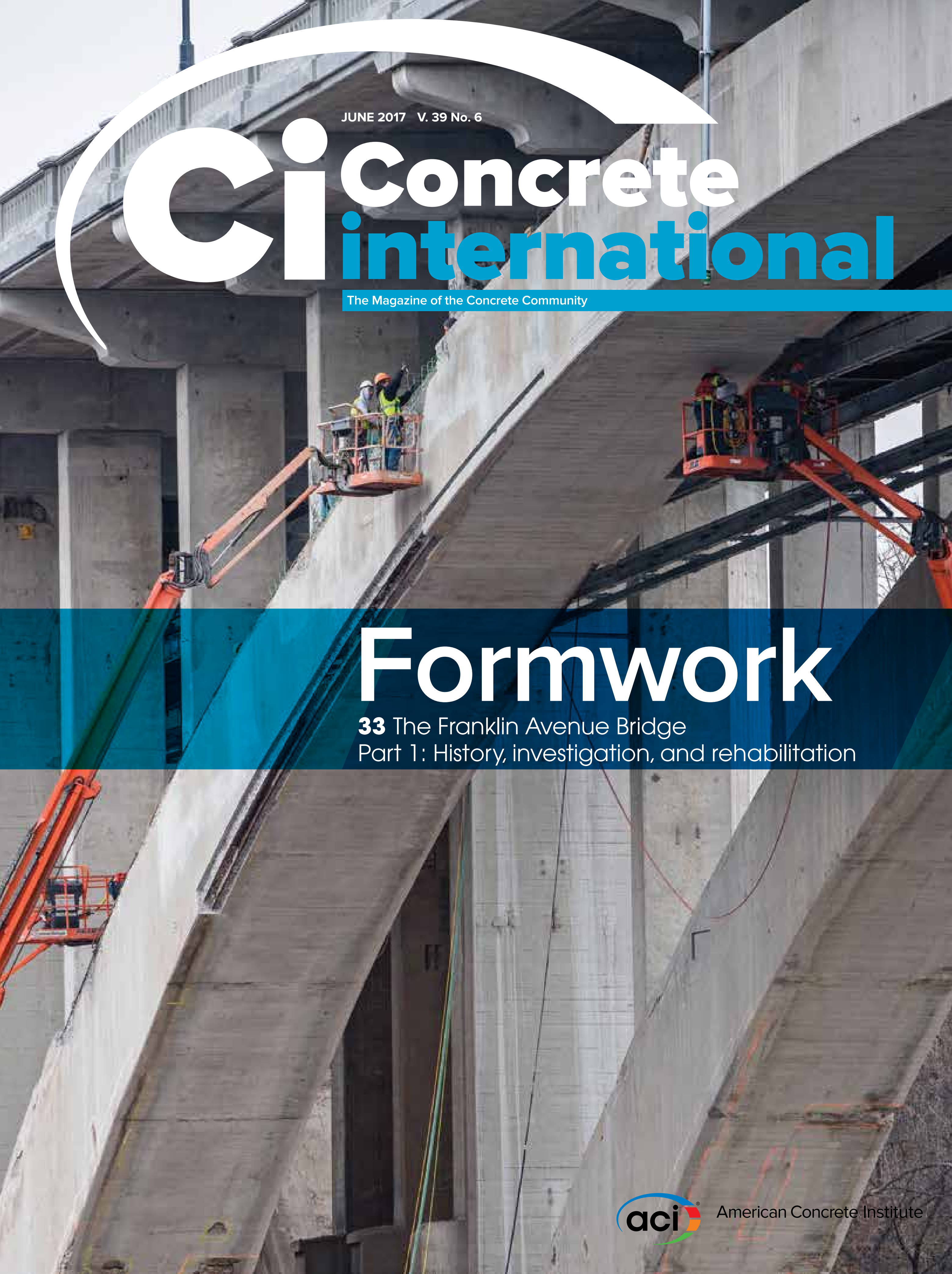

JUNE 2017 V. 39 No. 6

Ci Concrete international

The Magazine of the Concrete Community

Formwork

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Part 1: History, investigation, and rehabilitation

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The Franklin Avenue Bridge

Part 1: History, investigation, and rehabilitation

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This is the first of a two-part series on the restoration of the Franklin Avenue Bridge in Minneapolis, MN. Part 1 focuses on the rehabilitation aspects and Part 2 will focus on the structural analysis and the accelerated bridge construction methods used to replace the deck. The engineering firms Wiss, Janney, Elstner Associates, Inc. (WJE), which led investigation and rehabilitation design, and HNTB Corporation, which led structural analysis and new deck design, worked together with Hennepin County on the project and on these articles.

History and Description

Open-spandrel, concrete arch bridges were common designs in the United States during the early 1900s. The long spans and tall profiles characteristic of this bridge type are well suited to a topography of deep ravines and river valleys.

One of these arch bridges, the Franklin Avenue Bridge, was constructed from 1919 to 1923. It is located near Minneapolis and St. Paul, MN, crossing over the Mississippi River Valley with five elegant arch spans. The bridge was designed by Frederick Cappelen, assisted by Kristoffer Oustad. Both were innovative and influential Norwegian-American engineers working in the Minneapolis city engineer's office, and they also contributed to the design of several other concrete arch bridges in the area, including the Third Avenue Bridge constructed in 1914-1918 in Minneapolis. Per historical sources, the style of these bridges was intended to convey a sense of permanence and monumental beauty in their scenic surroundings.[1,2] Officially the F.W. Cappelen Memorial Bridge (named in honor of the designer, who died during construction), the Franklin Avenue Bridge was listed in the National Register of Historic Places in 1978.

The main arch of the Franklin Avenue Bridge spans 400 ft (122 m) and rises 88 ft (26.8 m) above its spring line. Flanking the center span are two 199 ft (60.7 m) side spans and two 55 ft (16.8 m) end spans. Each of the five spans consists of two 12 ft (3.7 m) wide arch ribs spaced 37 ft (11.3 m) apart. The arch ribs are widely spaced because the new bridge was constructed around an existing five-span steel bridge that was kept in place for transport of construction materials and to serve as a pedestrian crossing during the construction (Fig. 1).[1]

The bridge was constructed using the Melan system, which was patented in 1892 by Joseph Melan, an Austrian bridge engineer. Melan wrote, "the essence of the…construction consists…of the combination of iron arch ribs and a concrete vault [arch], with the latter being properly reinforced. This reinforcement is especially important when a non-uniformly distributed load…would cause tensile stresses…. The arch ribs enable a simple fixing of the shuttering timbers [formwork] for casting the concrete vault, so a special scaffolding is totally unnecessary, even for longer spans."[3]

The arch ribs for the Franklin Avenue Bridge were reinforced with steel trusses composed of double-angle chords connected with riveted steel gusset plates and diagonal cross braces. The steel trusses were erected between the piers and then the arch rib concrete was cast around the trusses. The arch geometry provided high load-carrying capacity for the steel trusses and the finished concrete arches, and the prefabricated steel trusses facilitated rapid construction.

For many bridges, the Melan truss construction technique saved cost and time by eliminating the need for wooden falsework—especially for structures with unfavorable falsework support conditions.[3] In the case of the Franklin

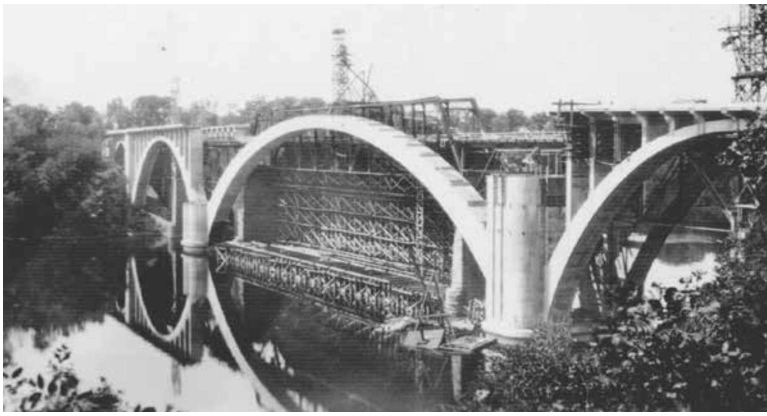

Fig. 1: Franklin Avenue Bridge during construction, circa 1923 (from Reference 1)

Avenue Bridge, however, wooden falsework was still used (Fig. 1), apparently because the "shallow stream and firm subsoil presented highly favorable support conditions."[1] The designers reportedly selected the arch geometry and proportions such that no extreme fiber tension occurred under any loading conditions. As such, the steel truss reinforcement was "designed by judgment, to give a good inter-bonding of all parts of the rib"[1] and "for the general purpose of toughening the concrete rather than for specific tensile service."[1]

The steel trusses were shop fabricated in sections, and the concrete was batched on site at a mixing plant erected near the west end of the bridge. The concrete mixture incorporated gap-graded local aggregates, including a 2-1/2 in. (64 mm) top size, angular crushed basalt coarse aggregate, and a fine-grained largely siliceous sand. With this coarse aggregate type and gap grading, a relatively high water-cement ratio (*w/c*) of about 0.50 (based on petrographic analysis) was required to facilitate placement. The original concrete was not air-entrained, as air-entrainment technology was not developed until the 1930s.

The arch ribs frame into massive reinforced concrete piers—two in the river and two on the river banks—and similarly massive concrete abutments. The lower portions of the piers are solid concrete and the upper portions are hollow pier walls. The bases of the river piers are highlighted by protruding horizontal concrete bands (nosings) just above the water line. The round nosings enhance the aesthetics of the piers and protect the piers from winter ice flows.

The bridge was originally designed for two vehicular lanes plus streetcars and was surfaced with wooden planks on a sand bed over the concrete deck. In 1940, a water main was suspended beneath the bridge deck, the streetcar tracks were removed, and two vehicular lanes were added for a total of four lanes. In 1970, due to advanced deterioration of the superstructure during its first 45 years of service, the entire bridge deck, cap beams, and spandrel columns were replaced. Localized concrete repairs were performed on the arch ribs, piers and abutments, and cracks along the tops of the arch ribs were repaired. The new deck was thicker and 6 ft (1.8 m) wider than the original deck, and the new spandrel columns were spaced twice as far apart as the original columns. Four of the 15 expansion joints in the then-new deck were located above the faces of the river bank piers, two were located above the faces of the abutments, and the balance were centered over cap beams. In 1984, the 1970 deck was scarified and overlaid with a low-slump concrete overlay, and the expansion joints were replaced. Traffic lanes were reduced from four to two and bicycle lanes were added in 2005.

Investigation

The rehabilitation of the Franklin Avenue Bridge was firmly rooted in a comprehensive investigation of the condition, performance, and historical importance of the structure. This initial study was commissioned by Hennepin County in 2007 as a responsible first step toward the rehabilitation. This is consistent with the requirements of National Park Service Preservation Brief 15 - Preservation of Historic Concrete,[4] which recommends a thorough condition assessment be performed to determine concrete condition and identify characteristics and mechanisms of deterioration. A follow-up assessment was conducted in 2013 during the repair design phase to refine the repair approach and update repair quantity estimates.

The scope of both condition assessments consisted of an overall visual examination of the bridge and subsequent detailed surveys and testing at representative study areas. Field testing in the representative study areas included delamination surveys, reinforcing bar cover surveys with ground-penetrating radar (GPR), corrosion potential (half-cell) surveys to assess risk of corrosion of embedded steel, concrete resistivity testing to assess compatibility with cathodic protection repair methods, concrete carbonation depth measurements, ultrasonic thickness testing to assess corrosion-related section loss of arch rib truss steel reinforcement, and adhesion (pulloff) testing to quantify bond of existing surface treatments (Fig. 2). Additionally, concrete cores and segments of reinforcing bars and Melan truss steel were removed for laboratory examination and testing. Laboratory testing included chloride ion profile analysis,

Fig. 2: Bridge investigation activities in 2007: (a) inspection, field testing, and material sample collection; and (b) half-cell corrosion potential measurements

Table 1:
Findings of testing of historic concrete

Property	Range of observations	Interpretation
Reinforcing bars		
Tensile strength (yield)	Square bars: 44.5 to 45.0 ksi Round bars: 67.9 ksi	Square bars are intermediate grade* (min. yield of 40 ksi); round bars are hard grade* (min. yield of 50 ksi)
Cover depth	Arch ribs: 2 to 10 in. Piers: 0 to 11 in. Abutments: 2-3/8 to 4-1/2 in.	See interpretation relative to carbonation depth and chloride concentrations
Concrete		
Carbonation depth	Arch ribs: 1/8 to 3/4 in. Piers: 3/8 to 5/8 in. Abutments: 1/4 to 1-1/8 in.	Reinforcement with shallow cover in piers susceptible to carbonation-related corrosion in the presence of adequate moisture. Reinforcement in arch ribs and abutments not susceptible because carbonation depth less than clear cover
Petrographic examination	*w/c* of approximately 0.50 or less (variable); non-air-entrained. Freezing-and-thawing damage: arches to depth of 11-1/2 in.; land piers to depth of 8-1/4 in; at water line in river piers to depth of 22 in. No other durability limiting mechanisms (for example, alkali-silica reaction) identified	Wherever concrete becomes critically saturated, additional freezing-and-thawing distress is expected. Otherwise, no inherent material deficiency identified
Chloride concentrations (acid soluble) in range of observed bar depths, % by weight of concrete	Arch ribs: 0.000 to 0.358 Piers: 0.020 to 0.409 Abutments: 0.082 to 0.367	Reinforcement in arch ribs, piers, and abutments susceptible to chloride-related corrosion. Additionally, water-soluble chloride test results indicate background chloride, such as admixed chloride, not present
Compressive strength†	Arch ribs: 5870 to 9850 psi Piers: 6870 psi Abutments: 8250 psi	Strength of no more than 5000 psi was used for analyses; measured strengths did not indicate durability concerns

*Steel grades as defined in ASTM A15-14, "Specification for Billet-Steel Bars for Concrete Reinforcement" (grade definitions applicable through the 1960s)

†Correction factors given in ASTM C42, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," applied (despite strength greater than 6000 psi)

Note: 1 in. = 25.4 mm; 1 psi = 0.007 MPa

compressive strength testing, and petrographic examination of the concrete, as well as chemistry and tensile testing of the steel reinforcement. Some of the key findings of the investigation of the historic concrete elements (arch ribs, piers, and abutments) are summarized in Table 1.

The assessments identified widespread concrete deterioration in the original concrete piers, abutments, and arch ribs. The nature of this deterioration consisted of spalls and delaminations; surface erosion, disintegration, and sub-planar cracking to a depth of as much as 11 in. (280 mm); and longitudinal cracking along the top and bottom surfaces of the arch ribs (generally aligned with the legs of the embedded steel truss angles). The causes of the deterioration were primarily chloride-related corrosion of the embedded reinforcement and long-term exposure to moisture and freezing-and-thawing cycles. Some carbonation-related corrosion was also present where bars had shallow cover. In some cases, the corrosion and freezing-and-thawing mechanisms had combined to produce the observed distress—cracks from freezing-and-thawing action permitted ingress of chlorides, which promoted corrosion, which further opened cracks allowing more moisture ingress, which encouraged still more freezing-and-thawing damage.

The distribution of the concrete deterioration was largely determined by the exposure to chloride-laden water from deicing salts leaking through expansion joints or by poor drainage of water away from the concrete surfaces. The river bank piers and both abutments exhibited widespread damage associated with leakage from the deck joints above them (Fig. 3(a)). Distress, particularly corrosion-related deterioration, was most severe near the top of these elements. Localized distress was also observed over the full height, especially near corners where direct moisture runoff and two-sided exposure to moisture and freezing-and-thawing cycling occurred.

The distress on the river piers consisted of isolated areas of corrosion-related deterioration, mostly due to carbonation and shallow cover over the steel reinforcement (Fig. 3(b)). Severe freezing-and-thawing damage was present at the waterline due to deep saturation and freezing-and-thawing cycling of the non-air-entrained concrete, and spalling was likely encouraged by occasional impact from river ice (Fig. 3(c)).

The arch ribs exhibited a range of distress conditions (Fig. 4). The most widespread was longitudinal cracking due to corrosion of the embedded steel angles that make up the Melan trusses (Fig. 4(b)). The corrosion was most severe below expansion joints and at the corners of the ribs, where it led to wide cracks, delaminations, and spalls. Isolated delaminations were also present on the faces of the arch ribs, and longitudinal cracks were often present in the top and bottom surfaces along the embedded truss members (Fig. 5). Freezing-and-thawing distress was also present in localized areas where water exposure was most severe. Previous concrete repairs of various vintages and types were present.

Away from expansion joints, the cap beams and spandrel columns, which date from the 1970 rehabilitation, were in very good condition. However, the deck soffit, cap beams, and some of the spandrel columns located below expansion joints exhibited widespread and sometimes advanced deterioration

Relevant Deterioration Mechanisms

Freezing-and-thawing damage—Freezing-and-thawing damage occurs in non-air-entrained concrete when the concrete is critically saturated with water and is subjected to repeated freezing-and-thawing cycles. The damage first manifests in internal microcracking, progresses to paste deterioration or map cracking visible on the surface, and culminates in disintegration of the concrete from the surface inward. Modern air entrainment avoids this damage mechanism by providing voids in the concrete into which internal water can expand when it freezes.

Chloride-induced corrosion damage—When chloride ions accumulate to a sufficient concentration around reinforcing bars, corrosion can initiate when moisture and oxygen are available. At the Franklin Avenue Bridge, the chlorides are from deicing salts. Chlorides can also result from admixtures used to accelerate strength gain during initial construction, exposure to seawater or seawater spray, and contaminated aggregates such as marine beach sand. Because the volume of corrosion by-products is greater than the volume of the base metal, corrosion generates expansive forces within the concrete that lead to cracks, delaminations, and spalls.

Carbonation-induced corrosion damage—Carbonation is a natural chemical process in which carbon dioxide in the air reacts with the cement paste of concrete and lowers the concrete's pH. When the carbonation front reaches the level of the steel reinforcement, the normally passive film on the bars starts to break down, and corrosion, much like atmospheric corrosion, can initiate. Carbonation-induced corrosion manifests in concrete distress in a similar manner to chloride-induced corrosion.

Ring anode effect—The ring anode effect is the phenomenon in which corrosion of the steel reinforcement around the perimeter of a new concrete repair is accelerated after the repair is installed, due to locally large differences between the noncorrosive environment in the new concrete and the relatively higher-corrosive environment in the original concrete. The effect is most prominent when the original concrete is chloride contaminated or carbonated.

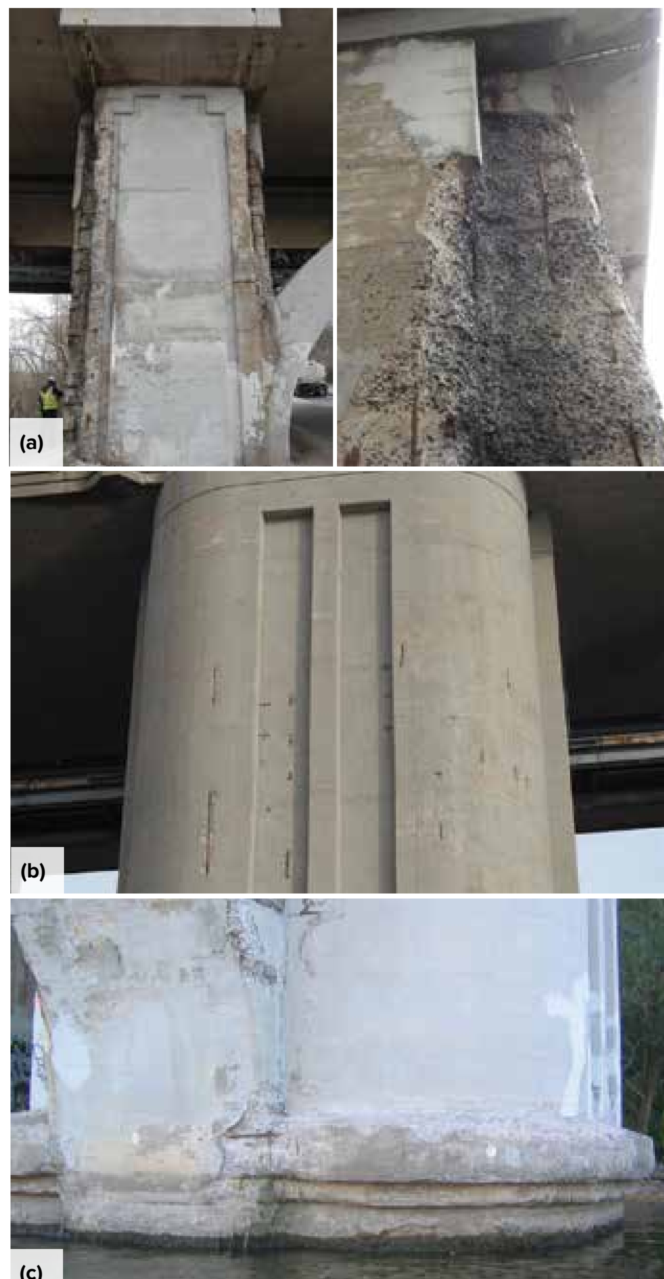

Fig. 3: Examples of observed concrete deterioration: (a) severe spalling of river bank pier located below deck expansion joints; (b) isolated corrosion-induced spalls at shallow cover conditions on river pier; and (c) severe freezing-and-thawing distress near waterline on river pier

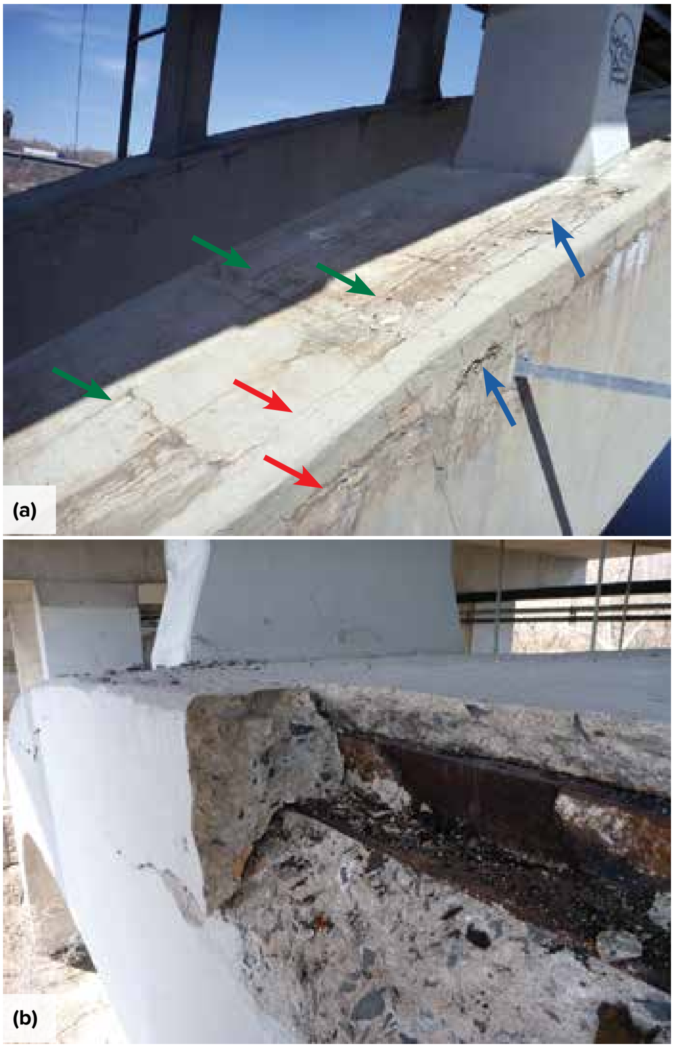

Fig. 4: Examples of arch rib distress conditions: (a) red arrows indicate cracking and delamination at corners, green arrows indicate cracking along interior lines of steel truss reinforcement, and blue arrows indicate freezing-and-thawing damage; and (b) loss of corner concrete caused by corrosion of steel angle

in the form of delamination, spalling, and corrosion of embedded reinforcement due to chloride contamination of the concrete (Fig. 6). High levels of chloride in the top of the deck (0.205% or more acid-soluble by weight of concrete at the bar depth), below the overlay, had begun to produce corrosion of the top steel reinforcement that resulted in associated concrete delaminations.

Another feature of the bridge significant to the rehabilitation approach was the presence of previous surface treatments, including one or a combination of the following: polymer-modified cementitious parge, cementitious mortar wash, and multiple layers of paint. The parge was applied over widespread areas in the 1970 rehabilitation. It appears the mortar wash was brushed or broomed to blend the repairs with the surrounding concrete surfaces. The paint was most prevalent on the abutments, piers, and arch ribs of end spans, as it was used primarily to cover graffiti.

Preliminary structural analysis was conducted as part of the 2007 investigation to assess the load-carrying capacities of the bridge elements, study whether expansion joints could be eliminated, and evaluate the structural feasibility of various rehabilitation schemes. Because leakage of salt-laden water through joints was a primary cause of the deterioration, reducing the number of joints would significantly improve performance. Structural analysis that included consideration of volume change effects showed that, while significant deck widening was not feasible, reducing the number of joints, perhaps by half, was possible. More refined analysis and a final load-rating of all elements of the bridge was conducted by HNTB at the beginning of the rehabilitation design in 2013 (details will be in Part 2 of this series). In the end, the new deck design reduced the number of deck expansion joints from 15 to only six, with none located above a pier.

Rehabilitation Design

The 2007 investigation showed that the bridge was generally structurally sound and competent to support vehicle

Fig. 5: Longitudinal cracking along interior lines of steel truss reinforcement

Fig. 6: Reinforcing steel corrosion and spalling due to leakage of expansion joint over cap beam

loading. However, deterioration in many of the bridge elements, particularly those located near expansion joints, prompted the following rehabilitation recommendations:

- Development of plans for replacement of the deteriorated portions of the deck or the entire deck;
- Repair or replacement of the deck soffit and deck framing near expansion joints;
- Elimination of deck expansion joints where possible; and
- Restoration of the original concrete piers, abutments, and arch ribs, as these embody the historic character of the bridge and were the only remaining elements of the original structure. Targeted corrosion mitigation measures were to be considered to improve future performance.

Based on these general recommendations, a range of preliminary rehabilitation strategies was developed, including specific maintenance efforts at defined intervals. To provide a basis for selection, the service life of the structure was estimated under each alternative. This information, in combination with estimates of initial and future costs, were then used to perform a life-cycle cost analysis for each alternative.

With the 2007 investigation as a foundation, the rehabilitation design effort was initiated in 2013. At that time, the rehabilitation alternatives were further refined based on an updated structural condition assessment, refined structural analysis and load rating, traffic operations, and the effect on historic properties. The final recommended rehabilitation alternative included:

- Complete removal and reconstruction of the deck and cap beams, with two central vehicle lanes flanked by barrier-separated pedestrian and bicyclists lanes along the bridge length, and a wider four-lane roadway on the east end to transition into a challenging five-legged intersection just off the bridge;
- Rehabilitation of the original historic concrete (piers, abutments, and arches) and the spandrel columns, using historically sensitive, durable concrete repair methods supplemented with targeted corrosion mitigation along the arch rib corners; and
- Restoration of historic features, including historic cap beams with scrolled ends, exterior ornamental barriers, light fixtures, deck fascia entablature, and recreated observation bays over the river piers.

Rehabilitation Details

The details of the concrete repair design were developed and communicated through carefully prepared specifications and drawings to achieve historic sensitivity and high-quality, durable repairs. The guiding principle behind the repair design was to detail the repairs in ways that would address the root deterioration mechanisms identified in this structure.

Based on the hands-on inspection of the bridge, concrete surface repairs were specified for all locations where delaminations and spalls were present, and repair details were developed for each typical location. Unique details were provided to address the severe corrosion-related distress at the arch rib corners, longitudinal cracking at the tops and bottoms of the arch ribs, and areas where freezing-and-thawing damage was particularly deep (as shown in Fig. 7). The specifications demanded high-quality concrete repair techniques, including perimeter sawcutting, removal to sound concrete using light chipping hammers, substrate preparation via sandblasting, sandblast cleaning and coating of exposed reinforcement, and anchorage using epoxy-grouted bars. The concrete repair specifications were designed to allow the contractor to choose either form-and-pour, form-and-pump, or shotcrete methods with either prepackaged or ready mixed concrete for each type of

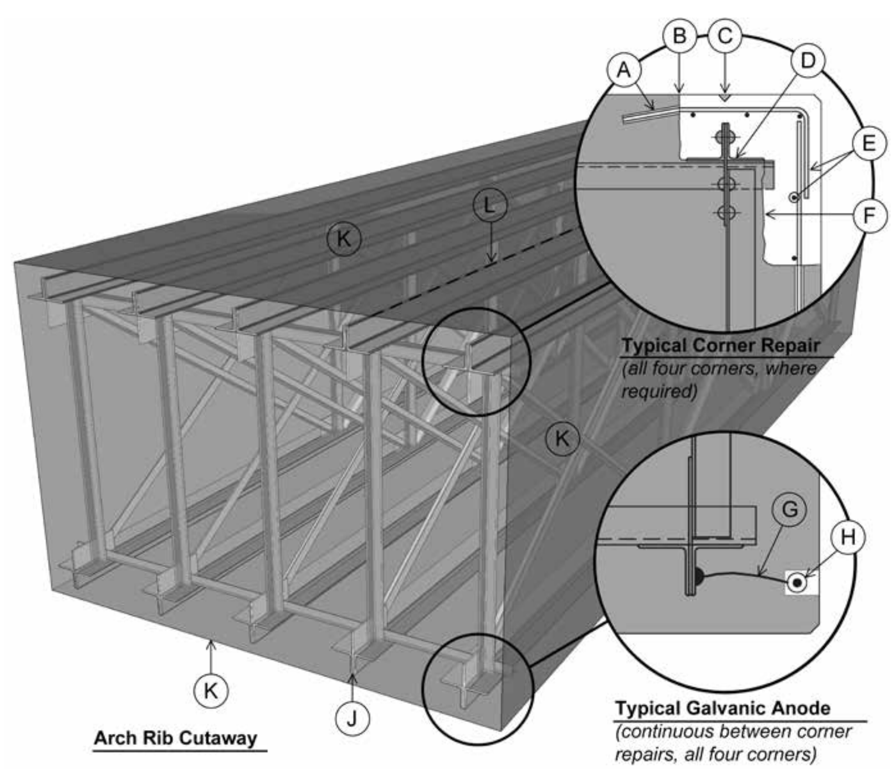

Fig. 7: Cutaway through arch rib showing Melan truss reinforcement and typical concrete repairs: (A) epoxy-grouted dowels for anchorage; (B) saw cuts at repair perimeter; (C) tooled joint with sealant; (D) clean and coat existing steel; (E) crack control reinforcement; (F) properly prepared sound concrete substrate; (G) intermittent slots and wire connections to existing steel; (H) continuous zinc anode in sawcut slot; (J) Melan truss reinforcement; (K) typical surface repairs (where required); (L) rout and seal crack repairs (where required)

Fig. 8: Dry-method shotcrete repair on arch rib corner

Fig. 9: Form-board finish on new repairs (red arrows) adjacent to original surface texture (blue arrow)

repair. The contractor chose to use predominantly prepackaged dry-mix shotcrete for most repairs (Fig. 8). In portions of the bridge most visible to the public, the western arch span and pier, the new concrete repairs were specified with a form-board finish to match the original surface texture (Fig. 9). The new surface coating and concrete repair materials were colored to a single light buff color that was selected by the historian on site to be within the range of the original concrete color.

For historic structures, mockups and field trials are essential to allow the owner and design team to evaluate the contractor's materials and methods from both a technical and aesthetic standpoint before full-scale repairs proceed on the structure. Aspects of the work for which mockups were deemed necessary included concrete surface repairs, cathodic protection galvanic anode installations, crack repairs, and concrete coating. For each, a three-step process was implemented—shop and field samples (Fig. 10), and trial repairs.

Shop samples are a range of small samples that are prepared off site to select materials and colors and then transported to the jobsite for comparison to existing surfaces. Field samples are larger samples prepared on the jobsite in forms that are situated in actual repair orientations, but are located off the structure. These samples verify selected materials and techniques and, when approved, serve as a portable standard for color, texture, and workmanship throughout the project.

Trial repairs are the final step in which the full repair process is implemented by the contractor on the structure at small, predetermined locations. Trial repairs must be approved before full-scale production work can proceed. This step-wise process provided confidence to all parties at the start of the project that the repairs would be implemented in a manner that was historically appropriate and in conformance with the project specifications.

One of the greatest challenges in the rehabilitation design was that the historic concrete that appeared sound and in good condition was still vulnerable to rapid future deterioration—corrosion-induced damage (because the concrete is chloride-contaminated) and freezing-and-thawing damage (because the

Fig. 10: Shop samples showing range of colors and field samples showing form-board finish

concrete is non-air-entrained). However, without sufficient moisture, corrosion and freezing-and-thawing damage will not occur. While the deck and joint replacement was the important first step in addressing the source of the moisture, this would not eliminate moisture exposure to the historic concrete. Therefore, concrete protection measures were explored, including penetrating sealers (for example, silane), water-resistant film-forming coatings, active and passive cathodic protection systems, and migrating corrosion inhibitors.[5] Ultimately, an approach combining a film-forming coating and passive cathodic protection at targeted locations was selected.

According to NPS Preservation Brief 15,[4] film-forming coatings are often inappropriate for use on a historic structure, unless the structure was coated historically. In the case of the Franklin Avenue Bridge, previous surface treatments had been applied, including cementitious wash, cementitious parge, and paint. Furthermore, absent bulk protection of the concrete surface from moisture, future deterioration, and loss of historic fabric would be likely. After thorough discussions between historians and technical experts, it was agreed that a high-performance, film-forming, water-resistant coating would be applied to all historic concrete surfaces. Relatively

(a)

(b)

Fig. 11: Galvanic strip anodes in slots cut along arch rib corners: (a) general view; and (b) close-up

thin acrylic-based coating products were selected so as not to mask the original form-board lines. This type of coating is more vapor permeable than most other concrete coatings. It also can be removed, which is important for historic structures, and it enhances the appearance of the concrete by masking multiple generations of different colored patches.

The corrosion of the steel angles of the Melan trusses was most severe at the corners of the arch ribs. Although significant section loss had not occurred in the steel angles, corrosion buildup had caused concrete damage (cracks, delaminations, or spalls). At these locations, the distressed concrete was excavated, the steel was sandblast-cleaned and coated with a zinc-rich primer and epoxy topcoat, and concrete surface repairs were installed using epoxy-grouted dowels for anchorage and reinforcing bars for crack control (Fig. 7). To avoid the “ring anode” effect and future damage resulting from hidden but ongoing corrosion of the steel angles, passive cathodic protection, consisting of zinc anode strips embedded in narrow slots cut into the concrete near the four corners of the arches, was applied to the remainder of the arches. The anodes were limited to the spaces of original concrete between the concrete repairs and were positioned along the tops and sides of the arch ribs to avoid disruption of the form-board lines on the more visible bottom surfaces (Fig. 7 and 11).

Fig. 12: Completed concrete jacket around base of river pier (after removal of coffer dam)

The design life of the anodes is 20 years, although performance of the anodes will be influenced by many factors, including the presence of chlorides remaining in the existing concrete and future moisture exposure. Commissioning was performed at two test stations to verify satisfactory performance of the anodes.

Freezing-and-thawing damage at the bases of the river piers was as great as 22 in. (560 mm) deep, based on coring and petrographic examination. Repair was accomplished by constructing a circumferentially reinforced concrete jacket around the base of each pier. Coffer dams were installed, the outer 12 in. (305 mm) of deteriorated concrete was removed to well below the water line, epoxy-grouted bars were installed in holes drilled into sound underlying concrete, and the reinforced jacket was constructed flush with the original concrete profile (Fig. 12). While the freezing-and-thawing damage to the piers at these locations extends deeper than 12 in., the confinement to the interior concrete by the new reinforced jackets will ensure structural performance, and the jacket concrete will limit moisture intrusion and forestall continued deterioration.

The bridge restoration efforts began in 2015, with rehabilitation of the historic concrete in the end spans and preparation for deck replacement. For access, the contractor initially used a suspended shoring system and then transitioned to using personnel lifts based on the ground or barges in the river. In 2016, while rehabilitation of the historic concrete continued, the deck and cap beams were replaced under a full roadway closure. Using precast elements and accelerated bridge construction methods, the deck replacement required only 116 days (details in Part 2 of the series). Consideration was given to the timing of the concrete repairs on the arch ribs relative to active traffic on the deck and unloading and reloading of the arch ribs during deck replacement. Limitations were placed on the amount of concrete repair that could be performed in different zones of the arches while traffic was on the bridge. The remaining restoration efforts, including surface repairs in the center span and coating system installation over the entire bridge, will be completed in 2017.

Critical Factors for Success

A thorough, early investigation—The project illustrates how an early condition assessment and feasibility study of alternatives sets the stage for a successful rehabilitation. Realizing the complexities at hand, Hennepin County astutely commissioned a comprehensive investigation including sufficient inspection, field testing, and laboratory analysis of collected samples to accurately identify the range of deterioration mechanisms present in the structure. Using that information, rehabilitation alternatives targeted to address the root causes of the deterioration could be identified, and realistic service-life predictions and life-cycle cost comparisons could be prepared for the different alternatives. With that information, informed planning decisions could be made.

Adherence to historic preservation principles while protecting vulnerable historic fabric—A film-forming coating is usually not ideal from a historic preservation perspective. However, in this case, the existing historic concrete fabric was found to be extremely vulnerable to future deterioration and eventual loss if moisture continued to penetrate. Further, various surface treatments had been applied to the concrete in the past. After a thorough vetting process involving historic agencies and technical experts, the decision was made to apply a high-performance, water-resistant, opaque coating to extend the life of the historic concrete. For historic sensitivity, the coating installation was selected to be reversible (removable), sufficiently thin to allow the original form-board lines to show through, and color-matched to the original concrete.

Efficient use of cathodic protection—Cathodic protection systems are designed to enhance concrete durability by slowing corrosion of the embedded steel. Chloride-contaminated concrete, such as the remaining historic concrete on the subject bridge, is particularly benefited. However, cathodic protection on large bridge structures can be very expensive and can alter historic appearance. In this case, cathodic protection was made more affordable by limiting its use to the locations where it would provide the most benefit—only at the arch rib corners and only in the regions between the concrete repairs. Historic impact was minimized by detailing the cathodic protection in discrete slots sized and positioned to be least visible from the ground (surface treatments such as arc-sprayed zinc would have obscured original form-board lines).

Step-by-step mockups—Given the cultural value and irreplaceable nature of historic concrete fabric, trial and error on the structure is not appropriate in the repair of a historic structure. Validating and finalizing the color, texture, and quality of the repair methods off the bridge, and then at a single location on the bridge, is critical before full-scale implementation. A carefully prescribed mockup process, such as the three-step process for this project (shop samples, field

Finishing form-board lines in wet shotcrete

Form-Board Finish on Shotcrete

To maintain the historical appearance of the Franklin Avenue Bridge, the contractor recreated the form-board finishes by working a float or 4 ft (1.2 m) long screed board into the fresh shotcrete. This action created raised lines that simulated the board-formed fins in the adjacent, original concrete. The plane of the original concrete was maintained by stringing piano wire tightly across the repair areas and using it as a guide for the finishing tools.

To avoid degrading the durability of the repair, timing of the finishing operation was critical. Workers had to avoid working the surface too early, which could tear or debond the repair material. They also had to avoid working the surface after setting, which could lead the worker to "water" the surface for lubrication, thus creating a low-strength surface layer. Efforts were taken to minimize shotcrete overspray. Where it did occur, overspray was promptly removed to avoid obscuring the original surface texture. Finally, once initial curing was completed, the surface was abraded by light sandblasting to match the somewhat weathered texture of the original surfaces.

The manual finishing method used for replicating form-board finishes on the Franklin Avenue Bridge offered the contractor the flexibility needed to finish the variety of sizes, surfaces, and locations of shotcrete repairs on this project. On projects that have larger surfaces that require replication of a form-board finish, alternative approaches may offer greater efficiency. For the rehabilitation of Union Depot in St. Paul, MN, for example, several prefabricated custom forms, comprised of 3/4 in. (20 mm) thick dimension lumber attached to a plywood backing, were pressed into overhead shotcrete repairs just after initial set. It is also possible to use altered finishing tools, such as trowels modified with notches at the correct spacing and depth necessary to replicate existing fins. Further, if a wood grain finish is required, boards can be lightly pressed into the surfaces between the lines.

samples, and trial repairs), both respects and protects the historic integrity.

Success through collaboration—Rehabilitation of a large, complex historic structure of this nature relies on the expertise of many different firms and many different individuals. Continual collaboration and mutual respect among these experts are keys for success. In this case, historic concrete assessment and rehabilitation experts, bridge analysis and design experts, historic preservation agencies, county engineers, technicians and inspectors, community stakeholders, and a contractor experienced in historic concrete repair all contributed their individual expertise and worked together to achieve a tremendous outcome.

Take a trip to Minneapolis and experience for yourself this renewed landmark bridge. We think you will agree that its restoration fulfills once again the original designers' intent—to service the traveling public while conveying a sense of beauty, monumentalism, and permanence in concert with its picturesque surroundings.

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Received and reviewed under Institute publication policies.

Note: Additional information on the ASTM standards discussed in this article can be found at **www.astm.org**.

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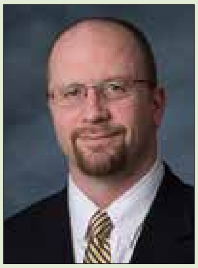

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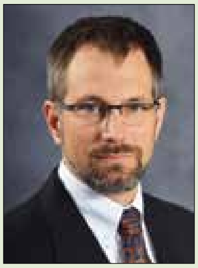

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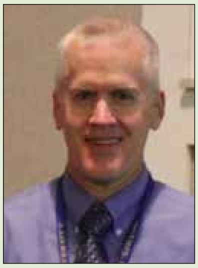

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