Durability of Structural Lightweight Concrete

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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/m^3 (110 lf/ft^3). 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.

Table 10
Requirements for Concrete Exposed to Sulfate-Containing Solutions

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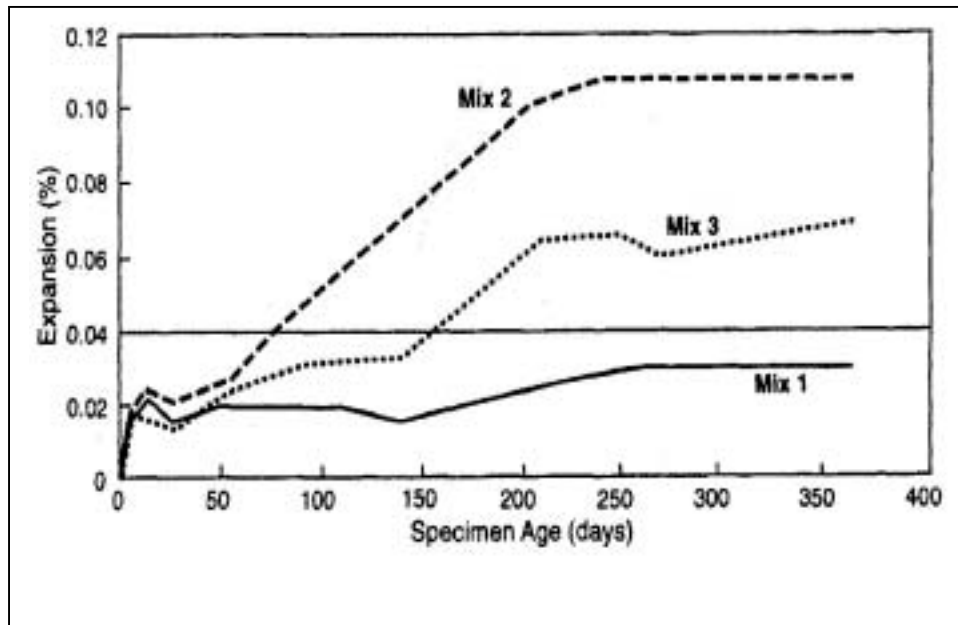
Figure 18. Reduction of expansion when low-density aggregates are used (Bremner et al. 1998) (Mixture 1 - Nonreactive normal-density fine and coarse aggregate; Mixture 2 - Reactive normal-density coarse aggregate and nonreactive fine aggregate; Mixture 3 - Reactive normal-density coarse aggregate and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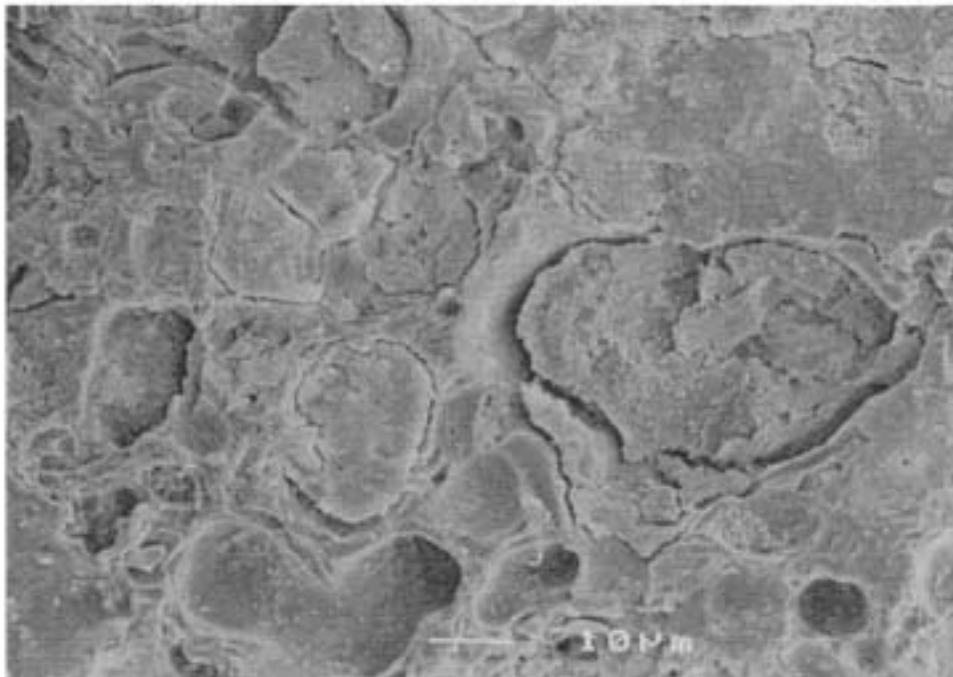

Figure 19. Concrete with a precipitation of alkali-rich material in the pores of low-density aggregate (Boyd 1998)

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 Kc is calculated as follows:

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

where

Kc = 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.)	K_c $(mm/\sqrt{years})$
Cape Charles, Virginia	Concrete ships	All LDC (35 MPa, 1,730 kg/m^3)		
		(A) Hull bulkhead	1 (0.04)	0.2
		(B) Wing-wall	1 (0.04)	0.2
		(C) Superstructure deck-top	1 (0.04)	0.2
		(D) Superstructure deck-bottom	2 (0.08)	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	1 (0.04)	0.2
		(B) Bottom surface, truss span	5 (0.20)	0.8
		(C) Top surface, approach span	8 (0.31)	1.4
		(D) Bottom surface, approach span	13 (0.51)	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	5 (0.20)	1.3
		(B) Bottom surface	10 (0.39)	2.6
Japan	Bridges/viaducts, 19 years	Sanded LDC (23 MPa, 1,820 kg)	16 (0.63)	3.7
		Sanded LDC (26 MPa)	18 (0.71)	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, aquaducts, 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 $K_c = d_c/\sqrt{t}$ in $(mm/\sqrt{years})$	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
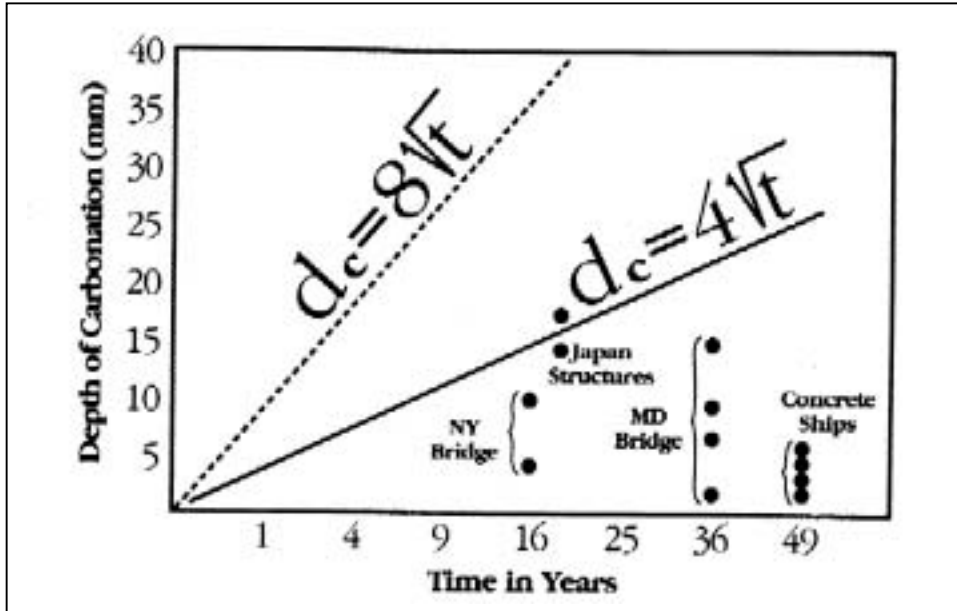
Figure 20. Measured depth of carbonation (in millimeters) of exposed low-density concrete (from Holm, Bremner, and Vaysburd 1988, with permission of ACI)
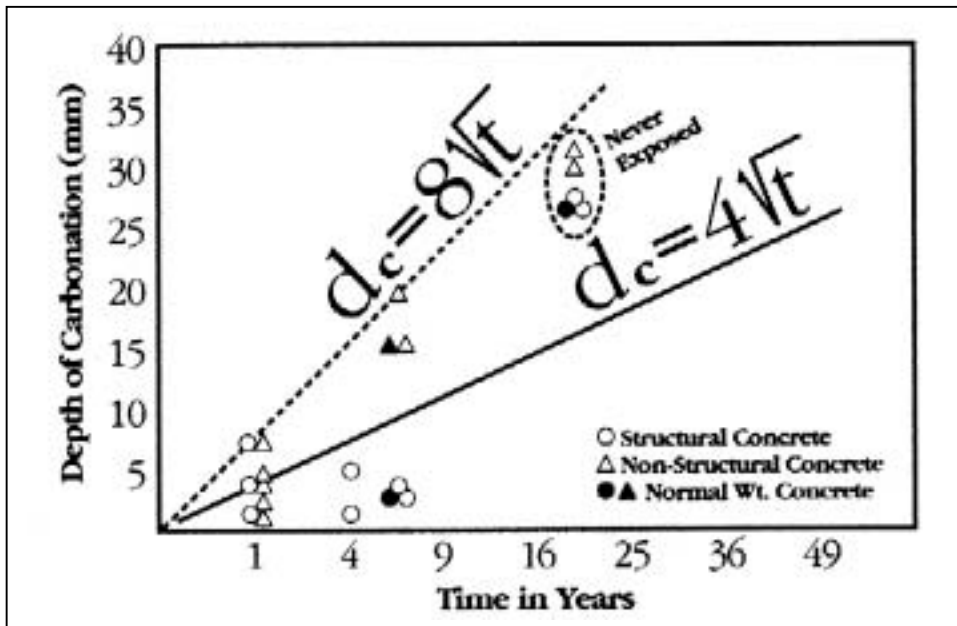

Figure 21. Measured depth of carbonation (in millimeters) of laboratory specimens of low-density concrete (from Holm, Bremner, and Vaysburd 1988, permission of ACI)

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/cm^2 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/ft^3) 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
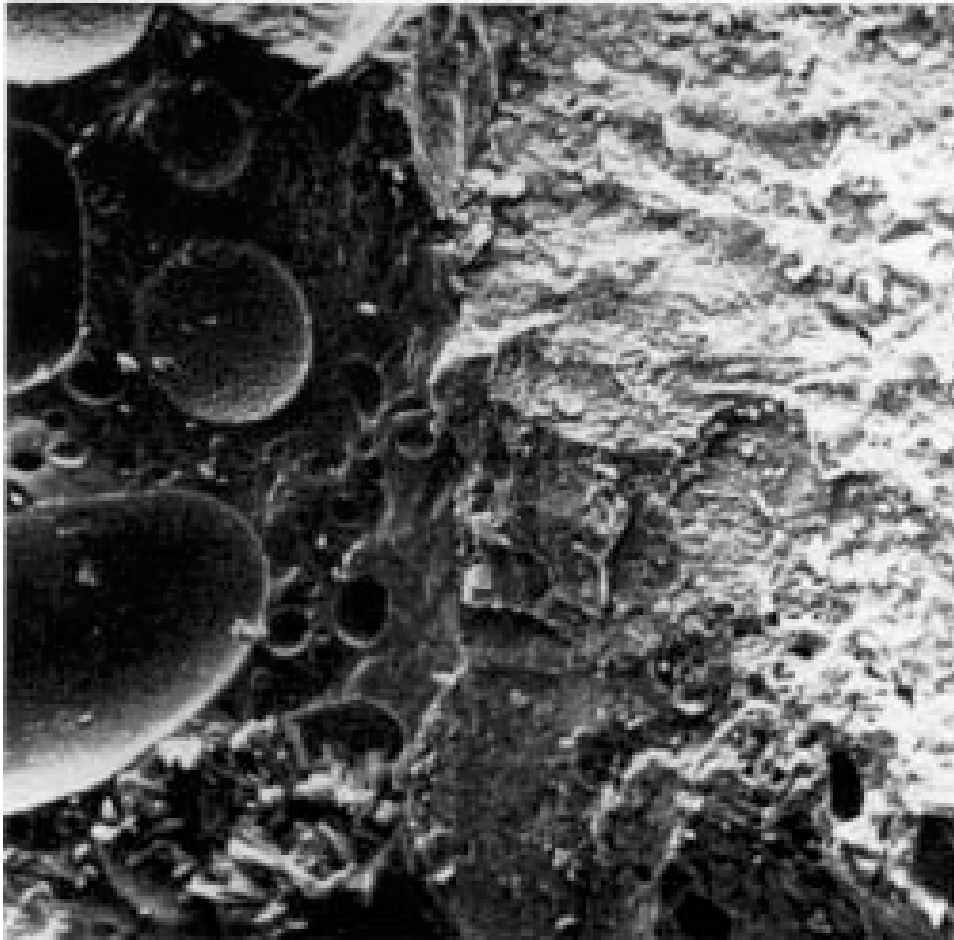
Figure 22. Fractured surface of concrete from USS *Selma*. (Expanded aggregate is on the left, and the cement paste is on the right. Micrograph width is 550 μm) (from Bremner, Holm, and Morgan 1996, with permission of ACI)

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 lbf/in.^{2}) while outside the contact zone the hardness measured 60 to 80 kgf/mm^2 (85,400-113,800 lbf/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

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.

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.

Table 10
Requirements for Concrete Exposed to Sulfate-Containing Solutions

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 alkalies in the cement, or from the ingress of alkalies 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).