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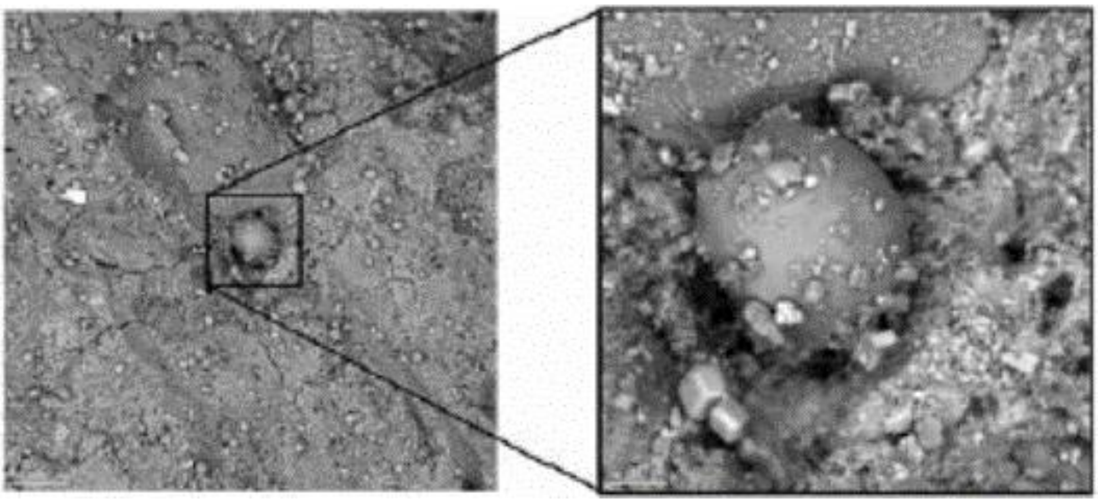

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MyConcrete:
The Bulletin of the American Concrete Institute – Malaysia Chapter

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ONLINE WEBINAR

A Q&A SESSION ON ADMIXTURE WATERPROOFING FOR BASE SLAB DESIGN

FRIDAY
28 OCT 2022
8:30 PM

SPEAKER: MR. JAMES LIM
DIRECTOR OF
CRT SPECIALIST (M) SDN BHD

James Lim completed his degree in Civil Engineering from the University of Auckland in 1996. He started his career as a structural engineer with a consultancy firm and subsequently developed his interests in concrete repair and waterproofing. He then went on to work for companies such as Hilti, MC Bauchemie and Fosroc in product sales and specification work. In 2002, he obtained his Executive MBA from the University of Bath UK. In 2005, he ventured out from the corporate world to start his own specialist contracting company specializing in concrete repair and waterproofing servicing the construction industry in Malaysia.
He specializes in the repair of cracks in concrete elements by method of injection. He has helped many contractors resolve their troubled leaks with specialized application. In addition, he also has vast interests in basement and roof slab waterproofing system especially in the spray polyurea lining system. His recent experiences include KVMRT Line 1 and KVMRT Line 2 underground station waterproofing work.

Preceding Events

FB Live Tech Talk

UHPFRC Technology in Malaysia
- UHPFRC Design & Construction

Ir. Jasson Tan Jhen Shen
Technical Director (Bridge)

Qualification:

- B. Eng Civil, Universiti Tun Hussein Onn Malaysia, 2010
- M. Eng Structural Engineering and Construction, Universiti Putra Malaysia, 2013.

Professional Societies:

- Professional Engineer of Board of Engineer Malaysia (P120462)
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Experience:

- 12 years experience in Conventional and Ultra High-Performance Concrete bridge design and water retaining structure design.
- Involved in single-span and multiple-span bridge design.
- Experienced in conventional beam design to British Standard and France Standards for UHPFRC beam design
- Involved in multiple-span bridge design and construction .

Dura Technology Sdn Bhd

Thursday 24 Nov 2022

8:30pm-9:30pm

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Disclaimer: The opinions expossed in the talk are of the individual, speaker's and not necessarily those of the American Concrete Institute - Malaysia Chapter.

Reprint from CI Magazine, Volume 42, No 7, Page 33-35

Contraction Joints for Residential Post-Tensioned Slabs

by Harvey Haynes and Kenneth B. Bondy

Contraction joints are generally not installed in residential post-tensioned (PT) slabs-on-ground—yet they should be. It is common to find random cracks in PT slabs because cracks can develop before the posttensioning is applied. Thermal contraction and drying shrinkage affect PT slabs in the same manner as non-PT slabs, which typically do receive contraction joints.

There are two likely explanations for the lack of joints in PT slabs. First, many designers are aware that post-tensioning is used in warehouse slabs to eliminate contraction and construction joints and thus reduce forklift maintenance. Second, many designers have the perception that PT slabs should not have cracks, so joints are unnecessary.

In the arena of construction defect litigation, some owners claim that cracks in a residential PT slab comprise a construction defect. If cracks are present, they assume that a design or construction problem occurred. Some engineers also apparently believe that a cracked section of a concrete slab-on-ground cannot function properly as a post-tensioned member. These technical issues have been addressed by Bondy[1] and will be discussed herein.

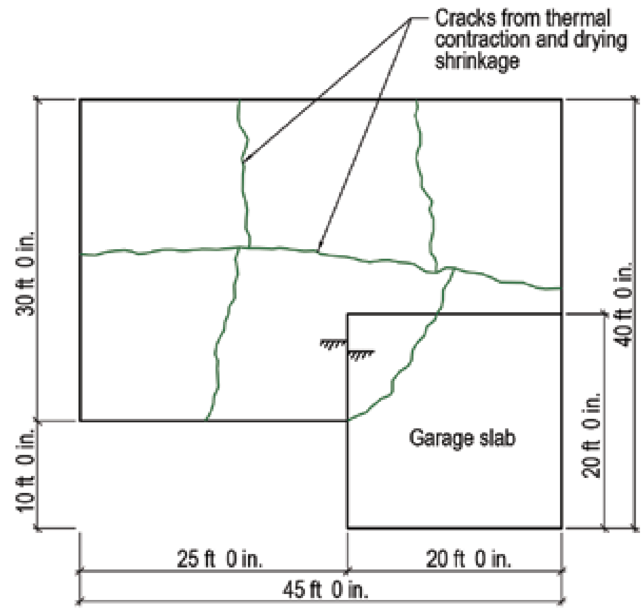

Fig. 1: Representative cracking pattern in PT slab-on-ground having a uniform thickness of 10 in. (250 mm) (Note: 1 ft = 0.3 m)

PTI Recommendations

As noted in PTI DC10.1-08, "Design of Post-Tensioned Slabs-on-Ground,"[2] contraction joints are permitted in slabs-on-ground. This document states: "Control joints, which are weakened planes formed by tooling, sawcuts, or mechanical devices, can be used effectively to attract and conceal restraint-to-shortening cracks in slabs-on-ground." This document has significance, as it is the referenced PT slab design document in ACI 332-14, "Residential Code Requirements for Structural Concrete and Commentary."[3]

Contraction joints in PT slabs are desirable because random cracks will be eliminated or substantially mitigated, thus minimizing aesthetic concerns. Further, joints shown on construction documents convey design intent and acceptability from a structural standpoint.

Contraction Joints in PT Slabs

As previously noted, Bondy[1] discussed the structural influence of thermal contraction and drying shrinkage cracks on PT slabs. Figure 1 shows representative random cracks in a PT slab having a uniform thickness. The maximum moment in PT slabs-on-ground usually occurs from center lift conditions, in which shrinkage of the soil by drying can leave the concrete slab unsupported at the edges over a distance e_m (Fig. 2). The maximum moment is approximately located a distance β from the edge.

$$\beta = (1/12)(E_{cr} I/E_s)^{1/4} \text{ in U.S. Customary units}$$

or

$$\beta = (1/1000)(E_{cr} I/E_s)^{1/4} \text{ in SI units}$$

where β is the distance from the slab edge to the location of

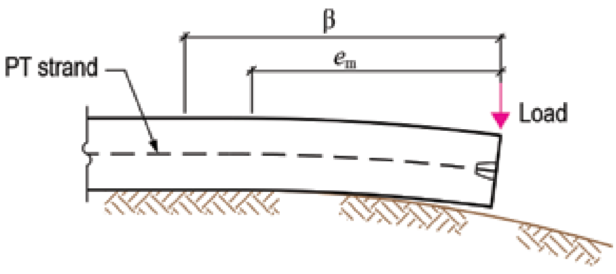

Fig. 2: Center lift condition, where the soil is dry at the slab edges, causing a portion of the slab to be cantilevered the distance e_m. Maximum moment location is approximately at β

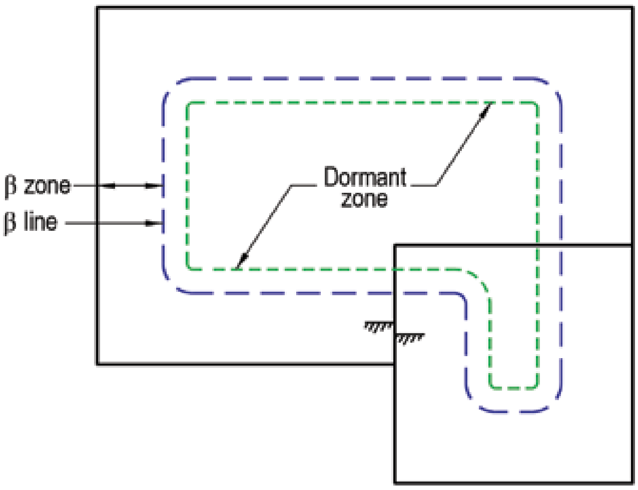

Fig. 3: The β-line is the approximate location of maximum moment. The dormant zone is the area of small moments and shears

maximum moment, in ft or m; E_{cr} is the long-term or creep modulus of elasticity of concrete, in psi or MPa; E_s is the modulus of elasticity of soil, in psi or MPa; and I is the gross moment of inertia of the slab perpendicular to the direction of bending, in in.^{4} or mm^4.

Later documents provide practical recommendations for the moduli values. "Design and Construction of Post-Tensioned Slabs-on-Ground"[4] states that E_s can be estimated as 1000 psi (7 MPa) if the modulus of elasticity of the clay soil is not known. PTI DC10.1-08 further states that E_{cr} may be taken as 1,500,000 psi (10,000 MPa) unless more refined calculations are used. In general, β ranges from 6 to 10 ft (1.8 to 3 m). For example, a slab with a uniform thickness of 10 in. and widths of 30 and 60 ft (9 and 18 m) will have β values of 6.8 and 8.1 ft (2.1 and 2.5 m), respectively.

The bending moment increases from zero at the slab edge to maximum at the β-line (Fig. 3) and then decreases rapidly over a short distance. The area inboard of the β-line is the dormant zone and is a region of low moments and shears.

Thermal and drying shrinkage cracks develop perpendicular to the maximum moment, so the presence of these cracks is innocuous. To mitigate the tendency for these cracks, it is common for designers to lay out joints at a

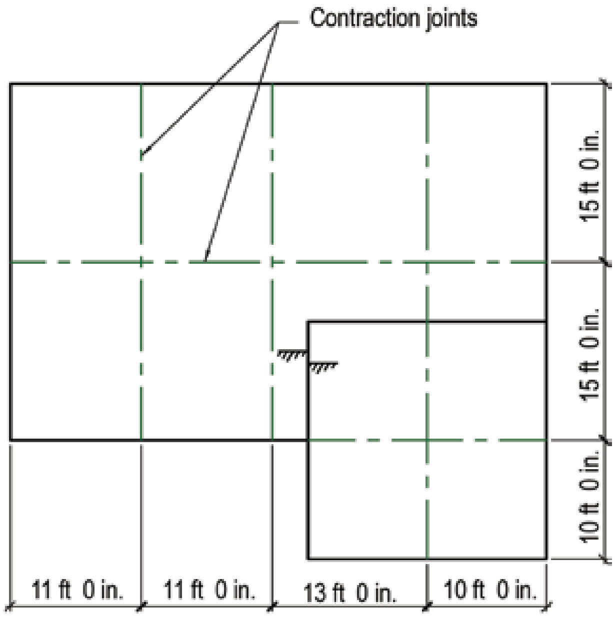

Fig. 4: Contraction joint layout for a PT slab (Note: 1 ft = 0.3 m)

spacing of 12 to 15 ft (3.7 to 4.6 m), starting about 10 to 12 ft (3 to 3.7 m) from the edge of the slab, regardless of the slab thickness. The maximum spacing of joints in PT slabs is the designer's prerogative—any spacing of joints will assist in minimizing or mitigating random cracks. Further, the aspect ratio of the panel defined by the joints is not as critical as it is for non-PT slabs. It is acceptable, for example, to use a joint layout that results in panels with aspect ratios exceeding 1.5. The design professional should determine the joint layout and show the joints on the construction drawings. Figure 4 provides an example of a joint layout. For aesthetics, joints in the garage area shown in the example would be expected to intersect the slab at midpoint locations. The interior slab areas will likely receive floor coverings, so joints in the interior slab areas do not need to be straight lines parallel to the slab edges.

Installation methods

Scoring, installing inserts, and making saw cuts are the standard methods used to produce contraction joints. Scoring is the preferred method for small slabs, installing polymer inserts for medium-sized slabs, and saw cutting for medium to large slabs. While saw cutting could be used for residential PT slabs, it introduces the risk of nicking the slab tendons. Residential slabs can be considered medium-sized slabs, so installing polymer inserts in fresh concrete is the preferred method. Two insert types are generally available: rigid strips[5] or flexible tape[6]. (Fig. 5). The latter method is new to the industry and holds promise as a significant advancement.

Summary

Currently, contraction joints are not commonly used in residential PT slabs, even though joints can minimize random

Fig. 5: This manually operated tool is used to embed a polymer tape into fresh concrete. The tape creates a stress concentration in the hardened concrete, typically resulting in a relatively straight crack that opens as the concrete shrinks *(photo courtesy of Aaron Hilbert LLC)*

cracking. PTI permits the use of contraction joints in PT slabson-ground for residential foundations, as joints will not detrimentally affect the structural behavior of the slab. It is essential, however, that a design professional determines the joint locations and provides the joint layout on the construction drawings. A defined and rational joint pattern will provide aesthetic benefits to the designer as well as the contractor and the owner.

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2. PTI DC10.1-08, "Design of Post-Tensioned Slabs-on-Ground," third edition with 2008 Supplement, Post-Tensioning Institute, Farmington Hills, MI, 2008, 156 pp.
3. ACI Committee 332, "Residential Code Requirements for Structural Concrete (ACI 332-14) and Commentary," American Concrete Institute, Farmington Hills, MI, 2014, 54 pp.
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5. Superior Profiles, Inc., 2014, www.superiorprofiles.com.
6. Wood, G., "New Tool for Slab Joints Reduces Concrete Cracking," *Concrete Contractor*, Jan. 2020, pp. 14-16.

Selected for reader interest by the editors.

Harvey Haynes, FACI, is a Principal of Haynes & Associates, a concrete consulting firm in Oakland, CA. He is a member of ACI Committees 201, Durability of Concrete; 224, Cracking; and 332, Residential Concrete Work. He is a past Chair of ACI Committee 357, Offshore and Marine Concrete Structures, and a past President of the Northern California and Western Nevada Chapter – ACI.

ACI Honorary Member **Kenneth B. Bondy** specialized in design and construction of post-tensioned concrete building structures for over 50 years. Now retired, he was a member of ACI Committees 132, Responsibility in Concrete Construction; 318, Structural Concrete Building Code; and 332, Residential Concrete Work, and Joint ACI-ASCE Committee 423, Prestressed Concrete. He is a Post-Tensioning Institute "Legend of Post-Tensioning" as well as an Honorary Lifetime Member and Fellow. He received his BS and MS in civil engineering in 1963 and 1964, respectively, from the University of California, Los Angeles, Los Angeles, CA. He is a licensed civil and structural engineer in California and has been licensed in many other states.

TECHNICAL REPORT

Reprint from CI Magazine, Volume 41, No 2, Page 36-41

A contribution from ACI Committee 212, Chemical Admixtures

Polymeric Microspheres Provide Resistance to Harsh Winter Conditions

by Edward (Ted) G. Moffatt and Michael D.A. Thomas

In cold climates, cyclic freezing and thawing is a leading cause of premature deterioration of concrete structures. The distress is characterized by scaling, cracking, and crumbling of the concrete.[1] Surface scaling is particularly exacerbated by the presence of deicing chemicals, and because scaling facilitates deeper ingress of these chemicals and moisture, it consequently increases the risk of other durability concerns, such as corrosion of steel reinforcement.[2]

However, deterioration due to cyclic freezing can be delayed by adding air-entraining admixtures (AEAs) to fresh concrete mixtures. AEAs protect concrete by creating millions of microscopic bubbles that alleviate internal stresses caused when water freezes within the concrete. Depending on the nominal aggregate size and exposure conditions, studies have shown that the air content required for protection of the concrete is typically in the range of 5 to 8% by volume of the concrete. Studies have also shown that the air void structure (bubble size and spacing) must be maintained within specific limits for protection of the concrete.[1,3-5] This is not a simple task, however, as air content and air void structure are affected by several factors, including characteristics of the cementitious materials, aggregates, and chemical admixtures; water content; type of mixer; and construction practices.[6] Alone or in combination, these factors can result in coalescence, size variability, and nonuniform dispersal of air voids within the concrete matrix.[7] For example, concrete mixtures with high workability are sensitive to excessive vibration and extended finishing operations, which can cause air bubbles to escape from the fresh concrete and bubbles to coalesce to create larger pores.

Issues with providing an optimum air entrainment can, however, be eliminated through the use of dimensionally stable particles known as microspheres. Such materials have been in use since 1977,[8] when the first patent was written for the application of hollow gas-filled synthetic polymeric spheres in concrete. Sommer[9] and Vanhanen[10] demonstrated the performance of dry, hollow microspheres to combat the effect of freezing and thawing.

While wider use of microspheres in concrete has been hindered by production and logistics costs, we believe they could be cost effective for applications that require highly reliable protection against freezing-and-thawing damage. This article compares performance test data for concretes produced with polymeric microspheres against test data for concrete produced with a conventional AEA.

Test Program

Materials and concrete mixture

The resistance to freezing and thawing was evaluated for three concrete mixtures (Table 1). All mixtures comprised a portland cement meeting the requirements of ASTM C150/C150M,[11] a 19 mm (3/4 in.) graded siliceous gravel, and a natural sand meeting the requirements of ASTM C33/C33M.[12] A polycarboxylate high-range water-reducing admixture was used to achieve a target slump of 125 to 180 mm (5 to 7 in.). The microspheres were dosed as a slurry mixture (of water and microspheres). The average particle size of the microspheres was 40 µm (0.0016 in.), shown in Fig. 1. Two mixtures, designated as MS-075 and MS-100, were produced using slurry dosages required to achieve microsphere contents of 0.75 and 1.00% of the total volume of the mixture. The water content of each mixture was adjusted to account for the water in the slurry.

Table 1:
Mixture proportions

Mixture	Proportions, kg/m³				AEA dosage, mL/100 kg (fl oz/cwt)	Microsphere admixture, % by volume
	Portland cement	Fine aggregate	Coarse aggregate	Water		
Reference	400	729	980	168	98 (1.50)	–
MS-075		842	980		–	0.75
MS-100		835	980		–	1.00

Note: 1 kg/m^3 = 1.7 lb/yd^3

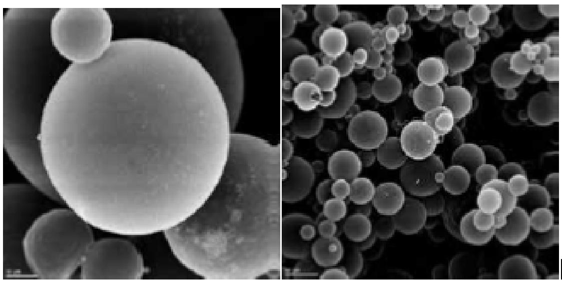


Fig. 1: Polymeric microspheres with nominal diameter of 40 μm (0.0016 in.)

A reference mixture was produced using a commercially available, vinsol resin-based AEA at a dosage needed to achieve an air content of about 6%. The mixtures incorporating microspheres were modified slightly relative to the reference mixture, as nonentrained concrete contains only entrapped air—typically less than 3% by volume of concrete. While maintaining the same portland cement content and water-cementitious materials ratio (w/cm), volume proportions were maintained by increasing the fine aggregate content to account for the lower air content in both MS mixtures.

Specimens and testing

All concrete was mixed using an inclined, rotating mixing pan. Slump, air content, and unit weight were measured in accordance with ASTM C143/C143M,[13] C231/C231M,[14] and C138/C138M,[15] respectively. The compressive strength was determined in accordance with ASTM C39/C39M[16] on 100 x 200 mm (4 x 8 in.) concrete cylinders at ages of 3, 7, 28, and 91 days after mixing. Resistance to freezing and thawing was evaluated using 75 x 100 x 405 mm (3 x 4 x 16 in.) prisms in accordance with ASTM C666/C666M, Procedure A[17]; testing commenced after 14 days of moist curing. Resistance to deicing salt scaling was evaluated on 300 mm diameter x 100 mm deep (12 x 4 in.) specimens in accordance with ASTM C672/C672M.[18] Specimens were cured under wet burlap and plastic at a room temperature of 23.0 ± 2.0°C (73.5 ± 3.5°F) for 24 hours and then stripped and moist cured for an additional 13 days, followed by 14 days drying in laboratory air. A polymer dam was then installed around the samples and the top surface was ponded with 100 mL of a 4% calcium chloride solution. Specimens were then exposed to 50 freezing-and-thawing cycles. Each cycle consisted of a freezing period of 16 hours and a thawing period of 8 hours. The only deviation from the standard test procedure (ASTM C672/C672M) was to rate the mixtures using a cumulative mass of scaled material after 50 cycles, in addition to the standard visual rating.

Larger test specimens were also produced for placement on field exposure sites. One 600 x 600 x 100 mm (24 x 24 x 4 in.) concrete slab was cast for each mixture and cured under wet burlap and plastic for 24 hours, after which the slabs were demolded and moist cured at room temperature for an additional 6 days. Specimens were then placed in laboratory air to dry for 21 days before being transported to the University of New Brunswick (UNB) exposure site. After each snow event, the snow was cleared from the top of the slabs and the surface was sprayed with 180 mL of 30 g/L NaCl solution (Fig. 2).

Fig. 2: Spraying slabs with NaCl solution at the UNB exposure site

Table 2:
Fresh and hardened concrete properties

Mixture	Slump, mm (in.)	Air, %	Density, kg/m³ (lb/yd³)	Compressive strength, MPa (psi)			
				3 days	7 days	28 days	91 days
Reference	140 (5.5)	6.2	2255 (3810)	29 (4200)	35 (5080)	48 (6970)	49 (7110)
MS-075	160 (6.3)	1.8	2364 (3995)	32 (4640)	42 (6090)	55 (7980)	62 (8990)
MS-100	150 (5.9)	1.7	2372 (4008)	38 (5510)	47 (6820)	59 (8560)	63 (9140)

Table 3:
Results of tests for resistance to freezing and thawing (ASTM C666/C666M) and deicer salt scaling (ASTM C672/C672M)

	Reference	MS-075	MS-100
Durability factor (*DF*)* after 300 cycles, %	105	101	103
Scaled mass loss after 50 cycles, g/m²	425	520	566
Visual rating after 50 cycles	2.5	2.5	2.5

*Per ASTM C666/C666M, Procedure A Note: 1 g/m² = 0.00328 oz/ft²

Additionally, six 6 x 6 x 21 in. (150 x 150 x 530 mm) concrete beams were cast for each mixture. These were cured under wet burlap and plastic for 24 hours, after which they were demolded and cured under wet burlap for an additional 27 days. Three beams from each mixture were stored in laboratory air for 9 months before being shipped to Eastport, ME, where they were placed on the mid-tide wharf at Treat Island. Treat Island is an outdoor exposure site operated by the U.S. Army Corps of Engineers. Located in Passamaquoddy Bay, which forms part of the Bay of Fundy, the site is exposed to the highest tides in the world (6.2 m [22 ft]). The site is also exposed to 100 to 160 freezing-and-thawing cycles per annum, making it one of the harshest concrete environments in the world.[19,20] The remaining three beams from each mixture were transported to the UNB exposure site. Following 3 years of natural exposure, all beams were removed from the exposure site.

Results

Fresh and mechanical properties

The fresh concrete properties and compressive strength results up to 91 days are presented in Table 2. ACI 318-14[21] provides requirements for the air content to ensure resistance to freezing and thawing. For severe exposure, an air content of 6% is required for a nominal aggregate size of 19 mm. Table 2 presents the air contents for the three mixtures determined in accordance with ASTM C231/C231M (the "pressure meter" test). For the concrete mixtures containing microspheres (MS-075 and MS-100), the air contents in Table 2 represent the entrapped air in the mixtures. This is a result of the inability of the pressure meter to measure the volume of microspheres in the concrete. Although the microspheres are hollow and compressible, the pressure used in the meter is not sufficient to compress and change the volume of spheres. The air content of such mixtures can be determined using ASTM C173/C173M[22] or a newly developed test method, called the "Microsphere Recovery Test," which has been submitted to CSA, ASTM International, and AASHTO for adoption as a standard test method for fresh concrete with microspheres.[23]

As shown in Table 2, concrete mixtures with microspheres (MS-075 and MS-100) achieved compressive strengths greater than that of the reference mixture at all ages. This was expected, as the microsphere concrete mixtures had significantly lower air contents than the reference mixture. It was expected that at an age of 91 days, MS-075 would have a significantly higher compressive strength than MS-100; however, this was not the case. This may be due to a slightly increased entrapped air content of this mixture, as indicated by the slightly lower unit weight.

Durability performance

Table 3 presents the resistance to freezing and thawing and deicer salt scaling of the three mixtures. Both microsphere mixtures (MS-075 and MS-100) showed slightly lower durability factor (DF) relative to Reference but significantly greater than the 80% minimum requirement per ASTM C260/C260M.[24] Also, Cordon and Merrill[25] found that a concrete achieving DF of 80% or higher is considered adequate to withstand a freezing-and-thawing environment.

The microsphere mixtures also showed an increased mass loss in ASTM C672/C672M testing, however; all mixtures fell well below the accepted limit of 0.80 kg/m2 (2.6 oz/ft2) after 50 cycles of freezing and thawing specified by the Ministry of Transportation Ontario.[26,27] Also, all specimens received a visual rating of 2.5, which falls between the rating of 2 and 3 in ASTM C672/C672M and represents concrete with slight to moderate scaling with some coarse aggregate visible.

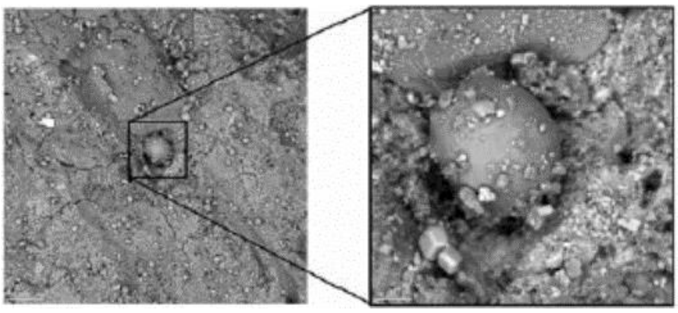
Field exposure

The visual appearance of concrete prisms following 3 years of marine exposure on Treat Island is presented in Fig. 3. All specimens were visually inspected on an annual basis using the rating system in ASTM C672/C672M. In general, all performed very well after 300 to 450 freezing-and-thawing cycles. Both Reference and MS-075 mixtures achieved a visual rating of 1—very slight scaling (3 mm [1/8 in.] depth, maximum, no coarse aggregate visible). MS-100 achieved a visual rating of 0—no scaling. The Reference specimens exhibited the most surface scaling, with mass loss along a few of the faces and along the edges of two of the three specimens.

After 3 years and about 50 NaCl treatments, slabs placed in an outdoor exposure site at the UNB campus showed minimal scaling deterioration. No difference was observed between the Reference mixture and the mixtures containing microspheres.

Discussion

The ability of concrete containing microspheres to resist freezing and thawing can largely be attributed to the mechanical properties of microspheres. Relative to the concrete matrix, the microspheres have low permeability, low modulus of elasticity, high resilience, and high rate of thermal contraction. The microspheres are also hydrophobic. These properties allow the formation of an interfacial zone between the polymeric microspheres, which provides stress relief during the expansion of freezing water in concrete (Fig. 4(a) and (b)).



a) b)

Fig. 4: Following 4 years of natural exposure at the UNB exposure site, cores were taken from test specimens and concrete was examined using a scanning electron microscope. These photomicrographs show: (a) a polymeric microsphere embedded in the concrete matrix (MS-100); and (b) the interfacial zone (void) surrounding the embedded microsphere

When water freezes, it increases in volume by 9%.[1,28,29] This volume increase results in a hydraulic pressure and a flux away from the zone of freezing, where water moves from capillary and gel pores (less than 100 nm [0.000004 in.]) to larger air voids (10 to 1000 µm [0.0004 to 0.04 in.]), due to the Gibbs-Thomson effect.[29,30] Preventing deleterious expansion is also achieved through an adequate void spacing, which Powers and Helmuth31 found must be less than 300 µm (0.01 in.). Due to the extremely small size of microspheres, Sommer[9] found that they are capable of achieving a spacing factor of approximately 7 µm (0.00027 in.).

Ong et al.[32] found that the performance of mortar containing 20 µm (0.0008 in.) diameter hollow microspheres at dosages of 1, 2, or 4% of the mortar volume was similar to that of conventional air-entrained mortar. They found that the coefficient of thermal expansion (CTE) of the mortar was initially 14 × 10–6/°C [7.8 × 10–6/°F]), then increased to 32 × 10–6/°C (17.8 × 10–6/°F) as ice formed in the interfacial zone, and then decreased to 10 × 10–6/°C (5.6 × 10–6/°F), when the temperature fell below −10°C (14°F). They hypothesized that the decrease was the result of ice crystals being pulled away from the surface of the mortar due to the thermal contraction of microspheres, which have been found to have a CTE ranging from 100 to 200 × 10–6/°C (56 to 111 × 10–6/°F). Because the microspheres contract 10 to 20 times faster than the surrounding mortar, the interface zone grows with decreasing temperature (Fig. 5).

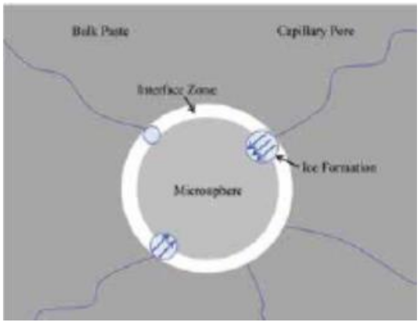


Fig. 5: Schematic representation of the hollow compressible microsphere providing freezing-and-thawing protection (based on Reference 32)

Ong et al.[32] also found that the freezing-and-thawing resistance is enhanced as the number of interfacial zones increases with an increase in volume fraction of the microspheres; however, little difference in performance was observed between mixtures with microsphere contents of 0.75 and 1.00% by volume of concrete. The results of the present study would suggest that these microsphere contents are sufficient to match traditional air entrainment in protection against resistance to freezing and thawing and salt scaling.

The improved mechanical properties of mixtures containing polymeric microspheres over mixtures prepared using conventional, liquid-based AEA gives the concrete producer several options. While these options include using microspheres to achieve a higher early strength relative to a reference mixture with AEA, they also include reducing the cementitious material content to achieve the same design strength as a reference mixture. The latter option provides a more sustainable alternative to conventional concrete containing entrained air bubbles.

Conclusions

Based on the results presented within this article and in previous studies, the following conclusions can be made:

- Concrete mixtures containing 0.75 to 1.00% by volume of polymeric, hollow microspheres with an average particle size of 40 µm have very high resistance to damage when exposed to cyclic freezing and thawing under saturated conditions, and in the presence of deicing salts;
- This protection is provided by an interfacial zone that forms around each microsphere as the result of the hydrophobic nature of the polymer and the higher CTE value of the polymer relative to the cementitious matrix; and
- Mixtures protected using microspheres have increased mechanical properties relative to mixtures protected using a conventional AEA. This indicates that mixtures with microspheres can have lower cementitious material contents yet match the strength of mixtures with conventional AEA—resulting in sustainability and cost benefits.

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CASE STUDY

Reprint from CI Magazine, Volume 41, No 2, Page 37-47

Assessing Corrosion of Reinforcing Steel

Lessons learned from select examples

by Andrew Fahim, Pouria Ghods, Aali R. Alizadeh, and Sarah Decarufel

Corrosion of reinforcing steel is the leading cause of concrete deterioration in North America and many other parts of the world.1 Normally, the corrosion rate of steel in concrete is limited by the high pH (> 12.6) provided by the concrete pore solution, which promotes the formation of a passive oxide layer on the reinforcement surface. However, active corrosion can initiate if pH-reducing reactions such as carbonation occur or if the reinforcement surface is exposed to chloride contents higher than a certain critical threshold. Corrosion propagation results in reaction products that occupy volumes greater than the original bars, and this leads to concrete cover cracking, spalling, and delamination.2 Following corrosion initiation, it is crucial to accurately determine the rate at which corrosion is propagating and the extent of the areas suffering corrosion to assess the type, urgency, and location of the required repair and rehabilitation.

Several methods have been used for corrosion detection and evaluation. These can range from simple visual observations or chain-dragging to electrochemical corrosion monitoring. This article presents two case studies in which we have used multiple methods for corrosion assessment. The aim is to introduce the capabilities and limitations of these methods and to describe how they can be synergistically used to achieve a more complete picture of the degradation mechanisms in reinforced concrete structures.

Corrosion Evaluation Methods

Visual and acoustic

The traditional corrosion detection method is visual inspection for rust stains or spalling and delamination of the concrete cover. Although this is, certainly, the simplest and least expensive option for detecting corrosion, it has a major drawback—the observer can detect corrosion signs only after corrosion has propagated significantly enough to damage the surrounding concrete. At this stage, repairs are rather costly and time-consuming to implement, since the concrete cover has already suffered an appreciable extent of damage. Other traditional methods include hammer sounding and chain-dragging (ASTM D4580/D4580M, “Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding”), in which the acoustic response of the concrete cover to hammer impact or a dragged chain is evaluated to detect delamination. ASTM D4580/D4580M notes that hammers or chains provide a clear ringing (high frequency) sound over nondelaminated concrete and a dull or hollow (low frequency) sound when delaminated concrete is encountered. Although these methods allow detection of internal cracking before external signs are apparent, they are only applicable after sufficient corrosion products have formed for cracking and delamination to occur. Acoustic methods also provide no information regarding degradation mechanisms.

Electrochemical

Because corrosion is an electrochemical process, electrochemical methods are the most theoretically sound methods of monitoring corrosion and determining its propagation rate. The most widely used electrochemical method for corrosion assessment is the corrosion potential method (ASTM C876, “Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete”). In this method, a $Cu/CuSO_4$ reference electrode (or another similar half-cell) is connected to the reinforcement through a hole induced in the concrete cover and the potential difference between the reference electrode and the reinforcing bars is recorded via a voltmeter. Through moving the half-cell over the concrete cover, different potentials are recorded, indicating

Table 1:
Summary of evaluation criteria for corrosion potential, corrosion rate, and electrical resistivity methods

Corrosion potential method		Corrosion rate method		Resistivity method	
Potential versus $Cu/CuSO_4$, mV	Corrosion probability	Corrosion rate, $\mu A/cm_2$	Corrosion state	Resistivity, ohm·m	Corrosion state
> −200	< 10%	< 1	Low	> 20	Low
−200 to −350	Uncertain	1 to 3	Moderate	10 to 20	Moderate
< −350	> 90%	3 to 10	High	5 to 10	High
—	—	> 10	Severe	< 5	Severe

different corrosion states (or different anodic dissolution states). The obtained potential is then used in conjunction with criteria listed in ASTM C876 to determine the risk of corrosion. This method is rather simple. However, there are drawbacks because the method:

- Results in localized damage to the cover to allow connection to the reinforcement;
- Is generally qualitative and does not provide information on the rate of corrosion propagation; and
- Is affected by variables such as cover depth, extent ofsaturation and oxygen availability, concrete resistivity, andanode-to-cathode ratios within the reinforcing network.[3,4]

The rate of corrosion propagation can be determined using electrochemical corrosion rate measurements. In this process, a surface-mounted device is used to apply a polarizing current ΔI. The shift in potential ΔV is measured and the polarization resistance R_p is determined using $R_p = \Delta V/\Delta I$. The corrosion current density (or corrosion rate) can then be found from the polarization resistance through the Stern-Geary equation $i_{corr} = B/(AR_p)$, where B is the Tafel constant (typically assumed to be 26 mV) and A is the bar area polarized by the applied current.

The traditional method of determining corrosion rate involves inducing a hole in the concrete cover from which a connection with the bar network can be established. This is necessary in determining the shift in the bar corrosion potential. This connection requirement, however, was overcome in the past few years through the introduction of the connectionless electrical pulse response analysis (CEPRA) concept, in which corrosion rate measurements can be obtained without the need for bar connection through a Wenner-probe setup, similar to that used for determining concrete resistivity.[5,6]

The electrical resistivity method is also widely used to assess concrete quality, thereby determining the risk of corrosion. The resistivity method determines the concrete resistance to the propagation of a high-frequency AC current (>1 kHz), determined through the Wenner array probe setup.7 The concrete resistivity is a direct measure of its porosity and pore-structure connectivity and tortuosity. Because porosity directly relates to concrete permeability and to the ionic transfer between anodes and cathodes formed over the bar during corrosion, this method can be used to indirectly estimate the risk of corrosion and its rate of propagation. To determine the corrosion potential, corrosion rate, and concrete resistivity, a commercial device capable of simultaneously performing the three measurements is used in the presented case studies. Table 1 provides a summary of the criteria used to deduce the risk of corrosion from results of these test methods.

Case Studies

The first case study is for the Three Nations Bridge Crossing located in Cornwall, ON, Canada. The south channel bridge is one of the many bridges crossing the U.S. and Canada border. The south channel bridge crosses the Saint Lawrence River and connects Cornwall Island, ON, with the State of New York. The bridge carries the Akwesanse International Road with traffic of about 2 million vehicles per year. The bridge is a high-level suspension bridge that straddles the waterway used by large ocean-going ships navigating the Saint Lawrence Seaway. This bridge opened to traffic in December 1958. The bridge evaluation work included visual inspection and chain-dragging as well as corrosion potential, corrosion rate, and electrical resistivity measurements. Figure 1 shows the location of the bridge and electrochemical measurements performed on the bridge deck using the commercial handheld device.

The second case study is for the LaSalle Causeway, located in Kingston, ON, Canada. The causeway provides an important link within Kingston across the Cataraqui River, between downtown Kingston and the Barriefield/Canadian Forces Base (CFB) area. About 23,000 vehicles cross the causeway every day. The causeway consists of five interconnecting structures: the East Bridge, the East Wharf, the Bascule Bridge, the West Wharf, and the West Bridge.

The East Bridge, presented in this study, opened to traffic in 1917, and the original single span through truss bridge was replaced by the current structure in 1969. Figure 2 shows the location of the East Bridge and photographs of measurements being taken on the underside of the deck. Although detailed testing was performed for the entire East Bridge, including the abutments and the pier, the presented results focus on the underside of the bridge deck. The evaluation work included visual inspection, hammer sounding, corrosion potential, and corrosion rate measurements.


a)


b)

Fig. 1: The Three Nations Bridge was evaluated using visual, acoustic, and electrochemical measurements: (a) aerial view of Cornwall Island, ON, Canada (left) and Akwesasne, NY (right); and (b) closeup of the device used to take electrochemical measurements

(a) (b)

Fig. 2: The LaSalle Causeway was evaluated using visual, acoustic,and electrochemical measurements: (a) aerial view of downtown Kingston, ON, Canada (left) and Barriefield/CFB (right); and (b) a view of the platform used to inspect the deck soffit

Three Nations Bridge

Figure 3 shows an example of severe cracking observed on the bridge deck after the removal of the asphalt wearing layer. Similar damage was visually observed throughout the full span area, albeit to different extents. Damage ranged from minor cracks to severe spalling.

Figure 4 shows a summary of the delaminated zones identified by both the visual inspection and chain-dragging methods. The results showed distributed damage throughout the full presented zone in localized areas. It should be noted, however, that portions of the visually observed delaminations and cracks were attributed to freezing-and-thawing and saltscaling damage and not specifically to corrosion propagation.

Figure 5 shows resistivity, corrosion potential, and corrosion rate results. In general, both corrosion rate and corrosion potential tests identified similar high-risk zones—namely, in the middle of the investigated zone and in the East zone near both U.S. and Canadian sides. The resistivity values recorded were generally in the range of 100 to 500 ohm·m, which, according to Table 1, indicates a low risk of corrosion.

However, it is well-established that resistivity is significantly influenced by the concrete moisture content during investigation. Therefore, these results are attributed to the dry condition of the slab

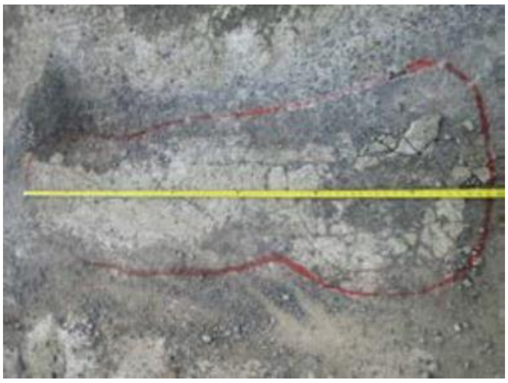

Fig. 3: An example of a damaged area on the Three Nations Bridge

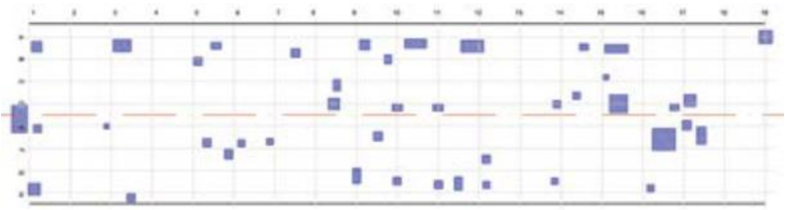

Fig. 4: Delamination zones (in blue) on the Three Nations Bridge identified by visual inspection and chain-dragging

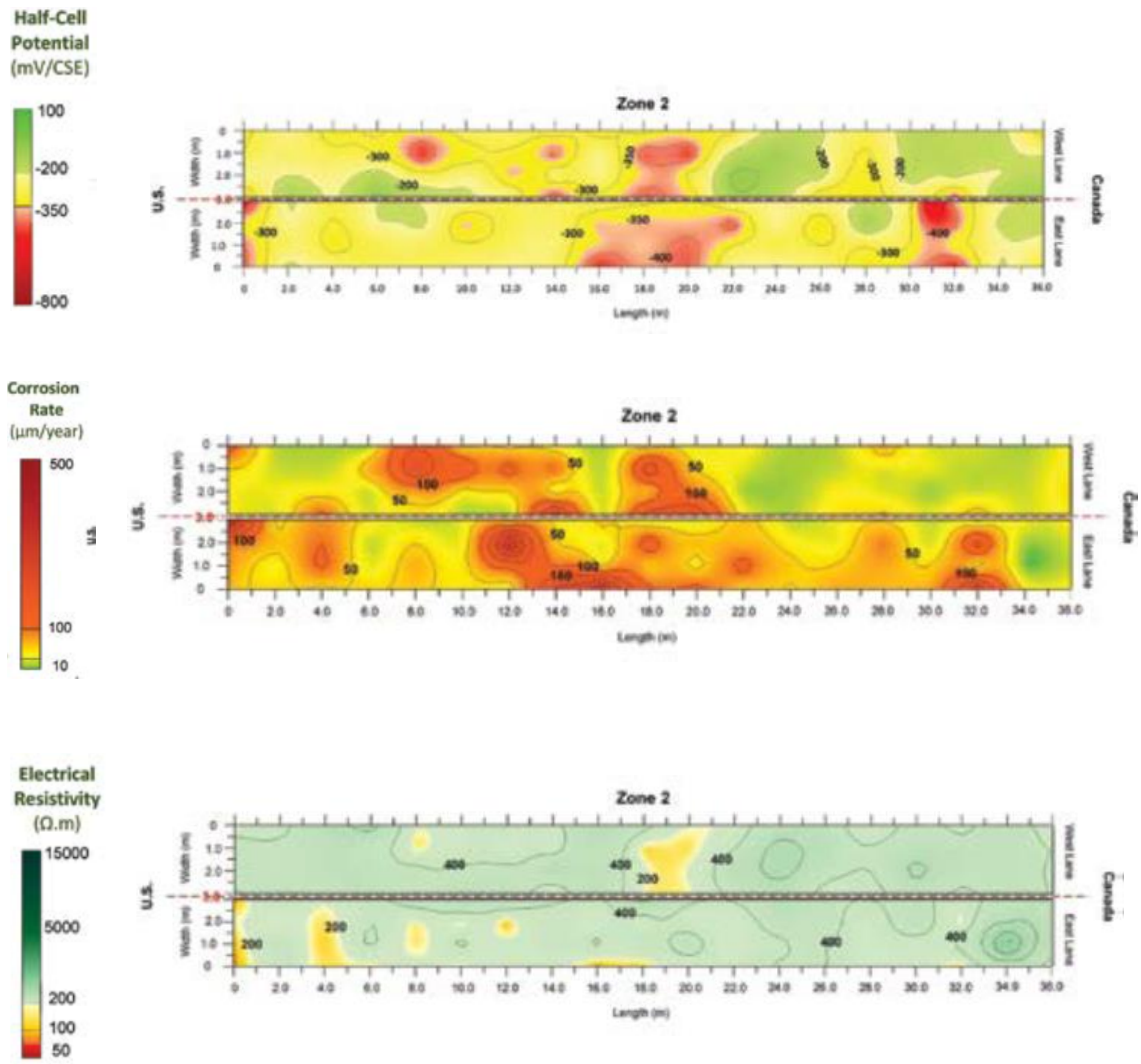

Fig. 5: Corrosion potential, corrosion rate, and electrical resistivity contour maps of the Three Nations Bridge

during the inspection. Nevertheless, the resistivity measurements were still able to detect anomalies (lower resistivity values) in the middle of the investigated zone and in the East lane near the U.S. border. This is in general agreement with the half-cell and corrosion rate results and is attributed to internal cracking or delamination caused by corrosion propagation.

The areas observed to be damaged by the visual inspection and chain-dragging methods were also identified by the combination of half-cell and corrosion rate monitoring. However, it should be noted that the inherent differences between these methods make them useful when applied in conjunction with each other. The chain-dragging and visual observation methods are only able to detect damage when sufficient corrosion propagation has occurred for corrosion to manifest itself. In contrast, the corrosion rate, corrosion potential, and resistivity methods provide information on the state of corrosion at the time of the measurement, even if corrosion has not propagated enough to cause sufficient damage to be visually or acoustically observable. This explains the greater area portions determined to be damaged through electrochemical methods (40 to 60% of the span surface area) when compared to visual and acoustic methods. It is expected that the areas with corrosion activity (determined electrochemically) will show signs of corrosion damage in the future.

It should also be noted that the electrochemical methods allow differentiation between areas degrading solely due to corrosion and those suffering damage due to deicing salt scaling or freezing and thawing (these are the minor areas found to be degrading [refer to Fig. 4] and were not found to have a corrosion activity). This information can be used to determine appropriate rehabilitation methods based on the cause of the damage. Furthermore, since corrosion rate measurements provide quantitative information regarding the rate of propagation, the urgency of repair can be determined and the time to cracking can be estimated.

Fig. 6: The underside of the LaSalle Causeway bridge deck exhibited visible damage only near the central pier

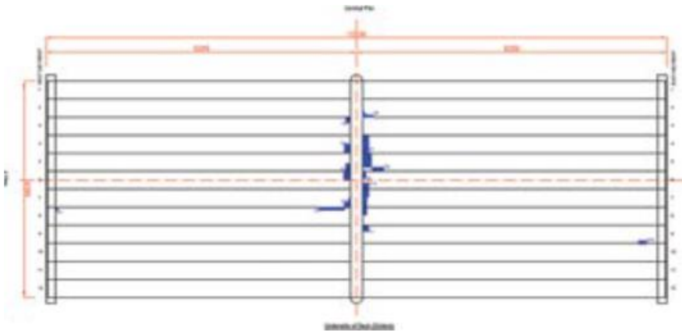

Fig. 7: Delamination zones (in blue) on the LaSalle Causeway bridge were identified through visual inspection and hammer sounding

LaSalle Causeway

Figure 6 shows the underside of the LaSalle Causeway bridge. It can be observed that the girders are in good condition with damage observed only near the central pier. Figure 7 shows the results of the visual inspection and hammer sounding survey on the bridge.

The survey showed significant damage due to corrosion at the area near the central pier, where delamination, cracking, and rust staining were found visually and via hammer sounding. The higher damage observed in this area is primarily attributed to the seepage of salt-laden water (from deicing salts) through the joint over the central pier.

No damage was observed in the area between the central pier and the two abutments. Figure 8 shows corrosion potential and corrosion rate contour plots for the underside of the bridge deck. The results from both methods agree with the visual inspection and show that the vicinity of the central pier is exhibiting significant damage as observed with the low corrosion potentials and the high corrosion rates. However, these methods also showed evident corrosion activity in girders 1, 2, 4, 11, and 12, and lower activity for the other girders. This corrosion activity was not observed visually, or by hammer sounding (refer to Fig. 6). This demonstrates the value of these methods in cases where corrosion has not yet led to concrete degradation. The results can act as an early alert for stakeholders, notifying them that these girders will eventually show some damage and allowing them to put into place early mitigation measures before concrete spalling or delamination occur. Such early detection of the susceptibility of members to corrosion damage cannot be done without the use of electrochemical techniques.

Conclusions

Visual inspection or delamination detection methods cannot detect corrosion initiation or early stage propagation, as these methods require some extent of damage to occur to the concrete cover. These methods also do not allow inspectors to differentiate between areas degrading due to corrosion from areas suffering other deterioration issues. A combination of corrosion potential, corrosion rate, and electrical resistivity testing can provide information on the location of the damage and the expected consequences of corrosion propagation. These methods generally

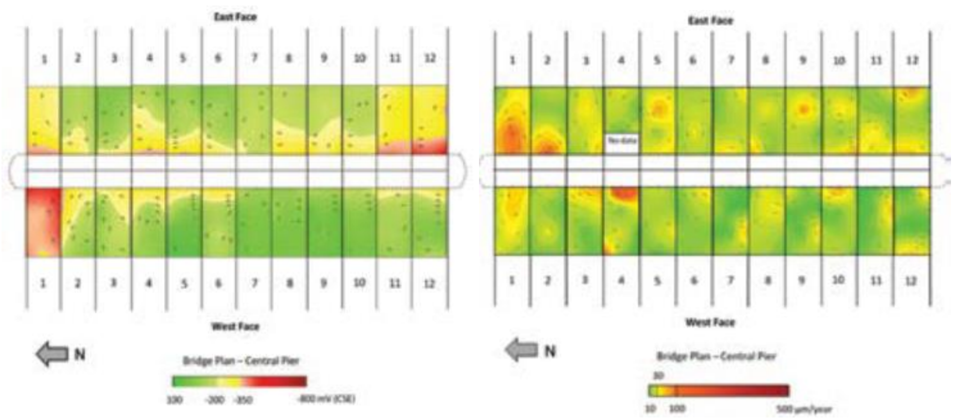

Fig. 8: Corrosion potential and corrosion rate contour maps of the LaSalle Causeway bridge

provide an earlier alert, before corrosion can manifest and cause significant degradation. They have been shown to be very effective when used in conjunction with each other and provide multiple points of view of concrete degradation, as shown in the presented case studies. Recent advances in commercial devices that can perform these measurements rapidly and simultaneously, while automatically collecting data and generating contour maps, can save significant time for inspectors and provide more information than the traditional inspection methods.

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ACI member **Aali R. Alizadeh** is the CEO of Giatec Scientific Inc. and an Adjunct Professor at the University of Ottawa, Ottawa, ON, Canada. He has over 15 years of research and practical experience in the development and commercialization of technologies related to concrete materials. He is a member of ACI Committees 207, Mass and Thermally Controlled Concrete; 236, Material Science of Concrete; 241, Nanotechnology of Concrete; and 347, Formwork for Concrete. He also serves on several committees of ASTM International and the CSA Group. He received his bachelor's and master's in civil engineering from the University of Tehran; his PhD from the University of Ottawa and the national research council of Canada; and a postdoctoral fellowship from Laval University, Québec City, QC, Canada.

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ACI member **Sarah Decarufel** is a Research and Development Associate at Giatec Scientific Inc. For the past 2 years, she has been focusing on the research and implementation of nondestructive testing technologies for the concrete industry. She received her bachelor's and master's in civil engineering from the University of Ottawa with a concentration in structural engineering.

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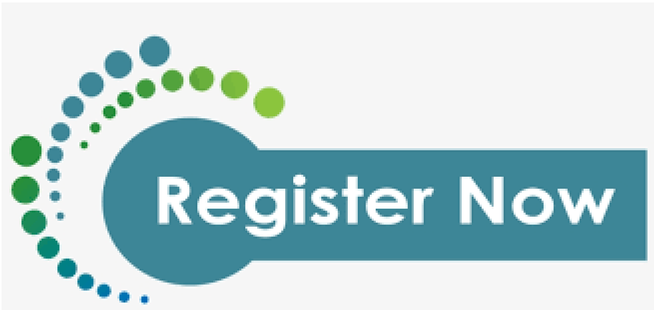

We look forward to your kind support and, more importantly, to your participation and registration as a member of ACI-Malaysia Chapter. It is our firm belief your involvement and together with your commitments will go a long way in our quest to uphold all our objectives to mutually benefits for all members.

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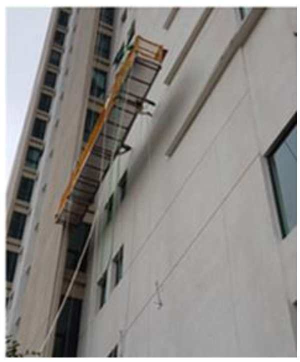

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