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State-of-the-Art Report on High-Strength, High-Durability Structural Low-Density Concrete for Applications in Severe Marine Environments

Thomas A. Holm and Theodore W. Bremner

August 2000



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State-of-the-Art Report on High-Strength, High-Durability Structural Low-Density Concrete for Applications in Severe Marine Environments

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Preface

This report was prepared for Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The study was conducted under Work Unit 33151, “Light-Weight, High-Strength Concrete for Float-In Construction,” managed at the U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, MS.

Dr. Tony C. Liu was the INP Coordinator at the Directorate of Research and Development, HQUSACE. Dr. Reed L. Mosher, ERDC, was the Laboratory Manager for the INP Program, and Dr. Stanley C. Woodson, ERDC, was the INP Program Manager.

This report was prepared by Dr. Theodore W. Bremner, University of New Brunswick, and Mr. Thomas A. Holm, Expanded Shale, Clay, and Slate Institute (ESCSI), under Contract No. DACW39-98-P-0146. The report was reviewed technically by Mr. John P. Ries, Executive Director, ESCSI; Dr. George C. Hoff, Hoff Consulting; and Dr. Basile Rabbat, Manager, Transportation Structures and Structural Codes, Portland Cement Association.

The work was monitored at ERDC by Mr. Billy D. Neeley under the general supervision of Dr. Michael J. O’Connor, Acting Director, and Dr. Bryant Mather, former Director, Structures Laboratory (SL), ERDC; and Dr. Paul F. Mlakar, Chief, Concrete and Materials Division (CMD), SL. Messrs. Neeley and Anthony A. Bombich, CMD, were the INP Principal Investigators for this work.

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At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL James S. Weller, EN, was Commander.

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1 Introduction

The need for specialized types of marine structures has challenged engineers to find novel answers to the design problems posed. The intrinsic nature of structural low-density concrete provides the design engineer with a special material to more effectively meet some of these design requirements. The purpose of this state-of-the-art report is to present information about the nature and properties of high-strength, low-density concrete so that the designer can use it effectively in a confident manner.

Prior to the adoption of the modern metric system in which different units are used for force (newtons) and mass (kilograms), it was customary to use “weight” incorrectly to mean mass, and hence, to use the expressions “lightweight aggregate” and “lightweight aggregate concrete.” In this report, however, these terms will not be used except where unavoidable as, for example, when incorporating commentary, figures, and tables from earlier publications that are cited as references. Following this logic, the word “weight” and “unit weight” will be replaced, where appropriate, with “mass” and “density.” Despite the current use of “normal weight” in many engineering journals, this report will use the expressions “normal-density aggregate” and “normal-density concrete.”

Concrete density can be reduced in a number of ways, such as incorporating low-density aggregates into the concrete, using cellular foams, high air contents, or no-fines aggregate mixtures. However, high-strength (>35 MPa (>5,080 psi)) low-density concrete can only be achieved by using structural-grade low-density aggregates. Therefore, in this report, where abbreviations are used for economy of space, it will generally not be necessary to use the letter “A” as it relates to concrete, as can be seen below:

LDA	Low-density, structural-grade aggregates
NDA	Normal-density aggregates
LDC	Low-density concrete (compressive strength 20 to 35 MPa (2,900 to 5,080 psi))
NDC	Normal-density structural concrete (compressive strength 20 to 35 MPa (2,900 to 5,080 psi))
HSLDC	High-strength low-density concrete (compressive strength >35 MPa (5,080 psi))
HSNDC	High-strength normal-density concrete (compressive strength >35 MPa (5,080 psi))

SDC	Specified-density concrete with partial or total replacement of normal-density aggregates with low-density, structural-grade aggregates
HSSDC	high-strength specified-density concrete with partial replacement of normal-density aggregates (compressive strength >35 MPa (5,080 psi))

Based upon a considerable body of research on concretes with a strength range of 20 to 35 MPa (2,900 to 5,080 psi), concretes containing structural low-density aggregates achieved full recognition as a structural material in the 1963 edition of the ACI 318 Code. Since then, in North America, LDCs with these strength levels have been commercially available for many decades, and considerable data on this strength range will be considered as baseline data. Consequently, for the purpose of this report, high strength will be defined as having a specified compressive strength (as measured on a 152 by 305 mm (6 by 12 in.) cylinder) of greater than 35 MPa (5,080 psi).

Several major structures such as long-span precast bridges and offshore oil platforms have been completed using concretes with specified densities intermediate between NDC and that presently defined as LDC in American Concrete Institute (ACI) documents 318, 213, and 301 and in American Society for Testing and Materials (ASTM) Standards C 330 and C 567. In general, these densities are achieved by replacing part or all of the ND coarse aggregates with structural-grade LD aggregates. Two examples of specified- or controlled-density concrete (CDC) are (1) a concrete density of 2,080 kg/m³ (130 lb/ft³) as was used in the Shelby Creek, Kentucky, precast, prestressed concrete bridge and (2) a concrete density of 2,170 kg/m³ (135 lb/ft³) as was placed in the gravity base structure of Mobil's Hibernia Platform (Janssen 1992, Hoff et al. 1995). This is less than the typical 2,400 kg/m³ (150 lb/ft³) associated with normal-density concrete and more than the 1,840 kg/m³ (115 lb/ft³) maximum defined as structural lightweight concrete in Chapter 2 of ACI 318 "Standard Building Code Requirements for Structural Concrete" and as also defined in the "Guide for Structural Lightweight Concrete" prepared by ACI Committee 213.

At this moment, there is virtually no reference in codes or standards for concretes of specified density. Due to the meager availability of data and design recommendations, and because of the screening process used in the aggregate selection process, the information contained in this report on CDC should be viewed as preliminary, limited to the unique combinations of aggregates used and should not be extrapolated. There is, however, a great potential for this class of concretes and, with further industry research, the ACI 318 Building Code and standards organizations will need to develop a seamless continuity of design criteria that apply to concretes of any density and with strength levels greater than 35 MPa (5,080 psi).

Several major publications were heavily relied upon, widely referenced, and served to provide baseline criteria from which this report seeks to move forward. These publications, for the most part, define present state-of-the-art criteria for commercial structural low-density concrete with compressive strengths of 20 to 35 MPa (2,900 to 5,080 psi). This body of data is then used as the basis from

which the properties and performance of high-strength, high-durability, low-density concrete presently being produced can be evaluated. The authors are grateful for the opportunity to integrate into this report the many charts, figures, tables, and references from these special publications, although limited in number. These publications include ACI 213R, ASTM STP 169C, “High-Performance Concretes and Applications” (Shah and Ahmad 1994), and the *Handbook of Structural Concrete* (Kong et al. 1983, Chap. 7).

Of particular use was ACI SP 136, which contains a three-part, 245-page report by Dr. George Hoff summarizing the research into low-density concrete by a joint industry study (Hoff 1992). This industry study was a comprehensive investigation into the physical characteristics and engineering properties of HSLDC specifically designed for severe exposure applications. The study was funded by Mobil Research and Development Corporation; Standard Oil of California; Exxon Production; Kajima Corporation; Taisei Corporation; Shimizu Construction Company; Takenaka-Komfen Company; Hazama-Guma, Ltd.; and PC Bridge Company, Ltd. Testing for this huge project was conducted by ABAM Engineers; Construction Technology Labs, Inc.; Wiss, Janney, Elstner and Association; Ben C. Gerwick, Inc.; Mitsui Engineering and Shipbuilding/Kajima Corporation; and Rensselaer Polytechnic Institute under the guidance of ABAM Engineers, Inc., who managed the study.

During the same time frame, several major investigations into the properties of HSLDC were funded and directed by Dr. Mohan Malhotra at the Concrete Technology Section, Canadian Centre for Mineral and Energy Technology (CANMET), Ottawa, Canada (Malhotra 1987). These contributions have continued to the time of the preparation of this report, with current attention focused on the performance of HSLDC exposed to the extremely intense hydrocarbon-based fires. Additionally, the performance of LD aggregate concrete containing high volumes of fly ash has been reported (Malhotra and Bremner 1996). With the completion of the Hibernia Field, Newfoundland, Mobil Oil offshore structure, Canada now has in place HSLDC structures exposed to the severe environments of the Atlantic and the Arctic.

In recent years, Europe has witnessed an outburst of research activity directed principally at the use of HSLDC in major offshore structures in the oil fields of the North Sea. In a fashion similar to North America, major joint-industry programs were funded. Of special interest was the early report by Gjerde (1982).

The authors of this report are also aware of a considerable quantity of research reports relating to the properties and applications of HSLDC authored by English, German, Japanese, and Russian authorities and only hope to provide adequate credit and reference to these papers for other investigators focusing on individual areas of research into LDC.

Low-density aggregate is produced by heating particles of shale, clay, or slate to about 1,200 °C (2,160 °F) in a rotary kiln (Holm and Bremner 1994). At this temperature the raw material bloats, forming a vesicular structure that is retained upon cooling. The individual vesicles are to various degrees not interconnected and

produce a dilation of as much as or more than 50 percent, which is retained upon cooling. This results in the particle density of the raw material changing from about 2.65 before heating to less than 1.55 after cooling.

Low-density aggregate is initially more costly than NDA, however, because of its structural efficiency, it can act in a more effective manner in concrete to achieve some desired end results. For example, these vesicular low-density manufactured aggregates have a stiffness that is similar to the stiffness of the cement paste matrix, thereby tending to create a uniform stress distribution within the concrete. Normal-density aggregates have a stiffness modulus up to 10 times that of the matrix, which results in high stress concentrations forming at the aggregate-paste interface. Normal-density concrete has a typically weak interfacial layer that is frequently the site of microcrack initiation. With low-density concrete, this weak interfacial layer is usually not present and, as a result, a lower level of microcracking is evident. Because of LDA's more favorable stress distribution, it is possible to produce a high-strength concrete with what, in comparison to a normal-density aggregate, is a relatively low-strength aggregate. The reduced stiffness of the low-density aggregates does, however, result in the stiffness of the concrete being reduced as well.

Low-density concrete was first used by the Greeks and Romans and, for their marine structures, seems to have been their material of choice. Some Roman marine structures, such as the Port of Cosa on the West coast of Italy, are still extant and serve as an index of how durable a concrete structure made with pumice and scoria aggregates can be (McCann 1987).

The currently manufactured low-density aggregates are similar to those first used for concrete ship construction some eight decades ago. Over the last 80 years, the method of manufacturing low-density aggregates using the rotary kiln has not changed significantly. The result is that the microstructure of aggregate from the first structure built using these aggregates (the *USS Selma*) is indistinguishable from aggregates made now (Bremner, Holm, and Stepanova 1994). Low-density concrete's resistance to eight decades of exposure to severe environments is well documented, and information is provided on both accelerated and long-term field exposure testing so that design professionals can use the product in a discriminating way.

Production of low-density aggregates makes use of readily available shale, clay, and slate, which do not compete with our limited supply of good-quality normal-density aggregates. The economics and potential future uses depend on how well design professionals are aware of the unique properties of low-density concrete and how it can be incorporated in imaginative ways to meet our future building needs. If the past is an indication of the future, then new and novel solutions are to be expected.

2 History

Natural Deposits of Low-Density Aggregates Used for the Port of Cosa in 273 B.C.

The origins of concrete are lost in antiquity, but whoever found a need for aggregates to make concrete and did not have access to suitable natural deposits of river gravel must have recognized that vesicular deposits of pumice and scoria were easier to reduce to size, not to mention easier to transport as compared to higher density aggregates. It seems that these early builders had also learned by 273 B.C. that porous aggregates were better suited for marine facilities than the locally available beach sand and gravel, as they went 40 km to the northeast to quarry volcanic aggregates at the Volcine complex for the harbor at Cosa (Bremner, Holm, and Stepanova 1994). This harbor is on the west coast of Italy and consists of a series of four piers (\approx 4-m cubes) extending out into the sea. For two millennia they have withstood the forces of nature with only surface abrasion and became obsolete only because of siltation of the harbor. They stand today as a testament to the wisdom of their designers whose prior experiences with marine concrete may have been limited to only several decades at the most.

Pantheon Dome

The early literature frequently mentions the collapse of domes due to improper design or the use of inappropriate materials, and all early builders must have been acutely aware of the risks involved. Thus, the materials used in domes would only be the ones in which the builder had the utmost confidence. Roman engineers during the reign of the emperor Hadrian had sufficient confidence in LDC to build a dome whose diameter of 43.3 m was not exceeded for over a millennium. The structure is in excellent condition and is still being used to this day for spiritual purposes (Bremner, Holm, and Stepanova 1994). Initially it was covered with metal, but the metal was soon stripped off to cover another structure. The domed structure stood exposed to the elements for many centuries before a lead roof was installed in recent times.

A second important aspect is that the porous aggregates were sorted so as to use the less-expanded aggregates near the base where stresses were greatest and then to use progressively more highly expanded aggregates for the upper portion of the dome where the stresses were lower (MacDonald 1976). There appears to be no

adverse effect in using even very highly expanded aggregates for this important application where both durability and strength are important.

The third factor of importance is that the dome contains intricate recesses to reduce the dead load. These recesses were formed with wooden formwork, and the imprint of the grain of the wood can be seen to this day. The excellent cast surfaces visible to the modern observer provide clear evidence that these early builders had successfully mastered the art of casting concrete made with low-density aggregates. Vitruvius took a special interest in building construction and commented on what was unusual. The fact that he did not single out LDC concrete for comment might simply imply that these early builders were fully familiar with this material (Morgan 1960).

The Origin of Manufactured Low-Density Aggregates

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, because of its distended size, does not meet the dimensional uniformity essential for a successfully fired brick. These rejected bricks were recognized by a Kansas City ceramic engineer, Mr. Stephen J. Hayde, as an ideal material for making a special concrete (Expanded Shale, Clay, and Slate Association 1971). When reduced to appropriate aggregate size and grading, these bloated bricks could be used to produce a LDC with mechanical properties similar to regular concrete. After almost a decade of experimenting with these rejected bricks, he patented in February 1918 the process of making these aggregates by heating small particles of shale, clay, or slate in a rotary kiln. A particle size was arrived at that, with limited crushing, produced an aggregate grading suitable for making a LDC.

About this time there was a great need for shipping because of the shortage of plate steel in World War I. With plate steel in short supply and with reinforcing steel in good supply as a result of curtailment of civilian construction, it appeared logical to the U.S. Emergency Fleet Building Corporation (the arm of government charged with solving this problem) to turn their attention to the success of the Scandinavian countries with concrete ships (Fougner 1922). The corporation found that, for the concrete to be effective in ship construction, concrete would need a maximum density of about $1,760 \text{ kg/m}^3$ (110 lb/ft^3) and a compressive strength of 28 MPa (4,060 psi). This high strength-to-density ratio was not possible using the various low-density volcanic aggregates available. In the summer of 1918, U.S. Naval architects learned of the work of Mr. Hayde in Kansas City, and the Corporation arranged with the National Bureau of Standards to conduct a series of tests that confirmed Hayde's findings. After this, Mr. Hayde patriotically granted free use of his patent rights to the Federal Government to produce aggregates for construction of their ships; they in turn authorized extensive research and experimental work to be conducted that enabled high-quality vessels to be produced (Expanded Shale, Clay, and Slate Association 1971).

Concrete Ships

Experience gained during 1918-22 in the design and fabrication of low-density reinforced concrete was of direct use to the civilian sector. The first commercial plant to produce low-density aggregates using a rotary kiln began operations in Kansas City, MO, in 1920 and, by 1941, there were eight licensed for operation in the United States and Canada. In 1923 the first lightweight concrete masonry units were being produced by Mr. Dan Servey. Between 1918 and 1941, the industry prospered as a result of the need for highly efficient concrete masonry units and structural concrete in high-rise buildings (Expanded Shale, Clay, and Slate Association 1971).

During the Second World War, 24 oceangoing ships and 80 seagoing barges were built. Although these vessels performed admirably during both wars, they were not economical in peacetime, as was the case of most construction of that time. Steel ships were broken up for scrap, while the destiny of the concrete ships often was to be sunk as breakwaters. This happened at Port Charles, VA, where the sunken ships have been examined and found to be in surprisingly good condition, confirming the high level of durability characteristics that can be achieved with LDC in marine exposure (Holm, Bremner, and Vaysburd 1988). Ten concrete ships, including one constructed in WWI, continue to serve as a floating log boom in Powell River, British Columbia (Bremner, Holm, and Morgan 1966).

High-Rise Construction

Low-density high-rise concrete construction became a reality when it was found that an addition of 14 stories could be added to the existing 14-story South Western Bell Telephone office building completed in 1929 in Kansas City. Without the reduction in dead load possible with LDC, only eight stories could have been added using normal-density concrete (Expanded Shale, Clay, and Slate Institute 1971).

Energy-Related Offshore Structures

In floating structures, great efficiencies are achieved when a lower density material is used. A reduction of 25 percent in mass in reinforced NDC will result in a 50-percent reduction in load when submerged. Because of this, the oil and gas industry recognized that LDC could be used to good advantage in its floating structures, as well as 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 low-density aggregate candidates deemed suitable for making HSLDC that would meet their design requirements. The work started almost two decades ago, with the results made available in 1992. As a result of this research, design information became readily available and has enabled LDC

to be used for new and novel applications where high strength and high durability are desirable (Hoff 1992).

3 Properties of Structural-Grade Low-Density Aggregate

Internal Structure of Low-Density Aggregates

Low-density aggregates have a low particle 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 which temperature gases are evolved within the pyroplastic mass, causing expansion that is retained upon cooling. Strong, durable, low-density aggregates contain a uniformly distributed system of pores that have a size range of approximately 5 to 300 μm and which are developed in a relatively crack-free, high-strength vitreous phase. Pores close to the surface are readily permeable and fill within the first few hours of exposure to moisture. Interior pores, however, fill extremely slowly, with many months of submersion necessary for saturation. A fraction of the interior pores are essentially noninterconnected and remain unfilled after years of immersion.

Particle Shape and Surface Texture

Depending on the source and the method of production, low-density aggregates exhibit considerable differences in particle shape and texture. Shapes may be cubical, rounded, angular, or irregular. Textures may range from fine pore, relatively smooth skins to highly irregular surfaces with large exposed pores. Particle shape and surface texture directly influence workability, fine to coarse aggregate ratio, cement content requirements, and water demand in concrete mixtures, as well as other physical properties.

Particle Density

The particle density of an aggregate is the ratio between the mass of the particle material and the volume occupied by the individual particles. This volume includes the pores within the particle, but does not include voids between the particles. In general, the volume of the particles is determined from the volume displaced while

submerged in water. Penetration of water into the aggregate particles during the test is limited by the aggregate's previous degree of saturation. The oven-dry density of an individual particle depends both on the density of the solid vitreous material and the pore volume within the particles, and generally increases when particle size decreases. The density of the pore-free vitreous material may be determined by pulverizing the low-density aggregate and then following procedures used for determination of the specific gravity of cement in ASTM C 188.

Bulk Density of Low-Density Aggregates

Aggregate bulk density is defined as the ratio of the mass of a given quantity of material and the total volume occupied by it. This volume includes the voids between, as well as the pores within, the particles. Bulk density is a function of particle shape, density, size, grading, and moisture content, as well as the method of packing the material (loose, vibrated, rodded) and varies not only for different materials, but for different sizes and gradings of a particular material. Table 1 summarizes the maximum densities for low-density aggregates listed in ASTM C 330, "Lightweight Aggregates for Structural Concrete" and C 331, "Lightweight Aggregates for Concrete Masonry Units."

Table 1 Bulk Density Requirements of ASTM C 330 and C 331 for Dry-Loose Low-Density Aggregates		
Aggregate Size and Group	Maximum Bulk Density	
	kg/m³	lb/ft³
ASTM C 330 and C 331		
-fine aggregate	1,120	70
-coarse aggregate	880	55
-combined fine and coarse aggregate	1,040	65

Total Porosity

Total porosity (within-particle pores and between-particle voids) can be determined from measured values of particle density and bulk density. If, for example, measurements on a sample of low-density coarse aggregate are

bulk dry loose density, 770 kg/m³ (48 lb/ft³);
 dry particle density, 1,400 kg/m³ (87 lb/ft³); and
 density of poreless vitreous material, 2,500 kg/m³ (156 lb/ft³),

then the fractional pore volume of an individual particle is

$$\frac{2,500 - 1400}{2,500} = 0.44$$

and the fractional interstitial void volume (between particles) is

$$\frac{1,400 - 770}{1,400} = 0.45$$

For this example, total porosity (pores and voids) would then equal

$$\{0.45 + [0.44 \Delta (1 - 0.55)]\} = 0.69$$

Grading

Grading requirements for low-density aggregates deviate from those of normal-weight aggregates (ASTM C 33) by requiring a larger mass of the low-density aggregates to pass through some of the finer sieve sizes. This modification in grading (ASTM C 330) recognizes the increase in density with decreasing particle size of low-density aggregates. This modification yields the same volumetric distribution of aggregates retained on a series of sieves for both low-density and normal-density aggregates.

Producers of structural low-density aggregates normally stock materials in several standard sizes such as coarse, intermediate, and fine aggregate. By combining size fractions or by replacing some or all of the fine fraction with a normal-density sand, a wide range of concrete densities can be obtained. The aggregate producer is the best source of information for the proper aggregate combinations to meet fresh concrete density specifications and equilibrium density for dead load design considerations.

Normal-density sand replacement will typically increase concrete density from about 80 to more than 160 kg/m³ (5 to 10 lb/ft³). Using increasing amounts of cement to obtain high-strength concrete may increase air dry density from 32 to 96 kg/m³ (2 to 6 lb/ft³). However, with modern concrete technology it will seldom be necessary to increase concrete cement content to obtain the reduced water-cementitious materials ratios needed to obtain higher strength, since this can be done using water-reducing admixtures or high-range water-reducing admixtures. [The water-cementitious materials ratio is referred to as W/C_m, where W is the mass of water and C_m is the mass of cementitious materials.]

Absorption Characteristics

Due to their cellular structure, low-density aggregates absorb more water than their normal-density aggregate counterparts. Based upon a 24-hr absorption test conducted in accordance with the procedures of ASTM C 127 and ASTM C 128, structural-grade low-density aggregates will absorb from 5 to more than 25 percent moisture by mass of dry aggregate. By contrast, normal-density aggregates generally absorb less than 2 percent of moisture. The important distinction in stockpile moisture content is that with low-density aggregates the moisture is largely

absorbed into the interior of the particles, whereas in normal-density aggregates it is primarily surface moisture (ASTM C 70). Recognition of this difference is essential in mixture proportioning, batching, and control. Rate of absorption of low-density aggregates is dependent on the characteristics of pore size, continuity, and distribution, particularly for those close to the surface. Internally absorbed water within the particle is not immediately available for chemical interaction with cement as mixing water, and as such, does not enter into water-cement ratio (w/c) calculations. However, it is extremely beneficial in maintaining longer periods of hydration essential to improvements in the aggregate/matrix contact zone. Internal curing will also bring about a significant reduction of permeability by extending the period in which additional products of hydration are formed in the pores and capillaries of the binder.

Modulus of Elasticity of Low-Density Aggregate Particles

The modulus of elasticity of concrete is a function of the moduli of its constituents. Concrete may be considered as a two-phase material consisting of coarse aggregate inclusions within a continuous “mortar” fraction that includes cement, water, entrained air, and fine aggregate. Dynamic measurements made on aggregates alone (Muller-Rochholz 1979) have shown a relationship corresponding to the function $E = 0.008 \psi^2$, where E is the dynamic modulus of elasticity of the particle in megapascals and ψ is the dry mean particle density in kilograms per cubic meter (Figure 1).

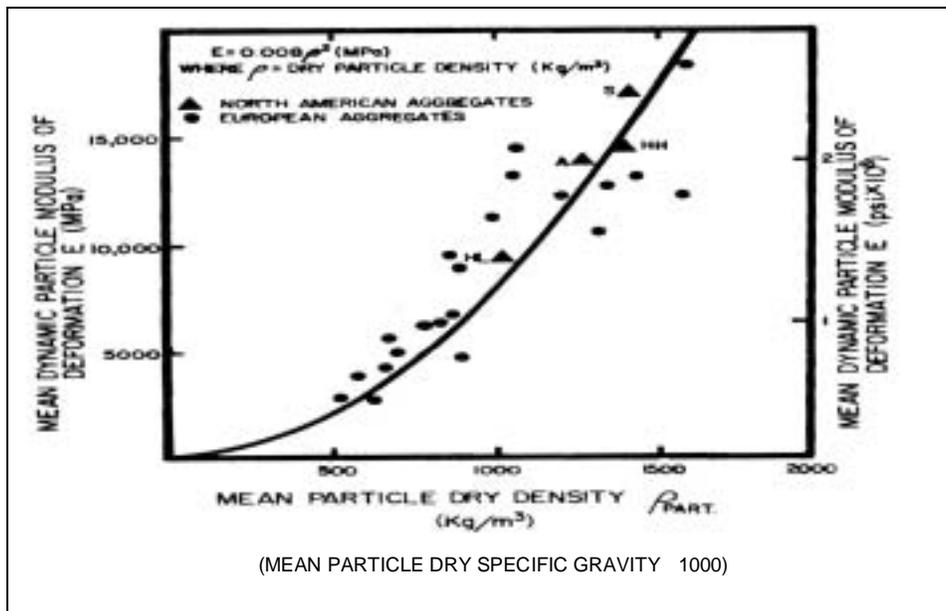


Figure 1. Relationship between mean particle density and the mean dynamic modulus of elasticity for the particles of low-density aggregates (from Bremner and Holm 1986, with permission of American Concrete Institute)

Dynamic moduli for typical expanded aggregates have a range of 10 to 16 GPa (1.45 to 2.3 $\Delta 10^6$ psi), whereas the range for strong normal weight aggregates is approximately 30 GPa (4.35 $\Delta 10^6$ psi) to 100 GPa (14.5 $\Delta 10^6$ psi).

4 Classification of Low-Density Concrete

The nature and use of low-density concrete are determined, to a large degree, by the properties of the aggregates and the cement paste. To simplify the design and construction procedure, various classifications have been derived, which will be discussed in order of increasing density. Nonstructural insulating concrete consists of a highly expanded aggregate with a cement paste matrix that is highly air entrained. This concrete is covered by ASTM C 332.

Very light nonstructural concretes, which are employed primarily for high thermal resistance, incorporate extremely low-density, low-strength aggregates such as vermiculite and perlite. With insulating concrete, the density seldom exceeds 800 kg/m^3 (50 lb/ft^3), and the thermal resistance is high. These concretes are not intended to be exposed to the weather and generally have a compressive strength ranging from about 0.69 to 3.45 MPa (100 to 500 psi).

ASTM C 332 limits thermal conductivity values for insulating concretes to a maximum of $0.22 \text{ W/m} \cdot \text{K}$ ($1.50 \text{ Btu} \cdot \text{in./h} \cdot \text{ft}^2 \cdot ^\circ\text{F}$) for concrete having an oven-dry density of 800 kg/m^3 or less, and to $0.43 \text{ W/m} \cdot \text{K}$ ($3.0 \text{ Btu} \cdot \text{in./h} \cdot \text{ft}^2 \cdot ^\circ\text{F}$) for those with densities up to $1,440 \text{ kg/m}^3$ (90 pcf). Lower density concretes are those made with Group I aggregates (perlites and vermiculite), while higher densities result from the use of Group II aggregates (expanded shales, expanded slags, and natural lightweight aggregates).

Thermal conductivity values may be determined in accordance with ASTM C 236 and ASTM C 177. Oven-dried specimens are used for both thermal conductivity and density tests on the insulating concretes. Moisture content of insulating materials directly affects both the thermal conductivity and density, but to varying degrees. A 1-percent increase in moisture content will increase density by an equivalent 1 percent but may increase thermal conductivity by as much as 5 to 9 percent (Holm 1994). Use of oven-dried specimens provides an arbitrary basis for comparison but clearly does not duplicate in-service applications. The controlled test conditions serve to permit classification of materials and to provide a standardized reference environment.

Widespread industrial applications that call for fill concretes require modest compressive strengths, with densities intermediate between the structural- and

insulating-grade concretes. These concretes may be produced in three ways: (1) high-air content mixtures with structural-grade LDA; (2) sanded low-density insulating aggregate mixtures; and (3) formulations incorporating both structural and insulating-grade LDA. Compressive strengths from 3.4 to 17 MPa (500 to 2,500 psi) are common, with thermal resistance less than that for concretes containing only structural-grade LDA. Because of its very low density, this fill concrete tends to reduce loads on supporting members and also to provide enhanced thermal-insulating properties.

With reduced air content and aggregates of particle density above 1.0, a structural/insulating concrete can be produced that has a compressive strength between 3.4 and 17 MPa (500 to 2,500 psi), which is covered by either ASTM C 330 or C 332. Table 2 summarizes the ASTM classification for low-density aggregate concrete.

Class of Low-Density Aggregate Concrete	Type of Low-Density Aggregate Used in Concrete	Typical Range of Mass of Low-Density Concrete kg/m³ (lb/ft³)	Typical Range of Compressive Strength MPa (psi)	Typical Range of Thermal Conductivities W/m · °K (Btu · in./h · ft² · °F)
Structural	Structural-grade LDA C 330	1,440 - 1,840 (90 - 115) at equilibrium	>17 (>2,500)	Not specified in C 330
Structural/Insulating	Either structural C 330 or insulating C 332 or a combination of C 330 and C 332	800 - 1,440 (50 - 90) at equilibrium	3.4 - 17 (500 - 2,500)	C 332 from 0.22 (1.50) to 0.43 (3.00) oven dry
Insulating	Insulating-grade LDA C 332	240 - 800 (15 - 50) oven dry	0.7 - 3.4 (100 - 500)	C 332 from 0.065 (0.45) to 0.22 (1.50) oven dry

Structural-Grade Low-Density Aggregate Concrete

Structural grade low-density concrete is normally considered to be of density 1,440 to 1,840 kg/m³ (90 to 115 lb/ft³) and have a strength in excess of 17 MPa (2,500 psi). This material was first used for concrete ships some 80 years ago, and its design and construction procedures are well known, as discussed in the next chapter.

Structural-grade low-density concretes generally contain aggregates made from pyroprocessed shales, clays, slates, expanded slags, expanded fly ash, and those mined from natural porous volcanic sources. Minimum compressive strength of structural-grade low-density aggregate concrete has, in effect, been jointly established by ASTM C 330 and ACI 318, which require that for structural concrete made with low-density aggregate, the air-dried density at 28 days is usually in the

range of 1,440 to 1,840 kg/m³ (90 to 115 lb/ft³). Although structural concrete with equilibrium density up to 1,920 kg/m³ (120 lb/ft³) is often used, most low-density aggregate concrete used in structures has an equilibrium density of approximately 1,800 kg/m³ (112 lb/ft³). High-strength requirements, above 35 MPa (5,080 psi), will generally limit aggregates to expanded shales, clay, slates, and pelletized sintered fly ash.

Structural-grade low-density aggregates are produced in manufacturing plants from raw materials including suitable shales, clays, slates, fly ashes, or blast-furnace slags. Naturally occurring lightweight aggregates are mined from volcanic deposits that include pumice and scoria. Pyroprocessing methods include the rotary-kiln process (a long, slowly rotating, nearly horizontal cylinder lined with refractory materials similar to cement kilns); the sintering process, wherein a bed of raw materials including fuel is carried by a traveling grate under ignition hoods; and the rapid agitation of molten slag with controlled amounts of air or water. No single description of raw material processing is all-inclusive and the reader is urged to consult local LDA for physical and mechanical properties of low-density aggregates and the concrete made with them.

ASTM C 330 requires fine low-density aggregates used in the production of structural low-density concrete to be properly graded, with a dry-loose bulk density as given in Table 1. Four coarse aggregates are provided for use in structural low-density concrete with a maximum dry-loose bulk density of 880 kg/m³ (55 lb/ft³). Combined fine and coarse aggregate formulations must not exceed a maximum dry-loose density of 1,040 kg/m³ (65 lb/ft³). Tests are conducted in accordance with ASTM C 29 using the shoveling procedure (Holm 1994).

Implicit in the definition of structural low-density concrete is the following:

- a. Specified compressive strength f_c' is equal to 20 to 35 MPa (2,900 to 5,080 psi).
- b. Virtually all cast-in-place LDC used in building construction throughout North America has had this strength range for many decades.
- c. Almost all structural-grade low-density aggregates that meet the requirements of ASTM C 330 can achieve these strength levels.
- d. ACI 318 requires low-density concrete structures exposed to freezing and thawing in a moist condition, or exposed to severe sulfate-containing solutions, to have a minimum compressive strength of 31 MPa (4,500 psi). For corrosion protection of reinforcement in concrete exposed to salt water, a minimum compressive strength of 35 MPa (5,080 psi) is required (see Table 3).
- e. Most State Department of Transportation specifications require a minimum compressive strength of 31 MPa (4,500 psi) and a maximum w/c ratio of 0.45 when using structural LDC for bridge decks.

- f. Most prestressed low-density concrete applications call for a compressive strength of 24 MPa (3,500 psi) at strand release and a minimum of 35 MPa (5,080 psi) at 28 days.

Table 3 Requirements for Special Exposure Conditions		
Exposure Condition	Maximum Water-Cementitious Materials Ratio, by Weight, Normal-Density Aggregate Concrete	Minimum f_c^{\prime} Normal-Density and Low-Density Aggregate Concrete, MPa (psi)
Concrete intended to have low permeability when exposed to water	0.50	27.5 (4,000)
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45	31 (4,500)
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources	0.40	34.5 (5,000)

High-Strength, Low-Density Concrete

In recent years, low-density concretes have achieved high strength levels (35-70 MPa, 5,080-10,160 psi) by incorporating various pozzolans (fly ash, silica fume, metakaolin, calcined clays and shales) combined with mid-range or high-range water-reducing admixtures or both. In addition, because of durability concerns, the water-to-cementitious material ratio has in many cases (i.e., for bridges) been limited to a maximum of 0.45 and, in special circumstances, even lower ratios have been specified. Limiting the mass of water used, combined with an air content on the lower end of the acceptable range, results in fresh equilibrium and oven-dry densities higher than generally recorded in past practice.

It may be argued that the first practical use of high-strength concrete took place in World War I when the American Emergency Fleet corporation built LDC ships with specified compressive strengths of 35 MPa (5,080 psi). Commercial NDC strengths of that time were approximately 14 MPa (2,030 psi).

While most structural-grade low-density aggregates are capable of producing concretes with compressive strengths in excess of 35 MPa (5,000 psi), a limited number of LDAs can be used in concretes that develop compressive strengths between 48 and 69 MPa (7,000 to 10,000 psi). Small increases in density may be necessary when developing compressive strengths approaching 69 MPa (10,000 psi) while still maintaining benefits of reduction in density. HSLDC with compressive strengths ranging from 40 MPa (5,800 psi) to 50 MPa (7,250 psi) are commercially available in some areas, and testing programs on HSLDC with ultimate strengths approaching 69 MPa (10,000 psi) are ongoing. For the purpose of this chapter, HSLDC may have a maximum equilibrium density of 2,000 kg/m³ (125 lb/ft³), as defined in ASTM C 567.

Specified-Density Concrete

Between the generally accepted maximum density (ranging from 1,850 to 1,900 kg/m³ (115 to 120 lb/ft³)) and the typically assumed normal concrete density of 2,400 kg/m³ (150 lb/ft³), any density may be specified and developed. Indeed, in one particular case, a precast concrete producer cast concrete mixtures where all components of the mixture were kept constant except that the normal-weight coarse aggregate was incrementally replaced by low-density aggregate. These replacements resulted in concrete densities of 1,840, 1,950, 2,080, and 2,210 kg/m³ (115, 122, 130, and 138 lb/ft³, respectively) up to NDC density of 2,370 kg/m³ (148 lb/ft³).

In this particular comparison between a quarried limestone and a rotary kiln-produced expanded shale, the strength levels of the concretes tested did not differ appreciably over the range of replacements, but the elastic modulus declined almost linearly with the amount of low-density coarse aggregate added. This investigation allowed the concrete producer to customize the concrete density (based upon product geometry and project span length) to ship more product or larger members per truck without exceeding the state highway loading. An example of this is the Shelby Creek bridge (Janssen 1992), which was produced using an air-dry concrete density of approximately 2,080 kg/m³ (130 lb/ft³). Depending on the characteristics of the normal- and low-density aggregate, any practical combination of strengths and densities can be achieved.

5 Engineering Properties of Structural Low-Density Concrete

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

Density

Although there are numerous structural applications of all LDC with LD coarse and LD fine aggregate, usual commercial practice in North America is to design sanded LDC where part or all of the fine aggregates used is natural sand. Long-span bridges using concretes with three-way blends (coarse and fine low-density aggregates and small supplemental natural sand volumes) have provided long-term durability and structural efficiency (density/strength ratios) (Holm and Bremner 1990). Earlier research reports (Kluge, Sparks, and Tuma 1949; Price and Cordon 1949; Reichard 1964; Shideler 1957) compared all LDC with “reference” normal-weight concrete, while later studies (Hanson 1964, Pfeiffer 1967) supplemented the early findings with data based upon sanded LDC.

The fresh density of expanded aggregate concretes is a function of mixture proportions, air contents, water demand, particle density, and moisture content of the LDA. Decrease in 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. Design professionals should specify a maximum fresh density for low-density concrete, as limits of acceptability should be controlled at time of placement.

Despite the ACI 213 definition of structural-grade low-density concrete that has a dry density ranging between 1,440 and 1,850 kg/m³ (90 to 115 lb/ft³), the report also adds that “it should be understood that this definition is not a specification. Job specifications may, at times, allow density up to 1,900 kg/m³ (120 lb/ft³). In the majority of applications in North America, HSLDC has been associated with

equilibrium densities of about $1,850 \text{ kg/m}^3$ (115 lb/ft^3) and, in some cases, as much as $1,900 \text{ kg/m}^3$ (120 lb/ft^3).

Equilibrium Density

Self loads used for design should be based upon equilibrium density that, for most conditions and members, may be assumed to be approached after 90 days. Extensive tests conducted during North American durability studies demonstrated that, despite wide initial variations of aggregate moisture content, equilibrium density was found to be 50 kg/m^3 (3.1 lb/ft^3) above oven-dry density (Figure 2). European recommendations for in-service density are similar (FIP 1983).

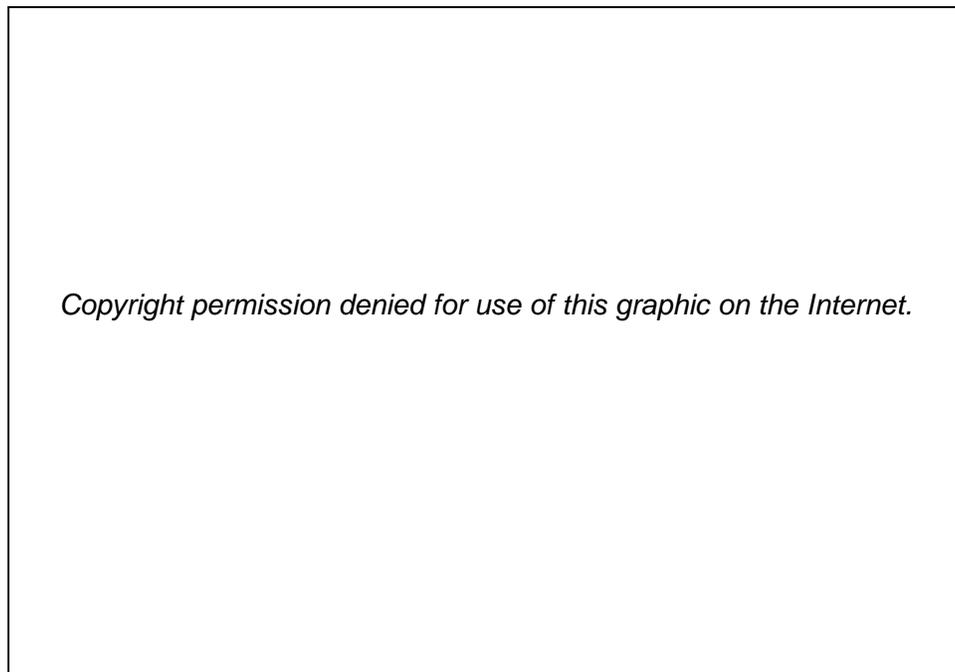


Figure 2. Concrete density versus time of drying for structural lightweight

When mass and moisture contents of all the constituents of the batch of concrete are known, an approximate calculated equilibrium density can be determined according to ASTM C 567 from the following equation:

$$O = (M_{df} + M_{dc} + 1.2 M_{ct} / V)$$

$$E = O + 50 \text{ kg/m}^3 \quad (E = O + 3 \text{ lb/ft}^3) \quad (1)$$

where

O = calculated oven-dry density, kg/m^3 (pcf)

M_{df} = mass of dry fine aggregate in batch, kg (lb)

M_{dc} = mass of dry coarse aggregate in batch, kg (lb)

1.2 = factor to account for water of hydration

M_{ct} = mass of cement in batch, kg (lb)

V = volume of concrete produced by the batch, m^3 (ft^3)

E = calculated equilibrium density, kg/m^3 (pcf)

Compressive Strength

Principles of elastic compatibility of a particulate composite

A particulate composite is by its very definition heterogeneous, and concrete is perhaps the most heterogeneous of composites—with size of inclusions varying from large aggregate down to unhydrated cement grains, and containing voids the size of entrained and entrapped air bubbles down to the gel pores in the cement paste. The general understanding of concrete as a particulate composite previously used in the analysis of regular-strength LDC may be extended to HSLDC (Bremner and Holm 1986).

Concrete can be considered as a two-phase composite composed of coarse aggregate particles enveloped in a continuous mortar matrix. This latter phase includes all the other concrete constituents, including fine aggregate, mineral admixtures, cement, water, and voids from all sources. This division, schematically shown in Figure 3, is visible to the naked eye and may be used to explain important aspects of the strength and durability of concrete.

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

Natural aggregates have an extremely wide range of elastic moduli resulting from large differences of mineralogy, porosity, flaws, laminations, grain size, and bonding. It is not uncommon for a fine-grained diabase rock to have an elastic modulus greater than 90 GPa ($13 \Delta 10^6$ psi) while poorly bonded, highly porous

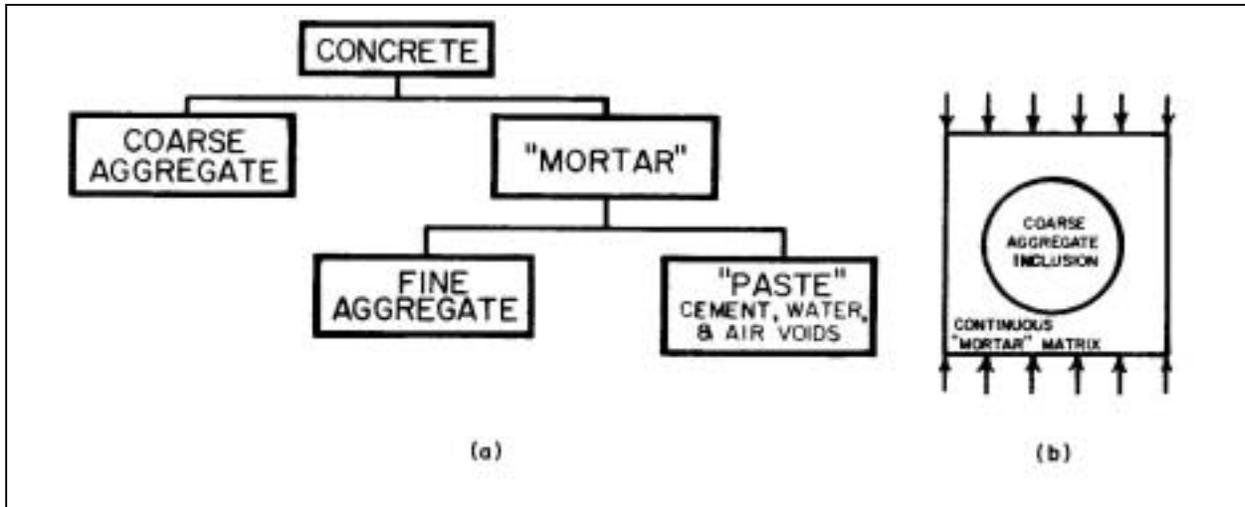


Figure 3. Cement, water, air voids, and fine aggregate combine in (a) to form the continuous mortar matrix that surrounds the coarse aggregate inclusion in (b) to produce concrete (from Bremner and Holm 1986, with permission of ACI)

natural aggregates have been known to have values lower than 20 GPa ($3 \Delta 10^6$ psi). Aggregate description by name of rock is insufficiently precise, as demonstrated in one rock mechanics text that reported a range of elastic modulus of 20 to 69 GPa (3 to $10 \Delta 10^6$ psi) for one rock type (Stagg and Zienkiewicz 1968).

Figure 4 is adapted from Stagg and Zienkiewicz (1968) and illustrates compressive strength and stiffness characteristics reported for several rock types and compares these wide ranges with the modulus of elasticity of concrete as calculated by the equation of ACI 318 Code:

$$E_c | 0.043 \varpi^{1.5} \sqrt{f'_c} \quad (\varpi = \text{density in kg/m}^3 \text{ and } f'_c \text{ in MPa}), \text{ or}$$

$$E_c | 33 \varpi^{1.5} \sqrt{f'_c} \quad (\varpi = \text{density in lb/ft}^3 \text{ and } f'_c \text{ in psi})$$

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

matrix w/c ratio may lead to faulty estimates of compressive strength and even greater misunderstanding of concrete's behavior from durability, permeability, and tensile-type loading conditions (Bremner and Holm 1986).

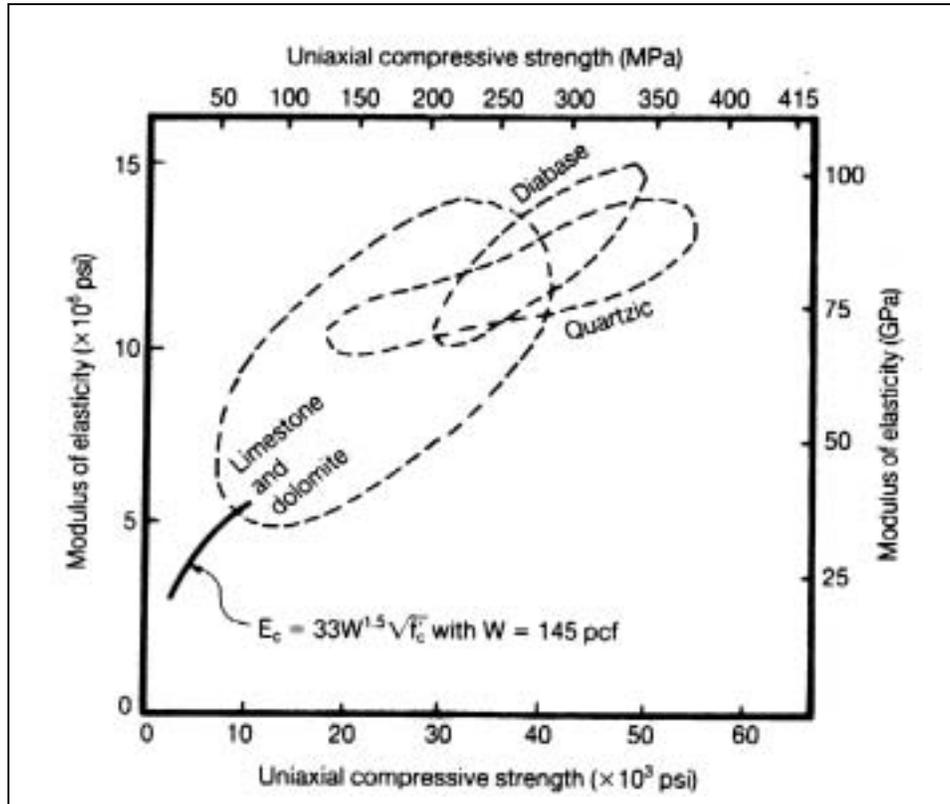


Figure 4. Range of stiffness of concrete caused by variability in the stiffness of the aggregate (after Stagg and Zienkiewicz 1968)

Obviously, the characteristics of the NDA will have a major effect on elastic compatibility. The interaction between the absolute volume percentage of coarse aggregate (± 35 percent) and the mortar phase (± 65 percent) will result in a concrete with a modulus intermediate between the two fractions. At typical commercial-strength levels, the elastic mismatch within LDC is considerably reduced due to the limited range of elastic properties of typical LDA particles.

Elastic matching of components of high-strength, low-density concrete (HSLDC)

Muller-Rochholz (1979) measured the elastic modulus of individual particles of LDA and NDA using ultrasonic pulse-velocity techniques. This report concluded that the modulus of elasticity of structural-grade LDA exceeded values of the cementitious paste fraction. This suggested that instances in which LDC strength exceeded that of companion NDC at equal binder content were understandable in light of the relative stress homogeneity.

made with high-quality LDA. Matching of the elastic properties of ordinary concrete using a high-modulus NDA such as a diabase will be possible only with the ultrahigh-quality matrix fractions incorporating superplasticizers high-range water-reducing (HRWR) admixtures and supplementary cementitious materials.

Air entrainment in concrete significantly reduces the stiffness of the mortar fraction and, as shown in Figure 5, results in a convergence of elastic properties of the two phases of sanded structural LDC while increasing the degree of elastic mismatch in normal-density concrete. This fact, combined with the slight reduction in mixing water caused by air entrainment, explains why the strength penalty caused by air entrainment is often less significant for LDC than for concretes using highly rigid NDA.

Elastic mismatching of components of HSLDC

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

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

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

Effect of curing conditions and age on the compressive strength of HSLDC

Hoff (1992), in summarizing the compressive strength results of the investigation of high-strength, low-density aggregate concrete for arctic applications, concluded that:

- a.* With the exception of HSLDC, the 28-day continually moist-cured strength is less than that obtained with 14-day moist curing followed by 14 days of air curing. This is fairly common behavior.
- b.* The very low W/C_m ratios used (0.26 to 0.31 by mass) do not provide much excess moisture to promote hydration once moist curing has ended. With the exception of the fly ash mixture, the increase in strength from 28 to

90 days for those specimens moist-cured for 14 days, is only 1 to 4 percent. For those specimens moist-cured 28 days, the strength change varies from a slight loss to a 10-percent gain. The mixtures containing fly ash show higher percentage gains with time due to the delayed reactivity of the fly ash.

- c. All of the mixtures obtained 86 to 92 percent of their 28-day strength at 7 days age.
- d. The target-specified design strengths, f_c^R (at 90 days age), of 48 and 62 MPa (7,000 and 9,000 psi) were exceeded by an amount sufficient to satisfy the statistical requirements of ACI 214 and ACI 318.
- e. The initial curing at elevated temperatures (steam curing) improves the 7-day strength but appears to cause a reduction in the 28-day strengths when compared to concretes not subjected to these high temperatures.”

Maximum strength ceiling

At a point termed the “strength ceiling,” there will be very little increase in compressive or tensile strength, despite improvements in binder quality (W/C_m) or increasing cementitious content. At this point, the strength of the coarse aggregate particle or the quality of the transition zone will determine the limiting strength. After reaching the strength ceiling, NDC will demonstrate a small positive slope for the strength/binder relationship, while for a strong LDC, the slope will be significantly less. In concretes containing highly expanded LDA, there will be essentially no increase in strength. Figure 6 demonstrates that the compressive strength ceiling for the particular 19.0-mm (3/4-in.) maximum size LDA tested was somewhat more than 55 MPa (8,000 psi) at an age of 75 days (Holm 1980a). When the maximum size of this aggregate was reduced to 9.5 mm, (3/8 in.), the strength ceiling significantly increased to more than 69 MPa (10,000 psi). Mixtures incorporating fly ash demonstrated higher strength ceilings at later ages than mixtures without fly ash.

Analyzing strength as a function of the quantity of cementitious binder, however (as shown in Figure 7), reveals that mixtures incorporating binder quantities exceeding an optimum volume are not cost effective (Holm and Bremner 1994). The schematic curves shown are for illustrative purposes only. In some areas it is not unusual to observe an overlap of strength/binder relationship when concretes containing a strong LDA are compared with concretes containing a midrange NDA.

Strength ceilings of LDA produced from differing quarries and plants will vary considerably. This variation is due to structural characteristics of the pore system developed during the firing process. The aggregate producer's goal is to manufacture a high-quality structural-grade LDA that has well-distributed pores of moderate size (5 to 300 μm) surrounded by a strong, relatively crack-free vitreous ceramic. The size, shape, and distribution of the vesicular pores will determine the

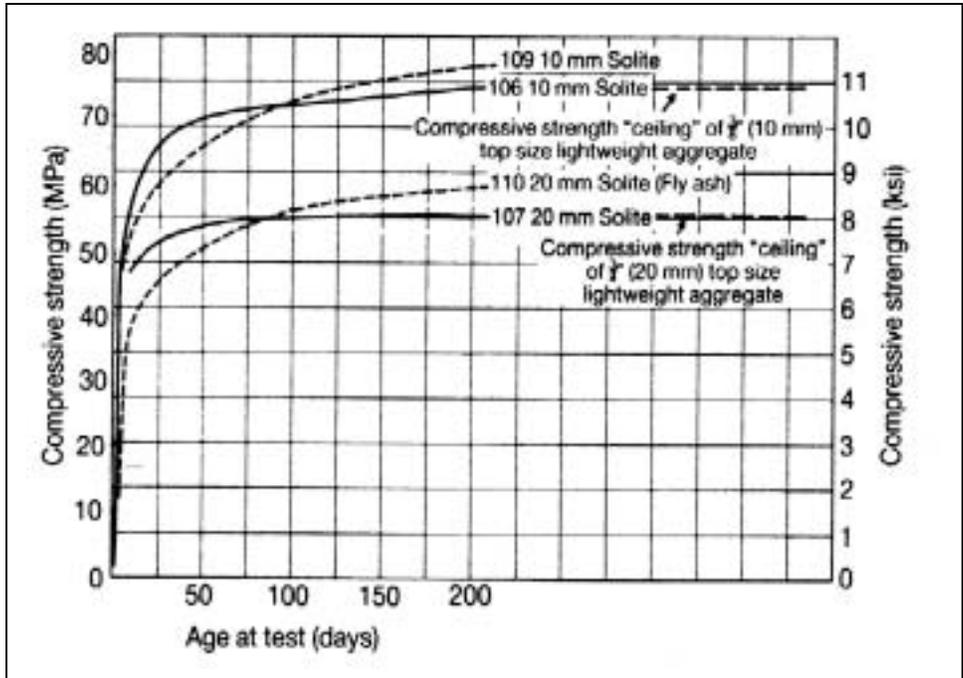


Figure 6. Compressive strength versus age of low-density concrete (from Holm 1980a, with permission of ACI)

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Figure 7. Strength versus concrete binder content

particle strength (compressive and tensile) of different sources of LDA. In general, greater pore volume will correlate with lower strength; however, there are exceptions to this relationship. The scanning electron micrograph in Figure 8 demonstrates a well-developed pore distribution system of a concrete sample cored from a highly exposed 30-year-old bridge deck (Holm 1983).

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

Contact zone

Micrographs of concretes obtained from mature structural LDC ships, marine structures, and bridges have consistently revealed minimal microcracking and a limited volume of unhydrated cement grains. The boundary between the cementitious matrix and coarse aggregates is essentially indistinguishable at the contact zone separating the two phases in all mature HSLDCs. The contact zone in LDC is enhanced by several factors, including pozzolanic reactivity of the surface of the lightweight aggregate developed during high-temperature production and the aggregate's surface roughness. With LDA there is an opportunity for the two-phase porous system to reach moisture equilibrium without developing the water-gain (bleeding) lenses that are frequently observed under and on the sides of NDA (Bremner, Holm, and deSouza 1984).

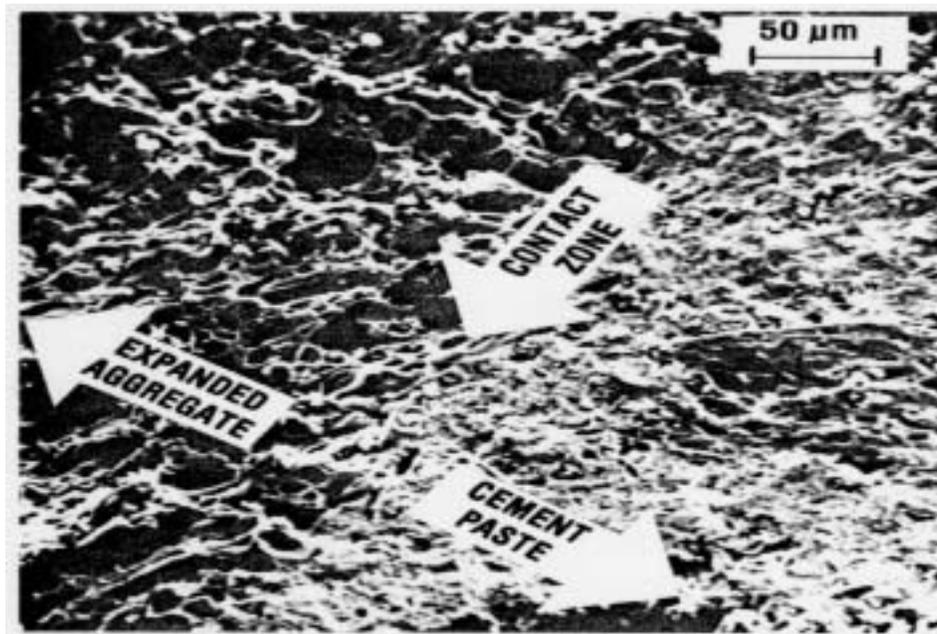


Figure 8. Contact zone of structural lightweight concrete (W. P. Lane Memorial Bridge over Chesapeake Bay, Annapolis, MD - constructed in 1952) (Holm 1983)

Shear and Tensile Strength

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

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

Shear and tensile strength of LDC

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

Tensile strength of HSLDC

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

hygral-equilibrium are significantly delayed and diminished in high binder content HSLDC.

At strength levels of 20 to 35 MPa (2,400 to 5,080 psi), the relatively similar tensile strength and elastic rigidity of the two components (LDA and matrix) will minimize stress concentrations and microcracking. At higher strengths, however, strong ND coarse aggregates will remain intact after matrix failure and provide a measure of post-elastic strain capacity and a greater resistance to splitting. Because of the lower post-elastic capacity of LDC, it appears prudent to limit the maximum strength levels for which the ACI 318 requirements govern shear, tension, torsion, development lengths, and seismic parameters to concrete compressive strengths no greater than 35 MPa (5,080 psi) unless compressive testing programs conducted on concretes containing specific combinations of aggregates demonstrate adequate performance at higher strength levels.

Tensile strength of HSSDC

Hoff et al. (1995) reported that the tensile splitting strength of the specified density concrete used in the Hibernia offshore platform slightly exceeded results for the NDC. This behavior is unusual and should not be anticipated with other aggregates.

In general, the splitting ratio as defined by $\sqrt{\frac{f_{ct}^2}{f_{ct}}}$ will be reduced as compressive strengths are increased. This is certainly true when the concrete is air-dried. However, Hoff (1992) found that, for moist-cured concrete, ACI 318 can apparently be used to predict the splitting tensile strength of HSLDC. The splitting tensile strength was generally 4 to 5 percent of the compressive strength when dry-cure regimes were used. For continuously moist-cured concrete, the range was generally 6 to 7 percent.

Modulus of rupture of low-density concrete

According to the ACI Code (ACI 318), the modulus of rupture is also related to the splitting ratio approach. Hoff (1992) reported the following for the high-strength, LDC tested in the Arctic Concrete investigations:

For moist-cured concrete, ACI 318 can apparently be used to predict the modulus of rupture of high-strength lightweight aggregate concrete. The modulus of rupture was generally less than 4 percent of the compressive strength when dry-cure regimes were used. For continuously moist cured concrete, the range was generally 9 to 11 percent. When compared to the splitting tensile strengths, the modulus of rupture was only 60 to 70 percent of the splitting tensile strength when the specimens were dry-cured. When moist cured, the modulus of rupture was generally 50 percent greater than the splitting tensile strength.

One must, however, be cautious about extending these conclusions, as they were based upon particular low-density aggregates that were chosen as superior from a large group of materials tested.

Modulus of Elasticity

The modulus of elasticity of concrete is a function of the modulus of each constituent (binder matrix, low- and normal-density aggregates) and their relative proportions in the mixture. The elastic modulus of NDC is higher because the moduli of the NDA particles (and parent rock formations) are greater than the moduli of LDA particles. For practical design conditions, the modulus of elasticity of concretes with densities between 1,440 and 2,500 kg/m³ (90 to 155 lb/ft³) and within strength ranges up to 35 MPa (5,000 psi) can be represented by the following formula (Pauw 1960):

$$E = 0.04 \bar{\omega}^{1.5} \sqrt{f_c'} \quad (2)$$

or

$$(\text{in non-SI}) \quad E = 33 \bar{\omega}^{1.5} \sqrt{f_c'}$$

where

E = denotes the secant modulus in MPa (psi)

$\bar{\omega}$ = density in kg/m³ (lb/ft³)

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

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

Structural low-density concrete

In general, structural-grade rotary kiln-produced LDAs have a comparable chemical composition and are manufactured under a similar temperature regime. They achieve low density by formation of a porous structure in which the pores are essentially spherical and enveloped in a vitreous matrix. It would be expected that, with such similarities, the variability in stiffness of the aggregate would be principally due to the density as determined by the pore volume system.

As with NDC, increasing matrix stiffness is directly related to matrix strength which, in turn, affects concrete strength. When large percentages of cementitious materials are used, the LDC strength ceiling may be reached, causing Equation 2 to overestimate the stiffness of the concrete. One factor affecting stiffness of normal-density concrete is the variation of aggregate modulus of elasticity within a particular density range. At the same specific gravity, LaRue (1946) found that the modulus of elasticity of natural aggregates could vary by a factor of as much as 3.

High-strength low-density concrete

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

$$E = C \bar{\omega}^{1.5} \sqrt{f_c \bar{R}} \quad (3)$$

where

$$\begin{aligned} C &= 0.040 \text{ for } 35 \text{ MPa} \quad (31 \text{ for } 5,000 \text{ psi}) \\ &= 0.038 \text{ for } 41 \text{ MPa} \quad (29 \text{ for } 6,000 \text{ psi}) \\ \bar{\omega} &= \text{density (kg/m}^3 \text{ or lb/ft}^3\text{)} \\ f_c \bar{R} &= \text{compressive strength (MPa or psi)} \end{aligned}$$

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

Poisson's Ratio

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

Maximum Strain Capacity

Several methods of determining the complete stress-strain curve of LDC have been attempted. At Lehigh University, the concrete cylinders were loaded by a beam in flexure (Figure 9). The approach at the University of Illinois, however, was

to load a concrete cylinder completely enclosed within a steel tube of suitable elastic properties (Wang, Shah, and Naamen 1978).

Despite formidable testing difficulties, both methods secured meaningful data; one of the more complete stress-strain curves obtained by loading the concrete through a properly proportioned beam in flexure is demonstrated in Figure 9 (Holm 1980b).

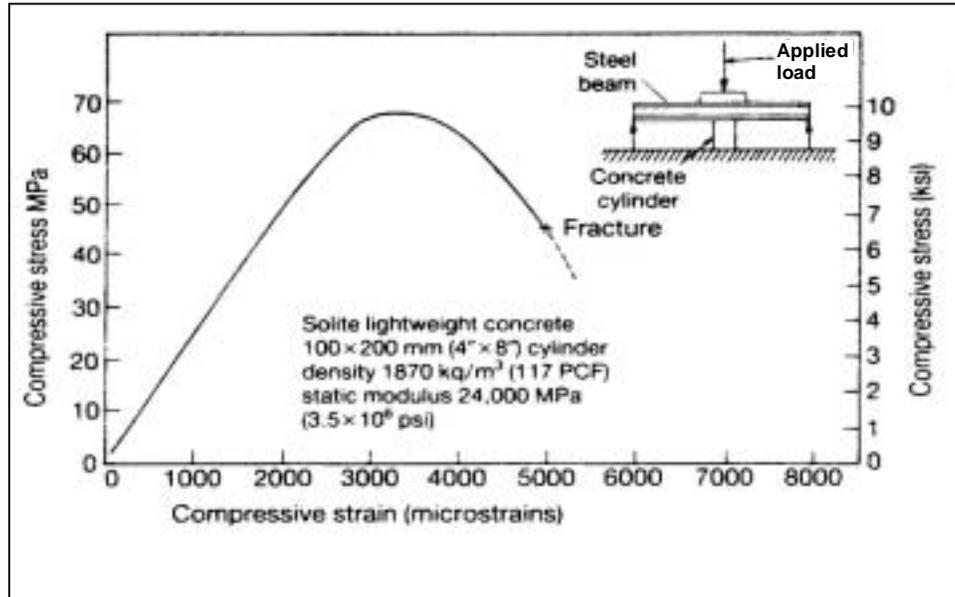


Figure 9. Stress versus strain under uniaxial compression (after Holm 1980b)

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

Abrasion Resistance

Abrasion resistance of concrete depends on the strength, hardness, and toughness characteristics of the cement paste and the aggregates, as well as on the bond between these two phases. Most LDAs suitable for structural concretes are composed of solidified glassy material comparable to quartz on the Moh scale of hardness. Structural LDC bridge decks that have been subjected to more than 100 million vehicle crossings, including truck traffic, show wearing performance similar to that of normal-density concretes. However, due to its porous system, the net resistance to impact forces is less than that of a solid particle of most NDA.

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) reports that specifically developed testing procedures that measured ice abrasion of concrete exposed to arctic conditions demonstrated essentially similar performance for LDC and NDC. However, LDC would not be appropriate for use in extreme applications, such as dam spillways, nor would it be economical for such applications.

Shrinkage

As with normal-density concretes, shrinkage of structural low-density concrete is principally determined by

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

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

Structural low-density concrete

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

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

High-strength low-density concrete

Figure 10 demonstrates the shape and ultimate shrinkage strains from one extensive testing program that incorporated both HSLDC and HSND (Holm 1980a). Shrinkage of the 9.5-mm maximum-size HSLDC mixture lagged behind early values of the HSND mixtures, equaled them at 90 to 130 days, and reached an ultimate value at 1 year, approximately 14 percent higher than the reference HSND. Shrinkage values of mixtures incorporating cement containing interground fly ash averaged somewhat greater than their high-strength non-fly ash counterparts.

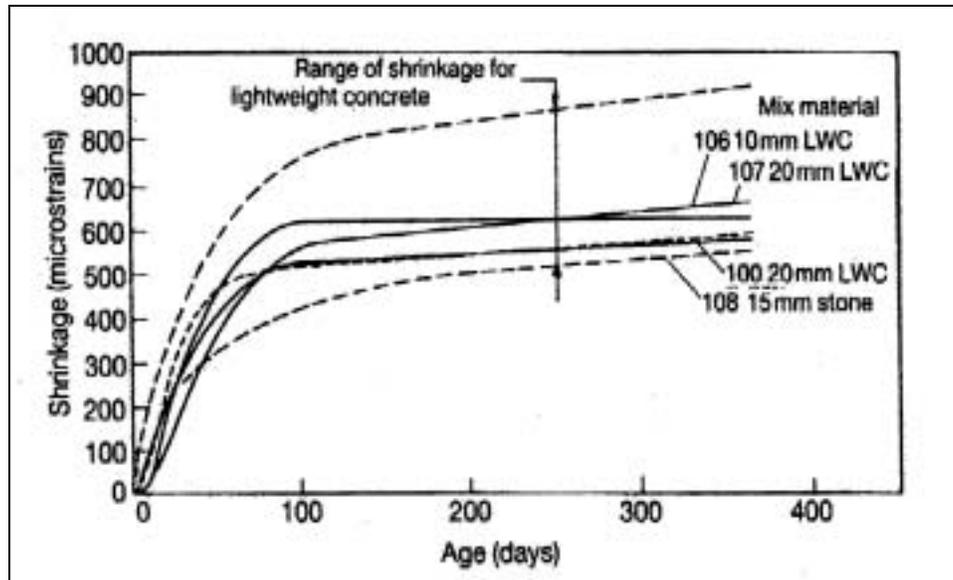


Figure 10. Shrinkage of high-strength, low- and normal-density concrete (after Holm 1980a, with permission of ACI)

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

Shrinkage measured on prisms exposed to similar curing conditions (77 days moist) reported by Hoff (1992) were of similar shape and magnitude to the LDC shown in Figure 10 reported by Holm (1980a). Specimens cured with 1 day of steam and 6 days moist curing prior to exposure in laboratory air had shrinkage strains approximately 20 percent less than the standard 7-day moist-cured specimens.

Creep

Time-related increases in concrete strain due to sustained stress can be measured according to procedures of ASTM C 512. Creep and shrinkage characteristics on any type of concrete are principally influenced by aggregate characteristics, water and cement content (paste volume fraction), age at time of loading, type of curing, and applied stress-to-strength ratio. Other second-level variables influence creep and shrinkage, but to a lesser degree. As creep and shrinkage strains will cause increase in long-time deflections, loss of prestress, reduction in stress concentration, and changes in camber, it is essential for design engineers to have an accurate assessment of these time-related characteristics as a necessary design input.

Structural low-density concrete

As shown in Figure 11, ACI 213R provides wide envelopes of 1-year specific creep values for low-density, normally cured concretes. Test results for higher strength, steam-cured, sanded LDC have a range of values that narrows significantly and closely envelopes the performance of the normal-density reference concrete. These values are principally based upon the results of the comprehensive testing program of Shideler (1957). Long-term investigations by Troxell, Raphael, and Davis (1958) on NDC report similar wide envelopes of results for differing natural aggregate types. Therefore, comparisons with reference concretes should be based upon data specific to the concretes considered.

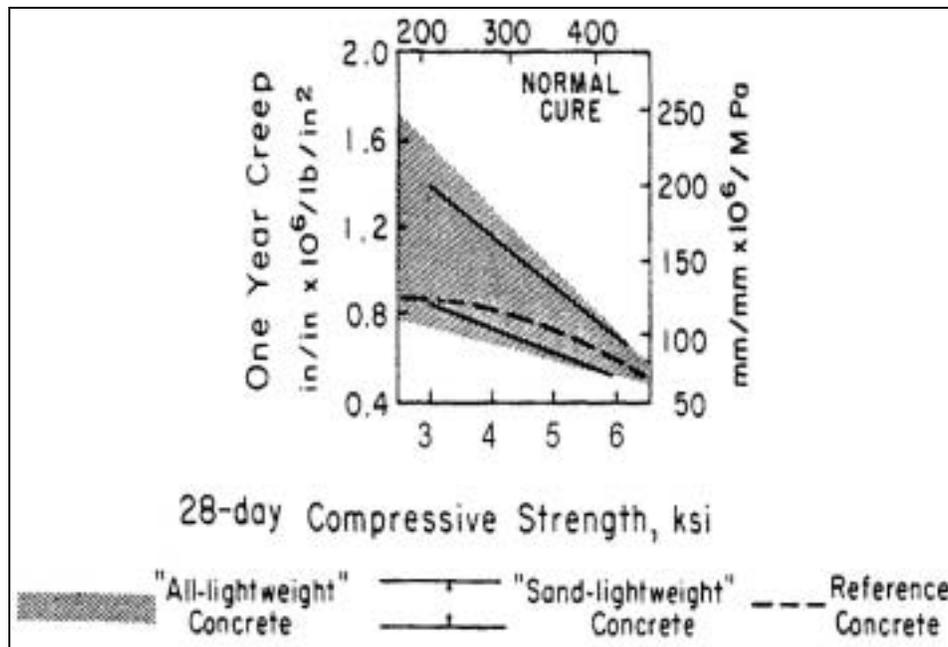


Figure 11. Creep of normally cured concrete (from ACI 213-87, with permission of ACI)

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

High-strength low-density concrete

Rogers (1957) reported that the 1-year creep strains measured on several North Carolina and Virginia LDCs were similar to those measured on companion NDC. Greater creep strains were reported by Reichard (1964) and Shideler (1957) on HSLDC containing both fine and coarse LDA, compared with reference HSNDC. These higher creep strains could be anticipated due to the significantly larger cement paste matrix volume required because of the angular particle shape of the LDA fines used in those testing programs.

The *Prestressed Concrete Institute Design Handbook* (1992) recommends a higher value of creep strain and an equal value of shrinkage when comparing LDC to NDC. It provides recommendations for increasing prestress losses when using LDC [207 to 379 MPa (30,000 to 55,000 psi)] compared with a range of 172 to 345 MPa (25,000 to 40,000 psi) for normal-density concrete. However, it may be advisable to obtain accurate design coefficients for long-span HSLDC structures by conducting prebid laboratory tests in accordance with the procedures of ASTM C 512.

Bond Strength and Development Length

Field performance has demonstrated satisfactory performance LDC with strength levels of 20 to 35 MPa (2,900 to 5,080 psi) with respect to bond and development length. Because of the lower particle strength, LDC have lower bond-splitting capacities and a lower post-elastic strain capacity than NDC. Usual North American design practice (ACI 318) is to require longer embedment lengths for reinforcement in LDC than for NDC. Unless tensile splitting strengths are specified, ACI 318 requires the development lengths for low-density concrete to be increased by a factor of 1.3 over the lengths required for normal-density concrete. With closely spaced and larger diameter prestressing strands that can cause high splitting forces, this increase may no longer be conservative. A conservative design approach or a preproject testing program may be advisable for special structures, short-span decks, or combinations of highly reinforced thin members using high-strength, low-density concrete. Additional research on development length requirements for prestressing strands in HSLDC and SDC is clearly warranted (Lane 1998).

Thermal Expansion

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

Low-density concrete

ACI 213R (1987) indicates a value of a 7 to 11 $\Delta 10^{-6}/^{\circ}\text{C}$ (4 to 6 $\Delta 10^{-6}/^{\circ}\text{F}$) depending on the volume and type of aggregates used.

High-strength low-density concrete

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

- a. 0 percent relative humidity, oven-dried to a constant mass of $105 \pm 2.8^{\circ}\text{C}$ ($221 \pm 5^{\circ}\text{F}$).
- b. 50 percent relative humidity, 50 ± 5 percent RH at $22.8 \pm 1.7^{\circ}\text{C}$ ($73 \pm 3^{\circ}\text{F}$).
- c. 100 percent relative humidity, submerged at a temperature of $22.8 \pm 1.7^{\circ}\text{C}$ ($73 \pm 3^{\circ}\text{F}$).

The results of this testing program are summarized in Table 4.

Hoff (1992) concluded that

In marine applications, lightweight aggregate concrete would most certainly have moisture contents between 50 percent and 100 percent. The data from this study suggest that at these moisture contents, the coefficients of thermal expansion for high-strength lightweight aggregate concretes containing supplementary cementing materials (silica fume fly ash, slag) will range from 7 to 13 microstrain / $^{\circ}\text{C}$ (4 to 7 microstrain / $^{\circ}\text{F}$).

Table 4 Coefficient of Thermal Expansion of High-Strength Low-Density Concrete			
Mixture	Moisture Condition, Relative Humidity, %	Specimen Size, mm (in.)	Coefficient of Thermal Expansion Between 21 °F (70 °C) and -30 °F (-22 °C) [microstrain /°C (°F)]
LWC1	100	13 Δ 76 (0.5 Δ 3)	6.1 (3.4)
LWC1	50	13 Δ 76 (0.5 Δ 3)	7.7 (4.3)
LWC1	0	13 Δ 76 (0.5 Δ 3)	6.3 (3.5)
LWC1	50	152 Δ 305 (0.5 Δ 3)	7.4 (4.1)
LWC3	100	152 Δ 305 (6 Δ 12)	12.8 (7.1)
LWC3	50	52 Δ 305 (6 Δ 12)	11.0 (6.1)
LWC3	0	52 Δ 305 (6 Δ 12)	5.8 (3.2)
LWC4	100	152 Δ 305 (6 Δ 12)	9.0 (5.0)
LWC4	50	152 Δ 305 (6 Δ 12)	8.1 (4.5)
LWC4	0	152 Δ 305 (6 Δ 12)	7.0 (3.9)
HSLWC	100	152 Δ 305 (6 Δ 12)	12.8 (7.1)
HSLWC	50	152 Δ 305 (6 Δ 12)	7.0 (3.9)
HSLWC	0	152 Δ 305 (6 Δ 12)	7.0 (3.9)

High-strength specified-density concrete

The authors are unaware of any reports of the measurement of the coefficients of thermal expansion of specified-density concrete, but would expect the coefficient to be intermediate to that of LD and NDC, and as mentioned earlier, would be highly dependent on the coefficients of the various aggregates used.

Specific Heat

The definition of specific heat is “the ratio of the amount of heat required to raise a unit mass of material 1 deg to the amount of heat required to raise an equal mass of water 1 deg.” In systems of units in which the heat capacity of water is 1.0 (either cal/g °C or Btu/lb · °F), the specific heat values are the same. In SI units, specific heat is expressed in Joules per kilogram kelvin, which can be obtained from customary values by multiplying by $4.1868 \Delta 10^3$. Tests for specific heat are generally carried out according to the procedures specified in the U.S. Army Corps of Engineers Test Method CRD-C124 (USACE 1998c).

Hoff (1992) reported specific heat results of 974 and 994 J/(kg °K) (0.233 and 0.238 Btu/lb · °F) for high-strength concretes with densities of 1,922 and 2,051 kg/m³ (120 and 128 lb/ft³), respectively. As shown in Figure 12, these results are comparable to other concretes of similar densities.

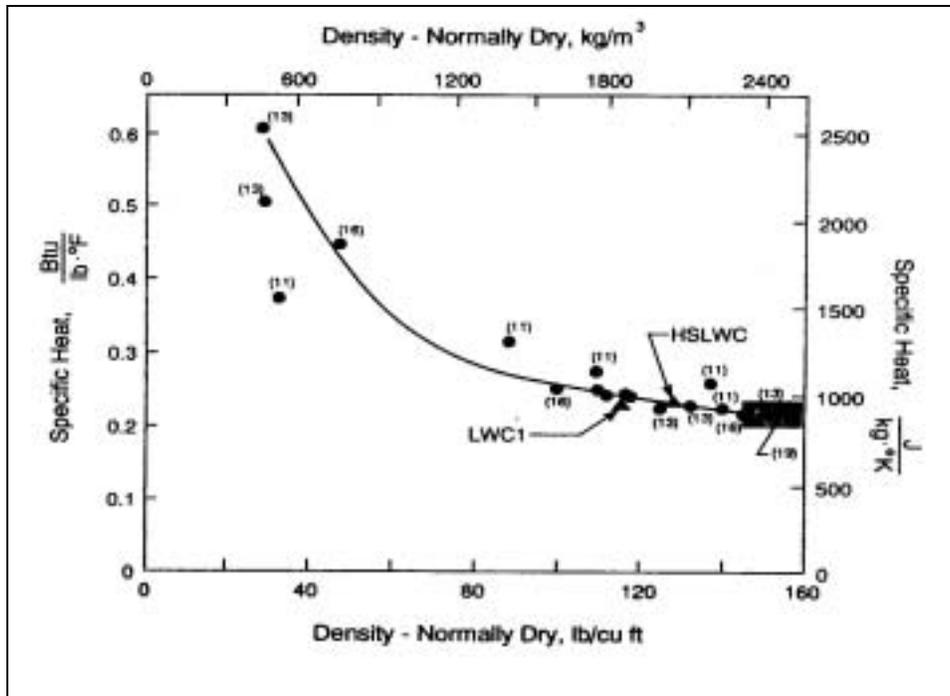


Figure 12. Specific heat of saturated surface-dry concrete as a function of density (from Hoff 1992, with permission of ACI)

Thermal Diffusivity

Thermal diffusivity is defined as thermal conductivity divided by the product of specific heat and density and relates to the rate at which temperature changes take place within a mass of material. Tests are generally conducted in accordance with CRD-C 36 (U.S. Army Corps of Engineers 1998a). The low value for expanded shale concrete shown in Table 5 is caused by the fact that thermal conductivity in the numerator has been shown to be exponentially influenced by density.

Hoff (1992) reported diffusivity results of 0.00183 and 0.00281 m²/hr (0.0197 and 0.0224 ft²/hr) for concrete densities of 1,913 and 2,043 kg/m³ (119.4 and 127.5 lb/ft³), respectively. These values are in line with those given in Table 5 and are consistent with values reported by other researchers (Figure 13).

Table 5 Typical Thermal Diffusivity Values (after Scanlon and McDonald 1994)		
Type of Aggregate in Concrete	Thermal Diffusivity	
	m ² /hr	ft ² /hr
Quartz	0.0079	0.085
Quartzite	0.0061	0.065
Limestone	0.0055	0.059
Basalt	0.0025	0.027
Expanded shale	0.0015	0.016

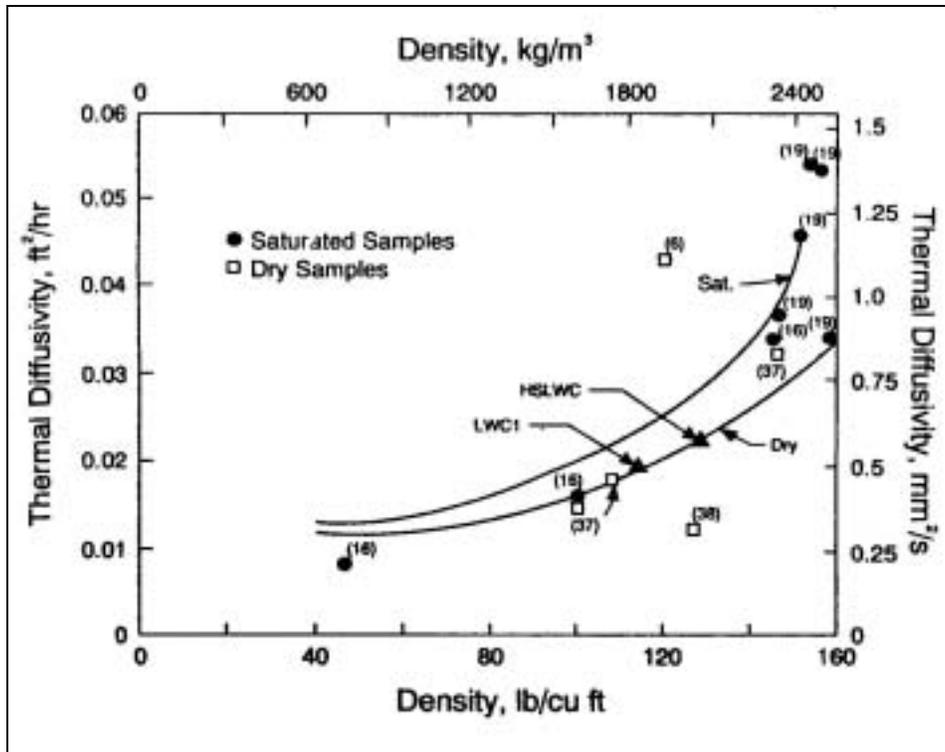


Figure 13. Thermal diffusivity of concrete as a function of density (from Hoff 1992, with permission of ACI)

Thermal Conductivity

Thermal conductivity is a property of a material and is a measure of the rate at which energy (heat) passes linearly through a unit area of homogeneous material of unit thickness for a temperature gradient of 1 deg. Thermal conductivity of concrete depends mainly on its density and moisture content but is also influenced by the size and distribution of the pores, the chemical composition of the solid components, their internal structure (crystalline or amorphous), and the test temperature. Crystalline materials (e.g. quartz) conduct heat better than amorphous materials (calcined clays, ceramics, etc.).

Low-density concrete

Thermal conductivity is generally measured on oven-dry samples in a guarded hot-plate assemblage according to ASTM C 177. Figure 14 shows the results of the analysis of conductivity tests on concretes with densities from 320 to 3,200 kg/m³ (20 - 200 lb/ft³) (Valore 1980), which suggest the equation

$$k = 0.072 e^{0.00125\varpi} \quad (4)$$

where k is the thermal conductivity in, $W m^{-1} \cdot K$, and ϖ is the density, in kg/m^3 .

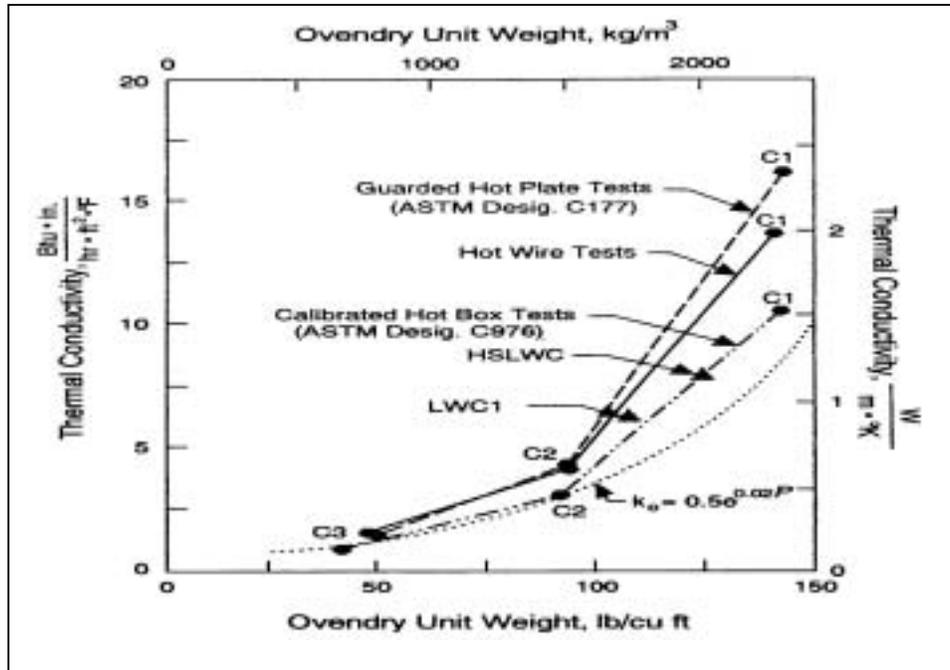


Figure 14. Thermal conductivity of concrete as a function of density (from Hoff 1992, with permission of ACI)

The corresponding equation in non-SI units is

$$k = 0.05\rho^{0.02}$$

where k is the thermal conductivity ($\text{Btu} \cdot \text{in.}/\text{hr} \cdot \text{ft}^2 \cdot \text{°F}$) and ρ is the density (lb/ft^3).

The Valore equation (Valore 1980) is accurate for concretes composed entirely of LDA up to a density of $1,600 \text{ kg}/\text{m}^3$ ($100 \text{ lb}/\text{ft}^3$) but becomes increasingly nonconservative for higher density concretes containing highly crystalline NDA. That being the case, it becomes essential to measure the thermal conductivity in a guarded hot plate for any specified-density concrete aggregate combination.

For a given concrete, an accurate value of the thermal conductivity based upon tests in a guarded hot plate (for oven-dry specimens) or a heat flowmeter (for rapid testing when specimens contain moisture) is preferable to an estimated value. However, the formula provides guidance for estimating the thermal conductivity in an oven-dry condition and, in addition, may readily be revised for air-dry conditions. When thermal resistance values are part of the project specifications, the addition of crystalline natural aggregates should be avoided, as the resulting thermal conductivity of the mixture will increase at a rate faster than that predicted by density alone (Schule and Kupke 1972).

Increasing the free-moisture content of hardened concrete causes an increase of thermal conductivity. As most conductivity data are reported for oven-dry concrete,

it is essential to know the moisture contents of the concrete in equilibrium with its in-service environment, and then apply a modification factor for estimating the conductivity under service conditions. Valore (1980) reported long-term moisture contents for concretes with the average for LDC being 4 percent by volume. As a practical matter, after considering the many variations of density, mixture composition, and in-service ambient conditions, Valore (1980) suggested that a reasonable approximation would be to increase the in-place thermal conductivity by 20 percent over test oven-dry values. CRD-C 44 (U.S. Army Corps of Engineers 1998b) provides a procedure for calculating thermal conductivity from the results of tests of thermal diffusivity and specific heat.

High-strength low-density concrete

With the exception that, in general, high-strength concretes have greater density (low w/c, low air content), there is only a modest increase in thermal conductivity with increased strength. The low porosity developed by the fully hydrated, rich cementitious fraction increases the thermal conductivity of the continuous matrix that encapsulates the aggregate fractions. The thermal conductivities of the oven-dried concretes reported by Hoff (1992) were 0.814 to 0.900 W/m · °K (5.64 to 6.24 Btu · in./hr · lb/ft³ · °F) for 1,922 kg/m³ (120.0 lb/ft³) concrete, and 1.10 to 1.07 W/m · °K (7.66 to 7.45 Btu · in./hr · lb/ft² · °F) for 2,051 kg/m³ (128.0 lb/ft³) high-strength concrete. These values compare well with the results estimated by the Valore equation (Valore 1980).

$$k = 0.072 e^{0.00125 \Delta 1922} = 0.80 \quad (k = 0.5 e^{0.02 \Delta 120} = 5.5)$$

$$k = 0.072 e^{0.00125 \Delta 2057} = 0.93 \quad (k = 0.5 e^{0.02 \Delta 128} = 6.5)$$

High-strength controlled-density concrete

The thermal conductivity of concrete is fundamentally influenced by the thermal conductivity of the aggregates that are used in the specified-density concrete mixtures. While the thermal conductivity of the mainly amorphous LDA does not differ significantly at any particular porosity, the thermal conductivity of NDA varies over a wide range that is principally determined by the degree of crystallinity. As reported by Scanlon and McDonald (1994) and shown in Tables 6 and 7, there exists a wide range of conductivity of concrete depending on aggregate type and moisture content.

Holm and Bremner (1987) reported the results of measurements of the thermal conductivity of LDC over a wide range of temperatures. Also included were measurements of the thermal conductivity of LD, expanded aggregates alone that averaged 0.47 W/m · °C (3.3 Btu · in./hr · ft²) over a temperature range of 42 to 1,400 °C (70 to 1,400 °F).

Table 6
Effect of Aggregate Type of Conductivity of Dry Concrete at Normal Temperatures

Aggregate Type	Density		Conductivity	
	kg/m ³	lb/ft ³	W/m °K	Btu in./hr ft ² °F
Hematite	2,870	179	2.6	18
Marble	2,290	143	1.7	12
Sandstone	1,920	120	1.4	10
Limestone	2,020	126	1.2	10
Dolerite	2,180	136	1.2	8.6
Barite	2,880	180	1.2	8.5
Expanded shale	1,430	89	0.62	4.3
Expanded slag	1,650	103	0.46	3.2
Expanded slag	960	60	0.22	1.5

Table 7
Effect of Aggregate Type on Conductivity of Moist Concrete at Normal Temperatures

Aggregate Type	Density		Conductivity	
	kg/m ³	lb/ft ³	W/m °K	Btu in./hr ft ² °F
Hematite	3,040	190	4.1	28
Quartzite	2,400	150	4.1	28
Quartzite	2,440	152	3.5	24
Dolomite	2,500	156	3.3	23
Quartzite	3.3	23
Limestone	2,450	153	3.2	22
Quartzite	2,350	147	3.1	21
Sandstone	2,130	133	2.9	20
Sandstone	2,400	150	2.9	20
Granite	2,420	151	2.6	18
Limestone	2,420	151	2.6	18
Marble	2,440	152	2.2	15
Limestone	2,440	152	2.2	15
Basalt	2,520	157	2.0	14
Rhyolite	2,340	146	2.0	14
Barite	3,040	190	2.0	14
Dolerite	2,350	147	2.0	14
Basalt	2,350	158	2.0	13
Expanded shale	1,590	99	0.85	5.9

Fire Resistance

Low-density concrete

When tested according to the procedures of ASTM E 119, structural LDC slabs, walls, and beams have demonstrated greater fire-endurance periods than equivalent-thickness members made with NDA. Superior performance is, according to ACI 213, due to a combination of lower thermal conductivity (lower temperature rise on unexposed surfaces), lower coefficient of thermal expansion (lower forces developed under restraint), and the inherent thermal stability developed by aggregates that have already been exposed to temperatures greater than 1,093 °C (2,000 °F) during pyroprocessing (see Figure 15).

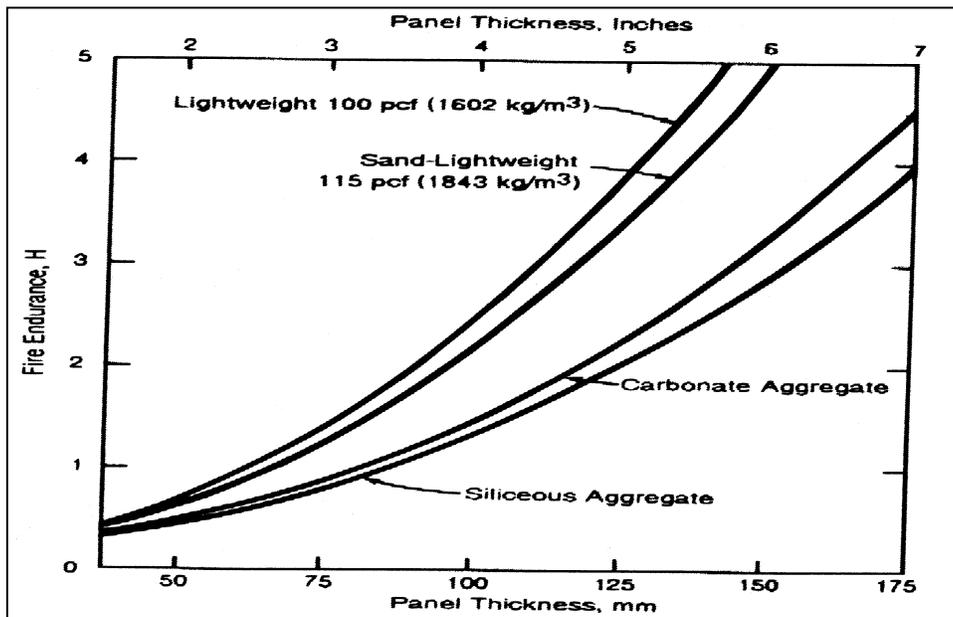


Figure 15. Fire endurance (heat transmission) of concrete slabs as a function of thickness for naturally dried specimens (ACI 213-87)

High-strength low-density concrete

While there is more than 50 years experience and a multitude of fire tests conducted on LDC of strength levels appropriate for commercial construction (20 to 35 MPa, 2,900 to 5,080 psi), the availability of data on HSLDC has, until recently, been very limited.

Recent testing (Bilodeau et al. 1995) has reported that, because of the extremely low permeability generally associated with HSC, there is a significantly reduced resistance to damage due to spalling. Because of the higher moisture contents of concretes containing LDA with high, as-batched absorbed water contents, there is even less resistance to damage due to spalling. Because of the use of HSLDC on several offshore platforms where intense hydrocarbon fires could develop, there was

even less resistance to damage due to spalling. Because of the use of HSLDC on several offshore platforms where intense hydrocarbon fires could develop, there was an obvious need for finding a remedy for this serious potential problem. Testing programs that are currently under way at Canada Centre for Mineral and Energy Technology (CANMET) continue evaluating and providing solutions for these high-risk conditions.

Several reports have documented the beneficial influence of adding small quantities of polypropylene fibers to HSLDC, as demonstrated by exposure to fire testing that was more intense than the exposure conditions (time-temperature criteria) specified by ASTM E 119. Jensen et al. (1995) reported the results of tests conducted at the Norwegian Fire Research Laboratories, Trondheim, Norway. 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.

Conclusions offered by the authors included the following:

- a. A considerable reduction in compressive strength and elastic modulus, even at relatively low temperatures between 100 and 300 °C (212 to 572 °F), was documented.
- b. Spalling is highly dependent on moisture content.
- c. The addition of 0.1 to 0.2 percent polypropylene fibers in the LDC mixture provided significant reduction of spalling. This was later confirmed by structural beam tests.
- d. Fire tests on beams confirmed earlier findings that severe spalling (exposed reinforcement) occurred on reinforced and prestressed LDC beams. Reduced spalling occurred on NDC beams. Reduced or no spalling occurred on LDC beams with polypropylene fibers. No spalling was observed on LDC beams with passive fire protection (a special cement-based mortar with expanded polystyrene balls).

At the same symposium at which the previously reported Jensen et al. (1995) paper was presented, Bilodeau et al. (1995) also commented on the behavior of several LDC exposed to hydrocarbon fires. This comprehensive report primarily, focused on the determination of mechanical properties, e.g. compressive, flexural and splitting tensile strengths, Young's modulus, drying shrinkage, and durability measurements. However, it also concluded that "all concretes without fibres were almost completely destroyed during the hydrocarbon fire. Based on the visual appearance, the use of polypropylene fibres improved considerably the fire resistance of the concrete." Apparently, the fibers melt and provide conduits for release of the pressure developed by the conversion of moisture to steam.

High-strength specified-density concrete

To a certain degree, the findings of these earlier tests have been paralleled by results from tests jointly sponsored by CANMET, Ottawa, Canada; Mobil, Dallas, TX; Synthetic Industries–Fibermesh, Chattanooga, TN; Headed Reinforcement Canada, Mt. Pearl, Newfoundland; and Health and Safety Executive, London, U.K. These tests used 500 by 1,000 by 1,000 mm (19.7 by 39.4 by 39.4 in.) reinforced prisms (Bilodeau, Malhotra, and Hoff 1998).

In tests conducted on NDC, LDC, and CDC, the authors reported the following results:

- a. The amount of spalling increased with the increase in the amount of LDA.
- b. Use of polypropylene fibers significantly reduced the spalling of concrete exposed to hydrocarbon fires.
- c. Reduced spalling resulted in a lower temperature increase in the core of the concrete and enhanced protection to the reinforcing steel.
- d. The properties of the concrete inside the block were not seriously affected by the fire exposure. However, the residual properties were slightly better for the concrete with fibers due to a smaller increase in temperature.
- e. The amount of fibers used in the concrete containing LDA was not fully adequate to prevent spalling. More research is needed to determine the optimum amount of fibers for the fire protection of different types of concrete.

Seawater Absorption

Because virtually all low-density concrete planned for marine applications will be, of necessity, high-strength, this section will not cover moderate-strength LDC. Early testing programs revealed that high-quality LDCs absorbed very little water and thus maintained their low density. This was not unexpected, as Bremner, Holm, and McNerney (1992) and Sugiyama, Bremner, and Holm (1996), in a series of publications, reported that the permeability of LDC was extremely low and generally equal to or significantly lower than that reported for NDC that were used as control specimens. Similar results by Russian, Japanese, and English investigators confirmed these findings. All attributed the low permeability to the profound influence of the high-integrity contact zone possessed by LDC. The zone of weakness demonstrated in concretes containing NDA, wherein layers of high w/c at the contact zone combine with bleed-water gaps, can be minimized if not eliminated in concretes containing pozzolanic materials such as silica fume, fly ash, and calcined clays, shales, and slates.

In investigations of high-quality concretes 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 40 kg/m^3 (2.5 lb/ft^3). Prior to the introduction of the test specimens into the seawater, all concretes lost mass during the drying phase of their curing, although concrete with a compressive strength of 62 MPa (9,000 psi) did not lose very much due to its very dense matrix.

The density changes for LDC2, LDC3, and LDC4 (LDC designed for 28-day strengths of 48 MPa (7,000 psi) and for HSLDC (62 MPa, 9,000 psi) are summarized in Table 8 (Hoff 1992).

Table 8			
Seawater Absorption			
Mixture	Total Time Under Pressure, days	Density Change with Time ¹	
		kg/m ³ (lb/ft ³)	kg/m ³ (lb/ft ³)
		4 m + 24 cc ²	14 m + 14 ³
0.61-MPa (89-psi) Pressure Test			
LDC2	246	51 (3.2)	38 (2.4)
LDC3	427	-3 (-0.2)	37 (2.3)
LDC4	470	11 (0.7)	30 (1.9)
HSLDC	333	46 (2.9)	42 (2.6)
Ambient Pressure Test, 0 MPa (0 psi)			
LDC2	246	14 (0.9)	10 (0.6)
LDC3	427	-21 (-1.3)	-26 (-1.6)
LDC4	470	-45 (-2.8)	-14 (-0.9)
HSLDC	333	5 (0.3)	14 (0.9)
¹ Positive values are density gain. Negative values are density loss. ² Cured 4 days in mold followed by 24 days in air after curing compound applied. ³ Cured 14 days moist followed by 14 days in air.			

It should be noted when considering these values that surface-to-volume ratio of the test specimens will be significantly different from that of actual in-service members.

The above density changes suggest that mixtures containing silica fume, which experience some drying during their initial curing period, will experience long-term density gains of 48 to 64 kg/m^3 (3 to 4 lb/ft^3) when subjected to hydrostatic pressures equivalent to 61 m (200 ft) of seawater. The very high-strength low-density concrete may be on the lower end of this range. At near-surface water depths (0 MPa (0 psi)), low-density concrete will have density increases of less than 16 kg/m^3 (1 lb/ft^3).

Ductility

The ductility of concrete structural frames should be analyzed as a composite system—that is, as reinforced concrete. Studies by Ahmad and Batts (1991) and Ahmad and Barker (1991) indicate, for the materials tested, that the ACI rectangular

stress block is adequate for strength predictions of HSLDC beams, and the recommendation of 0.003 as the maximum usable concrete strain is an acceptable lower bound for HSLDC members with strengths not exceeding 76.5 MPa (11,000 psi) and reinforcement ratios less than 54 percent of balanced ratio ψ_b . Moreno (1986) found that while LDC exhibited a rapidly descending portion of the stress-strain curve, it was possible to obtain a flat descending curve with reinforced LDC members that were provided with a sufficient amount of confining reinforcement slightly greater than that with NDC. Additional confining steel is recommended to compensate for the lower post-elastic strain behavior of LDC. This report also included study results that showed that it was economically feasible to obtain the desired ductility by increasing the amount of steel confinement.

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

Fatigue of Low-Density Concrete

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

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

This work was followed by Ramakrishnan, Bremner, and Malhotra (1992) who found that, under wet conditions, the fatigue endurance limit was the same for LD and NDC.

Because of the significance of oscillating stresses that would be developed by wave action on offshore structures, and due to the necessity for these marine structures to use a low-density concrete for buoyancy considerations, a considerable amount of research has been commissioned to determine the fatigue resistance of HSLDC and to compare these results with the characteristics of NDC. 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, HSLDC performs as well as HSNDC and, in many instances, provides longer fatigue life.” It is, however, the long-term service performance of real structures that provides improved confidence in material behavior rather than the extrapolation of conclusions obtained from laboratory investigations.

The long-term field performance of LDC bridge members constructed in Florida in 1964 (Figure 16) was evaluated in an in-depth investigation conducted in 1992 (Brown, Larsen, and Holm 1995). Comprehensive field measurements of service load strains and deflections taken in 1968 and 1992 were compared with the theoretical bridge responses predicted by a finite element model that is part of the Florida Department of Transportation bridge rating system (Brown and Davis 1993). The original 1968 loadings and measurements of the bridge were duplicated in 1992 and compared with calculated deflections, as shown in Figure 17 (Brown, Larsen, and Holm 1995). Maximum deflection for one particular beam due to a midpoint load was 7.1 mm (0.28 in.), measured at 18.4 m (60.5 ft) from the unrestrained end of the span. This compares very well with the original deflection, which was recorded to be 6.6 mm (0.26 in.) measured at 15.4 m (50.5 ft). Rolling load deflections measured in 1968 and 1992 were also comparable, but slightly less in magnitude than the static loads.

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

It appears that dynamic testing of the flexural characteristics of the 31-year-old long-span LDC bridge corroborates the conclusions of fatigue investigations conducted on small specimens tested under controlled conditions in several laboratories (Hoff 1994, Gjerde 1982, Gray and McLaughlin 1961). In these investigations, it was generally observed that the LDC performed as well as and, in most cases, somewhat better than companion normal-density control specimens. Several investigators have suggested that improved performance was due to the elastic compatibility of the LDA particles to that of the surrounding cementitious matrix. In LDC, the elastic modulus of the constituent phases (coarse aggregate and the enveloping mortar phase) is relatively similar, while with NDC the elastic modulus of most NDAs may be as much as 3 to 5 times greater than their enveloping matrix (Bremner and Holm 1986). With LDC, elastic similarity of the two phases of a composite system results in a profound reduction of stress concentrations and a leveling out of the average stress over the cross section of the loaded member. NDC having a significant elastic mismatch will inevitably develop stress concentrations that result in extensive microcracking in the concrete composite.

Additionally, because of the pozzolanic reactivity of the surface of the vesicular aggregate that has been fired at temperatures above 1,100 °C (2,012 °F) (Khokrin 1973), the quality and integrity of the contact zone of LDC 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 LDC will develop a lower incidence of microcracking (Holm, Bremner, and Newman 1984).



Figure 16. Barge-mounted frame placed beams (to the right is an old truss bridge; both bridges will carry U.S. 19 traffic) (Brown, Larsen, and Holm 1995)

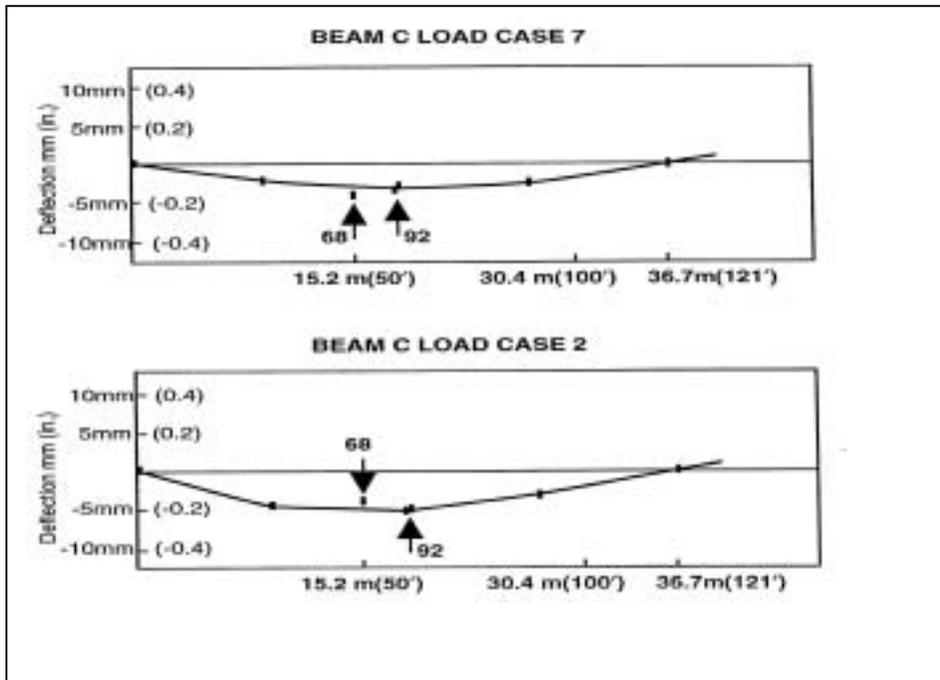


Figure 17. Florida DOT-predicted deflections compared with 1968 and 1992 measurements (Brown, Larsen, and Holm 1995)

Table 9 provides midrange values of various physical properties of LDC and NDC, normal-density concretes, which can be used for rough comparative purposes.

While significant progress has been reported in optimizing the physical characteristics and the engineering properties, there still remain areas of inadequate understanding. Additional research is now necessary to fully develop building code design criteria governing the shear, torsion, development length, and seismic behavior of high-strength, low-density and high-strength, controlled-density concretes. Additionally, the response to high temperatures developed during placement is not fully understood. Understanding this phenomenon would require a comprehensive integration of a number of time-related properties that differ substantially from concretes containing NDA.

Table 9 Summary of Typical Mechanical and Physical Properties of Structural Low- and Normal-Density Concretes			
Property		Structural Low-Density Concrete	Normal-Density Concrete
Design density	kg/m ³	1,850	2,400
	(lb/ft ³)	(115)	(150)
Compressive strength	MPa	20 - 50	20 - 70
	(psi)	(3,000 - 7,500)	(3,000 - 10,000)
Tensile strength	MPa	2.5	3.0
	(psi)	(360)	(435)
Modulus of elasticity	GPa	17 - 28	20 - 40
	(psi x 10 ⁶)	(2.5 - 4.0)	(3 to 6)
Poisson's ratio		0.2	0.2
Shrinkage at 1 year, microstrain		600	550
Specific creep	microstrain x 10 ⁻⁶ /MPa	70 - 150	70 - 120
	(microstrain x 10 ⁻⁶ /lb/in. ²)	(0.5 - 1.0)	(0.5 - 0.8)
Specific heat	J/kg · K	960	920
	(cal/g · °C)	(0.23)	(0.22)
Thermal conductivity	W/m · K	0.58 - 0.86	1.4 - 2.9
	(B · in./hr · ft ² · °F)	(4 - 6)	(10 - 20)
Thermal diffusivity	m ² /hr	0.0015	0.0025 - 0.0079
	(ft ² /hr)	(0.016)	(0.027 - 0.085)
Thermal expansion	microstrain Δ 10 ⁻⁶ /°C	9±	11±
	microstrain Δ 10 ⁻⁶ /°F	5±	6±

Note: Values shown are midrange numbers that vary depending on mixture constituents and strength levels; use for approximation purposes only.

6 Durability

Performance Record

The first structural application using rotary kiln-produced low-density concrete was the USS *Selma*, which was launched in 1919 (Holm 1980a). This 7,500 ton ship, now in Galveston Harbor and declared a National Monument, has been exposed to seawater continuously. In some areas it was damaged—when run onto a rock breakwater at Tampico Bay, Mexico, by an inebriated captain, and as a result of hard berthing. The concrete below the waterline both inside and outside the hull (after the barnacles had been scraped off) seemed in an as-cast condition with the form marks still visible.

Several of the *Selma's* holds contain water with about a 6-m head above the surrounding sea, providing ample proof of the low permeability of the concrete. In undamaged sections of the ship the 12- to 30-mm (0.5 to 1.2-in.) cover has proven surprisingly effective in protecting the reinforcing steel from corrosion. The strength of the concrete was in excess of 27.6 MPa (4,000 psi) at 28 days at a time when ordinary concrete had a strength of 13.8 MPa (2,000 psi). Cores taken from the ship in 1980 had compressive strengths in excess of 55.2 MPa (8,000 psi) for concrete with a unit weight of about 1,762 kg/m³ (110 lb/ft³). Given that the strength-to-density ratio is comparable to what is now commonly referred to as high-performance NDC, it would seem that there is an almost 8-decade headstart for high-performance, LDC.

Samples of concrete from the *Selma* below the waterline were examined in a scanning electron microscope, and it was noted that, other than in a region near the aggregate-cement paste interface, there was no propensity for the aggregate vesicles to become filled with marine or hydration products. Also, the aggregate-cement paste interface was of exceptional quality, with the transition between hydration product and aggregate in most instances difficult to discern, which is not the case for NDC (Holm, Bremner, and Newman 1984). With NDC, extensive microcracking typically occurs at the aggregate-cement paste interface. The hydration products are normally of inferior quality at the interface as well. In terms of validating long-term performance of current LDC projects, the aggregate from the *Selma* had a microstructure that was identical to aggregates produced by a modern rotary kiln, implying that long-term good performance can be expected from our current product, provided that changes to the portland cement are not a factor.

Resistance to Freezing and Thawing

Probably one of the most severe exposure conditions for concrete is in bridge decks in regions where de-icing agents are used. If concrete freezes at the beginning of winter and stays frozen until the end of winter, only one (or very few) cycles of freezing and thawing will have occurred, with little likelihood of damage until passage of many years. However, severe damage may be caused in relatively mild climates where large amounts of de-icing agents are applied. De-icing chemicals melt ice and snow and produce water that increases concrete saturation. The concrete then freezes again when the temperature drops, frequently resulting in daily cycles. If salt, and the sand holding the salt, are not promptly removed, steel corrosion is facilitated. Once corrosion begins, the concrete cover over the reinforcement starts to spall. The problem is most severe in the northeastern areas of the United States, making this region a useful location for comparative studies of the relative performance of LD and NDC. These areas have had a long history of bare bridge decks, whereas in Canada it is common to use a waterproof layer under the asphalt to prevent ingress of chloride ions into the concrete. A study of LDC bridge decks was completed in 1960 (Expanded Shale, Clay, and Slate Institute 1960). Based on published reports in the United States (FHWA 1985), England, and Japan, plus personal observations, the performance of LDC bridge decks is at least as good as NDC bridges (Brown, Larsen, and Holm 1995).

By 1935, over 34 low-density concrete bridges had been built in North America, including nine in Canada (Expanded Shale, Clay, and Slate Institute 1960). The good performance of several early bridges, built before concrete was air entrained, is surprising. The fact that chemical admixtures that entrained some air were found desirable in placing LDC might, in part, account for their good long-term performance (Holm 1983). Another reason for their good performance is that pores within the LDA can act as pressure relief chambers when the hydraulic pressure develops as the chemically uncombined water freezes. Crushed vesicular brick has also been shown to provide freeze-thaw protection in a similar manner when added to concrete that was subsequently exposed to freezing and thawing.

For the last several decades it has been common practice to use small amounts of entrained air in all LDC. When freezing and thawing is anticipated, 4 to 8 percent entrained air is recommended in LDC with a nominal maximum aggregate size of 19.0 mm, and 5 to 9 percent when the nominal maximum aggregate size is 9.5 mm. To achieve an effective air-void system in the concrete that will protect it from repeated cycles of saturated freezing and thawing, it is essential that the air voids be well distributed throughout the cement paste matrix. Normally, the longest distance from any point in the cement paste matrix to an entrained air void should be less than 0.2 mm. This can normally be achieved by using an air-entraining admixture meeting the requirements of ASTM C 260. In special situations where exposure conditions are severe or where unusual placing techniques are involved, the actual air-void spacing should be measured in simulated job site conditions to confirm that an adequate air void system will be achieved. This is done according to the procedures described in ASTM C 457.

Resistance to Sulfate Attack

As with NDC, the ACI 318 recommendations should be followed with respect to the level of sulfates in the groundwater (Table 10) (ACI 318). This entails limiting the tricalcium aluminate in the cement, which is the compound that combines with the sulfates to produce an expansion. Seawater also contains sulfates, but the presence of chlorides tends to inhibit the expansive reaction that is characteristic of attack by sulfates from groundwater or soils. It has become normal practice to allow up to 10 percent tricalcium aluminate in concrete exposed to seawater.

Sulfate Exposure	Water-Soluble Sulfate (SO ₄) in Soil, percent by mass	Sulfate (SO ₄) in Water, ppm	Cement Type	Maximum Water-Cementitious Materials Ratio, by mass, Normal-Density Aggregate Concrete	Minimum f' _c , Normal-Density and Low-Density Aggregate Concrete, psi ¹
Negligible	0.00-0.10	0-150	--	--	--
Moderate ²	0.10-0.20	150-1,500	II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	4,000
Severe	0.20-2.00	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00	Over 10,000	V plus pozzolan ³	0.45	4,500

¹ A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing.
² Seawater.
³ Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

As with most attacks from the surface, increased impermeability improves resistance to deterioration. A lower W/C_m, increased moist curing, and the use of air-entrained concrete are desirable. Also, the reduced microcracking in LDC and the improved quality of aggregate-to-cement paste bond tend to make the concrete more resistant to sulfate attack.

Resistance to Alkali-Aggregate Reaction

Concrete made from either natural LDA or manufactured LDA appears not to be adversely affected by any long-term interaction between silica-rich aggregates and the alkalis in the cement, or from the ingress of alkalis from natural sources such as seawater (Holm 1980a). In concrete mixtures that contain reactive NDA, replacement of either reactive or even the nonreactive NDA with LDA has been found to significantly reduce deleterious expansions (Bremner et al. 1998, Boyd 1998).

The heating of the aggregates tends to activate the aggregate surface such that it appears to act as a source of silica to react with the alkalis from the cement at an early age to counteract any potential long-term disruptive expansion (Boyd 1998). Another factor that enables a porous aggregate to reduce disruptive expansion is the availability of space within the expanded aggregate for reactive material to precipitate in a benign manner. In Figure 18, the beneficial effect of replacing some of the aggregates with low-density aggregates on expansion can be seen.

In Figure 19, precipitation of alkali-rich material in the pores of an expanded aggregate can be seen in concrete made with a well-known reactive ND coarse aggregates in which some of the nonreactive fine aggregates have been replaced with LD fine aggregates.

Carbonation

Carbonation in concrete is the reaction of carbon dioxide from the air with calcium hydroxide liberated from the hydration process. This reaction produces calcium carbonate that can neutralize the natural protection of steel reinforcement afforded by the concrete. While the rate at which the carbonation front advances into concrete has been noted, most studies have been of a relatively short-term nature.

Concern for carbonation is predicated on the pH in concrete lowering from approximately 13 to the vicinity of 9, which in turn neutralizes the protective layer over the reinforcing steel, making it vulnerable to corrosion. Two primary mechanisms protect steel from corrosion: the combination of an adequate depth of cover with a sufficiently high quality of the concrete. This quality is usually related to water-cement ratio or strength (relatively easy properties to quantify), but is more closely related to permeability and strain capacity of the concretes.

Measurements of Carbonation Depth in Mature Marine Structures

Concrete ships, Cape Charles, VA

Holm, Bremner, and Vaysburd (1988) reported the results of carbonation measurements conducted on cores drilled from several concrete ships built during World War II. The ships were used as breakwaters for a ferry-boat dock in the Chesapeake Bay at Cape Charles, Virginia. 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 35 MPa (5,080 psi) at 28 days with a density of $1,730 \text{ kg/m}^3$ (108 pcf) (McLaughlin 1944). Despite freezing and thawing in a marine environment, the hulls and superstructure concretes are in excellent condition after 5 decades of exposure. The only less-than-satisfactory performance was observed in some areas of the main decks. These

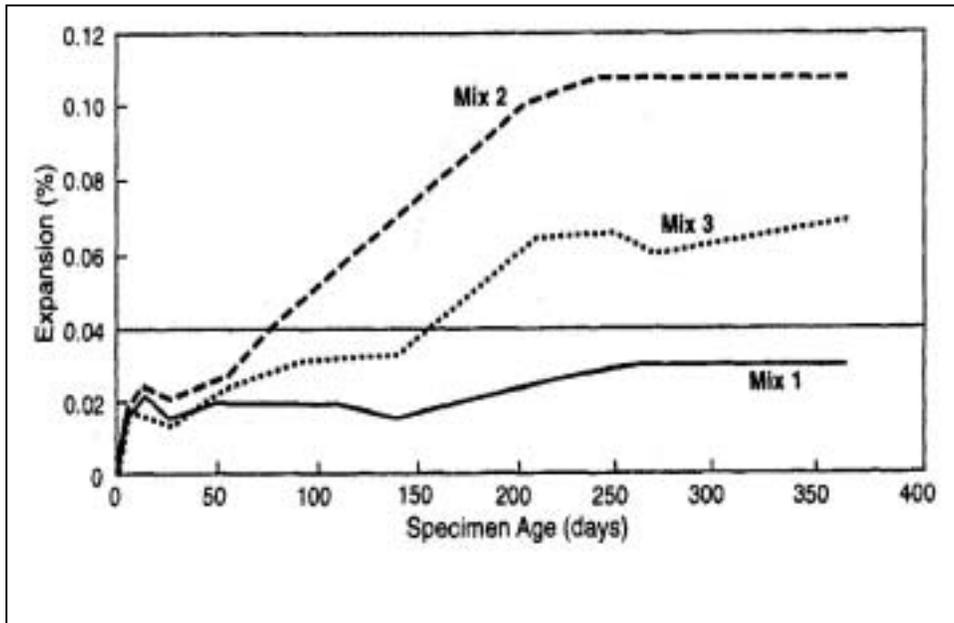


Figure 18. Reduction of expansion when low-density aggregates are used (Bremner et al. 1998) (Mixture 1 - Nonreactive normal-density fine and coarse aggregate; Mixture 2 - Reactive normal-density coarse aggregate and nonreactive fine aggregate; Mixture 3 - Reactive normal-density coarse aggregate and one half of absolute volume of nonreactive fine aggregate replaced by low-density fine aggregate)

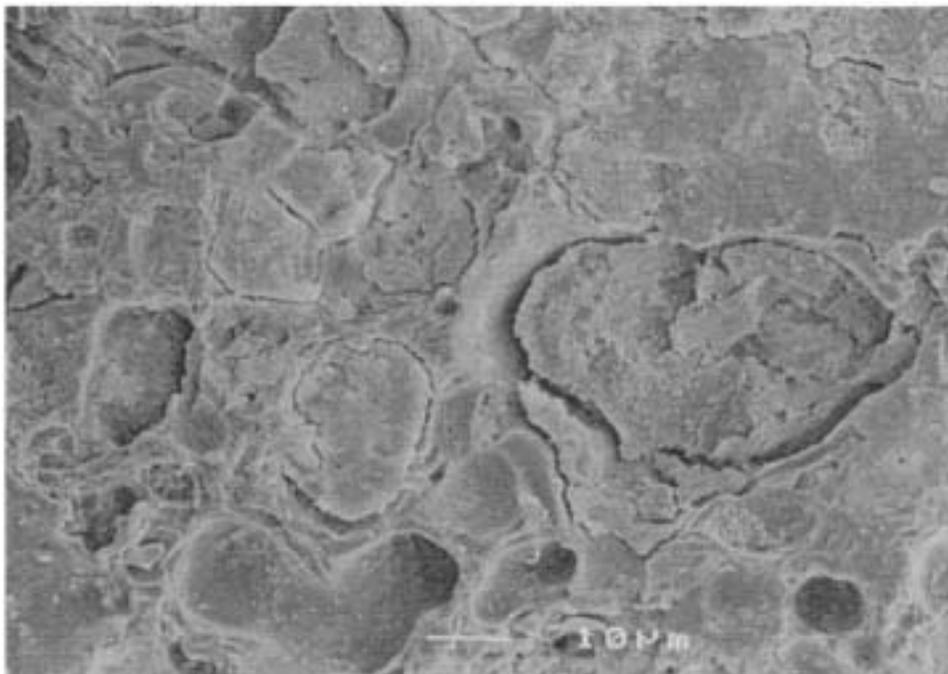


Figure 19. Concrete with a precipitation of alkali-rich material in the pores of low-density aggregate (Boyd 1998)

areas experienced a delamination of the 20-mm (0.78-in.) concrete cover protecting four layers of large sized undeformed reinforcing bars spaced 100 mm (4 in.) on centers. In retrospect, this failure plane is understandable and would have been avoided by the use of modern prestressing procedures. Cover for hull reinforcing was specified at 22 mm (7/8 in.), with all other reinforcement protected by only 13 mm (1/2 in.).

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. This was done by following the procedures of ASTM C 496. 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 depth (as revealed by spraying the freshly fractured surface with a standard solution of phenolphthalein) averaged 1 mm for specimens taken from the main deck, was between 1 and 2 mm (0.04 and 0.08 in.) for concretes in exposed wing walls, and was virtually nonexistent in the hull and bulkheads. Coring was conducted from the waterline to as much as 5 m (16 ft) above high water, and in no instances could carbonation depths greater than 2 mm (0.08 in.) be found. In isolated instances, flexural cracks up to 8 mm (0.31 in.) in depth were encountered, and these had carbonated in the plane of the crack. The carbonation did not appear to progress more than 0.1 mm (0.004 in.) perpendicular to the plane of the crack.

The result of these tests are given in Table 11. The value of K_c is calculated as follows:

$$K_c = \frac{d}{\sqrt{t}} \quad (5)$$

where

K_c = carbonation coefficient

d = carbonation depth (in millimeters) determined by spraying a freshly exposed surface with phenolphthalein

t = time (in years)

Two primary factors influence the carbonation coefficients. High-quality, low-permeability concrete will inhibit the diffusion of carbon dioxide, and the concrete with high moisture content will reduce the diffusion rate to that of a gas through water rather than that of a gas through air.

Chesapeake Bay Bridge, Annapolis, MD

Concrete cores taken from the 35-year-old Chesapeake Bay Bridge revealed carbonation depths of 2 to 8 mm (0.08 to 0.31 in.) from the top of the bridge deck

Table 11 Field Measurements of the Depth of Carbonation				
Location	Structure and Age	Concrete Data (Strength and Density)	Depth of Carbonation mm (in.)	Kc (mm/√years)
Cape Charles, Virginia	Concrete ships	All LDC (35 MPa, 1,730 kg/m ³) (A) Hull bulkhead (B) Wing-wall (C) Superstructure deck-top (D) Superstructure deck-bottom	1 (0.04) 1 (0.04) 1 (0.04) 2 (0.08)	0.2 0.2 0.2 0.3
Annapolis, MD Chesapeake Bay	Multispan, 4-mile bridge, 35 years	All LDC (24 MPa, 1,650 kg/m ³) (A) Top surface, truss span (B) Bottom surface, truss span (C) Top surface, approach span (D) Bottom surface, approach span	1 (0.04) 5 (0.20) 8 (0.31) 13 (0.51)	0.2 0.8 1.4 2.2
Coxsackie, NY (not over seawater)	N.Y. State Thruway interchange bridge, 15 years	Sanded LDC (27 MPa, 1,760 kg/m) (A) Exposed top deck (B) Bottom surface	5 (0.20) 10 (0.39)	1.3 2.6
Japan	Bridges/viaducts, 19 years	Sanded LDC (23 MPa, 1,820 kg) Sanded LDC (26 MPa)	16 (0.63) 18 (0.71)	3.7 4.1
All LDC: Lightweight fine and coarse aggregate.				

and 2 to 13 mm (0.08 to 0.51 in.) from the underside of the bridge deck. The higher carbonation depth on the underside reflects the increased gas diffusion associated with this drier portion of the bridge. The 36-mm (1.41-in.) asphalt wearing course appears to have inhibited drying and thus reduced carbonation depth on top. Physical and mechanical properties have been reported previously (Holm 1983; Holm, Bremner, and Newman 1984).

Coxsackie Bridge, New York

Cores drilled with the permission and cooperation of the New York State Thruway Authority from the 15-year-old exposed deck surface of the Interchange Bridge at Coxsackie revealed 5-mm (0.20-in.) carbonation depths for the top surface and 10 mm (0.39 in.) from the bottom. Despite almost 1,000 saltings of the exposed deck, there was no evidence of corrosion in any of the reinforcing bars cut by the six cores taken (Holm, Bremner, and Newman 1984).

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). These investigators studied the chloride penetration, depth of carbonation, and incidence of microcracking in both structural LDC and NDC 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) are similar to unpublished

studies (by the authors of this report) conducted on mature bridges on the eastern coast of North America.

Recommendations to Limit Rate of Carbonation

Field and laboratory experience was used to construct Table 12. The quality of the concrete, insofar as its resistance to the penetration of carbonation, may be categorized by maximum anticipated carbonation coefficients of 4 and 8, as shown in Figures 20 and 21. This approach for specific depths of cover can give an estimate of the period during which corrosion will not be initiated by carbonation factors.

Table 12		
Estimate of Time In Years for Carbonation to Reach Reinforcing Steel		
Concrete applications	Exposed marine, marine structures, bridge decks	Insulating and nonstructural concretes
Concrete inspection	Continuous	No
Concrete quality	High	Low
As measured by w/c ratio	< 0.45	> 0.65
As measured by compressive strength	> 30 MPa (4,350 psi)	< 20 MPa (2,900 psi)
Maximum Rate of Carbonation		
$K_c = dc/\sqrt{t}$ in (mm/ $\sqrt{\text{years}}$)	4 ¹	8 ²
Concrete cover of 20 mm (0.78 in.)	25 years	6 years
Concrete cover of 30 mm (1.18 in.)	56 years	14 years
Concrete cover of 40 mm (1.58 in.)	100 years	25 years
¹ As observed from field measurements of mature marine structures.		
² As observed from laboratory specimens.		

Permeability and Corrosion Protection

While current technical literature contains numerous reports on the permeability of concrete, only a limited number of papers report experiments in which structural LDC and NDC were tested under the same conditions. Furthermore, almost all studies measuring permeability use test conditions that are static, insofar as the concrete is concerned. While this approach is appropriate for dams and water-containing structures, it is not relevant to bridges and parking garages, which are constantly subjected to dynamic stress and strain. Cover concrete is expected to maintain its protective impermeable integrity despite the accumulation of shrinkage, thermal, and structural load-related strains.

Permeability investigations conducted on LDC and NDC exposed to the same testing criteria have been reported by Khokrin (1973), Nishi et al. (1980), Keeton (1970), and Bamforth (1987). It is of interest that, in every case, despite wide variations in concrete strengths, testing media (water, gas, and oil), and testing

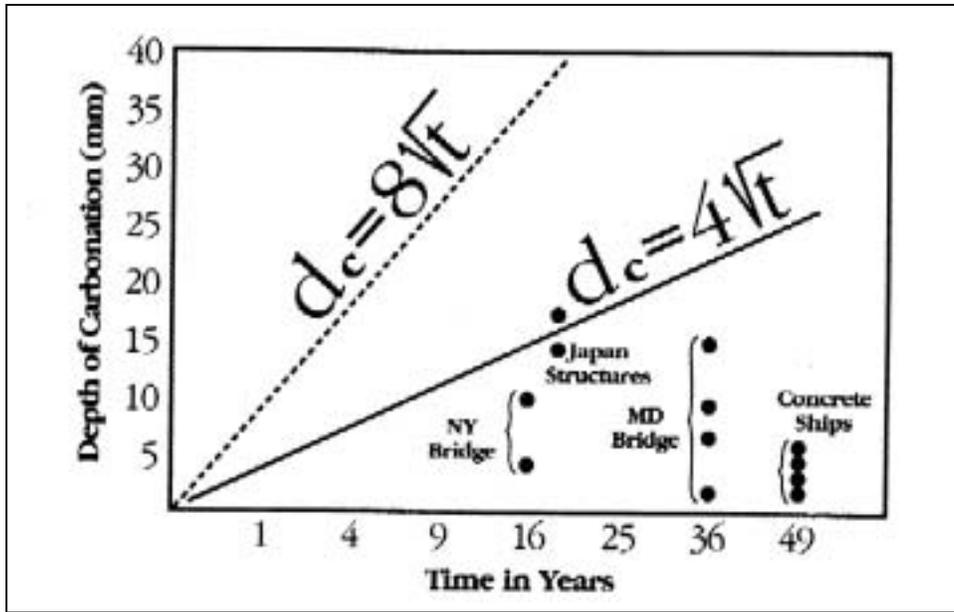


Figure 20. Measured depth of carbonation (in millimeters) of exposed low-density concrete (from Holm, Bremner, and Vaysburd 1988, with permission of ACI)

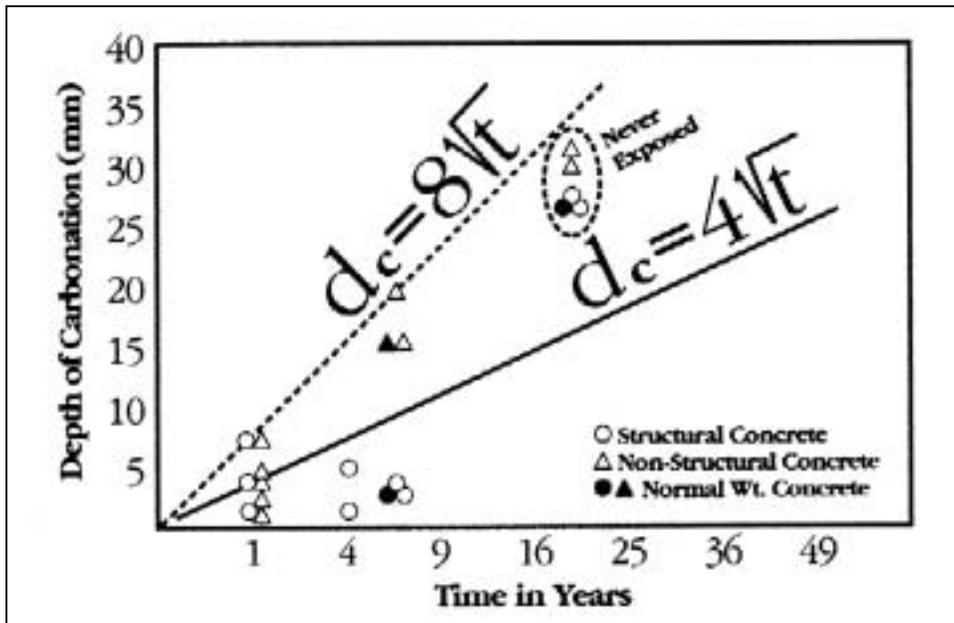


Figure 21. Measured depth of carbonation (in millimeters) of laboratory specimens of low-density concrete (from Holm, Bremner, and Vaysburd 1988, permission of ACI)

techniques (specimen size, media pressure, and equipment), structural LDC had equal or lower permeability than its ND counterpart. Khokrin (1973) further reported that the lower permeability of LDC 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), concretes with water-cement ratios of 0.55, moist-cured for 28 days when tested at 9 kg/cm² water pressure had depth of penetration of 35 mm (1.38 in.) for NDC and 24 mm (0.95 in.) for LDC. When tested with seawater, penetration was 15 and 12 mm (0.59 and 0.47 in.) for NDC and LDC, 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), who reported the lowest permeability with HSLDC. Bamforth (1987) incorporated structural LDC as one of the four concretes tested for permeability to nitrogen gas at 1 MPa (145 psi) pressure level. The NDC specimens included high-strength (90 MPa (13,000 psi)) concrete as well as concretes with a 25-percent fly ash replacement, by mass or volume. The sanded structural LDC (50 MPa (7,250 psi), 6.4 percent air) with a density of 1,985 kg/m (124 lb/ft³) demonstrated the lowest water and air permeability of all mixtures tested.

Fully hydrated portland cement paste of low w/c has the potential to form an essentially impermeable matrix that should render concretes impermeable to the flow of liquids and gases. In practice, however, this is not the case, as microcracks form in concrete during the hardening process, as well as 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 differentially responding to temperature gradients, service load-included strains, and volume changes associated with chemical reactions taking place within the concrete. In addition, channels develop in the transition zone surrounding coarse aggregates, giving rise to unimpeded moisture movements. While separations caused by the evaporation of bleed water adjacent to natural aggregates are frequently visible to the naked eye, such defects are almost unknown in structural LDC. The continuous, high-quality matrix fraction surrounding LDA 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 LDA. Bremner, Holm, and deSouza (1984) conducted measurements of the diffusion of the silica out of the coarse LDA 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 structural LDC extended approximately 60 μm from the LDA particles into the continuous matrix phase.

In addition, the contact zone in structural LDC is the interface between two porous media: the LDA particle and the hydrating cement binder. This porous media interface allows for hygral equilibrium to be reached between the two phases, thus eliminating weak zones caused by water concentration. In contrast, the contact zone of NDC is an interface between a dense, nonabsorbent component and a water-rich binder. Any accumulation of water at that interface is subsequently lost during drying, leaving voids.

Laboratory testing of NDC and LDC indicates that, in the unstressed state, the permeability of the two concretes is about equal. However, at higher levels of stress, the LDC can be loaded to a higher percentage of its ultimate compressive strength before the microcracking causes a sharp increase in permeability (Sugiyama, Bremner, and Holm 1996). This laboratory testing fails to take into account the more aggressive conditions that exist in the field, particularly at an early age. In the laboratory, the concrete is maintained at constant temperature, there are no significant shrinkage restraints, and field-imposed stresses are absent. All of these issues need to be accounted for. Because of the initial absorption of water by the LDA prior to mixing, this absorbed water can act as water 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 reaction is effective over a long time, laboratory testing that is usually completed in less than a few months may not adequately take this into account.

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 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 that cause separation, have not received the same attention as has the air void system necessary to protect the paste. Aggregates are frequently dismissed as being inert fillers and, as a result, they and the associated transition zone have until recently received very modest attention.

In order that concrete perform satisfactorily in severe exposure conditions, it is essential that a good bond develop and be 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. Deterioration will result, with pieces of apparently sound mortar separating from the bottom of the aggregate, usually with some of the mortar remaining firmly attached to the top side of the aggregate. An equally serious consequence of microcracking is the easy path provided for the ingress of salt water into the mass of the concrete. Here, it can react with the products of hydration and render ineffective the protective layer of concrete over the reinforcing steel. 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, Holm, and deSouza (1984).

The contact zone (the interface between the LDA and the surrounding mortar matrix) of LDC has been demonstrated to be significantly superior to that of NDCs that do not contain silica fume (Holm, Bremner, and Newman 1984; Khokrin 1973). This profound improvement in the quality, integrity, and microstructure stems from a number of characteristics unique to LDC, including but not limited to the following:

- a. The alumina/silicate surface of the fired ceramic aggregate, which is pozzolanic and combines with CaOH_2 liberated by hydration of the portland cement.
- b. Reduced microcracking at the matrix LDA interface because of the elastic similarity of the aggregate and the surrounding cementitious matrix.
- c. Hygral equilibrium between two porous materials (LDA and a porous cementitious matrix) as opposed to the usual condition with NDA, where bleed-water lenses around coarse natural aggregates have W/C_m 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 LDA is moderately enhanced. However, when used in concretes containing NDA, this zone of weakness is profoundly improved.

Contact zone of mature concrete subjected to severe exposure

Micrographs of the contact zone of specimens were prepared for examination in a Cambridge S4-10 scanning electron microscope equipped with a Tracor Northern NS-880 energy-dispersive X-ray analyzer. An example is Figure 22, which is a micrograph from the waterline of a more than 60-year-old concrete ship that was previously reported by Holm (1980a), and Holm, Bremner, and Newman (1984). Based on this micrograph and an examination of other areas, it would appear that a good bond develops between the LDA and the mortar matrix. NDC samples taken from bridge decks were also examined and revealed cracking between the aggregate and the matrix, as had been reported by Hsu et al. (1963).

Related studies on the contact zone

Russian studies on the durability of low-density concrete edited by Khokrin (1973) included results of scanning electron microscopy that revealed new chemical formations at the contact zone between the matrix and keramzite (rotary kiln-produced expanded clay or shale). These micrographs confirmed earlier tests in

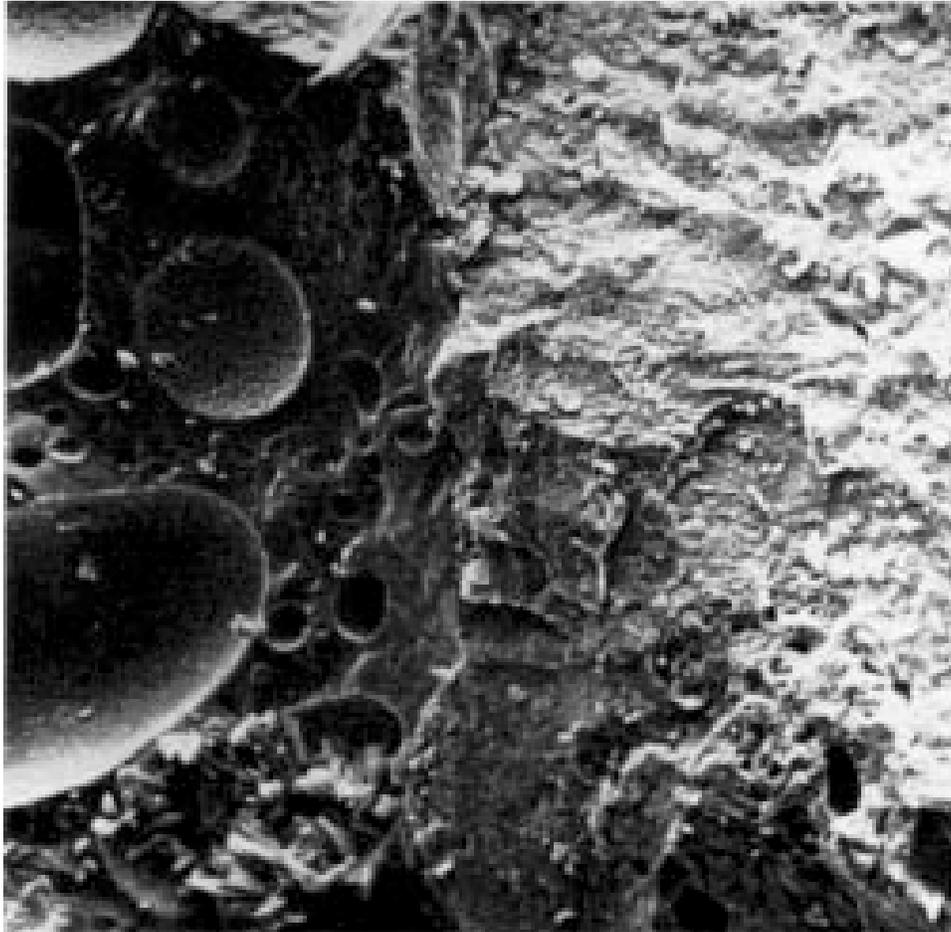


Figure 22. Fractured surface of concrete from USS *Selma*. (Expanded aggregate is on the left, and the cement paste is on the right. Micrograph width is 550 μm) (from Bremner, Holm, and Morgan 1996, with permission of ACI)

which X-ray photographs 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 (c/z) of LDC and NDC, which established the width of the c/z as approximately 60 μm . This research also concluded that the hardness of the matrix in the contact zone was in the range of 90 to 150 kgf/mm^2 (128,000-213,400 $\text{lbf}/\text{in.}^2$) while outside the contact zone the hardness measured 60 to 80 kgf/mm^2 (85,400-113,800 $\text{lbf}/\text{in.}^2$). Another investigation that included limestone, diabase, and rotary kiln expanded aggregates also varied the w/c. These results are shown in Table 13.

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 NDC was studied by Fenwick and Sue (1982). This 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. Reactions in mechanical properties are inevitable as a result of the interface flaws, as they limit interaction between the two distinctly different phases.

Table 13
Microhardness in and Beyond the Contact Zone (c/z) of Concretes with Differing Water-Cement Ratios and Various Coarse Aggregates (after Khokrin 1973)

Coarse Aggregate Type	Water-Cement Ratio					
	0.3		0.4		0.5	
	In c/z	Beyond c/z	In c/z	Beyond c/z	In c/z	Beyond c/z
Lightweight aggregate "B"	160	92	143	78	136	76
Lightweight aggregate "O"	167	94	138	73	125	68
Crushed diabase	81	79	--	--	--	--
Crushed limestone	81	81	--	--	--	--

However significant any reduction in compressive and tensile strength due to poor c/z, the effect on permeability is even greater. Increasing permeability inevitably leads to penetration of aggressive agents that accelerate corrosion of embedded reinforcement. The permeability of concrete is usually greater than the permeability of its two constituents. A plausible explanation could be the effect of the interface flaws linking up with microcracking in the mortar phase of the matrix.

The phenomenon of bleed water collecting and being entrapped under coarse particles of LDA 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 LDA. This has been verified in practice by the examination of the contact zone of LDC split cylinders, as well as by visual examination of sand-blasted vertical surfaces of building structures. This observation should not be surprising because, with structural LDC, the aggregate/matrix interface is a boundary between two porous media, while with NDC concrete there is an abrupt transition at the porous/solid phase interface.

Fagerlund (1972, 1978) has presented several reports that analyze the contact zone in mortars and concretes. 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 to cases involving expanded aggregates in which the boundary between the two phases was virtually indistinguishable due to surface chemistry and intergrowth of the two phases. The fact that the contact zones have maintained their integrity throughout the service life of the structures supports Fagerlund's suggestions and provides reassurance of long-term interaction of the components of the conglomerate.

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 in compression. The simplicity and ease of compression testing has perhaps diverted our 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, i.e., localized yielding and closing of temperature- and volume-change cracks. Weakest link mechanisms, however, are decisive in tensile failures in both dynamic and durability exposure conditions. In many concretes, the weakest link is, in fact, the long-term behavior of the contact zone.

Additionally, a full comprehension has yet to be developed regarding the accommodation mechanism—that process by which the pores closest to the aggregate-matrix interface provide an accessible space for products of various reactions without causing deleterious expansion. While considerable research has identified ettringite, alkali-silica gel, marine salts, and corrosion products in these near-surface pores, there remains the unfinished work of integrating these findings to explain how these products impact structural performance.

Long-Term Field Studies

Since 1978, Natural Resources Canada, through its Canadian Centre for Mineral and Energy Technology (CANMET), has installed 63 LDC prisms at the U.S. Army Corps of Engineers, Treat Island, Maine, exposure site. The specimens are prisms of dimensions 0.305 by 0.305 by 0.914 m (1 by 1 by 3 ft). They are located on a wharf at midtide level so that they are subjected to twice-daily tidal cycles that result in over 100 cycles of freezing and thawing per year. All LDC specimens were air entrained.

A recent paper on this work states that “with normal-weight concrete, there appears to be a potential for the mortar over the aggregates to come off in a sporadic fashion indicating a plane of weakness at the aggregate-cement paste interface. With semi-lightweight [LDC] concrete this is not noted; deterioration occurs by a uniform loss of the surface layer” (Malhotra and Bremner 1996). The paper goes on to report that “at this stage all specimens having cementitious contents of 360 kg/m^3 (607 lb/yd^3) or greater show excellent performance.” An analysis of these data indicates at least equal performance of LDC with NDC when compared at similar ages and with similar binders.

7 Constructibility Considerations

The use of LDC incorporating 100 percent LDA in the concrete ship construction program some 80 years ago is a clear indication that intricate shapes such as heavily reinforced hulls of oceangoing ships and barges were cast successfully. The ship-boiling program was completed in a short time period, when skilled labor was in short supply. Nevertheless, some lessons that should have been learned were not, as is the case with some of the main decks of World War I ships, which have performed poorly. In these ships the decks are relatively thin, heavily reinforced (frequently with large square bars), and with concrete cast in thin horizontal layers that were not consolidated properly. When it came time to construct the over 100 concrete ships and barges in World War II, the same high standard of construction was applied to the hulls. However, in far too many cases, the errors made in the first world war were repeated in the second, as far as deck construction was concerned. Fortunately, we seem to have learned how to construct decks successfully, as revealed by examining the many low-density concrete bridge decks constructed in the past 5 decades (Bremner, Holm, and Morgan 1996).

Subsequent advances in the use of chemical admixtures, pozzolans, and GBF slag cement and greater attention to placing, finishing, and curing of concrete have all enhanced the durability of construction and improved the construction process generally, and for LDC in particular.

Production of Low-Density Concrete

Normal-density aggregates are generally of higher density than the cement paste matrix and, when subjected to vibration, they tend to sink. The opposite occurs with LDA in that the aggregates tend to rise if a concrete mixture lacking cohesion is subjected to improper handling, placement, and consolidation procedures. Usually LDC is cast with a lower slump than NDC (usually in proportion to the reduction in density), and a nominal amount of air entrainment is used, even for concrete not subjected to freezing and thawing. Although LDC needs vibration for proper consolidation, it normally will require a shorter period of vibration than does NDC.

Low-density aggregates have a greater propensity to absorb water from the concrete mixture than do NDAs. As a result, dry aggregates generally are not used in the batching process. Usually, aggregates at a moisture content of at least their 24-hr absorption moisture content can be used with no significant slump loss in the LDC when placing with a crane or a conveyor. When concrete is to be pumped, LDA should be prewetted to a higher degree of saturation that varies with LDA source. The LDA supplier should be consulted for guidance.

Materials for High-Strength Low-Density Aggregate Concrete

In the past few decades, new materials have become available that have significantly improved the ease of concrete construction. In particular, high-range water-reducing admixtures (HRWRAs) have become well accepted by the industry. These materials greatly enhance the flow characteristics of fresh concrete without contributing to segregation. These and other chemical admixtures have made placing LDC less difficult in exacting circumstances.

Cementitious materials other than portland cement (such as fly ash, slag cement, silica fume, calcined clays and shales, and metakaolin) tend to increase the cohesion of the concrete mixture, rendering the concrete less prone to segregate. All of these silica- rich materials are beneficial in that they enhance the impermeability of concrete, which leads to enhanced durability. Also, the replacement of cement with a silica- rich material tends to reduce the maximum temperature of the concrete during the hydration process, which is an important consideration when thick sections are being cast. This is particularly true for HSLDC as its increased thermal resistance compared to that of NDC influences the problem of subsequent cracking when the concrete cools down. Comprehensive data on cements, pozzolans, slag, cement, and admixtures, etc., are reported in ACI 363R.

Cementitious materials

Portland cements used for HSLDC should conform to the requirements of ASTM C 150. Granulated blast furnace slags used as a replacement for portland cement should conform to ASTM C 989.

Pozzolans

Production of HSLDC requires the use of pozzolans. High-quality fly ash meeting the requirements of ASTM C 618 will reduce permeability, improve placing qualities, lower heat use, and improve long-term strength characteristics. Silica fume will improve compressive strength at all ages and also provide significantly greater resistance to chloride penetration.

Fly ash

Pozzolans function very effectively in HSLDC. Pozzolanic activity requires the presence of moisture to enhance the reaction of calcium hydroxide liberated during cement hydration with finely divided silica supplied by the pozzolans. Concretes incorporating mineral admixtures achieved higher strengths than the control concretes. Favorable hydrating environments will be provided for a longer time due to the internal curing provided by the LDA releasing moisture, thus promoting increased activity of the pozzolanic materials.

A comprehensive investigation by Bilodeau et al. (1995) reported the mechanical properties and durability of structural LDC containing high volumes of low-calcium fly ash (about 56 percent of cementitious fractions, by mass). This investigation, following the practice developed at CANMET on concretes incorporating NDA, used HRWRAs to produce concretes with a water-to-(cement + fly ash) ratio of 0.32. Typical mixtures used 115 kg/m^3 (193 lb/yd^3) water, 155 kg/m^3 (261 lb/yd^3) cement, and 215 kg/m^3 (363 lb/yd^3) fly ash. The targeted compressive strength of 35 MPa (5,080 psi) at 28 days age was met. Strength levels greater than 45 MPa (6,530 psi) were reached at 1 year for all the LDAs tested.

This investigation also measured fresh concrete properties (density, slump, air content, time of setting, and bleeding) as well as adiabatic temperature rise. Mechanical properties measured included compressive, flexural, and splitting tensile strengths at various ages, modulus of elasticity, abrasion resistance, drying shrinkage, and creep. A number of specimens were also evaluated for air-void parameters, water permeability, resistance to freezing and thawing, resistance to chloride-ion penetration, and depth of carbonation.

The researchers concluded that the “structural lightweight concrete containing large volumes of fly ash can be produced having satisfactory density ($< 1,900 \text{ kg/m}^3$, 119 lb/ft^3) and adequate compressive strength (35 MPa, 5,080 psi) at 28 days. The concrete so produced has excellent long-term strength properties and durability characteristics.”

Silica fume

The many well-known improvements brought about by the addition of silica fume to NDC are, in general, paralleled in concretes containing structural LDA. Tests reported by Wolsiefer and Clear (1995) demonstrated significantly improved physical properties when silica fume was added to concretes containing structural LDC and SDC.

Because of these enhanced characteristics, structural LDC and SDC have been widely investigated (Hoff 1992, Luthur 1992, Berner 1992) and used in many diverse applications. Examples include bridge deck overlays,¹ garage rehabilitation

¹ Personal Communication, 1997, E. J. Kluckowski, Northeast Solite Corporation, Mt. Marion, NY.

projects (Holm and Bremner 1992), high-strength precast members, offshore platforms (Hibernia Offshore Project), and secondary containment slabs at environmentally sensitive hazardous waste storage sites (Holm 1997).

When silica fume is added to concretes containing structural LDA, all strength properties are improved: compressive, tensile, flexure, shear, etc. In general, the strength improvements with LDC reach an earlier strength ceiling than is the case with NDC. The strength ceiling is that point in mixture composition where additional reduction in w/c provides very small improvements in strength. Most (but not all) NDAs have particle strengths greater than LDA. Although LDAs are composed of a very strong vitreous ceramic, the aggregate porosities may be 50 percent or greater. That being the case, the strength “ceiling” is generally (but not always) reached at lower binder quantities. After reaching the strength ceiling, LDCs containing silica fume have a lower increase in strength slope than that associated with most NDCs.

When silica fume is added to concrete containing either NDA or LDA, its small particle size and high surface area results in an increased water requirement. To compensate for the increased water requirement, HRWRAs are always used in concretes containing silica fume. The overall net effect can be a reduced water requirement if enough HRWRA is used. In fact, this is often the case, especially when the desired W/C_m is 0.40 or less. While this extra water reduction is beneficial in that it helps to keep the total cementitious content from becoming excessive, there is also a negative side to large water reductions. Reducing the absolute volume of free water (which has the lowest specific gravity of all the concrete ingredients) will result in increased fresh and equilibrium densities of the concrete. In some applications an increased volume of LDA may provide compensation, but the usual approach is for designers, precasters, contractors, etc., to design to higher densities. This increase in density may be approximately 3 percent, but in some nonexposed applications where the air content is lower than that required for durability considerations, the increase may be as much as 5 percent.

In a manner comparable to earlier statements regarding the strength ceiling, the elastic modulus of LDC containing silica fume will increase but eventually reach a practical limiting value. This occurs because the LDA has a higher porosity, and thus lower rigidity, than the surrounding silica fume-enhanced matrix.

One comprehensive investigation (Holm and Bremner 1992) that used high volumes of silica fume added to LDC mixtures demonstrated substantial permeability reduction, as well as strength improvement by 43 percent over mixtures not containing silica fume. The chloride permeability of the mixture containing silica fume, as measured by ASTM C 1202, was only 8 percent that of the control concrete (Table 14). The results of this cooperative research program show a dramatic improvement in matrix properties.

**Table 14
Physical Properties, Strength, and Chloride-Ion Permeability
of Structural Lightweight Concrete**

Mixture (+/- denotes addition/absence of silica fume)		1+	2+	3+	4+	5+	5-
Fresh concrete properties	Wet density kg/m ³ (lb/ft ³)	1,870 (116.5)	1,930 (120.5)	1,890 (117.9)	1,870 (116.5)	1,890 (118.2)	1,870 (116.8)
	Slump mm (in.)	190 (7.5)	215 (8.5)	215 (8.5)	225 (8.8)	205 (8.0)	150 (6.0)
Compressive strength at age, in Days MPa (psi)	2	43.9 (6,360)	33.4 (4,850)	28.8 (4,180)	40.8 (5,920)	40.0 (5,800)	19.9 (2,890)
	7	57.0 (8,270)	39.6 (5,740)	36.6 (5,310)	51.5 (7,470)	48.2 (6,990)	27.3 (3,960)
	28	66.5 (9,648)	47.2 (6,840)	41.7 (6,050)	59.3 (8,600)	51.0 (7,460)	34.8 (5,050)
	90	68.0 (9,855)	45.2 (6,550)	42.6 (6,225)	62.0 (8,990)	51.9 (7,525)	36.3 (5,270)
W/C _m		0.36	0.33	0.35	0.31	0.32	0.43
Charge passed (coulombs)		260	450	450	220	370	4,800

Note: Mixtures 1 through 5 incorporated five different sources of structural low-density coarse aggregates.

It is well known that thorough curing procedures must follow immediately after placing NDC containing silica fume. Because of the significant reduction in bleeding in NDC containing silica fume, immediate curing is extremely important. Because the high levels of absorbed water contained within vesicular aggregate available for extended internal curing, LDC containing silica fume is somewhat more forgiving of less than optimum curing procedures. Carefully developed curing specifications based upon the experience developed in placing NDC containing silica fume should be similarly followed for placements with LDC, with the advantages of internal curing considered only as an additional safety factor. The best source of information on the properties and performance of LDC containing silica fume is generally available through the experience obtained by local producers of LDA.

Admixtures

When used in HSLDC, water-reducing admixtures offer reduced water demand, enhanced durability, and improved workability in a manner comparable to that of HSNDC. Water-reducers, retarders, and high-range water-reducers should conform to ASTM C 494 and be dosed according to manufacturers' recommendations. LDC mixtures normally contain entrained air, which serves to increase the cohesiveness of the mixture and to make the concrete more resistant to the effects of freezing and thawing when in a wet environment. When freezing and thawing durability is not a consideration, then small amounts of entrained air (3 to 5 percent) are still recommended for workability. Entrained-air volumes should meet the requirements of ACI 201.2R according to the severity of the exposure conditions. While air entrainment may diminish the strength-producing characteristics of the cementitious

matrix, it will also lower water and sand volumes necessary to achieve satisfactory workability, with the net effect being only a modest reduction in the strength of HSLDC.

Coarse aggregate

HSLDCs normally use only coarse LDA. As reported earlier, most but not all HSLDC mixtures require a reduction of the LDA top size, particularly for concrete strengths greater than 48- to 70-MPa (7,000- to 10,000-psi) range. Certain LDAs, however, because of the strength of the vitreous ceramic enveloping the pores, have routinely used the 19.0 to 4.75-mm (3/4 in. to No. 4) grading in production of high-strength precast concrete for more than 4 decades. Most LDA manufacturing plants will limit coarse aggregate to two sizes to minimize production and stockpiling problems, but these plants will often make other gradings if project volumes warrant.

Gradings of 19.0 to 4.75 mm (3/4 in. to No. 4) or 12.5 to 4.75 mm (1/2 in. to No. 4) will normally be appropriate for commonly sized HSLDC members, while 9.5 to 2.36 mm (3/8 in. to No. 8) gradings may be necessary in highly reinforced members to allow for adequate placement conditions.

Fine aggregate

LDC normally incorporates normal-density sand as the fine-aggregate fraction. Quality criteria developed for sands used in HSNDC (e.g., fineness modulus of about 3.0 for optimum workability and strength, etc.) are identical to those used in HSLDC mixtures.

Proportioning of Concrete Mixtures

In general, proportioning rules and techniques used for NDC mixtures apply to LDC, with added attention given to the influence of the water absorption characteristics of the LDA (Table 15). Structural-grade LDCs are generally proportioned by absolute volume methods in which the fresh concrete produced is considered equal to the sum of the absolute volumes of cement, aggregates, net water, and entrained air. Proportioning by this method requires the determination of absorbed and adsorbed moisture contents and the as-used particle density of the separate sizes of aggregates. An often-used alternative to the absolute volume procedures is to proportion LDC mixes by the damp loose-volume method (ACI 211.2).

Specifications for structural-grade LDC usually require minimum values for compressive and tensile splitting strength, maximum limitations on slump, specified ranges of air content and, finally, a limitation on maximum fresh density. Reduction of concrete's high density leads to improved structural efficiency and is, therefore, an important consideration in proportioning LDC mixtures. While density reduction

Table 15
Basic Mixture Proportion Criteria for Structural Low- and Normal-Density Concrete
Exposed to a Marine Environment

Mixture Criteria	NDC	LDC	Comments
Cementitious materials (cement, fly ash, silica fume) kg/m ³ (lb/yd ³)	335-445 (560-750)	360-445 (610-750)	For equal strengths LDC may require additional 25-80 kg/m ³ (15-50 lb/yd ³) of cement
Water-cementitious materials ratios (W/C _m) or minimum compressive strength	<0.40	34.5 MPa (>5,000 psi)	ACI 318 requirements
Nominal size of coarse aggregate, mm (in.)	4.75-25.0 (No. 4 - 1 in.)	4.75-19.0 4.75-12.5 (No. 4 - ¾-in.) (No. 4 - ½-in.)	Check with local LDA producers
Coarse aggregate absolute volume	– 35%	– 35%	Influence of coarse aggregate on workability, other factors similar
Water absorption of coarse aggregate	Generally <1%	5 to 20%	Water absorbed by coarse aggregate must be accurately determined for control of strength and density
Water absorbed on coarse aggregate	Absorbed water affects W/C _m of both LDC and NDC and must be accurately measured and accounted for in W/C _m determination.		
Admixtures and other cementitious materials: Water-reducing admixtures HRWRA Retarding admixtures Fly ash Silica fume Other pozzolans GGBFS ¹ Air-entraining admixture	Admixture and pozzolans or slag influence the properties of the mortar fraction of the concrete. The many advantages and construction benefits obtained through their use in NDC are essentially paralleled in LDC, and their incorporation in structural concrete enclosed to a marine environment is strongly recommended. A LDC is air entrained whether or not it is to be exposed to freezing and thawing.	(See NDC)	(See NDC)

¹ Ground granulated blast-furnace slag.

depends primarily on the particle density of the LDAs, it is also influenced to a lesser degree by cement, water, and air contents, and the ratio of coarse-to-fine aggregate.

When expanded aggregates contain levels of absorbed moisture equal to or greater than those developed after a 1-day immersion, the rate of absorption will be very low. Under these moist conditions LDC may be batched, placed, and finished with the same facility as their NDC counterparts. Under these conditions, water-cement ratios, while not normally specified, may be established with the same precision as concretes containing NDA. Water absorbed within the LDA prior to mixing is not available for calculating the volume of cement paste at the time of

setting. This absorbed water is available, however, for continued cement hydration after external curing has ended. The general practice is to proportion LDC mixtures on the basis of a cement content at a given slump.

As with NDC, air entrainment in LDC significantly improves durability and resistance to scaling. In concretes made with angular LDAs, it is also an effective means of improving workability of otherwise harsh mixtures. With moderate air contents, bleeding and segregation are reduced and mixing water requirements are lowered while maintaining optimum workability. Because of the elastic compatibility of the LDA and cementitious binder phases, strength-reduction penalties due to high air contents will be lower for structural LDC than for NDC (Bremner and Holm 1986). Recommended ranges of total air content of usual structural LDCs are given in Table 16.

Table 16 Total Air Content of Low-Density Concretes	
Nominal Maximum Size of Aggregate	Air Content, % by Volume
19.0 mm (3/4 in.)	4 to 8
9.5 mm (3/8 in.)	5 to 9

Air content of LDC is determined in accordance with the procedures of ASTM C 173. Volumetric measurements assure reliable results while pressure meters provide erratic data due to the influence of aggregate porosity and should not be used.

Air contents higher than those required for durability considerations are frequently used for high thermal resistance, or for lowering the density of semistructural fill concrete, with reduced compressive strength as a natural consequence.

Mixing, Placing, Finishing, and Curing

When properly proportioned, structural LDC can be delivered and placed with the same equipment as NDC. The most important consideration in handling any type of concrete is to avoid separation of coarse aggregate from the mortar fraction. Basic principles required to secure a well-placed LDC include

- a. Well-proportioned, workable mixtures that use a minimum amount of water.
- b. Equipment capable of expeditiously moving the concrete.
- c. Proper consolidation in the forms.

d. Quality workmanship in finishing.

Well-proportioned structural LDC can be placed and screeded with less physical effort than that required for NDC. Excessive vibration should be avoided, as this practice serves to drive the heavier mortar fraction down from the surface where it is required for finishing. On completion of final finishing, curing operations similar to those for NDC should begin as soon as possible; however, membrane-forming curing compounds should not be applied until bleeding has stopped. LDCs with aggregates having high absorptions carry their own internal water supply for curing, and as a result are more forgiving to poor curing practices or unfavorable ambient conditions. This internal curing water is transferred from the LDA to the mortar phase as evaporation takes place on the concrete surface. This action maintains a continuous moisture balance by replacing moisture essential for an extended continuous hydration period.

Pumping

If not highly saturated, low-density aggregates will absorb part of the mixing water when exposed to increased pumping pressures. To avoid loss of workability, it is essential to raise the presoak absorption level of LDA prior to pumping. Presoaking is often accomplished at the aggregate production plant where uniform moisture content is achieved by applying water from spray bars directly to the aggregate moving on belts. This moisture content can be maintained and supplemented at the concrete plant by stockpile hose and sprinkler systems.

Presoaking will significantly reduce the LDA rate of absorption, minimizing water transfer from the mortar fraction and lowering slump loss during pumping. Higher moisture contents developed during presoaking will result in increased particle density that, in turn, develops higher fresh concrete density. Higher water content due to presoaking will eventually diffuse out of the concrete, developing a longer period of internal curing as well as a larger differential between fresh and equilibrium unit weight than that associated with LDC placed with lower aggregate moisture contents. Aggregate suppliers should be consulted for mixture proportioning recommendations necessary for consistent pumpability. These recommendations should include minimum levels of aggregate absorption, minimum slump of concrete prior to the addition of HRWRAs, and suggestions regarding pozzolans and other chemical admixtures.

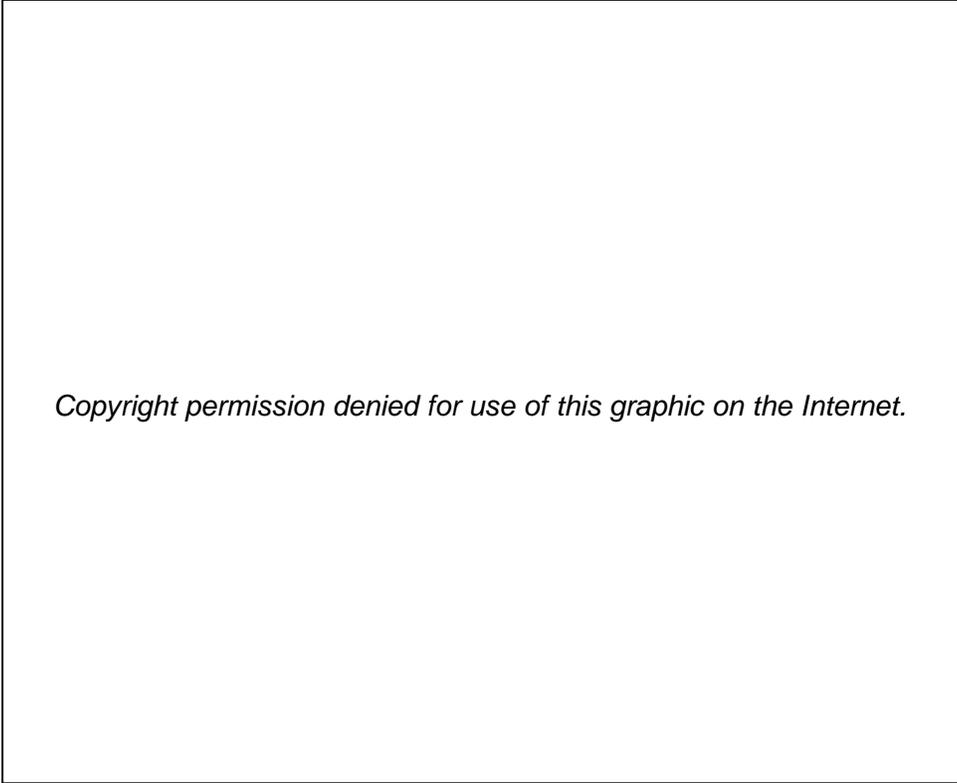
The addition of water-reducing admixtures into LDC mixtures has proven to be significantly useful in applications where the concrete had to be pumped long distances or to great heights. Job site experience has, however, demonstrated the need for a minimum slump, of about 75 mm (3 in.) prior to the addition of water-reducing admixtures. Concretes that start with a slump less than 75 mm will be difficult to pump, despite having high slumps (<200 mm, 8 in.) after the addition of the water-reducing admixtures.

In addition to mixture proportions, ACI 213.3R includes the following recommendations for the pumping system:

- a. Use the largest pump line possible with a preferable minimum of 125 mm (5 in.).
- b. Prior to pumping, ensure that all lines are clean, the same size, and “battered” with grout at the start.
- c. Avoid rapid size reduction from the pump to the line.
- d. Reduce operating pressures by limiting rate of placement, limiting the number of bends, using steel lines and as short a run of rubber lines as possible, and ensuring all lines are firmly braced and tightly joined and gasketed.

Following these recommendations, the concrete mixture No. 2 (shown in Table 17) was pumped 250 m (830 ft) to the 60 floors at the Nations Bank project in Charlotte, NC (see Figure 23). Several buildings over 60 stories have been successfully pumped with LDC.

Mixture No.	1		2 ¹		3	
Mixture Proportions:						
Cement, Type III, kg/m ³ (lb/yd ³)	250	(550)	295	(650)	341	(750)
Fly ash, kg/m ³ (lb/yd ³)	64	(140)	64	(140)	64	(140)
LDA 20 mm to #5, kg/m ³ (lb/yd ³)	409	(900)	409	(900)	409	(900)
Sand, kg/m ³ (lb/yd ³)	623	(1,370)	585	(1,287)	547	(1,203)
Water, L/m ³ (gal/yd ³)	134	(35.5)	138	(36.5)	141	(37.2)
WRA, L/m ³ (fl oz/yd ³)	0.78	(27.6)	0.90	(31.6)	1.01	(35.6)
HRWRA, L/m ³ (fl oz/yd ³)	1.56	(55.2)	2.31	(81.4)	2.27	(80.1)
Fresh Concrete Properties:						
Initial slump, mm (in.)	63	(2 1/2)	51	(2)	57	(2 1/4)
Slump after HRWRA, mm (in.)	140	(5 1/8)	191	(7 1/2)	171	(6 3/4)
Unit weight, kg/m ³ (lb/ft ³)	2.5	--	2.5	--	2.3	--
	1,887	(117.8)	1,890	(118.0)	1,890	(118.0)
Compressive Strength, MPa (psi):						
4 days	29.6	(4290)	35.2	(5,110)	39.4	(5,710)
7 days	33.6	(4870)	39.9	(5,790)	44.4	(6,440)
28 days (avg.)	43.2	(6270)	47.0	(6,810)	51.4	(7,450)
Splitting Tensile Strength, MPa (psi)	3.59	(520)	3.72	(540)	3.90	(565)
¹ Mixture selected and used on project.						



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Figure 23. Nations Bank, Charlotte, NC

Testing

In most instances, test procedures for measuring properties of LDC are the same as commonly used for NDC. In limited cases, special test procedures particularly suited to measure LDC characteristics have been developed, as for example, ASTM C 173.

Laboratory testing programs

Systematic laboratory investigations into the physical and engineering properties of HSLDC are too numerous to be elaborated here. Most early programs extending strength/density relationships were conducted by LDA manufacturers and innovative precast concrete producers striving for high early-release strengths, longer span flexural members, or taller one-piece precast columns (Holm 1980a). These in-house programs developed functional data directly focused on specific members supplied to projects. In general, project lead-times were short, the practical considerations of shipping and erection were immediate, and mixtures were targeted toward satisfying specific job requirements. This type of research brought about immediate incremental progress but, in general, was not sufficiently comprehensive.

Unfortunately, some investigations did not take advantage of the advanced admixture formulations or pozzolans and slag (i.e., HRWRA, silica fume, fly ash, ground granulated blast-furnace slag) that significantly improve matrix quality, and as such provide data of no commercial value. These investigations, as well as others incorporating unrealistic mixtures, inappropriate LDA, or impractical density combinations, are not reported.

Special requirements of offshore concrete structures have now brought about an explosion of practical research into the physical and engineering properties of HSLDC. Several large, initially confidential joint-industry projects have become publicly available as the sponsors release data according to an agreed-upon timetable. These monumental studies, one of which was summarized by Hoff (1992), are widely referred to throughout this report. In addition to providing comprehensive physical property data on HSLDC and HSSDC, these programs developed innovative testing methods: revolving disc tumbler and sliding contact ice-abrasion wear tests, freeze/thaw resistance to spectral cycles, and freeze bond testing techniques, etc., which measured properties unique to offshore applications in the Arctic.

Major North American laboratory studies into properties of HSLDC include those conducted at or sponsored by Expanded Shale, Clay, and Slate Institute (1960); Malhotra (1981, 1987); Seabrook and Wilson (1988); Ramakrishnan, Bremner, and Malhotra (1991); Berner (1992); and Luther (1992). Because of their special structural needs, much work has been conducted by Norwegian sources, with additional important contributions from other Russian, German, and UK sources, some of which have been referenced by Holm and Bremner (1994).

It has been estimated that the cost for these commercially supported research programs investigating the physical and structural properties of HSLDC has exceeded several million dollars (Hoff 1992). While much research has been already effectively transferred into actual practice on current projects, there remains a formidable task of analyzing, digesting, and especially codifying this immense body of data into design recommendations and code standards.

Laboratory and field control

Changes in LDA moisture content, grading, or particle density, as well as usual job site variation in entrained air, suggest frequent checks of the fresh concrete to facilitate adjustments necessary for consistent concrete characteristics. Standardized field tests for slump, fresh unit weight, and air content should be employed to verify conformance of field concretes with mixtures developed in the laboratory and the project specifications. Sampling should be conducted in accordance with ASTM C 172 and ASTM C 173. The ASTM describes procedures for calculating the in-service, equilibrium density of structural LDC. In general, when variations in fresh density exceed ± 3 percent, an adjustment in batch weights may be required to restore specified concrete properties. To avoid adverse effects on durability, strength, and workability, air content should not vary more than ± 2.0 percent from specified values.

8 Applications

Applications of High-Strength Low-Density Concrete

HSLDC with compressive strength targets ranging from 35 to 41 MPa (5,000 to 6,000 psi) has been successfully used for almost four decades by North American precast and prestressed concrete producers. Presently there are ongoing investigations into somewhat longer span lightweight precast concrete bridges that may be feasible from a trucking/lifting/logistical point of view. Parking structure members with 15- to 20-m (50- to 63-ft) spans are generally constructed with double tees composed of LDC with air-dry density of approximately $1,850 \text{ kg/m}^3$ (115 lb/ft^3) (Figure 24).

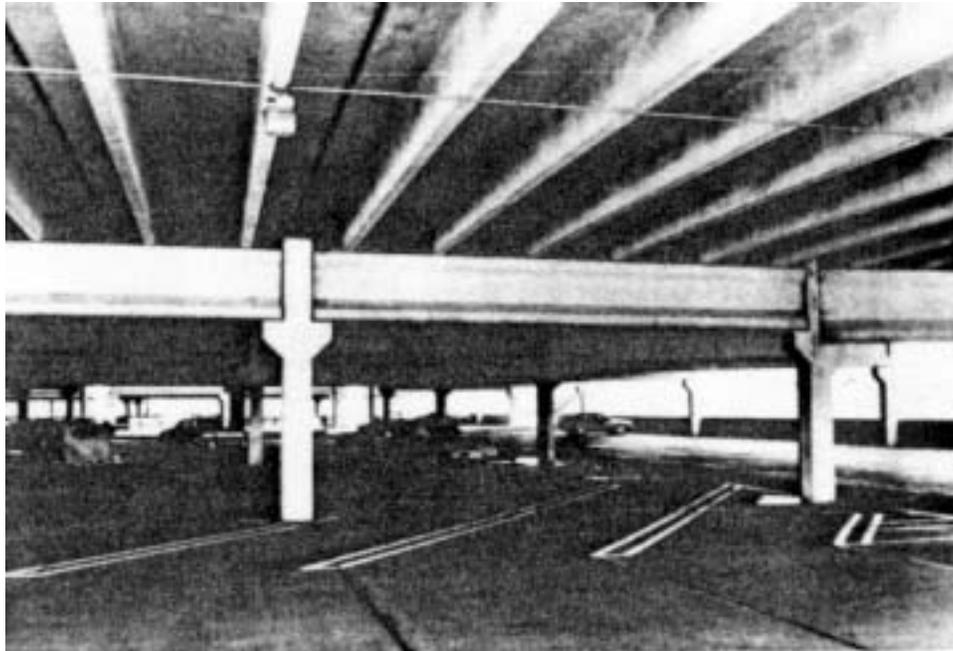


Figure 24. Typical precast lightweight concrete parking structure (Holm 1980b)

Reduction in mass is primarily for lifting efficiencies and lower transportation costs. One prestressed parking structure project is of interest from the perspective of the precast producer's quality-control adjustments of the mixture components. These adjustments were made after statistical studies of the plant's first use of HSLDC indicated unduly high strengths. The first series of statistical tests were on a mixture that included 450 kg/m^3 (755 lb/yd^3) of cement and produced compressive strengths of 51 MPa (7,450 psi) at 7 days age and compressive strengths in excess of 62 MPa (9,000 psi) at 28 days age. Reducing the cement content to 429 kg/m^3 (705 lb/yd^3) resulted in a compressive strength of 54.5 MPa (7,910 psi) at 28 days age. A further reduction of the cement content to 390 kg/m^3 (660 lb/yd^3) resulted in a compressive strength of 52 MPa (7,500 psi) at 28 days, which was near the specified strength at 28 days.

Buildings

The first major New York City building application of post-tensioned floor slabs was the 140-m (450-ft) multipurpose Federal Office Building, constructed in 1967 with five Post Office floors and 27 office tower floors. Concrete tensioning strengths of 24 MPa (3,500 psi) were routinely achieved at 3 days for the 9- by 9-m (30- by 30-ft) floor slabs with a design target strength of 41 MPa (6,000 psi) at 28 days. Approximately $23,000 \text{ m}^3$ ($30,000 \text{ yd}^3$) of structural LDC was incorporated into the floors, and the cast-in-place architectural envelope serves a structural as well as aesthetic function. Despite the polluted urban atmosphere, the buff-colored concrete has maintained its handsome appearance (Figure 25) (Holm and Bremner 1994).

The North Pier Tower (Chicago-1991) used HSLDC in the floor slabs as an innovative structural solution to avoid construction problems associated with the load transfer from HSNDC columns through the floor slab system. ACI 318 requires differences in compressive strength between column concrete—which in this project was 62 MPa (9,000 psi)—and the intervening floor slab concrete to be less than a ratio of 1.4. By using HSLDC in the slabs with a strength greater than 44 MPa (6,430 psi), the floor slabs could be placed using routine techniques, thus avoiding scheduling problems associated with the “mushroom” technique (Figure 26). In this approach, high-strength column concrete is overflowed from the column and intermingled with the regular-strength floor slab concrete. The technique used in the North Pier project avoids delicate timing considerations that are necessary to avoid cold joints.

Bridges

Of the more than 800 LDC bridge decks constructed throughout North America, most have been produced with concretes at higher than usual commercial-strength levels. The Sebastian Inlet Bridge, which used extra-long HSLDC in the precast, prestressed drop-in spans during its construction in 1965, is included in one LDA supplier's listing of more than 140 completed bridges. Transportation engineers generally specify higher concrete strengths on bridge decks, primarily to

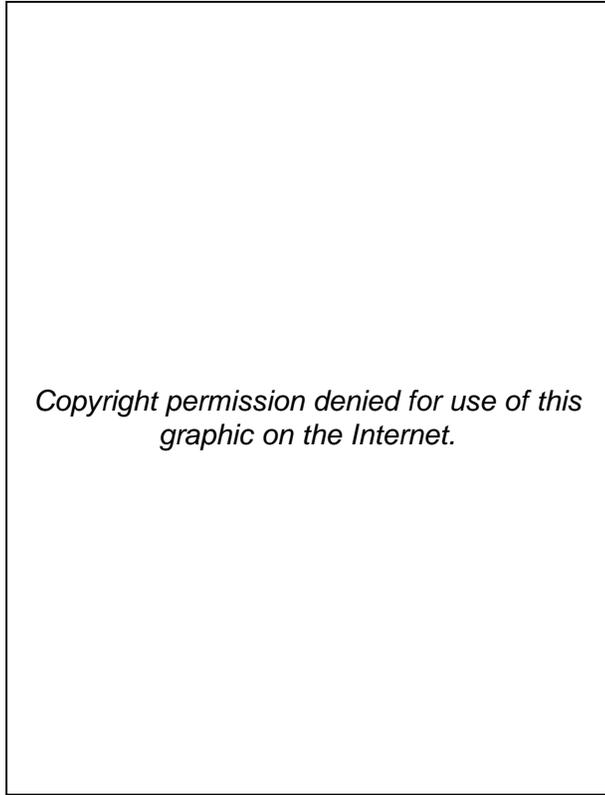


Figure 25. Federal Post Office and office building in New York, constructed in 1967

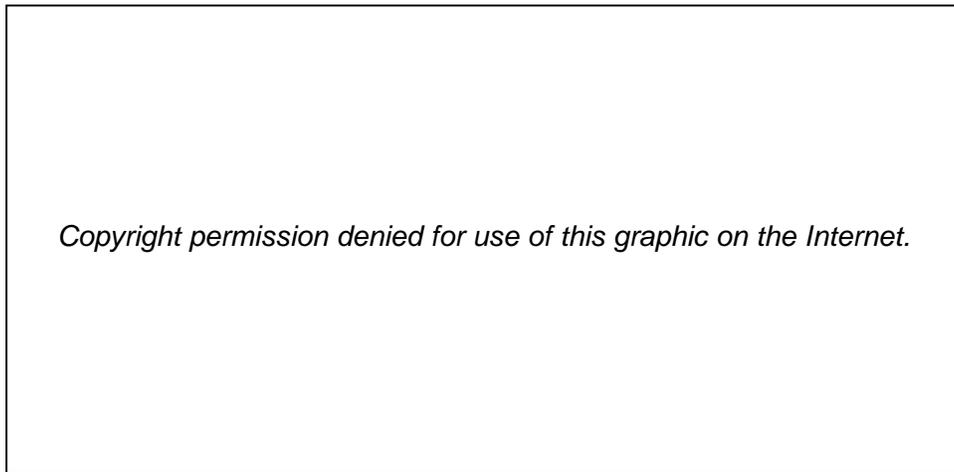


Figure 26. Alternative construction schemes for transfer of high-strength, normal-density concrete column loads through floor slabs

ensure high-quality mortar fractions (high strength combined with high air content) that will minimize maintenance costs. One mid-Atlantic state transportation authority has completed more than 20 bridges using HSLDC using a target strength of 36 MPa (5,200 psi), 6 to 9 percent air content, and an air-dry equilibrium density of 1,850 kg/m³ (115 lb/ft³). Recent studies have identified tens of thousands of bridges in the United States that are functionally obsolete, with low load capacity, unsound concrete, or insufficient number of traffic lanes. To remedy limited lane capacity, Washington, DC, engineers have replaced a four-lane bridge originally constructed with NDC with five new lanes constructed with LDC. This construction has provided a 50-percent increase in one-way, rush-hour traffic without replacing the existing structure, piers, or foundations. Perhaps the best testimony to successful performance is repeated usage, typified in Figure 27, which shows the first Chesapeake Bay Bridge constructed with LDC decks in 1953, followed by the parallel span built in 1975.

Bridges using both low- and normal-density concrete

For a number of bridges, HSLDC has been used to achieve unsymmetrical free cantilever construction. On the Sandhornoya Bridge, completed in 1989 near the Arctic Circle city of Bodo, Norway, the 110-m (360-ft) sidespans of a three-span bridge were constructed with HSLDC with a cube strength of 55 MPa (7,975 psi). The center span of 154 m (505 ft) used NDC with a cube strength of 45 MPa (6,525 psi). The mixture proportions are shown in Table 18 (Fergestad 1996).

Table 18 Mixture Proportions for Low-Density Concrete Used on Sandhornoya Bridge		
Material	Mass (kg/m³)	Mass (lb/yd³)
Cement	400	671
Silica fume	25	42
Sand	575	964
LDA 4 - 8 mm	250	419
LDA 4 - 16 mm	400	671
Water in admixture (B, R, R)	28	47
Water added	144	241
Slump	200 mm	8 in.
Fresh density	1,850 - 1,900 kg/m ³	116 - 119 pcf
Compressive strength, 100 mm cube, 28 days	59.8 MPa	8,670 psi

Elevations and cross sections of the Sandhornoya Bridge are shown in Figure 28.

Contrasting with the Sandhornoya Bridge, another prestressed box-girder bridge north of the Arctic Circle at Stovset used HSLDC only in the 146-m (479-ft) center section of the 220-m (720-ft) center span. This approach minimized the unbalance between the center span and the adjacent 100-m (328-ft) sidespans.



Figure 27. Chesapeake Bay Bridges, Annapolis, MD

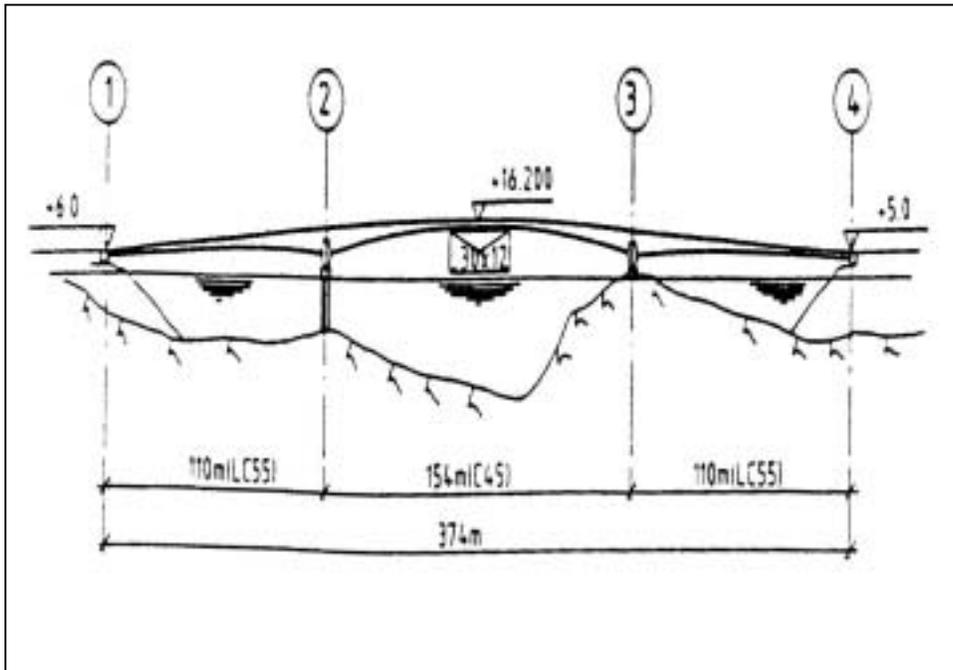


Figure 28. Sandhornoya Bridge, Norway (Fergestad 1996)

Marine Structures

Because many offshore concrete structures will be constructed in shipyards located in lower latitudes and then floated and towed to the project site, there is a special need to reduce mass and improve the structural efficiency of the cast-in-place structure. An additional consideration is that shallow-water conditions mandate lower draft structures. Therefore, the submerged-density ratio of

$$\frac{\text{HSNDC}}{\text{HSSDC}} = \frac{2.50 - 1.00}{2.00 - 1.00} = 1.50$$

which is greater than the air-density ratio

$$\frac{2.50}{2.00} = 1.25$$

becomes increasingly important.

Tarsiut Caisson retained island

The first Arctic structure using LDA was the Tarsiut Caisson retained island, built in 1981 by Dome Petroleum/Gulf Oil and barged to the Canadian Beaufort Sea. Four large prestressed concrete caissons, 69 by 15 by 11 m high (226 by 50 by 35 ft), were constructed in a graving dock in Vancouver, towed around Alaska on a submersible barge, and founded on a berm of dredged sand 40 km (25 miles) from land in the shear zone winter landfast ice and the moving Arctic ice. The space between the four caissons was then filled with dredged sand to form the working platform for the drill rig. When combined with the extremely high concentration of reinforcement, the resulting density was 2,240 kg/m³ (140 lb/ft³) (Figure 29).

Concrete island drilling system

The Tarsiut Caisson retained island project was followed in 1984 with the use of HSLDC to construct the concrete island drilling system, which was built in Japan and also towed to the Beaufort Sea (Fiorato, Person, and Pfeiffer 1984). In addition to reducing draft during construction and towing, use of HSLDC in offshore gravity-based structures can be justified by the improved floating stability as well as the opportunity to carry more topside loads. A large part of the intermediate level of this structure was constructed with HSLDC. Compressive strength was 45 MPa (6,500 psi), and the density was 1,840 kg/m³ (115 lb/ft³).

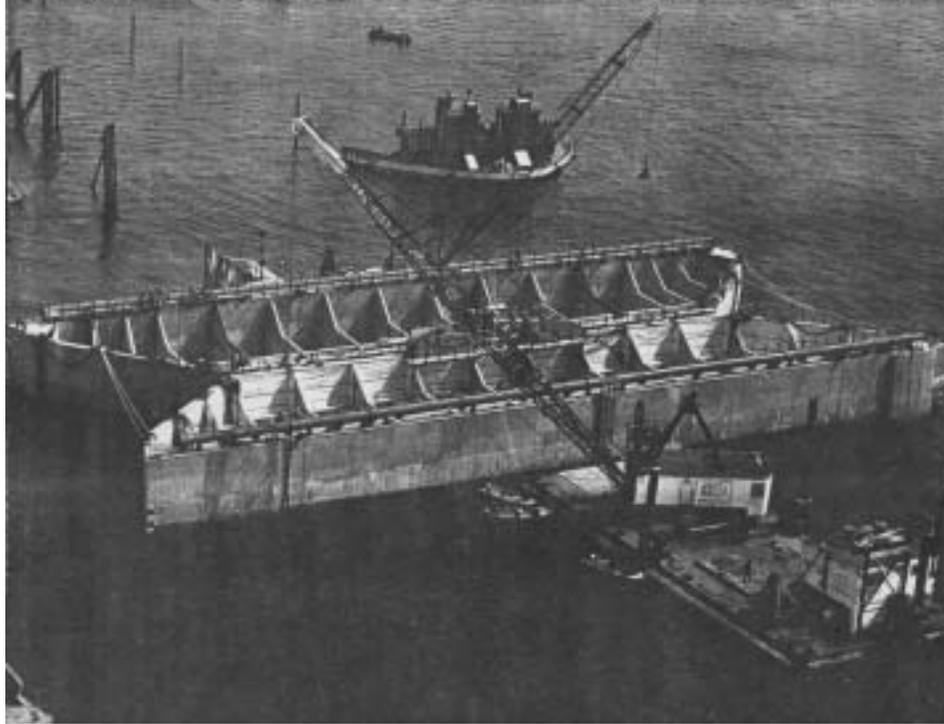


Figure 29. Tarsuit Caisson retained island (from *Concrete International* 1982, with permission of ACI)

Floating bridge pontoons

High-strength low-density concrete was used very 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. The pontoons are 42 m (138 ft) long and 20.5 m (67 ft) wide and were cast in compartments separated by watertight bulkheads. 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. Of interest are the design requirements, which included the following criteria:

- ∅ 100 mm as the minimum height of the compression zone in any cross section in order to ensure watertightness.
- ∅ maximum crack width of 0.2 mm in the splash zone and 0.5 mm for the remainder.

A similar floating bridge structure completed in 1992 near Kristiansund, Norway, consisted of steel trusses supported by LDC pontoons ($L \times W \times H = 34 \times 20 \Delta 5.7$ m) (112 Δ 66 Δ 19 ft) with a cross section as shown in Figure 30. The high-quality concrete combined with a 50-mm (2-in.) cover for the reinforcement results in a design life estimated at 100 years.

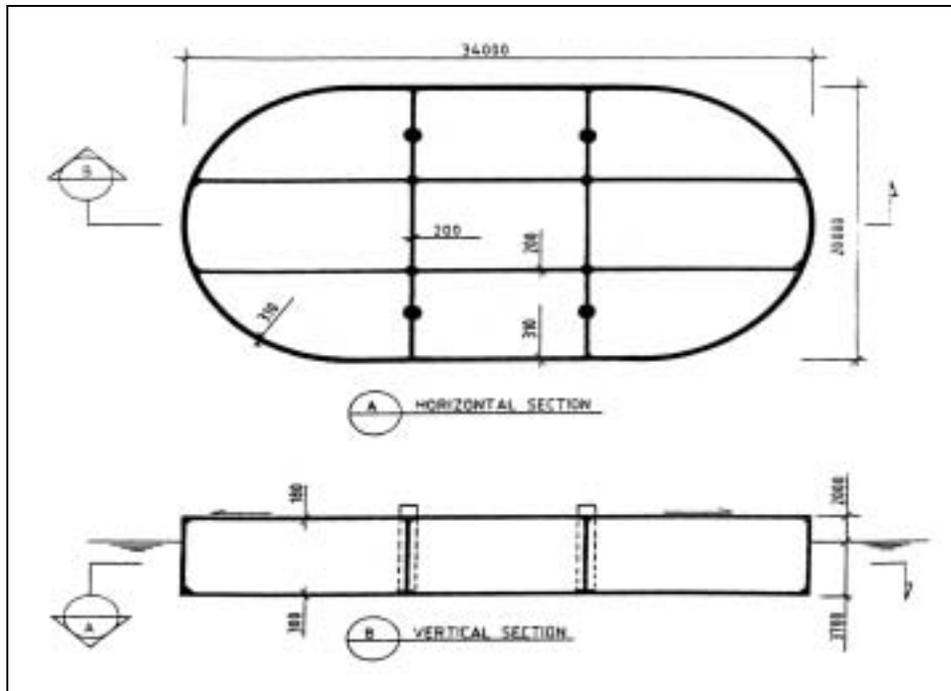


Figure 30. Typical low-density concrete pontoon, Bergsoysundet Bridge, Kristiansund, Norway (Fergestad 1996)

Hibernia offshore platform, Newfoundland, Canada¹

To improve the buoyancy of the largest floating structure ever built in North America, LDA replaced approximately 50 percent of the coarse aggregate fraction of the HSSDC in the gravity base structure. This structure, having a mass of more than 1 million tons, was successfully floated out of the drydock and towed to a nearby deepwater harbor area where construction continued. It was then towed to the Hibernia Oil Field site and set in place on the ocean floor (Figure 31). A comprehensive testing program evaluating the results of numerous mixture compositions was reported by Hoff et al. (1995). The fresh and hardened properties of the SDC are shown in Table 19. Results of the tests on a number of alternative mixture proportions are shown in Table 20.

Heidron floating concrete platform, North Sea, Norway

Because of the deep water (345 m, 1,130 ft) overlaying the Heidron oil fields, and due to the prior experience of the operator with an earlier tension-leg steel floating concrete structure, a decision was made to construct the first floating platform with HSLDC. Because of the need to achieve the required buoyancy, the concept of using HSLDC was introduced early in the planning stages. The hull of the floater was constructed entirely of LC60 (60-MPa, 8,700-psi cube strength).

¹ Personal Communication, April 1999, Ken Harmon, Carolina Stalite Company, Salisbury, NC.



Figure 31. Hibernia offshore platform (photo courtesy of Carolina Stalite Company)

Table 19 Fresh and Hardened Properties of the Specified-Density Concrete Used in the Hibernia Platform	
Physical Properties	
<i>Fresh Properties</i>	
Density	2.170 kg/m ³ (135.4 lb/ft ³)
Air content	2.1%
Slump	210 mm (8.25 in.)
Water/cement	0.33
<i>Hardened Properties (28 Days)</i>	
Compressive strength	79.9 MPa (11,588 psi)
Splitting tensile strength	5.87 MPa (851 psi)
Modulus of elasticity	30.5 GPa (4.4 x 10 ⁶ psi)
Poisson's ratio	0.22

Almost 70,000 m³ (91,000 yd³) of HSSDC [with a maximum density of 2,000 kg/m³ (125 lb/ft³) and a required elastic modulus of 22 GPa (3.19 Δ 10⁶ psi), incorporating 0.15 percent polypropylene fibers] was placed in the concrete above the waterline for increased resistance to spalling in the event of a hydrocarbon fire.

**Table 20
Physical Properties of Other Specified-Density Concrete
Alternatives Tested and Evaluated for the Hibernia Offshore Oil
Platform**

Mix ID	LDA, as a Replacement of Coarse Normal Density Aggregate	Test Results: Hardened Concrete			
		Unit Weight		Compressive Strength	
	(%)	(kg/m ³)	(lb/ft ³)	(MPa)	(psi)
LDC-2	100	2,020	126	70.9	10,281
LDC-7	75	2,140	134	74.9	10,861
LDC-8	50	2,230	139	79.2	11,484
LDC-9	25	2,320	145	87.8	12,731
LDC-6		2,410	150	89.8	13,021
Mix ID	Splitting Tensile Strength		Air Content	Elastic Modulus	
	(MPa)	(psi)	(%)	(GPa)	(psi x 10 ⁶)
LDC-2	4.6	667	2.3	28	4.1
LDC-7	5.4	783	1.6	30	4.4
LDC-8	5.5	798	1.8	33	4.8
LDC-9	6.1	885	1.8	35	5.1
LDC-6	5.4	783	1.6	37	5.4

Rehabilitation of bridges and parking decks

Numerous opportunities exist for the efficient rehabilitation of existing deteriorated bridges and parking decks. For example, replacing 75 mm (3 in.) of deteriorated NDC with 100 mm (4 in.) of low-permeability HSLDC will also provide opportunities to improve deficient surface geometry (for example, increasing slopes for drainage and improving superelevation on curves). There are many examples of bridges that have upgraded load limits or added additional traffic lanes when the NDC decks were replaced with LDC decks.

9 Economics, Material Costs, Transportation, and Environmental Considerations

The high-temperature manufacturing costs, combined with the large capital investment required for a modern rotary-kiln plant that is similar in many respects to a cement production plant, result in production and selling costs higher than NDA obtained from quarries, deposits, and dredging. An engineering analysis that takes into account the greater material costs often indicates value-added features that overcome this first-cost difference. Indeed, in many instances, the use of LDC will make a project practical or feasible. Such has been the case for HSLDC used in major marine projects located in severe environments, as for example the offshore platforms constructed in the Arctic.

Structural Efficiency of Low-Density Concrete

The entire hull structure of the USS *Selma* (and of the more than 100 subsequent ships) was constructed with HSLDC in a shipyard in Mobile, AL, and launched in 1919. The concrete strength/density (S/D) ratio (structural efficiency) of concrete used in the USS *Selma* was extraordinary for that time (Holm 1980a). Improvements in structural efficiency of concrete since that time are shown schematically in Figure 32, revealing upward trends in the 1950s with introduction of prestressed concrete, followed by production of HSNDC for columns of very tall cast-in-place concrete frame commercial buildings. It would appear that the S/D ratio for the HSLDC produced in the 1918 ship-building program was only exceeded by HSNDC 40 years later.

Compressive strength of the HSLDC cores taken at the waterline from the USS *Selma* and tested in 1980 were found to be twice the 28-day specified strengths, and from a structural efficiency standpoint are not appreciably different from the HSNDC of today. Analysis of the physical and engineering properties of the HSLDC in the ships of World War I, the 104 HSLDC World War II ships, as well as numerous recent bridges built, can be found in other reports that amply prove the

almost 80-year long-term successful performance of HSLDC (Bremner, Holm, Stepanova 1994).

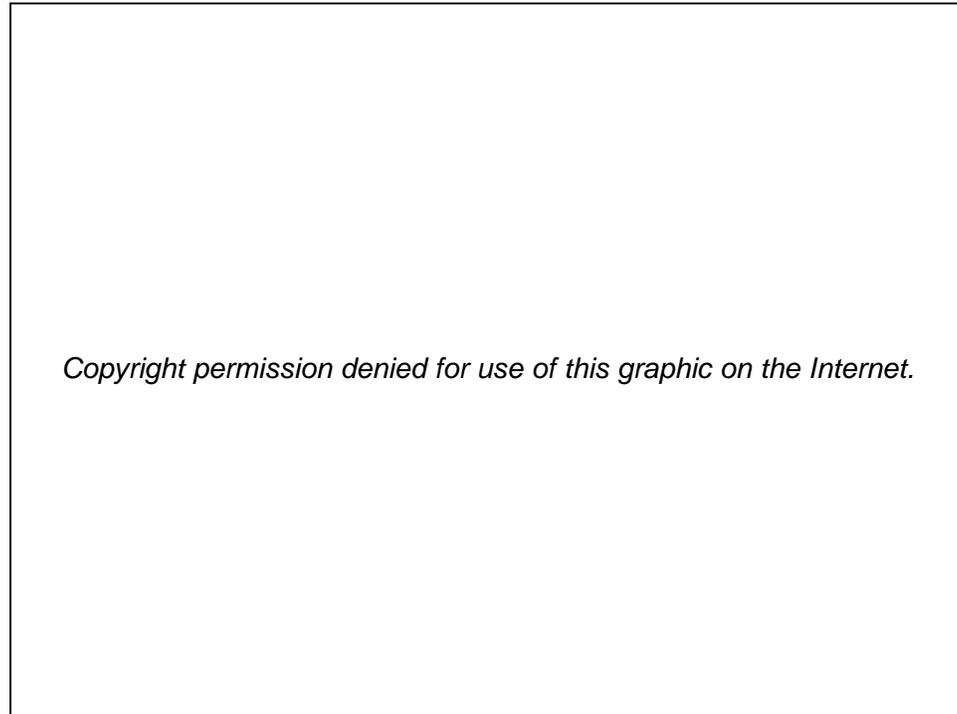


Figure 32. Structural efficiency of low-density concrete (from Holm 1994, with permission of ASTM)

Material Costs

An analysis frequently used in comparing the value-added potential as opposed to the increased material costs of LDC for bridge construction projects may be useful. The analysis is predicted upon cost per ton of aggregate delivered, but a similar analysis may be made using volume criteria (see Table 21).

Transportation Costs

In situations where transportation costs are directly related to the mass of concrete products, there can be significant economies developed through the use of low-density concrete. The range of products includes large structural members (girders, beams, walls, hollow-core panels, double tees, etc.) to smaller consumer products (precast stair steps, fireplace logs, wall board, imitation stone, etc.). Potential for lower costs is possible when shipping by rail or barge, but is most often realized in trucking where highway loadings are posted. Two examples

developed recently for economies in transportation observed in large-scale precasting plants are shown in Table 22 (Speck 1999).¹

Table 21 Effect of Aggregate Cost on Cost of Concrete					
Analysis	A		B		Relative Cost Increase A/B x 100
	Low-Density Aggregates and Concrete		Normal-Density Aggregates and Concrete		
Cost of aggregate, \$/tonne (\$/ton)	49.50	(45)	11	(10)	+450%
Aggregate required for 1 m ³ (yd ³) of concrete, kg (lb)	535	(900)	1,010	(1710)	-
Cost of coarse aggregate used in concrete, \$/m ³ (\$/yd ³)	26.50	(20.25)	11.15	(8.50)	+238%
Cost increase due to use of low-density aggregate, \$/m ³ (\$/yd ³)	14.35	(11.75)	-	-	-
Typical cost of concrete delivered to project, add small increase for additional cement in low-density concrete, \$/m ³ (\$/yd ³)	111	(85)	92	(70)	+21%
Cost of concrete in-place including formwork, conveying, finishing, curing, reinforcing (bridge deck), \$/m ³ (\$/yd ³)	478*	(365)	458	(350)	+4%
Value-Added Considerations					
<p>* The final in-place cost of the LDC does not include the following potential value-added considerations:</p> <ul style="list-style-type: none"> Reduced foundation loads resulting in smaller footings, lower number of piles, smaller pile caps. Reduced dead loads may result in smaller supporting members (decks, beams, girders, piers). Reduced inertial loads in seismic zones. In bridge rehabilitation, the new deck may be wider, or additional traffic lanes are possible. Bridge decks or overlays may be thicker (yet of equal dead load) allowing better drainage. In precast, prestressed long-span bridges, longer members result in fewer joints, and may be more practical to make, lift, transport, and erect. In several documented cases, the reduction in shipping costs was several times greater than the increase in material cost. In marine application, the reduced draft of low-density concrete structures will permit movement out of drydocks and through shallow shipping channels. 					

Environmental Considerations

Increased use of processed LDA is evidence of environmentally sound planning, as these products use materials with limited structural applications in their natural

¹ Personal Communication, April 1999, Jeff Speck, Big River Industries, Alpharetta, GA.

state, thus minimizing construction industry demands on finite resources of natural sands, crushed stones, and gravels.

Table 22 Analysis of Shipping Costs of Low-Density Concrete Products (Courtesy of Big River Industries, Inc.)		
	Project Example Number 1	Project Example Number 2
Shipping Cost per Truck Load	\$ 1,100	\$ 1,339
<u>Number of Loads Required</u>		
<i>Normal Weight</i>	431	87
<i>Lightweight</i>	<u>287</u>	<u>66</u>
Reduction in Truck Loads:	144	21
<u>Transportation Savings</u>		
Shipping Cost per Load	\$ 1,100	\$ 1,339
Reduction in Truck Loads	<u>x 144</u>	<u>x 21</u>
Transportation Savings:	\$158,400	\$28,119
<u>Profit Impact</u>		
Transportation Savings	\$158,400	\$28,119
Less: Premium Cost of LWC	<u>17,245</u>	<u>3,799</u>
Increase in Gross Margin:	<u>\$141,155</u>	<u>\$24,320</u>

10 Conclusions

The past two decades have witnessed the widespread extension of the knowledge of the properties and the application of high-strength, low-density concrete. Improved structural efficiency (strength-mass ratio), achieved through lower self-load of the structure, has provided economic advantages to thousands of commercial structures and made feasible offshore marine megastructures in the Arctic and on both sides of the Atlantic Ocean. Additionally, innovative bridge design has permitted the functional rehabilitation of hundreds of bridges where, for example, additional lanes were placed on existing girders, piers, and foundations. All of this has been made possible because of improvements in the matrix of HSLDC. These improvements have been aided by the development of new admixtures (e.g. HRWRA) that effectively combine with supplementary cementitious materials that include silica fume, fly ash, metakaolin, calcined clays, and shales.

Recent laboratory research programs, combined with the detailed physical examination of 80-year-old LDC ships, have provided ample proof of long-term durability. The excellent performance of these vessels provided confidence to the designers of several Arctic marine structures built in the early 1980s, which in turn gave assurance to the designers and owners of the construction of the multibillion-dollar Hibernia and Heidron megastructures.

Extensive laboratory research, coupled with the in-depth examination of severely exposed structures, has established LDC as a viable, cost-effective alternative to NDC. This concrete is now understood to be unique in several respects, and is being designed into structures to take advantage of these unique, structurally efficient properties.

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ASTM C 331. “Standard specification for lightweight aggregates for concrete masonry units.”

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Appendix A

Structural Low-Density Concrete

Guide Specifications

All Corps of Engineers specifications for concrete are to be followed, with the additions as outlined below.

Structural Low-Density Aggregate

Low-density aggregate shall be an expanded shale, clay, or slate produced by the rotary-kiln process, and shall meet all the requirements of ASTM C 330. Nonstructural lightweight aggregates shall not be acceptable. ASTM C 330 certification shall have been verified by an independent testing laboratory within 2 years of the submission of data to the engineer. The low-density aggregate producer shall furnish test reports from an independent testing laboratory certifying that concrete made with the aggregate and containing approximately 6 percent air content shall have a minimum durability factor of 85 percent when tested in accordance with the procedures of ASTM C 666 Procedure A and ASTM C 330.

Coarse expanded aggregate shall conform to the grading requirements of 19.0 mm (3/4 in.) to 4.75 mm (No. 4), or 12.5 mm (1/2 in.) to 4.75 mm (No. 4), or 9.5 mm (3/8 in.) to 2.36 mm (No. 8) of ASTM C 330. In addition, the resistance to degradation of the coarse aggregate, when tested by the Los Angeles abrasion method of ASTM C 131, shall not exceed 50 percent.

Structural Low-Density Concrete

Cement, aggregates, water, and admixtures shall be proportioned in accordance with ACI 211.2, "Recommended Practices for Selecting Proportions for Structural Lightweight Concrete." The water added to the mixture using saturated surface-dry aggregate shall not cause the w/c to exceed 0.45. Water-cement ratios shall be established by trial mixtures in accordance with ACI 211.2.

An air-entraining admixture meeting ASTM C 260 shall be used to produce an air content in the fresh concrete between 4 and 8 percent (ACI 318). Air content shall be determined by the volumetric method described in ASTM C 173.

Slump shall be 100 \pm 25 mm (4 \pm 1 in.). Maximum fresh unit weight (for control and acceptance) shall not exceed _____ kg/m³ (_____ lb/ft³). The calculated equilibrium density shall average less than _____ kg/m³ (_____ lb/ft³). Equilibrium density shall be calculated or measured in accordance with ASTM C 567. When the fresh density varies by more than 32 kg/m³ (2 lb/ft³) from the proposed fresh density, adjust the mixture as promptly as possible to bring the density to the desired level. Do not use concrete for which the fresh density varies by more than 48 kg/m³ (3 lb/ft³) from the specified level.

Mixture proportions, creep, and shrinkage of low-density concrete produced from the approved expanded aggregate shall be made available to the engineer for prior approval. The manufacturer of the expanded aggregate proposed for the project shall make available to the engineer results of tensile strength tests conducted in accordance with ASTM C 496. The tensile splitting strength obtained on concrete composed of coarse expanded aggregate and natural sand should yield values in excess of 0.85 times those called for in ACI 318 for the compressive strength specified. The tests should give values exceeding 0.75 times those called for in ACI 318 when the concrete is composed of fine and coarse expanded aggregate (i.e., natural sand is not included). A linear interpolation between 0.75 and 0.85 can be used when natural sand is included with fine expanded aggregate.

Prequalification of Structural Low-Density Concrete Mix Proportions

After the materials have been accepted for this work, the contractor will determine the proportions for concrete and equivalent batch weights to produce concrete with a compressive strength of _____ MPa (_____ psi) at 28 days.

Trial mixes

The contractor will determine the proportions on the basis of trial mixtures conducted with the materials to be used in the work in accordance with ACI 211.2. The corresponding cement content for each trial batch shall be determined by means of a yield test in accordance with ASTM C 138.

Proportions

The engineer shall be provided a copy of the trial mixture proportions that include the following:

- a. The mass in kilograms (pounds) of fine and coarse aggregate (saturated surface-dry condition), per cubic meter (pounds per cubic yard) of concrete.
- b. The cement content in kilograms per cubic meter (pounds per cubic yard).
- c. Amount of water in kilograms per cubic meter (pounds per cubic yard).

These values shall be used to manufacture all low-density concrete for this project. The proportions shall not be changed unless the engineer is informed at least 3 working days prior to the change being made. Further, the contractor shall provide to the engineer his reasons for making the changes.

Batch quantities

The engineer will approve the batch proportions by mass. Since the proportions are designated in terms of aggregates in saturated surface-dry conditions, the equivalent batch quantities by mass used by the contractor shall be corrected periodically to account for the moisture content of the aggregate at the time of use.

Strength of concrete mixture

Where a concrete production facility has appropriate testing records on a concrete similar to that proposed for the project, the concrete proportions may be submitted for approval in accordance with the procedures of ACI 318. Where no prior data on similar concrete are available, the concrete shall be proportioned in accordance with the procedures and requirements of ACI 318. Evaluation and acceptance of concrete shall be in accordance with ACI 318.

Workability

The concrete shall be of such consistency and composition that it can be worked readily without segregation of materials or the excessive collection of free water on the surface. Subject to the limiting requirements above, the contractor shall, if the engineer requires, adjust the proportions of cement and aggregates so as to produce a mixture that will be easily placeable at all times, due consideration being given to the methods of placing and compacting used on the work. Do not vibrate low-density concrete to the extent that large aggregate particles float to the surface. Do not finish low-density concrete to the extent that mortar is driven down and expanded coarse aggregate appears at the surface.

Aggregate storage

Pre-wet expanded aggregates when recommended by the aggregate supplier, or as specified in the contract documents. Follow the recommendations of the expanded aggregate supplier for storage, handling, and pre-wetting procedures.

Expanded coarse aggregates, together with approximately two-thirds of the total mixing water, shall be introduced into the mixer and mixed for a minimum of _____ minutes. The fine aggregate, cement, admixtures, and the remaining mixing water shall then be added and mixed completely.

Structural low-density concrete shall not be placed when the temperature of the concrete mixture is _____ °C (_____ °F) or greater. No placement of structural lightweight concrete will be allowed if weather forecasts indicate that air temperatures of 38 °C (100 °F) or greater will occur within 4 hr of the proposed placement time.

The contractor may place concrete at night with the prior permission of the engineer. Night placement will be restricted to those times, in the engineer's opinion, during which daytime temperatures are too high to result in acceptable concrete placements during daylight hours. All extra costs attributable to nighttime placement operations will be borne by the contractor. Permission to perform nighttime placements may be rescinded by the engineer at any time, with no advance notice.

Handling and placing

Concrete shall be transported from the place of mixing to the point of deposition as rapidly as practicable. Methods that will prevent the separation or loss of ingredients shall be employed. Concrete shall not fall freely more than _____ m (_____ ft). Depositing a large quantity at any point and working it into final position will not be permitted.

Manufacturer's representative

The manufacturer of the expanded aggregate shall have a service representative at the site for the initial placement of structural low-density concrete. The manufacturer's representative shall be given the authority by the contractor to assist in all aspects of low-density concrete mixing and placement operations and provide liaison with the concrete supplier as approved by the engineer. A technical report shall be submitted to the engineer by the expanded aggregate supplier regarding any observations or test results relative to the concreting practices at the work site.

REPORT DOCUMENTATION PAGE

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14. ABSTRACT <p>This report presents an overview of the current knowledge related to high-strength, high-durability structural low-density concrete (compressive strength 85 MPa (5,080 psi)) and its application in severe marine environments.</p> <p>Low-density concrete (LDC) is normally made with a manufactured low-density aggregate produced by heating particles of shale, clay, or slate to about 1,200 °C (2,160 °F) in a rotary kiln. At this temperature the raw material bloats, forming a vesicular structure that is retained upon cooling. The individual vesicles are generally not interconnected and produce a dilation of more than 50 percent that is retained upon cooling. This results in the particle density of the raw material changing from about 2.65 before heating to less than 1.55 upon cooling. It is the use of this low-density aggregate that enables the production of high-strength, high-durability structural low-density concrete, which is sometimes referred to by its obsolete term "lightweight concrete."</p> <p>Since the 1970s, the use of high-strength, low-density concrete (HSLDC) has seen widespread expansion. Coupled with an enhanced high-strength matrix, achievable compressive strength levels for these advanced, structurally efficient concretes have increased by more than 40 percent. Improved structural efficiency has provided economic advantages to thousands of commercial structures and made feasible the construction of offshore marine megastructures in the Arctic and on both sides of the Atlantic Ocean. In addition, innovative bridge design has permitted the functional rehabilitation of hundreds of bridges, where additional lanes were placed on existing girders, piers, and foundations.</p> <p>Extensive laboratory research, coupled with in-depth examination of severely exposed structures, has now essentially eliminated unfounded prejudices toward LDC. The economics and potential future uses depend on how well design professionals can incorporate the unique properties of LDC to meet future building needs.</p>											
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