerate may come from the failure of one of the two phases or from a breakdown in the contact zone causing a separation of the still-intact phases. The various mechanisms that act to maintain continuity or cause separation have not received the same attention as has the air-void system necessary to protect the paste.

Aggregates are often inappropriately dismissed as being inert fillers and, as a result, they and the associated transition zone have until recently received very modest attention.

For concrete to perform satisfactorily in severe exposure conditions, it is essential that a satisfactory bond develops and is maintained between the aggregate and the enveloping continuous mortar matrix. A high incidence of interfacial cracking or aggregate debonding will have a serious effect on durability if these cracks fill with water and subsequently freeze. An equally serious consequence of microcracking is the easy path provided for the ingress of salt water into the mass of the concrete. To provide an insight into the performance of different types of concrete, a number of mature structures that have withstood severe exposure were examined. The morphology and distribution of chemical elements at the interface were studied and reported by Bremner et al. (1984) and Zhang and Gjørv (1990).

The contact zone in lightweight concrete has been demonstrated to be significantly superior to that of normalweight concrete that does not contain silica fume (Holm et al. 1984; Khokrin 1973). This profound improvement in the quality, integrity, and microstructure stems from a number of characteristics unique to lightweight concrete including, but not limited to, the following:

a) The alumina- and silicate-rich surface of the fired ceramic aggregate, which is pozzolanic and combines with $Ca(OH_2)$ liberated by hydration of the portland cement

b) Reduced microcracking at the matrix lightweight aggregate (LWA) interface because of the elastic similarity of the aggregate and the surrounding cementitious matrix

c) Moisture equilibrium between two porous materials, LWA and a porous cementitious matrix, as opposed to the usual condition with normalweight aggregate where bleed-water lenses around coarse natural aggregates have *w/cm* significantly higher than in the bulk of the matrix. When silica fume is added, the high-quality microstructure of the contact zone of concrete containing LWA is moderately enhanced. When used in concrete containing normalweight aggregates, however, this zone of weakness is profoundly improved.

The enhanced hydration afforded by internally stored water is believed to further decrease permeability at the contact zone in lightweight-aggregate concretes (refer to 7.5).

6.3.2 *Contact zone of mature concrete subjected to severe exposure*—Petrographic thin sections of the specimen contact zone were prepared for examination in a scanning electron microscope equipped with an energy-dispersive x-ray analyzer. An example is Fig. 6.3.2, which shows a micrograph from the waterline of a more than 60-year-old concrete ship that was reported by Holm et al. (1984). This micrograph confirms that a tight bond develops between the LWA and the mortar matrix. Normalweight cores taken from bridges with lightweight decks were also examined and revealed separation between the normalweight aggregate and the matrix, but not at the lightweight-aggregate interface.

Russian studies on the durability of lightweight concrete (Khokrin 1973) included results of scanning electron microscopy that revealed improved cement hydration and pozzolonic reaction at the contact zone between the matrix and keramzite, which is the name of rotary kiln-produced expanded clay or shale in Russia. These micrographs confirmed earlier tests in which x-ray diffraction of ground keramzite taken before and after immersion in a saturated lime solution attested to the presence of a chemical reaction.

Khokrin (1973) also reported on microhardness tests of the contact zone of lightweight concrete and normalweight concrete, which established the width of the contact zone as approximately 2.4 in. (60 mm). These results are shown in Table 6.3.2.

Virtually all commercial concrete exhibits some degree of bleeding and segregation. This is primarily due to the difference in density of the various ingredients, and can be minimized with the use of proper mixture proportioning. The influence of bleeding upon the tensile strength of normalweight concrete was studied by Fenwick and Sue (1982). Their report described the effects of the rise of bleed water through the mixture, the entrapment of air pockets below the larger coarse aggregate particles, and the poor paste quality at the interface due to the excessive concentrations of water. Reductions in mechanical properties are inevitable as a result of the interface flaws, as they limit interaction between the two distinctly different phases.

Permeability leads to increased penetration of aggressive agents that accelerate corrosion of embedded reinforcement. The permeability of concrete is a function of the permeability of the matrix, the aggregate, and quality of the interface between them.

The phenomenon of bleed water collecting and being trapped under coarse particles of LWA is considerably diminished, if not essentially eliminated, by the absorption of a small but significant amount of water from the fresh concrete into the interior of the LWA. This has been verified in practice by the examination of the contact zone of lightweight concrete split cylinders and visual examination of sand-blasted vertical surfaces of North American building

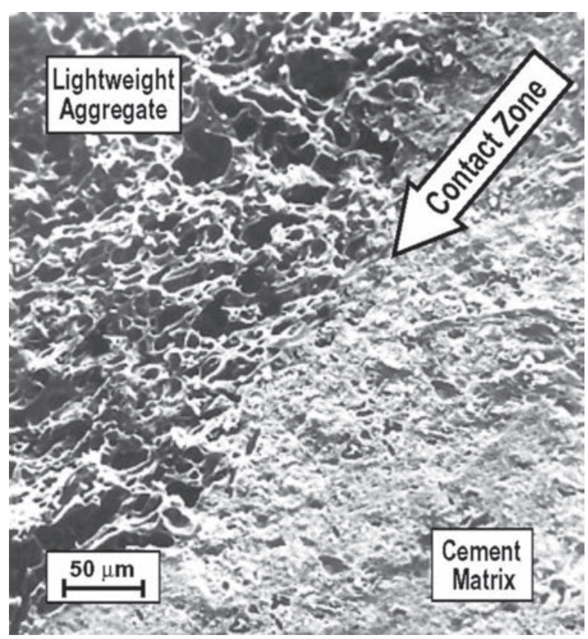


Fig. 6.3.2—Micrograph of contact zone.

Table 6.3.2—Microhardness in and beyond the contact zone *c/z* of concrete with differing *w/cm* and various coarse aggregates (Khokrin 1973)

Coarse aggregate type	*w/cm*					
	0.3		0.4		0.5	
	In *c/z*	Beyond *c/z*	In *c/z*	Beyond *c/z*	In *c/z*	Beyond *c/z*
LWA B	160	92	143	78	136	76
LWA O	167	94	138	73	125	68
Crushed diabase	81	79	—	—	—	—
Crushed limestone	81	81	—	—	—	—

structures. This observation should not be surprising because with lightweight concrete, the aggregate/matrix interface is a boundary between two porous media, whereas with normalweight concrete there is an abrupt transition at the porous/solid phase interface (Lamond and Pielert 2006).

Fagerlund (1972, 1978) has presented reports that analyze the contact zone in mortar and concrete. These reports provide equations that describe the influence of the contact zone on strength parameters. Fagerlund supported the analyses with micrographs that clearly identified various degrees of interaction, from almost complete phase separation for normalweight aggregates to cases involving expanded aggregates in which the boundary between the two phases is not possible to discern. The fact that the high-quality contact zones in lightweight concrete have maintained their integrity throughout their service life of the structures provides reassurance of effective long-term interaction of the components of the concrete composite (Holm 1983). Lopez (2005) and Lopez et al. (2007) studied strains in contact zones of lightweight concrete due to elastic deformation, creep, and shrinkage using strains maps obtained through image analysis. Strain maps revealed that the lightweight concrete contact zone presented lower strains than those of normalweight concrete. The lower strains were attributed to the better strain match between LWA and the matrix and to the enhanced hydration due to the internally stored water.

6.3.3 *Implications of contact zone on failure mechanisms*—Exposed concrete must endure the superposition of a dynamic system of forces, including variable live loads, variable temperatures, moisture gradients, and dilation due to chemical changes. These factors cause a predominantly tensile-related failure. Yet, the uniaxial compressive strength is traditionally considered the preeminent single index of quality despite the fact that concrete almost never fails under compression. The simplicity and ease of compression testing has, perhaps, diverted the focus from a perceptive understanding and development of appropriate measurement techniques that quantify durability characteristics.

In general, weakest-link mechanisms are undetected in uniaxial compression tests due to concrete's forgiving load-sharing characteristics in compression, for example, localized yielding and closing of stress related, temperature, and volume change cracks. Weakest-link mechanisms, however, are important in tensile zones that arise from applied loads and exposure conditions. In many types of concrete, the contact zone may be the weakest link that is decisive in determining the long-term behavior of the concrete.

6.3.4 *Accommodation at aggregate-matrix interface*—A full understanding has not been developed regarding the accommodation mechanism where the pores closest to the LWA aggregate-matrix interface provide an accessible space for products of various reactions to form without causing deleterious expansion. This was confirmed when ettringite, alkali-silica gel, marine salts, and corrosion products were found in these near-surface pores of high performance concretes (Holm and Bremner 2000).

6.4—Resistance to corrosion

6.4.1 *Carbonation in mature marine structures*

6.4.1.1 *General*—Carbonation in concrete is the reaction of carbon dioxide from the air with calcium hydroxide liberated from the hydration process. This reaction reduces pH and removes the conditions that promote depassivating conditions that can neutralize the natural protection of steel reinforcement afforded by the concrete.

The issues associated with carbonation depend on the pH in concrete lowering from approximately 13 to 9, which in turn neutralizes the protective layer over the reinforcing steel, making it vulnerable to corrosion. A combination of two primary mechanisms protects steel from corrosion: an adequate depth of cover with a sufficiently high-quality cover concrete. The quality of the cover material is usually related to *w/cm* or strength but is more closely related to permeability and strain capacity.

6.4.1.2 *Concrete ships, Cape Charles, VA*—Holm et al. (1988) reported the results of carbonation measurements conducted on cores drilled from the hull of several concrete ships built in early 1940s. The ships were used as breakwaters for a ferryboat dock in the Chesapeake Bay at Cape Charles, VA. They were constructed with carefully inspected high-quality concrete made with rotary kiln-produced fine- and coarse-expanded aggregates and a small volume of natural sand. High-cement contents were used to achieve compressive strengths in excess of 5000 psi (35 MPa) at 28 days with a density of 108 lb/ft^3 (1730 kg/m^3) (McLaughlin 1944). Despite freezing and thawing in a marine environment, the hulls and superstructure of this non-air-entrained concrete are in good condition after more than five decades of exposure. The only less-than-satisfactory performance was observed in some areas of the main decks. These areas experienced a delamination of the 3/4 in. (20 mm) concrete cover protecting four layers of large-sized undeformed (typically 1 in. [25 mm] square) reinforcing bars spaced 4 in. (100 mm) on centers. This failure plane is understandable and would have been avoided by the use of modern prestressing procedures. Cover for hull reinforcing was specified at 7/8 in. (22 mm), with all other reinforcement protected by only 1/2 in. (13 mm). Without exception, the reinforcing steel bars cut by the 18 cores taken were rust-free. Cores that included reinforcing steel were split along an axis parallel to the plane of the reinforcing in accordance with the procedures of ASTM C496/C496M. Visual inspection revealed negligible corrosion when the bar was removed. After the interface was sprayed with phenolphthalein, the surfaces stained a vivid red, indicating no carbonation at the steel-concrete interface.

Carbonation depths, averaged 0.04 in. (1 mm) for specimens taken from the main deck, were between 0.04 and 0.08 in. (1 and 2 mm) for concrete in exposed wing walls. The carbonation depth was virtually nonexistent in the hull and bulkheads. The carbonation was revealed by spraying the freshly fractured surface with a standard solution of phenolphthalein. Coring was conducted from the waterline to as much as 16 ft (5 m) above high water. In no instances could carbonation depths greater than 0.08 in. (2 mm) be found. In isolated instances, flexural cracks that had evidence of carbonation up to 0.31 in. (8 mm) in depth were encountered. The carbonation did not appear to progress more than .004 in. (0.1 mm) perpendicular to the plane of the crack.

6.4.1.3 *Chesapeake Bay Bridge, Annapolis, MD*—Concrete cores taken from the 35-year-old Chesapeake Bay Bridge revealed carbonation depths of 0.08 to 0.31 in. (2 to 8 mm) from the top of the bridge deck and 0.08 to 0.51 in. (2 to 13 mm) from the underside of the bridge deck. The higher carbonation depth on the underside reflects increased gas diffusion associated with the drier surface of the bridge. The 1.14 in. (36 mm) asphalt-wearing course appears to have inhibited drying and thus reduced carbonation depth on top (Holm 1983; Holm et al. 1984).

6.4.1.4 *Coxsackie Bridge, New York*—Cores drilled with the cooperation of the New York State Thruway Authority from the 15-year-old exposed deck surface of the Interchange Bridge at Coxsackie revealed 0.20 in. (5 mm) carbonation depths for the top surface and 0.39 in. (10 mm) from the bottom. Despite almost 1000 saltings of the exposed deck, there was no evidence of corrosion in any of the reinforcing bars cut by the six cores taken (Holm et al. 1984).

6.4.1.5 *Bridges and viaducts in Japan*—The results of measurements of carbonation depths on mature marine structures in North America are paralleled by data reported by Ohuchi et al. (1984) in Japan. These investigators studied the chloride penetration, depth of carbonation, and incidence of microcracking in both lightweight and normalweight concrete on the same bridges, aqueducts, and caissons after 19 years of exposure. The high-durability performance of those structures, as measured by the carbonation depths, microcracking, and chloride penetration profiles reported by Ohuchi et al. (1984), is similar to studies conducted on mature lightweight concrete bridges in North America (Holm et al. 1984).

6.4.2 *Permeability and corrosion protection*—Permeability investigations conducted on lightweight and normalweight concrete exposed to the same testing criteria have been reported (Khokrin 1973; Nishi et al. 1980; Keeton 1970; Bamforth 1987; Bremner et al. 1992; Zhang and Gjørv 1991; Thomas 2006). Interestingly, in every case, despite wide variations in concrete strengths, testing media (water, gas, and oil), and testing techniques (specimen size, media pressure, and equipment), lightweight concrete had equal or lower permeability than its normalweight counterpart. Khokrin (1973) further reported that the lower permeability of lightweight concrete was attributed to the elastic compatibility of the constituents and the enhanced bond between the coarse aggregate and the matrix. In the Onoda Cement Company tests (Nishi et al. 1980), concrete with a *w/cm* of 0.55, moist-cured for 28 days when tested at 128 psi (0.88 MPa) water pressure had a depth of penetration of 1.38 in. (35 mm) for normalweight concrete and 0.95 in. (24 mm) for lightweight concrete. When tested with seawater, penetration was 0.59 and 0.47 in. (15 and 12 mm) for normalweight concrete and lightweight concrete, respectively. The author suggested that the reason for this behavior was, "a layer of dense hardened cement paste surrounding the particles of artificial lightweight coarse aggregate." The U.S. Navy-sponsored work by Keeton (1970) reported the lowest permeability with high-strength lightweight concrete. Bamforth (1987) incorporated lightweight concrete as one of the four concretes tested for permeability to nitrogen gas at 145 psi (1 MPa) pressure level. The normalweight concrete specimens included high-strength 13,000 psi (90 MPa) concrete and concrete with a 25 percent fly ash replacement, by mass or volume. The sanded lightweight concrete (7250 psi [50 MPa]), 6.4 percent air with a density of 124 lb/ft^3 (1985 kg/m^3), demonstrated the lowest water and air permeability of all mixtures tested.

Fully hydrated portland cement paste of low *w/cm* has the potential to form a matrix that should render concrete impermeable to the flow of liquids and gases. However, in reality, microcracks form in concrete during the hardening process, and later due to shrinkage, thermal, and applied stresses. In addition, excess water added to concrete for easier placing will evaporate, leaving pores and conduits in the concrete. This is particularly true in exposed concrete decks where concrete has frequently provided inadequate protection for steel reinforcement.

Mehta (1986) observed that the permeability of a concrete composite is significantly greater than the permeability of either the continuous matrix system or the suspended coarse-aggregate fraction. This difference is primarily related to extensive microcracking caused by mismatched concrete components responding differentially to temperature gradients, service load strains, and volume changes associated with chemical reactions taking place within the concrete. In addition, channels develop in the transition zone surrounding normalweight coarse aggregates, giving rise to unimpeded moisture movements. While separations caused by the evaporation of bleed water adjacent to ordinary aggregates are frequently visible to the naked eye, such defects are essentially unknown in lightweight concrete. In lightweight concrete, the contact zone is the interface between two porous media: the LWA particle and the hydrating cement binder. The continuous, high-quality matrix fraction surrounding

LWA is the result of several beneficial processes. Khokrin (1973) reported on several investigations that documented the increased transition zone microhardness due to pozzolanic reaction developed at the surface of the LWA. Bremner et al. (1984) conducted measurements of the diffusion of the silica out of the coarse lightweight-aggregate particles into the cement paste matrix using energy-dispersive x-ray analytical techniques. The results correlated with Khokrin's observations that the superior contact zone in lightweight concrete extended approximately 2.4 in. (60 mm) from the LWA particles into the continuous matrix phase. Zhang and Gjørv (1990) examined the contact zone of lightweight concrete using scanning electron microscopy. They found that LWA with a porous external layer produced a denser and more homogeneous contact zone.

One laboratory report comparing normalweight concrete and lightweight concrete indicated that, in the unstressed state, the permeabilities were similar. At higher levels of stress, however, the lightweight concrete could be loaded to a higher percentage of its ultimate compressive strength before microcracking causes a sharp increase in permeability (Sugiyama et al. 1996). In laboratory testing programs, when the concrete is maintained at constant temperature, there are no significant shrinkage restraints, and field-imposed stresses are absent. Because of the as-batched moisture content of the LWA before mixing, this absorbed water provides for extended moist curing. The water tends to wick out from the coarse aggregate pores into the finer capillary pores in the cement paste, thereby extending moist curing. Because the potential pozzolanic surface reaction is developed over a long time, usual laboratory testing that is completed in less than a few months may not adequately take this into account.

6.5—Alkali-aggregate reaction

ACI 201.1R reports no documented instance of in-service distress caused by alkali reactions with lightweight aggregate (LWA). Mielenz (1994) indicates that although the potential exists for alkali-aggregate reaction with some natural LWA, the volume change may be accommodated without necessarily causing structural distress. The surface of fine aggregate fractions of expanded shales, clays, and slates are known to be pozzolanic and may also serve to inhibit disruptive expansion (Boyd et al. 2006; Holm and Bremner 2000). No evidence of alkali-lightweight-aggregate reactions were observed in tests conducted on 70-year-old marine structures and several more than 30-year-old lightweight concrete bridge decks (Holm 1994).

Though laboratory studies and field experience have indicated no deleterious expansion resulting from the reaction between cement and silica in the lightweight component of the aggregates, the natural aggregate portion of a sand-lightweight concrete mixture should be evaluated in accordance with applicable ASTM standards.

Many lightweight concrete mixtures designed for an equilibrium density in the range of 110 lb/ft^3 (1760 kg/m^3) and above are produced using either natural sand or a naturally occurring coarse aggregate. In either case, these natural aggregates should be considered a potential source to develop alkali-aggregate reactions until they have been demonstrated by an appropriate ASTM test procedure or by having an established service history to be of negligible effect.

6.6—Abrasion resistance

Abrasion resistance of concrete depends on strength, hardness, and toughness characteristics of the cement paste and the aggregates, and the bond between these two phases. Most lightweight aggregate (LWA) suitable for structural concrete are composed of solidified vitreous material comparable to quartz on the Mohs Scale of Hardness. Due to its pore system, however, the net resistance to wearing forces may be less than that of a solid particle of most natural aggregates. Lightweight concrete bridge decks that have been subjected to more than 100 million vehicle crossings, including truck traffic, show wearing performance similar to that of normalweight concrete (Holm and Bremner 2000). Limitations are necessary in certain commercial applications where steel-wheeled industrial vehicles are used, but such surfaces generally receive specially prepared surface treatments. Hoff (1992) reported that specially developed testing procedures that measured ice abrasion of concrete exposed to arctic conditions demonstrated essentially similar performance for lightweight and normalweight concrete.

CHAPTER 7—DESIGN OF STRUCTURAL LIGHTWEIGHT-AGGREGATE CONCRETE

7.1—Scope

The availability and proven performance of lightweight aggregate (LWA) has led to the improved functionality and economical design of buildings, bridges, and marine structures for more than 80 years. As engineers began using lightweight concrete, designs were based on the usual properties of concrete, adjusted by the engineers with little guidance from recommended practices specifically pertaining to lightweight concrete. With the adoption of the 1963 edition of ACI 318, lightweight-aggregate concrete received full recognition as an acceptable structural material. General guidelines for the architect/engineer and the construction industry were included.

This chapter assists in the interpretation of the ACI 318 requirements for lightweight concrete. It also condenses many practical design aspects pertaining to lightweight concrete and provides the architect/engineer with additional information for design.

The architect/engineer should obtain information on the properties of concrete made with specific LWA available for a given project. These aggregates should follow the recommendations of this guide, and the specifications should be prepared so that only LWAs suitable for the intended use are included.

7.2—General considerations

Lightweight concrete has been shown by test and performance (Chapter 5) to behave structurally in much the same manner as normalweight concrete, but at the same time, to provide some specific concrete properties—notably reduced weight, improved insulating properties, reduced stiffness (and therefore reduced cracking), and enhanced microstructure. For certain concrete properties, the differences in performance are minor. Generally those properties that are a function of tensile strength, for example shear, development length, and modulus of elasticity, are sufficiently different from those of normalweight concrete to require design modification.

7.3—Modulus of elasticity

If the value of E_c will have a significant effect on the design, the engineer should decide whether the values determined using formulas derived from experimental data (ACI 318) are sufficiently accurate or whether a more accurate value for E_c should be determined from tests on the specified concrete.

A lower E_c value for lightweight concrete means that it is more flexible. Where reduced concrete stiffness can be beneficial, the use of lightweight concrete should be considered instead of normalweight concrete. In cases where improved impact or dynamic response is required, where differential foundation settlement may occur, where tensile stresses are caused by restrained thermal and shrinkage deformations (such as bridge decks) (Shah et al. 1998), and in certain types or configurations of shell roofs, the reduced stiffness of lightweight concrete may be desirable. In other applications, however, lightweight concrete may lead to higher deformations than those of normalweight concrete of similar strength under the same live loads, as well as increased camber in prestressed concrete beams.

7.4—Tensile strength

Shear, torsion, anchorage, bond strength, development length, and crack resistance are related to tensile strength that is, in turn, dependent on the tensile strength of the coarse aggregate and mortar phases, and the quality of the bond between the two phases. Traditionally, tensile strength has been defined as a function of compressive strength. This, however, is understood to be only a first approximation that does not reflect aggregate particle strength, surface characteristics, or the moisture content and its distribution within the concrete. The tensile-splitting strength, as determined by ASTM C496/C496M, is used throughout North America as a simple, practical, and more reliable indicator of tensile properties of concrete than the beam flexural test. Reflecting the importance of the tensile strength, ASTM C330/C330M requires a lightweight aggregate for use in structural concrete to be able to produce concrete with a minimum tensile splitting strength of 290 psi (2.0 MPa).

Tests have shown that diagonal tensile strengths of beams and slabs correlate closely with the aggregate suppliers for laboratory-developed splitting tensile strength data. Tensile strength test data may need to be obtained before the start of projects where development of early-age tensile-related handling forces occurs such as in precast or tilt-up members.

7.5—Shear and diagonal tension

From a shear and diagonal tension perspective, lightweight concrete members behave in fundamentally the same manner as normalweight concrete members. Since the tensile capacity of lightweight concrete is less than for normalweight concrete, however, the concrete contribution of lightweight concrete members is also less. This is acknowledged in ACI 318 by multiplying the $\sqrt{f_c'}$ term by a modification factor λ. ACI 318 provides two approaches for determining the λ modification factor. The λ factor may be determined by using the splitting tensile strength f_{ct} for the specific aggregate to be used, or by using a fixed reduction factor based on the type of lightweight concrete.

If the first approach is used, λ is taken as the ratio $f_{ct}/6.7\sqrt{f_c'}$. A realistic value of f_{ct} for design purposes should be established for each desired compressive strength and composition of concrete. The f_{ct} values on which the structural design is based should be incorporated in the concrete specifications for the job. Splitting cylinder strength tests, if required, should be performed on laboratory mixtures that use the same materials that will be used for the project. These tests should be performed in accordance with ASTM C496/C496M. Splitting tensile strength is a laboratory evaluation test and should not be conducted in the field for acceptance purposes (refer to ACI 318-11, 5.1.5).

A second, generally conservative, approach in calculating the modification factor for shear may be used when the engineer does not specify f_{ct} values. Two modification factors have been established: 75 percent of normalweight values for concrete containing both fine and coarse LWAs; and 85 percent of normalweight values for combinations of natural sand fine aggregates and lightweight coarse aggregates.

Most of the research addressing tensile strength, shear strength, and development lengths of structural lightweight concrete that formed the basis for existing ACI 318 requirements are limited to concrete with a compressive strength of less than 6000 psi (41 MPa). When concrete strengths greater than 6000 psi (41 MPa) are specified, the determination of the appropriate tension, shear, and development length parameters should be based on a comprehensive testing program that is conducted on the materials selected for the project. For some LWAs, the tensile strength ceiling may be reached earlier than the compressive strength ceiling.

A comprehensive investigation into the shear strength of higher strength (41 to 69 MPa [6 to 10 ksi]) reinforced and prestressed lightweight concrete beams has been reported by Ramirez et al. (1999). Measurements during the beam tests and observations of the structural behavior enabled the evaluation of the 1995 version of ACI 318 and AASHTO (1994, 1995) shear design methods for the types of beams tested.

Ramirez et al. (1999) reported the following for the reinforced concrete specimens.

a) Despite the fact that the sand-lightweight concrete beams had higher measured shear capacities than those calculated using code/specification methods considered in their report, the lightweight concrete beams were, on average, 82 percent of the measured shear capacity of the companion normalweight beams. The 0.85 reduction factor used by the current specifications does not adequately account for the reduction of shear strength in sand-lightweight concrete beams. The trend observed is important especially for the case of beams with low to minimum amounts of shear reinforcement where the concrete contribution is a larger fraction of the total shear strength.

b) While all reinforced (nonprestressed) concrete beams had measured shear capacities that exceeded both the ACI 318-95 and AASHTO (1995) (simple method) and the AASHTO (1994) (general method), the degree of conservatism was greater for the normalweight concrete than the lightweight concrete beams.

c) The degree of conservatism in the calculated capacities decreases for the lightweight concrete beams tested.

d) For the beams tested, the ACI 318-95 and AASHTO (1995) (simple) method produced estimates of shear strength 6 percent more conservative than did the AASHTO (1994) (general method).

e) For the prestressed lightweight concrete beams tested, Ramirez et al. (1999) found the following Measured shear capacities of the prestressed concrete beams using normal strength (41 MPa [6 ksi]) and high strength (69 MPa [10 ksi]) lightweight concrete were nearly equal. Therefore, the minimum amount of transverse reinforcement required by the AASHTO LRFD (1994) did not provide the same level of conservatism for the higher strength lightweight concrete beams.

f) For the high strength prestressed lightweight concrete beams tested, both the AASHTO LRFD (1994) general method and the ACI 318-95 and AASHTO LRFD (1994) simplified method provide conservative estimates of the shear strength. For the high strength prestressed lightweight concrete beams tested both the AASHTO LRFD (1994) (general method) and the ACI 318-95 and AASHTO LRFD (1994) (simple method) provide conservative estimates of the shear strength.

g) For the prestressed high-strength lightweight concrete beams tested, the degree of conservatism afforded by the AASHTO LRFD (1994) general method was nearly equal to that provided by the ACI 318-95 and AASHTO LRFD (1994) simplified method. For the high-strength prestressed lightweight concrete beams tested, the degree of conservatism afforded by the AASHTO (simple) method was nearly equal.

Based on the results of this comprehensive testing program, Ramirez et al. (1999) recommended more research in the area of high-strength prestressed lightweight concrete beams, especially regarding the minimum area of transverse reinforcement requirements. Because a reduction in self-weight leads to a substantial reduction in total load on lightweight concrete members, the concrete contribution to the shear capacity, which may be reduced to as little as 75 percent of normalweight concrete, may not necessarily lead to a decrease in relative structural efficiency. It is usually easy to compensate for any reduced concrete shear capacity by appropriate design of the shear reinforcement.

7.6—Development length

7.6.1 *Passive reinforcement*—Because of the lower tensile and particle strength, lightweight concrete has lower bond-splitting capacities and a lower post-elastic strain capacity than normalweight concrete. ACI 318 requires longer embedment lengths for deformed 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 divided by a factor of 0.75, resulting in a 33 percent increase in the development lengths required for normalweight concrete.

7.6.2 *Active reinforcement*—Meyer and Kahn (2004) report that:

a) An evaluation of code provisions using the results of 12 tests on high-strength prestressed lightweight concrete girders showed the transfer and development length requirements of AASHTO LRFD (1994) and ACI 318-95 to be conservative.

b) Test results showed that shear cracking in the transfer length region across the bottom strands did not induce strand slip if stirrup density was doubled over the current AASHTO LFRD (1994) specified density in that region.

Thatcher et al. (2002) reported that while the ACI 318-95 and AASHTO LRFD (1994) codes provide a conservative estimate of the transfer length of normalweight concrete, their test results showed that transfer length of lightweight concrete was underestimated. Thatcher et al. (2002) suggested that the modulus of elasticity was a consistent factor in determining the transfer length for both normal and lightweight concretes and that most models do not accurately predict the behavior of lightweight concrete. Tests (Thatcher et al. 2002) indicate that the ACI 318-95 and AASHTO LRFD (1994) codes provide a conservative estimate of the development length for both normalweight and lightweight concretes.

Nassar's (2002) conclusions differ, however. Based on the results of tests on large-span high-performance prestressed lightweight concrete beams:

a) Until additional data emerges for transfer length in high-strength lightweight concrete beams, code guidance should be raised to $60d_b$ per AASHTO LRFD (1994) stipulation, $f_{si}d_b/3$, or both, to maintain a more conservative representation.

b) The development length results were inconclusive and the ACI 318-95 and AASHTO LRFD (1994) code requirements may be marginally acceptable for high-strength prestressed lightweight concrete. Until additional testing is conducted, it is recommended that the equation for the development length be modified by a factor of 1/0.85, resulting in an 18 percent increase in code requirements.

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 preproject testing program may be advisable for special structures, larger-diameter strands, short-span decks, or combinations of highly reinforced thin members using high-strength, lightweight concrete. Additional research on development-length requirements and the need for greater amounts of confining reinforcement for prestressing strands in high-strength lightweight concrete and specified-density concrete is clearly warranted.

7.7—Deflection

7.7.1 *Initial deflection*—ACI 318 specifically includes modifications of formulas and minimum thickness requirements that address the lower modulus of elasticity, lower tensile strength, and lower modulus of rupture of lightweight concrete.

ACI 318 also lists the minimum thickness of beams for one-way slabs unless deflections are computed and requires a minimum increase of 9 percent in thickness for lightweight members over normalweight. Thus, using the values suggested in the code, lightweight structural members with increased thickness are not expected to deflect more than normalweight members under the same superimposed load.

7.7.2 *Long-term deflection*—Analytical studies of long-term deflections can be made, taking into account the effects that occur from creep and shrinkage. Final deflection can then be compared with the initial deflection due to elastic strains only. Comparative shrinkage values for concrete vary appreciably with variations in component materials. In moderate strength cases, the shrinkage and creep of lightweight concrete may be somewhat greater than normalweight concrete of the same strength. As concrete strength increases, differences in creep and shrinkage between normalweight and lightweight concrete decreases. In one series of tests on high strength lightweight concrete, the internally stored water held in the lightweight aggregate (LWA) has shown to slowdown and reduce both creep and shrinkage. An analysis of deflection due to elastic strain, creep, and shrinkage leads to the same factor given in ACI 318, and this factor for obtaining long-term deflections should be used for both types of concrete. More refined approaches to estimating deflections are usually not warranted.

7.8—Columns

The design of columns using lightweight concrete is essentially the same as for normalweight concrete. The reduced modulus should be used in the code sections in which slenderness effects are considered.

Extensive tests (Pfeifer 1968; Washa and Fluck 1952) comparing the time-dependent behavior of lightweight and normalweight columns developed the following:

a) Instantaneous shortening caused by initial loading can be accurately predicted by elastic theory. Such shortening of a lightweight concrete column will be greater than that of a comparable normalweight column due to the lower modulus of elasticity of lightweight concrete.

b) Time-dependent shortening of lightweight and normalweight concrete may differ when small unreinforced specimens are compared. These differences, however, are minimized when large reinforced concrete columns are tested as both increasing size and amount of longitudinal reinforcement reduces time-dependent shortening. Measured time-dependent shortening was compared with those predicted by theory, and satisfactory correlations were found.

c) Measured ultimate strengths were compared with theory and good correlations were found. Both concrete type and previous loading had no effect on this correlation.

7.9—Prestressed lightweight concrete

7.9.1 *Applications*—Prestressed lightweight concrete has been widely used for more than 40 years in North America, in nearly every application for which prestressed normalweight concrete has been used. The most beneficial applications are those in which the unique properties of prestressed lightweight concrete are fully used.

Prestressed lightweight concrete has been used extensively in roofs, walls, and floors of buildings and has found extensive use in flat plate and beam types of construction. Reduced dead weight, lower structural, seismic, and foundation loads; better thermal insulation; significantly better fire resistance; and lower transportation cost have usually been the determining factors in the selection of prestressed lightweight concrete for these applications.

7.9.2 *Properties*—When lightweight concrete is used with prestressing, it should possess two important properties:

1. The aggregates should be of high quality
2. The concrete mixture must have high strength

Following is a summary of the properties of prestressed lightweight concrete.

7.9.2.1 *Equilibrium density*—The range is typically between 100 to 120 lb/ft^3 (1600 to 1920 kg/m^3). Several bridges have incorporated a specified equilibrium density of approximately 130 lb/ft^3 (2080 kg/m^3) (Holm and Ries 2000).

7.9.2.2 *Compressive strength*—Typically, higher-strength concrete is used with prestressing. In general, the commercial range of strength is between 5000 to 6000 psi (35 to 41 MPa), although design compressive strengths up to 10,000 psi (70 MPa) have been used.

7.9.2.3 *Modulus of elasticity*—An estimate of the modulus of elasticity of lightweight concrete for high-strength prestressed applications can be obtained by using the formula given in 5.5. In general, the ACI 318 equation for E_c (refer to 8.5.1 of ACI 318-08) tends to overestimate values for high-strength normalweight concrete, and the disparity is even greater for high-strength lightweight concrete. When accurate values of E_c are required, it is suggested that a laboratory test be conducted using the concrete mixture to be used on the project.

7.9.2.4 *Combined loss of prestress*—The Precast/Prestressed Concrete Institute (2009) provides guidance for estimating the prestress loss due to elastic shortening, creep, shrinkage, and other factors. Estimates for creep strains for lightweight concrete are shown to be greater than for normalweight concrete. No distinction is made between lightweight and normalweight concrete for estimated shrinkage after both moist and accelerated curing. The Precast/Prestressed Concrete Institute (2009) recommends that total loss of prestress in typical members will range from approximately 25,000 to 50,000 psi (170 to 340 MPa) for normalweight concrete and from approximately 30,000 to 55,000 psi (210 to 380 MPa) for members using lightweight coarse aggregate and natural sand.

The tests reported by Kahn and Lopez (2005) demonstrated that total prestressed losses on 8000 and 10,000 psi (55 and 69 MPa) compressive strength lightweight concrete AASHTO LRFD (1994) girders were lower than those predicted using PCI (2009), AASHTO LRFD (1994), and ACI 318-95 methods. That is, codes (PCI 2009; AASHTO LRFD 1994; ACI 318-95) were conservative in estimating total prestress losses.

7.9.2.5 *Thermal insulation*—The thermal insulation of lightweight concrete has a significant effect on prestressing applications due to the following:

a) Greater temperature differential in service between the side exposed to sun and the inside may cause greater camber. The top member of a stack of precast products should be covered during the initial drying stage.

b) Better response to steam curing

c) Greater suitability for winter concreting

d) Better fire resistance

7.9.2.6 *Dynamic, shock, vibration, and seismic resistance*—Prestressed lightweight concrete appears at least as good as normalweight concrete and may be even better due to its lower modulus of elasticity.

7.9.2.7 *Cover requirements*—Where fire requirements dictate the cover requirements, the insulating effects developed by the lower density and the fire stability offered by an aggregate preheated to 2192°F (1200°C) may be used advantageously.

7.10—Thermal design considerations

In concrete elements exposed to the environment, the choice of lightweight concrete will provide several distinct advantages over normalweight concrete (Fintel and Khan 1965, 1966, 1968). Physical properties covered in detail in Chapter 5 are as follows:

a) The lower thermal diffusivity provides a thermal inertia that lengthens the time for exposed members to reach any steady-state temperature.

b) Due to this resistance, the effective interior temperature change will be smaller under transient temperature conditions. This time lag will moderate the solar build-up and nightly cooling effects.

c) The lower coefficient of linear thermal expansion that is developed in the concrete due to the lower coefficient of thermal expansion of the lightweight aggregate (LWA) itself is a fundamental design consideration in exposed members.

d) The lower modulus of elasticity will develop lower stress changes in members exposed to thermal strains.

A comparative thermal investigation studying the shortening developed by the average temperature of an exposed column restrained by the interior frame demonstrated the fact that the axial shortening effects were approximately 30 percent smaller for lightweight concrete, and the stresses due to restrained bowing were approximately 35 percent lower with lightweight concrete than with normalweight concrete (Fintel and Kahn 1965, 1966, 1968).

For an exact structural analysis, use data on local aggregates obtained from previous studies.

7.11—Seismic design

Lightweight concrete is particularly adaptable to seismic design and construction because of the significant reduction in inertial forces. A large number of multistory buildings and bridge structures have effectively used lightweight concrete in areas subject to earthquakes.

The lateral or horizontal forces acting on a structure during earthquake motions are directly proportional to the inertia or mass of that structure. These lateral forces may be calculated by recognized formulas and are applied with the other load factors.

7.11.1 *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 318 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 reinforced lightweight concrete members with strength greater than 6000 psi (42 MPa). 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 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 postelastic strain behavior of lightweight concrete. This report also included 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 and normalweight concrete columns. Their 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. ACI 318 places a compressive strength limit of 5000 psi (35 MPa) for concrete members unless supported by test results for higher strengths.

7.12—Fatigue

The first recorded North American comparison of the fatigue behavior between lightweight and normalweight was reported by Gray et al. (1961). These investigators concluded that the fatigue properties of lightweight concrete are not significantly different from the fatigue properties of normalweight concrete.

This work was followed by Ramakrishnan et al. (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 lightweight concrete for buoyancy considerations, a considerable amount of research has been completed determining the fatigue resistance of high-strength lightweight concrete and comparing 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." The long-term service performance of real structures, however, is what 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 (Fig. 7.12a and 7.12b) was evaluated in an in-depth investigation conducted in 1992. 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 (FDOT) 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 Fig. 7.12c (Brown et al. 1995). Maximum deflection for one particular beam due to a midpoint load was 0.28 in. (7.1 mm) measured at 60.5 ft (18.4 m) from the unrestrained end of the span. This compares 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 lower in magnitude than the static loads.

Strain measurements across the bridge profile were also duplicated and compared closely for most locations in areas of significant strain. Comparison of the 1992 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 31-year-old long-span lightweight-concrete bridge corroborates the conclusions of fatigue investigations conducted on small specimens tested under controlled conditions in several laboratories (Hoff 1994; Gjerde 1982; Gray et al. 1961). In these investigations, it was generally observed that the lightweight concrete performed as well as and, in most cases, somewhat better than the normalweight control specimens. Several investigators have suggested that the improved performance was due to the elastic compatibility of the lightweight aggregate (LWA) particles to that of the surrounding cementitious matrix. In lightweight concrete, the elastic modulus of the constituent phases (coarse aggregate and the enveloping mortar phase) are relatively similar, while with normalweight concrete, the elastic modulus of most normalweight concrete may be as much as three to five times greater than its enveloping matrix (Bremner and Holm 1986). With lightweight concrete, the elastic similarity of the two phases of a composite system results in a profound reduction of stress concentrations and a leveling out of the

Fig. 7.12a—Barge-mounted frame-placed beams. To the right is the old truss bridge. Both carry U.S. 19 traffic (Brown et al. 1995).

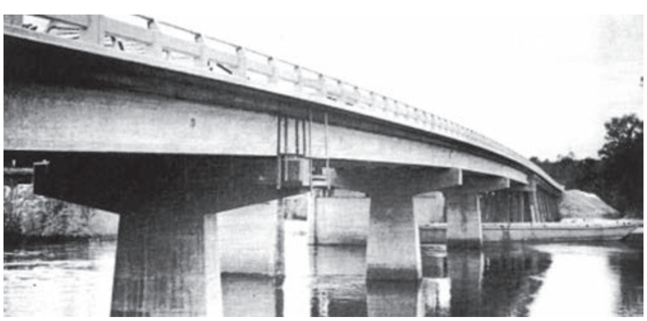


Fig. 7.12b—Concrete weighing less than 120 lb/ft^3 (1920 kg/m^3) permitted 120 ft (37 m) spans for Florida bridge (Brown et al. 1995).

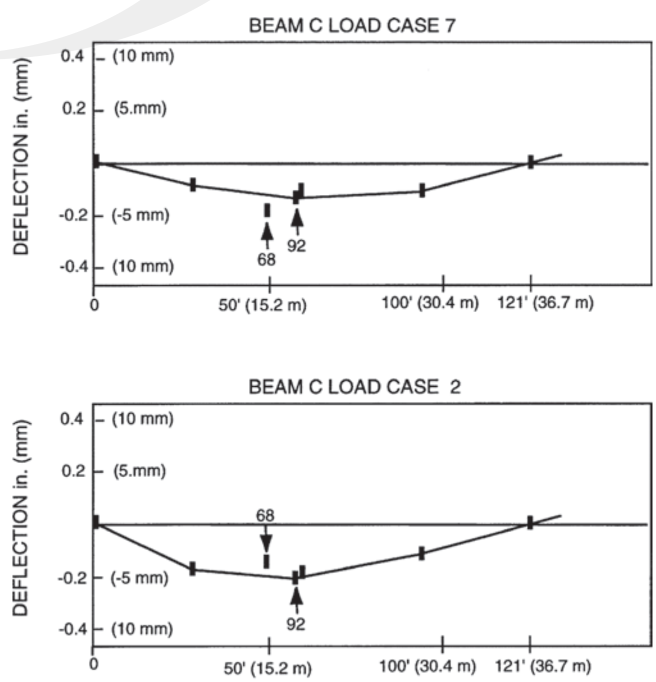


Fig. 7.12c—Florida Department of Transportation predicted deflections compared with 1968 and 1992 measurements (Brown et al. 1995).

average stress over the cross section of the loaded member. Normalweight concrete having a significant elastic mismatch will inevitably develop stress concentrations that may result in extensive microcracking in the concrete composite. This has been further demonstrated by strain measurements at the contact zone (Lopez et al. 2007; Lopez 2005)

Additionally, because of the pozzolanic reactivity of the surface of the vesicular aggregate that has been fired at temperatures above 2012°F (1100°C) (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 et al. 1984).

CHAPTER 8—PERFORMANCE AND APPLICATIONS OF LIGHTWEIGHT-AGGREGATE CONCRETE

8.1—Scope and historical development

While it is clearly understood that high strength and high performance are not synonymous, one may consider the first modern use of high-performance concrete to be when the American Emergency Fleet Corporation built lightweight concrete ships with specified compressive strengths of 5000 psi (35 MPa) during 1917 to 1920. Commercial normalweight concrete strengths of that time were approximately 2500 psi (17 MPa) (Holm 1980b).

Lightweight concrete has achieved high-strength levels by incorporating various pozzolans such as is fly ash, silica fume, metakaolin, calcined clays, and shales, combined with midrange water-reducing admixtures, high-range water-reducing admixtures, or both. Because of durability concerns, the *w*/*cm* has, in many cases, been specified for bridges and marine structures to be less than 0.45, and even significantly lower *w*/*cm* for severe environments. Limiting water content and designing to an air content of 4 to 5 percent may result in an equilibrium density higher than 120 lb/ft^3 (1920 kg/m^3).

While structural-grade lightweight aggregate (LWA) is capable of producing concrete with compressive strengths in excess of 5000 psi (35 MPa), several LWAs have been used in concrete that developed compressive strengths from 7000 to 10,000 psi (48 to more than 69 MPa). In general, an increase in density will be necessary when developing higher compressive strengths. High-strength lightweight concrete with compressive strengths of 6000 psi (41 MPa) is widely available commercially and testing programs on lightweight concrete with a compressive strength approaching 10,000 psi (69 MPa) are ongoing. High-strength lightweight concrete of 10,000 psi (69 MPa) is being used for precast prestressed concrete girders in some states.

8.2—Structural efficiency of lightweight concrete

The entire hull structure of the USS Selma and 18 other concrete ships were constructed with 5000 psi (35 MPa), high-performance lightweight concrete in the ship building program in Mobile, Alabama starting in 1917 (Holm 1980b). The structural efficiency as defined by the strength/density (*S*/*D*) ratio of the concrete used in the USS Selma was extraordinary for that time. Improvements in structural efficiency of concrete since that time are shown schematically in Fig. 8.2; an upward trend in the 1950s with the introduction of prestressed concrete, followed by production of high-strength normalweight concrete for columns of very tall cast-in-place concrete-frame commercial buildings. Most increases came as a result of improvements in the cementitious matrix brought about by new generations of admixtures such as high-range water reducers, and the incorporation of high-quality pozzolans such as silica fume, metakaolin, and fly ash. History suggests, however, that the first major breakthrough came as a result of the lightweight concrete shipbuilding program in 1917.

8.3—Applications of high-performance lightweight concrete

8.3.1 *Precast structures*—High-strength lightweight concrete with a compressive strength in excess of 5000 psi (35 MPa) has been successfully used for almost five decades in North American. Presently, there are ongoing investigations into longer-span lightweight precast concrete members that may be feasible from a trucking/lifting/logistical point of view (Holm and Bremner 2000).

The Wabash River Bridge, constructed in 1994 with 96 lightweight prestressed post-tensioned bulb-tee girders, is a good example where a 17 percent density reduction was realized (Expanded Shale, Clay and Slate Institute 2001). The 96 lightweight girders were each 175 ft (53.4 m) long, 7.5 ft (2.3 m) deep, and weighed 96 tons (87.3 metric tons). The 5-day strengths exceeded 7000 psi (48 MPa). High-performance concrete was used because it saved the owner $1.7 million, or 18 percent of the total project cost.

Parking structure members with 50 to 60 ft (15 to 18 m) spans are often constructed with double tees with an equi-

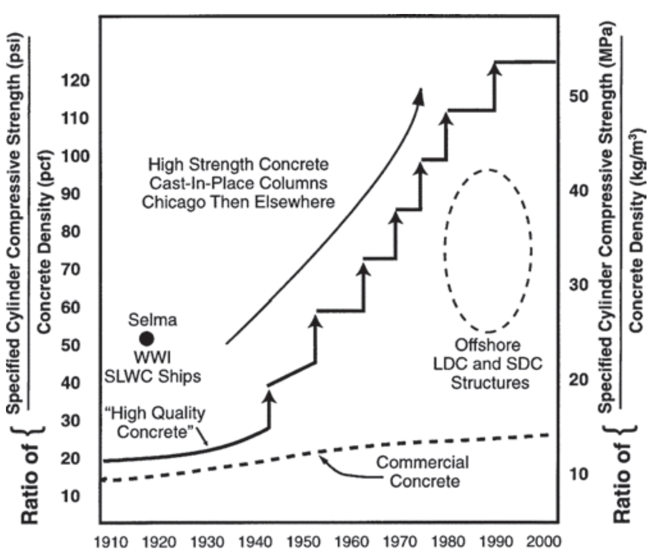


Fig. 8.2—Structural efficiency of concrete; ratio of specified compressive strength to density (psi/[lb/ft^3]) (Holm and Bremner 1994).

librium density of approximately 115 lb/ft^3 (1850 kg/m^3). This mass reduction is primarily for lifting efficiencies and lowering transportation costs.

8.3.2 *Buildings*—Among the thousands of buildings built in North America incorporating high-strength lightweight concrete, the following examples have been selected for their pioneering and unique characteristics.

8.3.2.1 *Federal Post Office and Office Building, New York, 1967*—The 450 ft (140 m) multipurpose building constructed in 1967 with five post office floors and 27 office tower floors, was the first major New York City building application of post-tensioned floor slabs. Concrete tensioning strengths of 3500 psi (24 MPa) were routinely achieved for 3 days for the 30 x 30 ft (9 x 9 m) floor slabs with a design target strength of 6000 psi (41 MPa) at 28 days. Approximately 30,000 yd^3 (23,000 m^3) of lightweight concrete were incorporated into the floors and the cast-in-place architectural envelope, which serves a structural as well as an aesthetic function (Holm and Bremner 1994).

8.3.2.2 *The North Pier Apartment Tower, Chicago, 1991*—This project used high-performance lightweight concrete in the floor slabs as an innovative structural solution to avoid construction problems associated with the load transfer from high-strength normalweight concrete columns through the floor slab system. ACI 318 requires a maximum ratio of column compressive strength, which in this project was 9000 psi (62 MPa) with the intervening floor slab concrete to be less than 1.4. By using high-strength lightweight concrete in the slabs with a strength greater than 6430 psi (44 MPa), the floor slabs could be placed using routine placement techniques, thus avoiding scheduling problems associated with the mushroom technique (Fig. 8.3.2.2). In the mushroom technique, the high-strength column concrete is overflowed from the column and intermingled with the floor slab concrete. The simple technique of using high-strength floor slab concrete in the North Pier project avoided delicate timing considerations that were necessary to avoid cold joints (Holm and Bremner 1994).

8.3.2.3 *The Bank of America, Charlotte, 1992*—This concrete structure is the tallest in the southeastern United States, with a high-strength concrete floor system consisting of 4-5/8 in. (117 mm) thick slabs supported on 18 in. (460 mm) deep post-tensioned concrete beams centered on 10 ft (3.0 m). The lightweight concrete floor system was selected to minimize the dead weight and to achieve the required 3-hour fire rating (Fig. 8.3.2.3 and Table 8.3.2.3) (Holm and Bremner 1994).

8.3.3 *Bridges*—More than 500 bridges have incorporated lightweight concrete into decks, beams, girders, or piers (Holm and Bremner 2000). Transportation engineers generally specify higher concrete strengths primarily to ensure high-quality mortar fractions with a high compressive strength combined with high air content that will minimize maintenance. Several Mid-Atlantic state transportation authorities have completed more than 20 bridges using a laboratory target strength of 5200 psi (36 MPa), 6 to 9 percent air content, and a density of 115 lb/ft^3 (1840 kg/m^3). The following are the principal advantages of using light-

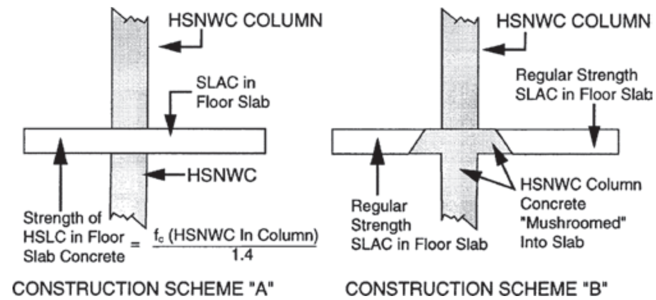


Fig. 8.3.2.2—Alternative construction schemes for transfer of high-strength normalweight concrete column loads through floor slabs (Holm and Bremner 1994).

Fig. 8.3.2.3—Bank of America, Charlotte, NC (Holm and Bremner 1994).

weight concrete in bridges and the rehabilitation of existing bridges:

a) Increased width or number of traffic lanes
b) Increased load capacity
c) Balanced cantilever construction
d) Reduced in seismic inertial forces
e) Increased cover with equal weight, thicker slabs
f) Improved deck geometry with thicker slabs
g) Increased span lengths and reduced pier costs

8.3.3.1 *Increased number of lanes during bridge rehabilitation*—Thousands of bridges in the United States are functionally obsolete with unacceptably low load capacity or an insufficient number of traffic lanes (Stolldorf and Holm 1996). To remedy limited lane capacity, Washington, DC,

Table 8.3.2.3—Mixture proportions and physical properties for concrete pumped on Bank of America project, Charlotte, NC

Mixture no.	1	2*	3
Mixture proportions			
Cement, Type III, lb/yd³ (kg/m³)	550 (326)	650 (385)	750 (445)
Fly ash, lb/yd³ (kg/m³)	140 (83)	140 (83)	140 (83)
LWA 20 mm to No. 5, lb/yd³ (kg/m³)	900 (534)	900 (534)	900 (534)
Sand, lb/yd³ (kg/m³)	1370 (813)	1287 (763)	1203 (714)
Water, gal./yd³ (L/m³)	296 (175)	304 (180)	310 (184)
WRA, fl oz./yd³ (L/m³)	27.6 (0.78)	31.6 (0.90)	35.6 (1.01)
HRWRA, fl oz./yd³ (L/m³)	53.2 (1.56)	81.4 (2.31)	80.1 (2.27)
Fresh concrete properties			
Initial slump, in. (mm)	2-1/2 (63)	2 (51)	2-1/4 (57)
Slump after HRWRA, in. (mm)	5-1/8 (130)	7-1/2 (191)	6-3/4 (171)
Air content	2.5	2.5	2.3
Unit weight, lb/ft³ (kg/m³)	117.8 (1887)	118.0 (1890)	118.0 (1890)
Compressive strength, psi (MPa)			
4 days	4290 (29.6)	5110 (35.2)	5710 (39.4)
7 days	4870 (33.6)	5790 (39.9)	6440 (44.4)
28 days (average)	6270 (43.2)	6810 (47.0)	7450 (51.4)
Splitting-tensile strength, psi (MPa)	520 (3.59)	540 (3.72)	565 (3.90)

*Mixture selected and used on project.

engineers replaced a four-lane bridge originally constructed with normalweight concrete with five new lanes made with lightweight concrete, providing a 50-percent increase in one-way rush-hour traffic without replacing the existing structure, piers, or foundations. Similarly, on Interstate 84, crossing the Hudson River at Newburgh, New York, two lanes of normalweight concrete were replaced with three lanes of lightweight concrete on a parallel span, allowing three-lane traffic in both east- and west-bound lanes.

8.3.3.2 *Increased load capacity*—The elevated section of the Whitehurst Freeway was upgraded to an HS20 loading criteria during the rehabilitation of the Washington, DC, corridor system structure with only limited modifications to the steel framing superstructure. An improved load-carrying capacity was obtained because of the significant dead load reduction brought about by using lightweight concrete to replace the normalweight concrete and asphalt overlay used in the original deck slab (Fig. 8.3.3.2a) (Stolldorf and Holm 1996).

The original elevated freeway structure was designed for HS20 live load according to the AASHTO LRFD (1994). With the significantly lighter replacement concrete deck, a minimum of the structural steel framing required strengthening, and little interruption at the street level below was required to upgrade the substructure to an HS20 live load criteria (Fig. 8.3.3.2b) (Stolldorf and Holm 1996).

8.3.3.3 *Bridges incorporating both lightweight-concrete spans and normalweight concrete spans*—A number of bridges have been constructed where high-performance lightweight concrete has been used to achieve balanced load-free cantilever construction. On the Sandhornoya Bridge, completed in 1989 near the Arctic Circle city of Bodo, Norway, the 350 ft (110 m) sidespans of a three-span bridge were constructed with high-strength lightweight concrete with a cube strength of 8100 psi (55 MPa) that balanced the

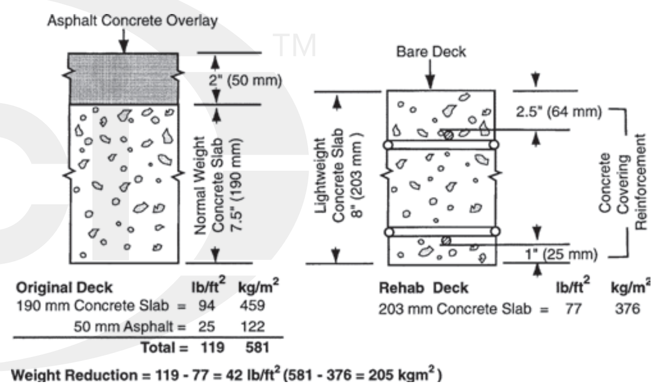


Fig. 8.3.3.2a—Original and rehabilitated decks for Whitehurst Freeway (Stolldorf and Holm 1996).

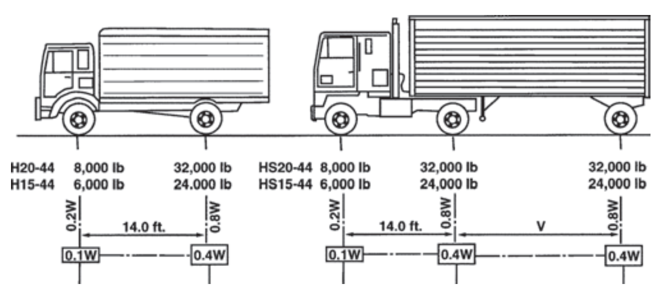


Fig. 8.3.3.2b—AASHTO LRFD (1994) H20-44 and HS20-44 loadings (Stolldorf and Holm 1996).

construction of the center span of 505 ft (154 m) that used normalweight concrete with a cube strength of 6500 psi (45 MPa) (Fergestad 1996).

The Raftsundet Bridge in Norway, north of the Arctic Circle, with a main span of 978 ft (298 m), was the longest concrete cantilevered span in the world when the cantilevers were joined in June 1998; 722 ft (220 m) of the main span was constructed with high-strength lightweight aggregate (LWA) concrete with a cube strength of 8700 psi (60 MPa). The side spans and piers in normalweight concrete had a cube strength of 9400 psi (65 MPa) (Fig. 8.3.3.3) (Expanded Shale, Clay and Slate Institute 2001).

The Benicia-Martinez Bridge across San Pablo Bay in California, is a 1.2 mile (1.9 km) long lightweight concrete segmental bridge. The bridge was constructed using a lightweight concrete mixture that resulted in compressive strengths between 10,000 and 11,000 psi (69 and 76 MPa) (Murugesh 2008). The use of lightweight concrete allowed for reduction of 20 percent of the mass of the superstructure and increased the flexibility of the bridge; both helped to minimize seismic excitation of the structure. The savings associated with the use of lightweight concrete were estimated to be up to $42 million (Murillo et al. 1994).

8.3.4 *Marine structures*—Because offshore concrete structures may be constructed in shipyards or graving docks located considerable distances from the site where the structure may be, then floated and towed to the project site, there is a special need to reduce mass and improve structural efficiency, especially where shallow-water conditions mandate lower draft structures. The structural efficiency is even more pronounced when lightweight concrete is submerged as shown.

A density ratio

$$\frac{\text{(heavily reinforced normalweight concrete)}}{\text{(heavily reinforced lightweight concrete)}}$$

in air of

$$(2.50[156\ \text{lb/ft}^3])/[2.00(125\ \text{lb/ft}^3)] = 1.25$$

when submerged is

$$(2.50 - 1.00)/(2.00 - 1.00) = 1.50$$

8.3.4.1 *Tarsiut Caisson Retained Island, 1981*—The first arctic structure using high-performance lightweight concrete was the Tarsiut Caisson retained island built in Vancouver, BC, and barged to the Canadian Beaufort Sea (Fig. 8.3.4.1). Four large, prestressed concrete caissons 226 x 50 x 35 ft (69 x 15 x 11 m) high were constructed in a graving dock in Vancouver, towed around Alaska on a submersible barge, and founded on a berm of dredged sand 25 mi (40 km) from shore. The extremely high concentration of reinforcement resulted in a steel-reinforced concrete density of 140 lb/ft^3 (2240 kg/m^3). The use of high-strength lightweight concrete was essential to achieving the desired floating and draft requirements (Expanded Shale, Clay and Slate Institute 2001).

8.3.4.2 *Heidron floating platform, 1996*—Because of the deep water (1130 ft [345 m] over the Heidron oil fields in the Norwegian Sea, a decision was made to improve buoyancy and construct the first floating platform with high-performance lightweight concrete. The hull of the floating platform, approximately 91,000 yd^3 (70,000 m^3), was constructed entirely of high-strength lightweight concrete with a maximum density of 125 lb/ft^3 (2000 kg/m^3). The platform was built in Norway and towed to the Norwegian Sea. A mean density of 121 lb/ft^3 (1940 kg/m^3), a mean 28-day cube compressive strength of 11,460 psi (79 MPa), and a documented cylinder/cube strength ratio of 0.90 to 0.93 were reported (FIP 2000; Expanded Shale, Clay and Slate Institute 2001).

8.3.4.3 *Hibernia oil platform, 1998*—The ExxonMobil Oil Hibernia offshore gravity-based structure is a significant application of specified-density concrete. To improve buoyancy of the largest floating structure built in North America, lightweight aggregate (LWA) replaced approximately 50 percent of the normalweight coarse fraction in the high-strength concrete used (Fig. 8.3.4.3). The resulting density was 135 lb/ft^3 (2160 kg/m^3). Hibernia was built in a dry dock in Newfoundland, Canada, and then floated out to a

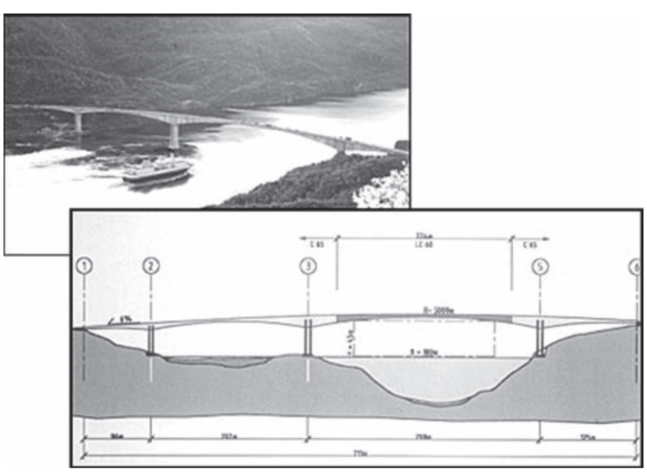


Fig. 8.3.3.3—Raftsundet Bridge (Expanded Shale, Clay and Slate Institute 2001).

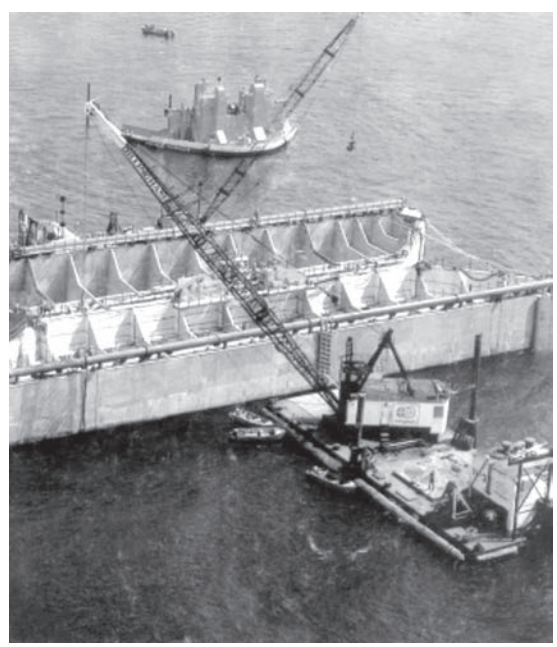


Fig. 8.3.4.1—Tarsuit Caisson Retained Island (Concrete International 1982).

deep water harbor area where construction continued. When finished, the more than 1 million ton structure was towed to the Hibernia oil field site and set in place on the ocean floor. A comprehensive testing program was reported by Hoff et al. (1995).

8.3.5 *Floating bridge pontoons*—High-performance lightweight concrete was used effectively in both the cable-stayed bridge deck and the separate but adjacent floating concrete pontoons supporting a low-level steel box-girder bridge near the city of Bergen, Norway (Fig. 8.3.5). The pontoons are 138 ft (42 m) long and 67 ft (20.5 m) wide and were cast in compartments separated by watertight bulkheads. The design of the compartments was determined by the concept that the floating bridge would be serviceable despite the loss of two adjacent compartments due to an accident.

Fig. 8.3.4.3—Hibernia Offshore Platform (Expanded Shale, Clay and Slate Institute 2001).

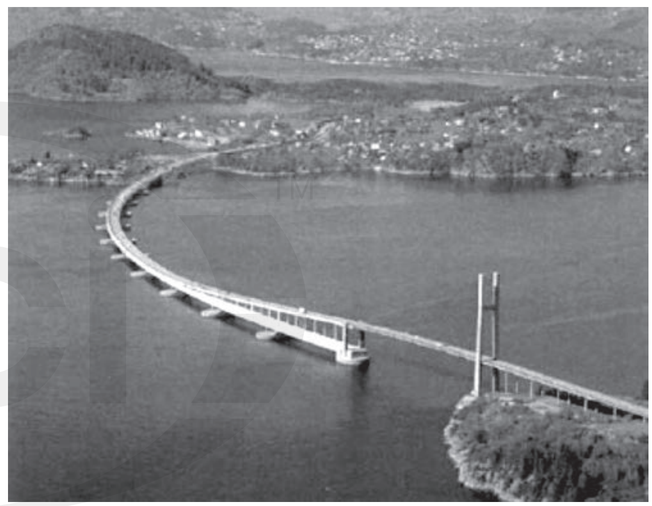


Fig. 8.3.5—Nordhordland Bridge, Bergen, Norway (Elkem Micro Silica 2000).

8.4—Self-consolidating lightweight concrete

Self-consolidating concrete (SCC) is a highly flowable yet stable concrete that can spread readily into place, fill the formwork, and encapsulate the reinforcement without any mechanical consolidation or undergoing any significant separation of material constituents (Okamura and Ouchi 1999) . Self-consolidating lightweight concrete, or lightweight SCC, is SCC made using structural-grade lightweight coarse aggregates, lightweight fine aggregates, or a combination of the two. Like normalweight SCC, lightweight SCC must flow and consolidate under the force of gravity and self-weight alone. In recent years, precast and prestress companies in the United States have begun using self-consolidating lightweight concrete in many applications, including precast seating sections, precast concrete sandwich panels, double tees and precast utility buildings, reinforced floating concrete structures, and insulated concrete form wall systems. Lightweight SCC has been in use in Japan since the early 1990s (Shi et al. 2006; Sugiyama 2003). The batching, mixing, and delivery of lightweight SCC has been accomplished with similar consistency as the normal density SCC. Reports indicate that the compressive strength and modulus of elasticity of lightweight SCC are similar to traditional lightweight concrete having similar *w*/*cm* ratios (Yao and Gerwick 2000). Since Lightweight SCC is easier to place than traditional lightweight concrete, less labor is required to place the concrete in precast elements. The ability of lightweight SCC to flow into tight sections and not require vibration allows for reduction or elimination of internal and external vibration equipment. Lightweight SCC differs from traditional lightweight concrete only in its fresh state, so all the properties present in traditional lightweight concrete and design methods used for traditional lightweight concrete are also valid for lightweight SCC.

As with traditional lightweight concrete, prewetted aggregates are preferable to dry aggregates at time of mixing, as they tend to absorb less water during mixing and, therefore, reduce the possibility of loss of slump flow as the concrete is being mixed, transported, and placed. The difference in specific density of the mortar and coarse aggregate, both normalweight and lightweight, is a driving force in segregation. Consequently, with lightweight SCC, the specific density of the mortar is higher than the specific density of the lightweight coarse aggregate particles. Because the mortar is heavier than the LWA particles, the segregation of the lightweight SCC is most commonly due to the coarse lightweight particles floating to the surface. The density of the mortar can be lowered using mineral additives such as fly ash, silica fume, or slag, and by adjusting the air content, thereby making the density of the mortar closer to the density of the LWA particles (Wall 2005). Because lightweight SCC has a density of 20 to 30 percent less than that of normal density SCC, a higher fluidity mortar is required in lightweight SCC to achieve the self-consolidating properties. The viscosity of the lightweight SCC mortar should be sufficient to prevent the lightweight coarse aggregate from moving to the concrete surface (Wall 2005; Yao and Gerwick 2000). Reportedly, the viscosity of lightweight SCC should be approximately equal to the viscosity of normalweight SCC to achieve similar segregation resistance. Finishing lightweight SCC can be a problem if the mixture viscosity and fluidity are not properly controlled. In situations where small amounts of lightweight coarse aggregate float to the top surface, contractors have troweled the loose coarse aggregate back into the lightweight SCC with little difficulty, making for a smooth surface finish. If segregation is occurring and more than the near surface lightweight coarse aggregate is loose on the top surface, the mixture should not be used and the mixture viscosity adjusted.

8.5—Advantages of lightweight concrete

The use of lightweight concrete is usually predicated on the reduction of project cost, improved functionality, or a combination of both. Estimating the total cost of a project is necessary when considering lightweight concrete because the cost per cubic yard (cubic meter) is usually higher than a comparable unit of normalweight concrete. The following example shows some advantages from the cost perspective of lightweight compared to normalweight concrete on a bridge project.

For example, assume the in-place cost of a typical short-span bridge may vary from 50 to 200 $\$/ft^2$ (540 to 2150 $\$/m^2$).

If the average thickness of the deck was 8 in. (200 mm), then one cubic yard (cubic meter) of concrete would yield approximately 40 ft^2/yd^3 (5 m^2/m^3).

In-place cost using lightweight aggregate is generally approximately 1 percent more than conventional normalweight concrete. This increase would easily be offset by significant increases in bridge, building, or marine structure functionality or any of the following economic factors:

a) The reduction in building weight foundation loads may result in smaller footings, fewer piles, smaller pile caps, and less reinforcing.

b) Reduced dead load selfweight may result in smaller supporting members (decks, beams, girder, and piers), resulting in a major reduction in cost.

c) Reduced dead load selfweight will mean reduced inertial seismic forces.

d) In bridge rehabilitation, the new lightweight deck may be wider or an additional traffic lane may be added without structural or foundation modification.

e) On bridge deck replacements or overlays, the deck may be thicker than the original normalweight deck or overlay to allow more cover over reinforcing or to provide better drainage without adding additional dead load to the structure.

f) With precast-prestress use, longer or larger elements can be manufactured without increasing overall mass. This may result in fewer columns or pier elements in a system that is easier to lift or erect, and fewer joints or more elements per load when transporting. At some precast plants, each element's shipping cost is evaluated by computer to determine the optimum concrete density.

g) Lightweight aggregate concrete presents an enhanced durability (refer to Chapter 6) that may result in a longer service cycle or reduced maintenance.

h) In marine applications (that is, bridges, piers, and oil platforms), increased allowable topside loads and the reduced draft resulting from the use of lightweight concrete may permit easier movement out of dry docks and through shallow shipping channels.

i) Due to the greater fire resistance of lightweight concrete as reported in ACI 216.1, if the slab thickness is controlled by fire rating, the thickness of slabs may be reduced, resulting in significantly less concrete volumes.

Products made out of lightweight concrete are often used to enhance the expression of a structure. In building construction, this usually applies to cantilevered floors, expressive roof design, taller buildings, or additional floors added to existing structures. Improved constructability may result in cantilever bridge construction where lightweight concrete is used on one side of a pier and normalweight concrete used on the other to provide weight balance while accommodating a longer span on the lightweight side of the pier. The use of lightweight concrete might also be considered when better insulating qualities are needed in thermally sensitive applications like hot water, petroleum storage, or building insulation (5.13).

8.5.1 *Transportation advantages*—In projects where transportation is relevant, there can be significant transportation savings developed through the use of lightweight concrete. Studies demonstrated that the transportation cost savings were seven times more than the additional cost of LWA. Savings vary with the size and mass of a product and are most significant for the smaller consumer-type products. Less truck traffic in congested cities is not only environmentally friendly but also generates fewer public complaints. The potential for lower costs is possible when shipping by rail or barge, but is most often realized in trucking where highway loadings are posted. Meyer and Kahn (2002) include the important economic benefits of using high-strength lightweight concrete on bridge girders to save on the transportation costs.

For more than 20 years, precast manufacturers have evaluated trade-offs between physical properties and transportation costs. In one study, a typically used limestone control concrete was paralleled by other mixtures in which 25, 50, 75, and 100 percent of the limestone coarse aggregate was replaced by an equal absolute volume of LWA. Results are reported by Holm and Ries (2000) of the testing program that measured and compared compressive strength, tensile strength, and modulus-of-elasticity-with-density data, as shown in Fig. 8.5.1. By adjusting the density of the concrete, precasters are able to minimize the number of truck deliveries without exceeding highway load limits and produce important savings.

8.6—Sustainability of lightweight concrete

The word sustainability, when used in the concrete industry, generally refers to the following conditions.

a) Saving on materials being used in the project

b) Extended lifecycle through enhanced durability in the environment for which it was designed

c) Overall environmental impact of manufacturing, transporting, and placing the concrete product in the final structure

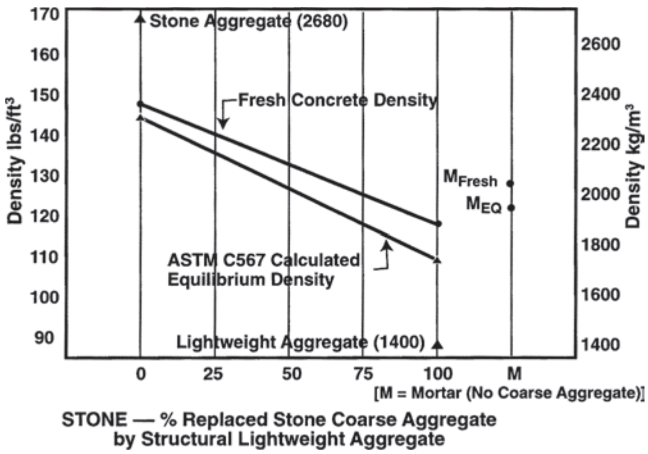


Fig. 8.5.1—Fresh and ASTM C567/C567M-calculated equilibrium concrete density with varying replacements of limestone coarse aggregate with structural LWA (Holm and Ries 2000).

Material savings in construction and transportations savings (8.5) and energy saving in heating and cooling are an example of the reduced impacts that lightweight concrete can provide in a determinate project. The overall cycle costs need to be determined case by case; however, in many cases, the added energy and cost required in manufacturing lightweight aggregate (LWA) often contributes to the overall energy savings, the service life, and sustainability of concrete structures.

The increased usage of processed LWA is evidence of environmentally sound planning. Products made with LWA require less trucking and use materials that have limited structural applications in their natural state, thereby minimizing construction industry demands on finite resources of natural sands, stones, and gravels

CHAPTER 9—ENHANCED PERFORMANCE DUE TO INTERNALLY STORED WATER (INTERNAL CURING)

Concrete containing lightweight aggregate (LWA) is becoming increasingly used to supply internal curing (Jensen 1993). This section describes the concept of internal curing, reviews mixture proportioning procedures, and describes the influence of the LWA on the properties of the concrete.

9.1—Concept of internal curing

A benefit of using prewetted LWA in concrete was first noticed by Klieger (1957), who stated that, "Lightweight aggregates absorb considerable water during mixing which apparently can transfer to the paste during hydration," based on industry observation that the use of LWA appeared to improve concrete performance. Campbell and Tobin (1967) performed an experimental program that compared normalweight and lightweight concrete strengths of cores taken from field-cured exposed slabs with test results obtained from laboratory specimens cured strictly in accordance with ASTM procedures. Their tests confirmed that availability of absorbed moisture within the expanded LWA produced a

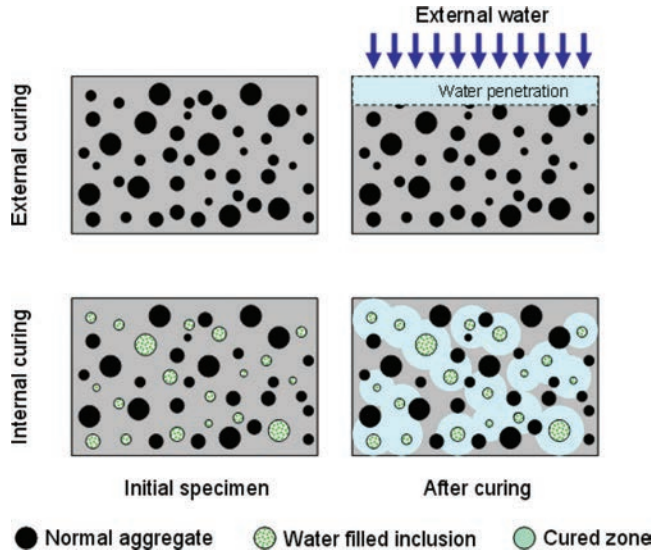


Fig. 9.1—Illustration of the difference between internal and external curing. The water-filled inclusions should be distributed uniformly and spaced close enough to provide coverage for the entire paste system (Castro et al. 2010a).

more forgiving concrete that was less sensitive to poor field-curing conditions. Philleo (1991) suggested that internal curing was more useful for higher performance concrete to compensate for self-desiccation. While these describe beneficial effects of the time release of water from prewetted LWA when it is needed, it is only recently that this technology has become more commonly used and commonly referred to as internal curing. Figure 9.1 shows that when external curing is used, water is supplied from the surface; however, when internal curing is used, it enables the water to be distributed more uniformly throughout the cross section.

Internal curing has emerged as a new technology that holds promise for producing concrete with increased resistance to early-age cracking and enhanced durability (NISTIR 776-2011). While this document provides a general overview of internal curing and its benefits to concrete performance, it should be noted that several reports have been developed over the last decade that include additional reference information that can provide the reader with additional details on internal curing (Hoff 2002; RILEM 2007; Bentz and Mohr 2008; ACI 231R-10). This section is based largely on a report by NISTIR 776-2011.

The need for internal curing comes directly from the basic nature of cement hydration reactions. As cement and water react, the hydration products generally occupy less space than the original constituents (that is, the paste volume decreases, resulting in an external volume change known as autogenous shrinkage and vapor filled pore spaces developing in the paste). As a result, hydration and pozzolanic reactions produce a net volume change termed chemical shrinkage. For example, Bentz (1997) showed that the hydration of tricalcium silicate involves a net reduction in volume of 9.6 percent, or approximately 0.07 mL/g C3S (tricalcium silicate). The benefits of internal curing are increasingly important when pozzolans, for example silica fume, fly ash, and metakaolin, are included in the mixture. The benefit of internal curing is particularly useful because the pozzolanic materials can have a chemical shrinkage that is two to three times greater than cement (NISTIR 776-2011). Another benefit is that the pozzolanic reaction itself is highly dependent on the consistent presence of moisture.

9.2—Mixture proportioning for internal curing

Determining the mixture proportions for internally cured concrete revolves around the need to provide additional water to fill the vapor-filled porosity that is created by chemical shrinkage. Providing additional water allows the concrete to maintain a higher degree of saturation and enables it to hydrate more of the cement while minimizing the development of autogenous shrinkage and early-age cracking.

Three key considerations are important to the mixture proportions of an internally cured concrete (Bentz et al. 2005; Henkensiefken et al. 2008): 1) How much internal curing water is necessary? 2) How far into the surrounding cement paste can the needed water readily travel? and 3) How are the lightweight aggregate (LWA) reservoirs spatially distributed within the concrete?

The first consideration, as described in Eq. (9.2) (Bentz et al. 2005), equates the water demand of the hydrating mixture (right hand side of Eq. (9.2)) to the supply that is available from LWA (left hand side of Eq (9.2)).

$$M_{LWA} \times S^{*}\Phi_{LWA} = C_f \times CS \times \alpha_{max} \qquad (9.2)$$

More details for Eq. (9.2) and its use are available in the literature (Bentz et al. 2005; Castro 2011). Espinoza-Hijazin and Lopez (2011) showed that internal curing can enhance hydration and concrete performance in mixtures of *w*/*c* above 0.42 under poor external curing. Under drying conditions, the water lost to the environment can be replaced by the water within the internal curing agent, thereby minimizing the actual drying of the paste and the negatives impacts associated with it.

The second consideration is water movement. This has been investigated by several researchers using x-ray or neutron radiography or tomography (Bentz et al. 2006a, 2006b, 2007). Lura et al. (2003) and Henkensiefken et al. (2011) used x-ray radiography to estimate water movement and observed that after 24 hours the travel distance was at least 0.08 in. (2 mm). Bentz et al. (2005) used x-ray microtomography and confirmed the relationship between supply and demand as mentioned previously. Trtik et al. (2011) used neutron radiography and observed that water could move at least 0.12 in. (3 mm) during the first 20 hours of curing without moisture gradients from the aggregate surface.

The third consideration is lightweight aggregate spacing. Bentz and Snyder (1999) developed a computer model that enables the volume of paste within a certain distance of the LWA to be quantified using a concept similar to the protected paste volume concept (NISTIR 6265). The user provides the sieve size distribution for the aggregates, the fractional replacement of LWA for normalweight aggregates for each sieve on a volume basis, and the total volume fraction of all aggregates in the mixture to obtain a table of the protected paste volume (0 to 1), as a function of distance from the LWA surfaces. Figure 9.2a provides an example two-dimensional color-coded image that is also provided to the user from the program by Bentz and Snyder (1999). In this example (Fig. 9.2a), a fine-graded LWA is used that provides a good distribution of the LWA throughout the matrix. Because all of the cement paste is within a 0.04 in. (1 mm) distance of an LWA surface, the water can be transported to the paste. If the LWA were spaced further apart (that is, a lower volume of LWA or a larger LWA), the paste would appear as well, indicating that the paste would not receive the benefit of internal curing.

Using the hard core/soft shell model described previously, Henkensiefken et al. (2009c) modeled 16 different mixtures with different size aggregates and sands with varying fineness modulus values. Figure 9.2b shows the volume of protected paste in concrete when 30 percent of the aggregate (either coarse or fine) on a volume basis is replaced by prewetted LWA and water movement is assumed to be limited to 0.08 in. (2 mm). If 30 percent of the coarse aggregate is replaced, regardless of the selected size, none of the tested coarse LWA will protect more than 50 percent of the paste volume. Conversely, if the LWA used for internal curing had a lower fineness modulus (finer sand), nearly all the paste would be within 0.08 in. (2 mm) of an LWA particle. When a coarser LWA (higher fineness modulus) is used, less paste is protected. Any LWA with a fineness modulus greater than 3.2 would protect less than 90 percent of the paste. If such LWA were used, the volume of the LWA fine aggregate might have to be increased to ensure that an adequate fraction of the paste was protected. This information is only intended to illustrate that the particle size distribution (fineness modulus) is an important consideration, not that LWAs with a fineness modulus greater than 3.2 should not be used or will not work.

9.3—Properties of the aggregate for internal curing

While ASTM C128 uses the cone method to determine the aggregate moisture content, alternative tests using cobalt chloride or a paper towel method (New York Department of Transportation 2008) can provide similar information without the complexities associated with the angular nature

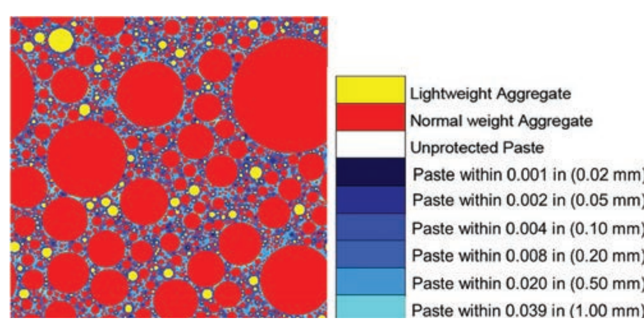


Fig. 9.2a—Example of two-dimensional image 1.8 x 1.8 in. (30 x 30 mm) from internal curing simulation (Bentz et al. 2005)

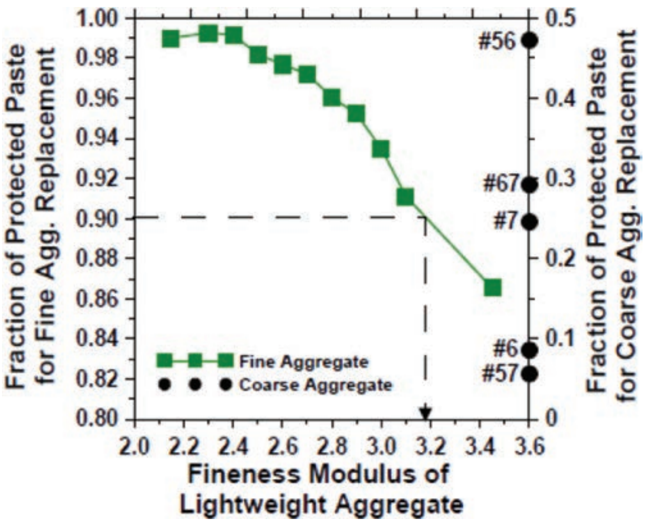


Fig. 9.2b—Volume of protected paste in concrete where 30 percent of aggregate by volume is replaced with different coarse aggregate or sand with different fineness modulus (Henkensiefken et al. 2009c) based on ASTM C33/C33M guidelines

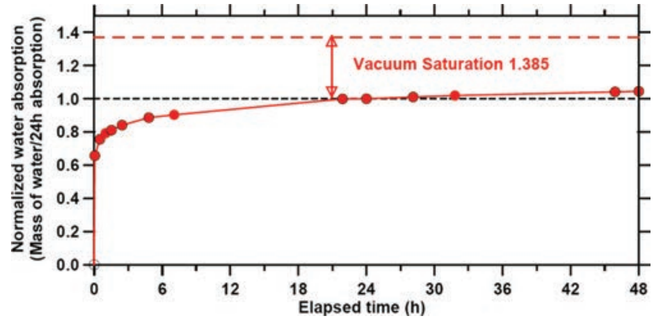


Fig. 9.3a—Typical time-dependent water absorption of LWA (Castro et al. 2011).

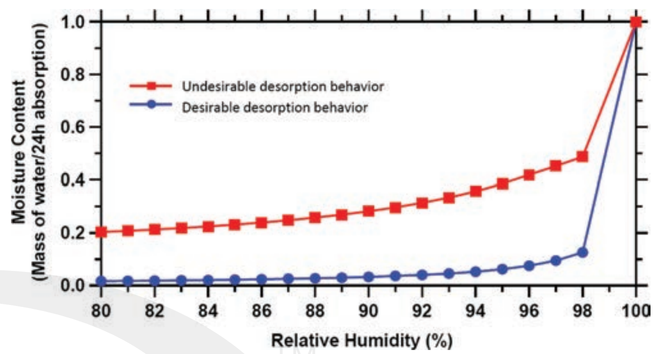


Fig. 9.3b—Illustration of desirable and undesirable aggregate desorption behavior (Castro et al. 2011).

of the particles (Castro et al. 2011). Typical 24-hour absorption values for a variety of lightweight aggregate (LWA) from North America range between 6 and 31 percent. While the 24-hour absorption provides a single value, it is important to note that the absorption of many LWAs varies over time. Therefore, the absorption properties must be clarified with a time descriptor, such as reporting the 24-hour absorption capacity of a specific LWA (Fig. 9.3a). Castro et al. (2011) observed a similar rate of water absorption for the first 48 hours for a wide variety of LWA, when the data was normalized by their individual 24-hour absorptions.

For LWA to function successfully as an internal curing reservoir, the pores containing the water must be larger than those in the surrounding cement paste so that water will preferentially move from the LWA to the hydrating cement. This ability of the LWA to release water at high relative humidities can be quantified by measuring the absorption/desorption properties of the LWA particles, as show in Table 9.3. The data in Table 8.3.2.3 reflects the normalized percentage of the 24-hour measured absorption capacity that remains in the aggregate when it is brought into equilibrium with a given relative humidity.

Several studies have used saturated salt solutions to control the relative humidity over prewetted LWAs to examine their mass loss or desorption (Bentz et al. 2005; Radlinska et al. 2008). Castro et al. (2011) employed dynamic vapor desorption in which the sample was placed in a high resolution balance in an air stream within a carefully controlled relative humidity environment to measure its desorption isotherm. These measurements indicated that approximately 90 percent of the 24-hour absorbed water is readily released at high relative humidities (greater than 93 percent) from nearly all of the examined expanded clay, shale, and slate LWAs currently produced in the U.S. (Castro et al. 2010b). This is not true, however, for all porous materials as shown in Fig. 9.3b. While the ideal aggregate shows a release of approximately 90 percent of its water during drying, that is the remaining normalized percentage of the 24-hour absorption is approximately 10 percent, the less than ideal aggregate shows a release of only 60 to 70 percent. As a result, an approximately 25 percent increase in the volume of the less than ideal aggregate would be required for equivalent internal curing performance. While the dynamic desorption approach provides data below 98 percent relative humidity, alternative methods may be applied at higher relative humidities. Pour-Ghaz et al. (2011) used a pressure plate method suggested by Johansson (2005) to examine the desorption of aggregates at these higher relative humidities. This method also has the advantage of being able to test a larger sample and a range of aggregate sizes. The testing, however, requires a longer duration of approximately 1 week for each relative humidity selected.

9.4—Influence of internal curing on concrete properties and behavior

Internal curing can influence the performance or properties of concrete in a variety of ways. This section describes the properties of typical internally cured concretes as compared with conventional concrete.

Time-dependent improvement in the quality of concrete containing lightweight aggregate (LWA) is typically greater than that observed with normalweight aggregate under sealed or drying conditions. This occurs because internal curing enables additional hydration of the cementitious fraction provided by moisture available from the LWA. This can

Table 9.3—Desorption behavior of expanded clay, shale, and slate aggregates in North America (Castro et al. 2011; Pour-Ghaz et al. 2011)

LWA No.		24-h absorption, %	Relative humidity, %					
			99.9*	99.6*	98.9*	98†	96†	94†
Clay	1	15.3	—	—	—	0.20	0.14	0.11
	2	30.5	0.70	0.69	0.61	0.17	0.10	0.06
	3	17.7	0.87	0.70	0.52	0.24	0.16	0.12
Shale	4	17.5	0.91	0.61	0.45	0.20	0.12	0.09
	5	14.1	0.61	0.48	0.50	0.08	0.04	0.03
	6	10.0	—	—	—	0.11	0.06	0.04
	7	15.6	0.78	0.54	0.48	0.17	0.08	0.05
	8	15.0	0.69	0.57	0.61	0.18	0.08	0.06
	9	15.7	0.63	0.44	0.39	0.10	0.06	0.04
	10	19.5	0.87	0.68	0.48	0.29	0.20	0.15
	11	18.1	0.93	0.68	0.47	0.29	0.21	0.17
	12	18.5	0.94	0.77	0.54	0.28	0.19	0.14
Slate	13	12.2	0.91	0.50	0.36	0.14	0.07	0.05
	14	6.0	0.93	0.52	0.38	0.10	0.06	0.04

*Determined from the pressure plate method (Pour Ghaz et al. 2011).

†Determined from the dynamic vapor desorption (Castro et al. 2011).

be seen in Fig. 9.4, where the heat of hydration is higher at 3 days for the mixtures with a *w/c* of less than 0.45. Similar results have been observed by Bentz et al. (2005) using calorimetry and Lura et al. (2010) using chemical shrinkage concurrently with calorimetry.

9.4.1 *Effect of internal curing on plastic shrinkage*—Concrete can be susceptible to cracking at the time of placement if the evaporation rate is high (Villarreal and Crocker 2007). While these cracks are not generally a cause for concern in terms of the load the structure can carry, they are often unsightly and can lead to the ingress of aggressive agents that could accelerate the corrosion of reinforcing steel (Lura et al. 2007). Studies have been conducted to compare the plastic shrinkage and cracking tendencies of concretes with and without internal curing (Henkensiefken et al. 2010). Plastic shrinkage cracking is reduced with internal curing as water in the LWA replenishes water lost due to evaporation (Henkensiefken et al. 2010). Note, however, that when internal curing water is used to compensate for evaporation, it is not available for internal curing at a later age.

9.4.2 *Effect of internal curing on concrete strength*—The effects of internal curing on compressive strength depend on the specific mixture proportions, curing conditions, and testing age. While mixtures with internal curing could increase strengths due to an increase in the degree of hydration of the cementitious binder, a decrease in strength could be observed as the internal curing agents are likely mechanically weaker than the normalweight aggregates they are replacing. In practice, both increases and decreases in strengths have been observed due to these competing effects (Hoff 2003). In general, decreases are observed at earlier testing ages (less than 7 days) whereas increases are obtained at later testing ages. The following provides reviews of experimental studies. Note also that curing conditions are as equally important.

Weber and Reinhardt (1999) investigated the effect of internal curing in a mixture with *w/cm* of 0.3 where 10 percent by mass of the cement was replaced by silica fume.

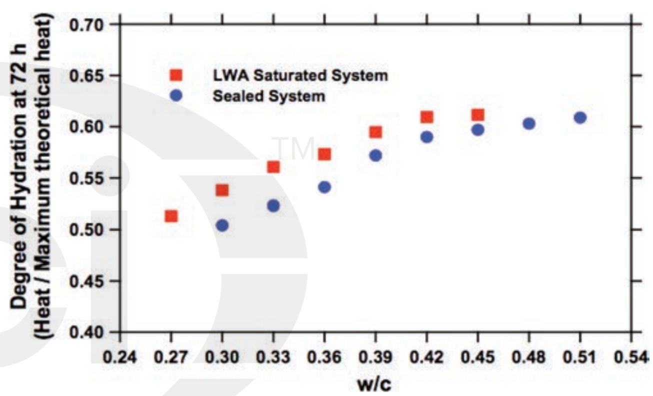


Fig. 9.4—Influence of w/c and internal curing on degree of hydration (Castro et al. 2011).

Twenty-five percent of the fine aggregate was replaced by prewetted LWA to apply internal curing. After 1 year, the difference in strength between sealed and continuously external cured specimens was only 3 percent, which demonstrated that the gain in strength due to external curing was insignificant when compared with a concrete in which an adequate internal curing was provided.

Lopez et al. (2008a) compared two high-performance concrete mixtures at 0.23 *w/cm* with 15 percent Class F fly ash and 10 percent silica fume cement replacement using prewetted LWA or air-dried aggregate in the coarse aggregate fraction. At 24 hours, both mixtures had a compressive strength of 10,585 psi (73 MPa). After 1 year, however, the prewetted LWA mixture had a compressive strength of 12,685 psi (87.5 MPa) whereas the air-dried LWA produced a lower strength of 11,235 psi (77.5 MPa). This showed that the internal curing provided by the prewetted LWA allowed for development of higher long-term compressive strength through enhanced hydration.

In systems with supplementary cementitious materials, internal curing often enhances strength at later ages, as the additional water supplied by the internal curing reservoirs is available for the longer term pozzolanic and hydraulic reactions (Bentz 2007). Figure 9.4.2a shows an example of strength gain for high-volume fly ash (HVFA) mortars with a *w/cm* of 0.30 using Class C fly ash and proportioned according to Eq. (9.2). Strength increases were observed for the HVFA mixtures with either a 40 or a 60 percent volumetric replacement of cement by fly ash. Furthermore, the benefits of internal curing at later ages are clearly evident in Fig. 9.4.2a as the strength gain in the internally cured mixtures is greater than that of the plain mixture due to the LWA providing additional water to increase the hydration of the paste.

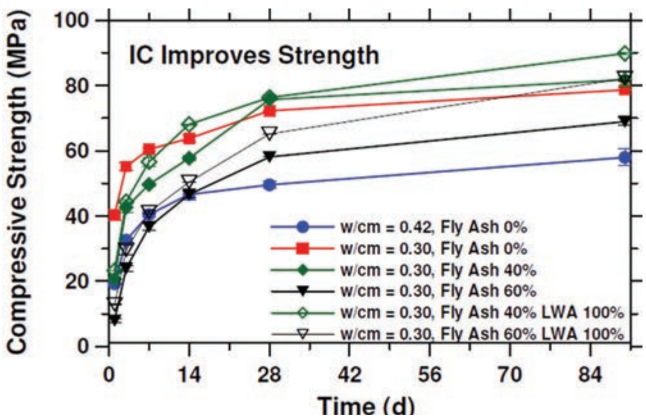


Fig. 9.4.2a—Influence of internal curing on improved strength of mixtures containing supplementary materials (De la Varga et al. 2011)

Golias (2010) examined the benefits of internal curing in four mortar mixtures with *w/c* of 0.30 or 0.50 (with and without internal curing) with four exposure conditions (drying at 50 percent relative humidity, sealed, moist room, or under water). Results are presented in Fig. 9.4.2b. In the water-cured or moist-cured samples, little difference exists between the internally cured mortar and the plain mortar without internal curing. This is expected because both mortars were provided with sufficient external water to aid in hydration. Although the performance of the sealed specimens was similar to that of the moist-cured ones at early ages, the influence of additional curing water becomes evident at later ages, for example, 91 days. The influence of internal curing is most dramatic for the samples stored in a drying environment (50 percent relative humidity), where the plain mortar shows a substantially reduced strength relative to the mixtures that incorporated internal curing.

Byard and Schindler (2010) evaluated the splitting tensile behavior of internal curing mixtures made with expanded

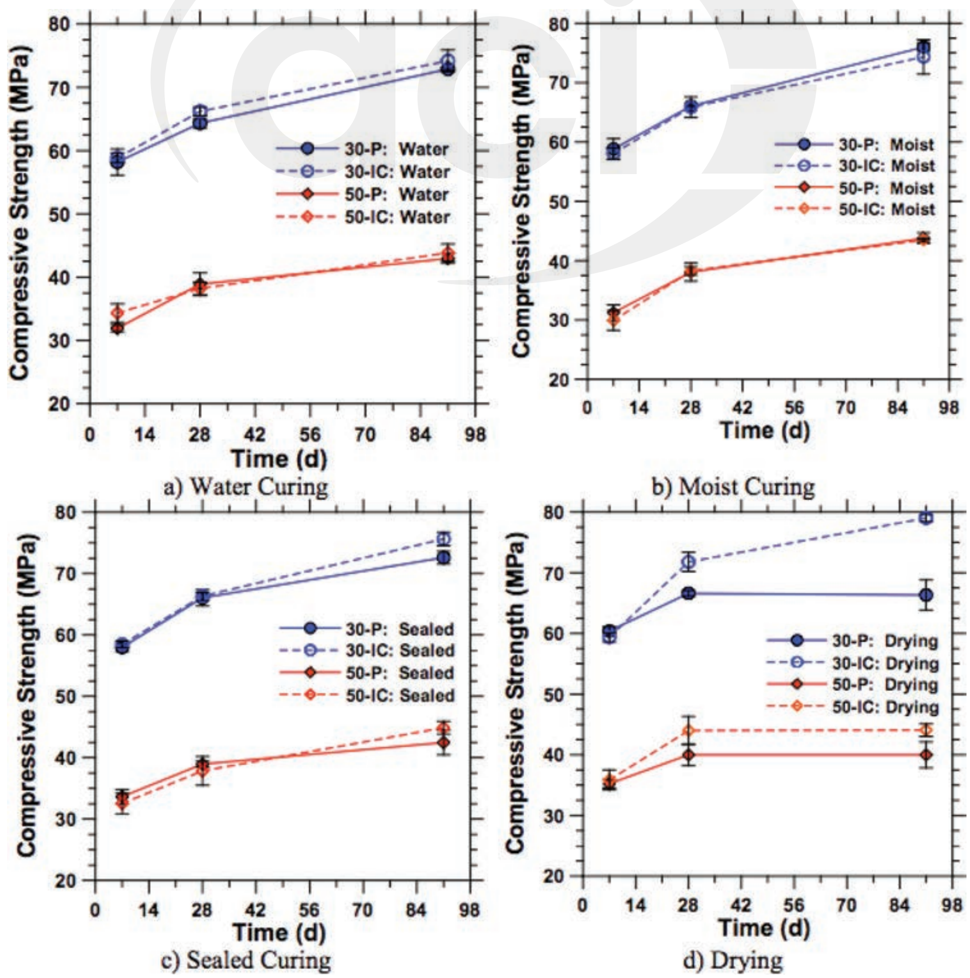


Fig. 9.4.2b—Influence of curing conditions on compressive strength (Golias 2010).

shale, clay, and slate fine LWAs relative to a normalweight concrete in bridge deck applications. The only difference between these mixtures is that some fine LWA was used in place of normalweight fine aggregate. The concrete mixtures were made and cured under conditions that simulate summer and fall placement conditions. Results from this study are summarized in Fig. 9.4.2c. All internal curing concretes exhibited an increase in tensile strength when compared to the normalweight control concrete. The increase in tensile strength was greater under summer conditions than fall conditions, which is attributed to the effects of internal curing. Additionally, the increase in tensile strength was greater for the clay and shale than the slate internal curing concretes, which occurred because these specific mixtures provided more internal curing water, which promotes increased cement hydration.

Paul and Lopez (2011) related the beneficial effects of internal curing from the hydration point of view with the structural capacity of some LWAs. They concluded that the combination of desorption capacity and intrinsic strength of the LWA maximizes the effect of internal curing in the compressive strength of concrete.

9.4.3 *Effect of internal curing on elastic modulus*—The influence of internal curing on elastic modulus is shown in Fig. 9.4.3a. The modulus is generally lower for systems containing LWA. Influence of the LWA is more pronounced in a lower *w*/*c* system because the modulus of the paste is higher. The reduced elastic modulus can also be related to the reduction in residual stress and a reduction in cracking potential (Weiss et al. 1999; Shah and Weiss 2000; Shin et al. 2011). Raoufi et al. (2011) conducted a series of simulations to better understand the influence of reduced concrete stiffness on early age cracking potential. It was determined that the stresses that build up are typically reduced by approximately 10 to 20 percent due to the reduction in elastic modulus caused by the LWA in systems with internal curing. This is in addition to the internal curing benefit from the water in the prewetted LWA.

Byard and Schindler (2010) also evaluated the modulus of elasticity behavior of internal curing mixtures made with expanded shale, clay, and slate fine aggregates relative to a normalweight concrete in bridge deck applications. Results from this study are summarized in Fig. 9.4.3b. The use of LWA in the concrete decreased the unit weight and, thus, the modulus of elasticity of the concrete. The effect of using LWA on the modulus of elasticity was accounted for with reasonable accuracy through the use of Eq. (9.4.3) (ACI 318).

$$E_c = 33 w_c^{1.5} \sqrt{f_c} \qquad (9.4.3)$$

9.4.4 *Effect of internal curing on creep*—Few studies concerning creep of systems with internal curing have been conducted. Lopez (2005) and Lopez et al. (2008a, b) showed that internally stored water reduced both basic and drying creep of high performance lightweight concrete mixtures loaded at 1 and 28 days of age and kept under load for 500 days. The reduction in creep was reported to

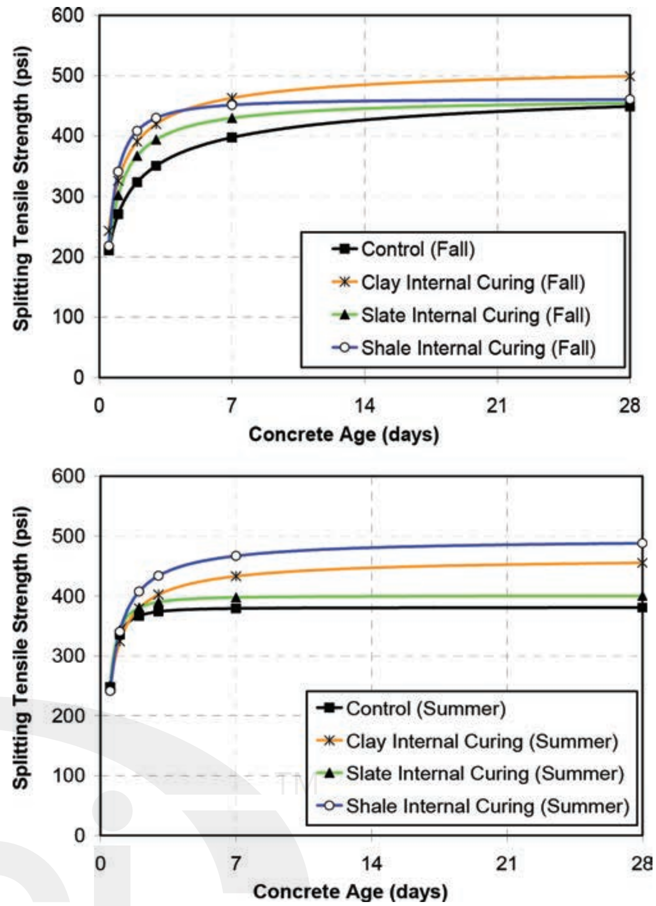


Fig. 9.4.2c—Effect of internal curing on splitting tensile strength for specimens cured under fall and summer conditions (Byard and Schindler 2010).

be caused by hydration enhancement, expansion afforded by internal curing, and by inhibition of water seepage. Conversely, Cusson and Hoogeveen (2005) measured a moderate increase in the tensile creep coefficient of *w*/*c* = 0.34 concrete mixtures with internal curing measured at 7 days versus a control mixture.

9.4.5 *Effect of internal curing on volume change and cracking*—One of the most important benefits of having internally stored water is the reduction or elimination of self-desiccation and the shrinkage associated with it. Several authors have investigated the use of internal curing for reducing autogenous shrinkage (Kohno et al. 1999; Bentz and Snyder 1999; Bentur et al. 1999; Jensen and Hansen 2001, 2002; Zhutovsky et al. 2002a, b; Henkensiefken et al. 2008). All concluded that the mixtures with LWA had considerably less autogenous shrinkage than their counterparts with no internal curing. Some of the mixtures in those studies also expanded.

Radlinska et al. (2008) illustrated the differences between the shrinkage of sealed (autogenous) and unsealed (autogenous and drying) shrinkage for mixtures with internal curing. The benefit of reducing autogenous shrinkage or delaying drying shrinkage (Bentz 2005; Cusson and Hoogeveen 2008; Henkensiefken et al. 2009a) is that the age of cracking can be delayed or cracking eliminated. Figure 9.4.5a shows

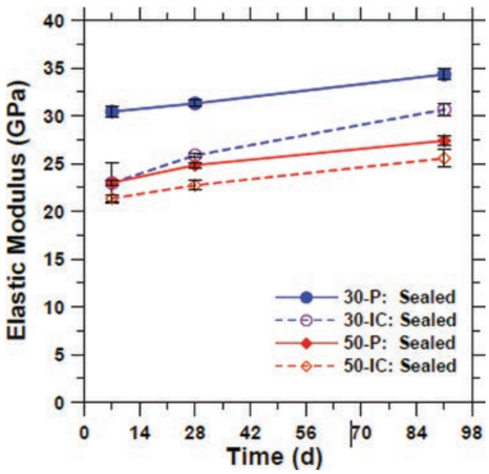


Fig. 9.4.3a—Influence of internal curing on elastic modulus in sealed concrete (Golias 2010).

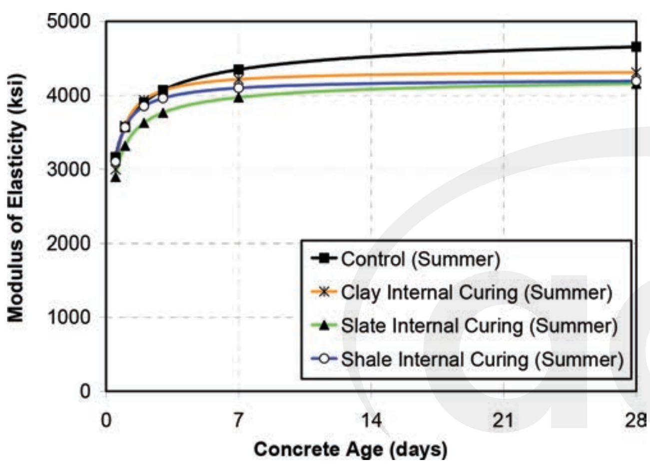


Fig. 9.4.3b—Effect of internal curing on modulus of elasticity for specimens cured under summer conditions (Byard and Schindler 2010).

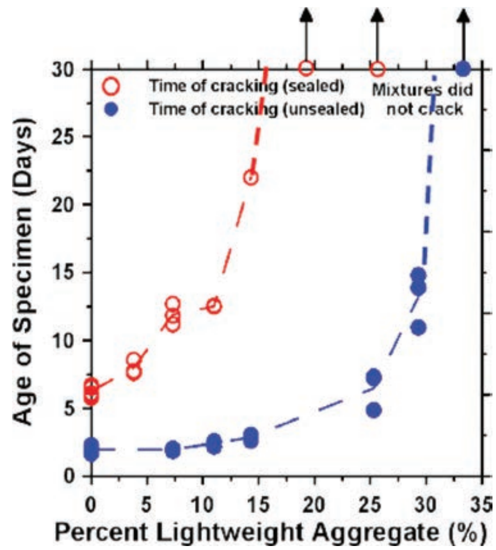


Fig. 9.4.5a—Influence of internal curing on restrained shrinkage cracking (Henkensiefken et al. 2009a).

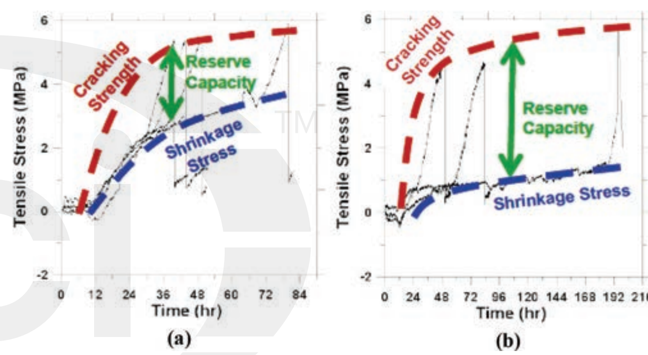


Fig. 9.4.5b—Influence of internal curing on residual stress development and reserve stress capacity. (a) Plain mixture; and (b) mixture containing internal curing (Schlitter et al. 2010).

the age of cracking for these mixtures as a function of the amount of internal curing; for example, the volume of LWA used. Note that the percent of LWA that corresponds to that predicted by Eq. (9.2) is 23.7 percent by volume. Mixtures exposed to drying in addition to autogenous shrinkage are more likely to crack than mixtures experiencing autogenous shrinkage only. As the volume of LWA increases, the age of cracking is delayed until an asymptote appears to be reached.

In addition to reducing the potential of cracking due to autogenous shrinkage, Fig. 9.4.5b shows examples of the residual tensile stress development in a plain mixture and a mixture with internal curing (Schlitter et al. 2010). By reducing the stress that is developed due to autogenous shrinkage, a greater reserve capacity exists in the internally cured mortar to resist stresses due to thermal loading, applied loading, or both. In a typical concrete structure, the temperature increases during the first few days of curing due to the heat released by the hydration reactions and then decreases toward ambient. As the structure cools from its maximum temperature to ambient, thermal cracking could occur. During the first 72 hours, no cracking occurred in the internally cured specimens when the temperature was reduced by as much as 58°F (32°C) whereas the plain mortar specimens cracked when the temperature was reduced by only –18°F (10°C) or –22°F (12°C). This shows a substantial increase in the potential robustness of materials made using internal curing at early ages with respect to thermal shock, cooling, or diurnal temperature changes.

Byard and Schindler (2010) evaluated the effect of using LWA for internal curing on the cracking tendency of bridge deck concrete by rigid cracking frame testing techniques. The rigid cracking frame used (Fig. 9.4.5c(a)) was adapted from the configuration in RILEM Technical Committee 119 (1998). The test setup used allowed the measurement stress development, due to thermal and autogenous shrinkage effects, from setting until the onset of cracking. Temperature of the specimens was controlled to simulate the concrete temperature history of each specific mixture as it would develop in an 8 in. (200 mm) thick bridge deck. Results for a fall placement scenario in the southeastern parts of the United States from this study are summarized in Fig. 9.4.5c(b). Byard and Schindler (2010) concluded that the use of LWA to produce internal curing concretes delays the occurrence of cracking at early ages in bridge deck concrete applications when compared with the normalweight control concrete. This improvement in cracking behavior is attributed to the increase in tensile strength and the decrease in modulus of elasticity, coefficient of thermal expansion, and autogenous shrinkage of the internal curing concretes when compared with the normalweight control concrete.

9.4.6 *Effect of internal curing on porosity*—The microstructure of high performance-blended cement mortars with and without internal curing has been examined using scanning electron microscopy by Bentz and Stutzman (2008). Although representative images are provided for cements blended with silica fume in Fig. 9.4.6, systems with fly ash and slag have also been examined (Cusson 2008). In contrasting the microstructures of specimens with internal curing and those without, the former contain fewer and smaller unhydrated cement particles, (indicating enhanced hydration), fewer and smaller empty pores (indicating less self-desiccation), and a denser and more homogeneous interfacial transition zone (ITZ) between LWAs and hydrating cement paste, which has been observed in previous studies on lightweight concrete (Holm et al. 1984).

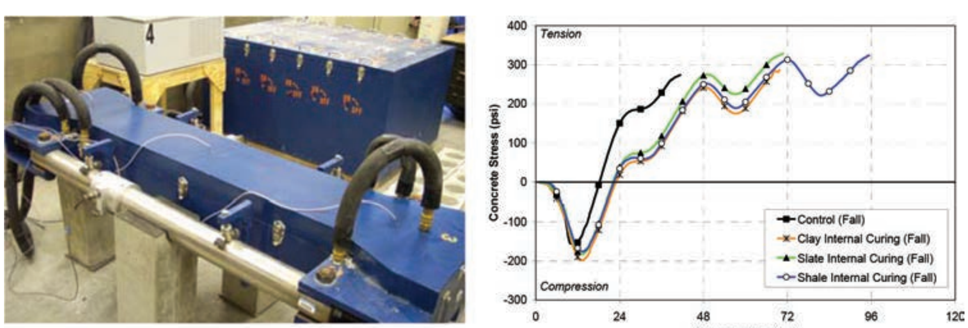
9.4.7 *Effect of internal curing on fluid transport*—Internal curing rests in the reduction of permeability that develops from a significant extension in the time of curing. Powers et al. (1959) showed that extending the time of curing increased the volume of cementitious products formed, which then caused the capillaries to become segmented and discontinuous. Weber and Reinhardt (1997) obtained an important decrease in average pore size between 180 and 365 days in concrete with prewetted LWA, even though no external curing was provided for enhancing hydration. Thomas (2006) found a considerable reduction in chloride permeability between 1 and 3 years in concrete mixtures with prewetted LWA. Zhang and Gjørv (1991) observed that the permeability of high-strength lightweight concrete is more dependent on the properties of the cement paste than the porosity of the LWA. Espinoza-Hijazin and Lopez (2011) examined the role of internal curing as a method to combat poor curing in the field. They observed that internal curing could compensate for water lost to drying and had, on average, a 15 percent increase in degree of hydration. Pyc et al. (2008) and Castro et al. (2011) performed mass measurements that suggest that once the pores in LWA empty while supplying water to the hydrating cement paste during internal curing, they are not subsequently resaturated, even after complete immersion of the specimen.

Often, the ITZ regions surrounding normalweight aggregates are more porous than the bulk hydrated cement paste and can provide preferential pathways for the ingress of deleterious species (Halamickova et al. 1995). The ITZ formed between LWA and its surrounding cement paste is denser and more homogeneous than the ITZ formed between a normalweight aggregate and its surrounding cement paste (Bentz 2009; Peled et al. 2010). Bentz et al. (2009) showed that the chloride penetration depth was reduced by 20 to 40 percent in a mortar with internal curing relative to that of a control mixture. Henkensiefken et al. (2009b) examined the sorption characteristics of mortars prepared with and without internal curing as shown in Fig. 9.4.7a(a). The mixtures with internal curing (11.0 percent or 23.7 percent LWA by volume) exhibited a sorption behavior similar to that of a mortar without internal curing of a significantly reduced w/c ratio (on the order of $w/c = 0.23$), as illustrated in Fig. 9.4.7a(a). Furthermore, a reduction in the electrical conductivity was measured for specimens with internal curing, even when they were vacuum-saturated prior to the electrical measurements (Fig. 9.4.7a(b)).

These benefits of internal curing are not limited to low w/c mixtures as illustrated in Fig. 9.4.7b (Castro 2011). While the results at early ages show little impact of internal curing, the total amount of absorbed water was reduced when the amount of LWA was increased for all investigated w/c (100 percent internal curing corresponds to the volume of water predicted from Eq. (9.2)). Electrical conductivity tests performed on sealed samples a year after casting also show a reduction when using internal curing.

9.5—Field experience

Internal curing has been employed in a variety of concrete mixtures for diverse applications including bridge decks, pavements, transit yards, and water tanks (Bentz and Mohr 2008). One of the first documented field studies of concrete with internal curing was a large railway transit yard in Texas (Villarreal 2008). In this application, an intermediate-sized



(a) (b)

Fig. 9.4.5c—(a) Rigid cracking frame used; and (b) effect of internal curing on restrained stress development for specimens cured under fall conditions (Byard and Schindler 2010).

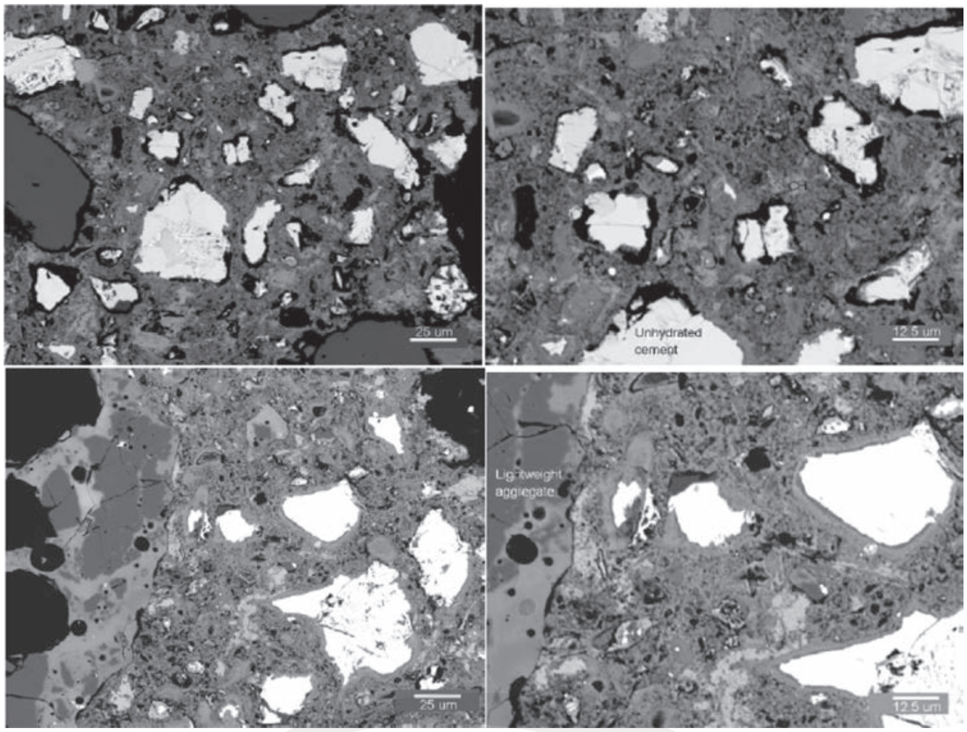


Fig. 9.4.6—BSE/SEM images of mortar microstructures for silica fume blended cement without (top) and with (bottom) internal curing at low (left) and high (right) magnifications (Bentz and Stutzman 2008). Scale bar for each image is located in lower right corner.

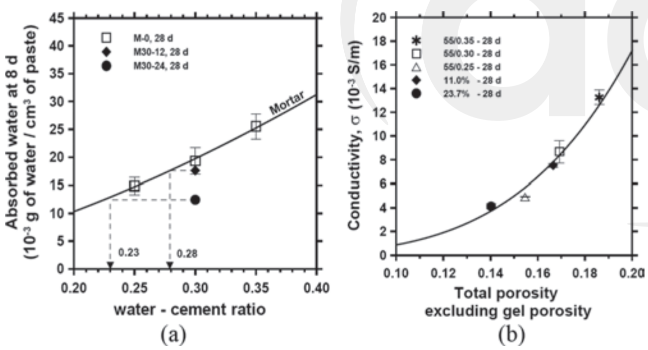


Fig. 9.4.7a—(a) Influence of internal curing on water absorption; and (b) influence of internal curing on electrical conductivity (Henkensiefken et al. 2009b).

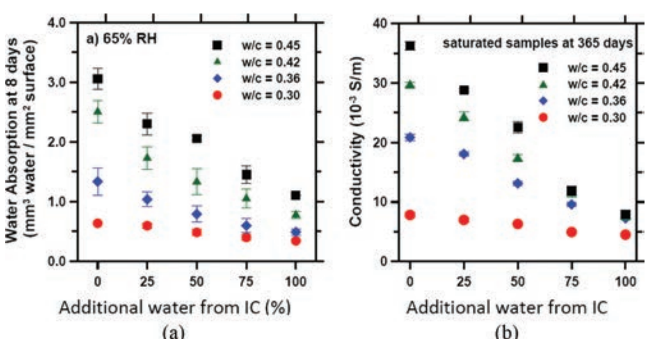


Fig. 9.4.7b—Influence of internal curing on water absorption and electrical conductivity for samples with different w/c (Castro 2011).

lightweight aggregate (LWA) was blended with normalweight aggregates to fill a gap in the overall aggregate gradation. The internal curing provided by the prewetted intermediate LWA resulted in a greater than 15 percent increase in 28-day strength, elimination of plastic and drying-shrinkage cracking, and a reduction in concrete unit weight that could translate into reductions in fuel requirements and equipment wear (Villarreal and Crocker 2007). Since 2007, several informal crack surveys have been conducted at the railway transit yard, with only two or three cracks found.

In 2006, internal curing was employed for a continuously reinforced concrete pavement placed using a slip-form paving machine (Friggle and Reeves 2008). Ten months after successful placement of the pavement, a crack survey indicated, "an overwhelming reduction in the number of cracks (21 versus 52 in a comparable section of normal concrete) and a significant reduction in the measured width of the cracks," for the test section placed using the mixture with internal curing relative to a control section placed with the Texas Department of Transportation (TxDOT) standard mixture (Friggle and Reeves 2008).

Internal curing has been used in bridges in New York, Ohio, and Indiana. In Ohio, the department of transportation employed a modified high-performance concrete (Grade No. 4) that contained 22 lb/ft^{-3} (356 kg/m^3) of cementitious materials and silica fume with a natural sand replacement with 7.5 lb/ft^3 (120 kg/m^3) of LWA. The mixture was pumped to the deck without incident. The mixture was reported to have strengths that were similar or greater than the conventional mixture without internal curing. The New York Department of Transportation (NYDOT) used internal curing on nine bridges using a special mixture design that is similar to their conventional deck design (nearly 24 lb/ft^3 [385 kg/m^3] of cementitious material with silica fume), where 7.50 lb/ft^3 (120 kg/m^3) of fine LWA are used (Streeter et al. 2011). A 2 to 10 percent increase in strength was noted between 7 and 28 days with the use of internal curing on the Court Street Bridge, a stone arch bridge in Rochester, and a 5 percent reduction in strength at 7 days with a 15 percent increase in strength at 28 days on the Bartell Road Bridge in Onondaga County (Streeter et al. 2011). In discussions with NYDOT, it was reported (Streeter et al. 2011) that there were no negatives associated with using internal curing; however, the potential benefits still needed to be quantified through comparisons with conventional concrete bridge deck materials. The NYDOT permits these concretes to be pumped and no problems have been reported. Air is typically monitored using a pressure meter.

Two bridges were cast in close proximity in Monroe Co., IN, just outside of Bloomington, in September of 2010 (Di Bella et al 2012a, b). The bridges were box girder assemblies and a 4 in. (100 mm) topping slab was made in one case for a conventional Indiana Department of Transportation (INDOT) Class C mixture. The mixtures were similar; however, approximately 15 lb/ft^{-3} (240 kg/m^3) of fine LWA was used. The LWA used in Indiana was less absorptive (approximately 10 percent by mass) than the LWA used in Ohio or New York (approximately 16 percent by mass). The benefit of casting these decks at the same time, with the same materials, using similar construction procedures is that the field performance is more directly compared. Approximately 40 days after casting, the bridges were surveyed and no cracking was observed in either deck. Higher strength and lower fluid transport properties have been measured in the internal curing concrete mixture.

9.6—Internal curing summary and potential impact on sustainability

Mixtures with internal curing show similar or improved mechanical properties, reduced risk of cracking, and the reduced chloride ingress. The additional costs of concrete with internal curing are estimated to be between 0 and 14 percent of the materials cost. Internal curing may require additional quality control and aggregate management. With time and increased familiarity with internal curing, it is expected that new opportunities will rise to use internal curing.

Internal curing is just one of many tools that might increase the sustainability of concrete elements. Internal curing has the potential to improve the durability and reduce the life-cycle costs of concrete structures. Cusson et al. (2010) compared the service lives of theoretical high-performance concrete bridge decks with and without internal curing. The high-performance concrete deck without internal curing was assumed to exhibit early-age autogenous and thermal cracking. The high-performance concrete with internal curing was assumed not to exhibit such early-age cracking and provided a further 25 percent reduction in the expected diffusion coefficient. Based on these and other assumptions,

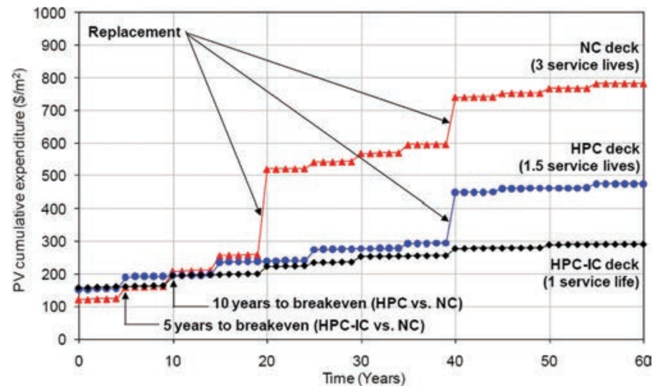


Fig. 9.6—Comparison of present value cumulative expenditures for three bridge deck alternatives (Cusson et al. 2010).

service life estimates of 22 years for conventional concrete, 40 years for high-performance concrete without internal curing, and 63 years for high-performance concrete with internal curing were reached (Fig. 9.6). In this case, internal curing should produce a bridge deck with an increased service life and a significantly reduced life cycle cost.

Recent work with the use of supplementary cementitious materials has suggested that substantially less cement clinker can be used in a mixture, resulting in a lower carbon footprint (De la Varga et al. 2011). This may also be true for mixtures with increased limestone powder replacement for cement (Bentz et al. 2009).

CHAPTER 10—REFERENCES

Committee documents are listed first by document number and year of publication followed by authored documents listed alphabetically.

American Concrete Institute

ACI 201.1R-08—Guide for Conducting a Visual Inspection of Concrete in Service

ACI 201.2R-08—Guide to Durable Concrete

ACI 209.1R-05—Report on Factors Affecting Shrinkage and Creep of Hardened Concrete

ACI 209.2R-08—Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete

ACI 211.1-91—Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete (Reapproved 2009)

ACI 211.2-98—Standard Practice for Selecting Proportions for Structural Lightweight Concrete (Reapproved 2004)

ACI 216.1-07—Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

ACI 231R-10—Report on Early-Age Cracking: Causes, Measurement and Mitigation

ACI 232.2R-03—Use of Fly Ash in Concrete

ACI 233R-03—Slag Cement in Concrete and Mortar (Reapproved 2011)

ACI 234R-06 Guide for the Use of Silica Fume in Concrete (Reapproved 2012)

ACI 302.1R-04—Guide for Concrete Floor and Slab Construction

ACI 304R-00—Guide for Measuring, Mixing, Transporting, and Placing Concrete (Reapproved 2009)

ACI 304.2R-96—Placing Concrete by Pumping Methods (Reapproved 2008)

ACI 308.1-11—Specification for Curing Concrete

ACI 309R-05—Guide to Consolidation of Concrete

ACI 318-11—Building Code Requirements for Structural Concrete and Commentary

ACI 318-08—Building Code Requirements for Structural Concrete and Commentary

ACI 318-95—Building Code Requirements for Structural Concrete and Commentary

ASTM International

ASTM C31/C31M-12—Standard Practice for Making and Curing Concrete Test Specimens in the Field

ASTM C33/C33M-13—Standard Specification for Concrete Aggregates

ASTM C78/C78M-10—Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)

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ASTM C128-12—Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate

ASTM C138/C138M-13—Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete

ASTM C143/C143M-12—Standard Test Method for Slump of Hydraulic-Cement Concrete

ASTM C150/C150M-12—Standard Specification for Portland Cement

ASTM C172/C172M-10—Standard Practice for Sampling Freshly Mixed Concrete

ASTM C173/C173M-12 Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method

ASTM C177-13—Standard Test Method for Steady-State Heat Flux Measurements and Thermal Transmission Properties by Means of the Guarded Hot-Plate Apparatus

ASTM C188-09—Standard Test Method for Density of Hydraulic Cement

ASTM C231/C231M-10—Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method

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ASTM C330/C330M-13—Standard Specification for Lightweight Aggregates for Structural Concrete

ASTM C331/C331M-10—Standard Specification for Lightweight Aggregates for Concrete Masonry Units

ASTM C332-09—Standard Specification for Lightweight Aggregates for Insulating Concrete

ASTM C469/C469M-10—Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

ASTM C494/C494M-13—Standard Specification for Chemical Admixtures for Concrete

ASTM C496/C496M-11—Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens

ASTM C512/C512M-10—Standard Test Method for Creep of Concrete in Compression

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ASTM C1017/C1017M-07—Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete

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