STEEL BRIDGES 2015

NATIONAL STEEL BRIDGE ALLIANCE
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Reprinted from **MODERN STEEL CONSTRUCTION**

Welcome to Steel Bridges 2015!

This publication collects all the bridge related articles that were published in Modern Steel Construction in 2016. Topics ranged from informative and academic to fun and enlightening. As always, we are proud of the amount and variety of information we provided.

These articles would not have been possible without the efforts of the authors, most of whom volunteered their time. Their willingness to share their experience and expertise benefits the entire bridge community. As we look forward to 2016, if you are aware of a project or topic that should be featured, don't hesitate to contact us. There are great stories out there and we want to share them.

It's hard to believe 2015 is already behind us but I hope you share in my enthusiasm for all 2016 holds. We look forward to working with you in the coming year.

Bill McEleney
NSBA Managing Director

The National Steel Bridge Alliance is dedicated to advancing state-of-the-art steel bridge design and construction. The NSBA stands united with industry businesses and agencies interested in the development, promotion, and construction of cost-effective steel bridges.

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312.670.2400 www.steelbridges.org

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bridge crossings

ARE YOU SURE THAT'S FRACTURE CRITICAL?

BY ROBERT J. CONNOR, PH.D., KARL FRANK, P.E., PH.D., BILL MCELENEY AND JOHN YADLOSKY. P.E.

Understanding which steel bridge elements are fracture critical members will provide the required protection while saving on in-service inspection.

ONE OF THE MOST NOTEWORTHY BRIDGE FAILURES in the United States occurred in 1967, when the Point Pleasant Bridge over the Ohio River (also known as the Silver Bridge) collapsed, resulting in 46 deaths.

The collapse was due to brittle fracture of one of the eyebars that formed the suspension system of the bridge. The subsequent failure investigation revealed that the fracture was due to brittle propagation of a tiny crack in the eyebar. Because the fracture toughness of the eyebar was extremely low, a relatively small crack led to a brittle fracture of the eyebar, which in turn led to the collapse of the bridge.

This collapse was the catalyst for many changes in material specifications, design, fabrication and shop inspection of steel bridges. These requirements are codified in the AASHTO *Bridge Design Specifications* and the AASHTO/AWS D1.5 *Bridge Welding Code* (AWS) and are applied to tension members whose fracture could lead to bridge collapse. (Another bridge incident—the failure of a pin-and-hanger assembly, which triggered the collapse of one span of the Mianus River Bridge in 1983—served as the impetus for enhanced field inspection requirements for these same members.)

The Three-Legged Stool

Today, a total fracture control plan (FCP) is often illustrated as a three-legged stool, where each leg is made up of a part of the plan, as illustrated in Figure 1. (Since the introduction of the FCP, the authors are not aware of any failures in fracture critical members fabricated to the FCP. Hence, the FCP concept appears to be serving its intended purpose.)

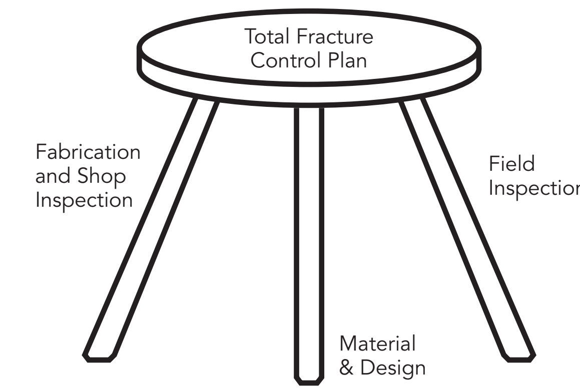
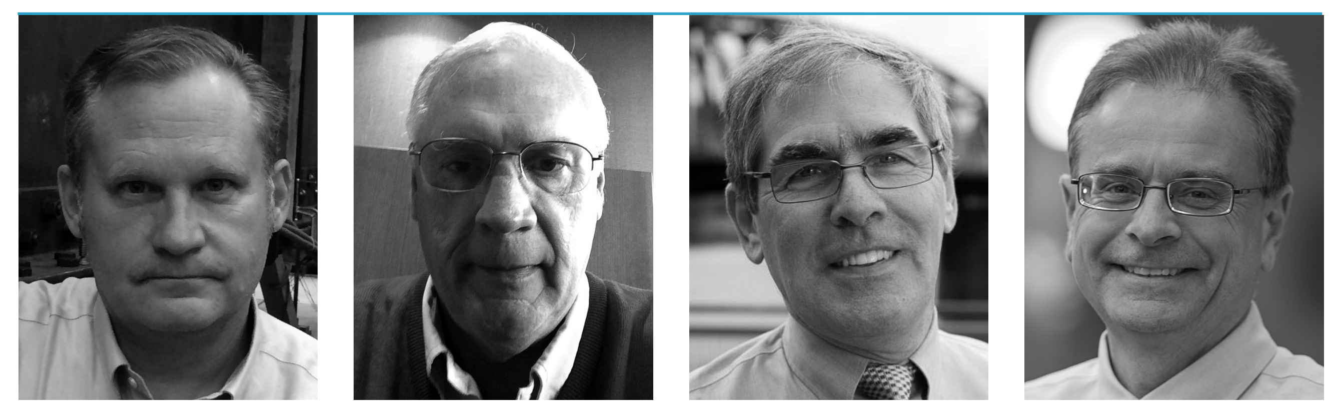


Figure 1 – The three "legs" of a total fracture control plan for bridges.

It is essential to understand that the FCP was specifically developed in response to failures (i.e., brittle fractures) in non-redundant tension members that occurred in the 1970s. Such members, which may be either entirely (e.g., a truss member) or partially (e.g., a flexural member) in tension became known as fracture critical members (FCMs). An FCM is defined by the *Code of Federal Regulations* (23CFR650 – Bridges, Structures and Hydraulics) as "a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse."



Robert Connor (rconnor@purdue.edu) is an associate professor of civil engineering and director of the S-BRITE Center at Purdue University. **Karl Frank** (karl.frank@hirschfeld.com) is chief engineer with Hirschfeld Industries. **Bill McEleney** (mceleney@aisc.org) is managing director of NSBA. **John Yadlosky** (john.yadlosky@hdrinc.com) is HDR's bridges and structures operations director.

Prior to the FCP, the design of tension members was based solely upon prevention of yielding; there were minimal requirements on steel toughness (i.e., no Charpy V Notch toughness requirements) and less stringent fabrication and shop inspection requirements. In fact, there was no AWS bridge welding code in existence. Researchers and engineers alike recognized that control of brittle fracture in non-redundant tension members, or portions of members in tension, was important.

In short, the primary objective of the FCP is to prevent brittle fractures of non-redundant tension members and tension components. The material, fabrication and shop inspection portions of the FCP are intended to minimize the frequency and size of discontinuities that might initiate a crack and also to ensure that materials with greater flaw tolerance are used for these members. Arms-length in-service field inspection is intended to discover fatigue cracks before they become a critical size.

Classifying a Member as an FCM

To be classified as an FCM, two basic yet specific criteria must be met:

1. An FCM must be subjected to net tensile stresses from either axial or bending forces. For example, a member that carries 100 kips in dead load compression but 200 kips in live load tension would satisfy this portion of the definition since the *net* force is tension. It is recognized that for brittle fracture to occur and propagate, tensile forces that exceed any compressive forces *must* be present in the member. As another example, in a simple-span beam only the components of the beam in tension (i.e., bottom flange and portion of the web in tension) would meet this requirement.

2. An FCM must be determined to be non-redundant. While definitions vary slightly, the concern is for members whose fracture would result in collapse of the bridge or a portion of the bridge. A member with an alternate load path—i.e., a redundant member—should not be considered fracture critical. Members such as the lower tension chord of a truss, single or double eyebars or pin and link hangers are typically considered as non-redundant members and identified as FCMs because it is presumed that if the member were to fail in brittle fracture, it could trigger the collapse of the bridge. In the absence of a more rigorous system analysis, this is of course a reasonable assumption. It is these types of members that were on the minds of the individuals who developed the FCP. In contrast, however, the tension flanges of multi-girder bridges are not considered FCMs because the adjacent girders provide a redundant load path and load capacity in the event of a fracture of any given girder.

If either of the above criteria is not met, the member shall not be considered an FCM. That is true of every specification in the United States governing steel bridge design, fabrication and in-service inspection that includes the concept of an FCM.

The responsibility to designate a member or member component as an FCM is incumbent on the design engineer. Once it is determined that the element meets both of the above criteria, the member must be clearly labeled as FCM on the design plans. This is essential as it alerts the fabricator to obtain the proper material and fabricate the member to the FCP. However, in addition to the more stringent material and fabrication requirements, the member will also be subject to more rigorous and costly arms-length in-service inspection every two years for a highway bridge.

Applying an FCP

Interestingly, during the development of the FCP, those who crafted the provisions recognized that engineers, given the choice, will often specify the most conservative option provided in a specification and in this case, potentially require the FCP regardless of member loading, type, etc. simply because it would be perceived to be "safer." To avoid this, the commentary to the FCP in AWS explicitly states that it is not intended to be used for members the engineer simply deems "important." In fact, the commentary goes so far as to state that the FCP is not intended to be used for anything but bridges. For example, see this wording from the commentary:

"The fracture control plan should not be used indiscriminately by the designers as a crutch 'to be safe' and to circumvent good engineering practice. Fracture critical classification is not intended for 'important' welds on non-bridge members or ancillary products; rather it is only intended to be for those members whose failure would be expected to result in a catastrophic collapse of the bridge."

Thus, although a member may be deemed "important," if it does not meet the two criteria cited above the member shall not be classified as an FCM. For example, failure of an end-post of a simple span truss will most likely cause collapse of the span. However, since it is never subjected to tension, it would be incorrect to label it as an FCM simply because it is a critical or "important" member in the bridge. This commentary leaves little to interpretation.

Despite the guidance in the specifications, it has become apparent that some design engineers occasionally incorrectly classify steel members as FCMs. This is likely due to inexperience and lack of familiarity with the spirit and objective of the AASHTO/AWS FCP. Nevertheless, in order to properly identify when a member should be classified as an FCM, it is best to first examine the definitions contained in various specifications (underlines are for emphasis):

From AWS:

➤ AASHTO/AWS D1.5 *Bridge Welding Code*, Article 12.2.2–Definitions

"Fracture critical member (FCM). Fracture critical members or member components are tension members <u>or tension components of bending members</u> (including those subject to reversal of stress), <u>the failure of which would be expected to result in collapse of the bridge</u>. The designation 'FCM' shall mean fracture critical member or member component. <u>Members and components that are not subject to tensile stress under any condition of live load shall not be defined as fracture critical.</u>"

➤ AASHTO/AWS D1.5 *Bridge Welding Code*, Article C12.2.2–Commentary on Definitions

"Tension members or member components whose failure would not cause collapse of the bridge are not fracture critical. Compression members and <u>portions of bending members in compression</u> may be important to the structural integrity of the bridge, but <u>do not come under the provisions of this plan</u>. Compression components do not fail by fatigue crack initiation and extension, but rather by yielding or buckling."

From the American Railway Engineering and Maintenance-of-Way Association (AREMA):

➤ AREMA *Manual for Railway Engineering*, Chapter 15, Article 9.1.14.2a

"Fracture critical members (FCM) are defined as those tension members or tension components of members whose failure would be expected to result in collapse of the bridge or inability of the bridge to perform its design function. The identification of such components must, of necessity, be the responsibility of the bridge designer since virtually all bridges are inherently complex and the categorization of every bridge and every bridge member is impossible. <u>However, to fall within the fracture critical category, the component must be in tension</u>. Further, a fracture critical member may be either a complete bridge member or it may be a part of a bridge member."

➤ AREMA *Manual for Railway Engineering*, Chapter 15, Article 9.1.14.2b

"Members or member components whose failure would not cause the bridge to be unserviceable are not considered fracture critical. <u>Compression members and member components in compression may, in themselves, be critical but do not come under the provisions of this Plan</u>."

As clearly stated in these specifications, compression members or components of members in compression are not to be considered FCM. Both AREMA and AWS use essentially the same definitions and state that compression members "do not" come under the provisions of the FCP. Further, redundant members do not come under the provisions of the FCP. The use of the phrase "do not" also leaves no interpretation and differs from other typical specification type verbiage, such as "should" or "may."

FCM or not?

In the interest of providing guidance, a few typical members found in steel bridges are listed along with basic rationale for either classifying or not classifying the member as an FCM.

Multi-girder bridges and stringers. Bridges with multiple longitudinal members, such as girder bridges with three or more girders or stringer beams of long-span bridges, are examples of members with alternate load paths in the event of a fracture. Their criticality is similar to the bridge deck, where fracture would result in local failure of the deck but not collapse of the bridge. As an example, fatigue cracks were found in late 1970 at cover plate terminations on the Yellow Mill Pond bridge, which carried I-95 in Connecticut. The girders had numerous small cracks and although one girder almost completely fractured, the bridge continued to carry traffic.

While a portion of these members is subjected to tension due to bending, failure of a single stringer or girder would not result in collapse of the bridge or even a part of the roadway. Multiple stringers supported by transverse floor beams are also inherently redundant.

Floor beams. Some engineers have chosen to classify floor beams fracture critical, perhaps in consideration of the support of the roadway. Floor beams should be assessed for FCM status in the same manner as any other bridge member—i.e., is fracturing of a floor beam likely to result in the collapse of the bridge? Regarding roadway support, consider the following:

1. Is the bridge deck composite with the stringers and floor beams? If so, in order for the riding surface to collapse, the entire floor system must suffer a fracture.
2. Are there continuous stringers over the floor beams? Continuous stringers offer an alternate load path for the vehicle load.
3. How are the floor beams framed into the main longitudinal elements? Can a failed floor beam in conjunction with the bridge deck carry load via an arching action spanning across the fracture?
4. Assuming the tension side of the floor beam fails, is it reasonable to assume the entire floor beam would suffer a full-depth fracture?

In most cases, floor beams in conjunction with continuous stringers and the continuity of the deck will provide a redundant system capable of carrying the vehicle load without a collapse.

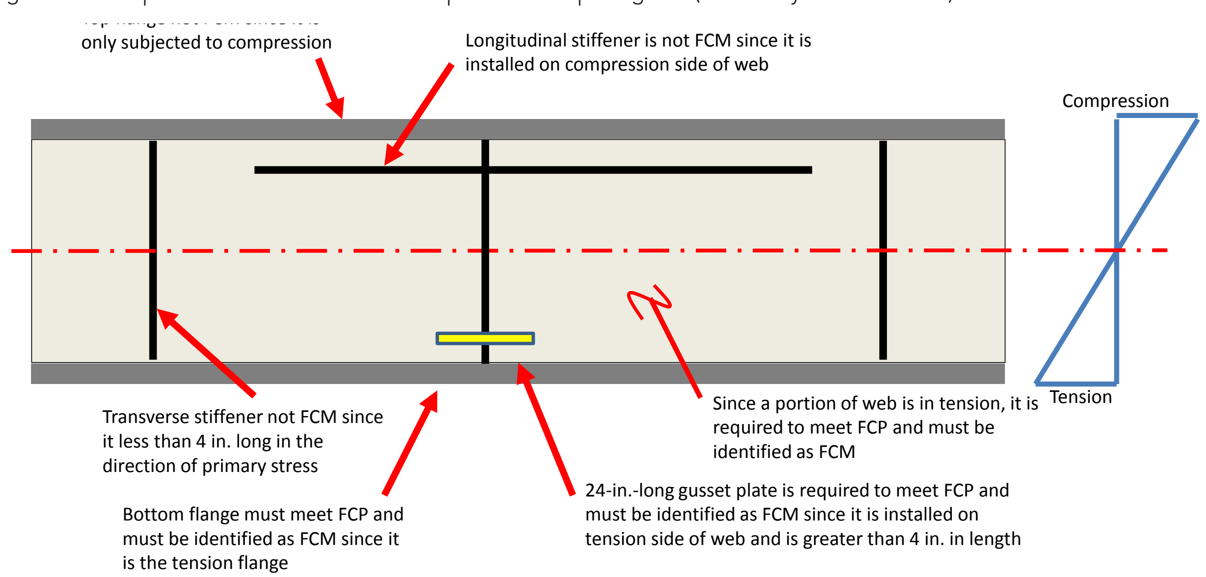
The authors have observed cases where engineers have classified floor beams as FCMs on bridges where the floor beams are spaced very closely, such as three feet or less. It is difficult to imagine that failure of a floor beam spanning from main girder to main girder spaced so closely could result in collapse of the bridge or roadway. If one were to idealize the main girders as supports between which the floor beams span, the cross section that carries the load would be comprised of multiple girders (i.e., floor beams). Hence, by definition, the floor beams could not be classified as FCMs at such close spacing.

If a floor beam is judged to be fracture critical, only the portion subjected to tensile stresses should be subjected to the FCP. If the floor beam is a rolled beam, while the entire beam would be required to meet the more stringent CVN material requirements, only the portion in tension is subjected to the FCP fabrication and inspection requirements. Hence, welds made to the compression flange would not be subjected to the FCP even though the rolled beam is a single piece of steel. If the floor beam is a fabricated plate girder, the tension flange and the web must meet the more stringent CVN material requirements of the FCP. However, only the portion of the web that is in tension needs to meet the FCP fabrication requirements. The top flange, which is only in compression, would not be considered fracture critical. Also, if the floor beam is designed as a simply supported member, small negative moments that may be produced due to a shear connection at the ends would not justify classifying the top flange as FC material.

Primary longitudinal girders. While the FCP applies to various elements, it was failure in elements such as primary longitudinal girders that led to the development of the plan. The classic main girders of a "two-girder" bridge can reasonably be classified as FCMs since failure of one of the beams may be expected to lead to collapse of the bridge. In the absence of any rigorous system analysis, the portions of the girders subjected to tension (flange and web) would be classified as FCMs and be required to meet the FCP, while the portion of the girder that is only subjected to compression does not, as illustrated in Figure 2.

Tension chords or diagonals in trusses. Generally speaking, most tension diagonals and chords in trusses would be classified as FCMs.

Figure 2 – Example of classification of FCM components on a plate girder (created by Robert Connor).



Tie girders. Generally speaking, tension ties would be classified as FCMs.

Miscellaneous attachments to FCMs. In addition to primary members, certain attachments must also be classified as FCMs and be fabricated to the requirements of the FCP. The reason for this is to ensure that components such as longitudinal stiffeners meet the same requirements as the base metal of the primary member. Further, the welds used to attach these components to the primary member must also meet the provisions of the FCP. For example, see this excerpt from AWS Article 12.2.2.2 Attachments:

"Any attachment welded to a tension zone of an FCM member shall be considered an FCM when any dimension of the attachment exceeds 100 mm [4 in.] in the direction parallel to the calculated tensile stress in the FCM. Attachments designated FCM shall meet all requirements of this FCP."

The FCP clearly states the attachment must be located on the portion of the member subjected to *tensile* stresses. Hence, a longitudinal stiffener that is welded to a girder in the tension zone of the web plate must meet the FCP, while a longitudinal stiffener in the compression zone of a web plate does not need to meet the FCP, as shown in Figure 2. Note that even though the attachment is welded to a web plate—which is designated as FCM in terms of the *material* selection (see AWS C12.2.2.2)—due to the fact that a portion of the web is in tension (since the welding of the longitudinal stiffener is on the compression portion of the web) there is no need to invoke the FCP. Note also that short attachments, such as a transverse stiffener, which is always less than 4 in. long in the direction of primary stress, need not be classified as FCM.

Ongoing Research

There are currently several research projects under way focusing on bridges and bridge members traditionally classified as fracture critical. Individual projects are studying the following areas:

Member-level redundancy. This research effort is examining the strength and fatigue performance of both riveted and bolted built-up members. While it is accepted that built-up members possess some level of internal redundancy, it has not been fully quantified through large-scale experimental or analytical research. Pooled fund study TPF-5(253) is characterizing this behavior and will result in evaluation and design guidelines for such members to ensure sufficient redundancy exists.

System redundancy. Several studies are under way, such as NCHRP Project 12-87a (research funded by AISC/NSBA focusing on twin-tub girders) as well as research sponsored by other agencies that are working to develop modeling, evaluation and design guidance related to analyzing bridges traditionally classified with FCMs. While it is generally presumed that failure of an FCM will cause collapse of the structure, field experiences where such failures have occurred suggest otherwise in all but extreme cases, such as in the Silver Bridge. These projects will result in rational criteria to characterize the benefits of load redistribution provided by the structural contributions of the deck slab, secondary members, parapets and other components not traditionally used. Further, the minimum live load capacity that is to be maintained in the faulted state will also be defined.

Exploitation of superior-toughness steel. It is well known that modern steels, in particular the HPS grades, offer far superior fracture toughness than "older" steels. However, the current A709 toughness requirements for HPS grade, while good, do not fully exploit the potential benefits of the HPS grades in terms of fracture resistance. These grades are consistently produced with toughness levels that far exceed minimum requirements. The research being conducted through pooled fund study TPF-5(238) explores the benefits of increasing the toughness requirements of some steel grades so that brittle fracture is no more likely than any other limit state, thereby effectively "taking fracture off the table" so to speak. In the extremely unlikely event a fatigue crack were to develop, tolerable crack sizes will be large enough to be reliably detected during normal inspections. By treating brittle fracture like any other limit state (e.g., buckling), it can be effectively mitigated eliminating the need for the term "FCM" in terms of long-term inspection.

Safer Bridges

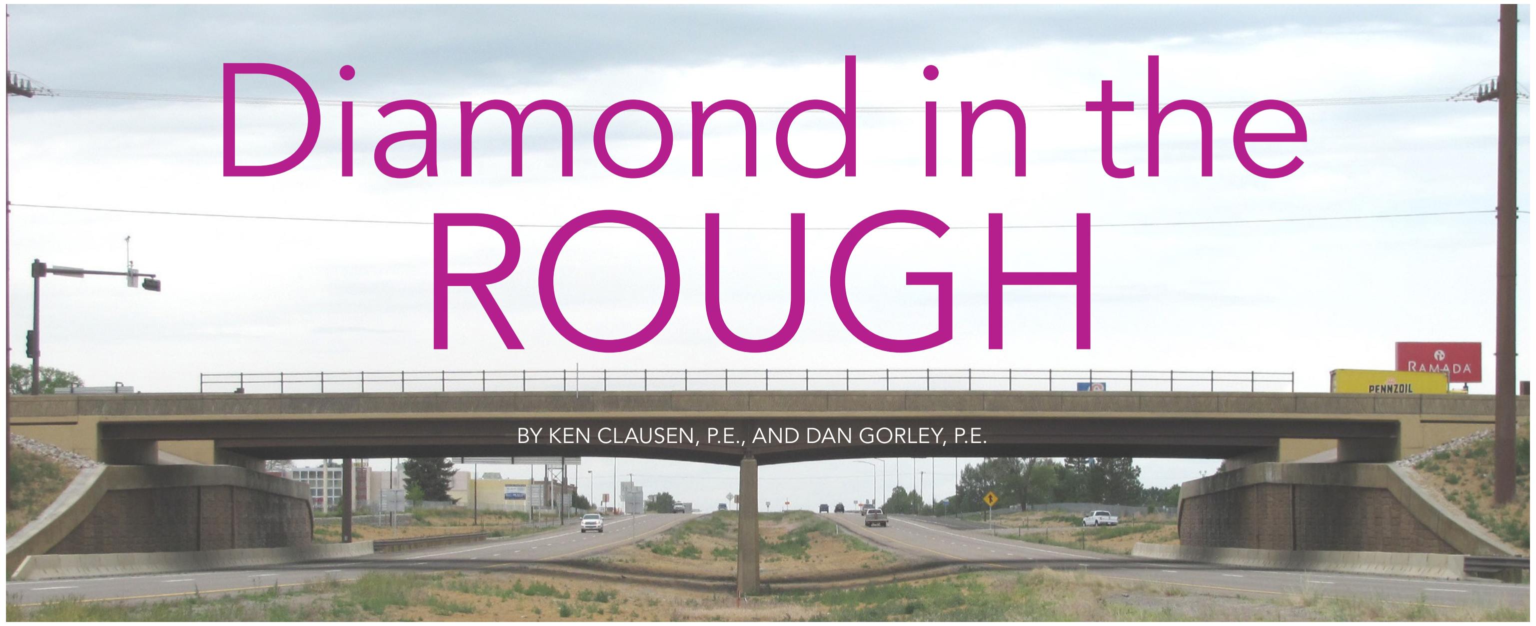
The AASHTO/AWS D1.5 FCP has been in place for nearly 35 years and appears to have eliminated brittle fractures in steel bridges through improved material toughness, fabrication practices and shop inspection. Additionally, the modern steels, in particular the HPS grades, possess far superior toughness than those used before the introduction of the FCP. The combination of these factors provides much greater safety than our legacy bridges built before the FCP.

While the additional first cost associated with the FCP have been estimated to be 5% to 10% of the total steel fabrication cost, the FCP should not be invoked based on the false assumption that this will somehow make the bridge "better." Designers and owners must appreciate that once a member is classified as an FCM, it is subjected to arms-length biennial inspections for the life of the bridge. As a result, the *long-term* costs associated with inspection greatly increase the life-cycle cost of the structure. When invoked arbitrarily, this simply increases costs, with little or no increase in actual performance of the structures.

In summary, engineers are encouraged to become familiar with the existing AASHTO/AWS D1.5 *Bridge Welding Code* provisions to ensure they are specified only when necessary and appropriate. Doing so will result in the most economical steel structure and is in the best interest of the owner, fabricator and public. Further, as current research progresses and is moved into practice, the meaning of the term fracture critical will certainly evolve. In fact, with modern steels, modern fatigue design approaches and advanced analytical tools, we may see a time when the term fracture critical will no longer be relevant. ■

A diverging diamond interchange in southeast Idaho is the first of its kind for the state and facilitates increased integration between two growing communities.

Diamond in the ROUGH



BY KEN CLAUSEN, P.E., AND DAN GORLEY, P.E.

THE CHUBBUCK INTERCHANGE needed a change.

Situated at the intersection of I-86 and US-91 in southeast Idaho, it joins the towns of Chubbuck (to the north of I-86) and Pocatello (to the south). Both communities have seen increased population growth and its associated traffic for many years, resulting in the need for increased capacity at this interchange. The old five-span concrete girder structure, built in the early 1960s, restricted capacity and traffic flow and was below current standards in several areas. It had insufficent load capacity, was too narrow for the increased traffic on US-91, had spans that were too short and clearance that was too low over the Interstate—and it had been hit by overheight loads on multiple occasions. It also wasn't able to accommodate potential future widening of I-86.

Designing a Diamond

After reviewing several options, Idaho Transporation Department (ITD) District 5 chose to build the state's first diverging diamond interchange (DDI) with the idea that it could be modified in the future to a single-point urban interchange (SPUI). The main structural difference between a DDI and an SPUI is that the DDI requies two relatively narrow structures set apart to accommodate the weaving traffic pattern; in the case of the Chubbuck Interchange, the clear distance between the two parallel twin bridges is 48 ft. On the other hand, an SPUI, which can handle higher volumes of traffic, requires a wide single bridge to opperate as a center point for all traffic movements. Because the SPUI can handle a higher volume of traffic, the DDI was designed to accommodate a future SPUI if or when traffic warrents the change.

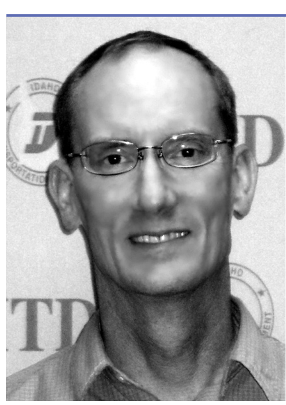
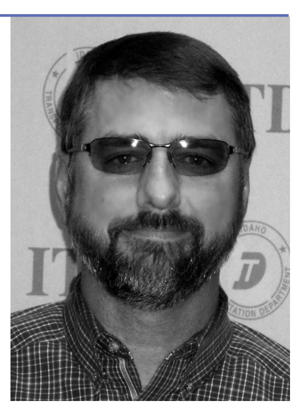
Due to the required sight distances and the need to minimize the grades on the interchange ramps, it was determined that the new US-91 crossing (which also includes pedestrian access across the bridges) could not be raised above its existing grade. Therefore, with the constraints of 17 ft of vertical clearance, no grade raise and two 85-ft spans, the superstructure could not exceed a depth of 36 in. over interstate traffic lanes, including the 8-in. deck.

In addition to these functional requirements, an important objective of this project was meeting the desire by the local community for an aesthetically pleasing structure. This interchange is in a highly visible part of the community and is the gateway to both cities for eastbound traffic on I-86. While there were no formal requirements or guidelines for what constituted an attractive bridge in this particular setting, the general principles of form following function, clean lines, slender profile and appropriate coloring were used to guide the design.

Because the geometry already dictated that the structure needed to be very shallow over the westbound and eastbound lanes of the Interstate, an effective solution was a two-span continuous girder with a variable-depth web. Welded steel plate girders also fit well with the aesthetic aspects of the project. The parabolic shape of a variable-depth girder is not only structurally efficient, but the curved line of the web haunch at the pier is also visually appealing. And, the color of the weathering steel chosen for the project fit well into the overall theme of earth tones.

A tapered wall pier was selected over a more traditional column and cap type pier for a couple of reasons. First was the need to be able to extend the pier between the twin structures if or when it becomes necessary to join the two bridges into a single structure to accommodate a future SPUI. This is easy to do with a wall because the load at each girder bearing is transferred directly through the wall to the footing and is relatively independent of the other girder loads. Second, because the overall structure depth is shallow and the bridge appears very light, a slender wall pier would continue that look. The wall can also be tapered to a thin bearing seat and does not require a massive cap to support the girder loads between columns.

Mechanically stabilized earth (MSE) walls were constructed in front of the abutments in order to minimize the span lengths yet still provide for future interstate widening. To accommodate widening, corrugated metal pipe (CMP) sleeves were used in the MSE fills to prevent damaging the soil reinforcement when piles are driven in the future. The integral abutments eliminated the need for expansion joints and bearings at the ends of the girders, two items that have historically required regular maintenance.

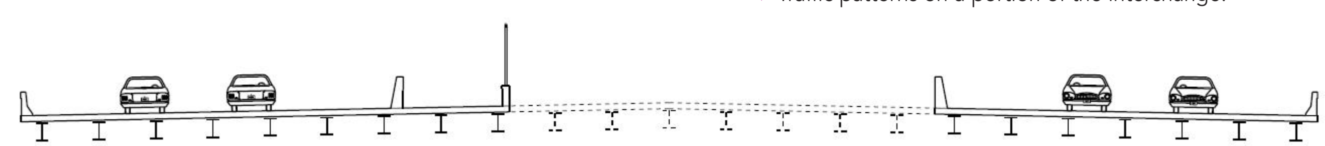



Ken Clausen (**ken.clausen@itd.idaho.gov**) is the bridge design engineering manager and **Dan Gorley** (**dan.gorley@itd.idaho.gov**) is a bridge design engineer, both with ITD.

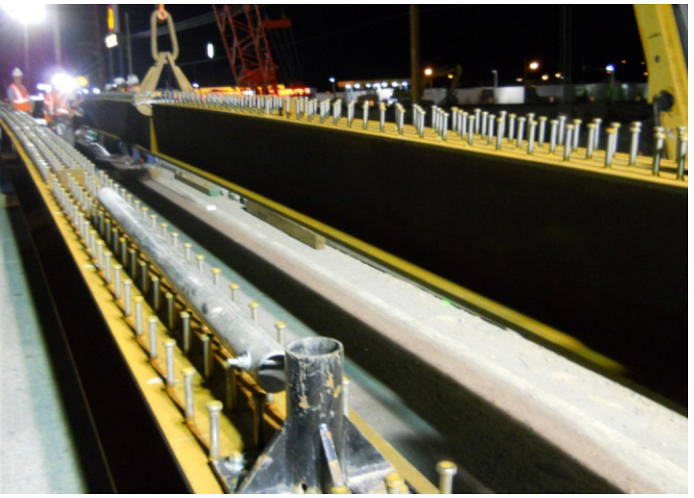
The new Chubbuck Interchange replaces a structure that had insufficent load capacity, had become too narrow for current traffic and whose clearance was too low over the Interstate.

The project is Idaho's first diverging diamond interchange. It can be modified to a single-point urban interchange in the future, as necessary.

With the constraints of 17 ft of vertical clearance, no grade raise and two 85-ft spans, the the superstructure couldn't exceed a depth of 36 in. over Interstate traffic lanes, including the 8-in. deck.



Traffic patterns on a portion of the interchange.

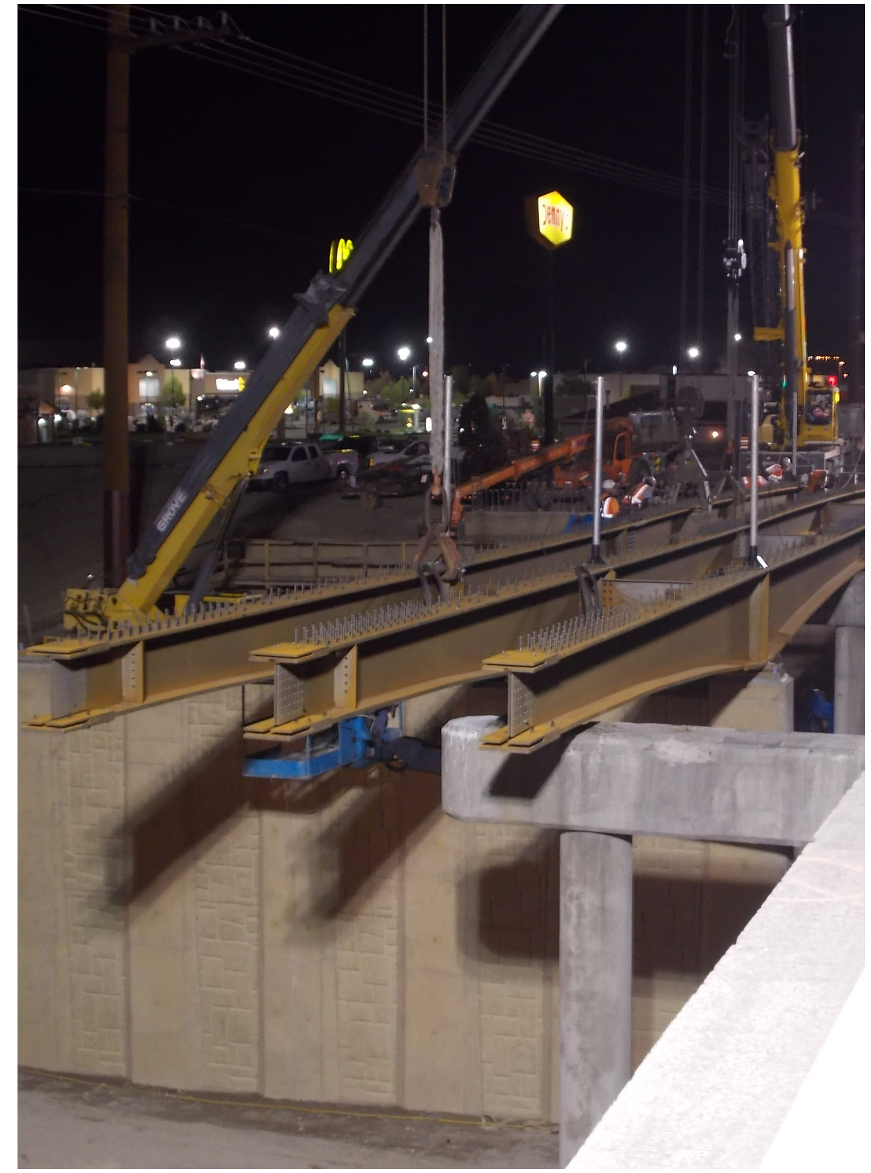


Girders were erected at night to limit disruption to traffic and consisted of three sections with a 60-ft variable-depth section over the pier.

To accommodate widening, corrugated metal pipe (CMP) sleeves were used in the MSE fills to prevent damaging the soil reinforcement when future piles are driven. The integral abutments eliminated the need for expansion joints and bearings at the ends of the girders.

Mechanically stabilized earth (MSE) walls were constructed in front of the abutments in order to minimize the span lengths, yet still provide for future Interstate widening.

Steel superstructure connecting to the piers.



The bridge uses 276 tons of steel.

It has been operating since the fall of 2013, and the local community has quickly adapted to the new traffic pattern of a DDI.

Bridging the Gap (Between Bridges)

As mentioned above, the two bridges are designed to be one bridge in the future. Both bridges were built on a 2% shed, so if they are indeed connected the combined bridge will have a 2% crown. The bridges were built on an accelerated schedule, using conventional, staged construction methods, in one building season (spring, summer and fall of 2013) and the DDI was open to traffic before winter. Stage 1 consisted of removing the eastern portion of the existing structure while still allowing enough width to carry four lanes of traffic while the east bridge was built. In Stage 2, a portion of the west side of the existing structure was removed and traffic was reduced to two lanes on the existing bridge while two lanes were in place on the new east bridge, maintaining four lanes; this allowed enough room to build the new west bridge. Once the west bridge was complete and traffic moved to the new birdge, Stage 3 could proceed, which involved removing the final portions of the existing bridge. Landscaping and aesthetic treatments were completed the following spring (2014).

The girders were erected at night to limit disruption to traffic and consisted of three sections with a 60-ft variable depth section over the pier; the web ranged from 23 in. to 42 in. There were also 55-ft sections at both ends with a constant web depth of 23 in. The contractor and steel fabricator chose to assemble the pier section and one of the end sections of the girder prior to shipment in order to accelerate placement. Additionally, the shear studs were installed at the fabrication facility. The total weight of structural steel on the project is 276 tons.

The bridge has been operating since the fall of 2013, and the local community has quickly adapted to the new traffic pattern of a DDI; drivers and pedestrians now enjoy improved access over and onto the freeway. ■

Owner and Structural Engineer
Idaho Transportation Department

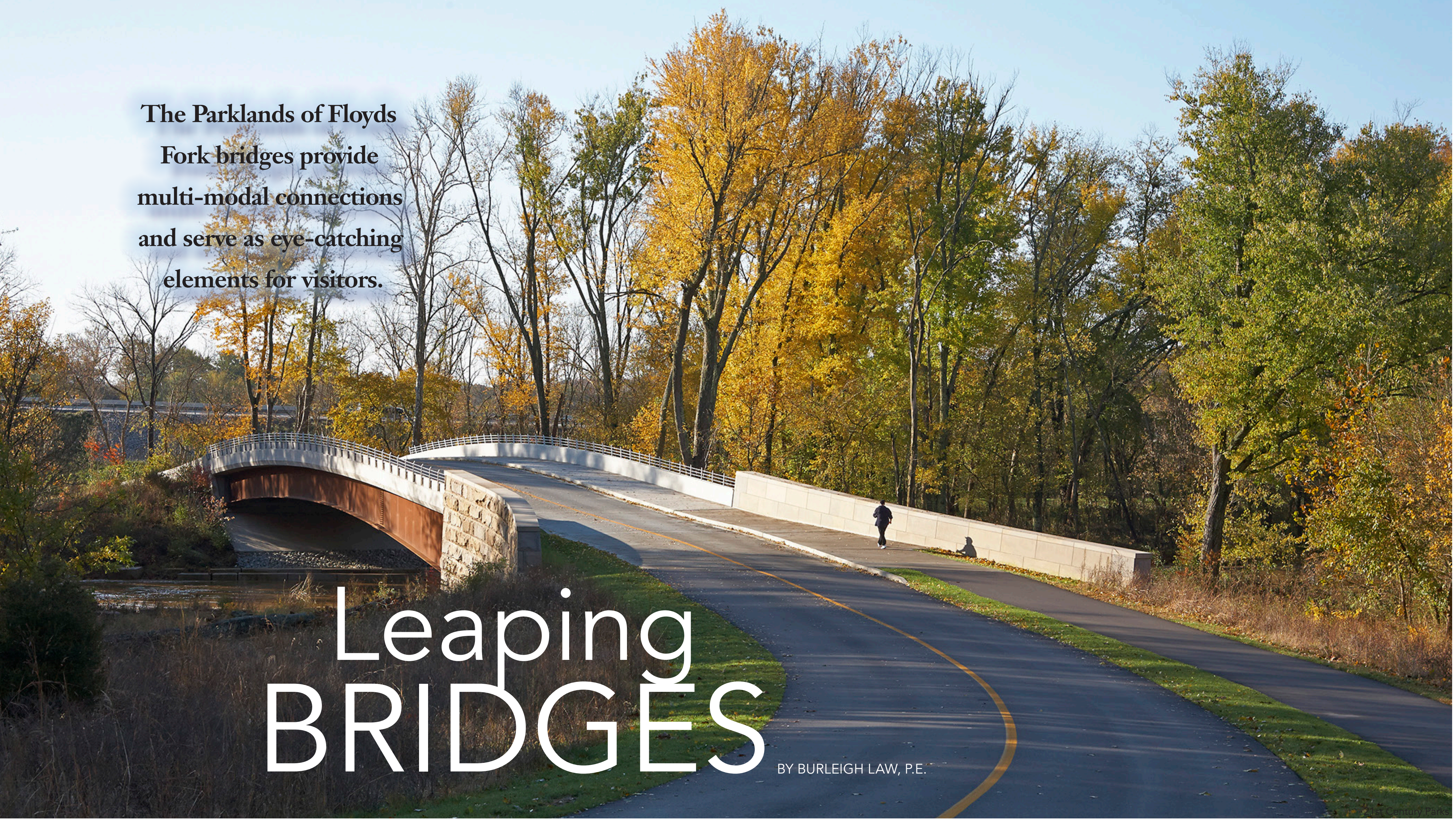
General Contractor
Ralph L. Wadsworth

Steel Team

Fabricator
Utah Pacific Bridge and Steel, Lindon, Utah (AISC Member/NSBA Member/AISC Certified)

Erector
Ralph L. Wadsworth Construction Co., LLC, Draper, Utah (AISC Member/Advanced Certified Steel Erector)

Detailer
Tensor Engineering, Indian Harbour Beach, Fla. (AISC Member/NSBA Member)



The Parklands of Floyds Fork bridges provide multi-modal connections and serve as eye-catching elements for visitors.

Leaping BRIDGES

BY BURLEIGH LAW, P.E.

▲ The leaping bridge grades are 8% at both the leaping and landing ends, resulting in an arch effect.

▲▼ A curved girder, gradually making its way over Floyds Fork. The high arches accentuate the passage of water beneath the bridge and provide overlooks from the bridge centers and abutments.

Burleigh Law (blaw@hntb.com) is a senior project engineer with HNTB Corporation.

GREAT PARKS are part of Louisville's history.

The city's park system was designed in the 1890s by world-renowned landscape architect Frederick Law Olmsted, who sought to incorporate nature into neighborhoods and enhance social interactions and economies.

With more than one million visitors in the past year, the Parklands of Floyds Fork is a world-class park that serves as a model for other communities across the country. The park's four new leaping bridges (three cross Floyds Fork in the Beckley Creek section of the Parklands and the fourth is in the Broad Run section) fulfill its goals of fostering a well-used and well-loved place and playing a role in shaping the community, while also providing access and circulation.

Inspired Jump

Inspired by a deer leaping over a fence, the bridges are composed of haunched welded steel plate girders fabricated from M270 Grade 50W weathering steel to match the natural surroundings of the park. Depths vary from 60 in. to 120 in., and the girders are painted at both ends and include drip bars to mitigate staining of the abutment. The high arches accentuate the passage of water beneath the bridge and provide overlooks from the bridge centers and abutments. At 38 ft wide with single spans ranging from 160 ft long to 170 ft long, the bridges are shared by pedestrians and vehicles. Each of the bridges features coordinated elements, but each is also place-specific to create distinct landmarks.

For example, instead of a typical flat bridge, the leaping Thornton Bridge grades are 8% at both the leaping and landing ends, resulting in an arch effect. When the bridge's five 170-ft girders, each weighing approximately 40 tons, were lifted into place to splice the field pieces together, the steel erector initially feared a fabrication error because the girders were unlike anything they had previously seen. Steel girders, by design, require upward camber in the web to account for all the dead loads that eventually deflect the bridge downward, but this unique profile required 50 in. of additional camber.

No Skew

As the bridges (which each use approximately 245 tons of steel) are straight with no skew, this enabled an architecturally rich structure to be simplified. The girders were designed in steel bridge design program MDX using a line girder analysis in lieu of the more complex girder system analysis required for curved or sharply skewed bridges. The fabricator asked for one splice plate thickness change to be consistent with other sizes used on the girders, but otherwise the shop drawings were very consistent with the design plans. Camber met expectations and resulted in no unexpected challenges while setting overhang forms and brackets, the stay-in-place metal forms or the screed elevations.

Details and recommendations found in the AASHTO/NSBA *Steel Collaboration Guidelines for Design Details* and the *Mid-Atlantic States Structural Committee for Economical Fabrication Details* resulted in simplified design and detailing while providing a cost-effective solution for fabricators. The architect sought to minimize the visible connections and miscellaneous members beyond the girder itself. Maximum fabrication depth and shipping lengths dictated the need for field splices, but the number of transverse stiffeners and connection plates was minimized by increasing the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▲ The bridges are composed of haunched welded steel plate girders fabricated from M270 Grade 50W weathering steel.

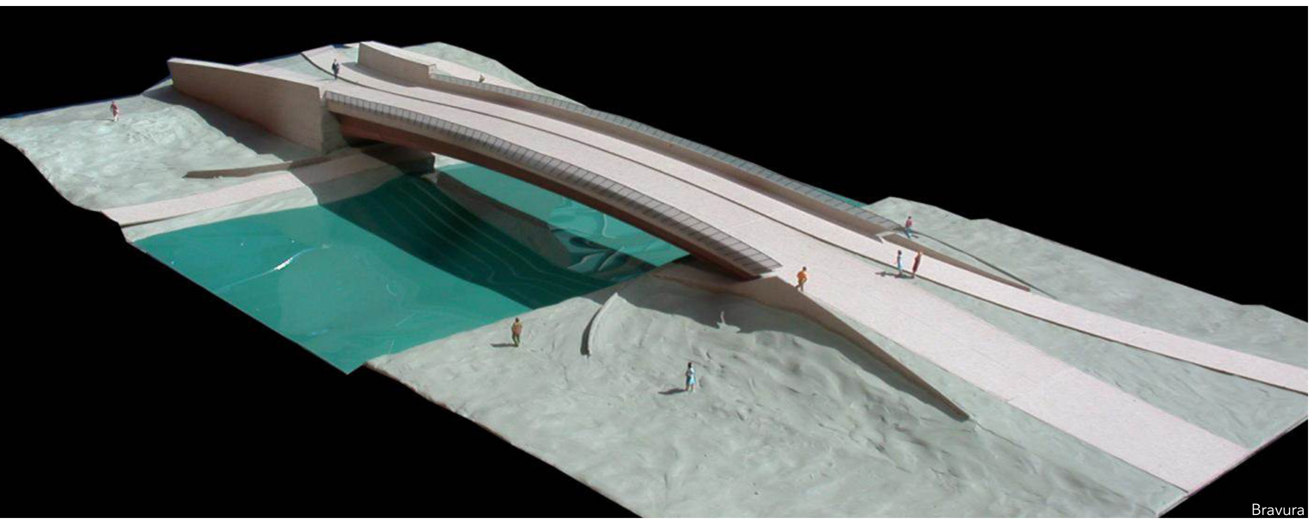
▲ A second girder following the first one.

▲ The staging and laydown area was limited, but the contractor was able to place cranes next to the rear of the abutments and hoist the field sections out over the river.

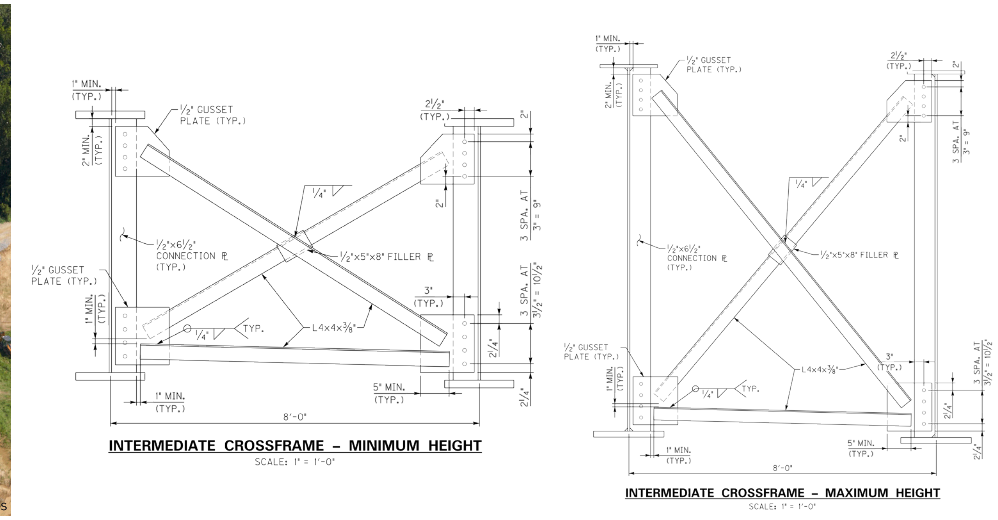
▼ A model of one of the bridges.

▼ The bridges are inspired by the arc of a deer leaping over a fence.

▼ Crossframe drawings for the Thornton Bridge.


Bravura


© Quadrant; Courtesy of 21st Century Parks



web thickness, only providing them at the intermediate cross-frames, and by pushing the intermediate cross frame spacing to the code-allowed maximum. The abutment and end bent are semi-integral with concrete end diaphragms and no steel end diaphragms, so the first intermediate cross-frames were placed within 10 ft of the substructures to strengthen stability during erection and slab pour.

The bridge arch not only met the architect's aesthetic requirements, but also increased the hydraulic opening for a bridge that hugs the river's edge. Much of the Parklands is located in a floodplain and if the bridges had been designed to meet typical Kentucky Transportation Cabinet hydraulic requirements, higher profiles and additional spans would have been required. Because the land is owned by 21st Century Parks, a private nonprofit organization, they and the architect were able to select a smaller-footprint, single-span bridge with a high-arch opening that provided sufficient freeboard in the middle of the bridge. These bridge components are designed to withstand flooding conditions for a 75-year design life, including large debris moving down Floyds Fork.

Weathering steel, with its dark, rustic brown color, was chosen for its ability to blend in with the park surroundings. Prestressed precast concrete beams would have weighed almost three times more and required unique forms and fabrication to create the same variable-depth arch shape, but for a much higher cost. In addition, a cast-in-place concrete beam option would have required falsework within the river, which was not permissible by regulatory agencies and would have restricted river traffic. Structural steel resulted in lighter members and smaller cranes during the erection process, keeping the bridges' costs lower. The staging and laydown area was limited since the regulatory agency permits required a 250-ft limit of disturbance and tree clearing along the length of the river. Fortunately, the contractor was able to place cranes next to the rear of the abutments and hoist the field sections out over the river.

To create a sense of movement, the steel pedestrian bridge railings include posts that are set at different angles. An incremental difference of 1.2° varies the post railings from -7.2° to match the leaping abutment sloping face to 45° to match the landing end bent sloping face. The differing angles and the profile grade added complexity to the fabrication and installation of the railings, while further enhancing the arch.

A Sense of Place

From the air, the Louisville Loop Trail (part of a 100-mile multi-recreational trail) winds under the bridge to mimic the movement and flow of Floyds Fork. Along the trail and under the bridge is a decorative variegated limestone block stepping and seating area. The massive stone blocks are as big as 3 ft high by 5 ft wide by 8 ft long and weigh up to 10 tons each.

Fishermen can cast for trout, catfish, bass and bluegill, while visitors can enjoy picnics and a view of the expansive Egg Lawn, a 22-acre oval-shaped lawn, surrounded by a 0.7 mile paved, tree- and light post-lined walking trail. The Thornton Bridge, part of the Beckley Creek Park area, provides a connection to trails, lakes, a dog park, picnic areas, a canoe launch and other park amenities including a community center, an education/interpretation center, playgrounds, a splash pad and pavilions.

The leaping bridges are the result of collaboration between structural engineer HNTB Corporation and architect Bravura. At the beginning of the planning and design process, the team drew inspiration from walking the 3,200-acre park, of which 80% is naturally restored or managed woodlands, wetlands and meadows. In addition to preserving large natural areas, the park provides active recreation areas, including sports fields, community parks, multi-use trails for bikers, horses, hikers and launches for boaters. Final design of the bridges was completed in three months, with no major changes during the design process, and construction took just under a year.

While the cost of a typical highway bridge is about $110 per sq. ft, the leaping bridges cost around $400 per sq. ft due to the added aesthetics and the limestone-clad abutment, for a total of $2.6 million for the Thornton Bridge. The park's bridges and roads were federally funded through $38 million in federal funds from the Federal Highway Administration's "Safe, Accountable, Flexible, Efficient Transportation Equity Act, A Legacy for Users" for the park's infrastructure. Private donors provided about $70 million, which funded several of the park's amenities.

Through this collaborative process and by using ordinary materials in unique ways, the signature leaping bridges help define the world-class park that is and will be well-used and well-loved by current and future generations. ■

Owner
21st Century Parks

General Contractor
MAC Construction and Excavating, New Albany, Ind.

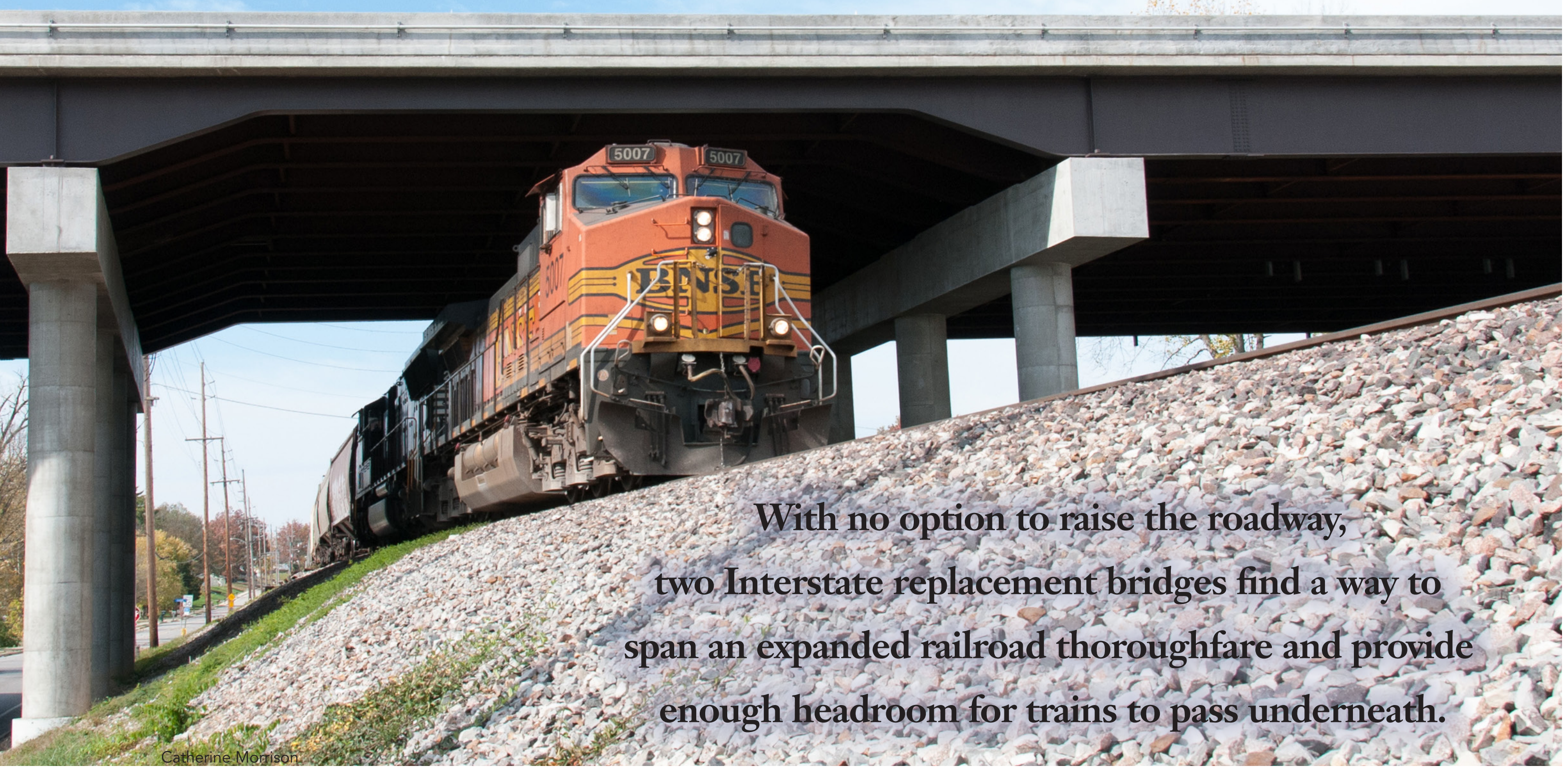
Architect
Bravura

Structural Engineer
HNTB Corporation

Steel Fabricator and Detailer
Stupp Bridge Company, Bowling Green, Ky. (AISC Member/NSBA Member/AISC Certified)

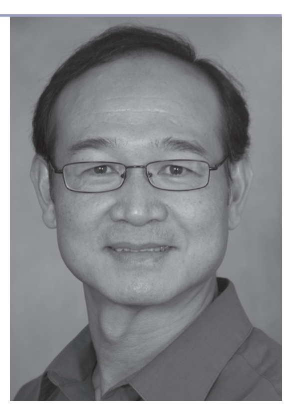
Making HEADWAY

BY ROBERT BRENDEL AND ANOUSONE AROUNPRADITH, P.E.



Catherine Morrison

With no option to raise the roadway, two Interstate replacement bridges find a way to span an expanded railroad thoroughfare and provide enough headroom for trains to pass underneath.



Robert Brendel (robert.brendel@modot.mo.gov) is a special assignments coordinator– customer relations and **Anousone Arounpradith** (anousone.arounpradith@modot.mo.gov) is a structural project manager– bridge division, both with MoDOT.

SAFE AND SOUND is the name of the game for Missouri bridges—and not just figuratively.

In one of the most ambitious statewide bridge improvement programs in the country, the Missouri Department of Transportation's (MoDOT) Safe and Sound Bridge Improvement Program has improved 802 bridges in three-and-a-half years: 554 replacements through a single design-build contract and 248 rehabilitation projects managed through its more conventional monthly lettings.

As part of the program, two Interstate 55 bridges, northbound and southbound traveling over the BNSF railroad and 5th Street in Festus, Mo., were identified to be re-decked. These bridges, built in 1967, had six spans each ranging from 37 ft to 60 ft long for a total length of 300 ft. In the center, over the railroad tracks, was a 60-ft span.

However, this stretch of I-55, which lies about 35 miles south of downtown St. Louis, is experiencing rapid growth and carries increasingly higher traffic volumes as commuters from Jefferson County's bedroom communities travel to work in St. Louis. Consequently, there was a desire to widen the bridges for an eventual third lane in each direction. This could not be accommodated if the bridges were to merely be re-decked. Plus, the Safe and Sound budget had only $1.5 million to allocate to the project—not enough to replace the structures.

Luckily, MoDOT's St. Louis District chipped in $3 million, which enabled MoDOT's Bridge Division to search for the design solution to economically build two new bridges that would accommodate the additional lanes while also fitting within multiple on-site constraints.

Catherine Morrison

◄ The notched-out section of the center span provides the necessary 23 ft of clearance for trains below.

▲ MoDOT's Safe and Sound Bridge Improvement Program has improved 802 bridges in three-and-a-half years, including the two I-55 bridges.

▼ A notched-out section in Stupp's fabrication facility.

Stupp Bridge

Snug Space

MoDOT's first goal was to have a jointless deck to ensure longevity for the new bridges—which effectively meant the old six-span configuration was out of the question. But fewer spans also translated into deeper girders—and could those fit within the other site limitations?

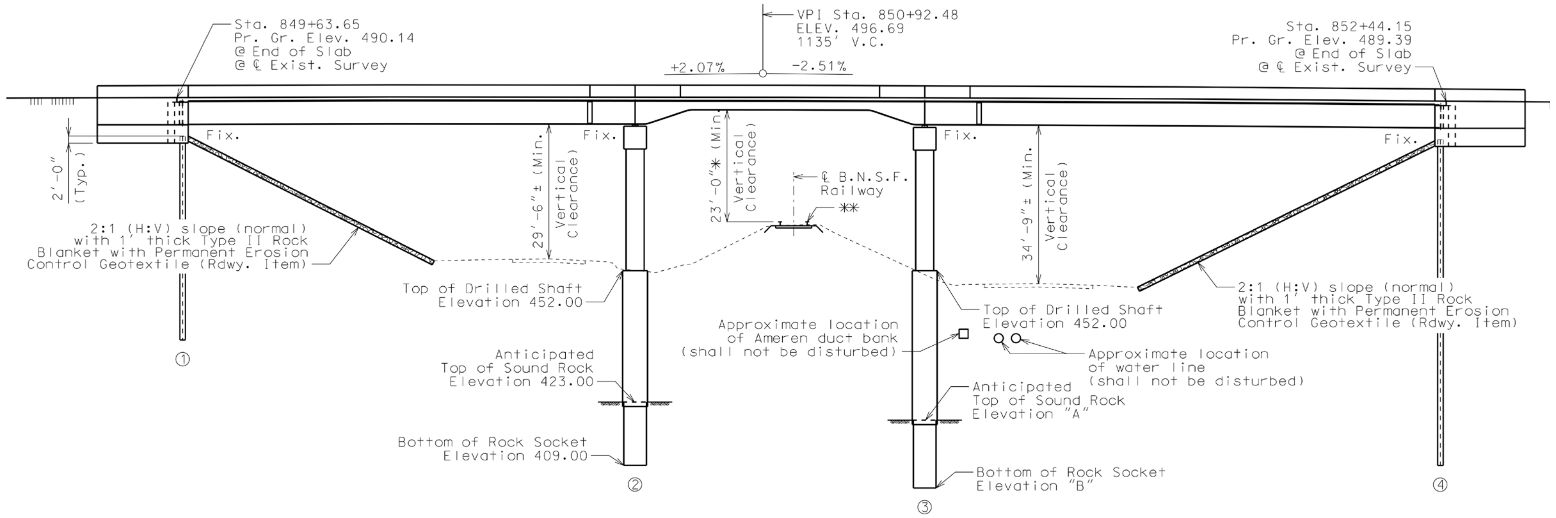
The most significant of these was the railroad. BNSF wanted the new bridge to accommodate space for a second track and also required 23 ft of vertical clearance—a snug proposition in both directions. In addition, utilities in the area that could not be disturbed, plus maintaining the flow of 5th Street and a private driveway, limited the options for locating the bents. Because there was no possibility of raising the highway above or lowering the railroad track below, vertical clearance parameters were unyielding and any new design would have to stay within them.

There were other concerns as well. First, if mechanically stabilized earth (MSE) walls were used for the abutments, geologists were concerned they could settle as much as 3 ft. If concrete girders were chosen, there were very real concerns about their weight as well as the size of crane that would be needed to lift them into position during staged construction.

Ultimately, a three-span continuous composite plate girder design was chosen for the bridges, with spans of 100 ft, 64 ft and 114 ft. The lighter-weight steel superstructure also helped address seismic concerns, as the bridges are within the New Madrid Seismic Zone.

While it's atypical to have the shortest span in the middle of a bridge—typically they're on the ends—the arrangement worked. Positioning the 64-ft span in the middle not only allowed enough room for the railroad and its future second track but also allowed for placing bents that would not disturb utilities or the track bed embankment.

However, the vertical clearance issue remained. Achieving the three-span continuous girder design required a girder depth of 60 in. The only way to get the needed 23 ft of clearance over the railroad was to notch-out a section of the center span girders. The




Anousone Arounpradith


Stupp Bridge

▲ An elevation drawing of the bridges.

◀ The center girder "notches" up to a 22-in. × 9⁄16-in. web plate with 15-in. × 1 1⁄8-in. flanges.

end span girders were 60-in. × 9⁄16-in. web plates with 15-in. × ¾-in. top flanges and 15-in. × 1⅛-in. bottom flanges, and the center girder "notches" up to a 22-in. × 9⁄16-in. web plate with 15-in. × 1⅛-in. flanges. As such, the steel was fabricated to gradually taper the depth of the girder from 60 in. at the ends to 22 in. over the centermost 40 ft of the span—which not only solved the vertical issue but also saved on material weight and provided a unique look to the bridges. Weathering steel was used for the spans, with the exterior girders painted to ensure a consistent appearance that fit in with the aesthetics of the bridges' surroundings.

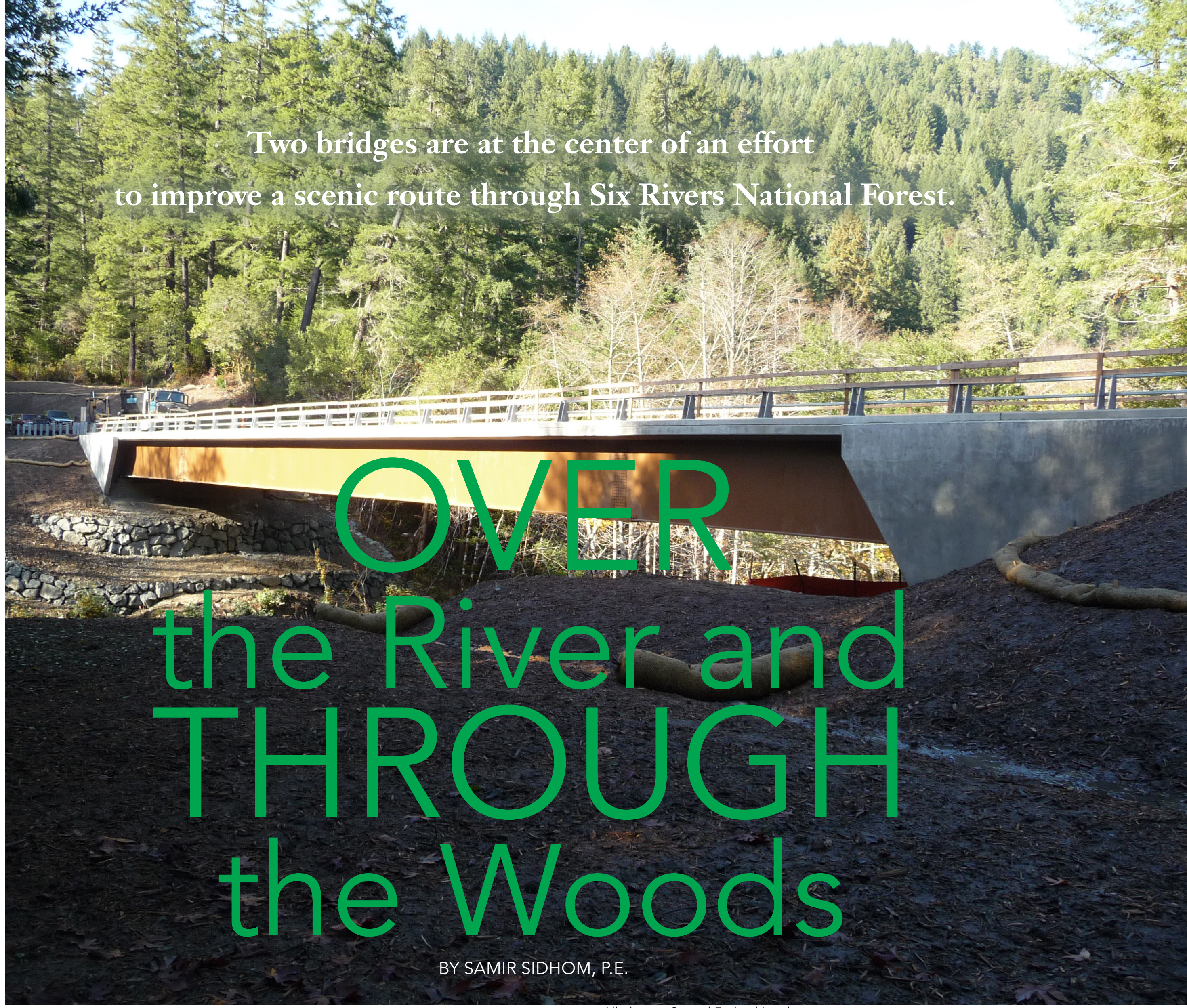
Because of the site complexities, general contractor Gershenson Construction Company was given a long window to complete the project over two construction seasons. The northbound bridge was completed within a year of the start of construction and the southbound bridge was finished six months after that; the bridges use 375 tons of steel in all. ■

Owner and Structural Engineer
Missouri Department of Transportation

General Contractor
Gershenson Construction Company, Eurkea, Mo.

Steel Fabricator and Detailer
Stupp Bridge Company, Bowling Green, Ky. (AISC Member/NSBA Member/AISC Certified)

◀ The two new bridges use 375 tons of steel in all.



Two bridges are at the center of an effort to improve a scenic route through Six Rivers National Forest.

OVER the River and THROUGH the Woods

BY SAMIR SIDHOM, P.E.

All photos: Central Federal Lands

SIX RIVERS NATIONAL FOREST has certainly earned the right to be named a National Forest.

Established in 1947 by President Harry Truman, it is nearly 1,500 sq. miles in size, including 137,000 acres of old-growth forest, and has 366 miles of wild and scenic rivers, distinct botanical areas and public use areas for camping, hiking and fishing—a nature-lover's paradise in northwestern California.

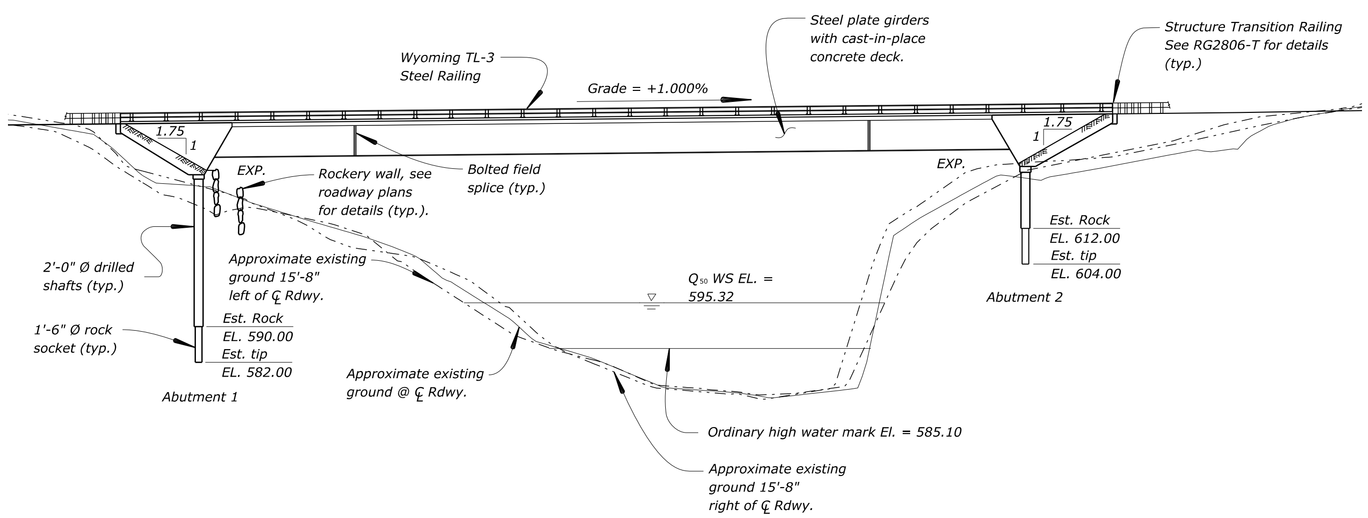
Given the sheer number and mileage of rivers in the park, there are, of course, several bridges. In 2012, the Central Federal Lands Highway Division of the Federal Highway Administration (FHWA), in cooperation with Six Rivers and Del Norte County, Calif., began the process of constructing the second phase of an improvement plan project to CA FH 112, also known as the South Fork Smith River Road, which spans the Smith River in the northernmost section of the forest. The project included the replacement of two bridges on the road: the Steven Memorial and Hurdy Gurdy Creek Bridges, both designed to AASHTO LRFD *Bridge Design Specifications*. The $8.6 million project is entirely funded by FHWA and is part of a larger project to upgrade all of the one-lane sections of South Fork Smith River Road to allow traffic in both directions.

A bridge selection study evaluated structural options based on: required bridge length, the remoteness of the construction sites, transporting girders to these sites via forest roads with difficult angles, initial cost, the use of deep foundations in a high-seismic area, maintenance and minimal impact of the bridge piers on the environmentally sensitive areas and waterways that they would cross—and a steel plate girder design was chosen as the best option for both bridges. In addition, staged construction wasn't required for either bridge.

Samir Sidhom (samir.sidhom@fhwa.dot.gov) is the bridge design team leader with the Central Federal Lands Bridge Office.

▲ The Hurdy Gurdy Creek Bridge spans were placed with two cranes, one on each side of the river.

▲ The bridge consists of four 189-ft, 4-in. spliced girders, 7 ft deep, with a 110-ft long midsection and two 37-ft, 2-in. end sections.



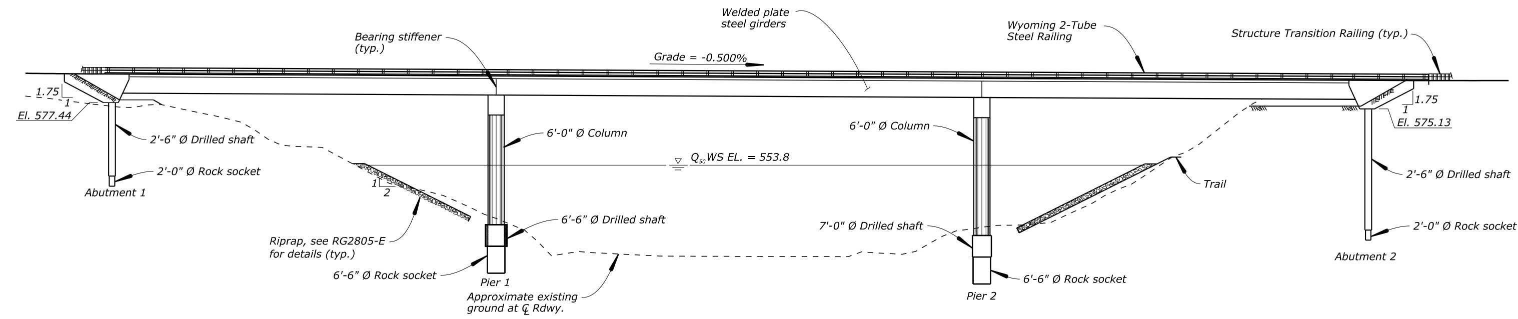
▲ An elevation drawing of the Hurdy Gurdy Creek Bridge.

▼ Installing the new Hurdy Gurdy superstructure; the original bridge is visible just below the new steel.

Hurdy Gurdy Creek Bridge

The original Hurdy Gurdy Creek Bridge was a one-lane, 170-ft-long bridge with two simple spans: a rolled beam approach span and a riveted steel plate girder main span. The approach was 30 ft long and the main span was 140 ft long, and the bridge's total width was 15 ft, 6 in., resulting in a clear roadway width of 14 ft.

For the replacement, a single-span bridge was selected to avoid the need for, significant cost of and environmental degradation inherent to pier construction in a streambed. The new bridge is a two-lane, 190-ft-long, steel plate girder bridge consisting of four 189-ft, 4-in. spliced girders (7 ft deep) with a 110-ft long midsection, two 37-ft, 2-in. end sections and a cast-in-place concrete deck.



▲ An elevation drawing of the Steven Memorial Bridge.

▼ The new bridge is 470 ft long.

▲ The new design increased the crossing's width to 31 ft, 4 in., providing a clear roadway width of 28 ft.

▼ The superstructure rests on two piers, 48 ft tall and 52 ft tall, as it crosses the river.

The new width is 31 ft, 4 in., providing a clear roadway width of 28 ft. Bridge abutments were founded on 2-ft, 6-in. drilled shafts socketed into bedrock.

The new alignment was shifted to the upstream (north) side of the existing structure to allow the existing bridge to stay in service during construction. The county and Forest Service approved WYDOT standard TL-3 bridge rail to be used on the structure, with a WYDOT standard corrugated beam approach guardrail transition section. The chosen rail was ideal for scenic views, with minimal obstruction, while also meeting safety requirements.

Approach slabs were connected to the cantilevered end walls at both ends of the bridge, and these end walls were designed to engage the soil behind the abutments in case of an earthquake. In addition, a gravity-retaining rockery was used to provide slope stability at the abutment of one embankment, a solution that ended up being cost-effective, aesthetically appealing and sustainable.

Steven Memorial Bridge

The original Steven Memorial Bridge was a one-lane, 330-ft-long, three-span riveted steel plate girder bridge with a suspended middle span. The span configuration was 94.4 ft-140 ft- 94.4 ft, and the bridge's total width was 16 ft, 4 in., providing a clear roadway width of 15 ft. The new bridge is a two-lane, 470-ft-long, three-span steel plate girder bridge consisting of four spliced girders (6 ft deep) with a 180-ft-long center span, two 145-ft outer spans and a cast-in-place concrete deck. The new design increased the crossing's width to 31 ft, 4 in., providing a clear roadway width of 28 ft, and 15-ft and 20-ft approach slabs were constructed at the ends of the bridge. As with the Hurdy Gurdy Bridge, abutments were founded on 2-ft, 6-in. drilled shafts socketed into bedrock, and the 48-ft-tall and 52-ft-tall, 6-ft-diameter piers were founded on 6-ft, 6-in.-diameter drilled shafts socketed into bedrock and encased with 1-in.-thick galvanized steel.

➤ The three-span Steven Memorial Bridge consists of four 6-ft-deep spliced girders.

In addition, hammerhead pier caps with round columns were used to create a slender and open structure. To evaluate the effects of seismic forces on the superstructure, the piers and the drilled shafts supporting the piers and abutments, a complete 3D finite element model of the structure was developed to accurately predict the bridge's behavior.

The new bridge was constructed adjacent to (north of) the existing one to allow the latter to stay in service during construction, and the new alignment was shifted to the downstream side of the existing structure.

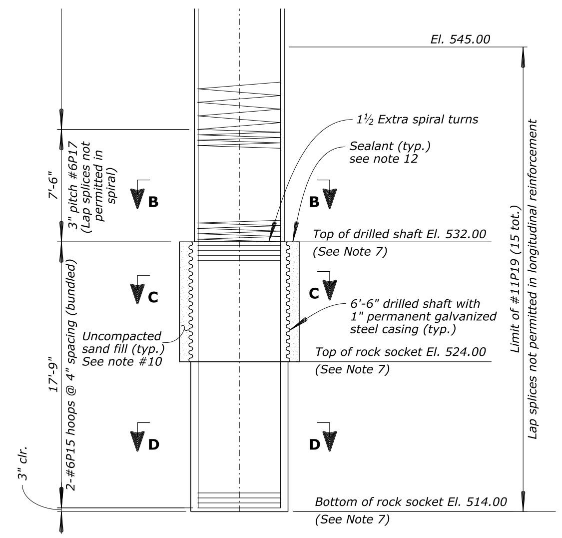
Over 415 tons of structural steel were used to build both bridges. Girders were shop painted using a three-coat system and were all preassembled at the fabrication shop for quality control purposes. Construction of the bridges began in May of 2013 and was completed in less than 12 months. The new pair now provides improved, scenic access through Six Rivers National Forest. ■

Owner
Six Rivers National Forest

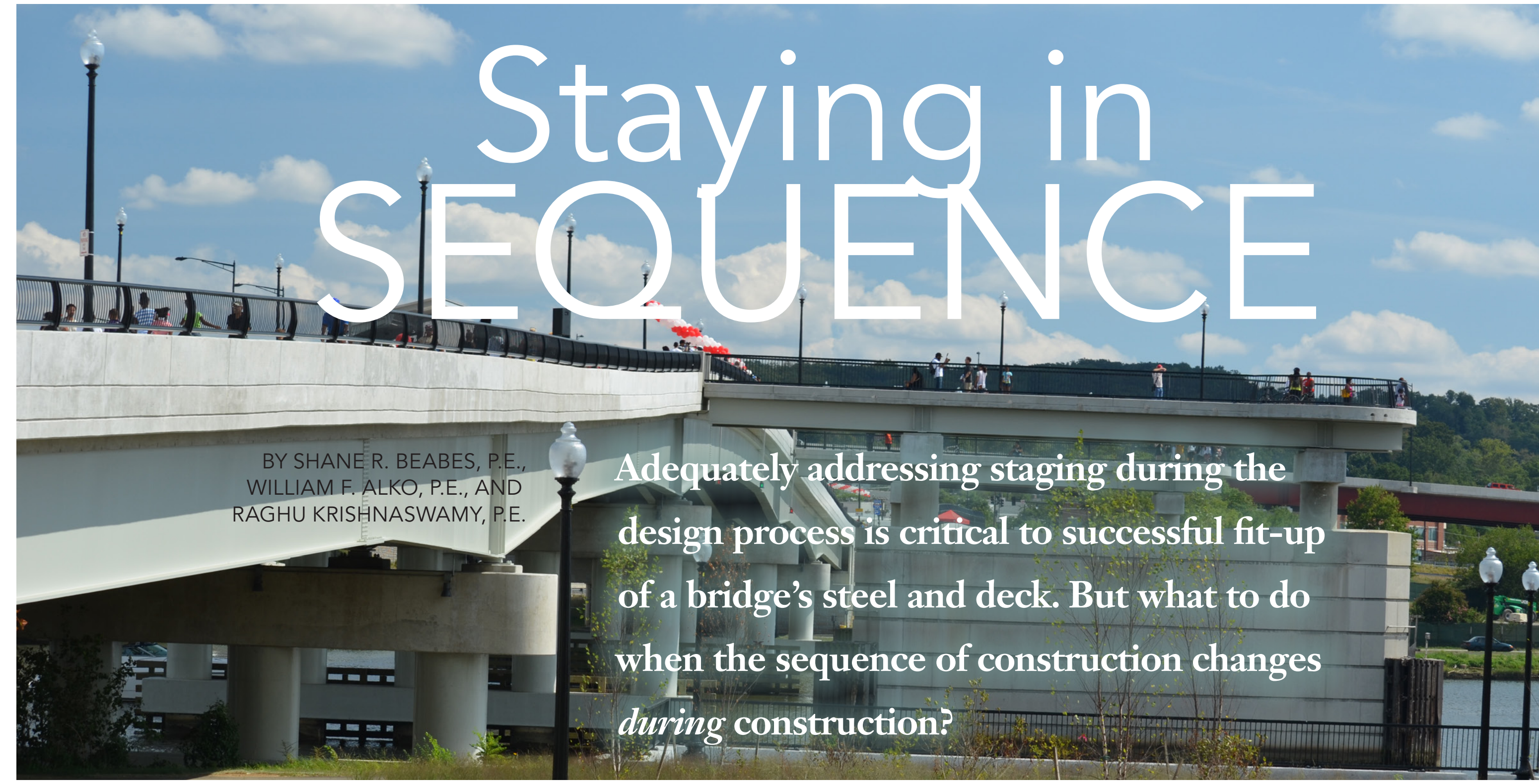
Structural Engineer
Central Federal Lands Bridge Office, Lakewood, Colo.

General Contractor
West Coast Contractors, Inc., Coos Bay, Ore.

Steel Fabricator, Erector and Detailer
Fought and Company, Inc., Tigard, Ore. (AISC Member/NSBA Member/AISC Certified)



➤ A column-shaft rock socket connection detail for Pier 1 of the Steven Memorial Bridge.



Staying in SEQUENCE

BY SHANE R. BEABES, P.E., WILLIAM F. ALKO, P.E., AND RAGHU KRISHNASWAMY, P.E.

Adequately addressing staging during the design process is critical to successful fit-up of a bridge's steel and deck. But what to do when the sequence of construction changes *during* construction?

SEQUENCING MATTERS.
Whether it is shored or un-shored, non-composite or composite or single or multi-stage construction, all of these techniques represent a sequence of construction for bridges that must be recognized by the engineer during the analysis and design process.

Considering the sequence of construction is critical to the methodology of load application, distribution of forces and prediction of deflections for the bridge structural framing system. In bridges that are tangent or mildly skewed, a line girder analysis (1D) is typically used to predict forces and deflections in a bridge system. Although not complex, the model's ability to best predict the performance of the girders will depend on the development and application of loads to the individual girders that are compatible with the staged sequence of construction.

For bridges that are curved or significantly skewed, a more rigorous analysis is often warranted. Typically, this will involve a 2D or 3D model. In these models, similar to the line-girder model, the application of load will depend on the staged sequence of construction. However, since the more rigorous models rely on the cross frames to distribute both dead load and live load forces to adjacent girders in the system, staged construction will dictate which girders are connected by cross frames at any given time, and ultimately the distribution of forces within the structural framing. Regardless of how simple or complex, the engineer must consider the sequence of construction to successfully predict the performance of the bridge.

Across the Anacostia

On the complex end of the spectrum is the 11th Street



Shane R. Beabes (shane.beabes@aecom.com) is district chief engineer–bridges and associate vice president at AECOM. He is an active member of several AASHTO/NSBA Joint Collaboration Committees on bridges, including Task Group 13 – Analysis of Steel Bridges (Chair), Task Group 12 – Design for Economy and Constructability (Member) and Task Group 10 – Erection (Member). **William F. Alko** (william.alko@aecom.com) is a bridge technical leader at AECOM and has over 26 years of experience in the analysis and design of steel bridges and transportation-related structures. **Raghu Krishnaswamy** (raghu.krishnaswamy@aecom.com) is deputy structures manager at AECOM. He has 10 years of experience designing steel bridges and transportation-related structures.

Bridge in Washington, D.C., part of an overall design-build-to-budget project let by the District Department of Transportation. The project was awarded based on a $260 million best-value, design-build, procurement process and involved the construction of three main river bridges and extensive ramp reconfigurations on both sides of the Anacostia River. The project included two new parallel Interstate bridges and one new local bridge, the 11th Street Bridge, with the objective of separating the Interstate movements from the local traffic, pedestrians and bicyclists.

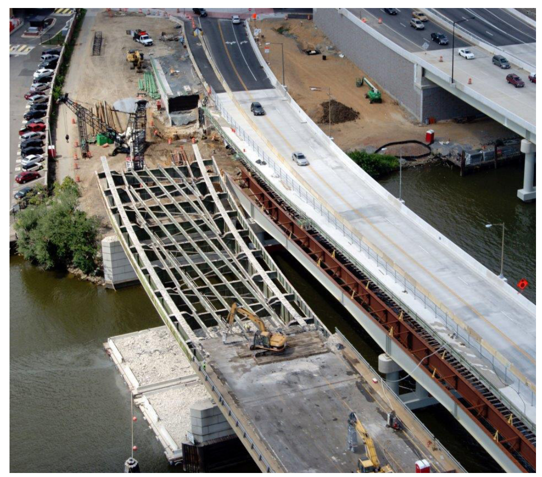
The 11th Street Bridge is a 916 ft-long, five-span continuous steel I-girder bridge. Using 1,663 tons of steel (1,446 tons for fabricated I-girders and 217 tons for cross frames), it has spans of 170 ft, 170 ft, 234 ft, 171 ft and 171 ft; the longest span is located over the navigable channel. The superstructure framing includes sections of splayed, kinked, skewed and horizontally curved girders. To comply with project aesthetic requirements, the girder webs linearly vary in depth from approximately 76 in. within the positive moment regions to 108 in. within the negative moment regions, transitioning from the bolted field splices to the piers. The cross frames are inverted K-frames with a top chord, and the members are shop welded to the gusset plates and field bolted to the connection plates. The cross frames vary in spacing from 19 ft to 25 ft, are contiguous between bays, and are oriented perpendicular to the girders. The bearings are high-load, multi-rotational (HLMR) using a combination of non-guided expansion, guided expansion and fixed types. The substructures are oriented at 90° to the construction baseline.

Each girder line in the framing plan includes nine field sections and eight bolted field splices (FS) located near the dead load inflection points; the field splices are numbered sequentially from FS1 to FS8. Girder lines (G) are numbered from G1 to G7 (see plan on following page).

The girder spacing varies from 10 ft to nearly 13 ft to accommodate the flared geometry at each end of the bridge, and the out-to-out width varies from approximately 68 ft to 75 ft. The bridge supports four 11-ft lanes of vehicular traffic and a 17-ft-wide, multiuse sidewalk.

Stage 2 framing and closure bay with an upward load on G1 (jack stand) and downward load on G1 (concrete block).

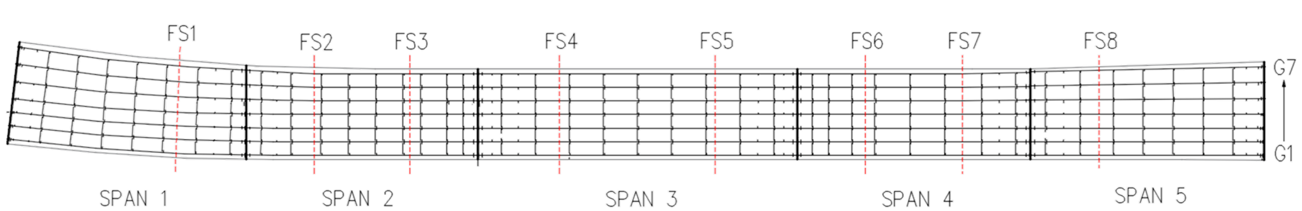
A conflict between the existing bridge (left) and proposed bridge (right); notice the discontinuous girder lines (G1 and G2) on the proposed bridge.

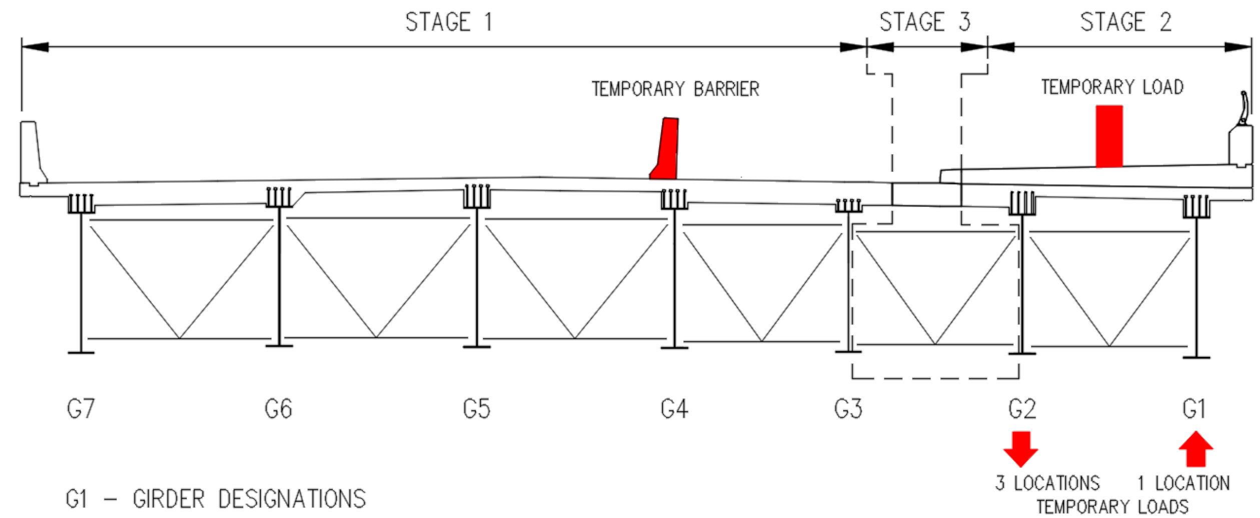


Using 1,663 tons of steel (1,446 tons for fabricated I-girders and 217 tons for cross frames), the bridge has spans of 170 ft, 170 ft, 234 ft, 171 ft and 171 ft.

Girder spacing varies from 10 ft to nearly 13 ft to accommodate the flared geometry at each end of the bridge.

The framing plan.





A typical section for the bridge (illustrating sequence of construction).

Setting the Stage

Staged construction is defined in accordance with AASHTO LRFD as the situation in which the superstructure is built in separate units with a longitudinal joint. This is to be distinguished, and held separate from, the longitudinal deck placement sequence.

The originally proposed sequence of construction for the 11th Street Bridge required all seven girders to be erected and the deck placed using a typical, longitudinal deck placement sequence over the full width of the structure. The bridge sidewalk and barriers were then to be constructed following the placement of the deck. As such, the original design of the bridge did not include considerations for a staged sequence of construction. However, during construction, the contractor decided to re-sequence the maintenance-of-traffic plan for the project, requiring a change in the sequence of construction for the bridge.

But the girders and cross frames were already fabricated and substantially erected based on the original sequence of construction. Therefore, the bridge had to be reanalyzed, not only to check the girders and cross frames for strength, but just as importantly to check the girders, cross frames and deck for camber shape and for relative positioning and fit-up between the stages of construction—all while minimizing the changes to the structural steel.

Another challenge associated with re-sequencing the bridge construction was that the last field sections in Span 1 for girderlines G1 and G2 would not be erected until after the existing 11th Street Bridge was removed; this was due to a spatial conflict between the existing and proposed bridges. As the AASHTO/NSBA Joint Collaboration guideline G13.1-2011 *Guidelines for Steel Girder Bridge Analysis* indicates: "On continuous bridges, girder deflections are influenced by adjacent spans. Just as the presence of girders in one span reduces the deflections in the adjacent spans, when the girders in an adjacent span are not present, deflections are greater." This phenomenon ultimately led to the implementation of a full-length closure bay to separate the G1 and G2 girder system from the G3 through G7 girder system in the revised sequence of construction.

Rethinking the Plan

Several options were assessed to accommodate the re-sequencing, and the most viable to minimize structural steel changes was to implement a three-stage sequence of construction. Since most of the girders and cross frames were already erected, the decision was made to disconnect the cross frames between G2 and G3 and introduce a closure bay between the two girders, thus creating a five-girder (Stage 1) and two-girder (Stage 2) system, separated by a full-length, longitudinal deck closure pour (Stage 3). This was necessary in addressing the predicted high stresses in the cross frames based on the analysis of the system without the Stage 3 closure pour. The high stresses were the result of the five-girder system deflecting downward under the loads of the deck and traffic, while the two-girder system remained partially erected and for the most part, unloaded.

The revised sequence largely mitigated predicted adverse force effects for the in-place girders and cross frames, but did not fully address the challenges with predicted deflections and relative positioning for fit-up between Stages 2 and 3—a result of the girders and cross frames not being originally detailed for the deflections associated with the revised sequence of construction, as well as the girders in Stage 2 not being fully erected. The design team developed a solution using both temporary and permanent loads strategically placed to allow fit-up of the cross frames and deck in the closure bay once the last field sections for G1 and G2 were erected. The sequence of loading involved the following:

- The traffic barrier, used for the maintenance of traffic in Stage 1, was temporarily relocated in Stage 2 over G4 in Span 5 and over G4 in the remaining spans.
- An opposing temporary force-couple of 30 kips upward force in G1 and 20 kips downward force in G2 was applied in Stage 2 using temporary loads. This was done

- to counteract G1 and G2 from twisting away from G3 due to curvature effects and the lack of torsional restraint, since the cross frames were disconnected in the closure bay.
- Temporary downward forces of 20 kips each were applied in Stage 2 on G2 in Spans 2 and 4. These loads were applied to the non-composite girder section to assist in aligning G2 in Stage 2 with G3 in Stage 1. The principle of structural continuity was leveraged by loading one span and obtaining the required deflection in another span.
- A temporary, composite, uniform load was applied on the deck in Stage 2, Span 2 between G1 and G2 using concrete traffic barrier.
- The permanent sidewalk was placed partial-width in Spans 1, 3, 4 and 5 and omitted in its entirety in Span 2 during Stage 2, to further control the relative positioning of G2 with respect to G3.
- The sidewalk barrier was initially constructed only in Spans 4 and 5.

Following the application of the strategic loading, the cross frame connections between G2 and G3 were largely constructable in their relative positions. For the cross frame connections that were out of alignment by more than 3/8 in., details were developed to use air-arc gouging to remove the welds connecting the cross frame members to the original gusset plates, and then replace the existing gusset plates with new pre-drilled gusset plates. The cross frames in the closure bay would first be connected to G2, Stage 2. Subsequently, the new, pre-drilled, gusset plates would be bolted to the connection plates on G3, Stage 1, and then the cross frame members would be field welded to the new gusset plates to achieve the final connection between Stages 1 and 2.

Once all of the cross frames between G2 and G3 were installed, the Stage 3 deck closure pour was placed and the temporary loading in Stage 2 was removed. The remaining portions of the sidewalk and sidewalk barrier were then finished, thus completing the bridge's construction.

Redistributing the Load

During the staged construction operations, the existing cross frame bolted connections were reassembled at all but three cross frame locations within the closure bay. These cross frames were located in Span 2 within the zone where the analysis predicted the largest differential deflections.

As a result of the sequence change, load redistribution occurred within the girder and cross frame system. When compared to the results in the original sequence, some members experienced larger forces, while other members experienced smaller forces. In combination with the countermeasures to achieve fit-up at the connections, 13 out of 276 cross frames within the framing system required retrofit to resist the redistributed larger forces, and limited zones within the bottom compression flanges of G4 and G6 required lean-on bracing to address lateral torsional buckling, since these girders were experiencing higher moments from the redistributed forces.

Modifying the Model

In order to evaluate the effect of the revised construction sequence on the various bridge components, the analysis considered the sequence of loading, the magnitude of loading and the time-dependent stiffness of the girders and deck system, as well as the lateral bracing conditions, during each stage.

The analysis of the staged sequence of construction used the original 2D design model with the necessary adjustments. Due to the complexity of the sequencing of the loads and the ultimate introduction of temporary loadings in Stage 2 construction, multiple design models were required to successfully predict the behavior of the system. Since the behavior of the girders and cross frames remained in the linear elastic range, the individual model results for stresses and deflections were combined using the principle of superposition to predict the behavior during each stage of construction and in the final configuration. Ultimately, nine models were developed, and the results superimposed to achieve the interim and final stresses and deflections.

Successful Re-sequencing

The girder and cross frame fit-up was achieved by strategically using permanent and temporary loads including concrete barriers, concrete sidewalks, concrete block counterweights and a hydraulic jack system to bring the structural steel framing of the two independent stages into relative position for connection. The finished deck slab geometry was achieved by using a full-length closure pour between Stages 1 and 2.

With the rigorous analyses, the revised sequence of construction was successful, achieving girder relative positions and a constructable means for cross frame fit-up. The deck closure pour was placed and the final deck geometry was achieved to obtain both the required structural depth and cross slope geometry of the deck.

Regardless of the complexity of the bridge, consideration of the sequence of construction is critical to reasonably predict the applied loads and the resulting forces and deflections within the system. Whether it is a simple-span tangent bridge built in stages or a multi-span continuous plate-girder bridge, the design must consider the sequence of construction. For bridges that are tangent and mildly skewed, the sequence of construction will dictate the load application on the line girders. For bridges that are curved or highly skewed, the sequence of construction will dictate not only the load application, but also how the dead load and live load forces are distributed through the 2D or 3D girder-and-cross-frame system. As seen with the 11th Street Bridge, re-sequencing the construction caused a redistribution of the loading in the girders and cross frames. Diligently developing the loads for each stage of construction and recognizing the sequential stiffness of the system led to a reasonable prediction of the forces and deflections for girder, cross frame and deck fit-up—illustrating that sequencing indeed matters. ■

Owner
District Department of Transportation

General Contractor
Skanksa/Facchina Joint Venture

Structural Engineer
AECOM

conference preview

HIGH-PERFORMANCE STEEL BRIDGE COATING OPTIONS

BY ROBERT KOGLER AND LAURA ERICKSON

There's more than one way to coat a bridge.

INDUSTRIAL PAINT SYSTEMS have been and continue to be the workhorse corrosion protection system for steel highway bridges.

For about the first 100 years of steel bridge construction, paint systems consisted of primarily simple, single-package, easy-to-apply, inexpensive, lead-containing paints. The lead pigment served as a corrosion inhibitor, and these coatings were easy to use in both new construction and maintenance painting applications. They were typically applied directly over intact mill scale and were used as a "one-size-fits-all" corrosion protection system.

Several key factors came together during the 1970s and 1980s to force the evolution of bridge painting systems toward the much more durable systems in use today. The advent of high-production centrifugal blasting equipment coupled with increased demands by bridge owners for durability allowed for truly clean, profiled surfaces for paint application—thus opening the door for use of high-performance coating systems. Additionally, concerns over environmental and worker health and safety issues associated with lead-containing paints helped force change.

Specifically, zinc-rich coating systems eventually became the standard due to their greatly improved performance in salt-rich environments. With the continuous pressure on owners to maintain open roads and "dry pavement" at all times in all seasons, the use of deicing chemicals increased the demands on corrosion protection systems nationwide. These factors conspired against the older steel bridges painted with no surface preparation and mediocre paint. When the use of deicers increased dramatically, these older systems were ill-suited to perform for long periods of time, and the condition of the steel bridge inventory suffered. However, for those structures built or repainted more recently with modern paint systems, performance has dramatically improved. So it is important to note that when considering design options for new or replacement bridges, the historical corrosion protection performance of a "painted steel bridge" in a specific environment will likely not be representative of the improved performance expected from a more modern "high-performance coating system" in the same bridge today.

Zinc-Rich Systems

The shift to zinc-rich coatings as the primary steel bridge corrosion protection system has greatly increased the performance of painted steel in salt-rich environments. This includes bridges located on the coast or exposed to chemical-containing runoff, drainage and traffic splash in areas that receive significant deicing treatment in the winter. While real-time data regarding performance of modern paint systems is difficult to find, there is a significant body of published information (from the American Galvanizers Association, the Society for Protective Coatings, the Federal Highway Administration, the American Society for Testing and Materials and others) indicating that better zinc-rich paint systems last 20 years or longer in harsh marine environments (and likewise in the areas and details of non-marine bridges that are directly impacted by deicing chemicals). In fact, FHWA recently revisited the "Corrosion Protection of Steel Bridges" section of the *Steel Bridge Design Handbook*, Volume 19 specifically to enhance the discussion on performance of modern bridge coatings. This revision is presently in final review and should be published in late 2015.

Also important to modern coating performance is the fact that "failure" of these types of sacrificial paint systems is typically localized on the structure. Except for in the harshest marine exposures, there are usually specific areas of the structure that show coating breakdown and corrosion first, before the vast majority of the steel. These "micro-environments" concentrate the factors that drive coating breakdown and corrosion. The areas directly beneath failed or open deck joints, members directly in the way of traffic splash or details that tend to trap


Robert Kogler (bobkogler@verizon.net) and **Laura Erickson** (lauraerickson4@gmail.com) are both with Rampart, LLC.

and hold debris, moisture and salts are usually the leading areas for failure. By identifying these areas in maintenance practices, the life-cycle maintenance burden of the structure can be focused on and greatly reduced when compared to the traditional approach of a regular blast and repaint cycle for the entire bridge, which is taken with so many older structures.

One Size does not Fit All

For continued progress in corrosion protection, bridge owners must get over the mindset that there is only one approach for corrosion protection. Many states have maintained a list of several different acceptable paint systems over the years. Typically, the various systems are targeted toward different required levels of durability, and for states that have many bridges in areas that are far from natural salt water and do not deice, this approach seems like a rational way to decrease the cost associated with coatings on the lower performance end. However, with ever-increasing performance demands in more corrosive applications, owners are increasingly looking toward use of hot-dip galvanizing and metalizing to enhance steel corrosion protection in a more targeted manner. The data available for both galvanizing and metalizing show excellent long-term performance, even indicating up to 40 years of protection for metalized exposed steel in marine environments. For many structures, this level of performance represents the potential for a "life of structure" corrosion protection system applied on new construction. That value proposition is gaining recognition within the owner and fabricator community, particularly for bridges in severe marine environments.

The next logical step in this evolution of coatings is to move toward the application of corrosion protection systems to specific bridge elements on an as-needed basis. That is, the areas of the bridge expected to be impacted by high levels of salt and moisture can be constructed with an appropriately durable coating system, while other areas expected to have a far less severe service environment can be fabricated with a less costly (and more efficiently constructed) system. Some possibilities include:

- priming interior girders with zinc-rich coatings and applying topcoats to fascia beams only
- preferential galvanizing or metalizing of bridge elements or areas known to have more corrosion incidents than the bulk of the bridge (e.g., beam ends under joints or horizontal members)
- the use of topcoats over galvanizing and metalizing in very aggressive environments

Corrosive Environments and Design Detailing

Bridge corrosion protection design must consider not only the macro-environment (e.g., marine, heavy deicing, urban, rural, etc.) but also, and perhaps more importantly, the micro-environments created by the detailing of the structure—e.g., the specifics of designed drainage, unintentional (but likely) life-cycle drainage paths caused by failed deck joints and splash created by traffic (both vertically and laterally) by considering these areas up front in the design process, these potential problem areas can be minimized or addressed specific high-durability coating treatments if not fully eliminated.

This general approach has already become increasingly popular for primarily aesthetic reasons. The beam ends of weathering steel bridge members are frequently painted as a risk mitigation measure for anticipating deck joint failure or to prevent staining of concrete in the vicinity. Also, fascia beams are frequently painted for aesthetics while the remainder of the members (out of obvious public view) are left as bare weathering steel. This general approach of selective application could provide a benefit for the many bridges constructed in non-marine areas that have only specific areas and details expected to require periodic maintenance repainting.

Details to Consider (and Avoid)

There are many steel bridge details on existing structures that have played a role in the initial failure of coating systems and have driven the need for maintenance actions. Built-up riveted members and boxes with lacing bars used in older designs tend to trap moisture and debris, causing coating breakdown and pack rust, and are notoriously difficult to clean and re-coat. The good news is that the majority of these details are no longer frequently employed on modern steel bridge designs, and there are a few items to consider that can provide a significant long-term benefit.

For example, splice and cover plates should be designed to consider as-constructed drainage paths for salt carrying water on flanges. The leading edges of these plates can either act as a dam and collection area for debris or, depending on fabrication angle, as an effective "drip bar" helping to move water off of the steel. Snipes in stiffeners have the same issue. A snipe small enough to easily become clogged with debris over time will create a small, focused area of coating failure and eventual corrosion. Welds should not leave small gaps between members that may serve as moisture traps to initiate corrosion. Smaller cross frames should also be placed in such a manner that allows proper access for blasting, painting and inspection (at least several inches apart).

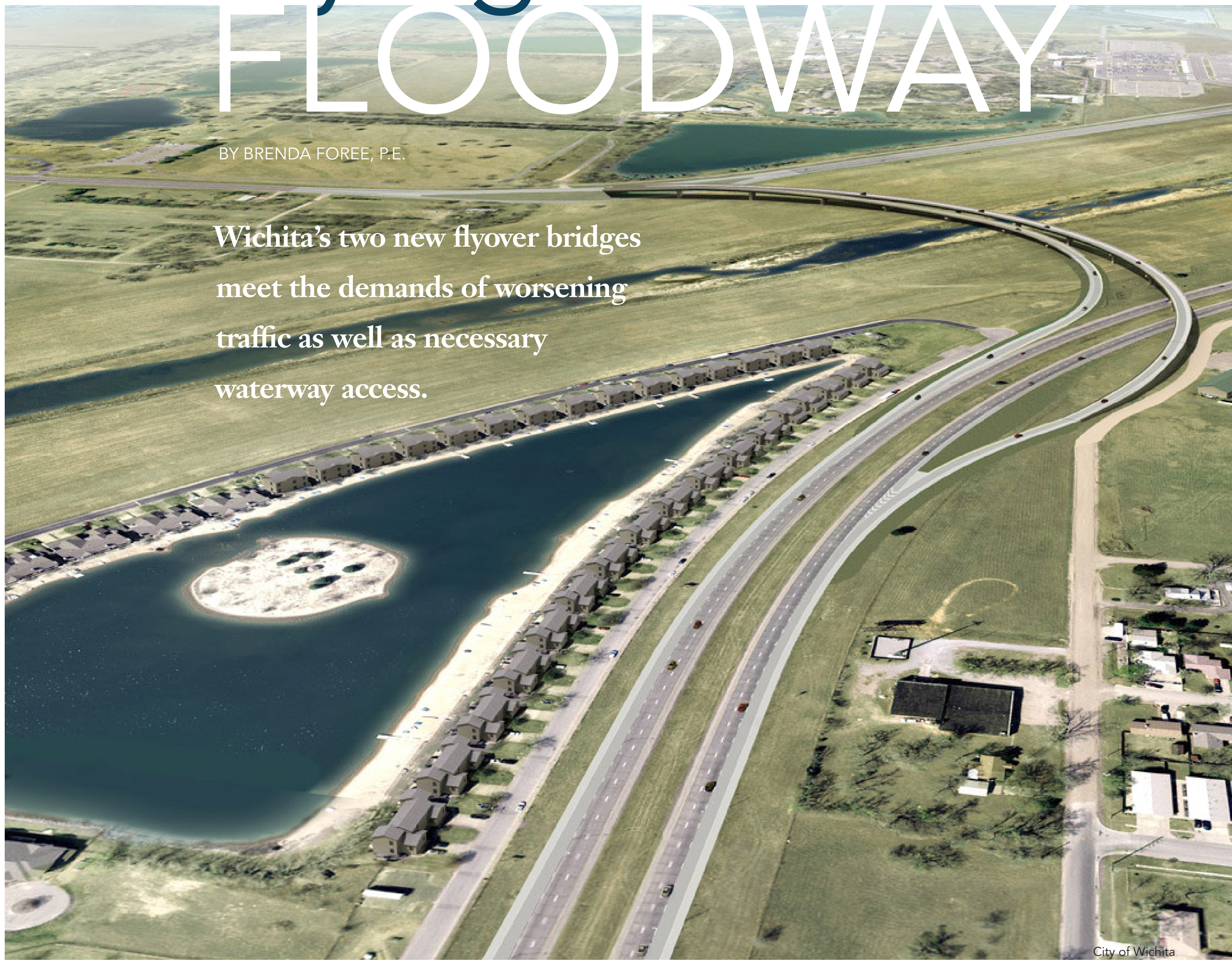
Long-term durability in modern steel bridge design requires consideration of the global or macro-environment for the bridge location, but also important is the use of proper selection of modern, high-durability coatings and considerate design detailing to mitigate areas and details that present known risks for corrosion initiation. The high level of performance of modern zinc-rich coatings is significant when compared to the older "paint-over-the-mill-scale" approach, which has created the recent maintenance burden in the existing bridge inventory. ■

This article is a preview of Session B6 "High-Performance Steel Bridge Coating Options" at NASCC: The Steel Conference, taking place March 25-27 in Nashville. Learn more about the conference at www.aisc.org/nascc.

Flying Over the FLOODWAY

BY BRENDA FOREE, P.E.

Wichita's two new flyover bridges meet the demands of worsening traffic as well as necessary waterway access.



City of Wichita

▲ The two flyover bridges opened this past November in Wichita.

TWO NEW FLYOVER BRIDGES, which sweep gracefully over a Wichita, Kan., floodway, provide a welcome relief for commuters from decades of traffic congestion—while also preserving critical flood protection for the community.

The structural steel plate girder bridges—2,273 ft long and 1,690-ft long, respectively—are part of a new partial interchange with 13th Street and Interstate 235 in this city of nearly 400,000. The bridges were integral to the design of a suitable alternative for meeting traffic demands in a highly constrained area and to achieving the city's aesthetic objectives.

Before the opening of the $24 million interchange this past November (five days ahead of schedule) Wichita commuters were routinely suffering bottlenecks accessing the limited crossings over a large flood protection channel that separates the city from newer development to the northwest.

Brenda Foree (**bforee@hntb.com**) is a project manager with HNTB Corporation.

One of the bridge assemblies in the field, which starts at pier 6 and ends at pier 8. The assembly is nearly 534 ft long, with a maximum elevation of over 24 ft and a maximum sweep offset of nearly 39 ft.

For more than two decades, the City of Wichita had sought solutions to relieve this worsening traffic congestion at the I-235 interchange with Zoo Boulevard, which provides access across the Wichita-Valley Center Floodway, known locally as the "Big Ditch." The Floodway is a manmade channel on the west side of Wichita that was constructed in the 1950s to protect a large portion of the city from flooding by intercepting and diverting flow from the Arkansas River and other Sedgwick County watercourses.

To address the safety issue of traffic accessing Zoo Boulevard backing up onto I-235 during peak hours, the bridge across the Floodway at the I-235/Zoo Boulevard interchange was reconstructed in 2000 with additional left-turn lanes. Bottlenecks remained, however, and given the proximity to the Floodway, a pedestrian/pipeline bridge and railroad tracks would not allow any other major improvements to that interchange. The city thus turned to other options.

Traffic studies indicated that a new crossing over the Floodway would help alleviate congestion in the area. A new partial access interchange at 13th Street and I-235, between Central Avenue and Zoo Boulevard, and over the Floodway was determined to be the most appropriate solution for congestion relief, given the constraints of the area. The Kansas Department of Transportation (KDOT) and the Federal Highway Administration (FHWA) approved this break in access for I-235 after a study demonstrated that the proposed improvements would improve safety and operations on I-235 and improve local access.

Even with this new option, the City still had to contend with several challenges. The design team had to carefully locate the flyover bridges over 1,000 ft of floodway and around its levees, as well as around

The bridges use 2,875 tons of structural steel.

The exterior steel girders employ two different-colored elements constructed from HSS6×3×5/16 to visually subdivide the girder mass.

Both bridges are 32 ft, 6 in. wide, with four plate girders spaced at 8 ft, 8 in. The girders have 84-in.-deep webs, 2-in.-thick flanges and lengths ranging from 100 ft to 110 ft (the average girder weight is 15 tons to 20 tons).

I-235, other roadways, a lakeside residential development and a county park.

The Right Pace

The 45-mile-per-hour design speed was a major factor in setting the bridge's geometric features, such as longitudinal grades, super-elevation rates and curve radii. Steel plate girders were chosen as the preferred structure type early in the preliminary design process due to the bridges' horizontal curvature and span lengths up to 225 ft.

Flyover NB235-WB13 connects northbound I-235 to westbound 13th Street, while flyover EB13-SB235 connects eastbound 13th Street to southbound I-235. Flyover NB235-WB13 is a 12-span bridge on a 1,150-ft radius with a total bridge length of 2,273 ft. The south unit crosses I-235 with spans of 155 ft, 200 ft and 155 ft. The center unit passes over the east levee and Floodway channel with spans of 157 ft, 215 ft, 215 ft, 215 ft and 167 ft. The west unit spans the west levee and 13th Street to Windmill Road Ramp. West unit spans are 190 ft, 225 ft, 210 ft and 169 ft.

Flyover Ramp EB13-SB235 is a nine-span bridge on a 950-ft radius with a total bridge length of 1,690 ft. The west unit crosses over 13th Street to Windmill Road Ramp and the west levee with spans of 145 ft, 180 ft, 225 ft and 190 ft. The south unit passes over the Floodway channel and east levee with spans of 156 ft, 212 ft, 212 ft, 225 ft, and 145 ft.

Both bridges are 32 ft, 6 in. wide, with four plate girders spaced at 8 ft, 8 in., and the girders have 84-in.-deep webs, 2-in.-thick flanges and lengths ranging from 100 ft to 110 ft (the average girder weight is 15 tons to 20 tons). Weathering steel was chosen to minimize future maintenance requirements, and the project uses 2,875 tons of structural steel.

After fabricating the girders, each unit required a full vertical inspection. Essentially, the bridge had to be erected in the shop using falsework (mock piers and blocking points), including the installation of cross frames and diaphragms, to verify that the elevations, camber and horizontal sweep of the bridge would match in-field conditions. These assemblies reached up to 600 ft in overall length with an overall elevation difference of 30 ft, and were able to get within ¼ in. over spans exceeding 500 ft. In order to verify all dimensions, a transit was used to verify horizontal offsets and camber. Before each assembly began, the outdoor assembly bay was surveyed and re-leveled in order to properly assemble the units.

Floodway and Levees

Construction had to accommodate the needs and requirements of the City of Wichita/Sedgwick County Flood Control Section, which is responsible for maintaining the Floodway in accordance with standards established by the U.S. Army Corps of Engineers (USACE). Construction could not restrict access to the levees for maintenance and emergency vehicles, but the bridges' vertical profile allowed for an access road to be constructed on top of the west levee and on the dry side of the east levee. Temporary bents for steel erection were allowed to be located on the levees since emergency access was available from Zoo Boulevard (north of the project) and Central Avenue (south of the project). Vertical bridge profiles were set to provide room for an access road on top of the levee at three crossings, and the access road was placed adjacent to the dry side of the levee at the fourth crossing.

Two mechanically stabilized earth (MSE) walls are located at the west end of the bridges, and one MSE wall is located at the south end of Flyover NB235-WB13. Additional project improvements included addition of acceleration/deceleration lanes to I-235 between Central Avenue and the flyover ramps, realignment of 13th Street/Windmill, arterial intersection improvements at two locations, roadway improvements to Hoover Road and Lakewind Street and revised access to the Sedgwick County Park.

Bridge piers were located a minimum of 20 ft from the toe of the east and west levees in order to avoid impacts to the integrity of the levee system. In addition, USACE required a geotechnical seepage analysis be completed for bridge piers adjacent to the dry side of the levees to demonstrate that the piers would have no substantive impact upon seepage potential through or beneath the existing levees.

The team performed a hydraulic analysis, which determined that the project would have only a minimal effect

upon the capacity of the Floodway. The minor rise of the base flood profile attributed to the bridge piers was acceptable because the existing levees are sufficiently high to contain floods exceeding the 500-year event.

Bridge Aesthetics

The City of Wichita was committed to creating visually appealing bridges with clean, elegant lines, and the bridges' steel components helped to achieve this goal. Project designers worked with artist Greg Turner to develop an aesthetic concept that was approved by the Wichita Design Council. The two flyover bridges sweep across the Floodway in an ever-widening curve inspired by the flared pier design. Curved rustications in the pier face recall floodplain grasses bending in the breeze, and the exterior steel girders employ two different-colored elements constructed from $HSS6\times3\times\frac{5}{16}$ to visually subdivide the girder mass. These elements extend from abutment to levee, to be seen from adjacent viewpoints, and represent the colors of the changing Kansas seasons.

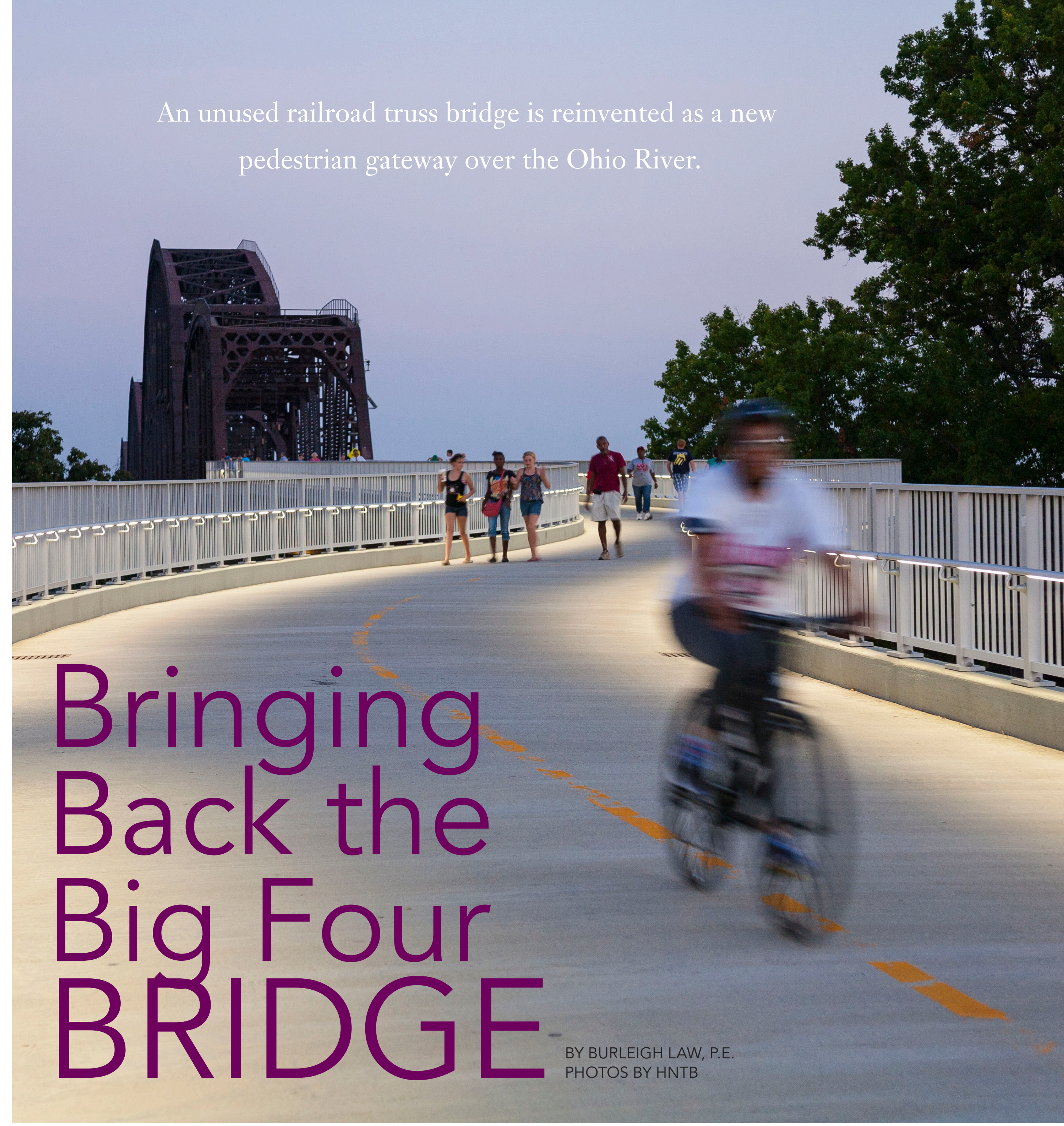
Project retaining walls and abutment wingwalls have a ribbed finish and are tan in color. The retaining wall along Windmill Road has a stylized impression of a windmill, clouds and birds, and southbound Windmill Road runs parallel to the west end of Flyover NB235-WB13, giving a prolonged view of the artwork on the retaining wall. ■

▲ Weathering steel was chosen to minimize future maintenance requirements.

Owner
Kansas Department of Transportation

General Contractor
Dondlinger and Sons Construction Company, Inc., Wichita

Structural Engineer
HNTB Corporation, Overland Park, Kan.



An unused railroad truss bridge is reinvented as a new pedestrian gateway over the Ohio River.

Bringing Back the Big Four BRIDGE

BY BURLEIGH LAW, P.E.
PHOTOS BY HNTB

THE BIG FOUR BRIDGE had a big name to live up to.

Built in 1885 and replaced in 1929, the 2,525-ft-long six-span railroad truss bridge was named for the now defunct Cleveland, Cincinnati, Chicago and St. Louis Railway—also known as the Big Four Railroad—and carried a single track over the Ohio River between Louisville, Ky., and Jeffersonville, Ind. The replacement bridge operated for four decades before falling into disrepair and was eventually deemed a safety hazard. Rail operations ceased in 1969, when rail traffic was rerouted to another bridge, and the approach spans were removed and sold for scrap.

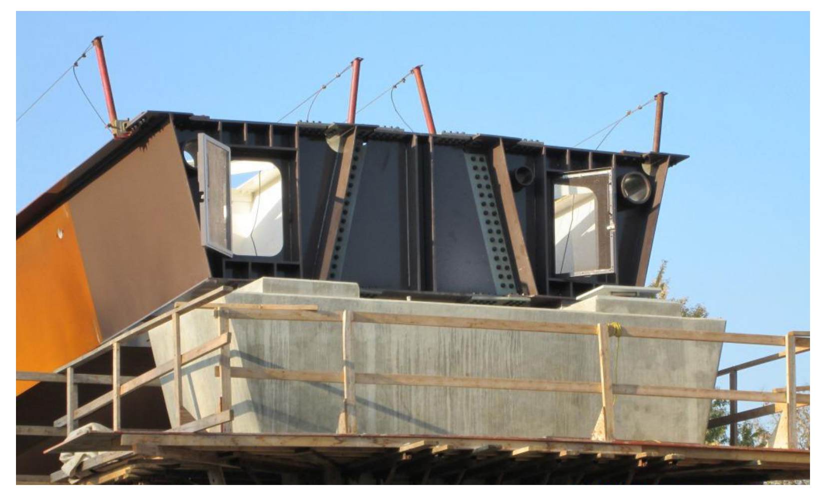
For decades, the bridge was unused, with no access to the main span sitting atop piers that rose 50 ft in the air, earning the bridge the unfortunate nickname of "the bridge to nowhere." The Louisville Waterfront Development Corporation acquired the bridge in 2005 with the goal of converting it into

Burleigh Law
(blaw@hntb.com)
is a senior project engineer with HNTB Corporation.

While steel box girders are often used on curved long-span highway bridges for stability and structural efficiency, they are not generally used on pedestrian bridges. However, this girder type was chosen for the Jeffersonville approach because the girder fascia and bottom soffit create a streamlined look through the S-curve span and the sharp 90° curve at the end of the bridge.

A stair tower (50 ft from top of footing to highest point) provides access to the Ohio River Greenway and Jeffersonville waterfront 1,000 ft south from where the bridge lands at the Big Four Station.

The portions of the girders immediately above the piers are painted to prevent staining caused by runoff from the weathering steel as it develops its patina.



Special details at the pier diaphragms ensure easier passage of the drain piping and conduit.

a pedestrian bridge. At the time, pedestrian and bicyclist access over the Ohio River was accommodated by the Clark Memorial (2nd Street) Bridge on the other side of the Interstate 65 bridge, but the sidewalks and shoulders were narrow and adjacent to fast-moving vehicular traffic.

The newly constructed and rehabilitated 21-ft-wide bridge is designed for a 75-year life for pedestrian loading as well as emergency vehicles. The Jeffersonville approach is 1,240 ft long (1,033 ft of bridge and 207 ft of fill approach), the main span is 2,547 ft long and the Louisville approach is 1,181 feet long (693 ft of bridge and 488 ft of fill approach). The bridge was completed in phases, with the Louisville approach opening in 2010 and the main span truss being rehabilitated in 2013.

Outside the Box (Girder)

The last portion, the curvaceous Jeffersonville approach—designed by HNTB—opened just last year, completing the crossing. While steel box girders are often used on curved long-span highway bridges for stability and structural efficiency, they are not generally used on pedestrian bridges. However, this girder type was chosen for the Jeffersonville approach because the girder fascia and bottom soffit create a streamlined look through the S-curve span and the sharp 90° curve at the end of the bridge. There are two steel box girders and, given the length of the spans, field splices were required for fabrication, shipping and erection. The 60-in.-deep box girders for the Jeffersonville approach had spans of 128.5 ft, 128.5 ft, 160.67 ft and 160 ft for Unit 1 and143 ft, 160 ft and 143 ft for unit 2.

The 60-in.-deep box girders for the Jeffersonville approach had spans as long as160.67 ft.

The girders also conceal utilities, which would have detracted from the clean lines if mounted on the outside. Each girder contains an 8-in.-diameter drain pipe and four 2-in.-diameter conduits, including junction boxes and hanger assemblies, all of which run the length of the bridge. The internal intermediate cross frames are standard and the placement of the drain pipes, conduit and hanger assemblies are placed to fit around these. Special details at the pier diaphragms ensure easier passage of the drain piping and conduit. Holes are provided in the bottom flange near the end bent for the drain pipes to exit the bridge and tie into a storm water system, and intermittently spaced standard 2-in.-diameter vent holes with "critter screens" are provided in the webs and bottom flanges to prevent moisture accumulation in the girders.

The girders also house the utilities, which would have detracted from the clean lines if mounted on the outside.

Joining the new approach to the historic main span of the bridge.

The S-curve alignment was selected to minimize utility and right-of-way impacts, cost and coordination, allowing the bridge to avoid historic homes and other buildings along the east side of Mulberry Street. The approach crosses over streets, a proposed future canal and an existing floodwall, which was modified with a wider opening to better connect the new bridge with the Ohio River Greenway trail. To avoid ending the bridge at an intersection, a green space—the Big Four Station—was designed with fountains, a stage, pavilions and playgrounds.

To accommodate a 54-ft elevation change, ADA considerations dictated a long bridge with a constant 4.79% grade. Post-tensioned box girders were initially

The Jeffersonville approach is 1,240 ft long.

OBEC Consulting Engineers

The aptly named Jughandle project in suburban Portland eases traffic flow through one of Oregon's biggest bottlenecks.

Getting a HANDLE on Traffic

BY BOB GOODRICH, P.E., AND JASON KELLY, P.E.

THE INTERSECTION OF OR 213 AND WASHINGTON STREET in Oregon City, Ore., had the distinction of being the state's busiest signalized intersection.

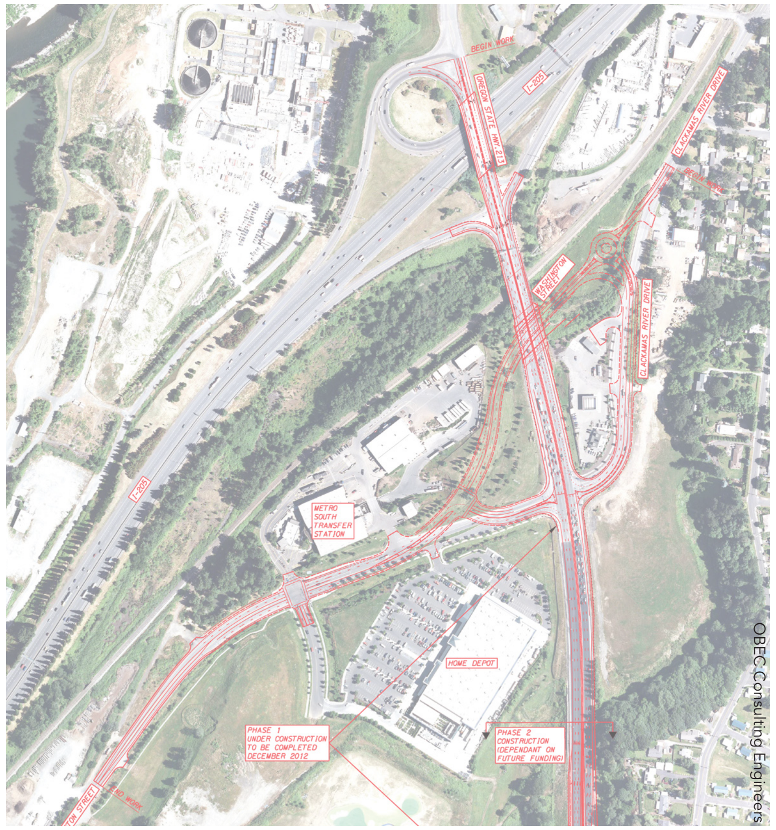
It's near the northern end of the OR 213 corridor, which stretches from Salem to this southern suburb of Portland. With an average daily traffic (ADT) count of 65,000 vehicles, OR 213 is one of the state's busiest transportation corridors and until recently struggled to accommodate this high capacity.

Luckily, relief has been provided in the form of the $25 million OR 213: I-205 to Redland Road project (also known as the Jughandle project), which involved building a new bridge along OR 213 and realigning Washington Street so that it now passes under the highway—thus creating new, safer traffic patterns. Left turns have now been eliminated at the intersection and additional travel lanes have been added, thereby increasing capacity and separating out traffic merging onto the adjacent I-205 freeway. And a new roundabout, which accommodated the traffic passing under OR 213, avoided the need to add a signal.

Pencil Sketch

The Jughandle concept started in 2007 as a rough pencil sketch by the traffic engineer, Hermanus Steyn, of Kittelson and Associates, of how to improve OR 213 with limited funding. The project was initially led by a private developer looking to build on an adjacent property, but with the downturn in the economy in 2009, they decided not to proceed. However, Oregon City staff,

Bob Goodrich (rgoodrich@obec.com) is the bridge division manager at OBEC and has more than 15 years of experience designing and managing bridge projects. **Jason Kelly** (jkelly@obec.com) is a construction project manager and leads OBEC's construction and inspection work in northern Oregon and southwest Washington. He has more than 12 years of engineering experience, with an emphasis on bridge projects.



This project plan view from the design phase illustrates the various changes the project would implement, as well as potential future improvements (depending on available funding).

An aerial view of the overall project site illustrates the reworked interchange, including the new roundabout and the new bridge to the west.

The completed OR 213 bridge, as seen from Washington Street below. The new roundabout can be seen in the distance, directly east of the bridge.

recognizing the vital importance of this project, picked it up and ultimately secured state and federal funds to see it completed.

In order to eliminate the left-hand turns at the OR 213/Washington Street intersection that were making congestion worse and creating unsafe conditions for drivers and pedestrians, the team knew it was necessary to extend Washington Street underneath OR 213 through a grade-separated undercrossing and connect it to S. Clackamas River Drive via a roundabout. Due to the very high traffic volumes and the proximity of the new bridge to an interchange with I-205, traffic staging and constructability required careful consideration. The project's structural engineer, OBEC, analyzed four traffic staging alternatives to construct the new bridge:

1. Full closure for the duration of construction.
2. A temporary detour alignment.
3. Close one lane in each direction and construct the bridge in three stages.
4. Implement accelerated bridge construction (ABC) and do a full closure for a very short duration (four days, give or take).

Closing the highway for the entire duration of construction would have impacted thousands of commuters and freight traffic for an extended period of time (as much as 60 days for total closure and 30 weeks for single-lane staged closures) by essentially closing a prominent interchange; impacts to the region and nearby businesses such as Home Depot and the Metro Transfer Station would have been too severe. Constructing a detour alignment was cost-prohibitive for several reasons, including crossing Union Pacific Railroad tracks and maintaining connections to the I-205 interchange. And constructing the bridge in stages would have still resulted in significant traffic impacts given the ADT and available capacity—and closing even a single lane during daylight hours would have created unacceptable traffic delays every day for the 12 to 18 months of construction. As such, ABC was ultimately selected as the preferred alternative. While it did come with a large impact—full closure of the highway for 104 hours to move the bridge superstructure into its final position—it balanced the variety of site constraints and resulted in the shortest overall project duration.

A shot from the live project webcam shows the new bridge, which was constructed along the existing highway, shortly before crews pushed it into place.

The new OR 213 bridge after it was moved into place and traffic was reopened four hours earlier than originally scheduled.

An aerial view of the project site (looking southwest) shows the project with excavation underway prior to pushing the new bridge into place.

Bridge Design

The new OR 213 crossing over Washington Street is a six-lane, single-span bridge with a multimodal walkway on one side and additional width for a future travel lane, for a total width of approximately 112 ft, and is 130 ft long; the clearance is 16 ft. The bridge was designed per 2010 AASHTO LRFD *Bridge Design Specifications* and in accordance with the Oregon Department of Transportation Bridge Design and Drafting Manual. The superstructure comprises nine steel plate girders and a conventional concrete deck. The girders, fabricated by Fought and Company, are made from ASTM A709 Grade 50 weathering steel and are 4 ft, 7¼ in. deep.

Because of the complexity and high-profile nature of the project, it was critical to employ a highly qualified contractor with a strong understanding of, and approach to, the project's challenges. To achieve this, the project team used an alternative bidding process that awarded the contract based on not only a) price but also a technical component consisting of b) qualifications and c) project approach (while not common, this method is used more with historic rehabilitations). Through this process, the project team selected Mowat Construction Company, who scored highly in all three areas. Mowat's approach generally followed the plan outlined by the design team, and its qualifications consisted of several ABC projects in Oregon and Washington State using a horizontal moving system similar to that which was to be employed on this project.

The project started with the bridge foundations, which were constructed at night during single-lane closures. A sheet pile shoring system was constructed across the highway on each side of both abutments, then the roadway was excavated between the shoring and covered using precast concrete panels to maintain traffic in all lanes during the day. The steel pipe pilings were then driven and were followed by the concrete pile caps, which were constructed during the day below traffic thanks to the shoring and panels.

Concurrent with foundation construction, the superstructure, consisting of the steel girders, concrete deck and bridge rail, was constructed during the day adjacent to the bridge's final position. Temporary steel piling and cap foundations, mimicking the permanent abutment skew and grade line, were constructed to support the superstructure until it was moved into position. Special care had to be taken during layout of the temporary foundations so that the alignment and grade of the final bridge location were an exact match. And the temporary foundation also had to not only provide vertical support but also a surface for jacking and rolling.

Upon completion of the bridge foundations and superstructure, the highway was closed for 100 hours (four hours shorter than originally scheduled) from a Thursday night at 7:00 p.m. to Monday night at 11:00 p.m. to complete the new undercrossing. During the first 24 hours of closure, crews relocated approximately 5,000 cubic yards of soil directly north of the bridge site to create an opening for the superstructure. Then the superstructure was pulled into place using a system of hydraulic jacks and rollers. It took approximately 24 hours to pull the superstructure more than 155 ft horizontally and lower it 18 in. vertically into position.

Naturally, it did not all go as planned. Some of the temporary foundation was in conflict and had to be removed. The hydraulic system that pulled the bridge into place could only move 18 in. per iteration. The jacking system also had a limited range of movement, requiring several iterations to lower the bridge. During the remaining 53 hours of closure, Mowat constructed the precast impact panels, bridge joints and asphaltic concrete transitions and reconfigured traffic signals to reopen the highway.

The 3,200-kip load was lifted using a system of 32 hydraulic jacks ranging from 50- to 70-ton capacities and controlled from a central manifold that moved with the bridge. The bridge was designed with an extra-large reinforced back wall that allowed for lifting and lowering and was then pulled into place using $1\frac{1}{4}$-in. coil rods actuated with twin 40-ton rams pulling against the permanent wing wall/thrust blocks constructed on one side; it rolled into its final position via 34 50-ton Hillman rollers placed inside a steel guide channel stretching 267 ft across both the temporary and permanent foundations. The bridge not only traversed laterally 155 ft but also vertically 2.5 ft due to the superelevation of the road.

Since opening in 2013, the Jughandle project has vastly improved mobility in the area, easing congestion and reducing delays. It may still be a busy interchange in terms of ADT, but it is certainly a safer and more efficient one. ■

Owner
City of Oregon City

Structural Engineer
OBEC Consulting Engineers Eugene, Ore.

General Contractor
Mowat Construction Company, Woodinville, Wash.

Steel Fabricator
Fought and Company, Tigard, Ore.

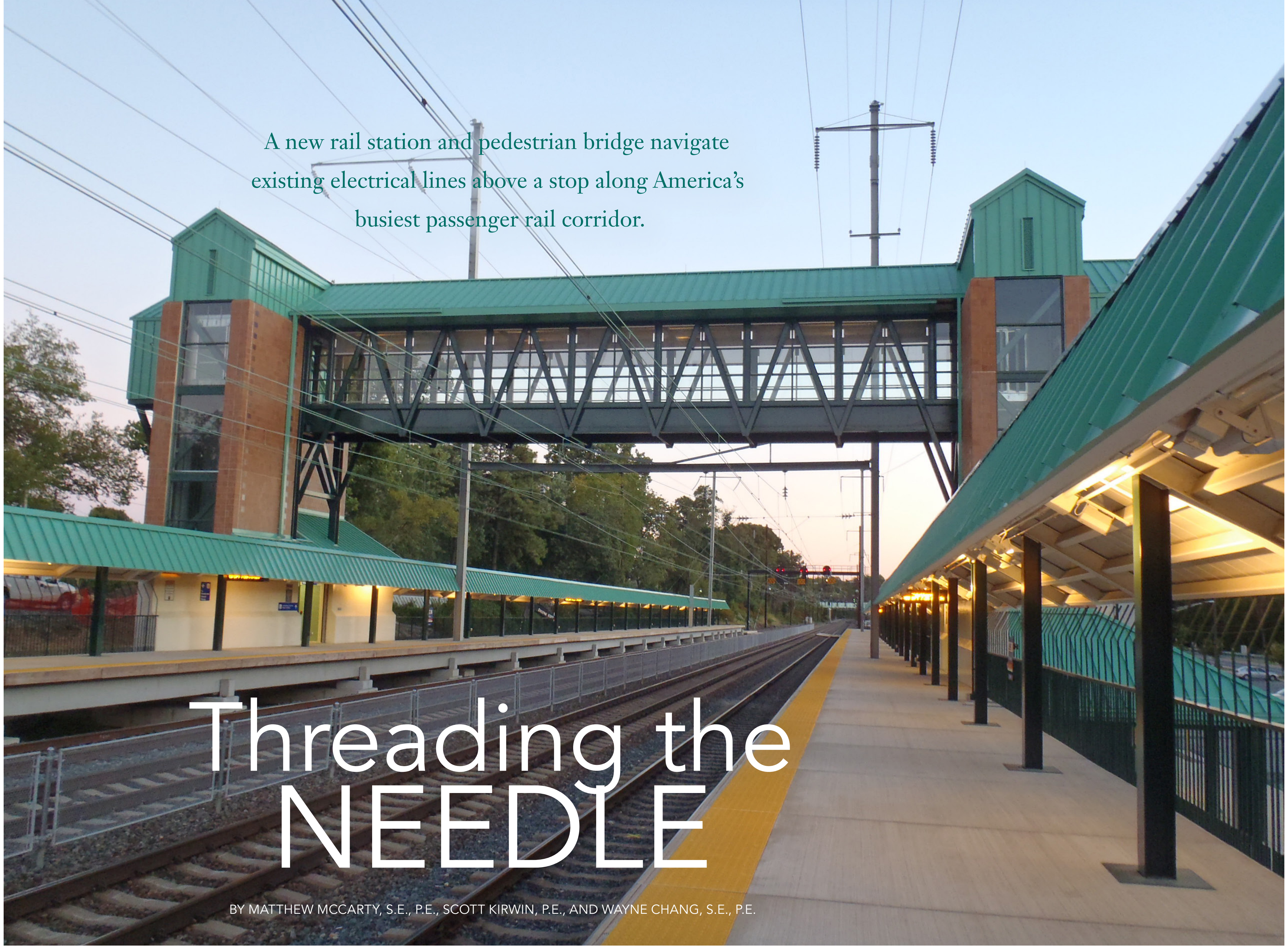
Public Outreach

Due to the high visibility of the project and the impact the four-day closure would have, the project team developed a robust and proactive outreach program to keep the public up to date on project progress and inform them of the impending closure. Extensive public involvement efforts included public meetings, a newsletter and a web page that featured a live construction camera for the duration of the project. Leading up to the four-day closure, the project team used extensive outreach (social media, print, radio and TV) to inform the public of delays in the area and point out available detour routes. Ultimately, the outreach was very successful, reducing traffic in the immediate area by 75% during the closure.

For a time-lapse video of the bridge move in action, go to **tinyurl.com/pkmg5dp**.


OBEC Consulting Engineers

▲ Crews place the girders of the new OR 213 bridge. The girders are fabricated from ASTM A709 Grade 50 weathering steel and are 4 ft, $7\frac{5}{8}$ in. deep.

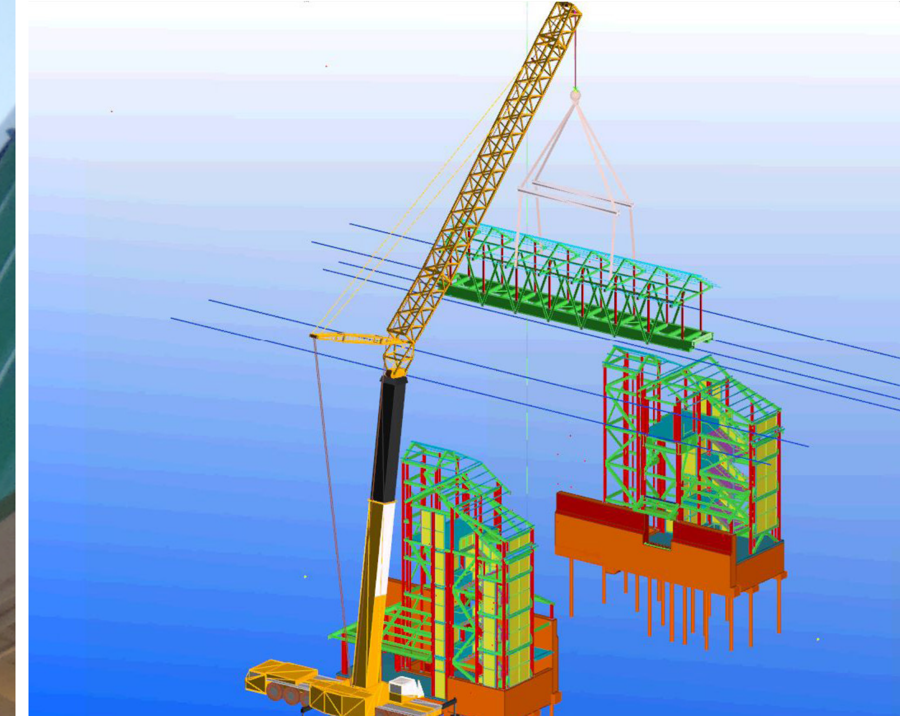


A new rail station and pedestrian bridge navigate existing electrical lines above a stop along America's busiest passenger rail corridor.

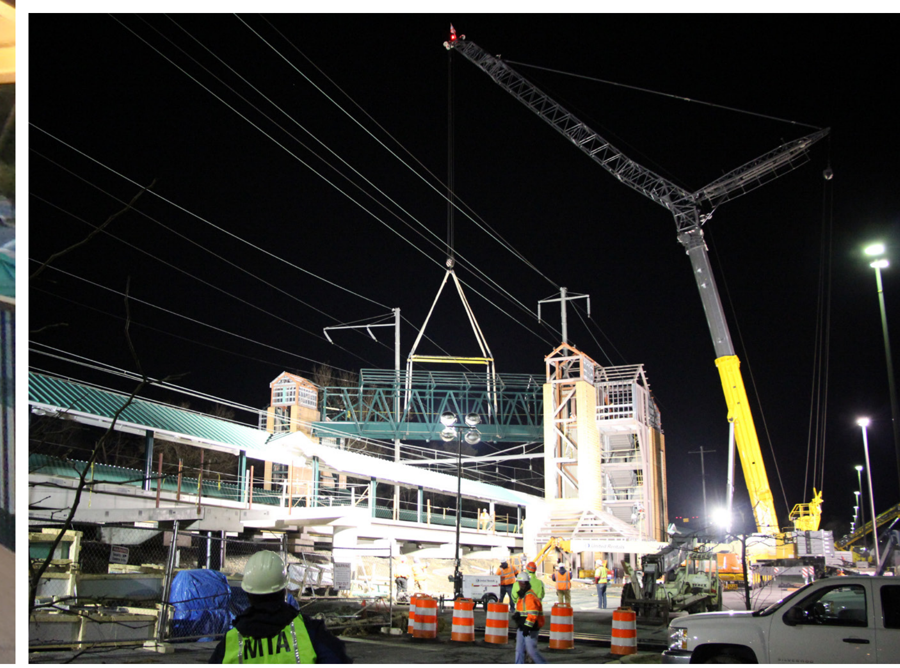
Threading the NEEDLE

BY MATTHEW MCCARTY, S.E., P.E., SCOTT KIRWIN, P.E., AND WAYNE CHANG, S.E., P.E.

The project uses 245 tons of structural steel in all.

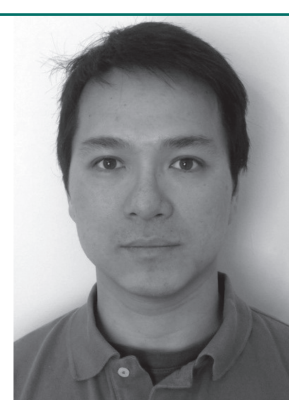


A detailing model of the pedestrian bridge erection process.



Erection of the bridge at night. For a time-lapse video of the bridge's erection, visit: vimeo.com/82219115.

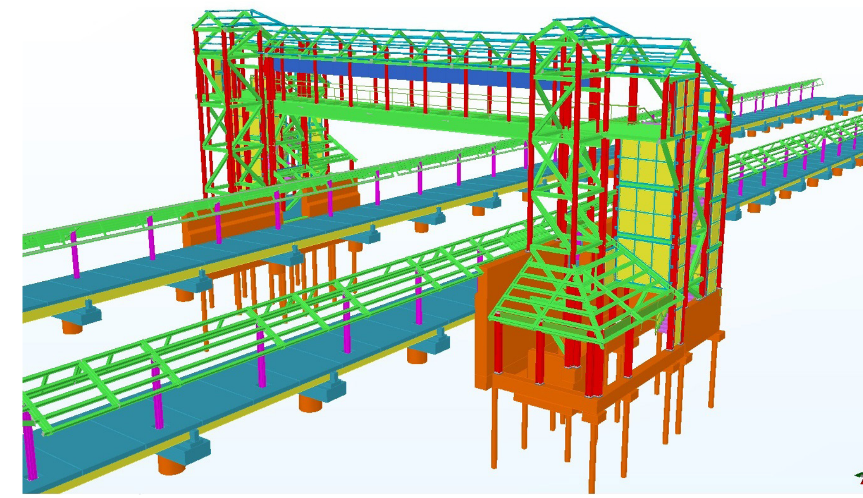




Matthew McCarty (mmccarty@wrallp.com) is a project engineer and **Scott Kirwin** (skirwin@wrallp.com) and **Wayne Chang** (wchang@wrallp.com) are associates, all with Whitman, Requardt and Associates, LLP.

THE HALETHORPE Station is a key link in a long and crucial chain.

Each day, the station (in Halethorpe, Md.) accommodates 1,300 of the 39,000 passengers served by the Maryland Rail Commuter Service (MARC), making it one of the five busiest stations in the system and a key point along Amtrak's Northeast Corridor (NEC), the busiest passenger rail corridor in the U.S. However, the station didn't meet Americans with Disabilities Act (ADA) requirements, so in 2002, the Maryland Transit Administration (MTA) engaged Whitman, Requardt and Associates (WRA) to study and subsequently design a new station that would be both ADA-compliant and able to accommodate current and potentially larger future passenger capacity. In addition, MTA requested that the station be unmanned and require minimal maintenance. It is also intended to serve as a prototype for future new and upgraded MARC stations. Finally, the facility needed to be constructed without interrupting MARC rail service and that of the NEC. WRA completed design contract documents in 2008, Amtrak electric traction modifications began in January 2010, the general contractor's construction was underway by March 2011 and the ribbon cutting for the new station took place in August 2013.



Welding the canopy steel.

The preassembled pedestrian bridge before erection.

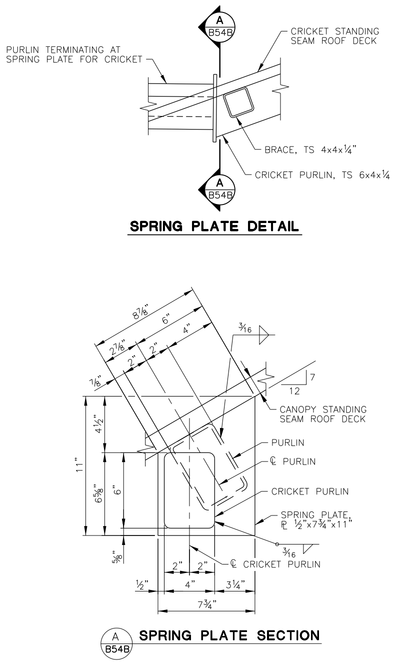
Finished pedestrian bridge interior.

A steel detail model, from AIW, used for review.

The completed station.

Spring plate details.

HSS field-welded connections.



East tower framing with interior precast panels.

Old and New

The previous Halethorpe Station consisted of two 150-ft-long at-grade platforms: one for trains bound for Washington, D.C., and the other for Baltimore. These pre-ADA, low-level platforms required conductors to assist patrons with the use of a small stepstool while boarding or alighting trains. Only the platform for Washington-bound trains provided any relief from the elements, in the form of two 33-ft-long shelters. The ticketing building was located 100 ft from the Washington-bound platform and even farther from the Baltimore service. In addition, patrons were required to walk up two flights of steep, open stairs to the sidewalk of the Francis Avenue Bridge then descend another two flights of stairs to get from one platform to the other, all while exposed to the elements.

The replacement station provides two 700-ft-long, high-level platforms that allow patrons to board and alight from trains at any point along the platform, thereby improving safety, accessibility and reducing dwell time for trains. Full-length platform canopies provide protection to commuters from the weather, and the new station is fully ADA-accessible, with elevators, ramps, stairwells and doorways that lead to a covered pedestrian bridge to provide easy access to and from each platform. The ticketing area is located at the main parking level entrance, which is near the middle of the Washington-bound platform. The style of the station echoes transportation architecture of the late Victorian/Industrial Revolution era; iron spot brick with accent bands, sloping metal roof components and an exposed structure evoke a historic flavor. These elements combine with the modern landscaping to provide a pleasant environment for the patrons to await their train's arrival. In all, the project uses 245 tons of structural steel (131 for the station building and 114 for the canopy).

Limited Flexibility

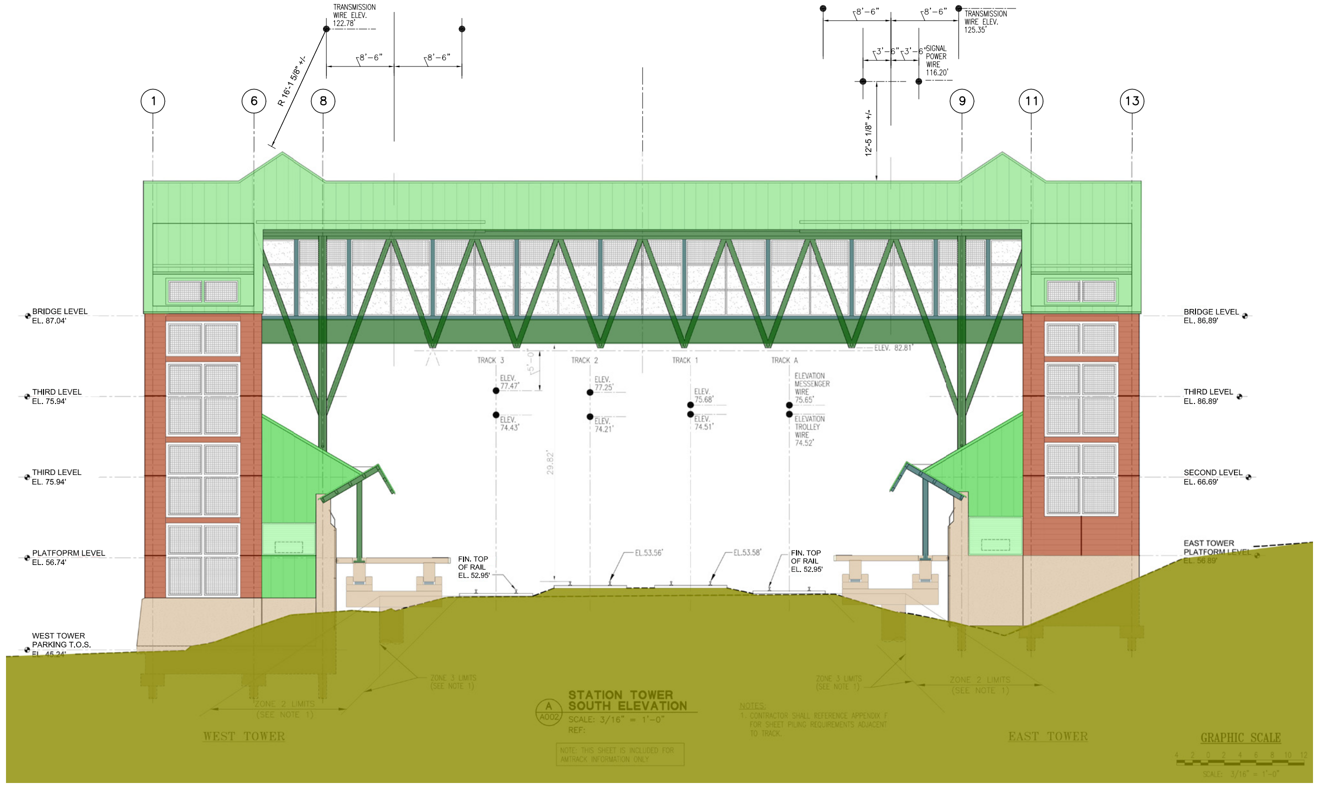
The project site's location along the NEC placed severe limitations on the overall construction. During construction, the daily operations of 122 passenger trains and the existing MARC station were required to be unimpeded, resulting in extremely restricted work schedules with limited flexibility. On the east side of the site, the contractor, W.M. Schlosser, had approximately 12,500 sq. ft of MTA property available for construction trailers and staging during construction; on the west side, it was limited to a small area of existing parking spaces immediately adjacent to the new construction. Finding parking around the MARC station during peak times was already very difficult, and MTA deemed it unacceptable for Schlosser to use any more than the bare minimum amount of parking lot for construction laydown. For temporary activities, which absolutely required more laydown area (e.g., prep for the bridge lift) Schlosser had to arrange to perform the work during lower commuter days or off-peak times of day.

Schlosser was limited to a six-hour nighttime work window allowing track closure for all activities adjacent to the southbound platform track and only a two-hour nighttime work window for activities affecting all tracks. As such, most foundation, concrete and structural steel work required at least one track closure. In addition, the overhead catenary power system and overhead transmission lines posed further constraints because they could only be de-energized within similar work windows. The overhead transmission lines run parallel to and are directly overhead of the entire platform and canopy structures. These lines limited the height of the equipment that could be used to install the drilled shafts for the platform foundations as well as the height of the cranes setting precast platform panels and canopy steel. On top of that, other Amtrak projects on the corridor upstream and downstream of Halethorpe Station occasionally removed these work windows entirely. These unexpected and unpredictable removals had a significant impact on the originally estimated construction duration. Schlosser was granted additional contract calendar days when they could prove that the delay was due to Amtrak requirements/limitations. For the most part, the company requested work windows and Amtrak approved or declined them as necessary for its own work needs. However, on occasion, Schlosser was told daily before close of business whether or not they would be working that night.

Platform Canopy

Each platform is protected by a steel-framed gable roof canopy using wide-flange columns and rectangular hollow steel structural sections (HSS) beams and purlins. The canopy columns and attached main sloped beams are W8×31s, and the canopy purlins are HSS6×4×¼. HSS were selected over open structural sections to eliminate a bottom flange where debris and wildlife can collect, as well as for its ability to better accommodate irregular connection geometry.

A standing-seam metal roof is applied directly over the structural steel purlins. Given the canopy's length, the designers selected a scheme where numerous short, structurally independent sections comprise each canopy. Each run of canopy is made up of 19 independent framing sections of lengths between 27 ft and 50 ft. This scheme was favored because it allowed nearly all steel connections to be performed off-site to speed on-site construction and minimize required track outages. The frequent joints between the framing sections also provide all necessary room for thermal expansion and contraction of the canopy. At the request of the fabricator, AIW, keeper bars were added to each joint in the framing to keep the independent canopy steel framing sections appropriately aligned to accept the standing-seam roof. These series of keeper bars, which slide past one another when the canopy thermally expands and contracts, hold the abutting cantilevered spans of canopy framing in vertical and horizontal



The south elevation of the station building.

alignment. All canopy steel was shop primed and given final field coats of high-performance forest green or white Tnemec paint.

Towers

Steel columns for the east and west towers are launched from the tops of 30-in.-thick reinforced concrete crash walls and surrounding grade beams. The gravity load resisting system of the towers consists of HSS beams and columns with non-composite concrete on metal deck, and the lateral load resisting system is a series of braced and moment frames. Multiple braced frames tie into the tops and sides of the crash walls and engage them as part of the lateral system. The designers chose steel HSS sections for the same reason as the canopy: The aesthetic and geometry of the buildings also required irregular member connection geometry, which was more easily accomplished with HSS rather than open sections. All exposed HSS sections were shop primed and given final field coats of high-performance forest green Tnemec paint.

The station building is clad in a combination of precast concrete panels, metal panels and wire mesh, and is designed to be an open structure. The use of exposed HSS in an open structure necessitated that the connections between elements be seal welded all-around for aesthetic and corrosion reasons. These connections are used as moment resisting connections in multiple locations and participate in the towers' vertical lateral load resisting system. Moment connections in the horizontal plane are also used to create a frame in some areas as a substitute for a traditional building diaphragm. During construction, AIW welded together complete building frames in the shop and erected them in one piece as much as possible. In cases where the seal-welded connections did not accommodate field fit-up tolerances in member length, the tolerances were achieved by either cutting off slivers of member ends where pieces ran long or by building up a sufficient weld width to bridge the resulting gap where pieces ran short.

Due to the towers' open nature, the precast panels were mostly placed around the elevator and stair shafts and below steel roof beams, which necessitated very close coordination between the steel and concrete panel erectors and fabricators. During construction, steel framing was advanced up until the roof beams were to be set, then paused while the interior precast panels were set. Once all the internal panels were set, roof framing was completed.

The structural steel shop drawings were produced by AIW via Tekla. During the steel shop drawing review period, WRA requested and was sent "for information only" copies of the Tekla detailing model. Due to the complexity and irregularity of the stair/elevator towers, this model was incredibly helpful in verifying the shop drawings and visualizing the structure. A number of issues, which had been hardly noticeable in the printed shop drawings, were readily identified in the model and were quickly resolved. For instance, the initial set of shop drawings was missing several rows of short cantilevered purlins on top of the stair tower. Using 2D drawings, this omission was easy to overlook, as the purlins only extend 1 ft outwards and support a small section of roof and gutter. However, comparing the shop model to contract drawings and architectural renderings made the missing purlins blatant. Additionally, the initial shop drawings misplaced a number of the steel channels required for attaching wire mesh panel cladding. Reviewing the model made verifying proper placment of the various cladding system elements much more intuitive.

Pedestrian Bridge

The pedestrian bridge provided perhaps the biggest erection challenge because it could only fit within a narrow vertical window. The elevation and height were restricted in order to maintain effective viewing time to Amtrak's signals coupled with maintaining vertical clearances from power cables above and below the bridge. Just below the bridge are electrical trolley lines; just above the bridge are electrical transmission and signal lines. These physical constraints were the driver of most design decisions related to the bridge. Given the need to shut down all four tracks and trolley wires to install the bridge, ease and speed of erection were of paramount concern. The vertical load-resisting structure of the bridge consists of a pair of 80-ft-long W36×194 girders laced together with horizontal angle bracing. HSS frames are launched from atop the girders and are used to support the bridge's roof and glazed and wire mesh cladding.

The bridge was designed to give Schlosser the flexibility to either erect the girders first and then build the HSS frames atop or fully preassemble the bridge and set it in one piece. Schlosser ultimately decided to set the bridge almost completely assembled, as this allowed them to perform the maximum amount of work while still on the ground, without track outages and during the day. A few of the HSS frames at the end of the bridge were left off during the bridge pick to allow it to fit between the already constructed stair and elevator towers and to reduce pick weight. Although the bridge assembly weighed 40 tons when set, it required a 550-ton-capacity Grove GMK 7550 crane to erect. To clear the electrical transmission lines, Amtrak tracks and already constructed west stair tower, the pick radius was an impressive 90 ft.

Unfortunately, not all of Amtrak's electrical lines could be shut down during the bridge pick. At least one set of transmission lines is required to remain energized at all times to maintain proper phasing between all the electrical substations along the NEC. To provide an adequate clearance around all electrified lines, the eastern set of transmission lines were permanently relocated approximately 10 ft further away from the tracks by installing new steel tower armatures in the weeks preceding the bridge lift.

Schlosser further leveraged the steel detailing model to fully model the surveyed locations of the bridge bearings, electrical lines, bridge pick rigging and crane to be used. AIW then created a series of animations to show that the entire bridge lift could be performed successfully and demonstrate that every contingency had been evaluated; the entire project team recognized the potential for a prolonged and very costly closure of the NEC if something went wrong with the lift. Thanks to the months of planning and preparation, the night of the bridge erection went off without a hitch. The bridge went from sitting on the ground to being lifted over 120 ft in the air to clear the de-energized transmission wires to sitting on its bearings with connections bolted within two hours—which was just in time to let a 3:30 a.m. diesel train pass through the site. ■

Owner
Maryland Transit Administration

Operator
Maryland Rail Commuter Service

Architect and Structural Engineer
Whitman, Requardt and Associates, LLP, Baltimore

General Contractor
W.M. Schlosser Company, Inc., Hyattsville, Md.

Steel Fabricator, Erector and Detailer
AIW, Inc., Hyattsville, Md.

The Kansas Department of Transportation finds a new solution for stream crossings.

KANSAS Crossover

BY TIM LEAF, P.E., AND SCOTT UHL, P.E.

The new Highway K-4 bridge replaces a deteriorating corrugated metal arch culvert.

JUST WEST of Topeka, Kan., where Highway K-4 crosses Blacksmith Creek, sat a deteriorating corrugated metal arch culvert that was badly in need of replacement.

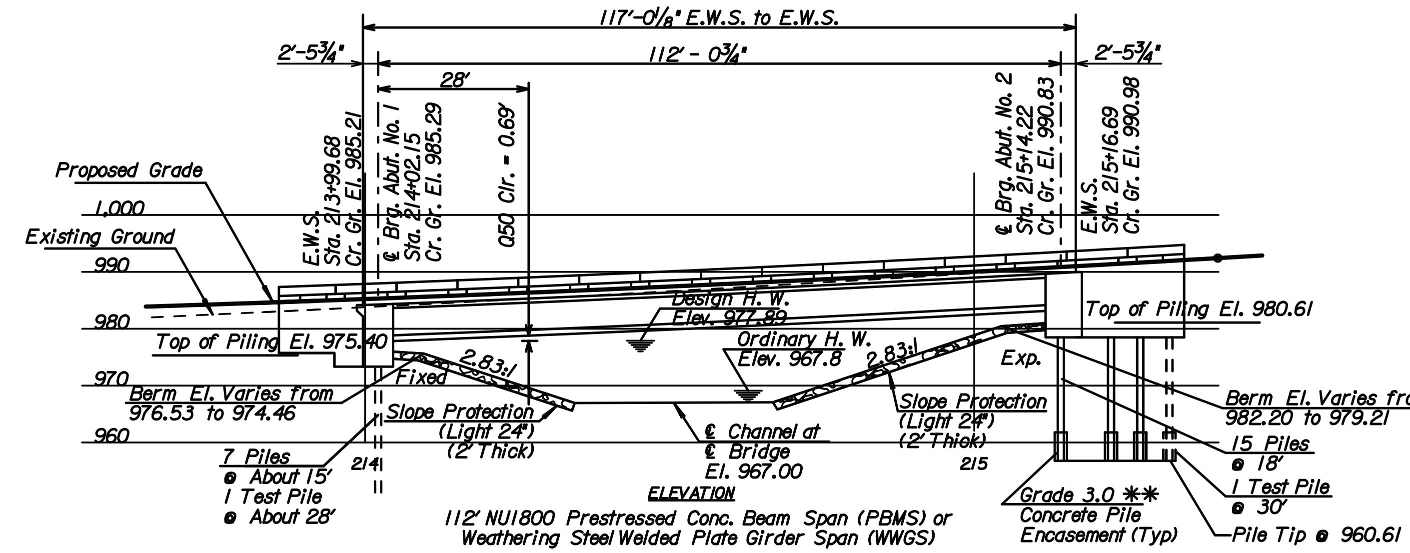
One side of the arch was deflecting inward and maintenance crews reinforced it with railroad ties as a temporary measure, but the Kansas Department of Transportation (KDOT) recognized that the span would eventually need to be replaced.

On the surface, this project seemed fairly simple; it's a relatively short bridge over a creek. However, the site was not without its challenges. The old, arched culvert had a maximum height of 12 ft in the center and a maximum width of 30 ft at the bottom. The road above was 19 ft to 20 ft higher than the bottom of the streambed and sloped at roughly a 6% grade. Also, the stream cuts the road at a 45° angle, which meant a replacement structure would require a similar skew.

KDOT routinely relies on three-span reinforced concrete haunched slab (RCHS) bridges for stream crossings and has predetermined span arrangements to simplify design and construction work. Because of this preference, local contractors are accustomed to building these standard structures. However, considering the high skew at this location, the substructure units for the RCHS option would need to be longer than a typical application, thus adding to the overall cost for each intermediate pier. In addition, the smallest standard RCHS span arrangement that could accommodate the required hydraulic capacity and keep all of the piers out of the channel would be longer than necessary.

Tim Leaf (tim.leaf@bartwest.com) is a senior project engineer and **Scott Uhl** (scott.uhl@bartwest.com) is a senior project manager, both with Bartlett and West.

The stream cuts the road at a 45° angle, which meant a replacement bridge would require a similar skew.



An elevation view of the new bridge.

The next solution considered was a single-span girder bridge. It would be shorter yet still have a sufficient hydraulic opening, and came with the benefits of lower maintenance costs and ease of construction. Bartlett and West investigated this option in the hopes that by reducing the overall bridge length and eliminating two piers, the single-span bridge would be more cost-effective than the three-span RCHS. In addition, by eliminating two of the piers, drift accumulation and scour concerns would be greatly reduced.

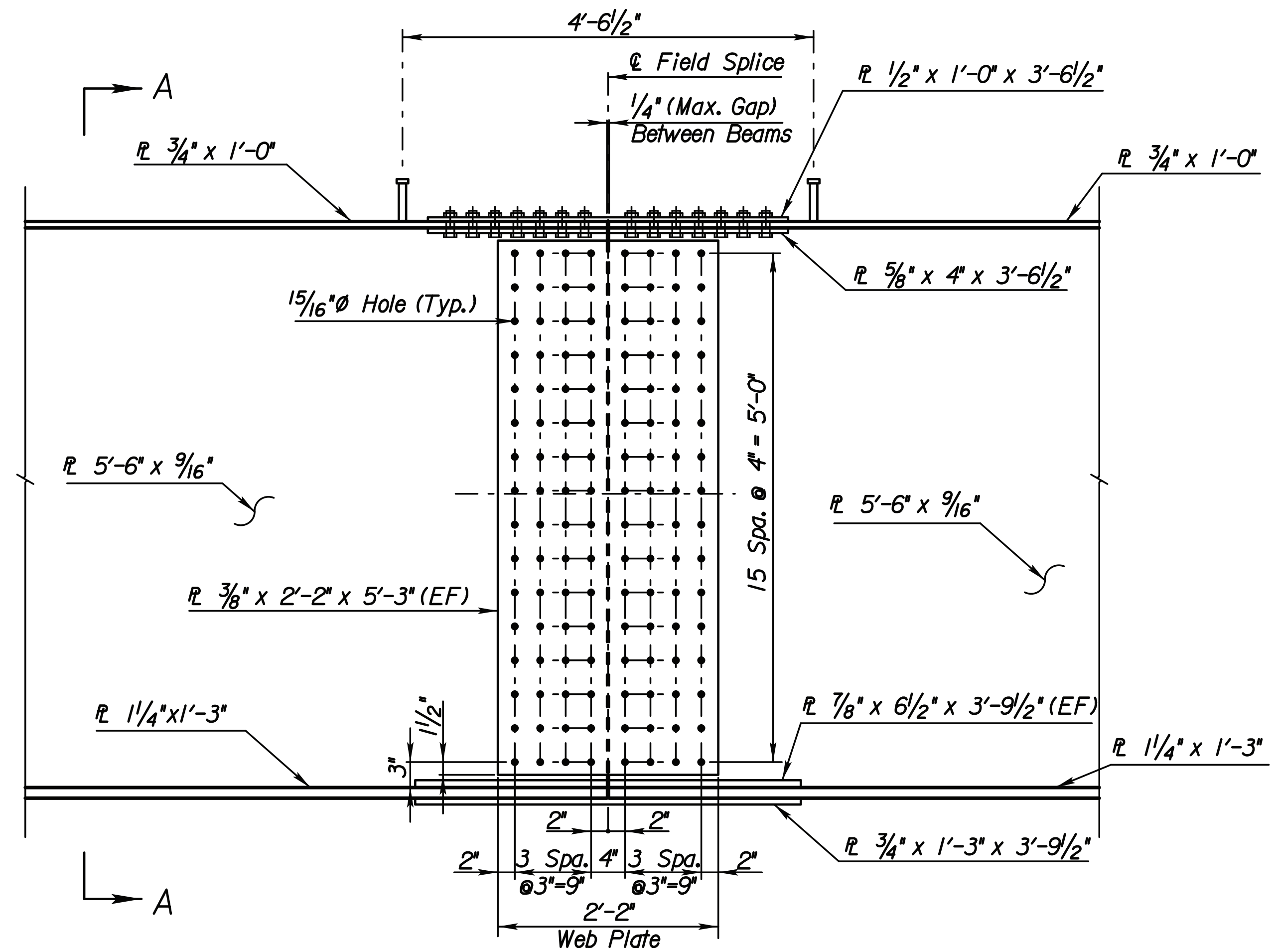
Separate preliminary cost analyses for both steel and concrete superstructures were performed to see which would be the more cost-effective solution. The estimates showed that the difference in cost for a bridge with a steel plate girder superstructure versus pre-stressed concrete girders was marginal. KDOT's assumption was that labor costs more than materials and therefore predicted that placing a long single-span concrete girder in one piece would be more economical than the steel option, which would require additional intermediate diaphragm and field splice work. By bidding both superstructures, this theory was put to the test.

The old, arched culvert had a maximum height of 12 ft in the center and a maximum width of 30 ft at the bottom.

The final girder layout of the new bridge was a single span of 112 ft, using five girders at a 9-ft spacing; the steel plate girders are made from weathering steel.

Out of four contractors that bid on this project, only King Construction, Inc., went with the steel superstructure option—and ultimately secured the contract. King chose the steel option for the following reasons:

1. Due to the small size of the job and the single-span scheme, they were not planning to have a large crane on-site. They could lift the steel girder in place with the crane they had available and would have needed a larger crane just for placement of the concrete girders (each concrete girder was 36 tons heavier than each steel girder).
2. Forming would be easier for the concrete diaphragms at the end bents with the steel girder option. The difficulty in forming the concrete girders was due to the large and thin top flanges, which could potentially be broken during the formwork stage of construction.
3. Steel would be quicker for construction.



A connection drawing of one of the web plates.

A field splice was located 33 ft, 9 in. from the end of the girders, which allowed them to be shipped more easily and without the need for special permits.

The final girder layout was a single span of 112 ft, using five girders at a 9-ft spacing, and the steel plate girders are made from weathering steel. Each girder used a 12-in. × ¾-in. top flange, a 66-in. × 9⁄16-in. web and a 15-in. × 1¼-in. bottom flange. No adjustments were made to the size of the flanges to account for differences in moment envelopes across the span. According to the steel fabricator, DeLong's, Inc., this was due to the fact that the fabrication costs of adding shop splices to adjust the flange dimensions can end up costing more than the cost per weight of steel saved from putting in smaller flange sections near the girder ends.

A field splice was located 33 ft, 9 in. from the end of the girders, which allowed these longer assemblies to be shipped more easily and without the need for special permits. Since this was a single-span bridge, the splice location could not be put at the dead load contra-flexure point and instead was located at 30% of the total span length. This location helped avoid an unnecessary shop splice as well as

The bridge crosses Blacksmith Creek near Topeka, Kan.

kept the field splice away from the maximum moment region.

For KDOT, this project was an opportunity to test a new system that it wasn't yet familiar yet and expand its bridge portfolio, as well as correct some of its cost-related assumptions. And most importantly, it opened the door for another economical solution for future stream crossings. ■

Owner
Kansas Department of Transportation, Topeka, Kansas

Engineer
Bartlett and West, Inc., Topeka, Kansas

General Contractor
King Construction Company, Inc., Hesston, Kansas

Steel Fabricator
DeLong's Inc., Jefferson City, Mo.

Getting a GRIP

BY CHARLES-DARWIN ANNAN, P.ENG., PH.D., MARIO FAFARD, ING., PH.D., MAXIME AMPLEMAN, ING., AND ÉRIC LÉVESQUE, ING.

An emerging coating option for bridge components—metallizing—shows promise thanks to slip-resistance testing based on parameters set in U.S. and Canadian steel standards.

THE DESIRE for enhanced, long-term performance for both new steel bridge construction and maintenance applications is shifting the paradigm from today's paints to coatings with more complex chemistry and application requirements.

Metallizing has recently emerged as a protective coating for steel bridge elements and is seeing increased recognition by multiple transportation agencies, including the U.S. Federal Highway Administration (FHWA) and the Canadian ministère des Transports du Québec. The practice can be used alone or in combination with compatible topcoats to not only provide an extended service life but also to add additional aesthetic quality to the bridge structure.

So what, exactly, is metallizing? The term is commonly used to describe the practice of thermally spraying molten zinc, aluminum or zinc/aluminum alloy on surfaces of exposed steel elements to provide both physical barrier and effective sacrificial protection through galvanic action. It can be applied to steel bridge components either at fabrication shops or in the field, and there is no size limitation on members that can be metallized. Strict surface preparation is essential for reliable adhesion, and a minimum of SP-10 (near-white blast-cleaned surface) is required per SSPC-CS 23.00.

In order to derive the maximum benefits of metallizing, bridge designers need to know the slip coefficient of metallized faying surfaces required to develop slip-critical connections in the bridge structure. This helps to eliminate the current labor-intensive and time-consuming practice of masking off all connection faying surfaces to preserve their conditions prepared in accordance to prevailing design standards. Therefore, the ability to design for and supply coated faying surfaces is an important option—and achieving a reliable slip coefficient is an essential variable in this option.

As no code provision for this design coefficient exists, Université Laval and Canam-Bridges (NSBA Member) decided to perform their own research in accordance with the slip tests described in Appendix A of the *Specification for Structural Joints Using High Strength Bolts* published by the Research Council on Structural Connections (RCSC), with the results being based on the slip coefficient values in the 2006 *Canadian Highway*

Charles-Darwin Annan is an associate professor, **Mario Fafard** is a professor and **Maxime Ampleman** is a graduate student, all at Université Laval. **Éric Lévesque** is engineering manager of new products with Canam-Bridges.

Design Code and the 2010 AISC *Specification for Structural Steel Buildings*. For maximum slip resistance, the highest established slip class in both standards is Class B, with a slip coefficient of 0.50, which represents a blast-cleaned connection surface or blast-cleaned with Class B coatings.

Testing Slip Resistance

Laval and Canam performed two sets of tests to evaluate the slip resistance of zinc-based (99.99% pure) metallized faying surfaces with no top coat. Short-duration slip tests in tension and compression were conducted first to determine the mean slip coefficient. Subsequently, long-term creep tests were performed under sustained tension loading to ensure that the coating did not undergo excessive deformation (meaning creep deformation did not affect the observed slip resistance).

For the short-duration tests, close to a hundred specimens were fabricated and prepared for testing in compression and tension. The metallizing coating was applied through an electric arc spray gun from zinc wire. Other parameters investigated other than the testing regime included the thickness of coating (6 mils and 12 mils), plate thickness (½ in. and ⅝ in.), and the amount of bolt preload (70% and 90% of the tension capacity of bolt material). The specimen plates were fabricated from weathering steel and the plates were clamped using ⅞-in.-diameter ASTM A325 high strength-bolts. For each set of parameters, the mean slip coefficient was obtained from five replicates.

Figure 1 shows comparisons of the evaluated mean slip coefficient for different sets of parameters. All specimens tested far exceeded the Class B slip coefficient value of 0.5. The lowest mean slip coefficient was evaluated as 0.77, representing a ⅝-in.-thick plate specimen with 6-mils metallized coating and 90% bolt preload tested in compression. The highest mean slip coefficient was obtained as 0.98, representing a 12-mils coating on a ½-in. plate with 70% bolts preload tested in tension. Most importantly, we discovered that for the same set of parameters, an increase in coating thickness from 6 mils to 12 mils resulted in an increase in slip resistance, while the bolt preload, plate thickness and test regime had no significant effect.

In the long-term creep tests, three replicate assemblies were clamped and loaded in series for 1,000 hours in ten-

Metallizing could become a viable option for bridges such as the Highway 15-640 overhaul project in Boisbriand, Quebec, Canada, fabricated by Canam-Bridges.

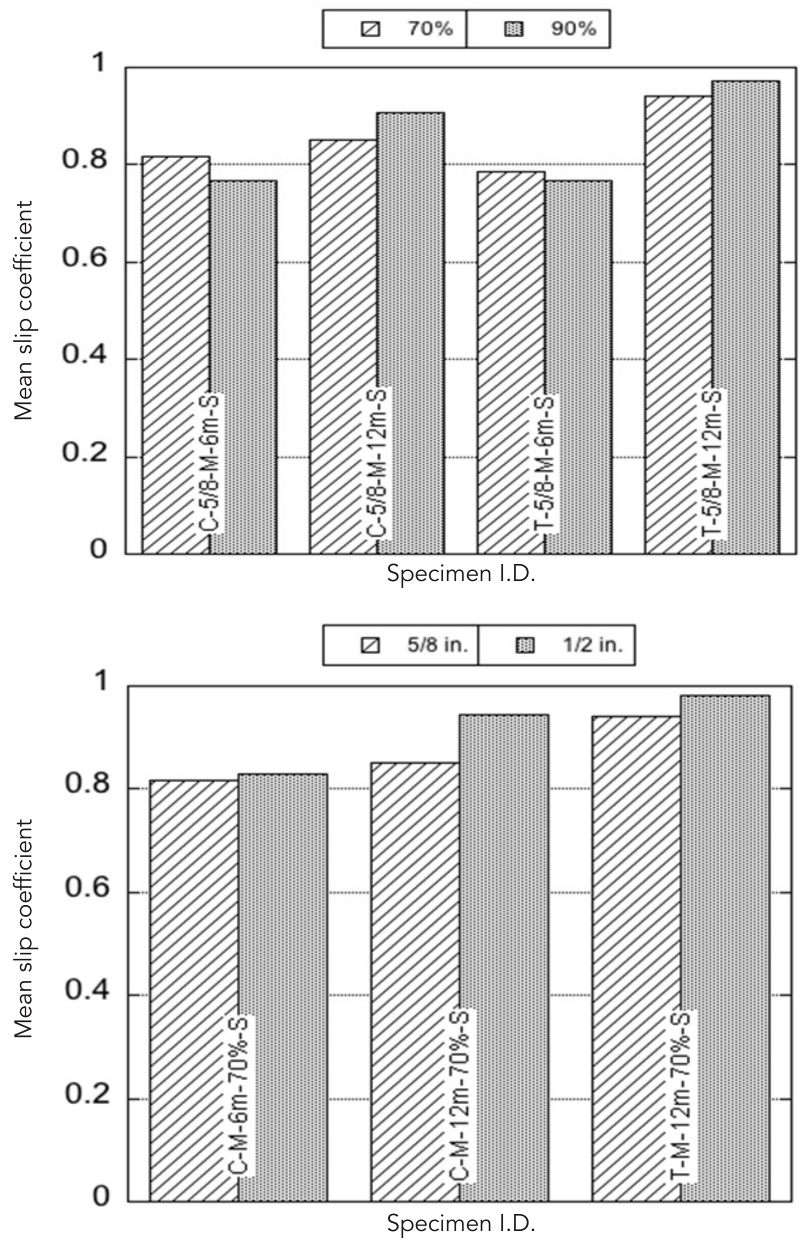


Figure 1. Comparisons of mean slip coefficient—e.g., notation C-M-6m-70%-S represents compression test (C) with 6 mils metallization and a 70% bolt preload.

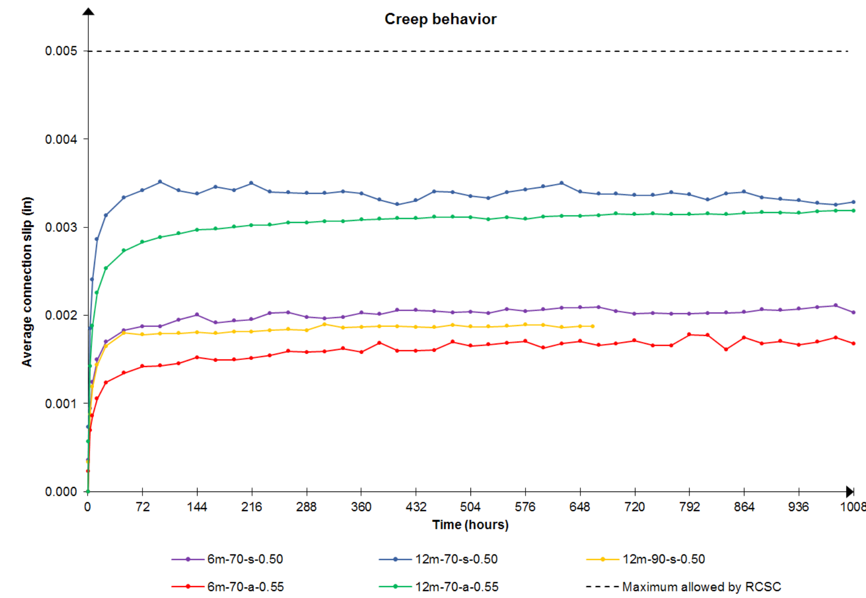
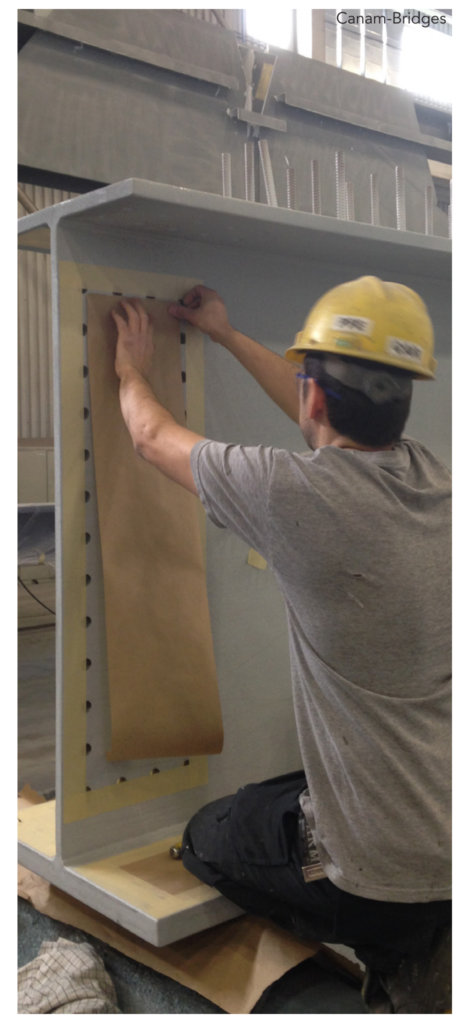


Figure 2. Creep deformation versus time.



Masking off of connection faying surface before metallizing.

Long-term creep test set-up.

at the service load associated with the design slip coefficient of class B. Specimens were evaluated for two design slip coefficients, 0.5 and 0.55, to verify creep performance in accordance with the revised Class B coefficient of 0.52 specified in the 2014 *Canadian Highway Bridge Design Code*. The creep deformation, defined as the relative displacement between adjacent plates in a clamped specimen, was measured using extensometers in compliance with the RCSC specifications and compared with the acceptable limit of 0.005 in. The applied clamping force was monitored continuously from the time of assembly through to the end of testing to assure that relaxation in the bolt preload wasn't excessive. At the end of the creep loading, the test assemblies were loaded to the design slip load to ensure that the creep behavior did not adversely affect the design slip resistance. Figure 2 shows plots of average creep deformation versus time for five sets of parameters and also shows the maximum allowed deformation.

All the specimens showed acceptable creep behavior, with the 12-mils metallized coating exhibiting more creep deformation than the 6-mils coating. For the 12-mils coating, the specimen with a 70% bolt preload showed higher creep deformation compared with specimens with 90% bolt preload. Also, more relaxation of the clamping force was observed for the 12-mils metallized coating versus the 6-mils coating. When loaded to the design slip load at the end of the creep test, all the test assemblies showed a slightly increased deformation, much lower than the RCSC specified limit of 0.015 in.

Additional research is in the works, but these initial results are very encouraging. The fact that metalizing has been demonstrated to meet the Class B requirements for slip-critical connections without having to perform additional and potentially expensive connection preparation means that it could potentially become a viable, efficient option for bridge components. ■

Pedestrians in Memphis now have an attractive and innovative way to wind up at the Mississippi riverfront.

Ramping UP

BY ANDY KIZZEE, P.E., LEE SCHULZ, P.E., AND DANIEL SOURS, S.E., P.E.

Aerial Innovations

Andy Kizzee was a structural engineer with Smith Seckman Reid, Inc., in Memphis and is now serving as an Engineering Ministries International director in India. **Lee Schulz** (engineer of record) was a senior structural engineer with SSR and is now enjoying retirement after a 40-year career with the company. **Daniel Sours** (**dsours@ssr-inc.com**) is currently a senior structural engineer with SSR.

MEMPHIS VISITORS used to be able to stroll west down Beale Street past the bronze statue of Elvis Presley until they encountered scenic Riverside Drive at the mighty Mississippi River.

The sprawling Tom Lee Park, with 4,500 ft of river frontage lies, just to the south of the intersection while pre-Civil War cobblestones pave the riverbank north of the site. But right at the intersection, the neglected seven-acre riverbank was covered in broken concrete revetment and overgrown vegetation—not an attractive transition.

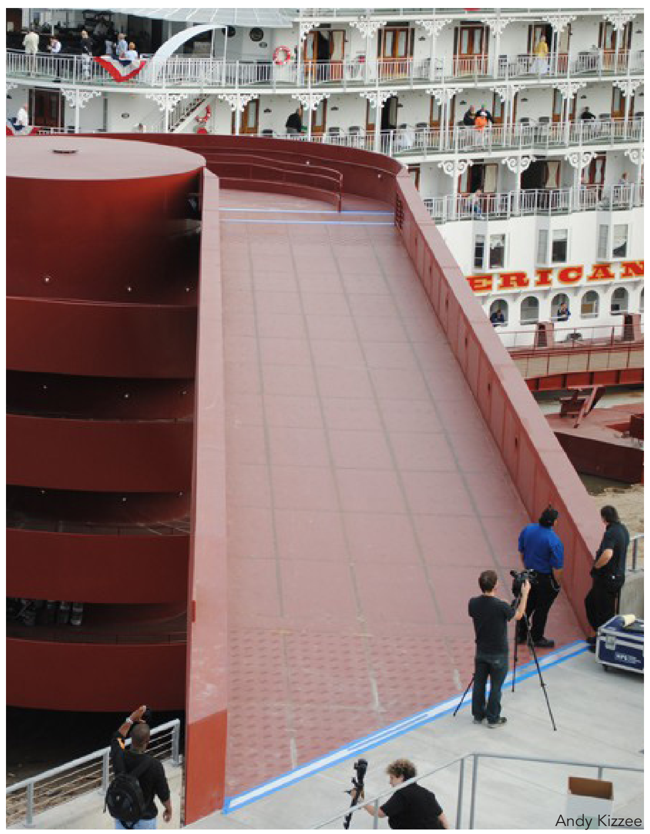
In 2003, Riverfront Development Corporation, under authority from the City of Memphis, decided to develop this prime location. An international competition was held for a design solution to highlight the intersection where Beale Street meets the river and where Tom Lee Park connects to the cobblestone riverbank. The resulting Beale Street Landing, which is now open, serves as the port for riverboat traffic as well as a high-profile public park.

A Roundabout Solution

The structures include a floating dock accessed by a helical ramp, which helps the facility handle river fluctuation, and a grass-covered terminal building that provides pedestrians a link between the park and the cobblestones. Visitors access the riverboats tied up at the floating dock by a helical ramp and a connector walkway—both completely steel-framed structures. The connecter walkway is 16 ft wide and spans 130 ft between the terminal building and the helical ramp and is framed with an upturned W40×183 girder on each side, while the bridge is supported on two 48-in. hollow structural steel (HSS) columns; the girders are covered in light-gage steel panels for a cleaner look.

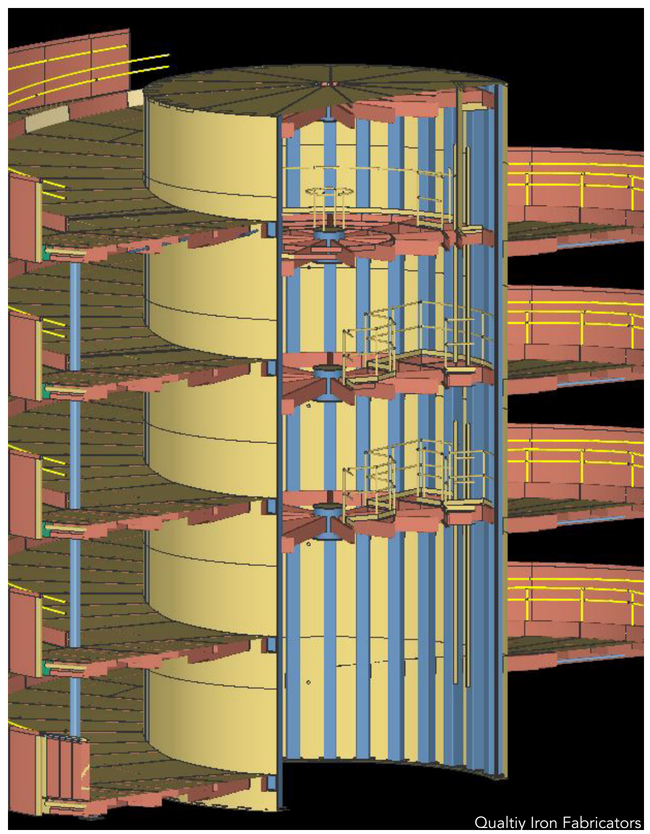
The new terrace park descends into the Mississippi, and at high water levels the river floods the park while permanent structures dubbed "islands" remain above the 100-year flood level (these pile-supported concrete slab structures appear as islands during periods of high water).

Visitors access the floating dock by a helical ramp and a connector walkway. The connecter walkway is 16 ft wide and spans 130 ft between the terminal building and the ramp.



Andy Kizzee

A 3D look inside the steel-framed helical ramp.

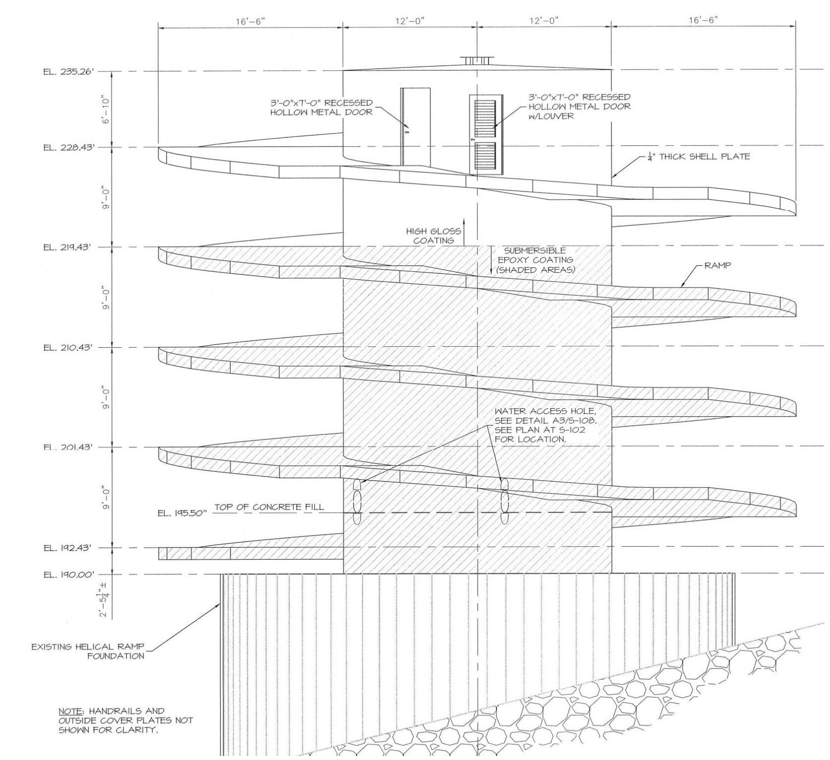

Qualtiy Iron Fabricators

The bridge is supported on two 48-in. HSS columns.

The helical ramp accommodates elevation changes of the floating dock.

High Water

The dock is designed to operate during river level changes of more than 45 ft. It operates in all but the extremes of low and high water when it is neither safe nor possible to navigate the Mississippi. To give an idea of just how much the Mississippi's water level can fluctuate in this area, the spring floods of 2011 to the drought of 2012 saw the water change by nearly 60 ft.



Most floating docks on the lower Mississippi are accessed by relatively short gangways, which, along with the floating dock, accommodate the river's elevation change by being pulled in or pushed out along the sloping riverbank. The designers for the Beale Street Landing wanted a floating dock moored by large arms anchored to pile caps set at the edge of the channel. A stationary connector walkway connects the terminal building to the helical ramp, which accommodates the elevation changes from the connector walkway to the floating dock. The helical ramp provides five access gates that vary in elevation by 9 ft (one complete circumferential turn of the ramp), and a hydraulic ramp on the first floating barge accommodates the elevation difference between the gates). At high water levels, access to the floating dock will be from the highest level of the helical ramp. At lower water levels, pedestrians walk down the spiral to the lower levels to access the dock. When one of the gates is opened for access to the floating dock, it blocks the access for pedestrians and cart service at lower levels, which would be underwater.

▲▼ Framing for the helical ramp consists of 55 HSS12×10×½ columns at the perimeter of a 24-ft-diameter core with ¼-in. steel plate forming the steel cylinder surface.

The heart of the river access is a helical ramp that accommodates elevation changes of the floating dock. During design, the requirements for the helical ramp seemed daunting. The ramp had to be ADA-compliant but also able to provide access for electric carts to carry luggage and supplies. And it would be located not just *on* the river but also *in* the river. Also, the helical ramp and its foundation work had to be installed during a short period of low water. The design solution was to use 480 tons of HSS and steel plates.

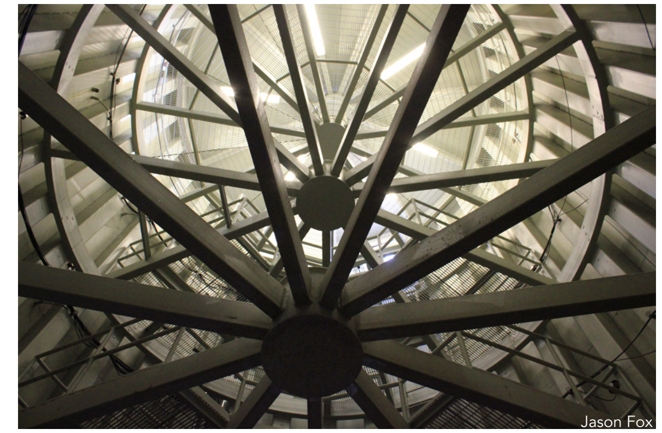
Framing for the helical ramp consists of 55 HSS12×10×½ columns at the perimeter of a 24-ft diameter core with ¼-in. steel plate forming the steel cylinder surface. Outrigger HSS12×8×½ beams cantilever 16.5 ft from the HSS columns, and infill HSS6×2×¼ members support the ⅜-in. floor plate. The steel plate elements, as well as two exposed inboard columns supporting the dock access landings, are intended to help the structure survive debris build-up from eddy currents.

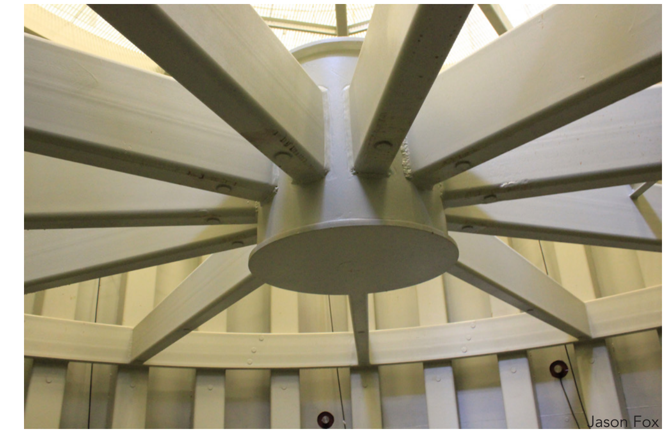
The sloping walkway of the helical ramp is divided into two parts. The first is an approximately 12-ft-wide continuous slope adjacent to the core that is dedicated to the electric carts since it is too steep for pedestrians; the outer 5-ft-wide walkway is separate from the cart access. The handicap-accessible pedestrian access is a series of ramps and landings, with the perimeter wall serving as a guard rail winding around the ramp at a constant slope and disguising the broken slope of the pedestrian ramp.

The framing for the multiple levels of the helix resembles a series of wagon wheels. The horizontal members tie the columns together to resist the horizontal forces resulting from the bending moments applied by the floor beams. Access into the core is at the highest framing level, the only level with grating and where the electrical panels are located, and ladders provide access to the levels below for inspection and maintenance. Two openings just above the core's base allow for water pressure equalization while an opening in the center of the roof serves as a relief air vent. The ramp is anchored to a concrete pile cap, which bears on 16 48-in.-diameter steel piles.

Since the steel structure is exposed inside and out to river water, a polysiloxane marine coating was used on the exterior surfaces and floating docks. The inside of the structure is protected by a high-build epoxy coating, and the HSS members were injected with expanding polyurethane foam to prevent water infiltration, which could freeze and damage the structure.

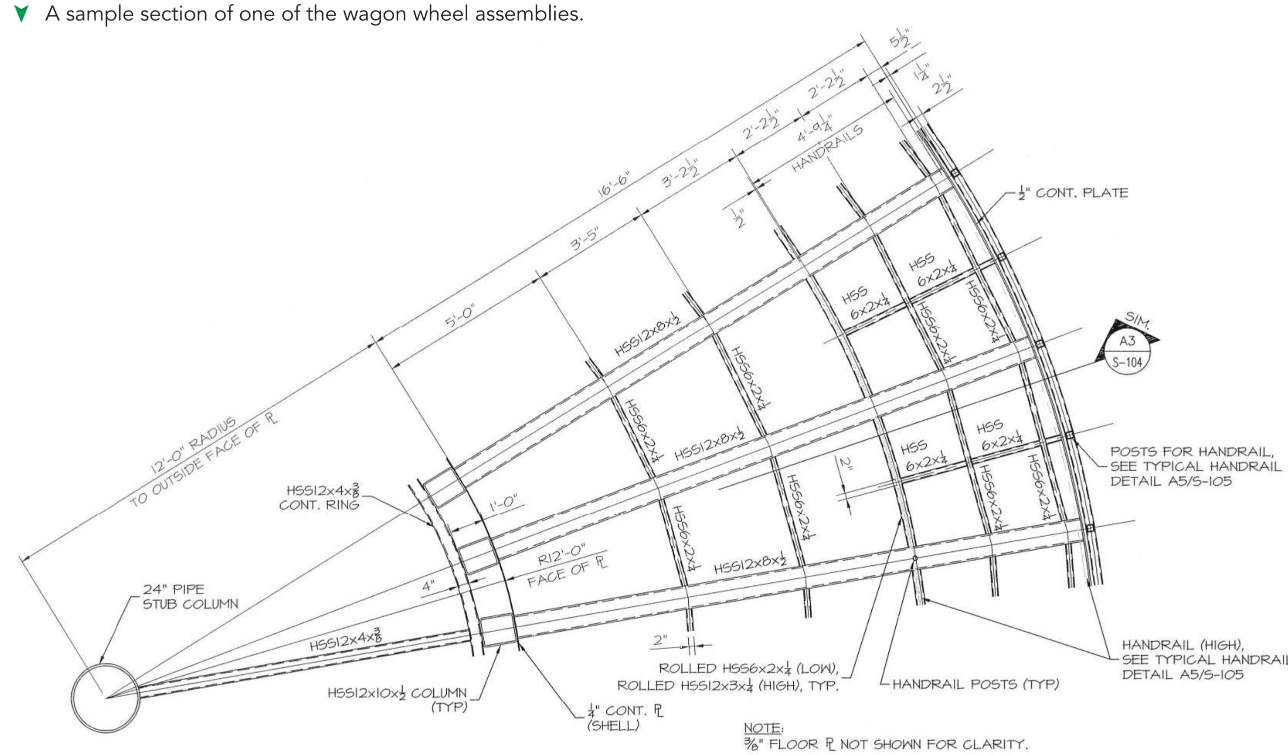
The Beale Street Landing project was built in phases over several years. The contractor had two low-water opportunities to drive piles and pour concrete pile cap foundations. There is no guarantee that the river will cooperate or fall to any specific level for any length of time, but luckily the water level remained low enough for the ramp foundation to be installed in the first low-water season. In the next construction phase, the erector of the helical ramp placed all 55 HSS columns and was able to pour the infill concrete base, and the pie-shaped walkway sections were erected as the river rose throughout the year. ■





▲ The framing for the multiple levels of the helix resembles a series of wagon wheels. The horizontal members tie the columns together to resist the horizontal forces resulting from the bending moments applied by the floor beams.

▼ A sample section of one of the wagon wheel assemblies.



Owner
City of Memphis Riverfront Development Corporation

General Contractors
LCI Construction
Webb Building Corporation

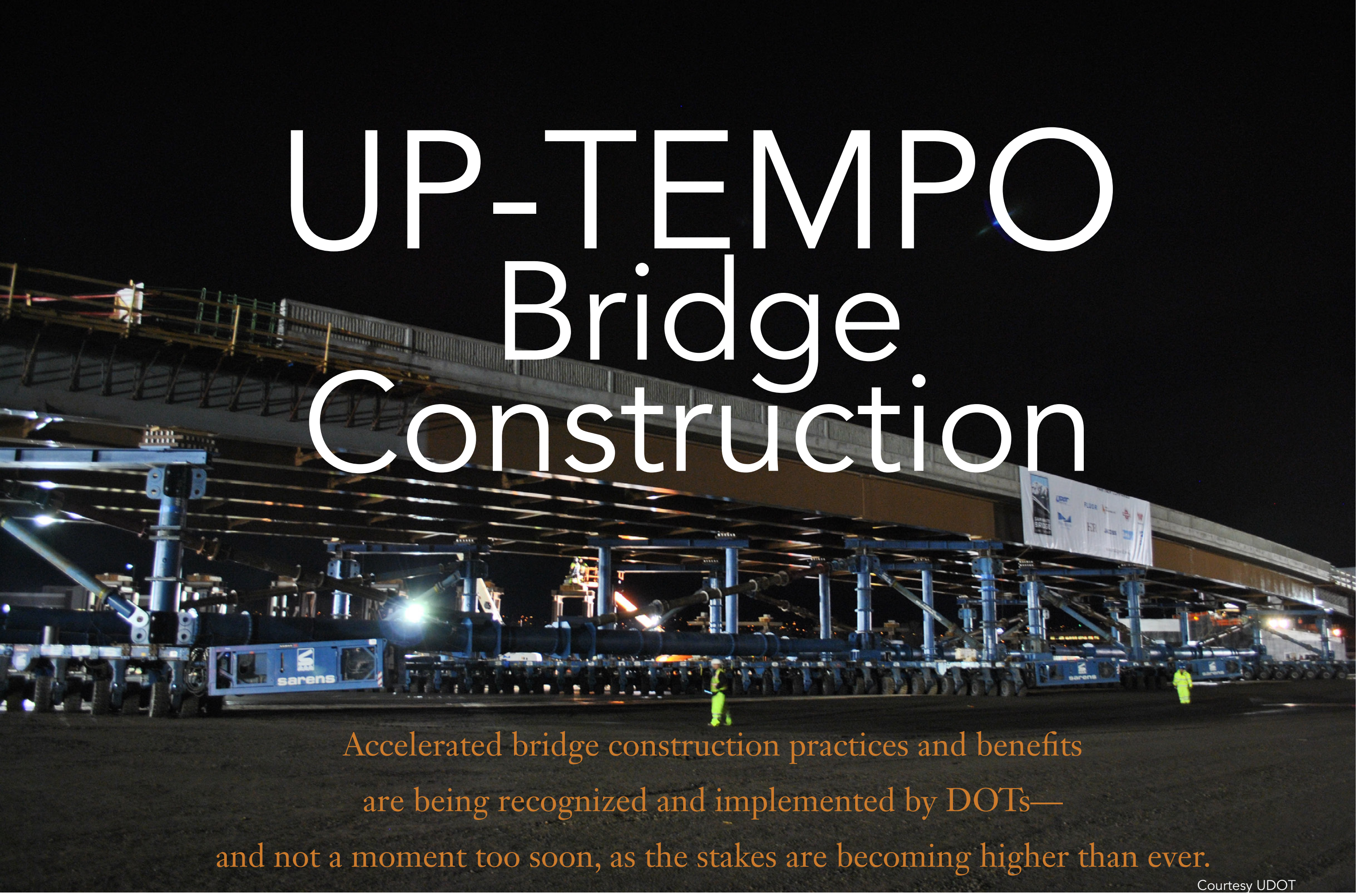
Architects
Bounds and Gillespie Architects, PLLC
RTN Architects

Structural Engineer
Smith Seckman Reid, Inc.

Steel Fabricator and Detailer
Quality Iron Fabricators, Inc.

UP-TEMPO Bridge Construction

Accelerated bridge construction practices and benefits are being recognized and implemented by DOTs—and not a moment too soon, as the stakes are becoming higher than ever.



Courtesy UDOT

Utah DOT's Sam White Lane Bridge over Interstate 15 was moved into position via self-propelled modular transporters.

BY MARY LOU RALLS, P.E.

WHAT IS the overall health outlook for the nation's bridges?

A quarter of the 607,380 bridges in the U.S. are classified as substandard (structurally deficient or functionally obsolete)—and 210 million vehicles cross these substandard bridges every day in the 102 largest metropolitan regions alone, according to the American Society of Civil Engineers' 2013 *Report Card for America's Infrastructure*. In addition, the average age of the nation's bridges is over 40 years, with an estimated 30% of existing bridges already older than their 50-year design life. To make matters worse, to upgrade existing substandard bridges and the bridges being added daily to this group would require billions of additional dollars every year for the next decade. While progress is being made to reduce substandard bridges, the above statistics resulted in a grade of C+ for bridges in the aforementioned report card.

Mary Lou Ralls (**ralls-newman@sbcglobal.net**) is principal of Ralls Newman, LLC, in Austin.

Traffic must continue to flow as these substandard bridges are being replaced, and cost efficiencies are needed to optimize the use of the limited available funding. Accelerated bridge construction (ABC) can help address these challenges, and much progress has been made in the use of ABC over the past decade. According to the 2014 *Annual State Bridge Engineers' Survey* of the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures (SCOBS)—in which 47 state departments of transportation (DOTs) responded—ABC has been used in 43 states, and only three state DOTs responded that they have not used ABC. During the same period, progress has also been made towards ABC as standard practice. One state, Utah, has adopted programmatic implementation of ABC, and a number of other states are moving in that direction.

Although sometimes overlooked due to the competitive nature of the transportation industry, construction contractors can be, and in some states are, significant partners with owner agencies in moving ABC to standard practice. And contractors are increasingly supporting the use of ABC principles for a variety of reasons. The improved constructability and cost savings when building multi-span bridges with repetitive elements is a primary reason. Others include safety concerns for crews and the traveling public when working in water or over electric power transmission lines, or working on bridge replacements in locations with limited site distance or space or high traffic volumes. The ability to minimize work in environmentally sensitive areas also provides an incentive for contractors to consider ABC technologies even on low-traffic-volume roads.

Massachusetts 93Fast14 involved the replacement of 41 spans on 14 bridges during 10 weekend closures.

Courtesy CME Associates

A folded plate girder during fabrication.

Courtesy CDR Bridge Systems

Courtesy Walsh Construction

The Milton-Madison Bridge over the Ohio River between Kentucky and Indiana during lateral slide of its four river spans.

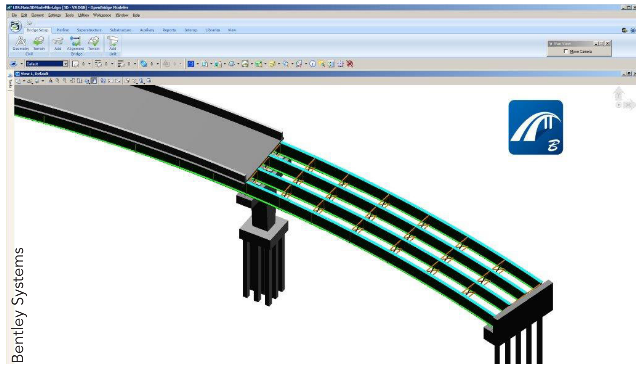
Prefab is the Key

So how is ABC defined? Perhaps its most widely recognized characteristic is the use of prefabricated bridge elements and systems (PBES)—and to fully grasp the meaning of ABC, one must first understand PBES as presently defined. The Federal Highway Administration (FHWA) has provided PBES definitions—search "PBES" at **www.fhwa.dot.gov**—that have generally been adopted by the SCOBS Technical Committee for Construction (T-4); SCOBS has designated T-4 as the focal point for ABC implementation among the states. *Element* categories are prefabricated decks, beams, piers, abutments and walls. In addition, the miscellaneous elements category includes precast approach slabs, prefabricated parapets, deck closure joints and overlays. Various elements in each of these prefabricated element categories have been constructed in the U.S. to date. Prefabricated *systems* include whole superstructure systems and combined superstructure/substructure systems that can be installed in one piece at one time. In general, prefabricated elements can be erected with conventional construction equipment, whereas prefabricated systems require innovative construction equipment due to the significantly heavier system self-weight. Below are three examples of the most commonly used PBES.

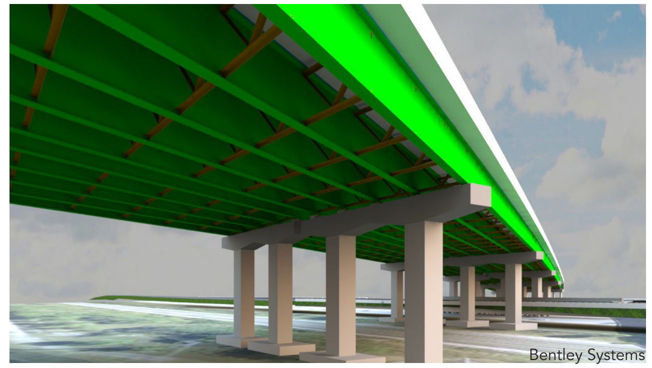
Modular decked beams. Currently, one of the most popular prefabricated ABC elements is the modular decked beam, consisting of either steel or concrete beams pre-topped with concrete deck. An example of modular decked beam use is the 2011 Massachusetts 93Fast14 project on Interstate 93 through the city of Medford. (The project, a 2012 NSBA Prize Bridge Awards winner, was featured in the September 2014 article "Piece by Piece," available at **www.modernsteel.com**.) In this project, 41 spans on 14 bridges were replaced during 10 weekend closures. The modular decked beams for this project were composed of two steel I-shaped girders pre-topped with a composite concrete deck. Deck closure joints between beams were 32 in. wide and filled with high-early-strength concrete. The width of these joints was selected to reduce the width and weight of the modular decked beams to facilitate shipping and handling as well as permit conventional reinforcing lap splices within the closure joints. (Filling narrow closure joints with ultra-high-performance concrete—UHPC—is becoming a popular option, one made possible through FHWA's extensive research and development activities in collaboration with state DOTs and industry.)

Another modular innovation was introduced at the University of Nebraska-Lincoln, with development continued at Florida International University: a streamlined modular decked steel beam cross section known as the "folded steel plate girder bridge system." In 2014 this solution was incorporated in Nebraska's Primrose East Bridge in Boone County. The 50-ft-long, 32-ft-wide, single-span bridge has four 28-in.-deep girders that were match-cast with 8-in.-thick concrete deck panels and end diaphragms by the contractor at a nearby staging area. The contractor then transported the decked beams to the site for erection, and the 8-in.-wide deck closure joints were filled with UHPC.

Self-propelled modular transporter moves. When it comes to prefabricated bridge systems, two installation methods are currently seeing wide use. They install complete superstructure spans composed of steel or concrete beams pre-topped near the final bridge location with full-width, full-depth composite concrete decks. The first installation method uses self-propelled modular transporters (SPMTs), which are ideal for use on bridge projects over Interstate highways or other high-traffic volume roadways.



An intelligent, parametric 3D steel bridge model.



Bridge information modeling (BrIM) can speed overall bridge project time while reducing clashes and enhancing accuracy.

The initiative for widespread use of SPMTs to move bridge spans in the U.S. began after the 2004 FHWA/AASHTO/Transportation Research Board (TRB) International Scan on Prefabricated Bridge Elements and Systems toured SPMT companies in Belgium and the Netherlands and observed the speed and flexibility with which bridge spans were being installed with SPMTs. The Florida Department of Transportation was the first in the U.S. to use SPMTs to remove and replace spans over a U.S. Interstate. Taking place in 2006, the project was on Interstate 4 northeast of the city of Orlando and incorporated SPMTs during partial overnight closures of the highway. Since then, scores of bridge spans have been installed with SPMTs. Another example is the Utah DOT's Sam White Lane Bridge over Interstate 15 in the city of American Fork. This 354-ft long, 77-ft wide, two-span continuous steel plate-girder superstructure—with a 48° skew and a 1,910-ton self-weight—was moved into position during an 8-hour overnight road closure in 2011.

Lateral slides using hydraulic jacks or winches. The second prominent ABC installation method for prefabricated systems is the lateral slide. This is an ideal technology for high-traffic-volume bridge replacement projects over low-traffic-volume roadways or river crossings. While lateral slides have been used occasionally over the past decade to move spans into place, their use has increased significantly since FHWA's 2013-2014 Every Day Counts 2 (EDC-2) "slide-in bridge construction" initiative focused on this technology. The largest truss slide to date is the Milton-Madison Bridge on U.S. Route 421 across the Ohio River between the towns of Milton, Ky., and Madison, Ind. In 2014 the four 48-ft-wide steel through-truss river spans, totaling 2,427 ft in length and 15,260 tons in weight, were slid into place using computer-controlled hydraulic strand jacks. (The project was featured in the August 2014 news section and also in the February 2012 article "Move that Bridge," both available at **www.modernsteel.com**).

Different Angles on ABC

In addition to PBES, the bridge design and construction community is taking a multifaceted approach to ABC and exploring and implementing other initiatives as well.

Bundling bridges. A primary goal of bundling bridges in a project is to reduce cost with volume. The Missouri DOT's Safe and Sound Bridge Improvement Program, completed in 2012, and the Pennsylvania DOT's Rapid Bridge Replacement Program, currently underway, are examples of two DOTs that consolidated improvement/replacement work on hundreds of substandard bridges into single projects. Bundling bridges can also be an effective tool on a smaller scale for bridge owners with multiple relatively short substandard bridges within a limited distance. For example, cost efficiencies can be achieved in a county or multi-county project with a half dozen single-span bridges within a short distance, all replaced with prefabricated elements of the same type and length stockpiled prior to construction.

Bridge information modeling. Bridge information modeling (BrIM) can speed overall bridge project time from planning through construction while reducing clashes and enhancing accuracy. This is accomplished by using data, developed in design, for fabrication and construction as well as other phases in a bridge's life cycle. Although BrIM is more widely known for its use on large projects such as the Tappan Zee Bridge, the general benefits of BrIM—data reuse, change management, and collaboration—can be realized in bridge projects of all sizes. Like its building counterpart (BIM), it can help ABC and other projects see faster production with fewer errors, resulting in time and cost savings.

State DOTs are starting to use this 3D intelligent modeling in their planning, design, and construction of bridges across the county. Currently 29 DOTs plan to implement it in their agency's culture during 2015 and 2016. An additional 15 states and the Federal Lands Highway plan to integrate it pending a two-year assessment cycle. By December 2016, it is expected that 16 DOTs will have the new methodology institutionalized, 17 will be in the assessment phase, 12 will be in the demonstration phase and two will be in the development phase.

Total cost estimation tools. ABC significantly reduces the number of days in the work zone, but to date, bridge owner and contractor savings related to the reduced number of days in the work zone are not typically included in cost comparisons between ABC and conventional construction. Similarly, the most frequent reason for the use of ABC is to reduce traffic congestion, but in many cases user costs are not included in cost comparisons between ABC and conventional construction. Work has been done in some states and is underway in others as well as at the ABC University Transportation Center (ABC-UTC) at Florida International University, to develop tools to estimate total costs of ABC and conventional construction. For example, the Connecticut DOT has recently developed an ABC decision matrix that includes estimated construction inspection overhead costs associated with differing project durations for conventional construction versus ABC. It also includes measures to weigh the cost of conventional construction with overbuild and/or temporary construction with minor long-term traffic impact, versus the cost of ABC with road closures, detours or more significant short-term traffic impacts. In addition, it captures contractor costs. Another example is an ongoing ABC-UTC research project to create a framework for evaluating and using construction and user costs as part of the decision-making processes associated with bridge construction, as well as a total cost analysis and estimation tool.

When it comes to such estimates, keep in mind that a specific project's design can be significantly different when taking an ABC approach versus a more conventional approach. A paradigm shift is needed when considering costs, as the idea of a conventional cost estimate versus an ABC cost estimate is an old train of thought. There should be no one type of estimate versus another. Proper project planning leads to the most appropriate project cost. Within the project planning process, the objective is to define the goals of a project—and in most cases this means to reduce the impacts to the public. If ABC is a tool that aides in meeting the established goals of the project, then any additional cost of using ABC is secondary to those goals. One should define the project goals and set the project budget to account for all project needs and requirements.

Future Opportunities to Advance ABC

Owner agencies are typically the stakeholders in the best position to take the lead in making ABC standard practice because of their obvious influence and their consideration of the traveling public that crosses their bridges. The collaboration of academia, industry organizations and consultants, in partnership with bridge owners and construction contractors and suppliers, provides the opportunity to accelerate the advancement of ABC as standard practice.

But making ABC standard practice does not mean that ABC is actually used on every bridge project. Instead, it means an owner agency, in support of its traveling public, considers ABC as the default in the initial planning phase of every bridge project and has a decision-making tool that evaluates whether ABC or conventional construction is the best solution for that specific project. It means an owner agency's leadership and staff members understand the benefits and challenges of transitioning to ABC as standard practice and that they are committed to following through in collaboration with their bridge community. Each owner agency must determine how best to transition to ABC within their organization; for starters, owners could designate a champion to lead a multi-disciplinary team specifically charged with transitioning to ABC as standard practice.

The framework and opportunity to take advantage of ABC's benefits are now known, and the momentum is growing across the country for ABC as standard practice. And in the face of the daunting statistics on substandard bridges in the U.S., ABC is becoming more important than ever. ■