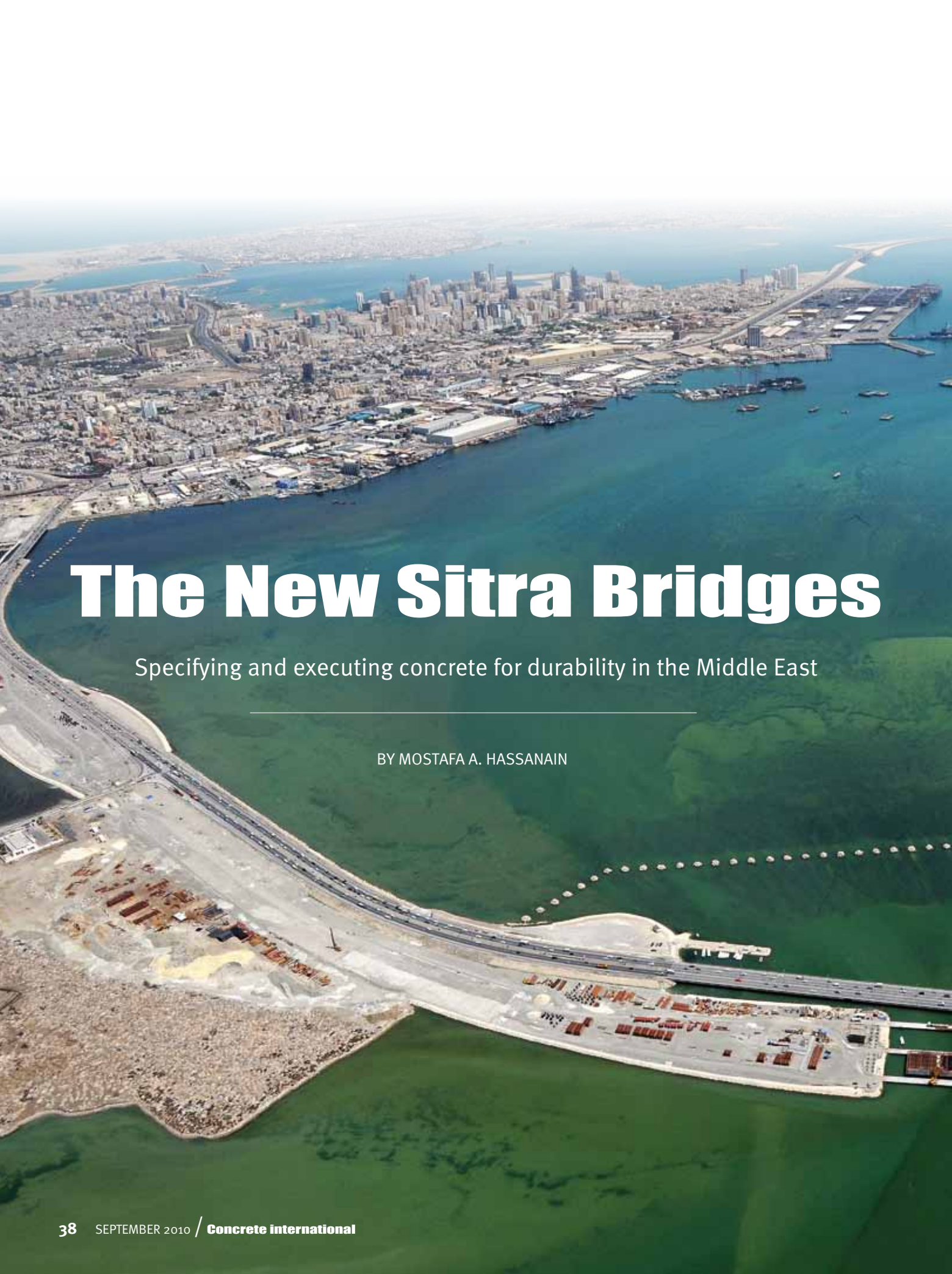

The New Sitra Bridges

Specifying and executing concrete for durability in the Middle East

BY MOSTAFA A. HASSANAIN

Fig. 1: Sitra Causeway (looking northeast)

The Sitra Bridges are part of a 3.2 km (2 mi) causeway linking the main island of Bahrain to the island of Sitra (Fig. 1). The causeway, which opened to traffic in 1976, is one of the most strategic road links in the Kingdom of Bahrain. Located in the severe marine environment of the Arabian Gulf, the causeway is exposed to warm, highly saline water; airborne salt spray and salt-laden dust; high temperatures and temperature gradients; and high humidity. Air temperatures in summer can reach 50°C (122°F), and the relative humidity can exceed 95%. These aggressive conditions make structural concrete highly vulnerable to deterioration.

Only 15 years after opening, the original concrete bridges started to show signs of deterioration[1]; and 30 years after opening, they had deteriorated beyond economically feasible repair. The causeway was also unable to accommodate existing traffic. The bridges are being replaced as part of a $280 million project to construct a new causeway—the largest single road project ever undertaken in Bahrain.

This article presents some aspects of the structural concrete needed to satisfy stringent requirements for durability. Other design, construction, and management challenges are discussed elsewhere.[2]

PROJECT OVERVIEW

The new causeway is 50 m (164 ft) west of the existing one and will double its capacity (Fig. 1). The new 200 m (656 ft) northern bridge has four spans (Fig. 2), and the new 400 m (1312 ft) southern bridge has seven spans (Fig. 3). The superstructures consist of cast-in-place concrete box girders with bonded post-tensioning. The box girders are up to 3 m (10 ft) deep. The substructures consist of cast-in-place, reinforced concrete piers on reinforced concrete pile cap crosshead beams over bored, steel-

Fig. 2: Completed western carriageway of the northern marine bridge. Temporary staging platforms are in place for the eastern carriageway

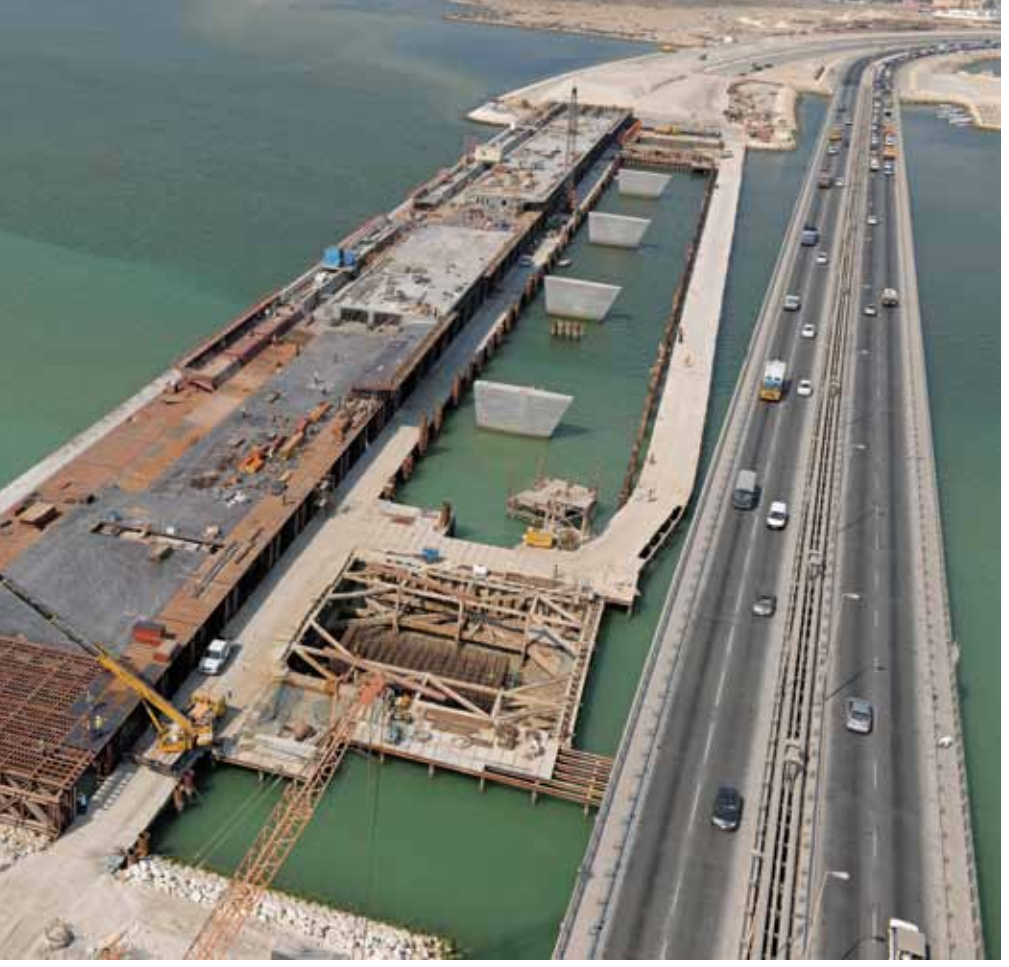

Fig. 3: Construction of the southern marine bridge

Fig. 4: Construction of the three-level, grade-separated interchange (looking south)

encased reinforced concrete piles embedded in bedrock; the abutments comprise cast-in-place reinforced concrete bank seats on similar piled foundations.

The project also includes the transformation of the northern approach to the causeway from an at-grade, signalized junction into a three-level, grade-separated interchange—the first in Bahrain. This involves the construction of a 560 m (1837 ft) underpass; a 26 m (85 ft) at-grade bridge; a 379 m (1244 ft) six-span flyover; and a 183 m (600 ft) five-span ramp (Fig. 4). The structural systems for the superstructures and substructures are generally similar to those of the marine bridges.

The underpass is about 26 m (85 ft) wide and consists of a watertight, open-trough structure with a 0.5 to 1.2 m (1.6 to 3.9 ft) thick reinforced concrete base slab. The structure is anchored to resist buoyancy. The side walls comprise steel sheet piles with inclined ground anchors, and the sheet piles are covered by cast-in-place, reinforced concrete on the traffic side (Fig. 5). The 26 m (85 ft) long, 73 m (240 ft) wide at-grade north-south bridge comprises a two-span continuous, reinforced concrete deck slab supported by the outer and central underpass walls (Fig. 5). The central wall is supported by bored, steel-encased reinforced concrete piles.

DURABILITY REQUIREMENTS

Due to the severe environment, the client set the design life of the structures at 120 years, in accordance with the then-current British Standard BS 5400-4.[3] (In March 2006, Bahrain adopted AASHTO LRFD.[4]) Additionally, the client required that the bridges would require only planned (nonstructural) maintenance during the first 40 years.

DURABILITY DESIGN

The main mechanism of concrete deterioration was considered to be

reinforcement corrosion due to chlorides or carbonation. Four operational bases were used for the service-life design:[5]

- The design service life is 120 years;
- Initiation of corrosion represents the nominal end of service life;
- There is a 90% probability of no corrosion initiation by age 120 years, corresponding to a reliability index of 1.3; and
- The nominal concrete cover is 80 mm (3.2 in.), with a tolerance of ±10 mm (0.4 in.).

Life-cycle cost studies were carried out to evaluate various design alternatives. The main alternative called for carbon steel reinforcement in high-performance concrete (HPC). However, the designer argued that the experienced workforce needed for HPC construction[6] is not normally available in the Arabian Gulf region, so an additional defense was needed.

REINFORCEMENT

The additional defense was to use stainless steel reinforcement in areas most highly exposed to chlorides and use carbon steel reinforcement in other locations. This has been suggested by Neville[7] and others in association with ensuring good concrete in the cover zone and an adequate thickness of cover. Life-cycle analyses showed that the additional cost of stainless steel would be offset by savings on maintenance and longer service life.

For the piles, all reinforcement is carbon steel; for the pile caps, only a few dowel bars are stainless steel. For the piers, stainless steel is used in the outermost layers, while carbon steel is used in the inner layers. For the superstructures, stainless steel is used in the outermost layers of exposed outer and inner surfaces, while carbon steel is used in diaphragms and as bursting reinforcement (Fig. 6). For the base slab in the underpass, only carbon steel is used; and for the cladding walls covering the sheet piling, stainless steel is used in the outermost layers, while carbon steel is used in the inner layers. While stainless steel has been used for bridge construction in North America, Europe, and Asia, it is believed that this is one of the most extensive applications of stainless steel reinforcement on record.

It should be noted that research[8] has shown that galvanic action between carbon steel and stainless steel embedded in concrete does not increase corrosion rates significantly.

Fig. 5: Underpass trough just before casting a segment of the base slab. A completed portion of the at-grade, north-south bridge is shown at the top of the photo

Fig. 6: Stainless steel bars were used in the outermost layers of exposed outer and inner surfaces of superstructure elements. Post-tensioning strands were protected using corrugated polyethylene ducts filled with grout

Even when the different bars are electrically connected by direct contact or through supports and tie wires, the corrosion rate has been found to be less than that of the combination of corroding carbon steel and noncorroding carbon steel.

Post-tensioning strands were protected using corrugated high-density polyethylene grout tubes (Fig. 6) and following grout guidance from Reference 9.

CONCRETE SPECIFICATIONS

Six different classes of concrete were specified according to the exposure conditions and possible deterioration mechanisms. The specified cement type was CEM I[10] (with minor changes and additions). The specified fly ash (pulverized fuel ash) content was 30% of the total cementitious material content, and the maximum water-cementitious material ratio (w/cm) is 0.40. The grading, type, and source of the fine and course aggregates were tightly controlled. Table 1 provides more details on the concrete mixture proportions.

The concrete strength was set at C40/50 per Reference 11, meaning the 28-day characteristic strengths for 150 by 300 mm (6 by 12 in.) cylinders or 150 mm (6 in.) cubes were 40 and 50 MPa (5800 and 7250 psi), respectively.

The nominal concrete cover to reinforcement steel was specified as 80 mm (3.2 in.). Exceptions are in the piles, where the cover is specified as 100 mm (4 in.) and at stainless steel bars, for which the cover is specified as 45 mm (1.8 in.). The tolerance on cover was set at ±10 mm (0.4 in.).

PERMEABLE LINER

It has been shown that concrete placed in formwork with a controlled permeability liner has a significantly enhanced resistance to chloride ingress.[12,13] The permeable liner allows excess water and entrapped air to pass as it retains the solids in the concrete. This gives a dense microstructure in the cover concrete.

A permeable liner, consisting of a drainage layer and a nonwoven polypropylene filter fabric, was specified for use on all formed surfaces (Fig. 7). The fabric was

TABLE 1:
CONCRETE MIXTURE PROPORTIONS

Concrete class designation	Minimum cementitious material content, kg/m³ (lb/yd³)	Coarse aggregate type	Maximum nominal aggregate size, mm (in.)
SUB50-NC40	360 (600)	Noncalcareous	40 (1.5)
SUB50-NC20	380 (640)		20 (0.75)
SUB50-C40	360 (600)	Calcareous or noncalcareous	40 (1.5)
SUB50-C20	380 (640)		20 (0.75)
SUP50-C40	360 (600)	Calcareous	40 (1.5)
SUP50-C20	380 (640)		20 (0.75)

SUB50: Concrete for piles, substructures and in contact with ground, fill material, or soil; compressive strength classification of C40/50.
SUP50: All other concrete structures; compressive strength classification of C40/50.
NC and C: Noncalcareous (gabbro) and calcareous coarse aggregate, respectively.
Final figure in designation indicates the maximum nominal aggregate size in mm (in.).

required to have a flow capacity of up to 3 L/m^2 (7×10^{-2} gal./ft^2) and storage capacity to retain 0.35 L/m^2 (9×10^{-3} gal./ft^2) of water in its structure.

CONCRETE EXECUTION

Placing mass concrete

The base slab of the underpass and the pile caps (Fig. 8) were sufficiently massive for thermal cracking to be an issue, particularly during the hot summer months. The specifications limited the maximum concrete temperature to 65°C (149°F) and the difference between the mean and surface temperatures of any element to 15°C (27°F). To reduce the temperature gradient, embedded small-diameter steel pipes were used to circulate cool water. Temperature and stress analyses were used to determine how much heat needed to be removed (and thus, the number of cooling pipes and their locations) to avoid thermal cracking.

One hour prior to the commencement of concreting, a chiller (Fig. 9) was used to cool water in a storage tank and the piping system to about 15°C (59°F). During placement, the water was circulated in pipes fitted with control valves that allowed small adjustments to the flow. The water returned to the chiller at about 18°C (64°F), was cooled to about 15°C (59°F), and returned to the embedded pipes. Thermocouples were used to monitor the concrete temperatures continuously during the first 14 days, but the cooling period was generally limited to 48 to 60 hours after casting, depending on the temperature and stress analyses. Afterward, the embedded pipes were cut at the concrete surface, drained, and filled with grout.

Concrete curing

Proper curing was rigorously practiced by the contractor. An evaporation retarder was sprayed immediately after final finishing to minimize drying and plastic shrinkage cracking. As soon as the bleed water disappeared, an ASTM-compliant curing compound was applied evenly over the surface. When the concrete hardened, wet curing was started immediately by placing wet burlap on the concrete and covering it with sealed polyethylene sheets. The specifications required the burlap to remain saturated for at least 10 days. All concrete surfaces were kept covered with the polyethylene sheets for at least 4 more days. Depending on ambient conditions, windbreaks were put up for additional protection against evaporation.

Throughout the project, thermal cracking has not been significant. The few cracks that occurred either sealed autogenously or were filled with a low-viscosity epoxy resin.

TESTING FOR CHLORIDE PENETRATION

In a saline environment, the predominant transport mechanism for chlorides is diffusion, a process through

Fig. 7: Controlled permeability liner (in white) attached to the formwork of the marine bridges

Fig. 8: Reinforcement cage for a pile cap of a marine bridge pier

Fig. 9: Water cooling system (showing chiller, generators, water tank, and control valves)

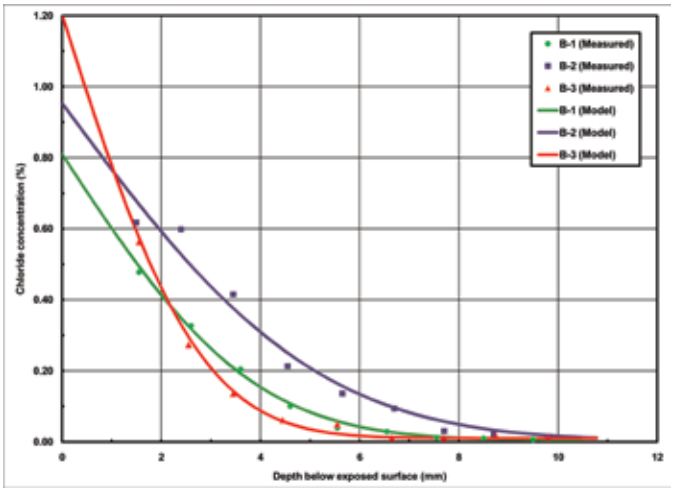

Fig. 10: Results of the Nordic test NT Build 443 for three specimens from the pile cap of the south abutment of the east-south flyover (Note: 1 mm = 0.039 in.)

which chloride ions move under a concentration gradient. The service life of reinforced concrete exposed to chlorides is closely related to the rate of chloride diffusion.

There are various accelerated test methods for determining chloride diffusion coefficients. On this project, the specifications called for Nordic test NT Build 443,[14] in which a limewater-saturated concrete core is exposed on one planar surface to sodium chloride solution for 35 days. The specimen is then evaluated by dry-grinding parallel to its exposed face to obtain at least six powder samples for a chloride profile. The chloride diffusion coefficient is obtained by fitting the measured profile to the solution of Fick's Second Law. Figure 10 illustrates typical results (in this case, for three specimens from the pile cap of the south abutment of the east-south flyover).

Chloride ions can also move through concrete under an electrical potential gradient. The specifications called for the use of the Nordic test NT Build 492[15] on drilled cores from the completed structures to determine the chloride migration coefficient. In this method, an electrical potential is applied across the specimen to force chlorides into it. The specimen is then axially split and sprayed with silver nitrate solution. The depth of chloride penetration can then be measured from the visible white-silver chloride precipitation. From this depth and the applied electrical voltage, the chloride migration coefficient can be calculated. The specification limited the coefficient to 3×10^{-12} m^2/s (3.2×10^{-11} ft^2/s) at 56 maturity days and 4×10^{-12} m^2/s (4.3×10^{-11} ft^2/s) at 28 maturity days.

ADDITIONAL MEASURES

Two coating systems for concrete surfaces were specified, each with an expected minimum service life of 20 years: a waterproofing system with a membrane for concrete in contact with the ground and a water-repellent surface impregnation system for all other concrete. The latter was specified as a monomeric alkyl (isobutyl)-trialkoxysilane with a minimum active content of 92%. Its function is to allow the concrete to "breathe" while keeping the water and chlorides out.

Mock-ups were used for the various structures to establish materials performance and provided reliable data for construction procedures.

The contract emphasized contractor quality control based on proven techniques and ISO standards, including material testing and construction inspection. The client also performed materials tests; these results were used for quality assurance. Effective and continuous construction supervision and inspection were implemented. Corrective measures are taken when needed, and enforcement has included demolition when cover requirements are not met.

It is believed that the materials, design, and construction procedures used on this project should provide adequate protection against concrete deterioration during the specified design life, provided that regular and systematic inspection and preventive maintenance are properly carried out.

Acknowledgment

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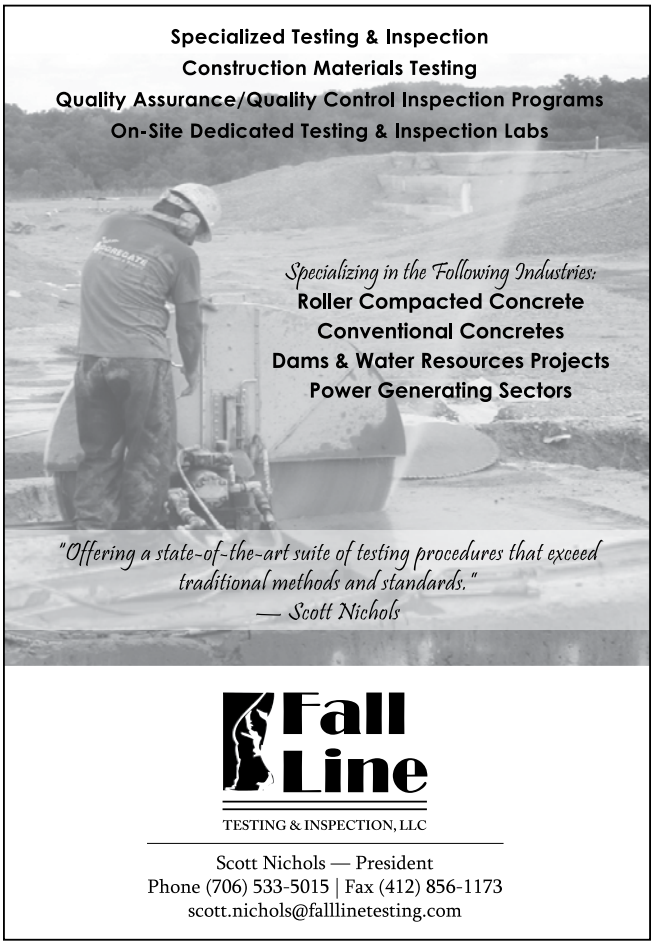
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