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Malaysia Chapter

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The Bulletin of the American Concrete Institute – Malaysia Chapter

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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.

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- 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.
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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.

Past Events

American Concrete Institute
Malaysia Chapter

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.

ARTICLE

Life-Cycle Resilience Assessment (LCRA) of Concrete Structures in the Tropics

Farnaz Jahandideh[1,2,a], Sudharshan N. Raman[3,b], Maslina Jamil[2,c]

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Introduction

Global warming and environmental sustainability are amongst the most critical challenges that are being debated across the world lately. Concrete is the most widely used construction material in most parts of the world, including in tropical regions [1,2]; consequently, concrete structures are responsible for significant environmental impacts, depletion of natural resources, and energy consumption through their life-cycle [3]. Due to the broad global use of concrete, it is essential to comprehensively assess the environmental impacts and CO2 emission of concrete structures [4].

This study focuses on a novel framework designated as Life-Cycle Resilience Assessment (LCRA). In the building industry, the term resilience is related to a combination of sustainability and recovery in the shortest possible time. This term is more tangible when an unexpected disaster occurs in urban infrastructures, and especially in residential buildings [5]. Thus, the aims of this study are to identify key parameters and factors that influence the resiliency of concrete structures in the tropics; to measure the resilience aspects of concrete structures; and finally to introduce a Life-Cycle Resilience Assessment (LCRA) framework for concrete structures by considering the identified parameters and factors, and the analytical findings.

Methodology

To assess the energy consumption and environmental impacts of concrete structures, a novel LCRA method which combines the aspects of sustainability, durability, and safety of structures, has been introduced and adopted in this study (Figure 1).

A critical review was been performed through a comprehensive bibliometric analysis related to the life-cycle sustainability assessment and environmental life-cycle assessment related to concrete structures. Consequently, a simplified LCRA analysis which relies on information available in public databases and the different phases of concrete structures' life-cycle (production, construction, operation, and maintenance) was accomplished. Based on Figure 2. the indicators considered in this study, are aspects of sustainability, durability, and safety; and the evaluated categories are energy consumption, acidification, eutrophication, human toxicity, and global warming potential.

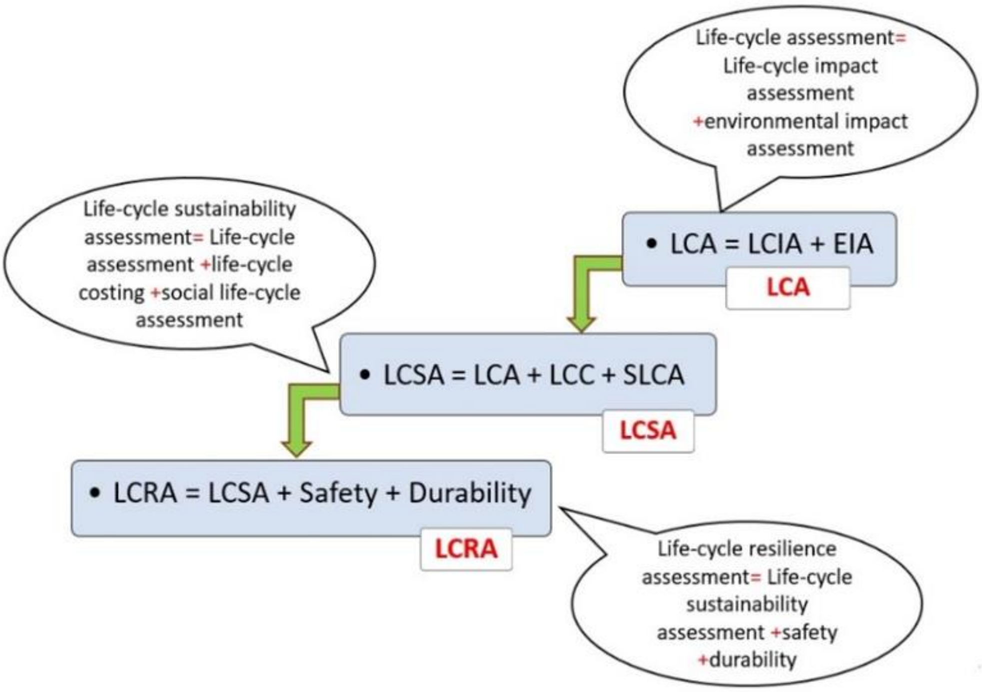

Figure 1. The stages of hierarchical formation of the LCRA framework

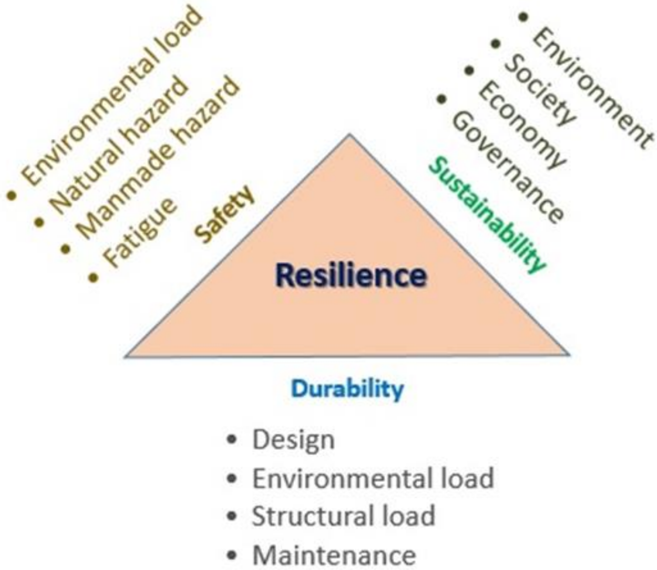

Figure 2. Resilience assessment aspects in building industry. Adapted from [6, 7, 8, 9]

The research adopted a mixed-method approach of qualitative and quantitative techniques in data collection and analysis. For quantitative data collection, a comprehensive questionnaire survey with a sample size of 382 respondents was conducted, and the findings were analyzed using the

SPSS software. Further, five residential buildings in Malaysia were selected as case studies for qualitative and quantitative data collection, and the IMPACT 2002+, CML 2001 in SimaPro computational tools were used for data analysis. Also, interviews with concrete and building professionals were undertaken to assess the durability and safety aspects of concrete structures in Malaysia.

Results and Discussion

The results obtained from the quantitative and qualitative studies indicated that the environmental sector and sustainability have the most impact (57%) on the LCRA of concrete structures. According to the survey analysis results, the maintenance of structures with 77% record is the most crucial aspect in durability assessment. The results of analyzing safety aspects showed that manmade hazards with 51% and disaster resistance with 50% have the highest efficacy on LCRA. Analyzing the life-cycle stages of concrete structures showed that the service life with more than 45% record, contributed the most effect on LCRA. The main findings of this study suggest that the environmental impacts of concrete structures are highly dependent on their location, the amount of concrete, and its material constituents; and the aspect of durability has a strong influence on the LCRA of concrete structures. The study demonstrated that substantial reformation in the production phase, reduction in transport in the construction phase, and regular quality control and inspection in the operation phase lead to a considerable reduction in CO_2 emissions, global warming potential, and energy consumption through the life-cycle of the structure [10,4].

Conclusion

This study revealed that the operation phase of a concrete structure's life-cycle, has the greatest environmental impact on acidification, global warming, and human toxicity; and the production phase is the second largest huge environmental influencer with the greatest impact on the global warming potential; while the overall effect of the construction phase is the lowest. In addition, in this study energy consumption is defined as the greatest environmental impact reason due to the use of fossil fuel as the key source for generating electricity in Malaysia [11]. Evidence revealed that fossil-fuel-generated electricity contributes to increasing greenhouse gas emissions that have led to global warming [12,13]. The findings of this study have shown that maintenance can be the most crucial factor; thus, it must be considered carefully during the durability assessment of a concrete structure. According to the results of this study, it can be seen that the construction industry in Malaysia does face challenges in the aspect of maintenance and repair of the structures during service life. Though these findings cannot be generalized due to the contribution of factors such as differences in location, definition, and objectives of each similar studies around the world, nevertheless, the results of this study can be used as a criterion for developing resilience assessment and sustainability of concrete structures in the tropics.

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TECHNICAL REPORT

Reprint from CI Magazine, Volume 41, No 2, Page 31-35

Precast Concrete Panels for Bridge Approach Slab Rehabilitation

Reducing lane closures to minimize traffic disruptions

by Leahann E. Powell, Timothy Barnard, and Shiraz Tayabji

In 2013, the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO) created the Implementation Assistance Program (IAP) to help transportation agencies deploy products developed under the Strategic Highway Research Program 2 (SHRP2). Seven rounds of the SHRP2 IAP were offered between February 2013 and April 2016.

In August 2015, as part of Round 6 of the program, the Florida Department of Transportation (FDOT) was selected as a lead adopter of precast concrete panel (PCP) technology. FDOT was awarded $300,000 to help offset the cost of constructing a PCP project that would provide a learning environment for operations needed to implement PCP installations, including panel fabrication, handling, setting, and grouting.

This article provides details of a project that used PCPs for rapid rehabilitation of a bridge approach slab. This type of rehabilitation is especially beneficial on roadways with high traffic volumes, where work can be performed only during short lane closures and where treatments are needed to ensure long service life.

Project Overview

The project involved the replacement of the existing approach slabs in the westbound direction on Bridge No. 500077 on I-10 (Florida SR 8), over the Apalachicola Northern Railroad near Quincy, FL[1]. The existing cast-in-place approach slab had been in service since the bridge was constructed in 1976, and it was exhibiting cracking and settlement (Fig. 1).

The existing slab was 20 ft. (6.1 m) long, and it was monolithic across the full width of the roadway. As is shown in Fig. 2, the existing slab had been constructed with two 12 ft. (3.7 m) travel lanes, a 10 ft. (3 m) outside shoulder (lane to gutter) and a 5.5 ft (1.7 m) inside shoulder (lane to gutter). Over a portion of the slab length, the shoulders incorporated barriers that were 16.5 in. (420 mm) thick at the base.

Fig. 1: The east-side bridge approach slab of westbound I-10.

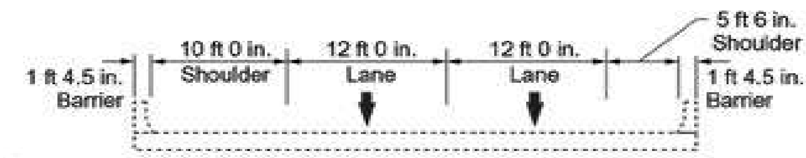

Fig. 2: Lane dimensions of the existing bridge approach slab (looking east). (Note 1 ft. = 0.3m; 1 in. = 25 mm)

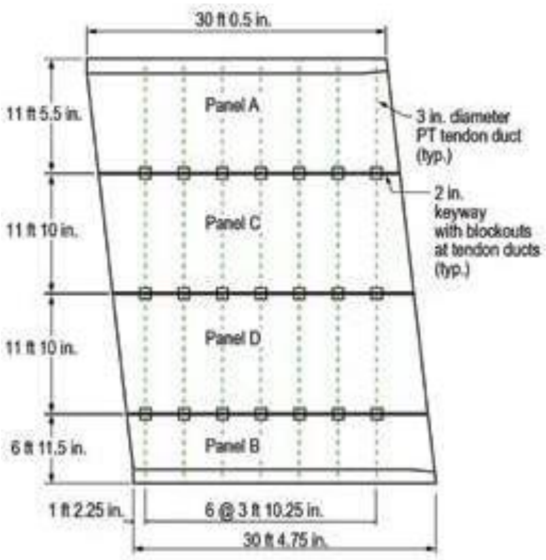

Fig. 3: Panel layout and post-tensioning duct locations at the I-10 bridge approach slab (Note: 1 ft. = 0.3 m ; 1 in. = 25 mm)

Before the panel installation work started, FDOT conducted a public awareness campaign to alert motorists to the possible disruptions to night-time traffic in the westbound lanes. The contract specifications required at least one lane be open to traffic throughout the project. This was accomplished by constructing the replacement slab using four precast concrete panels installed along the existing alignment during night-time lane closures, one at a time.

Panels were installed in the following order: Panel A (outside shoulder), Panel B (inside shoulder), Panel C (inside lane) and Panel D (outside lane). After all panels were in place, grout was placed in the longitudinal joints between panels, transverse post-tensioning tendons were stressed (tendon locations are shown in Fig. 3), a wax-based filler was pumped into the post-tensioning ducts, and bedding grout was installed under the panels.

Panel Design

All panels were 15 in. (381 mm) thick. The panel lengths varied from 30 ft. 0.5 in. to 30 ft. 4.75 in. (9.2 to 9.3 m). Panels were trapezoidal in shape to match the skew alignment at the bridge backwall. The panel widths (measured at the top of the units) were 11 ft. 5.5 in (3.5 m) and 6 ft. 11.5 in. (2.1 m) for Panels A and B, respectively. Panels C and D were both 11 ft. 10 in. (3.6m) wide (Fig. 3). All panels had top and bottom deformed bar reinforcement in both directions. They also included longitudinal prestressing stands. Panel B reinforcement was like that in Panel A, including reinforcement for integral barrier walls.

Fig. 4: Panel fit-up was verified at the precast plant on March 27, 2018.

Panel Fabrication

Panels were fabricated at a precast plant in Leesburg, FL, about 250 miles (400 km) from the project site, based on design drawings prepared by the design engineer, American Consulting Professionals, LLC, Pace, FL, and approved by FDOT. Project specifications called for Class IV concrete with 28-day compressive strength of 5500 psi (38 MPa). The concrete was produced at the precast plant and was regularly tested for fresh properties and for strength.

As required by the project specifications, panel fit-up was verified at the precast plant (Fig. 4). The test installation also helped to demonstrate how the panels would be placed at the project site.

Panels were stored at the precast plant and delivered to the project site a few days before the scheduled installation. At the project site, the panels were stored in the median area adjacent to the bridge slab.

Panel Installation

Panel installation began in April 2018 and was completed in June 2018. The planned installation had to be postponed on several occasions due to inclement weather. To ensure traffic flow during the work, the existing slab was removed and replaced only in one lane or shoulder zone at a time. The installation steps used for each of these phases included:

- Removing the existing slab section for the lane being replaced;
- Removing the existing base;
- Preparing the backwall seat (Fig. 5);
- Grading and compacting the sub-base;
- Placing a geogrid over the sub-base;
- Placing, compacting, and grading a new base;
- Placing a double layer of polyethylene sheeting over the base;
- Placing steel bearing plates on the sheeting (Fig. 6);
- Placing the precast panel while maintaining the longitudinal joint widths as specified;
- Adjusting the elevation of the panel, using levelling screws, to the specified surface elevation;

Fig. 5: The backwall seat was prepared by removing unsound concrete, repairing the seat with a high-early-strength patching material, drilling holes for the panel anchor dowels, and installing neoprene bearing pads.

Fig. 6: Bearing plates were placed over two layers of 12 mil (0.31 mm) polyethylene sheet. The bearing plates were positioned to match the location of levelling screws in the panels.

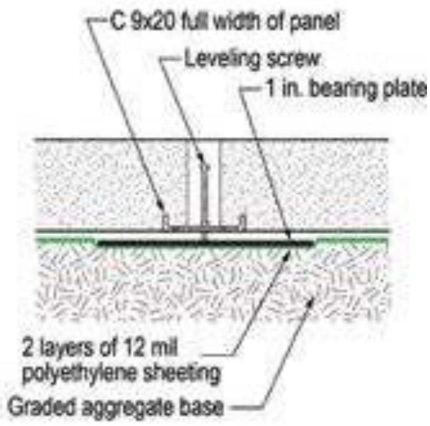

Fig. 7: Levelling screw schematic (Note 1 in. = 25 mm: 12 mil = 0.31 mm)

- Installing and grouting anchor bolts (dowels) along the backwall seat; and
- Finishing and opening to traffic.

In each of the traffic lanes, the existing approach slab was initially removed only in a 2 ft. (0.6 m) wide zone above the backwall. This step was taken to allow the contractor to repair the backwall and return the lane to traffic the next day (by covering the hole using steel plates). However, the contractor also had the option of continuing to work through the following day, removing all concrete in the affected lane and preparing the base.

Panel elevations were adjusted using levelling screws in the panels and bearing plates installed on the polyethylene sheets installed on the prepared base (Fig. 6 and 7). Each panel required two bearing plates, each 1 in. (25 mm) thick and 4 ft. (1.2 m) wide, positioned near mid-panel and at the end opposite to the bridge backwall. To ensure a low bearing stress on the base, the plate lengths were slightly less than the panel width. Each panel included six 2 in. (50 mm) diameter levelling screws (three at each plate, equally spaced along the width of the panel).

Panel A (the outside shoulder) was installed during the night of April 21, 2018, using a 220,000 lb (99,790 kg) capacity crane (Fig. 8a) Panel B (the inside shoulder) was installed during the night of May 7, 2018. Panel C (the inside travel lane) was installed during the night of June 5, 2018 (Fig. 8b). Panel D (the outside lane) was installed during the night of June 17, 2018.

After Panels C and D were installed, longitudinal keyway joints were filled with high-strength grout (Fig. 9 and 10) before the pavement was opened to traffic. The specification called for a minimum 28-day compressive strength of 5500 psi and minimum opening-to-traffic strength of 2500 psi (17 MPa). The specification also required a minimum strength of 5500 psi prior to tensioning of the transverse tendons.

The post-tensioning tendons comprised seven 1 in. diameter bars, spaced at 3 ft. 10.25 in. (1.2 m). Final applied load per bar was 57,800 Ibf (257 kN). To ensure adequate corrosion protection for the tendons, the specification required verification that the tendon duct system was leakproof and that the ducts could be completely filled with wax-based filler. The ducts were successfully tested on June 26, 2018, and the tendons were stressed on June 27, 2018. Figure 11 shows the post-tensioning jack applying load at an anchor plate.

The specification required that the bedding grout have a minimum 28-day compressive strength of 5500 psi. The bedding grout was installed under the panels after the tendons were tensioned. After all *finishing and clean-up* activities were completed, a coating of high-molecular-weight methacrylate was applied over all four panel surfaces to seal the concrete. The completed bridge approach slab is shown in Fig. 12.

a)

b)

Fig. 8: Panel installation: (a) Panel A; and (b) Panel C

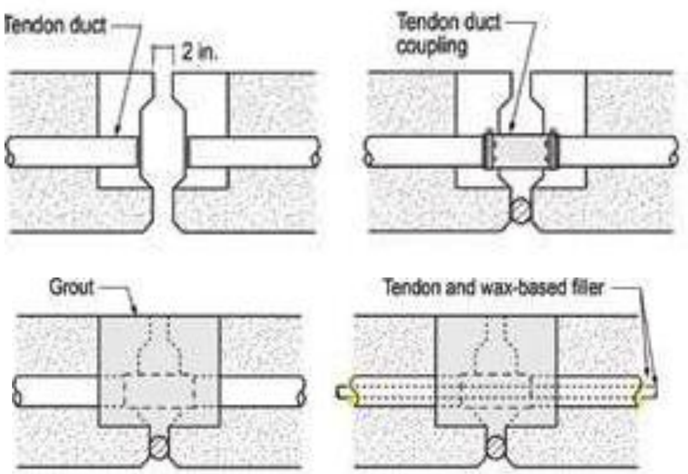

Fig. 9: The keyways and tendon ducts were designed to prevent leakage or blockage: (a) block-outs provided access to the tendon ducts; (b) a backer rod was installed over the length of the keyway and the tendon ducts in adjacent panels were coupled; (c) keyways were grouted after Panels C and D were installed; and (d) after all panels were in place, the tendons were installed, tensioned (starting from the center of the group), and protected with a wax-based filler (Note: 1 in. = 25 mm).

Fig. 10: Workers finish the keyway grout at the conclusion of a panel installation.

Fig. 11: A worker checks tendon extension during the tensioning operation

Summary

This project was an important step in the implementation of precast panel use for bridge approach slab rehabilitation in Florida. The FDOT project staff reported that the overall goal of the project – to reduce daytime lane closures on I-10- was accomplished. Traditional bridge approach slab projects have 14 days of lane closures, while this rehabilitation only required a total of 2.5 days to allow for continuous construction during the two travel lane panel placements, all the time maintaining one lane open for traffic.

Based on the experience gained at the I-10 project, the FDOT will refine the provisional specifications and plans with the goal of improving the efficiency and quality of panel placement at future bridge approach slab rehabilitation projects.

Fig. 12: Completed bridge approach slab

Selected for reader interest by the editors.

Acknowledgments

The information presented in this article is based on a case study report developed under FHWA contract DTFH16-13-D-00028. Samuel S. Tyson, FHWA Concrete Pavement Engineer, served as the FHWA Contracting Officer's Representative for the contract. The Support of the FDOT and Tyson during the preparation of the case study report is greatly appreciated.

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CASE STUDY

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Cathodic Protection and Concrete Repairs at Sound of the Sea II Condominiums

Assessment after a decade indicates long-lasting success.

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by David G. Tepke, Clement A. Firlotte, and Stephen P. Robinson.

Constructed in 1984, Sound of the Sea II is a six-level reinforced concrete condominium building with 36 individually owned units. It is on the 0.75 mile (1.2 km) wide Bogue Banks Island within a few hundred feet of the Atlantic Ocean in Emerald Isle, NC, USA. It includes exterior common access corridors (walkways) on the front side of the building and private balconies for each unit (Fig. 1).

The location of Sound of the Sea II makes it highly susceptible to chloride penetration from ocean-borne salts and associated corrosion-related distress of embedded steel. Deck coatings were reportedly installed on corridors and balconies circa 1999. These deck coatings did not effectively arrest corrosion, as was evidenced by the formation of subsequent delaminations. Given the cost, inconvenience, and impact on renter and owner occupancy associated with capital repair projects, the owners commissioned a comprehensive repair project in 2007/2008 to address corrosion-related concrete distress, including preservation of existing concrete with cathodic protection (CP). The repair project, totaling approximately $1.2 million, was completed in December 2008 and included:

- Structural concrete repairs;
- Impressed current cathodic protection (ICCP) on balconies;
- Localized galvanic cathodic protection (GCP) in corridors and roof parapet repairs;
- Steel stair repairs;
- Improved drainage; and
- Protective coatings.

Evaluation

Evaluation of the structure prior to design included review of previous testing and assessment reports, on-site assessment, and sample collection by the design team. The

a)

b)

Fig. 1: Sound of Sea II Condominiums. (a) viewed from the beach; and (b) viewed from the parking lot in 2019 (more than 10 years after repairs).

design team reviewed and used information from a preliminary durability assessment report and a delamination survey report prepared by a testing and consulting firm in 2006. The reports were provided to the design team by the management company for review and included results and general recommendations. The reports indicated generally low reinforcing steel cover, chlorides at reinforcing steel depth in sufficient concentrations to initiate corrosion (approximately 500 to 1500 parts per million in some locations), significant delamination, and evidence of corrosion based on half-cell potential testing. Considerably more delaminations were identified on balconies as compared to corridors.

In 2007, excavations were made as part of the design effort to review embedded steel and structural conditions. Core samples were extracted from one corridor area and one balcony for resistivity testing by a third party. Resistivity was found to be relatively low and thus conducive for active corrosion for both samples. Of note, the balcony sample (4800 ohm-cm as received; 3000 ohm-cm after 12 hours of saturation and removal from bath) had about 60% the resistivity of the sample taken from the corridor (7700 ohm-cm; 4700 ohm-cm). Excavations revealed significant reinforcing steel corrosion (Fig. 2).

Fig. 2: Example of test excavation during 2007 assessment

Design

Budget was of the utmost importance to the owners. Thus, it was necessary to evaluate repair/preservation options that would extend service life but not be overconservative and cost prohibitive. The designers discussed the merits of global ICCP and GCP with the owners. Given the spatial differences in exposure, identified differences in magnitude of distress, budgetary constraints, and the owners' directives, two separate approaches for repair and protection were adopted. Concrete repair—an alternate for ICCP—and cementitious deck coatings were specified for balconies where distress was more severe. Limited concrete repairs, urethane deck coatings, and an alternate for localized GCP were specified for corridors where distress was more sporadic and exposure somewhat less severe. Alternates were provided in the bid for CP to provide flexibility for the owners. A general overview of primary repairs to the corridors and balconies is summarized in Fig. 3.

As required by the owners, sliding glass doors remained in place during balcony repairs and existing embedded guardrails were not replaced. Ability to remove, store, and reuse guards to conduct repairs required special approval from the code official.

The design included a performance specification with additional prescriptive requirements for ICCP, including a 10-year no-corrosion warranty. Repair materials compatible with localized GCP and the global ICCP were specified.

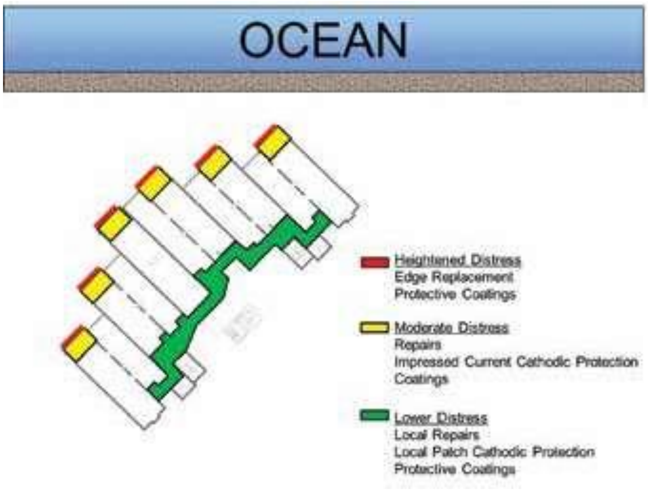

Fig. 3: Schematic showing tower position and final repair strategy

Construction

General

The owners elected to accept the alternates for ICCP for balconies and localized GCP for corridors and roof parapets. The Notice to Proceed was issued in October 2007. The project was substantially complete for all items except corridor coatings by December 2008, with completion of coatings following within a few months.

Repairs included horizontal, full-depth, vertical, and overhead orientations and were more substantial on balconies as compared to corridors. Repairs were completed by saw-cutting surfaces at repair extents, chipping deficient concrete, preparing surfaces via sandblasting, and conditioning to achieve a saturated-surface dry condition. Prepackaged portland cement-based repair materials were used for all repairs. Pre-extended repair mortars were used for deeper repairs. Repairs were moist-cured after installation.

All horizontal surfaces were shotblasted and coated with cementitious (balconies) and polyurethane (corridors) deck coatings after concrete repairs. Overhead and vertical surfaces were prepared and coated with a breathable acrylic coating where required.

a)

b)

c)

Fig. 4: Balcony repairs:
(a) general construction access with visible swing stage;
(b) extent of distress; and
(c) one of the more severely distressed balconies requiring shoring.
Note demising walls at balcony sliding glass doors.

Balconies

Swing stages were used for balcony access (Fig. 4 (a)). Due to project logistics and the amount of uncovered distress (Fig. 4(b)), it became immediately evident that the most efficient and cost-effective manner to address slab edges was to remove the entire slab edge. Although some sound concrete was removed, this method was less complicated and allowed for more efficient installation of supplemental reinforcement to address severely corroded reinforcing steel. It also allowed for effective coordination and installation of drip edges and guardrail posts after repairs. A significant challenge to the project was to minimize damage to guardrails during removal, storage, and reinstallation, and to make sure that slab edge installation was coordinated with new precisely located post pockets.

In general, less concrete removal than typical was necessary due to the use of ICCP. However, some balconies required full-depth repairs over substantial areas that required temporary support (Fig. 4(c)). Finite element analysis was used during the project to evaluate conditions for making field decisions. Mechanical splices and adhesive anchors were used where severely corroded reinforcing steel or inadequate steel was encountered and limited space was available for lap splicing. This also allowed for reduced quantities of concrete removal.

Balcony repairs required protection of sliding glass doors. Repairs generally terminated at the doors; however, distress at a limited number of areas required that repairs continue under thresholds into units. This required careful and well-planned execution of concrete repairs, as well as rapid cure repair materials to minimize impact to interiors and occupants.

Concrete repairs and reinstallation of guardrails were coordinated with the ICCP system. Reinforcing steel was tied together at repair areas to reduce the potential for stray current corrosion. Reinstalled aluminum guard posts were placed in polyvinyl chloride (PVC) pockets filled with epoxy to provide a dielectric barrier between guards and concrete subjected to ICCP. Textured, breathable cementitious deck coatings were installed at balconies for aesthetics and functionality.

Corridors and other areas

Lift and foot access were used on corridors. Repairs at corridors (Fig. 5(a)), columns, and walls were less frequent and generally smaller than those at balconies. Repairs at roof parapets (Fig. 5(b) and (c)) were variable. Commercially available alkali-activated galvanic zinc anodes were embedded in patch repairs to help prevent incipient anodes directly outside repair areas and associated distress. Anodes were tied to reinforcing steel within patches and all reinforcing steel and anodes within patches were confirmed electrically continuous. Reinforcing steel was coated with a cementitious/epoxy anticorrosion bonding agent. This had the likely added effect of diverting current from the anode to the existing substrate concrete.

5(a) 5(b) 5(c)

Fig. 5: In progress repairs: (a) exposed reinforcement in corridor; (b) exposed reinforcement in parapet; and (c) GCP and formwork installed at parapet repair.

Fig. 6(a) Fig. 6(b) Fig. 6(c) Fig. 6(d)

Fig.6: ICCP system features: (a) anode and isolation from low-cover reinforcing bar; (b) grouted anode slots; (c) balcony deck as finished with vertical conduit (photo taken 2010); and (d) control and power system components (May 20, 2019).

Flood testing was conducted to identify ponded areas. Additional drains to address significant ponding and a urethane deck coating were installed as water management and waterproofing measures. Deck coatings extended up the wall with mesh reinforcement at the transition.

Balcony ICCP System

The ICCP system (Fig. 6) was designed by the manufacturer to meet project and contract requirements during the project and was installed by an experienced specialty contractor. It includes 3/8 in. (10 mm) wide titanium/mixed metal oxide (MMO) mesh ribbon anodes with a current rating of 0.85 mA/ft (2.79 mA/m) grouted into 9/16 in. (14 mm) wide and 1/2 in. (13 mm) deep slots spaced approximately 12 in. (305 mm) on center and connected to 1/2 in. wide titanium headers. Testing was conducted to confirm continuity of reinforcement and separation of reinforcement from anodes. Embedments were grounded to the system negative. Where anodes and reinforcement were found to be continuous, they were isolated. System negative and anode connections were led through concrete to a junction box at each of the 36 balconies and then to the constant voltage transformer/rectifier in the mechanical room at the roof via PVC conduits painted to match the building.

Circuitry was split into three cathodic protection zones, each with 12 units over two vertical balcony stacks. Wiring for each balcony was connected to the assigned zone through a variable resistor for individual balcony control. Each zone included four embedded Ag/ AgCl/KCl reference electrodes (silver/ silver chloride reference electrodes immersed in a potassium chloride solution) for depolarization testing, commissioning, and future monitoring. Remote monitoring equipment and a modem were installed that allowed for system review.

The system was commissioned and adjusted in September 2008 with approximately 60 mA current delivered to the average balcony (approximately 0.73 $mA/ft^2_{steel,\ design}$ [7.86 $mA/m^2_{steel,\ design}$]; 0.48 $mA/ft^2_{concrete}$ [5.17 $mA/m^2_{concrete}$]; 0.41 mA/ft_{anode} [1.35 mA/m_{anode}]). Depolarization testing per NACE Publication 35108[1] over a 4-hour period was used to evaluate the system, with 100 mV depolarization being considered effective protection. Depolarization data during commissioning are shown in Fig. 7.

Overall Repair Totals

The final project cost was approximately $1.2 million. About 830 ft² (77 m²) of horizontal concrete repairs, 275 ft² (26 m²) of overhead repairs, and 30 ft² (3 m²) of vertical repairs were conducted. Approximately 4660 ft² (430 m²) of balconies are protected by ICCP.

Recent Site Visits and Closing Remarks

The management company has communicated that no major repairs associated with the work conducted in 2007/2008 have been needed and that only a minimal amount of maintenance has been necessary. This contrasts with other nearby properties requiring much more extensive repair and maintenance. Guardrails were reportedly replaced subsequent to the project in 2015; however, concrete repairs, coatings, and cathodic protection installed as part of the 2007/2008 project generally have performed well and are in serviceable condition, even withstanding a number of hurricanes over the past 10+ years in the very harsh, chloride-rich environment.

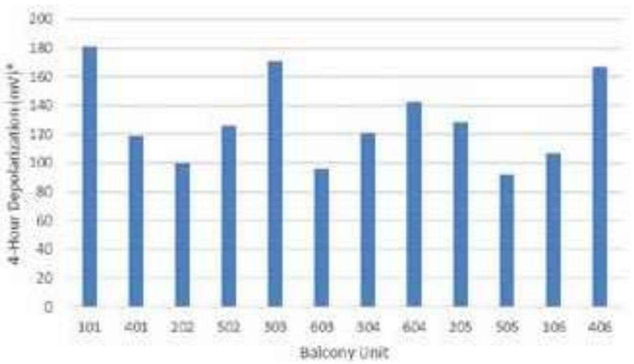

Fig. 7: Four-hour depolarization (September 25, 2008) (Note; Unit 505-97 mV at 6.25 hours; Unit 603-102 mV at 5.25 hours)

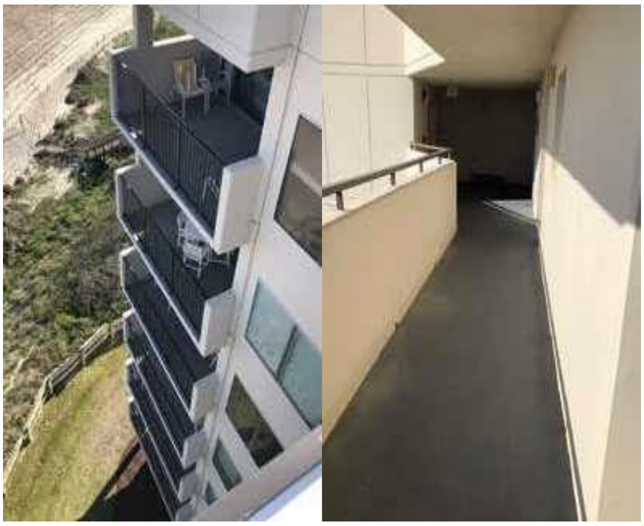

Fig. 8: Typical balconies and corridor area as of May 20, 2019.

Table 1:
Summary of 2008 and 2019 depolarization date.

ICCP depolarization*	September 2008	May 2019
Average 4-hour depolarization of reinforcing steel, mV.	130	182
Average depolarize potential of reinforcing steel, mV **	-269	-199

* *Based on 12 embedded Ag/AgC1 reference electrodes*
*** Converted to CSE*

During visits in 2019, it was observed that critical ICCP system components were functioning to protect reinforcing steel within the original design parameters. While some small discrete corrosion stains and one small delamination were observed on balcony soffits, likely associated with discontinuous steel embedments, no major concrete distress was observed on balconies repaired and protected with ICCP in 2007/2008. The remote monitoring device had its backup battery replaced as part of routine maintenance, and the site-monitoring device and wiring conduits on the roof required maintenance after a recent major hurricane; however, these items have not affected protection. Guardrail anchorages in the concrete from the 2015 guardrail replacement project had variable connectivity, demonstrating the need for coordination of all repairs to concrete subjected to ICCP. Some concrete distress was observed on walkway areas and roof parapets where ICCP was not installed, and some isolated deck coating damage was observed, but installed repairs and coatings generally have performed well. Balconies and corridor areas as of May 2019 are shown in Fig. 8.

Table 1 contains a comparison between data collected and reported by the System Designer in 2008 and 2019. After 11 years, the average depolarization increased and the average depolarized potential decreased, reaching, on average, a level of passivation (more positive than -200 mV versus a copper-copper sulfate reference electrode [CSE]) per NACE Publication 35108.[1] This project serves as an example of how ICCP can provide long-term service-life extension and shows that strategically selecting different preservation strategies on a building can provide economic, effective, long term benefits.

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Selected for reader interest by the editors

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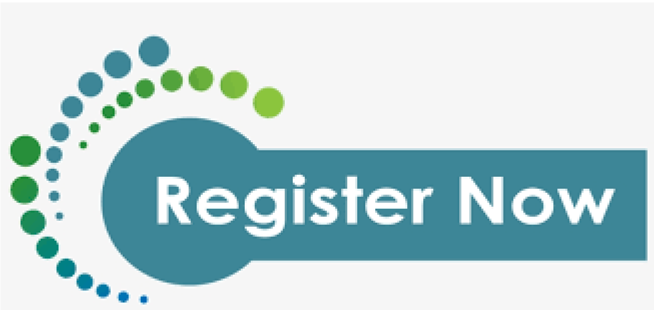

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