Chapter 8

Durability/Service Life of Structural Lightweight Concrete

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CHAPTER 8

Chapter 8 Durability/Service Life

8.0   Historical Performance

Lightweight Concrete Ships

The first structural application using rotary kiln-produced lightweight concrete was the USS Selma, which was launched in 1919 (Holm 1980a). This 7,500 ton ship now partially submerged and resting on the ground in Galveston Harbor and declared a National Monument has been exposed to seawater continuously. In many areas it was damaged – when run onto a rock breakwater at Tampico Bay, Mexico, and as a result of hard berthing and extensive physical damage when the steel and equipment was salvaged from the ship. Much of the concrete below the waterline both inside and outside the hull (after the barnacles had been scraped off) appear to be in an as-cast condition with the form marks still visible.
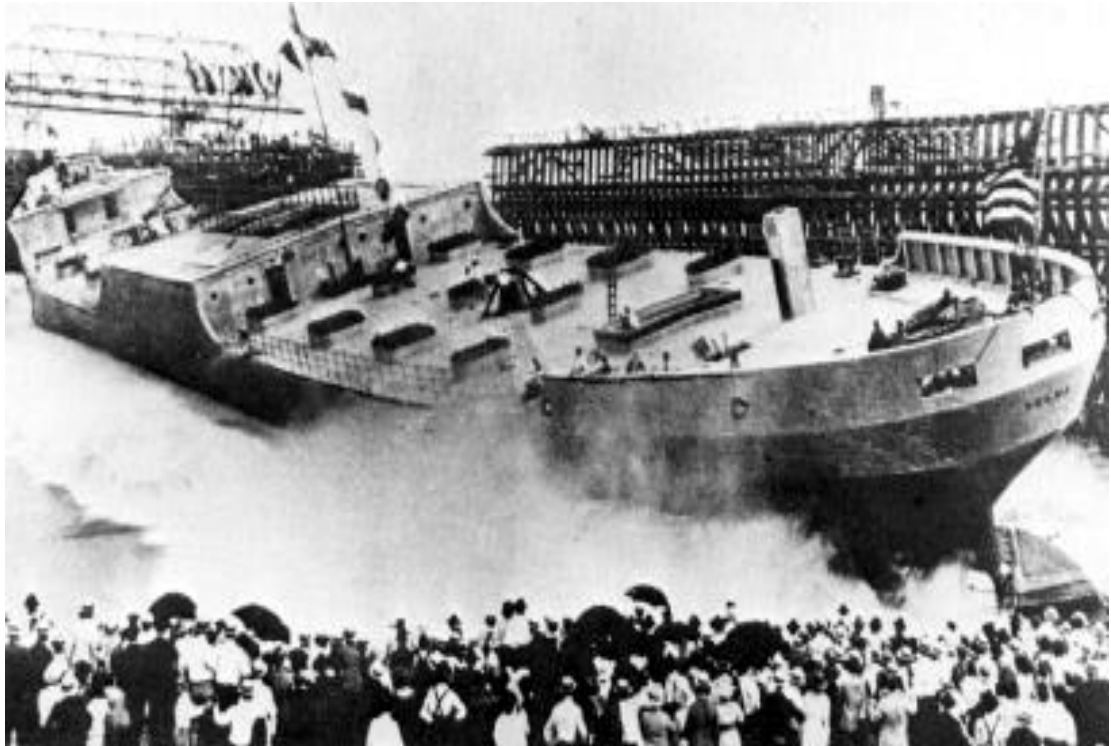

Figure 8.1. *Launching of lightweight concrete ship*
USS SELMA in 1919

Several of the Selma's holds contain water with over 6 ft. (2 m) head above the surrounding sea, providing ample proof of the low permeability of the concrete. In many undamaged sections of the ship the 0.5 to 1.2 in. (12 – to 30-mm) cover has proven surprisingly effective in protecting the reinforcing steel from

corrosion. The effectiveness of lightweight concrete in protecting the reinforcing steel was also shown by Sturm, McAskill, et. al. SP-189 ACI. The strength of the concrete on the Selma was in excess of 4,000 psi (27.6 MPa) at 28 days at a time when ordinary concrete had a strength of 2,000 psi (13.8 MPa). Cores taken from the Selma in 1980 had compressive strengths in excess of 8,000 psi (55.2 MPa) for concrete with a unit weight of about 110 lb/ft³ (1,762 kg/m³). Since the strength/performance ratio is comparable to what is now commonly referred to as high-performance concrete; it's impressive that structural lightweight concrete at 110 lb/ft³ had over 70 years head start for high-performance concrete with a much better strength/to density ratio.

Samples of concrete taken from the Selma below the waterline were examined in a scanning electron microscope. It was noted that, other than in a region near the aggregate-cement paste interface, there was very little evidence of aggregate vesicules becoming 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 normal density concrete (Holm, Bremner, and Newman 1984) (Sturm, et.al. 1999), with normal density concrete, 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 lightweight concrete projects, the aggregate from the Selma had a microstructure that was identical to aggregates produced by a modern rotary kiln; long-term good performance can be expected from current ESCS production, provided that changes to the portland cement are not a factor.
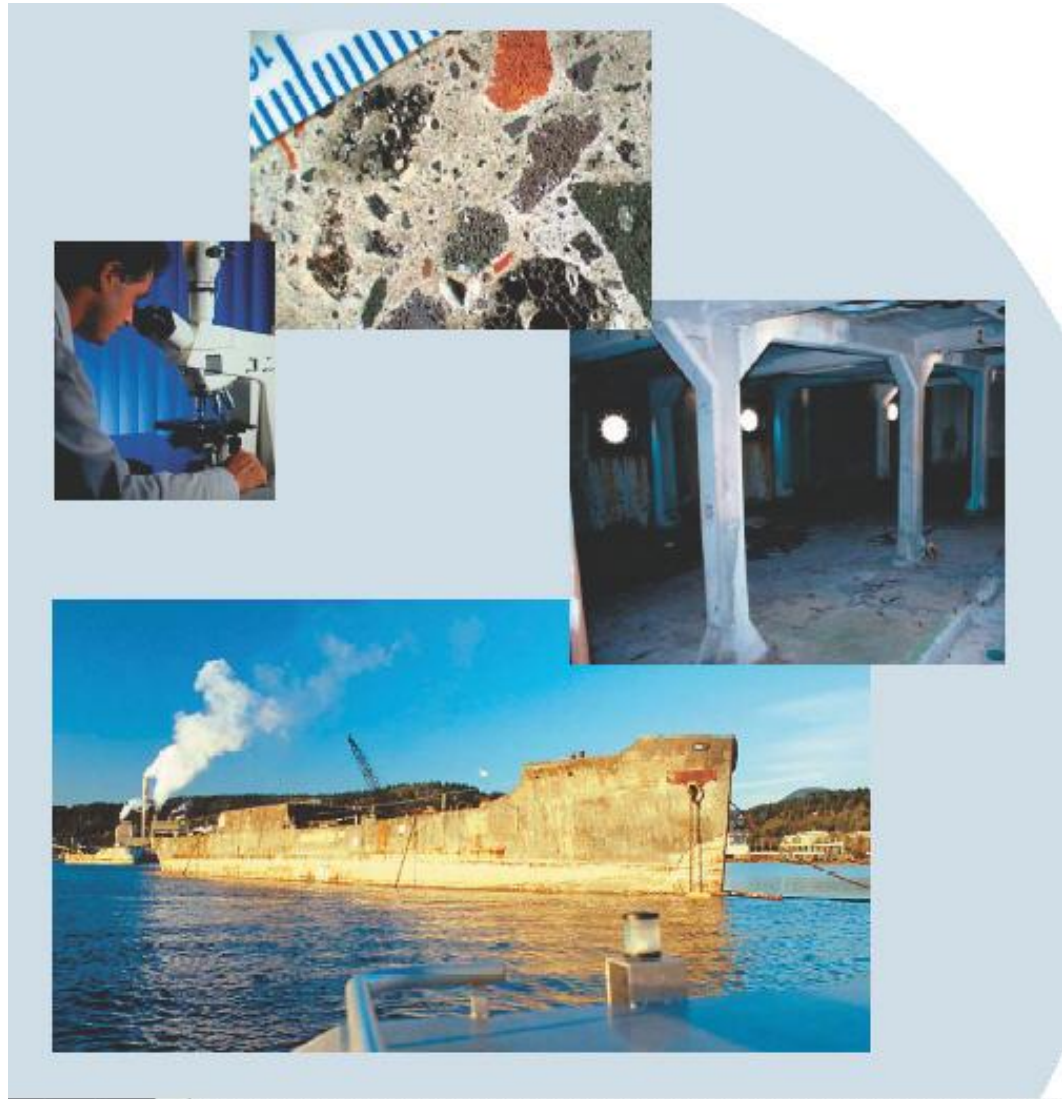
Figure 8.2. *Powell River Ship,*
Powell River, British Columbia, Canada

Floating Structural Lightweight Concrete Vessels Continue to Serve:
Ten lightweight concrete ships are being used as a floating breakwater at the Pacific Paper Powell River Plant. Evaluation of the ships for long-term service durability of the lightweight concrete was recently conducted (Sturm, et. a. 1999). The ships range in age from 55 to 80 years with one ship constructed in the 1920's, and the other nine in the 1940's. Theses ships demonstrate the excellent performance of structural lightweight concrete after more than a half-century of marine exposure.

Petrography studies conducted at CTL (Construction Technology Laboratory) revealed limited microcracking, excellent aggregate/matrix contact zone,

complete hydration of the cement, and insignificant damage due to freezing and thawing. This microscopic examination clearly showed evidence of continued hydration and development of the cement matrix. This continued maturation of the concrete has contributed to the development of compressive strengths up to 8700 psi in the ships' hulls well beyond the minimum design strength (5000 psi) of the concrete. Concrete densities range from 106 lb/ft³ to 130 lb/ft³.

According to the inspectors' report, the "paste-aggregate bond is consistently excellent in all the examined concrete specimens, in part attributed to mostly beneficial reactions along the paste-aggregate interface. Overall, the manufactured lightweight concrete used in the construction of the ships has performed exceptionally well in a harsh marine environment."

8.1 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 many freezing and thawing cycles occur each season and large amounts of de-icing agents are applied. De-icing chemicals melt ice and snow and produce water that increases concrete saturation and chloride concentration. 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 very severe in the northeastern areas of the United States, making this region a useful location for comparative studies of the relative performance of lightweight and normalweight concrete. These areas have had a long history of bare bridge decks, whereas in Canada it is common to use a waterproof membrane concrete under the asphalt to prevent ingress of chloride ions into the concrete. A study of lightweight concrete bridge decks was completed in 1960 (Expanded Shale, Clay and Slate Institute, ESCSI 1960). Based on published reports in the United States (FHWA 1985), England and Japan, the performance of lightweight concrete bridge decks is at least as good as normal density concrete bridges (Brown, Larsen and Holm 1995).

By 1935, over 34 lightweight concrete bridges had been built in North America, including nine in Canada (ESCSI- 1960). The good performance of several early bridges, built before concrete was air entrained, is surprising. The fact that water reducing admixtures in the 1950's entrained some air and were used in placing lightweight concrete might, in part, account for their good long-term performance (Holm 1983). Another reason for their good performance is that pores within the lightweight aggregate can act as pressure relief chambers when the hydraulic

pressure develops as the free water freezes. Crushed porous 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 entrained air in all lightweight concrete. When freezing and thawing is anticipated, 4 to 8 percent entrained air contents are recommended in lightweight concrete with a nominal maximum aggregate size of 3/4" (19.0 mm), and 5 to 9 percent when the nominal maximum aggregate size is 3/8" (9.5 mm). To achieve an effective air-void system that will protect the concrete from repeated cycles of saturated freezing and thawing, it is essential that the air voids be well distributed throughout the cement paste matrix. Generally, 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 usually 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.

If not properly protected, concrete made with lightweight aggregates with a high degree of saturation may be vulnerable to early freezing damage. As with normalweight, lightweight concrete should be protected from freezing conditions until it has been cured (time and temperature) in accordance to procedures in ACI 306. This is necessary to allow adequate strength to develop.

8.2   Resistance to Sulfate Attacks

As with normalweight concrete, the ACI 318 code recommendations should be followed with respect to the level of sulfates in the groundwater, see Table 8.1. 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.

Table 8.1 Requirements for Concrete Exposed to Sulfate-Containing Solutions from ACI 318 Section 4.3 Sulfate Exposure

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'_{ct} Normal Density and lightweight Aggregate, psi
Negligible	$0.00 \le SO_4 \le 0.10$	$0 \le SO_4 \le 150$	---	---	---
Moderate[2]	$0.10 \le SO_4 \le 0.20$	$150 \le SO_4 \le 1,500$	II, IP (MS) IS (MS), P(MS) I (PM)(MS), I(SM)(MS)	0.50	4,000
Severe	$0.20 \le SO_4 \le 2.00$	$1,500 \le SO_4 \le 10,000$	V	0.45	4,500
Very Severe	$SO_4 > -2.00$	$SO_4 > 10,000$	V plus pozzolan[3]	0.45	4,500

[1] 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/cm, increased moist curing, and the use of air-entrained concrete are desirable. Also, the reduced microcracking in lightweight concrete and the improved quality of aggregate-to-mortar matrix bond suggest that the concrete maybe more resistant to sulfate attack.

8.3   Resistance to Alkali-Silica Reaction

Concrete made from either natural lightweight aggregate or manufactured lightweight aggregate 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 normalweight aggregate, replacement of either reactive or even the nonreactive normalweight aggregate with lightweight aggregate has been found to significantly reduce deleterious expansions (Bremner et al. 1998, Boyd 1998) Figure 8.3.

The heating of the aggregate in a rotary kiln 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) Figure 8.4.   Another factor that enables a porous aggregate to reduce disruptive expansion is the availability of pores (space) within the expanded lightweight aggregate for reactive material to precipitate into, in a benign manner. "The glass phase of expanded clay, shale and slate may be alkali reactive but expansion of concrete from this cause has not been observed because any siliceous gels that are generated are accommodated within the abundant air-filled cavities that characterize the expanded particles," ASTM C294 *Standard Descriptive Nomenclature for Constituents of Concrete Aggregates,"* section 26.3.5. In Figure 8.3, the beneficial effect of replacing some of the aggregates with lightweight aggregates on expansion can be seen.
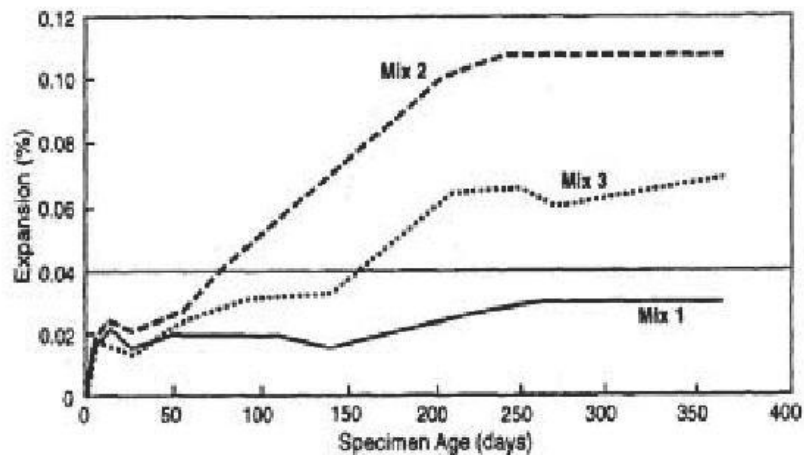

Figure 8.3. *Reduction of expansion when lightweight aggregates are used (Bremner et al. 1998) (Mixture 1 – Nonreactive normalweight fine and coarse aggregate; Mixture 2 – Reactive normalweight aggregate and nonreactive fine aggregate; Mixture 3 – Reactive normalweight coarse aggregate and one half of absolute volume of nonreactive fine aggregate replaced by lightweight fine aggregate)*

Figure 8.4. *Concrete with a precipitation of alkali-rich material in the pores of lightweight aggregate (Boyd 1998).*

In Figure 8.4, precipitation of alkali-rich material in the pores of an expanded aggregate can be seen in concrete made with a well-known reactive normalweight coarse aggregate in which some of the nonreactive fine aggregates have been replaced with lightweight fine aggregates.

8.4   CARBONATION

Carbonation is the reaction in concrete of carbon dioxide from the air with calcium hydroxide released during the hydration of cement. This reaction produces calcium carbonate that can diminish the natural protection of steel reinforcement afforded by the concrete. The very slow rate at which the carbonation front advances into concrete has been studied in many testing programs. This section of the reference manual reports on measurements of the penetration of carbonation into mature, exposed lightweight concrete structures.

Concern for carbonation is predicated on the lowering of the concrete's alkalinity, which in turn, removes the protective layer over the reinforcing steel, making it vulnerable to corrosion. Two primary factors protect the steel from corrosion, an adequate depth of cover over the top of the reinforcing and a low permeable high quality concrete.

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 the 19140's. 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 ESCS fine and coarse lightweight aggregates and a small volume of natural sand. High-cement contents were used to achieve compressive strengths in excess of 5,080 psi (35 MPa) at 28 days with a density of 108 pcf (1m7030 kg/m³) (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 areas experienced a delamination of the 0.78 in. (20 mm) concrete cover that protected four layers of large un-deformed reinforcing bars spaced 4 in. (100 mm) on centers. In retrospect, this failure plane is understandable and should have been avoided by the use of modern prestressing methods. Cover for hull reinforcing was specified at 7/8 in. (22 mm), with all other reinforcement protected by only 1/2 in. (13 mm) of cover.

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 reinforcement. 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 0.04 and 0.08 in. (1 and 2 mm) 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 16 ft. (5 m) above high water, and in no instances could carbonation depths greater that 0.08 in (2 mm) be found. In isolated instances, flexural cracks up to 0.31 in. (8 mm) in depth were encountered, and these had carbonated in the plane of the crack. The carbonation did not appear to progress more than 0.004 in. (0.1 mm) perpendicular to the plane of the crack.

The result of these tests is given in Table 8.2. The value of Kc is calculated as follows:

$$Kc = \frac{d}{\sqrt{t}}$$

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.

Table 8.2 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 > 70 years old	All LWC (35 MPa, 1,730 kg/m³) (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 LWC (24 MPa, 1,650 kg/m³) (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	Sand LWC (27 MPa, 1760 kg/m³) (A) Exposed top deck (B) Bottom surface	5 (0.20) 10 (0.39)	1.3 2.6
Japan	Bridges/Viaducts, 19 years	Sand LWC (23 MPa, 1820 kg) Sand LWC (26 MPa)	16 (0.63) 18 (0.71)	3.7 4.1
All LWC: Lightweight fine and coarse aggregate.				

Holm, Bremner, Vaysburd (1988)

Concrete cores taken from the 35-year-old Chesapeake Bay Bridge revealed carbonation depths of 0.08 to 0.31 in. (2 to 8 mm) from the top of the bridge deck and 0.08 to 0.51 in (2 to 13 mm) from the underside of the bridge deck. The higher carbonation depth on the underside reflects the increased gas diffusion associated with this drier portion of the bridge. The 1.41 in. (36 mm) 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).

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 0.20 in. (5 mm) carbonation depths for the top surface and 0.39 in. (10 mm) 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).

The results of measurements of carbonation depths on mature marine structures in North America (shown in Fig. 8.5) 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 lightweight concrete and normalweight concrete on the same bridges, aquaducts, and caissons after 19 years of exposure. The high-durability performance of those structures as measured by the test results of carbonation depths, microcracking, and chloride penetration profiles Ohuchi et al. (1984) are similar to reports by Khokrin (1973) who suggested that with regard to other variables (specimen size, media pressure, and equipment), structural lightweight concrete had equal or lower permeability than its normalweight counterpart. Khokrin (1973) further reported that the lower permeability of lightweight concrete was attributed to the elastic compatibility of the constituents and the enhanced bond between the coarse aggregate and the matrix. In the Onoda Cement Company tests (Nishi et al. 1980), 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 carbonation 1.38 in (35 mm) for normalweight concrete and 0.95 in. (24 mm) for lightweight concrete. When tested with seawater, penetration was 0.59 and 0.47 in. (15 and 12 mm) for normalweight concrete and lightweight concrete, respectively. The author suggested that the reason for this behavior was "a layer of dense hardened cement paste surrounding the particles of artificial lightweight coarse aggregate".
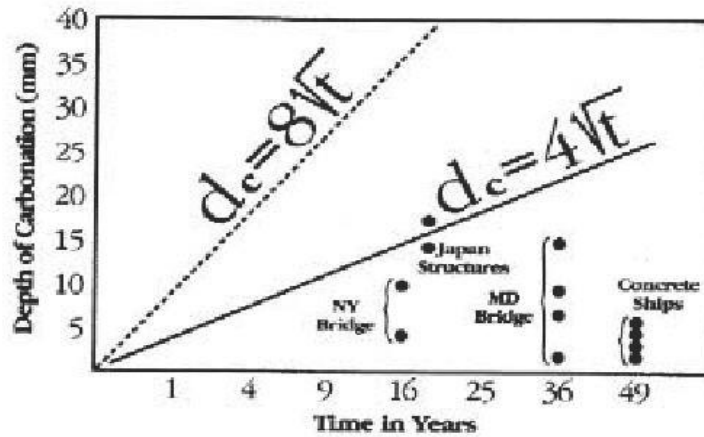
Figure 8.5. *Measured depth of carbonation (in millimeters) of exposed lightweight concrete (from Holm, Bremner, and Vaysburd 1988)*
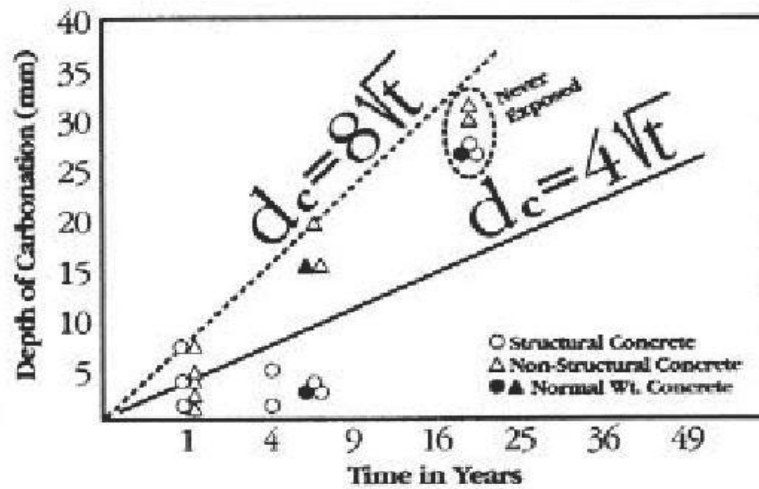

Figure 8.6. *Measured depth of carbonation (in millimeters) of laboratory specimens of lightweight concrete (from Holm, Bremner, and Vaysburd 1988)*

The effect of strength and density and moisture exposure on carbonation depth can be seen in Fig. 8.6 which covers specimens stored for over 20 years in a Richmond, VA., laboratory. These widely varying concrete types (normalweight, sanded lightweight, all lightweight and an insulating type concrete) had been part of an unrelated investigation into time related density and moisture variations in non-structural concrete. To develop fracture surfaces without saw cutting, the prisms were split by techniques similar to the tensile splitting method of ASTM C 496 as described earlier. The results clearly indicate that an increased rate of carbonation occurs in low strength concrete in a dry environment.

The U.S. Navy sponsored work by Keeton (1970), who reported the lowest permeability with high strength lightweight concrete. Bamforth (1987) incorporated structural lightweight concrete as one of the four concretes tested for permeability to nitrogen gas at 145 psi (1 MPa) pressure level. The normalweight concrete specimens included high-strength 13,000 psi (90 MPa) concrete as well as concretes with a 25-percent fly ash replacement, by mass or volume. The sanded structural lightweight concrete with a strength of 7,250 psi (50 MPa), and 6-4% air and with a density of 124 lb/ft³ (1,985 kg/m) 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 responding differently to temperature gradients, service load-induced strains, and volume changes associated with chemical reactions taking place within the concrete. In addition channels develop in the transition zone surrounding coarse aggregates, which enable 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 lightweight concrete.

The continuous, high-quality matrix fraction surrounding lightweight aggregate 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 lightweight aggregate. Bremner, Holm, and deSouza (1984) conducted measurements of the diffusion of the silica out of the coarse lightweight aggregate particles into the cement paste matrix using energy-dispersive X-ray analytical techniques. The results correlated with Khokrin's observations that the superior contact zone in structural lightweight concrete extended approximately 60 μm from the lightweight aggregate particles into the continuous matrix phase.

In addition, the contact zone in structural lightweight concrete is the interface between two porous media: the lightweight aggregate 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 normal density concrete 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 normalweight concrete indicates that, in the unstressed state, the permeability of the two concretes is about equal. However, at higher levels of stress, the lightweight concrete 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 that makes this effect of even greater importance. 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 considered when comparing laboratory and field conditions. Because of the initial absorption of water by the lightweight aggregate prior to mixing, this absorbed water can act as water for extended moist curing. The water tends to wick out from the coarse pores, of the aggregate into the finer capillary pores in the cement paste, thereby extending moist curing. This process is referred to as internal curing (As reported by Sugiyama 1999). The beneficial 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.

8.5   Influence of Contact Zone on Durability

The contact zone is the interface between the lightweight aggregate and the surrounding mortar matrix where a transition layer of material connects the coarse aggregate particle with the mortar matrix. Analysis of this linkage layer requires consideration of more than the adhesion developed at the inter face 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 matrix. 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.

For concrete to 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 water and chlorides 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, of lightweight concrete has been demonstrated to be significantly superior to that of normalweight concretes 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 lightweight concrete, 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 lightweight aggregate interface because of the elastic similarity of the aggregate and the surrounding cementitious matrix.

c.   Hygral equilibrium between two porous materials (Lightweight aggregate and a porous cementitious matrix) as opposed to the usual condition with normal density aggregate, 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 lightweight aggregate is moderately enhanced. However, when used in concretes containing normalweight aggregate, 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 8.7, 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 lightweight aggregate and the mortar matrix.

Figure 8.7. *Paste-Aggregate Interface*

Scanning electron microscope photograph of aggregate/matrix contact zone; from the cover of "*State-of-the-Art Report on High-Strength, High-Durability Structural Low-Density Concrete for Applications in Severe Marine Environments*", by Thomas A. Holm and Theodore W. Bremner.

Related Studies on the Contact Zone

Russian studies on the durability of lightweight 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 which X-ray analysis of ground keramzite taken before and after immersion in a saturated lime solution attested to the presence of a chemical reaction.
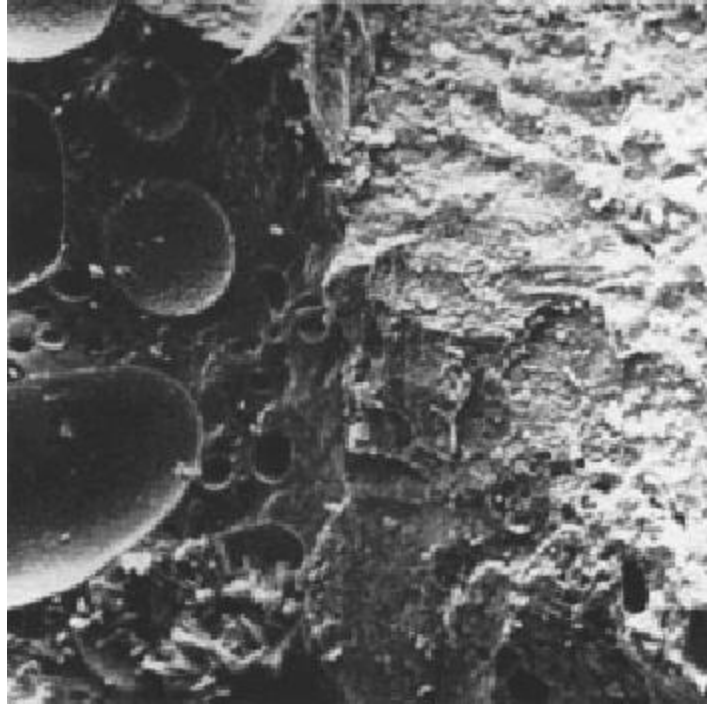
Figure 8.8. *Fractured surface of concrete from USS Selma. (Expanded aggregate is on the left, and the cement paste is on the right. Micrograph width is 550 μm) (from Bremner, Holm, and Morgan 1996)*

Khokrin (1973) also reported on microhardness tests of the contact zone of lightweight concrete and normalweight concrete, which established the width of this zone as approximately 60 μm. The research also concluded that the hardness of the matrix in the contact zone was more than 50% greater than outside the contact zone. The results of another investigation that included limestone, diabase, and rotary kiln expanded aggregates are shown in Table 8.3.

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

Table 8.3
Microhardness in and Beyond the Contact Zone (c/z) of Concretes
With Differing Water-Cement Ratios and Various Coarse Aggregates
(Khokrin 1973) (kg/m²)

| 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	126	68
Crushed diabase	81	79	---	---	---	---
Crushed limestone	81	81	---	---	---	---

However significant any reduction in compressive and tensile strength due to poor contact zone, 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 lightweight aggregate is considerably diminished, if not essentially eliminated in lightweight concrete, by hygral equilibrium that is established between two porous materials; lightweight aggregate and the enveloping cementitious matrix. This has been verified in practice by the examination of the contact zone of lightweight concrete split cylinders, as well as by visual examination of sandblasted vertical surfaces of building structures. This observation should not be surprising because, with structural lightweight concrete, the aggregate/matrix interface is a boundary between two porous media, while with normalweight 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 provided reassurance of long-term interaction of the components of the concrete 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 rarely 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 test 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 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.

8.6   Long-Term Performance

Since 1980, Natural Resources Canada, through its Canadian Centre for Mineral and Energy Technology (CANMET), has installed numerous lightweight concrete prisms at the U.S. Army Corps of Engineers, Treat Island, Maine, exposure site. The specimens are prisms of dimensions 1 by 1 by 3 ft. (0.305 by 0.305 by 0.914 m). They are located on a wharf at mid-tide level so that they are subjected to twice-daily tidal cycles that result in over 100 cycles of freezing and thawing per year. All lightweight concrete specimens were air entrained.

Figure 8.9. *Treat Island, Maine showing US Army Corps of Engineering Severe Weathering Station (Photo by R. Holm, 1998)*

Malhotra and Bremner reported that "with normalweight 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 concrete this is not noted; deterioration occurs by a uniform loss of the surface layer" (1960). The paper goes on to report that "at this stage all specimens having cementitious contents of 607 lb/yd³ (360 kg/m³) or greater show excellent performance". An analysis of these data indicates comparable performance of lightweight concrete with normalweight concrete when compared at similar ages and with similar binders.

Structural lightweight concrete has been placed at Treat Island as part of five separate investigations starting in 1980 and also in 1985, 1986, 1988 and 1990. More than 70 specimens are situated in one long line on the dock as shown in Fig. 8.10 and are referenced as Phase III, VI, VII, X and XI in the "2004 Review of Programs Natural Weathering Exposure Station, Treat Island", USA Corps of Engineers Report.

Figure 8.10 *Treat Island, Maine research center*
Lightweight concrete specimens in bottom row. Exposed
To two tidal cycles per day (R. Holm, 1998)

The 1980 Phase III program, by far the largest of the five investigations was designed to parallel earlier test on normalweight concrete that evaluated the influence of binder content type and ranges of additions of palletized blast furnace slag (PBFS) and content.

The 1985 Phase VI was under taken to evaluate the performance of air-entrained, semi-lightweight concrete incorporating 2.5% by mass 50mm long steel fibers. The normal cementitious content of the mixture investigated were (375, 500 and 625 kg/m³) that included 13.5% fly ash and 6.5% silica fume.

The 1986 Phase VII evaluated the performance of high-strength, semi-lightweight concrete delivered from a commercial ready-mix plant with cement binder contents of 843 pcy (500 kg/m³) and 674 pcy (400 kg/m³) that also included 6 and 7.5% silica fume respectively.

Phase X (1988) was designed to determine the comparative performance of air-entrained, semi-lightweight silica fume concrete made with structural lightweight aggregates from sources in Canada, Germany and the USA.

Phase XI (1990) was under taken to determine the comparative performance of semi-lightweight high volume fly-ash concrete made with aggregates from sources in Canada, Germany and the USA.

In general, when considering all the various air-entrained semi-lightweight concrete mixtures that also included combinations of slag, fly ash, silica fume, steel fibers, the durability of structural lightweight concretes tested at the USACE severe weathering site, was shown to be satisfactory when the binder content was adequate i.e. >600 pcf (295 kg/m³). These good results are in total agreement with the reported excellent performance of ships constructed with structural lightweight concrete exposed in sea water for 55-80 years. (Sturm, et. al., 1999)

8.7   DESIGN FOR 100 YEAR LIFE

History was made in North Carolina on August 16, 2002 when a new bridge opened. (See Fig. 8.11). The Virginia Dare Bridge is the longest bridge in the Carolinas; at 5.3 miles, it is 2 miles longer than any bridge in the Carolinas, and one of the longest concrete bridges on the East Coast. This bridge is designed to last a century, twice as long as the preceding generation of bridges. In the summer of 1996 the State of North Carolina and the Department of Transportation determined that a new bridge was required to replace the existing bridge connecting the Dare County mainland with the hurricane-prone East Coast. Traffic volume is expected to more than double by 2020. The present bridge, opened to traffic in April 1957 and nearly 43 years old, is unable to handle that amount of traffic and already causes congestion for vacationers. It also increases the risks associated with evacuation in the event of a hurricane.

At a cost of $91 million dollars, construction took a little more than 3.5 years and was completed in the summer of 2002. This is a pre-cast, pre-stressed concrete beam bridge with a cast-in-place lightweight concrete deck (riding surface). The logistics were challenging-each component had to be made on site or brought in by water or specially designed rail. Two concrete batch plants were needed-one was built in Manns Harbor, the other was built on a floating platform.

Figure 8.11. *The Virginia Dare Bridge spans the Croatan Sound From Manns Harbor to Roanoke Island, North Carolina.*

Proportions: For Lightweight Concrete, the North Carolina DOT uses a cement content of 715 lbs. per cubic yard with a 20% replacement rate of cement for class F fly ash using 1.2 lbs of fly ash to replace 1.0 lbs. of cement. The mix was as follows"

- 572 lbs cement
- 172 lbs fly ash, class F
- Approx. 825 lbs of 3/4" ESCS lightweight aggregate
- 6% entrained air
- Strength 4500 psi at 28 days
- Maximum fresh density = 120 lbs/ft³
- Maximum equilibrium density = 115 lbs/ft³