

DURABLE CONCRETE



Compilation 24

American Concrete Institute

Durable Concrete

ACI Compilation 24

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Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The Material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

W. Barry Butler

Chairman, ACI Committee 201

Durability of Concrete

James R. Clifton

Chairman, ACI Committee 365

Service Life Prediction

On The Cover: The largest public transportation project currently underway in the United States is in Seattle, Washington. This photo shows a section of Interstate 90 under construction between I-405 in Factoria and its intersection with I-5. When completed in 1994, the project will contain 1.1 million cu yd (0.85 million cu m) of concrete. (Photo by Robert I. Pearson, Engineering Editor, *Concrete International*.)



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Documents

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Long Service Life of Concrete

by L. H. Tuthill

The length of concrete service life depends on the correct choice and use of materials. Problems such as alkali-silica reaction (ASR) and the prospect of sulfate attack and corrosion need early and proper identification. These conditions must be controlled with resistant materials.

An often overlooked requirement for long concrete life is preventing or minimizing thermal cracking of the massive structural concrete found in power plants, bridge piers, foundation elements, and thick linings of large tunnels.

Ordinary concrete in municipal use, especially in new subdivisions, is often short on durability and exhibits cracking due to failure to follow the most fundamental rules of good practice: protecting the concrete from freezing weather, using enough cement, controlling slump, providing an ample number of joints, and curing properly. Sidewalks and driveways are too often distorted and disappointing.

Fortunately, the engineering and inspection process essential to long term results is in use and in force during the construction of most highway, water, power, and large building projects. Sometimes it is

necessary to remind those in charge that specifications and building standards requiring a "life of 50 years" are insufficient when the proposed facility, after 50 years of service, must be in condition to last 50 years again — and again. Hoover Dam has had its 50th birthday. There must be no scrimping. Every item and aspect must correctly use the best materials for long life under the service conditions.

Bids for doing such durable work should be taken on this basis, with the requirements and materials described in ample detail in the project specifications. Meetings should be held before the bid opening to assure that builders understand the standards and quality of performance that they will be required to furnish.

The quality, long serviceability, reduced maintenance costs, and continuing good appearance is well worth the added investment.

Aggregates

First, it should be determined if there is a preferable choice among the sources of aggregate and between crushed and better-shaped natural materials — gravel and sand. With crushed rock aggregate, the stronger and harder the stone, the flatter and sharper-edged the smaller sizes tend to be, particularly those less than 1/2 in. This detracts from workability and makes more sand and water necessary, especially in those mixes commonly used

in building construction with little large coarse aggregate.

If nothing else, coarse aggregate should be chosen by uniformity of grading and minimum variation of the percent of significant undersize (passing a screen having openings 5/6 the nominal minimum size of aggregate)¹ as provided by the various vendors and batching plants. Those having finish screening as the coarse aggregate goes to the batch plant bins will have minimum irregular amounts of significant undersize in sequential batches and correspondingly less variation in workability and slump. With the small, 1/2 in. maximum size of aggregate (MSA) sometimes used, it is unlikely to be much of a problem. Varying amounts of significant undersize in the successive batches will cause workability variations that will prompt addition of more sand and water to the proper mixes.

Cement

In most areas of construction, a number of different brands and types of cement are available. They should not be taken for granted. An effort should be made to see that the type and composition of cement used will best serve the particular service circumstance and environment.² Probably most concrete will be amply served by Type I cement. Up to one extra sack of it per cubic yard will often, at little or no overall extra cost, avoid the occasional need for Type III cement. How-

This article first appeared in SP-122, "Paul Klieger Symposium on Performance of Concrete," an ACI special publication containing 24 papers presented at The Paul Klieger International Symposium on Performance of Concrete, sponsored by ACI Committees 201, Durability, and 222, Corrosion of Metals.

There is no reason whatsoever for concrete placed anywhere to lack durability due to sulfate attack.

ever, Type I cement is not appropriate in some situations.

A common example occurs when the most favorable aggregate for a project contains alkali-reactive elements, creating a need for a low-alkali cement.³ Another special cement that can be a serious mistake to overlook is sulfate-resisting Type V cement. It has no more than 5 percent of tricalcium aluminate (C_3A) (the less the better). Concrete that will be in contact with soil and soil water should have this protection if it contains sodium sulfate. Accordingly, tests for sulfates should be made for concrete in low-lying flat areas, unless concrete already in the area made without Type V sulfate-resisting cement shows no evidence of attack and deterioration after several years of service. The composition of sulfate-resisting cement has been known for more than 55 years and it has performed well,⁴ so there is no reason whatsoever for concrete placed anywhere to lack durability due to sulfate attack.

Pozzolans

Any concrete that will not be sheltered from an occasional wetting will be improved by using a pozzolan for 20 percent or more of its cementitious material. This is particularly true if its least dimension is more than 12 in. and if reduced heat of hydration may increase its freedom from thermal cracking.

There are two primary pozzolans available today: Class N and Class E. ASTM C 618 also mentions Class C pozzolans, but fails to caution that some of them may not reduce heat of hydration or alkali-aggregate reactivity. Thus, they should not be used where either of these may be a problem unless there is strong evidence that a particular Class C pozzolan is reliably equal to a Class N or F pozzolan.

Pozzolans are siliceous or siliceous and aluminous materials which in themselves possess little or no cementitious value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Class N pozzolans are natural materials, some naturally fine, some requiring calcining and grinding. Class F pozzolans are fly ash pozzolans produced from burning anthracite or bituminous coal. Class C pozzolans are fly ash pozzolans produced from lignite or sub-bituminous coal.

Pozzolans were one of the earliest of concrete materials. They were used in the Roman Aqueduct. A section of this famous structure may be seen in the basement of the Smithsonian Institution in Washington, D.C.

Entrained air

Since the early 1940s, it has been known that wet concrete exposed to alternate freezing and thawing would be much more durable if the concrete contained a certain amount of extremely small air bubbles. The more mortar the concrete contains the greater is the percent of air needed. Air-entraining admixtures are used to produce air bubbles in the concrete.

Unfortunately, this entrained air has adverse effects that should be kept to a minimum. As the percent of entrained air becomes greater, slump increases and strength decreases. Thus, smaller amounts of air needed only for workability are usually sufficiently provided by water-reducing admixtures.

For example, to ensure ample workability for a certain tunnel-lining concrete, 6 percent entrained air and a normal dosage of a water-reducing admixture was specified. It was soon found that with the high amount of entrained air from those two sources, the variations in slump were considerable and troublesome. The air-entraining admixture was

Concrete should be designed to produce no more strength than necessary to permit the lowest practical cement content.

omitted and the air entrained by the water-reducing admixture was found to be ample for workability. The variation in slump was thus reduced to acceptable limits and the concrete was 17 percent stronger.

Admixtures

Several good water-reducing, workability increasing, and strength enhancing admixtures are available and for most ordinary structural concretes have been worthwhile additions. More recently, a new and special flowing-concrete producing admixture has come into use. Flowing concrete with a small MSA is required to properly enclose the closely spaced clusters of reinforcing bars needed in modern, ductile, reinforced concrete frames. The special admixture reduces water requirements and increases strength. It is usually used with silica fume, an extremely fine-grained pozzolan material that significantly reduces permeability and prevents corrosion of reinforcing steel, as well as having the other pozzolanic benefits.

Thermal cracking

Thermal cracking occurs on the surface of large structural elements unless fundamental procedures are taken to prevent it. If the interior becomes too warm as a result of hydration of the hardening concrete, or if the surface becomes too cool compared to the interior, cracks will appear. Fortunately, it is practical to control the interior and surface temperature differences.⁵ Cracks are likely to appear if the differential is more than 30 F.

Thermal cracking is most common in the massive structural concrete of water project construction, power plants, pumping plants, piers, abutments, and large building foundations. Every available means should be used to keep the

the temperature from heat of hydration in such concrete. Concrete should be at the lowest practical temperature as placed, and should be designed to produce no more strength than necessary to permit the lowest practical cement content. Modern admixtures may assist in achieving this. Forms should not allow fast changes in surface temperature or be removed in a way that allows fast temperature changes.

These practices and restraints do make a difference, as is clearly evident among the large pumping plants built by different agencies in California.

Curing

Curing is the final concern. For concrete to reach the ultimate potential of its materials, it is necessary for the cement mortar binding it to contain moisture until these ultimate strengths are developed. When the concrete is dry or a portion becomes dry, such as the important surface layer, nothing happens and the concrete is less than it should be. For at least its first year, concrete is benefitted every time it is wetted by rain, but this does not diminish the importance of job curing. Ideally, for at least the first 10 days, the concrete's water content should be maintained and its surface kept damp. Preferably, the concrete should remain covered for another week to slow drying.

Obviously, in areas or seasons where weather is cool, damp, and moist, curing requires little assistance other than to make sure there are no gaps in these favorable concrete curing conditions, particularly for the first few days. In warm, dry conditions, systems such as a saturated cover kept wet by automatic sprinklers should be used to provide fully adequate wet curing. Soil soaker hoses on the top of formed, non-massive concrete can be arranged and operated to keep the concrete wet all over by loosening the forms slightly as soon as the concrete is strong enough to safely do so. Surface-sealing compounds

are adequate for curing slab work such as pavements and canal linings placed on a well-wetted subgrade, which ensures a long-term supply of moisture within the concrete for full curing; they are not recommended as a first choice for formed concrete.

Conclusion

Concrete is a forgiving material. In many cases, despite construction practices that fall considerably short of bringing out its fullest quality and serviceability, concrete does serve its intended and expected purpose. Nevertheless, it is advisable and worthwhile for any important concrete project to take advantage of all the practical practices that will contribute to insuring the second and, maybe, third "50-year-life." The net actual cost will be very little, if any, considering the assured excellence of results.

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Selected for reader interest by the editors.

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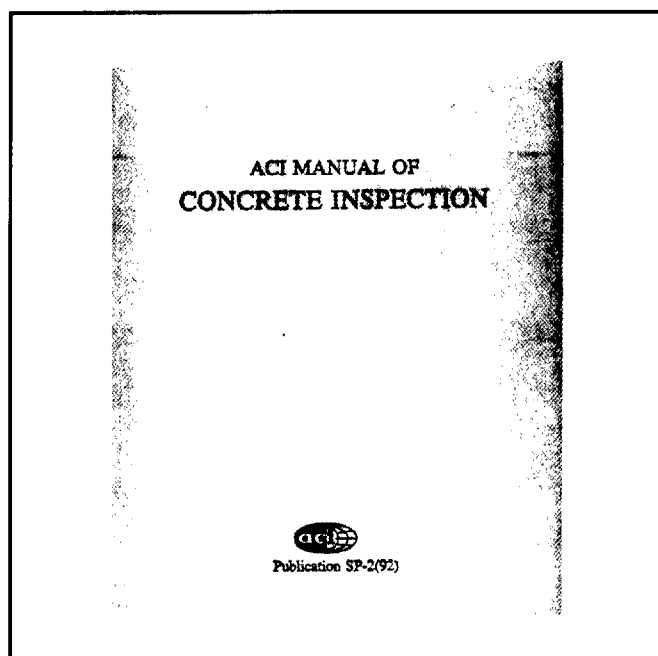
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Design Considerations for Service Life

By Eivind Hognestad

Concrete structures designed for service life are proportioned and built to serve their intended functions in a specified environment over a required number of years. Maintenance costs, if any, should be reasonable. The concept of design for a service life required by the owner or user of a structure is relatively new in concrete construction. However, it has long been practiced in design and maintenance of aircraft and automobiles.

North Sea structures

My first practical contact with service life prediction was in January 1972. At the time I was serving the Phillips Petroleum Company as a consultant regarding design and construction of the first major concrete structure for the North Sea:¹ a 1-million barrel oil tank (Fig. 1) to be located in the Ekofisk Oil Field.

Construction had barely started when Phillips headquarters requested an estimate of the service life expected from the oil tank. A life in excess of 30 years was needed in terms of production plans for gas and oil.

This inquiry may have been related to a letter received by the Chairman of Phillips in January 1972 from a prominent corrosion engineer. A halt of the Ekofisk Tank construction was strongly urged unless it was "properly instrumented to detect the onset of corrosion of the steel and inception of other causes of catastrophic failure." The letter further suggested that steel will rust even in the "best" of concrete, and that sulfur compounds in crude oil have been found to cause stress corrosion damage, such as "the catastrophic failure of the Silver Point Bridge (across the Ohio River) that resulted in the death of 46 people."

In a response prepared at the re-

quest of Phillips, I referred to three major U.S. field investigations carried out in 1969-71 reporting excellent durability of reinforced and prestressed concrete structures exposed to sea water.^{2,3,4} In addition, a 1968 book by Odd E. Gjörv⁵ reported performance inspections of 219 harbor structures built on the Norwegian coast with aggregates and cements similar to those used in the Ekofisk Tank. Gjörv concluded: "After 50 to 60 years in service, reinforced concrete wharves still showed a remarkable ability to withstand the combined effects of the most severe marine exposure and heavy structural loads . . . Where corrosion was observed, it was frequently in connection with improper placing of the concrete or the reinforcement." This performance is impressive indeed considering that these Norwegian structures were completed well before air entrainment was invented.

The Ekofisk Tank was to be built using entrained air, plus a cement content 50 percent higher and better quality control than the old

Norwegian structures inspected by Gjörv. Hence, my advice to Phillips concluded: "Provided that high quality materials, good engineering practices, and adequate construction are used, corrosion of the reinforcement in the Ekofisk Tank will be highly improbable."

The project was completed as planned and entered service in 1973, 17 years ago. The need for a durable, high quality structure was foremost in the minds of the entire construction team. Today, twenty-one large concrete structures are serving the petroleum industry in the North Sea.⁶ They are all doing well. Design for service life is feasible. It works!

Hot climate structures

Also in the early 1970s, specifications were written for the reinforced concrete foundations of a large petrochemical plant located in the Arabian Gulf. Without consideration of service life in a highly corrosive environment, it was written that "all concrete shall be mixed with sea water." The chloride in the sea caused rebar corrosion to begin



Fig. 1 — Oil tank under tow from Stavanger, Norway, to Ekofisk Field in the North Sea, Spring 1973.

shortly after completion of construction (Fig. 2). Cracking, spalling, and rust became widespread within a few years, calling for very extensive repairs. One structure collapsed (Fig. 3).

The service life concept

The owner of a structure must indicate a required service life clearly expressed in years at the very beginning of planning and design. This service life should later be shown on every concrete drawing and specification. Complete reliance on compressive strength must be abandoned.

Environmental exposure

Design codes

The British Standard 8110, "Structural Use of Concrete,"⁷ outlines exposures commonly met in Northern Europe (Table 1). The Euro-International Concrete Committee is preparing a 1990 Model Code⁸ that lists exposures with details beyond BS 8110, though "seawater environment" did not include the extreme case of sea water splash where air temperature frequently exceeds 32°C (90°F), such as the Arabian Gulf, Bahrain, the Florida Keys, Hong Kong, and the Philippines. The 1988 German Code⁹ lists exposures in a manner similar to the British and CEB-FIP Codes.

Various exposure conditions and corresponding steps in concrete design and construction have been thoroughly discussed by ACI Committee 201¹⁰ in "Guide to Durable Concrete." The 1989 ACI Building Code¹¹ was reformatted to emphasize durability requirements for freezing and thawing exposures, sulfate exposures, and corrosion of reinforcement.

Special exposures

Sulfate exposure is an old problem area that is quite well understood through research and practice. Design codes commonly stipulate SO₃ contents of exposure beyond which a cement with reduced C₃A content should be used. As alternatives, fly ash or granulated slag may be added to the concrete. Maximum

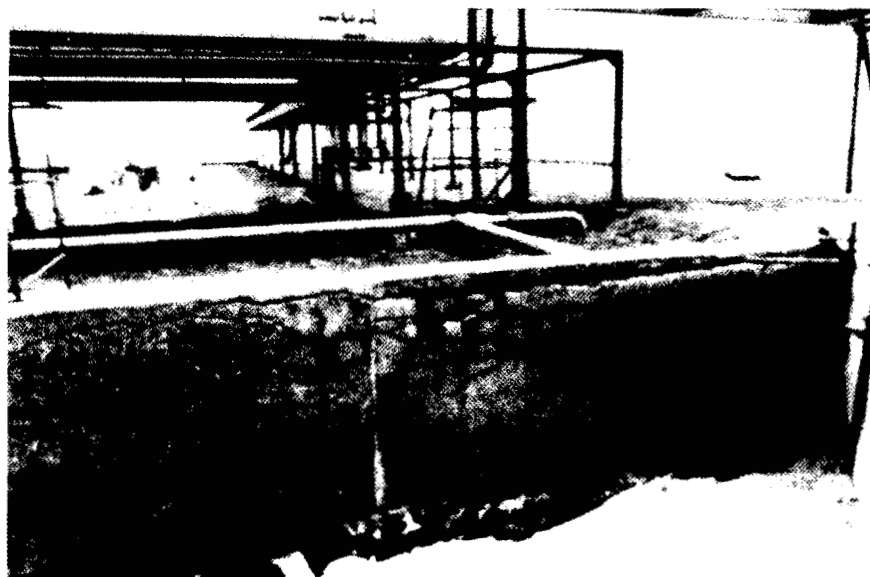


Fig. 2 — Corrosion of reinforcement in canal wall containing concrete mixed with sea water, Summer 1982.

water-cement ratios are also specified for the various exposures.

Alkali-aggregate reactivity is generally covered by reference to aggregate specifications. For example, the 1989 ACI Code refers to ASTM C 33-86. But the 1985 British Code outlines the use of cement with less than 0.6 percent Na₂O and addition of fly ash or granulated slag.

Fire exposure is covered by municipal codes rather than the 1989 ACI Code, while the British Code contains criteria for fire resistance.

Earthquake effects are fully covered by a special chapter in the ACI Code. This is also done in other countries of serious earthquake risk.

Resistance to exposures

In a 1986 paper, Dr. G. Somerville¹² summarized design steps providing resistance to exposure in terms of "the four Cs": Constituents of concrete; Cover of reinforcement; Compaction during placement; and Curing of the finished work.

Constituents

Several types of portland cement, such as normal, rapid hardening, low heat, sulfate resisting, and blended cement are useful under various exposures, together with such materials as granulated blast furnace slag cement. Fly ash, granulated slag, and silica fume are also suitable as admixtures, particularly for severe exposure and very high strength concrete.

Chlorides are a significant cause

of reinforcement corrosion. Specifications must carefully block all paths by which chlorides may enter the concrete mix. The use of calcium chloride as an accelerator should be completely banned, and aggregates dredged from the sea should be thoroughly washed in fresh water. Sea water should never be used to mix concrete.¹³

The water-cement ratio strongly influences service life. K.C. Clear¹⁴ placed 124 reinforced concrete slabs in an exposure yard to determine the time to initiation of rebar corrosion. The slabs were exposed to 830 daily applications of sodium chloride solution. Clear developed a model for the time to onset of corrosion, which should be somewhat less than the service life:

$$\text{Life to Corrosion Onset} = \frac{(129 \times \text{Cover}^{1.22})}{(w/c \times C^{0.42})} \quad (1)$$

Three governing variables were observed. Life (years) was inversely proportional to w/c , increased with the concrete cover (inches) to the power 1.22, and decreased with increasing chloride content (parts per million) of the exposure solution to the power 0.42.

Fig. 4 shows prediction by Clear's life model for exposure to sea water with 2 percent chloride. Life decreases in proportion to increasing w/c . Concrete cover over the reinforcement has a powerful life-extending effect. **Documents**

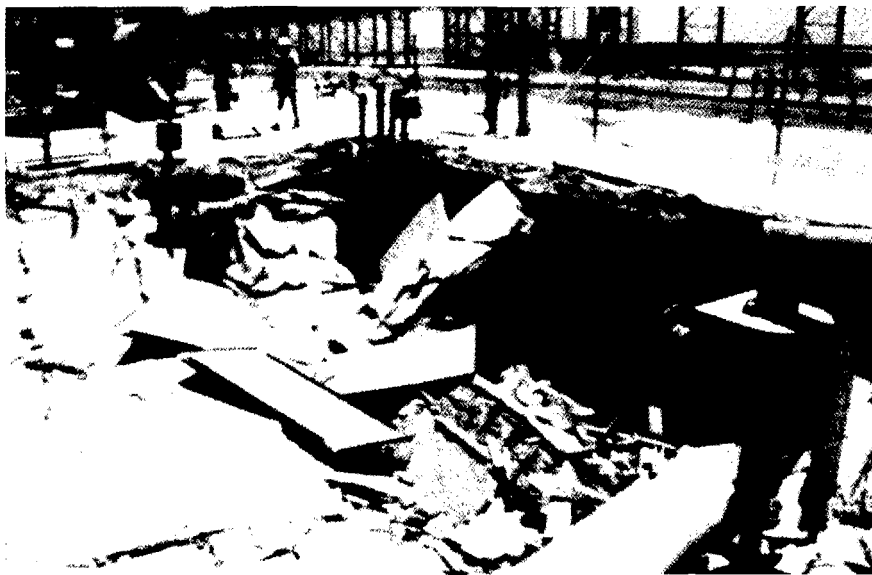


Fig. 3 — Collapse of roof structure containing concrete mixed with sea water, Spring 1984.

periments have omitted temperature as a variable. Corrosion will be substantially accelerated in the Florida Keys as compared to Newfoundland. The intensity of corrosion roughly doubles for each 10 C (18 F) increase in temperature.

Concrete cover

Cover over reinforcement is a critically important variable governing the service life of concrete structures. Clear's life model indicates that life increases with cover. The 1988 German Code points out that concrete cover serves several important functions affecting the performance of reinforced structures:

- It ensures the bond between concrete and reinforcing steel required to develop structural strength.
- It protects the reinforcement against corrosion.
- During fire exposure, it protects reinforcement against early loss of strength due to heat.
- It provides resistance to mechanical abrasion.
- Its surface texture and color govern the architectural appearance of the structure.

All codes throughout the world are currently recognizing the need for more detailed cover specifications to serve its various functions.

Compaction

In field investigations I have commonly found that premature corro-

sion of reinforcement is caused by localized areas of inferior concrete compaction, particularly at construction joints. Effective inspection is essential to proper consolidation. The 1985 British Code outlines the principles of proper compaction. The 1989 ACI Code also refers to ACI Committee 309, "Guide to Consolidation of Concrete."¹⁵

Curing

All properties of concrete improve with extended wet curing. This is particularly so for permeability, which strongly affects durability and service life. The British Code gives curing requirements in considerable detail. The ACI Code primarily refers to ACI Committee 308, "Standard Practice for Curing Concrete."¹⁶

Corrosion of reinforcement

Prevention of rebar corrosion¹⁷ must be given close attention in design, particularly when a service life of 50 to 100 years is indicated. We have noted that chloride content in the concrete mix or in the environmental exposure will decrease service life sharply. Air entrainment, low w/c, adequate concrete cover, and proper compaction are all strongly beneficial. More recently, epoxy coated reinforcement has provided excellent protection against corrosion. The NCHRP is currently planning further research

on epoxy coating for reinforcing steel, with emphasis on failure of coatings on fabricated bars in hot climate.

Numerous papers have been written about an additional design variable, namely flexural crack width. A control of such cracking was introduced in the 1971 ACI Building Code to permit the use of rebars having yield strengths above 40,000 psi. By the nature of the cracking mechanism, crack width limitations call for as thin a concrete cover as possible. This is in direct contradiction with the need for a substantial cover to prevent corrosion.

Exposure tests were run by the U.S. Army Corps of Engineers in the tidal zone of the Atlantic Ocean at Treat Island, Maine.¹⁸ They exposed 76 reinforced concrete beams with 2 in. cover to reinforcement at low tide. The beams were spring loaded to steel stresses from zero to 50,000 psi. Data after 25 years of exposure clearly indicated that crack width and steel stress may *not* be significant variables with respect to corrosion and service life.

The role of crack width in corrosion protection has also been studied by the German Committee on Reinforced Concrete (DAfS). Peter Schiessl reported in 1989:¹⁹ "The DAfS has produced a detailed report regarding the influence of cracks on the durability of reinforced concrete . . . The intensity of corrosion at cracks is subject to a wide scatter. The width of cracks crossing the steel at a right angle is of secondary importance to corrosion protection, as long as the yield strength of the steel is not exceeded, that is as long as the cracks are not wider than 0.4 to 0.5 mm [0.016 to 0.020 in.]. Corrosion protection is governed by the thickness and permeability of the concrete cover."

My words from the Commentary to the 1971 ACI Code remain unchanged in 1989: "Crack width is inherently subject to wide scatter even in careful laboratory work and is influenced by shrinkage and other time-dependent effects. The best crack control is obtained by

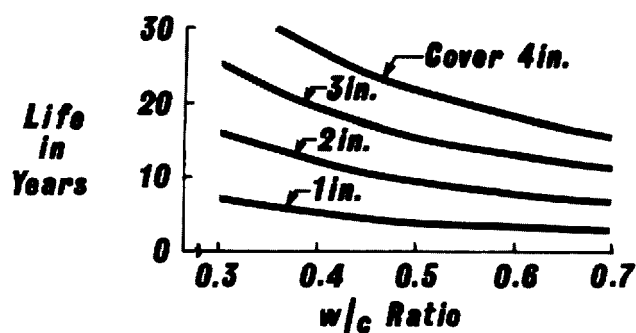


Fig. 4 — Clear's equation for sea water exposure with 2.0 percent chloride.

steel reinforcement is well distributed over the zone of maximum concrete tension."

Epilogue

Design for service life calls for sharply increased emphasis on the construction process and planned maintenance. Well done structural design calculations and drawings are not enough. Several factors beyond those discussed in this article will influence service life. Therefore, the design process must be broadened to include thorough consideration of:

- Initial and periodically repeated coatings.
- Cathodic protection systems.
- Competent and thorough inspection.
- Planned systematic maintenance.

Flexural crack width has been studied extensively in recent years. Some field observations indicate, however, that crack width is not a significant variable influencing service life.

Some 25 years ago, the Illinois Toll Road was built North and West of Chicago. In January 1990, I drove the Northwest spur from the Wisconsin border. I noted that the pavement was approaching the end of its service life. Shortly after I found in the Chicago Tribune that the entire system will be rebuilt during 1990-91. Wasn't this a major design mistake? Shouldn't these pavements have been designed to last at least 50 years?

Per Anders Ortendahl, Chief Executive Officer of the Swedish Road Administration, announced in October 1988²⁰ that Swedish bridges henceforth shall be designed for a

service life of at least 100 years. This increase from the current 65 years shall be achieved through improved quality.

In the spring of 1990, Chicago's relatively new Major Richard Daily inspected his streets and was horrified by the winter's damage. "We must find a substitute for salt to melt the ice without ruining the concrete," he told the press.

Finally, I would like to repeat that remarkable progress in design for service life would result if all known principles were extensively applied in design and construction.

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Table 1 — Environmental exposure as out-lined in British Standard 8110.

Environment	Exposure conditions
(1) Mild	Concrete surfaces protected against weather and aggressive exposure (ACI 318-89 "interior").
(2) Moderate	Concrete surfaces sheltered from severe rain or freezing while wet. Concrete surfaces continuously under water. Concrete in contact with non-aggressive soils.
(3) Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying, occasional freezing, or severe condensation.
(4) Very severe	Exposed to sea water spray, deicing salts, corrosive fumes, or severe freezing.
(5) Extreme	Exposed to abrasive action, such as sea water carrying solids, or from machinery, or vehicles.

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Point of View

Research Innovations for Durable Concrete

by G.M. Idorn, Vagn Johansen and N. Thaulow

Concrete production methods and materials have changed radically during the last 20 years. Concurrently, laboratory research on concrete and on concrete source materials has been furnished with powerful analytical and testing instrumentation and corresponding basic scientific knowledge. The aim of this paper is to stimulate discussions about how the parallel developments in practice and research can be effectively integrated for improved quality management of concrete in production and for improved performance quality of field concrete.

Concrete durability pros & cons

Fig. 1 shows the basic principles of manufacturing technology in modern commodity industries. From raw materials to products, the processing phase is monitored to ensure that the specified qualities are attained with a high degree of certainty and optimum economy. The figure first appeared as a display of a strategic goal setting for concrete research and technology development in a National Materials Advisory Board (NMAB) report:¹ "The Status of Cement and Concrete Research in the United States." At that time the peak of the "energy crisis" was about to be overcome, while the "concrete durability crisis" was beginning to attract public attention.

Between 1970 and 1980, in a remarkable response to the energy crisis, the cement industries reduced the energy consumption in cement manufacture from about 1450 kcal/kg cement in the best conventional plants to about 750 kcal/kg cement

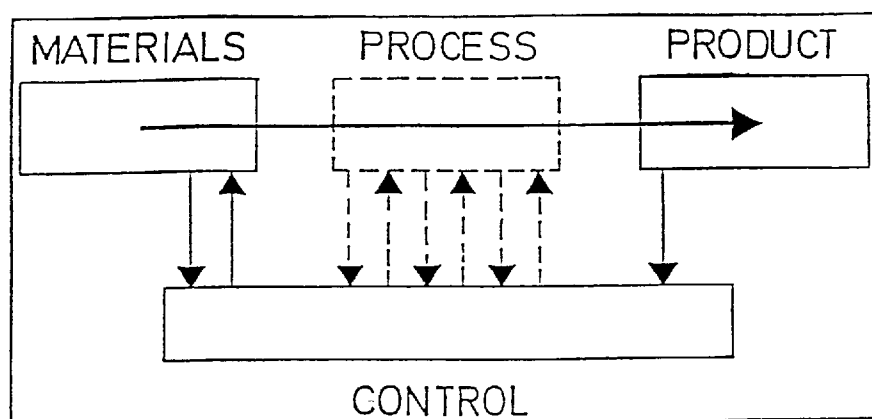


Fig. 1 — The flow chart shows the principle of industrial manufacturing technology with knowledge based monitoring processing from source materials to predesigned products. In conventional concrete making, the process control is made with samples of the fresh concrete that are tested for slump and standard cured to attain compressive strength. The process reactions in the concrete in place are not monitored, and the properties of the hardened concrete in place are therefore not assessed.

with dry process, suspension and precalciner kilns. A contributing factor was the implementation of the processing monitoring concept shown in Fig. 1. The concrete construction and precast sectors did not attempt similar innovations in the same decade. One could say that concrete production technology continued to waste the energy that the new cement making technology saved.

The NMAB report showed that institutional research had declined to a terribly low level of impact on practice, and did not have the capability and structure in 1980 to combat the rising public concern about the economic penalties of concrete deterioration. ACI became alerted and launched the Concrete Materials Research Council in 1984 (now known as the Concrete Research Council). The United States

government followed in 1987 with the Strategic Highway Research Program (SHRP), allocating about \$12 million (U.S.) to research on improving concrete highway pavement technology. SHRP introduced new goal oriented requests for concerted research project planning and management, and succeeded in attracting research cooperation with agencies and private research companies in other countries.

The SHRP concrete research projects made it obvious that industrial process monitoring (Fig. 1) does not yet exist as a mainstream technology concept for concrete production, and that the existing institutional research is not adequate for its development. This is an increasingly paradoxical situation because the levels of mechanization and automation of concrete construction sites and in precast plant and



Fig. 2 — A prestressed concrete railway tie with cracking due to alkali silica reaction. In the mid section, where the prestressing is fully effective, the cracks are longitudinal. At the end sections with lower inherent stress levels the cracking approaches map cracking, which is typical for deleterious alkali silica reaction.

vanced enough to profit from the introduction of process monitoring. The new European ISO 9000 certification system now requires the establishment of such systems with access for external quality auditing of all kinds of software and hardware manufacturing in the common market region. This is certain to raise the demands on competitiveness in concrete construction and precast industries inside and outside the EC countries, including in North America.

Initial moves

ASTM C 1074-87² "Standard Practice for Estimating Concrete Strength by the Maturity Method" is a recent precursor to the introduction of the maturity method for controlling the concrete hardening rate, which is dependant on the temperature rise due to heat of hydration. The ASTM test method does not comprise adaptation of monitoring of the heat-strength development in concrete during curing in place. This monitoring application of the maturity method has been adopted in major construction

works in Europe,^{3,4} and in precast industries.

Between 1980 and 1985, the former Atlantic Cement Company invested in a comprehensive research and development program in conjunction with the introduction in the eastern United States of ground, granulated blast furnace slag (GGBS) for blending with portland cement. The company researched the effects of the new materials in concrete⁵ and emphasized the benefits, particularly the impact of GGBS on the rheology of the fresh concrete and its curing.⁶ This research and development was subsequently acknowledged by ACI.⁷

Many less publicized research studies have also been made over the years by the ready-mixed concrete and chemical admixture industries to learn about the influence of special materials on concrete properties during processing.

The SHRP research project C201,^{8,9} "Microstructure of Concrete" is a recent American step towards deliberately using heat developed during curing to improve the economy and durability of concrete in service. The research focussed on ways to improve the microstructure of concrete by optimizing the packing and rheology of the fresh concrete, by evaluating the need to apply maturity-based curing technology, and by permeability measurements of hardened concrete. The final SHRP concrete program C206, "Optimization of Highway Concrete Technology" presents computerized concrete packing programs¹⁰ and curing technology tables.¹¹

Requirements for concrete durability research

The proceedings for major international conferences on durability held in recent years in the United States and Canada^{12,13,14,15} give a fair picture of the general state of research on concrete durability. The extent to which conclusions drawn from limited laboratory test series are presented in many papers as valid for field concrete on a 1:1 model scale basis is striking, if not frightening. SHRP was to a large extent forced to accept this kind of research philosophy as the prevailing working basis in several project contracts. This could not lead to a

concerted approach to new production technology for concrete highways, although a quite impressive number of research "products" have been offered for broad application in highway engineering practice. Concrete research methodology must be profoundly changed to reflect the following conditions:

- Research and practice must recognize that the durability of site and plant-made concrete is conceived during its production.
- Quality assurance in concrete production is not ensured by sampling and testing under arbitrary standard laboratory conditions.
- Research cannot simulate the properties and behavior of field concrete by studying small samples of homogenized cement paste, mortar bars, and concrete cured and stored under standard conditions.

An early ASTM perception

ASTM C 33,¹⁶ "Standard Specification for Concrete Aggregates," introduced the advice that evaluation of an aggregate "should be based upon judgement and on the interpretation of test data *and examination of concrete structures containing the aggregates and cements for use in the new work*" (authors' italics). This refers to the ASTM C 227 and C 289 methods for testing the alkali-silica reactivity of aggregates. In some ways, these two methods elegantly simulate the mechanical effects and the chemical nature of the reaction, albeit under arbitrary reaction conditions. ASTM C 33's good advice unfortunately has been long forgotten both in research on alkali-silica reaction and in concrete specifications requiring use of the test methods for acceptance of aggregate. The consequences of this oversight on the progress of eliminating alkali-silica reaction, now a worldwide cause of cracking in field concrete, are serious:

- Aggregates which pass the tests sometimes cause cracking in field concrete, while aggregates which do not pass the test often cause only a harmless reaction.
- Since 1940, where cracking in field concrete has been found to be caused by alkali-silica reaction there are usually many more structures with the same kinds of reactive aggregates and no cracking — only a harmless reaction.



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• Research continues in many countries to develop new, refined laboratory test methods disregarding the C 33 advice, despite acknowledging the reasons for it.¹⁷

Clearly, the alkali-silica reaction issue of concrete durability will not go away before the ASTM C 33 recommendation has regained its original recognition, and systematic analyses of both deleterious and harmless reactions in field concrete are incorporated in major research programs.

Renovating research

The sound advice in C 33 is relevant today as a basic principle for all kinds of laboratory testing and examination of cement paste, mortars, and concrete. The entire research profession must realize that the more sophisticated and numerous the data and observations in laboratory studies become, the more indispensable it is to convert such study results truthfully to make them valid for field concrete, both in production and under field performance. This is now the basic requirement for an urgently needed, long overdue renovation of general strategy and methodology in concrete research.

Two equally important aspects of this renovation will soon turn into profound changes of working methods and achievements for many currently employed in concrete research. Firstly, realistic model laws must be developed for the transfer of laboratory results to accurate information for field concrete behavior. Such model laws are routinely used in other branches of industrial research, and are applicable for concrete technology innovations.

Secondly, the same basic modeling principles must be developed for use when electron microscope observations of field concrete specimens are used to supplement thin section petrography and field observations of evidence of deterioration. In research studies with modern, high-power electron microscopes it is tempting to over-emphasize observations of visually spectacular cryptocrystalline features at the expense of the more dull and featureless main substance of the cement paste,¹⁸ and to forget the difficulties of ensuring representativity at the used magnifications.

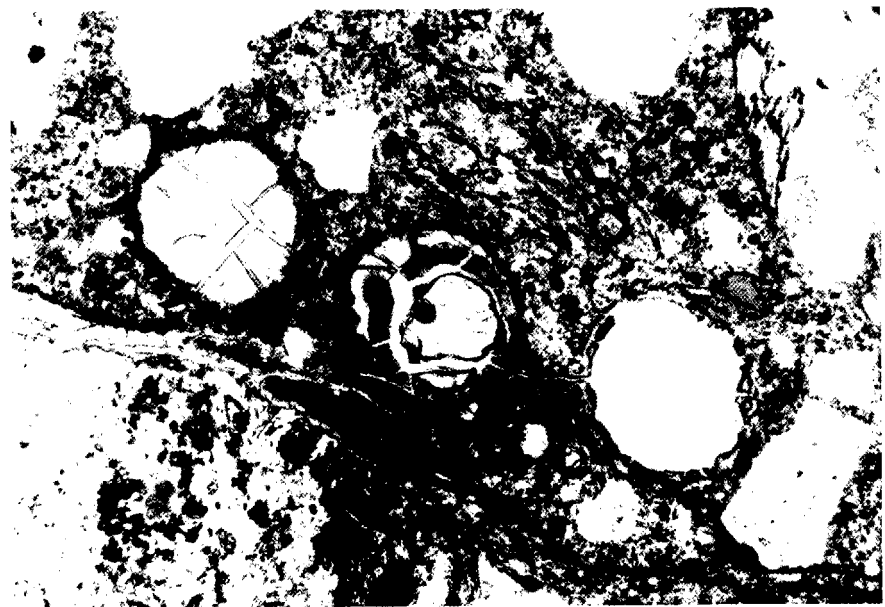


Fig. 3 — Coarse aggregate particle in thin section of prestressed concrete railroad tie. Microcrystalline quartz has made the aggregate particle alkali reactive and caused its fracturing and gel exudation out into the radiating crack and air bubbles in the adjacent cement paste. (The image is taken by polarized light and represents 2.7 x 3.9 mm).

Even at the commonly used scanning electron microscope (SEM) magnifications it is difficult to distinguish between pores, cavities, and cracks without comparing the images to thin section observations at the orders of magnifications used with the optical microscope. The education of research workers in the art of interpreting submicroscopic features seen in modern laboratory instruments and their relation to concrete properties needs thorough attention.

Simplistic lab modelling

The heat development in field concrete during curing creates temperature variations that are dependant upon the cement type and contents, the volume and geometry of the concrete in place, and the environmental temperature.¹⁹ Nevertheless, most scientific research on the hydration of cement is still based upon laboratory studies made at room temperature, which means that the heat as the driving force for the rate of hardening is deliberately prevented from affecting the chemical and physical reactions in laboratory specimens. Besides, laboratory specimens are made orders of magnitude more homogeneous than field concrete in order to achieve reproducibility within and between test series.

The ASTM specification C 618-91 and the test method ASTM C 311-90, which cover the use of fly ash in concrete with portland cement, are examples of this. The characterization of fly ash and its effects in concrete could be much improved by updating the test method with measurements of the adiabatic heat of hydration of the actual blends with portland cement in concrete. This would give data for maturity monitoring of the curing of the given concrete. As it is, the test procedure shows less early strength development than occurs in the field concrete. In contrast, the ASTM C 289 test method would, with minor adjustments, be suitable as a test of the early hydration reactivity of fly ash, partly because the +80 C reaction temperature quite realistically simulates practice, and partly because the early hydration of fly ash in concrete is actually primarily a modification of the alkali-silica reaction.

Field concrete investigation

According to ASTM C 33, the decisive observations to record when investigating deleterious chemical and physical reactions in field concrete is the visible evidence of damage in concrete surfaces: cracking, crumbling, scaling, and characteristics of environmental exposure.

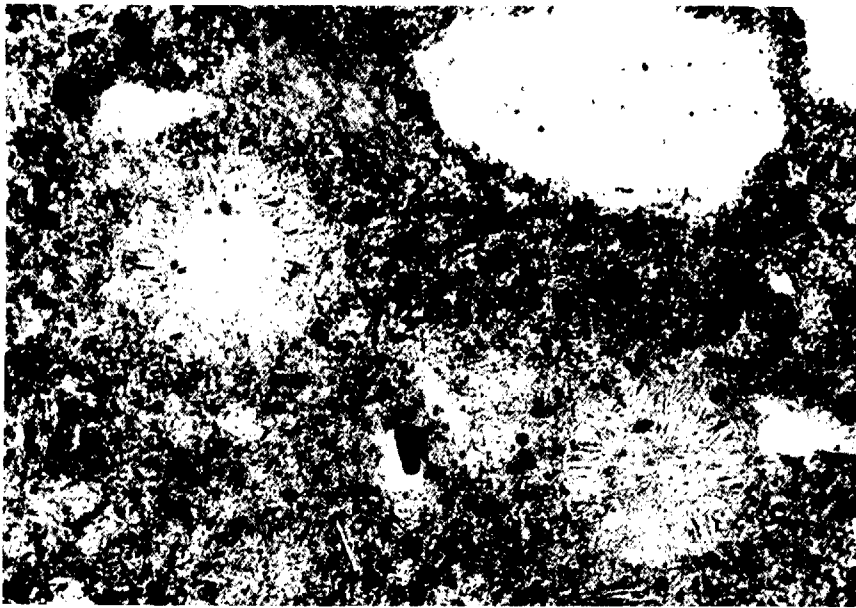


Fig. 4 — Secondary ettringite filling original air bubbles in the cement paste in thin section of a prestressed concrete tie. There is no evidence of radiating cracks created by the ettringite formation. (The image is taken by polarized light and represents 1.1 x 1.6 mm).

These observations may need to be supplemented by selected "third dimension" dissection of the concrete by means of drilled cores for macro examination and thin section petrography. Thin section observations may be further detailed by means of scanning electron microscopy. For complicated cases, chemical and x-ray analyses, non-destructive testing, and even structural analysis may also apply. The laboratory examinations must be interpreted stepwise back from the laboratory observations and data to correlate with the field evidence.

Fig. 2 shows that when alkali-silica reaction is the primary cause of deterioration, surface map or pattern cracking is the typical field evidence. Alkali-silica gel is the traceable consequence of the chemical reaction, and internal cracking of aggregate particles the incremental basis of surface cracking and, in rare cases, of gross structural expansions.^{20,21}

Fig. 3 shows typical evidence of alkali-silica reaction in a thin section of concrete. The aggregate particle shows internal cracking and gel precipitation in the radiating crack in the adjacent cement paste.

When freezing and thawing is the primary cause of deterioration, surface crumbling and scaling is the typical field evidence, except if the

aggregate is non-resistant to frost, in which case the field evidence may be popouts. The surface crumbling and scaling are due to the successive dilation and contraction deformations of the cement paste that result from the formation and thawing of ice in the pores of the cement paste. This mechanism causes loss of bond between aggregate particles and cement paste, but the cement paste cannot cause internal cracking in the much stronger aggregate.

Sulfate attack causes crumbling and scaling in field concrete and is dreaded in regions with sulfate rich soils,^{22,23} under certain conditions with reactive pyrites in the aggregate,²⁴ and in certain other acidic environments.²⁵ Sulfate attack may cause dilation of cement paste in concrete, like freezing-thawing, and additional chemical destruction of the binding capability of the cement paste.

Where sulfate attack occurs in regions with severe winters, the combined physical and chemical reaction may be severe. Sulfate attack may also be violent in hot countries due to the aggravating effects of high temperatures on the chemical reactions.

For all three mentioned causes of deterioration, initial microcracking of the cement paste in concrete, such as those provoked by intensive

steam curing or inadequately handled hot weather conditions, will reduce the concrete's resistance to subsequent deleterious reactions during the structural performance, unless the structure is well protected against the impact of weathering with cyclic wetting/drying and heating/cooling.

With the same three kinds of deterioration, ettringite (calcium aluminate sulfate hydrate) is usually found as a secondary reaction product in cracks and cavities in the concrete due to changes of the equilibria conditions of the pore liquid in the cement paste.

Fig. 4 shows a typical formation of secondary ettringite in a thin section of concrete. The ettringite has crystallized from the pore solution in the available empty space.

Generally, the amount of ettringite increases with the intensity of the deterioration. Under some circumstances ettringite has been found to be replaced or supplemented by gypsum.²⁶ The expansive effects of ettringite formation that have been observed in laboratory specimens have not been found as gross expansions in field concrete exposed to chemical sulfidic reactions. This is because the affected cement paste loses strength and expansive capacity during the chemical conversions.

As discussed above, tests such as ASTM C 227 and C 289 are helpful as model simulations of the reactions concerned, but are unable to provide quantitative information about the effects of the simulated reaction in field concrete. Logically, more complicated chemical reactions in cement paste in hardened concrete can only be simulated with difficulty by the simplistic reaction systems that are used in laboratory specimens, except for measurements of the mineralogical characterization of reaction products, such as ettringite, gypsum, magnesium hydroxide, calcite, aragonite etc.

Conclusions

The recognition in ASTM C 33 in the early 1950's that constant temperature conditioned laboratory testing of a chemical reaction in concrete does not represent the effects of the reaction in field concrete ought to be reacknowledged after many years in apparent oblivion. The later decision to

development of powerful laboratory instrumentation is a major reason why the institutional concrete research profession has insulated itself from studies of field concrete and engineering practice behind a high fence with the room temperature syndrome as the no-trespassing signpost.

The SHRP 1987-93 program was begun because this research strategy had slackened or prevented realistic applications of new scientific knowledge in concrete production technology and thereby created the alarming degradation of the highway systems. The SHRP research has achieved fragmentary progress towards new, realistic site concrete process monitoring. It has also revealed that the prevailing concrete research strategy must be profoundly changed. The nature of concrete during production and field performance must be the primary target for scientific and applied studies, and laboratory findings must be used with due regard to rational model laws. With such a new strategy, the research community will be able to attain promising returns on research investments, to achieve termination of the concrete durability crisis, and to win the confidence of concrete producers with realistic assistance to innovations for industrial concrete production technology.

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Gunnar Idorn

The concrete blocks forming a jetty in Hvide Sande, West Jutland were cast and placed in 1915. They were made without air entrainment from ordinary portland cement and local, unwashed beach sand and gravel. For 77 years they have resisted sea water, freezing and thawing, and wear and tear. Some of the blocks have map cracking due to alkali silica reaction, but they can last for at least 100 years. Most of the blocks have no damage, but they do have harmless alkali silica reactions. Their service life may easily be 200 years. The blocks illustrate the complexity of concrete durability as compared to the economy in concrete construction and uses. Seen in hindsight, the blocks certainly should be considered to represent high performance concrete.

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Point of View

For Durable Concrete, Fly Ash Does Not "Replace" Cement

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by W. E. Ellis, Jr.

High quality fly ash improves the properties of both fresh and hardened concrete. Many sources of fly ash now produce material having low loss on ignition and high fineness with predictable, consistent properties.¹ Stringent quality control/quality assurance programs by reputable ash marketers insure reliable supplies of consistent fly ash. However, in light of the available information on the positive benefits to concrete durability of the proper use of fly ash, specifications governing the use of fly ash in concrete often do not allow for the full exploitation of fly ash in improving concrete properties.

Historically, there has been a misconception that limiting the amount of fly ash in a mix will improve the quality of the concrete produced. Specifications often state "Fly ash may be used to replace up to 15 percent (or 10 percent or 20 percent) of the cement," or something similar. This is exactly the wrong way to specify the use of this valuable material.

Fly ash does not "replace" cement

Typical Class F fly ash, when substituted for cement on an equivalent mass basis, will produce concrete having lower compressive strengths at ages up to and sometimes beyond 28 days. A higher total cementitious material content (cement + fly ash) is almost always required to obtain equivalent 3, 7, or 28-day compressive or flexural strengths.² However, many concretes with Class C fly ash contents in the range of 20 to 40 percent of the total cementitious material^{3,4} will show equivalent or higher strengths than concretes containing portland cement only. Much research has placed fly ash (particularly Class F fly ash) at a significant disadvantage in durability testing by replacing cement by mass (or worse, by absolute volume), creating concretes of lower strength. In the real world, concrete is normally required to attain a specified minimum strength at a specified age (typically 28 days or earlier, regardless of the cementitious materials used. Comparisons should be made on this basis.

Concrete durability

Concrete in service is exposed to a range of external and internal stresses, environmental and material-related, which can lead to a reduction in service life or premature failure of the concrete section.

Some of these environmental stresses include:

- Aggressive chemicals in groundwater or soils
- Marine exposure conditions
- Cycles of freezing and thawing
- Exposure to de-icing chemicals

Material-related stresses can be a result of:

- Alkali-silica or alkali-carbonate reactions
- Excessive shrinkage and related micro-cracking
- Excessive chloride levels from concrete materials
- Inadequate air entrainment
- Thermal stresses from excessive heat of hydration
- High permeability

All or any of these conditions can cause or lead to premature deterioration of concrete, either directly (chemical attack), or indirectly (by allowing infiltration of moisture and salts).

It is becoming increasingly obvious that the traditional method of improving concrete durability — raising the portland cement content — is not sufficient to protect concrete in service from many of these destructive forces. Hoff⁶ has pointed out that "performance" concretes of a given strength contain less cement (higher w/c) today than in the past. These concretes are typically more finely ground and contain higher C₃S contents to achieve higher early strengths. Because of these characteristics more heat is generated more rapidly during cement hydration, and more water-soluble calcium hydroxide, is created.⁷ Increased cement content does not necessarily reduce the permeability of concretes containing these cements.⁸ (Fig. 1). Increased cement content also contributes to increased drying shrinkage and thermal stress, and provides additional alkalis to support potential alkali-aggregate reactions. Proper use of mineral admixtures such as fly ash will improve concrete properties related to durability. Augmentation of portland cement with fly ash reduces the permeability of concrete dramatically (Fig. 2).

How fly ash interaction improves durability

Permeability reduction

Permeability has long been ignored as a measure of the relative performance of concretes, largely because adequate test procedures to measure it have not been readily available. Only recently has ASTM C-1202 been adopted, to measure chloride permeability of concrete.

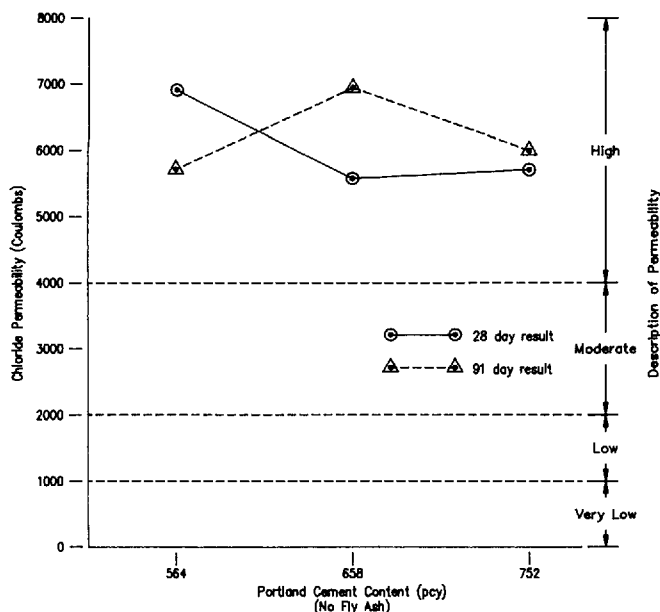


Fig. 1 — Rapid chloride permeability (AASHTO T-277) versus portland cement content (no fly ash).⁸

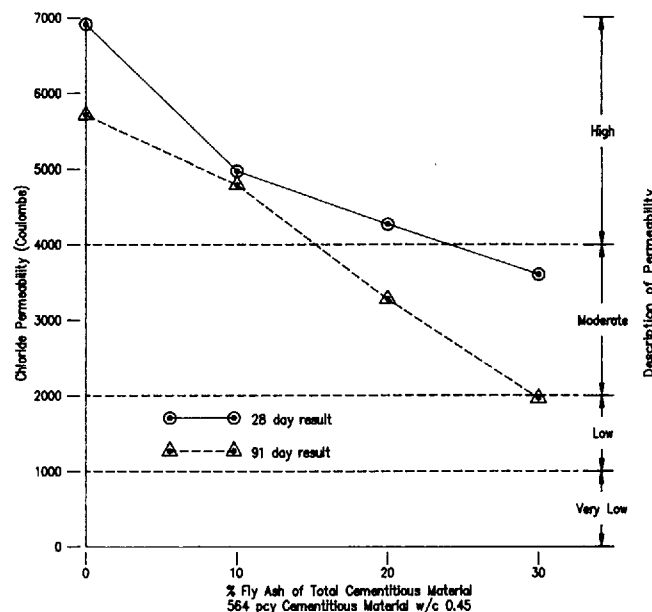


Fig. 2 — Rapid chloride permeability (AASHTO T-277) of concrete containing fly ash as a percentage of cementitious materials.⁸

cretes. In general, research has shown that this test (as the AASHTO T-277 test method) may also correlate well with water permeability.⁸

To achieve high durability, concrete requires a high degree of impermeability. Highly impermeable concrete will resist the intrusion of external chemical attack (sulfates and chlorides) as well as water penetration, which is critical to freeze-thaw durability. Moisture is also required to activate alkali-aggregate reactions, and to initiate corrosion of reinforcing steel. A decrease in concrete permeability will improve potential durability, all other factors (air entrainment, strength) being equal.

Li and Roy have detailed the mechanism by which the use of fly ash creates a discontinuous pore structure and clogs pore channels with additional hydration products⁵ (Fig. 3). The result of these reactions is dramatically slower transport rates for both water and chlorides through the cementitious paste. The consumption of water-soluble calcium hydroxide (produced as a by-product in the hydration of portland cement) by the pozzolanic reaction creates additional calcium silicate gel, which is much denser than the gel without pozzolan.⁹ In general, the consumption of $\text{Ca}(\text{OH})_2$ and the resulting decrease in permeability increases with fly ash content as a percentage of total cementitious material.^{5,9}

The permeability of concrete is also reduced significantly by increasing fly ash content, keeping a constant cement content^{8,10} (Fig. 4,5). Reducing concrete permeability with fly ash, including its use in addition to rather than merely replacing cement, will reduce the potential for corrosion of reinforcing steel, both from the effects of moisture and chloride penetration.^{11,12} The modification of pore structure and the permeability-reducing effect appears to be more pronounced with ASTM C-618 Class F fly ashes than with Class C.

Sulfate resistance

Large areas of the United States have soils or ground-water that contain sulfates of sodium, potassium, calcium, or magnesium. The mechanism by which sulfates attack concrete has been well documented:¹³ sulfates chemically combine with calcium hydroxide to form gypsum compounds, and sulfate ions combine with hydrated calcium aluminate to form ettringite.

The formation of gypsum compounds and calcium sulfoaluminates results in an increase in solid volume in the cement matrix, and subsequent disruptive expansion. Much research^{14,15} has shown that use of an effective pozzolan will reduce the potential for expansion due to the action of sulfates. The mechanism is three-fold: the pozzolanic reaction reduces the calcium hydroxide content of the concrete, the use of a Class F fly ash reduces the absolute amount of calcium aluminate available to support ettringite formation by reducing the cement content, and the permeability of concrete is reduced, thus diminishing the sulfate penetration.

Von Fay and Pierce,¹⁶ Tikalsky,¹⁷ and many others have shown that Class F fly ash is more effective in reducing sulfate attack, and that Class C fly ash usually decreases the sulfate resistance of concrete, especially if compared with Type II cement, unless used at levels exceeding forty to fifty percent of the cementitious content (Fig. 6). Class C fly ash typically contains relatively high levels of amorphous calcium aluminate, which contributes to the decreased performance of concrete containing Class C ash in a sulfate environment. In general, fly ashes with the highest CaO contents perform worst in sulfate conditions.^{18,19} However, in areas where mild sulfate exposure exists, (as defined in ACI Committee 201's "Guide to Durable Concrete," Table 2.2.3) concrete containing Class C fly ash can have satisfactory sulfate resistance.

Class F fly ash used at a relatively high percentage of the total cementitious content will provide a high de-

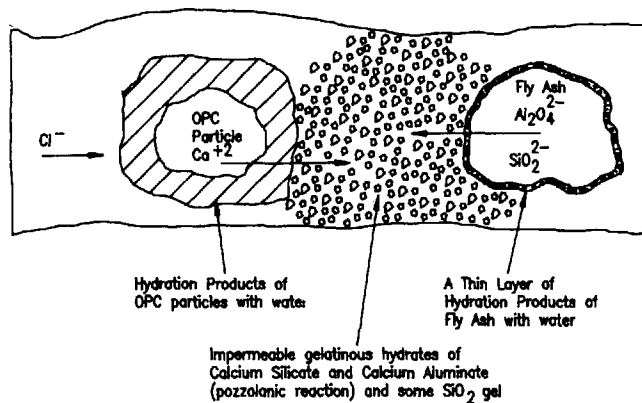


Fig. 3 — Diagram of an impermeable gel plug forming in a pore.⁵

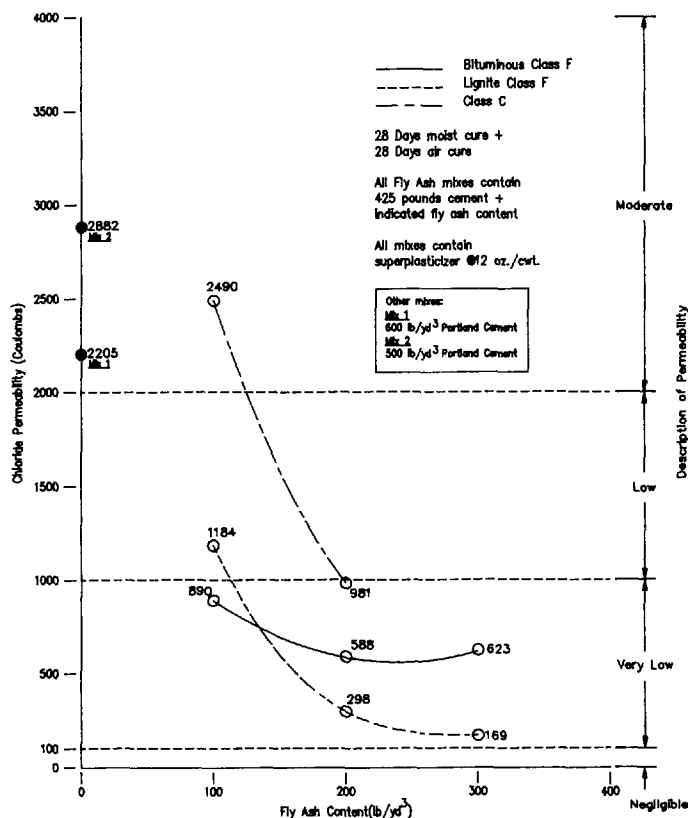


Fig. 5 — Chloride permeability (AASHTO T-277) of concrete with increasing fly ash content.¹⁰

gree of protection, when combined with the appropriate type of cement. All evidence suggests that Class F fly ash added beyond that required for compressive strength will improve durability.²⁰ The effect is even more pronounced in lean concretes, where additional fly ash produces dramatic permeability reductions.

Alkali-silica reaction

Aggregates that may react with portland cement alkalis over time have been identified in virtually all the United States. While certainly not all aggregates possess the potential for catastrophic expansion, sufficient concern about long-term serviceability exists to justify the now wide-spread specification of low alkali cement

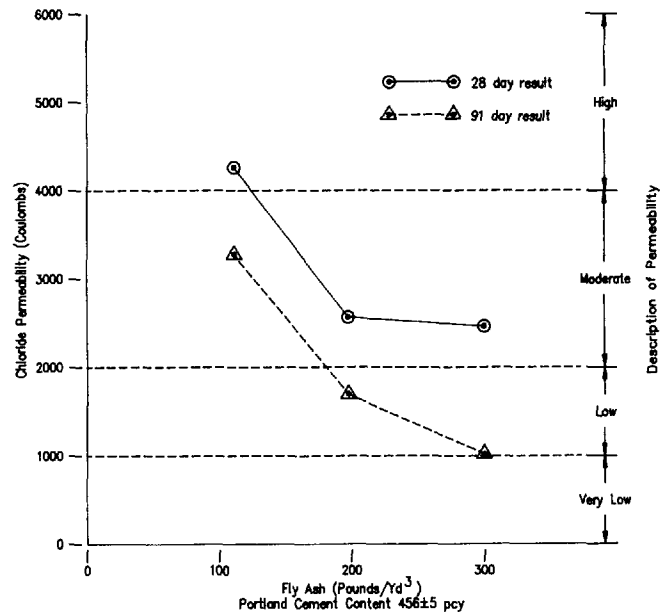


Fig. 4 — Effect on chloride permeability (AASHTO T-277) of increasing fly ash content with constant cement content.⁸

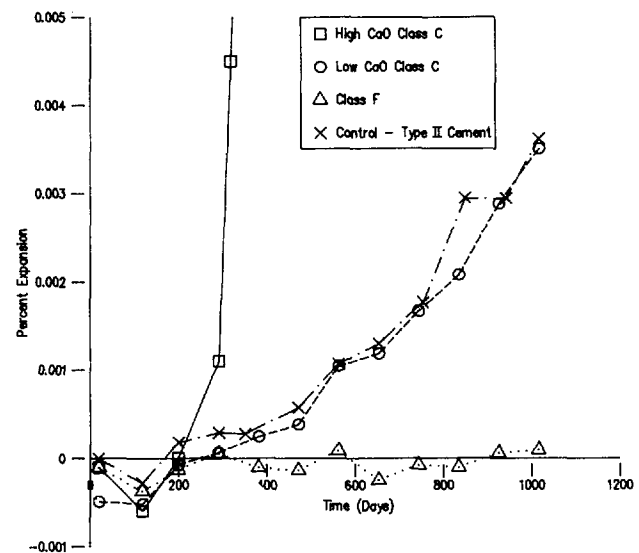


Fig. 6 — Expansion versus time for concrete with 645 lb/yd³ cementitious material exposed to 2.1 percent sodium sulphate solution.¹⁷

or the inclusion of an appropriate amount of fly ash, ground granulated blast furnace slag (GGBFS), or silica fume to ameliorate the potential for distress. In at least one state DOT specification, the cement alkali content has been specified as a maximum of 0.4 percent, with an alternative allowable minimum of 20 percent Class F fly ash (replacing 15 percent cement) or minimum amounts of GGBFS or silica fume. This minimum amount recognizes the existence of "pessimum" levels which must be exceeded for the fly ash, GGBFS, or silica fume to be effective.

Bhatty²² has published an excellent overview of the alkali-silica reaction mechanism, in which he details the reduction in expansion as a result of increasing the si-

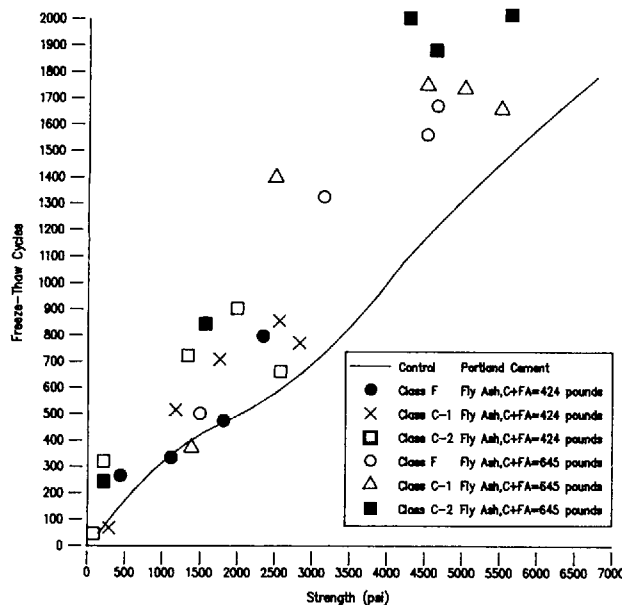


Fig. 7 — Durability versus compressive strength for three fly ashes (28-day fog cure).²⁷

ica content of the cementitious phase. Fly ashes meeting the optional maximum available alkali limit in ASTM C-618, and which are shown to be effective by test, compared with low alkali cement, typically contain high levels of silica, compared with cement. This silica, present as highly reactive silicates, is responsible for the pozzolanic properties of fly ash. Alkalies (sodium and potassium hydroxides) form during hydration of portland cement, and co-exist with calcium hydroxide in the gel/pore water structure. The fly ash pozzolanic reaction also combines some of the alkali metals into the calcium silicate matrix, rendering them unavailable to support alkali-silica reaction. As detailed above in the section on permeability, the pozzolanic reaction is more complete when sufficient fly ash is available. Bhatti also showed that the retention of alkalies was improved in the cementitious system containing a higher percentage of silica (more fly ash).

Studies have shown that alkali-silica reaction is substantially reduced in concretes containing very high percentages of fly ash.²³ Consistent with construction scheduling and strength requirements, use of sufficient fly ash, including in addition to as well as a replacement for cement, will reduce the potential for alkali-silica reaction.²⁴ Where alkali-silica reaction is a concern, cement contents (the major source of alkalies) should be kept as low as possible, consistent with desired results. In general, a higher fly ash content means a lower potential for alkali-silica reaction.²⁵

Insufficient data exists to support any finding that alkali-carbonate reaction is diminished by the use of fly ash, GGBFS, or silica fume.

Resistance to freezing and thawing

The freeze-thaw mechanism requires that concrete be saturated in service to be adversely affected by freezing conditions. ACI 201 sets forth specific guidelines for durable concrete under such circumstances, including air entrainment, adequate compressive strength (4000 psi prior to freezing is desirable), and a period of air

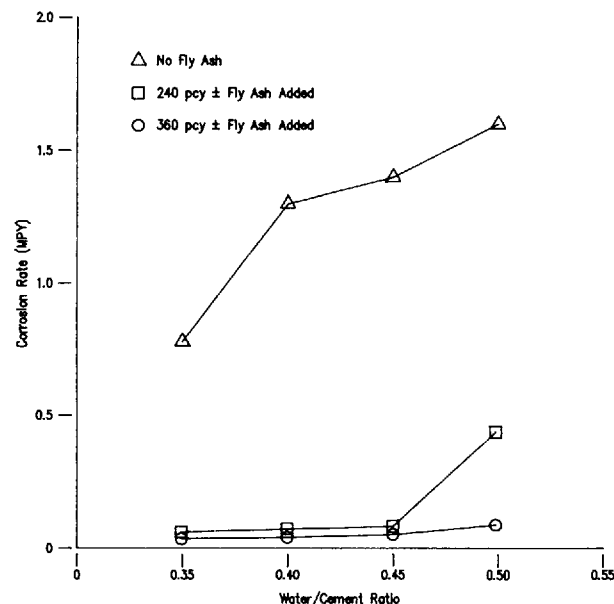


Fig. 8 — Corrosion rates of reinforcing bars in plain and fly ash concrete samples containing a constant 590 lb/yd³ of portland cement exposed to 5 percent NaCl solution.¹¹

drying prior to being subjected to freezing conditions. Since saturation is a prerequisite to freeze-thaw damage, reducing the permeability of concrete will be an asset. A Bureau of Reclamation study has shown that the use of either Class F or Class C fly ash produces concrete with resistance to freeze-thaw cycles generally superior to portland cement concrete of equivalent compressive strength²⁶ (Fig. 7).

As long as the recommended procedures for concrete exposed to freeze-thaw conditions are followed, good quality fly ash (which does not alter the amount of entrained air), will improve the performance of concrete in this environment.

Corrosion and marine concrete

Research has detailed the effect of the addition of fly ash (as sand replacement) with a constant cement content in reducing corrosion (and half-cell potential) of reinforcing steel in concrete exposed to chlorides¹¹ (Fig. 8). Research in Florida on the corrosion-inhibiting properties of Class F fly ash concretes²⁷ and the reduction in permeability⁸ through its use has led to the development of Section 346 in the Florida DOT concrete specification,²⁸ in which a minimum of 18 percent by weight (up to 50 percent maximum in mass concrete) of Class F fly ash is required in concrete placed in moderately or extremely aggressive environments. This is in recognition of the beneficial properties of fly ash concrete in areas of high chloride/sulfate concentrations. The study cited also showed that the corrosion resistance improved as the fly ash content increased as a percentage of total cementitious content, although the 28 day strengths decreased as cement was replaced.

The best available technology for durable marine concretes would dictate the use of a moderate C₃A content cement with the addition of relatively large quantities of Class F fly ash (greater than 200 lb/yd³ [119 kg/m³]) to produce a dense, impermeable concrete. The use of high-range water-reducing admixtures should be considered to reduce w/c and maximize density.

An example of this mix design approach was used in the replacement of the Sunshine Skyway in Florida. The concrete mixtures (over 90 percent were Florida DOT Class 4) contained 658 lb/yd³ (390 kg/m³) of total cementitious material. A Type II cement and 145 lb/yd³ (86 kg/m³) of Class F fly ash were used in all superstructure concrete. Up to 50 percent by weight of the cementitious content was Class F fly ash in all of the mass concrete. These design mixes were chosen to "lower heat of hydration, decrease concrete permeability, and improve sulfate resistance."²⁹

Exceptions

There are a few exceptions to the use of significant quantities of fly ash in concrete mixtures:

- Class C fly ashes appear to perform best in normal commercial concretes when limited to 100 to 120 lb/yd³ (59 to 71 kg/m³) (or perhaps 20 to 25 percent of the total cementitious material). Larger quantities may retard the setting time of concrete excessively. Most Class F fly ashes, on the other hand, do not in themselves retard concrete — larger quantities may be added without adversely affecting setting times.
- Floors that will be steel trowelled will benefit from relatively small amounts of fly ash (less than 100 lb/yd³ [59 kg/m³]) and minimal air entrainment (ACI 201, Section 3.4 recommends less than 3 percent for floors not exposed to freeze-thaw conditions). This limit will encourage bleeding and should reduce the possibility of blistering caused by premature finishing operations. Concrete mixes that bleed excessively will of course benefit from higher amounts of fly ash.

Summary

The science of making durable concrete is no longer as simple as adding another sack of cement to the mix. Awareness of the beneficial utilization of mineral admixtures — fly ash, GGBFS, or silica fume — in producing more durable concrete is increasing. However, these materials are not portland cement, and different rules apply to their use.

Compressive strength remains the single criterion by which most concrete mixtures are designed, although a maximum *w/c* may be specified where concern exists over exposure conditions or the competence of a concrete producer. The increasing trend is to demand high early strengths either for early form removal or as a safety factor. The development of cements designed to meet the needs of cement consumers for early strength development and which must also be cost-effective to produce is a contributing factor to a deterioration in concrete quality and durability.

The technical benefits of fly ash use and the mechanism of the fly ash-cement reaction are well known. However, fly ash is not being utilized effectively when only small amounts are allowed as a replacement for some of the cement. Even high early strength concretes will benefit from the inclusion of a proper amount of fly ash, although it may contribute little or nothing to very early strengths, due to the dramatic improvement in durability resulting from its use.

While the statement "Water-Cement Ratio is Passé"³⁰ is too strong, Robert Barton made some excellent points about the use of admixtures in concrete and the danger of too much cement. Both "Cement —

When to Say When"³¹ and "Principles Underlying Production of High-Performance Concrete"³³ should be scrutinized carefully before designing durable concrete mixes. Adequate amounts of pozzolans impart properties to concrete that cannot be achieved by increasing portland cement content alone.

Proper fly ash use in concrete decreases its permeability (one of the most important but least understood variables in durable concrete mix design), modifies the cement hydration products to render them much less harmful in reactions with sulfates or reactive aggregates, and decreases the water content required for concrete production. It reduces shrinkage,³² reduces the incidence of cracking, and improves the gel structure of concrete, particularly at the cement-aggregate interface where Ca(OH)₂ otherwise tends to accumulate.³⁴

Any concrete that is damp or wet in service (including areas of high humidity), is exposed to sulfates or chlorides, or may contain reactive aggregates, will benefit from inclusion of significant amounts of fly ash.

Since reactive combinations of aggregates and cement and exposure conditions cannot always be predicted accurately prior to concrete placement, the use of fly ash should be adopted prophylactically to eliminate or minimize the potential for future distress.

Adequate test procedures do not currently exist to identify all types of reactive aggregates. Unless soil borings and groundwater are analyzed for sulfate and chloride content, not enough information exists to adequately design a durable concrete mix (see Tuthill³⁵ for a case history). The goal of the concrete technologist today must be to produce concrete that will consistently perform and be durable in all types of exposure conditions for longer design and expected service lives. The best current technology for designing durable concrete *requires* the use of pozzolans (fly ash or silica fume) or GGBFS.

Surely there can be no argument that a concrete containing 400 lb/yd³ (237 kg/m³) of cement and 250 lb/yd³ (148 kg/m³) of fly ash will be more durable by any definition than one containing 400 lb/yd³ (237 kg/m³) of cement and 80 lb/yd³ (47 kg/m³) of fly ash, despite the fact that fly ash constitutes 38 percent by weight of the total cementitious material in the former and 17 percent in the latter? Subject to the limitations described herein, there are very few concretes that will not benefit from the inclusion of at least 150 lb/yd³ (89 kg/m³) of Class F or 120 lb/yd³ (71 kg/m³) of Class C fly ash, purely for durability enhancement. This may be as part of a normal economical concrete mix, or in the form of an additive where higher early strength requirements would normally limit the use of some fly ashes. "The incorporation of mineral admixtures in concrete can lead to so many technical advantages that no concrete should be made without them unless there are special reasons."³⁴

Selected for reader interest by the editors.

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The list of references for this article were deleted because of space limitations. They are available on request from *Concrete International* production editor Paula Schmalzriedt.



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