Steel Bridges 2016–2018

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Modern Steel Construction

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With budgeting the cost of Steel, Labor, Demolition, Design & Engineering from year to year being increasingly difficult to forecast, the option of a new Bridge has some Counties in Ohio electing to look into the cost of **REHAB** instead of **REPLACE**.

The Wyandot County bridge engineers, with the help of a local bridge design/build fabricator (US Bridge Cambridge, Ohio), who asked V&S Galvanizing to consult on a feasibility study to not only save a historic 103 year old county bridge, but compare the economic benefits, and in turn, the sustainability effect on our natural resources.

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OR

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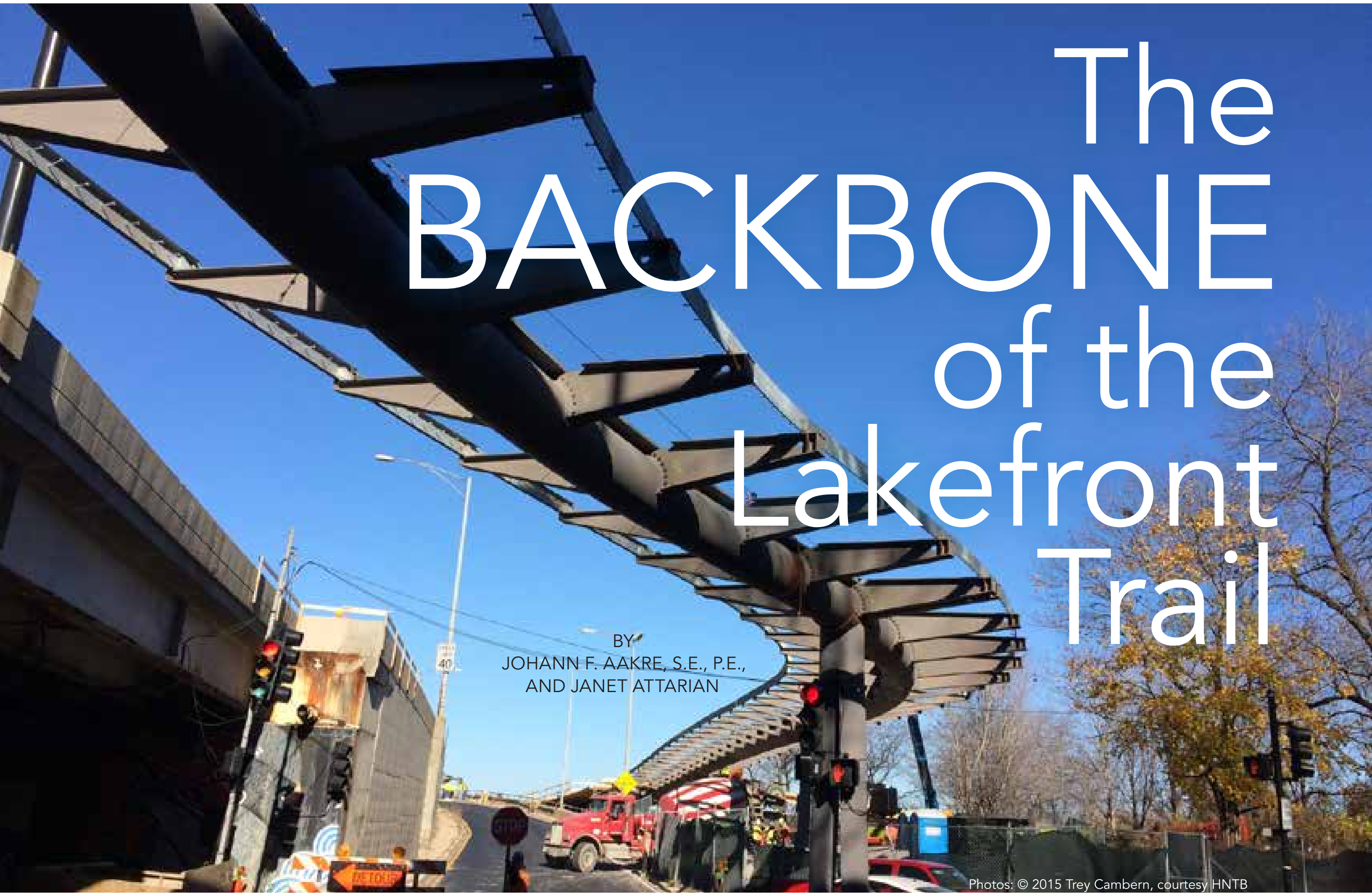
B018-18

A new elevated pathway provides safe passage for pedestrians and cyclists near one of the Windy City's premier tourist destinations.



The BACKBONE of the Lakefront Trail

BY JOHANN F. AAKRE, S.E., P.E., AND JANET ATTARIAN

Photos: © 2015 Trey Cambern, courtesy HNTB

CHICAGO'S LAKEFRONT TRAIL is a popular cycling, walking and running venue for residents and tourists alike.

The 1,750-ft-long, $60 million "Navy Pier Flyover" now under construction as part of Mayor Rahm Emanuel's "Building a New Chicago" program will eventually relieve congestion along the most heavily used section of the 18.5-mile trail.

Site constraints and limited clearances created by a parking garage, a residential high-rise, local streets, highway access ramps and several parks, a serpentine alignment was essentially the only option for the grade-separated pathway. Aesthetics were equally as important as functionality because the flyover will be visible from every angle in a highly visible part of the city. A dramatic steel spine-rib superstructure support system became the essential component that helped the team achieve objectives of both form and function.

Construction is being executed in three phases: The first, northern phase is between Jane Addams Park and Ogden Slip; the second stretches from this slip to the Chicago River; and the third phase will span the river by modifying the path on the Lake Shore Drive bridge. Construction of the supporting steel superstructure for phase 1 is now finished as the $27.9 million first segment nears completion.

Eliminating a Bottleneck

With 60,000 users traversing this segment of the Lakefront Trail during peak use, in the late 1990s the Chicago Department of Transportation (CDOT) began looking at ways to alleviate the bottleneck created when trail users crossed paths with pedestrians on city streets, local vehicle traffic and highway entrance ramps. All of this congestion, combined with missing or deteriorated pavement markings, lack of way-finding signage and poor trail surface conditions were contributing to frequent accidents.

In 2003, HNTB began working with CDOT and Muller and Muller Architects to develop a bridge configuration that was functional, aesthetically pleasing and contextually appropriate. The overriding objective for the new path was to separate shared crossing points between pedestrians and vehicles at Illinois Street and Grand Avenue, the heavily congested (not to mention only) entry and exit streets to the Navy Pier facilities.

Initially, finding a viable path for the structure was the most formidable challenge. In one area, the partial removal of the east shoulder of upper Lake Shore Drive was deemed necessary to squeeze the elevated path between it and Lake Point Tower, an iconic residential high-rise. Given the proximity of the path to the 70-story condominium tower—there will be only 9 in. separating them—concessions were made with residents to erect a mesh screen wall between the structures for added security.

In another section, the path alignment had to be shifted across the currently undeveloped DuSable Park, designated as a Superfund site by the U.S. Environmental Protection Agency because of radioactive thorium nitrate present in fill material placed in the early 1900s. To minimize the need for soil remediation at flyover support columns, HNTB used minimally invasive steel H-pile foundations.

A Flexible Spine

Tasked with designing a serpentine structure that would be practical, structurally sound and visually appealing, HNTB incorporated the use of an easily manipulated, central steel spine. The longitudinal spine-rib support system accommodates the complex bridge's need to curve both horizontally and vertically and provides the desired aesthetics.

The bridge's central spine is fabricated from 30-in.-diameter steel pipe, either 1¼ in. or 1¾ in. thick depending upon span and strength requirements. A T-shaped web and flange are welded on top of the pipe to provide added strength and a surface for shear studs, which enable composite action between the steel and the path's 6-in.-thick, 17-ft, 10-in.-wide concrete deck. For Phases 1 and 2, nearly 350 tons of API 2B steel pipe will be used in the spine.

The size of the pipe combined with the serpentine alignment did come with a challenge. Since the project is federally funded, the steel needed to meet Buy America provisions and most pipe of this size and thickness is produced overseas. In discussions with fabricators and the steel industry, it was determined that the pipe could be produced domestically in accordance with API 2B specifications, which is for pipe manufactured from plate that is rolled into cans and then longitudinally welded.

Rib elements, fabricated from steel plates, are connected to each side of the steel spine on 8-ft centers, tapering in

◄ ▲ ▼ The 1,750-ft-long, $60 million "Navy Pier Flyover," now under construction as part of Mayor Rahm Emanuel's "Building a New Chicago" program, will eventually relieve congestion along the most heavily used section of the 18.5-mile Lakefront Trail.

Johann F. Aakre is a project manager with HNTB Corporation and **Janet Attarian** is Livable Streets director with the Chicago Department of Transportation.

depth from approximately 2 ft, 2 in. at the central spine to less than 5 in. at the outer deck edge to create a sleek, graceful appearance. A longitudinal steel channel, running parallel to the steel pipe spine, is bolted to the ends of the steel ribs to facilitate construction of the deck and to support the path railing.

The design uses the frame action created by the spine-column rigid connection. This not only controls in-plane bending, but also resists out of plan bending and torsion in the spine. As such, the analysis considered the column supports of the structure, rather than treating the superstructure as a continuous beam supported atop various piers. Thermal range was also factored into the design, requiring the development of unique expansion bearing and column connections. In one location, where the spine is supported on Lake Shore Drive itself, a dapped connection was inserted into the spine so that its thermal displacements would act in conjunction with the Lake Shore Drive Bridge. At this location and at the expansion piers, the spine is supported by low-profile disc bearings, serving to maximize the amount of steel pipe available to carry the forces at the bridge supports.

Connections between the steel elements of this unique structure also came with a set of challenges. The connections needed to fit within the aesthetic constraints of the project and custom details were developed. Where possible, AISC design guidelines for HSS connections and CIDECT publications were used as resources. In some cases, however, refined 3D finite element models were relied upon where the details did not fit entirely within the context of the code.

Steel Substructure and Foundation

The steel spine is supported by steel columns created from 1¼-in. steel plate bent into a 30-in. by 22-in. elliptical shape along the main alignment and by cantilevered concrete abutments at its ends. At three locations, the path is supported directly from the existing bents of Lake Shore Drive. The elliptical column extends 2 ft below the bottom of the spine before separating into two half-elliptical column legs that splay apart to create a wider base for additional stability. This arrangement was devised to provide an aesthetically continuous and pleasing transition between the bridge elements.

The foundation system is primarily comprised of steel piles driven to 50-ton capacity and embedded into a 3-ft, 3-inch-thick rein-

▲▼ The flyover spine is supported by steel columns created from 1¼-in. steel plate bent into a 30-in. by 22-in. elliptical shape along the main alignment and by cantilevered concrete abutments at its ends. At three locations, the path is supported directly from the existing bents of Lake Shore Drive.

▲▼ The bridge's central spine is fabricated from 30-in.-diameter steel pipe, either 1¼ in. or 1¾ in. thick depending upon span and strength requirements.

▲ A full-scale mockup of the spine, deck and railings was constructed on-site to refine the details.

▼ The path will come as close as 9 in. to an adjacent 70-story residential high-rise.

Steel plate rib elements are connected to each side of the steel spine on 8-ft centers, tapering in depth from approximately 2 ft, 2 in. at the central spine to less than 5 in. at the outer deck edge.

The design uses the frame action created by the spine-column rigid connection. This not only controls in-plane bending, but also resists out of plan bending and torsion in the spine.

forced concrete cap. Steel piling was selected as the preferred foundation type since it minimized the amount of excavation and spoils, especially necessary when working at the Superfund site.

Attention to Detail

All details were highly scrutinized to ensure that the desired function and look would be achieved. Architectural cable steel railings and panels and custom steel deck nosings were incorporated into the design. In addition, the paint system is not the typical zinc/epoxy/urethane system used on highway bridges, but rather a three-coat system comprised of a primer, an intermediate coat and a fluoropolymer finish coat. Typically used on building applications, this system is highly durable and provides enhanced color and finish retention.

Highlighting these details is a comprehensive LED lighting system that will illuminate the ribs, spine and columns from below and will shine down from the cable railing posts above. Most conduits for lighting power supply are embedded in the concrete curb or are enclosed within the steel elliptical column sections. Drainage downspouts were custom detailed to follow the path of the ribs and columns to integrate with the structure. A ⅜-in.-thick stainless steel curb cover plate extends along the deck edges to provide a uniform shape and enhanced appearance. To ensure that all details were worked out during construction, the contractor created a full-scale mock-up before starting full production.

When all three phases are complete in 2018, the elevated pathway will significantly improve this key segment of the Lakefront Trail, safely guiding pedestrians and cyclists through one of the most heavily trafficked regions of the city. By maneuvering the path through a 3D obstacle course of existing structures, the pathway's steel spine system has proven itself to be up to the challenge. ■

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➤ When all three phases are complete in 2018, the elevated pathway will significantly improve a key segment of the Lakefront Trail.

Owners
Chicago Department of Transportation (primary), Chicago Park District, Illinois Department of Transportation

General Contractor
F. H. Paschen

Construction Manager
T.Y. Lin International Group

Architect
Muller and Muller Architects

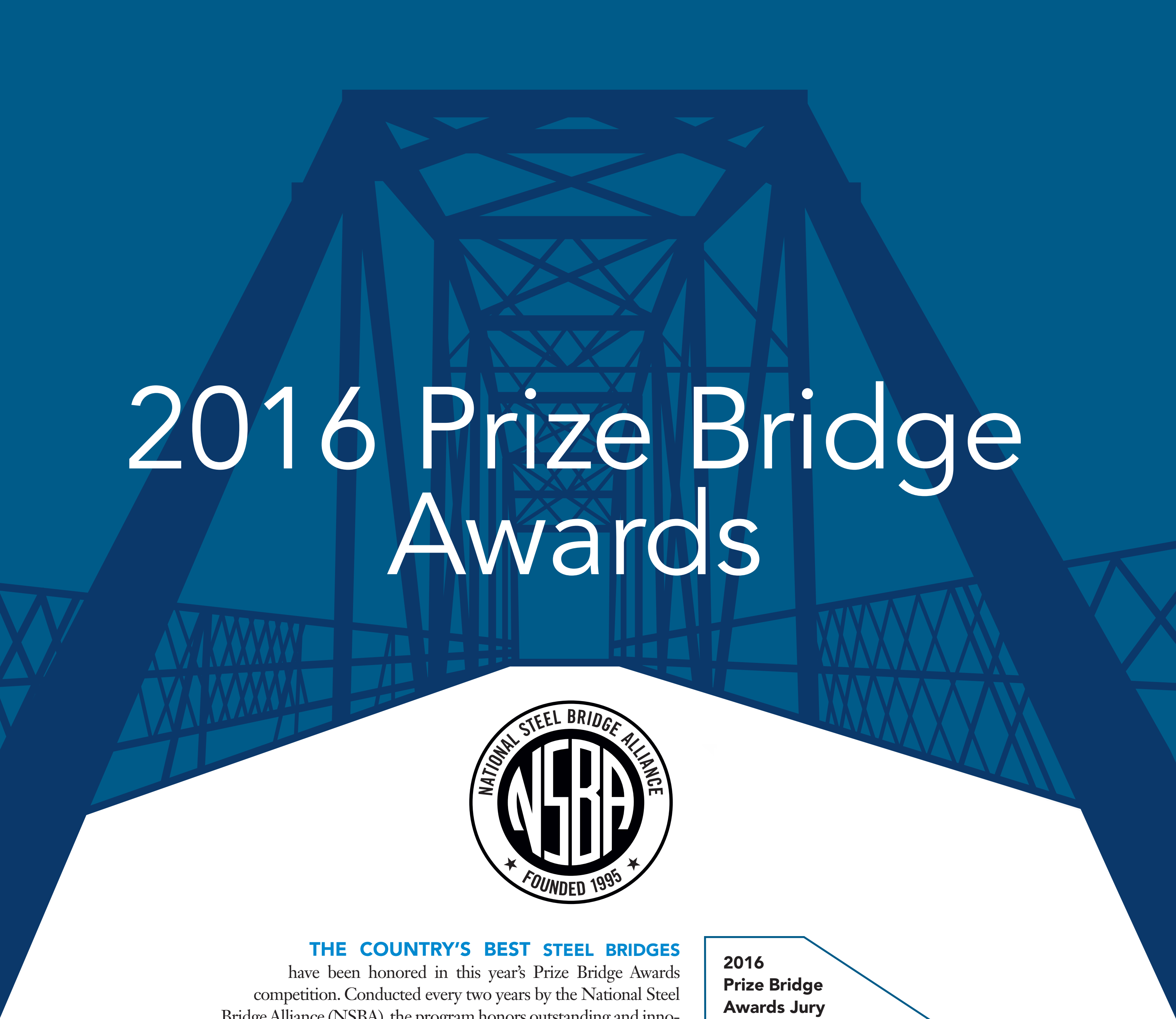
Structural Engineer
HNTB Corporation

Erection Engineer
Bowman, Barrett and Associates Inc.

Steel Team

Steel Fabricator
Hillsdale Fabricators, an Alberici Enterprise AISC CERTIFIED FABRICATOR

Bender-Roller
Bendco AISC ASSOCIATE



2016 Prize Bridge Awards

THE COUNTRY'S BEST STEEL BRIDGES have been honored in this year's Prize Bridge Awards competition. Conducted every two years by the National Steel Bridge Alliance (NSBA), the program honors outstanding and innovative steel bridges constructed in the U.S. The awards are presented in several categories: major span, long span, medium span, short span, movable span, reconstructed, special purpose, accelerated bridge construction and sustainability. This year's 16 winners, divided into Prize and Merit winners, range from a mammoth marquee Mississippi River crossing to the country's first steel extradosed bridge. Winning bridge projects were selected based on innovation, aesthetics and design and engineering solutions, by a jury of five bridge professionals.

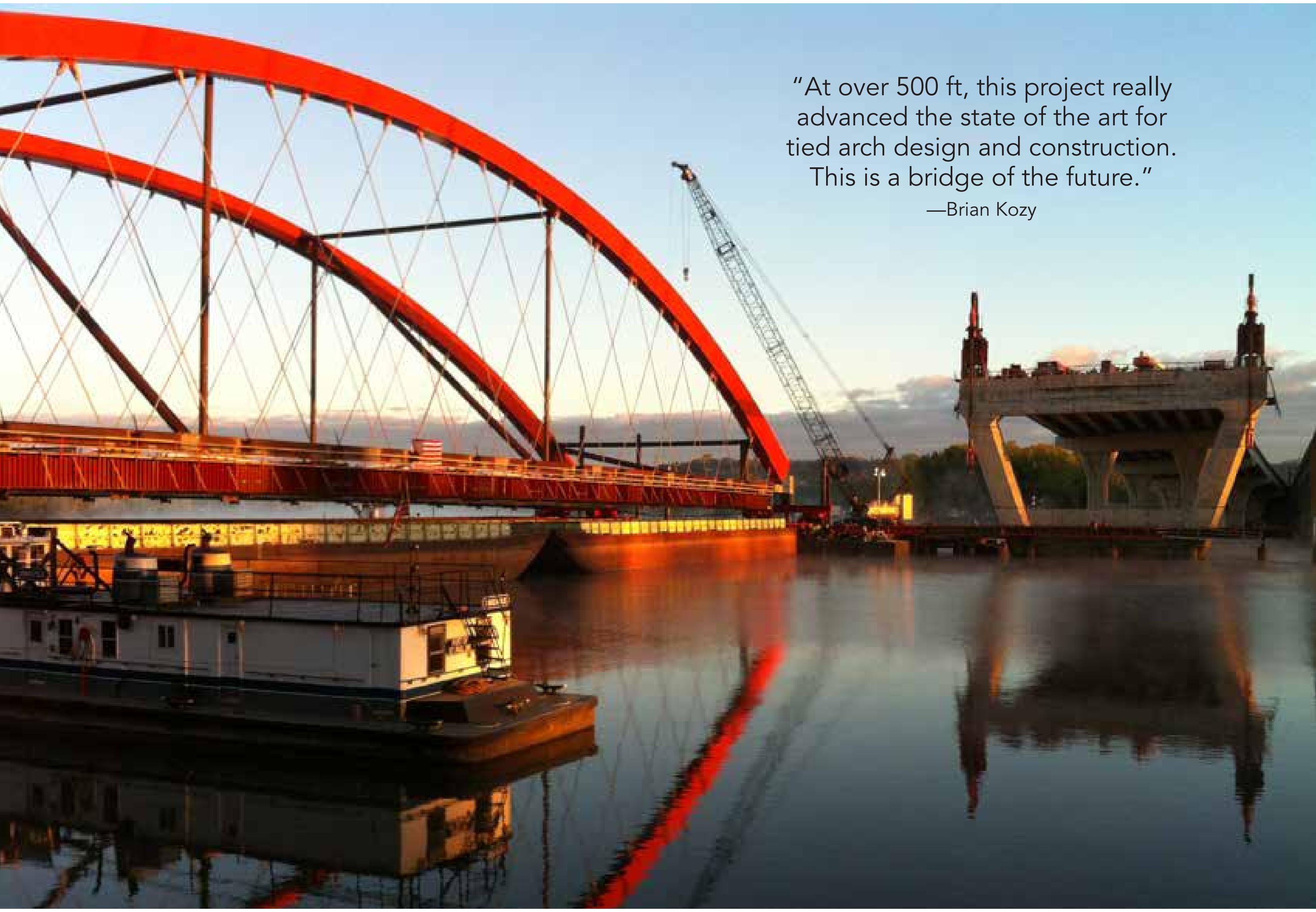
This year's competition included a variety of bridge structure types and construction methods. All structures were required to have opened to traffic between May 1, 2013 and September 30, 2015.

The competition originated in 1928, with the Sixth Street Bridge in Pittsburgh taking first place, and over the years more than 300 bridges have won in a variety of categories. Between 1928 and 1977, the Prize Bridge Competition was held annually, and since then has been held every other year, with the winners being announced at NSBA's World Steel Bridge Symposium. The following pages highlight this year's winners. Congratulations to all of the winning teams! And check out past winners in the NSBA archives at **www.steelbridges.org**.

2016 Prize Bridge Awards Jury

- **David Spires**, P.E. Senior Engineering Manager with WSP Parsons Brinckerhoff
- **Michael Culmo**, P.E. Vice President of Transportation and Structures with CME Engineering
- **Brian Kozy**, P.E., Ph.D. Structural Engineering Division Team Leader with FHWA
- **Steve Jacobi**, P.E. State Bridge Engineer for the Oklahoma Department of Transportation
- **Carmen Swanwick**, S.E. Chief Structural Engineer for the Utah Department of Transportation

PRIZE WINNER: MAJOR SPAN
Hastings Bridge, Hastings, Minn.



THE HASTINGS BRIDGE over the Mississippi River in Hastings, Minn., is a record-breaker.

Built as a replacement for the functionally obsolete Hastings High Bridge (built in 1950), the new 1,938-ft-long bridge—with a 545-ft main span—is the longest freestanding tied-arch bridge in North America. The overall project was accelerated through MnDOT's Chapter 152 Bridge Improvement Program following the I-35W bridge collapse. MnDOT identified this route as critical to the mobility and commerce of Minnesota because it carried the highest daily traffic volume of any two-lane trunk highway in the state.

The bridge was constructed using design-build procurement and required accelerated bridge construction (ABC) technology to meet the demanding schedule and limit impacts on the travelling public. The tied-arch structural system is comprised of two freestanding vertical structural steel arch ribs with trapezoidal cross sections and variable depth. Post-tensioning steel strands were used to resist the arch thrust and encased in cast-in-place concrete tie girders and knuckles. The structural steel floor system consists of a grid of floor beams, full-depth longitudinal stringers and secondary longitudinal stringers all made composite with a cast-in-place concrete deck. The knuckles and deck are integral with the piers, creating a fully framed system. A network of structural strand hangers is used to suspend the floor system from the arch ribs.

All structural tension members are load-path redundant for fracture at any point in a single member or connection subject to tension under permanent loads and vehicular live load. Consequently, there are no fracture-critical bridge elements on the structure. The structure was analyzed for fracture of all tension members using a 3D time-history analysis to determine appropriate dynamic effects. The transverse floor beams and full-depth longitudinal stringers form a grid floor system, which allows load transferring in both the longitudinal and the transverse directions. This structural steel grid forms a redundant system with the primary load path through the transverse floor beams. The full-depth longitudinal stringers provide multiple supports, which minimize deflections from the potential fracture of a floor beam and significantly reduce the resulting fracture energy release and dynamic impact.

The design-build team determined very early that the traditional methods of erecting the arch off-site on high towers and floating it in over the piers was too risky due to the high center of gravity and vari-

ability of river water elevations, which could delay move-in. Therefore, the team elected to erect the arch on land, transfer it onto barges with self-propelled modular transporters (SPMTs), float it in low, position it between the piers using a skid rail system and lift it into place with strand jacks on top of the piers.

The steel floor beams and longitudinal stringers were erected on land in the staging area by the river bank with temporary supports. A temporary tension tie system, consisting of two W36 sections to resist the thrust of each arch rib, was used to facilitate the erection and served to stabilize the floor system and support the formwork for the cast-in-place tie girder. A steel lifting connection served as a temporary knuckle connecting the arch rib with the temporary tie. Finally, the hangers were installed between the arch rib and the ends of each floor beam. The arch ribs were braced during erection, and the entire system was framed using the temporary rib bracing, floor system and the lower lateral bracing system.

After completing the steel member erection on land, eight 16-axle SPMTs were brought in and situated with two under each of the corners of the arch system. The vertical lifting ability of the SPMTs was used to lift each of the four corners of the arch in unison, bringing the arch off of its support towers and the floor system off the temporary supports. The total vertical lift was approximately 6 in. to account for the deflection of the arch and elongation of the tie as the arch picked up the weight of the floor system. The wheels of the SPMTs at one end of the arch were rotated 90° to allow them to roll with the elongation of the temporary tie girder. After a successful lift-off, the wheels were rotated back to prepare for the move down the slope to the river bank, while all the temporary supports and towers were taken down.

The SPMTs under the corners at each end were connected together to act in unison for moving the arch system transversely down to the river bank and over a trestle onto barges. Water level monitors at each corner of the arch were used to check the slope between the ends of each arch and the SPMTs were adjusted vertically to maintain a constant slope between the arches and avoid twisting the floor system as they marched the arch down a 3.5% slope to the river and onto the barges.

One barge was positioned at each end of the arch to allow each 104-ft-wide end to roll onto the barge from one end toward the center until both sides of the arch were positioned in the center of the two barges. The barges were constantly monitored and re-ballasted as the

SPMTs rolled each end of the arch onto the barges. The total move onto the barges took about 12 hours.

The arch was floated down stream to the bridge site and positioned adjacent to the piers. Due to the curve in the river bank and the south piers' position on the river bank, the arch was skidded south off the south barge onto the river bank with a skid track system until it lined up with the horizontal skid rails that were positioned between the piers. Once in position on the south end, the support was transitioned from longitudinal to transverse skid shoes. The north end of the arch was unloaded off the barge onto the skid rails during the transverse slide with the help of SPMTs on the barge. Once positioned between the piers, the arch was ready for lifting.

The lifting frame supporting the strand jack system was anchored directly to the top of the pier. The strand jack system was connected to the arch system-lifting connection and hoisted 55 ft onto the top of the piers. Pier deflections were monitored and checked to ensure clearance after liftoff from the skid rails. Once in place, a support frame was moved into position under the temporary knuckle and the bridge was lowered into its final position. The lifting connection and support frame were cast into the permanent concrete knuckle. The concrete tie girder and knuckle were post-tensioned sequentially as the knuckle, tie and deck concrete were placed. To compensate for creep, shrinkage and shortening, the piers were jacked apart 6 in. before casting the knuckle, and the temporary arch bracing remained in place until the deck was cast. The deck was placed in a single pour, beginning at the center of the bridge. Hanger adjustments for geometry and stress were made by modifying the heights of the shim packs at each hanger. The float-in and lift process for the 3,300-ton arch steel structure was completed within a 48-hour window to limit the amount of time the Mississippi River navigation channel was closed.

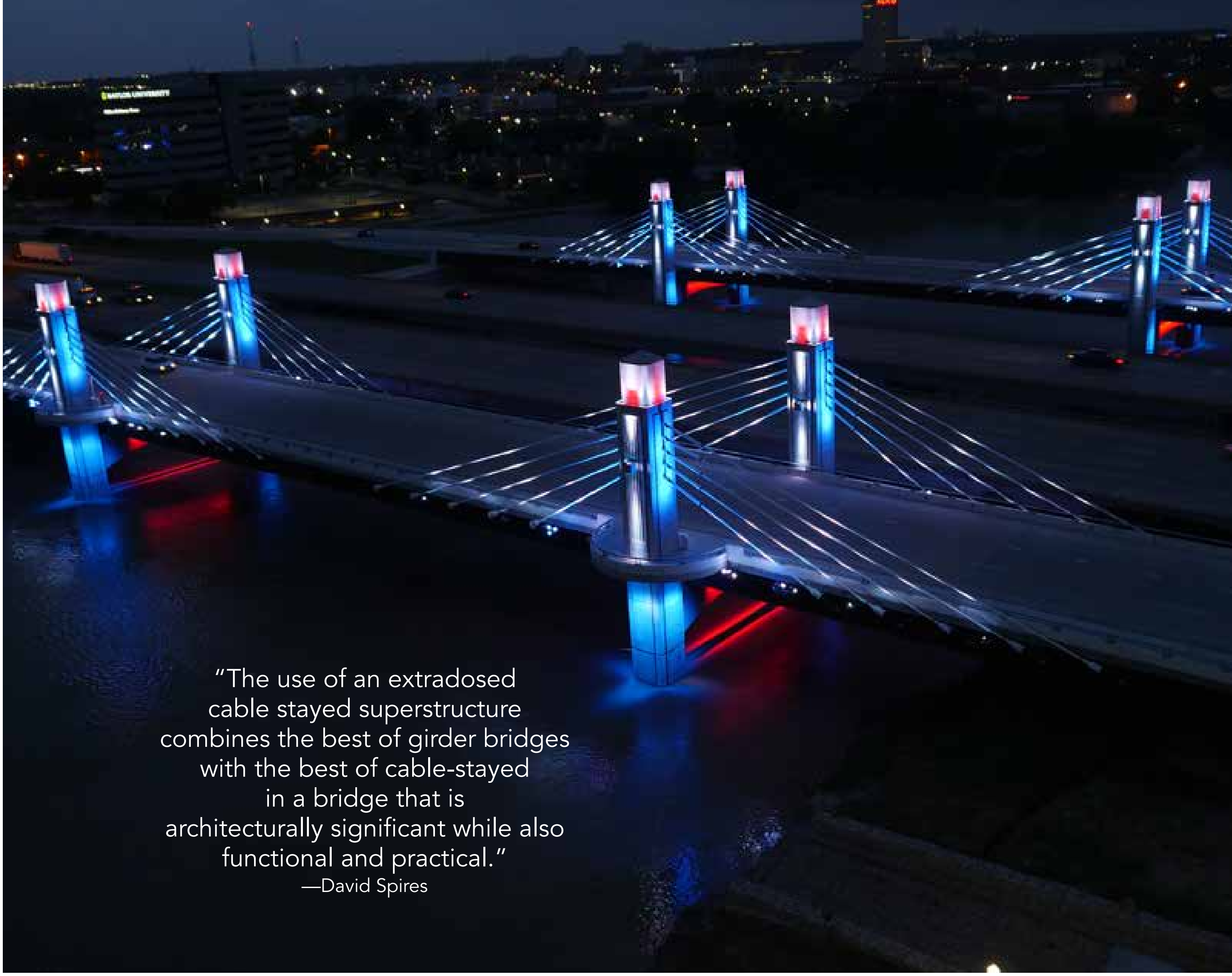
Owner
Minnesota Department of Transportation, St. Paul

Designer
Parsons Corporation, Chicago

Contractor
Lunda Construction Company, Rosemount, Minn.

Steel Fabricator
Veritas Steel, Eau Claire, Wis. AISC CERTIFIED FABRICATOR

Steel Detailer
Candraft Detailing, Inc., AISC DETAILER
New Westminster, B.C., Canada



PRIZE WINNER: LONG SPAN

IH-35 Bridges over the Brazos River, Waco, Texas

TEXAS, LIKE MANY OTHER SOUTHERN STATES, has seen substantial population growth recently.

This growth is one of the key drivers for expanding Interstate capacity, particularly in cities like Waco. I-35 meanders through the east Texas city, home of Baylor University, and the Waco District of the Texas Department of Transportation (TxDOT) wanted to do more than just add capacity with the $17.3 million IH-35 frontage road bridge project. It wanted to make a statement. By pioneering the application of extradosed bridges in the U.S., the city of Waco and TxDOT did just that (see sidebar for a description of extradosed bridge design).

Spanning the Brazos River and parallel to the existing mainline I-35 bridges, the new IH-35 frontage road extradosed bridges are 620 ft long and are the first extradosed cable-stayed bridges in the U.S. to use a steel superstructure. Traffic on each bridge is one way, with the new bridges placed to the outside of the mainline bridges. The existing mainline bridges will soon be replaced with new steel box-girder bridges as part of the IH-35 corridor improvements. In the final configuration, both the northbound and southbound frontage road bridges will be separated horizontally some 60 ft from the corresponding mainline bridges.

The roadway for each frontage road bridge carries three traffic lanes with shoulders, as well as a 10-ft, 6-in.-wide sidewalk for pedestrians and cyclists. In addition, scenic overlooks providing unobstructed river views are incorporated into the pylons to enhance the bridge experience for pedestrians. Each of the new twin structures is a three-span bridge with a 250-ft center span and 185-ft side spans. Matching the span configuration of the proposed new mainline bridges, this configuration aligns the piers within the river for all the bridges in their final condition.

Each bridge's superstructure consists of 6-ft, 6-in.-deep steel trapezoidal box edge girders, 3-ft, 6-in.-deep steel-plate I-girder floor beams and 10.5-in. cast-in-place concrete deck. Transverse floor-beam spacing varies from 13 ft, 3 in. in the end region of the side spans to 15 ft in the regions near the pylons. The trapezoidal box edge girders are composed of ¾-in. web plates, with 2-in.-wide top flange plates and a 5-ft-wide bottom flange plates. Top flange plate thickness varies from a typical 1 in. up to 3 in. for the regions

over the bearings at the pylons, and the bottom flange plate varies from a typical 1¼ in. to 3 in. over the pylons. The box edge girders are continuous for the entire length of the bridge, supported on single disc bearings at the abutments and pylons. The transverse I-girder floor beams consist of ½-in. web plates with 1-ft, 6-in.-wide by 1-in.-thick top and bottom flange plates. Each H-shaped pylon consists of two 9-ft, 3-in. by 9-ft, 3-in. rectangular towers with a haunched 5-ft, 3-in.-wide crossbeam that supports the superstructure.

The project team chose a steel-girder composite cross section for the design. While a concrete cross section is typical for extradosed bridges, TxDOT preferred steel girders with concrete decks because of its familiarity with this superstructure scheme. In addition, structural engineer AECOM evaluated the use of a cast-in-place concrete box girder superstructure but determined it to be economically untenable. In addition, the project team used a steel trapezoidal box section for the edge girders, rather than a steel-plate I-girder section more commonly used on composite cable-stayed bridges, in order to provide greater superstructure stiffness and less reliance on the cable stays. The team also wanted to visually match the new mainline bridges, which will be steel box-girder bridges. The cable system

An Extra Dose of Strength

Unlike traditional cable-stayed bridges, extradosed bridges use a combination of superstructure as well as cable stays to support the loads. These bridges have a distinguishing feature from traditional cable stayed bridges in that the tower height is much shorter in proportion to the main span. While cable-stayed bridges typically have tower heights around one-fourth to one-fifth of the main span length, extradosed bridges have tower heights equal to approximately one-tenth of the main span length. The shorter tower height results in shallower cable angles that in turn increase the axial compression in the superstructure and decrease the vertical stay forces that act as supports in a conventional cable-stayed bridge. In other words, cables on an extradosed bridge serve a prestressing function.

Also, due to the additional support of the cables, an extradosed bridge may have a shallower superstructure depth relative to a traditional girder bridge. The span-to-depth ratio of extradosed bridges is typically on the order of 35-to-1 versus approximately 20-to-1 to 25-to-1 for typical girder bridges. However, an extradosed bridge still acts as a girder bridge, so the superstructure depth is greater than a conventional cable-stayed bridge, with a typical span-to-depth ratio of approximately 100-to-1. In addition, due to an extradosed bridge's relatively stiff superstructure, which resists a majority of live-load forces (rather than having the stays carry the load), these bridges also are often characterized by low live-load stress ranges in the stay cables.

consists of a single plane of five cable stays at each pylon supporting the edge girders (a total of 20 stays for each bridge). The cable stays are anchored at the deck level to the web of the steel box girder and pass through cable saddles in the pylons. With 12 strands per cable, the stays are composed of 0.62-in.-diameter, seven-wire, low-relaxation strands. For improved corrosion resistance, each strand is coated with wax and encapsulated inside high-density polyethylene (HDPE) sheathing. The strand-bundled stays are further protected by an outside HDPE pipe.

Since an extradosed bridge has two load-carrying systems, cable support can be provided for only a portion of the span. Consistent with the geometry of many extradosed bridges, the first stay for the IH-35 frontage road bridges is offset from the pylon by approximately 20% of the main span. From this first stay, the cable support points are spaced 14 ft, 9 in. along the edge girder, for a total of 59 ft. This results in approximately 50% of the main span being cable supported, which is consistent with most existing extradosed bridges that have cables distributed across approximately 60% of the span.

Due to the use of the relatively more flexible steel superstructure (supported at the pylons by bearings), the resulting live-load stress range in the stays was greater than the Post-Tensioning Institute's (PTI) limit that would allow the stays to be designated as low-fatigue. The stress variation caused by live loads (AASHTO HL-93 live load with no pedestrians) varied up to approximately 15 ksi versus the 6.75 ksi limit for the stays to be considered extradosed. So the stays were designed accordingly, using the same provisions in the PTI manual as for conventional cable-stayed bridges.

A complete erection scheme was also developed during the design of the bridge to inhibit both cracking of the concrete deck during construction and slippage of the stay strand through the saddles. Further, the cable-stay and saddle system chosen was specified to allow for the installation and replacement of stay strands on an individual strand-by-strand basis. This was no small consideration since the ability to replace strands on an individual basis will allow future bridge maintenance workers to pull and inspect strands without needing to replace the entire stay. Stay strands will be placed within individual holes in the cable saddle, significantly reducing the risk of fretting corrosion and facilitating strand-by-strand replacement. Although using saddles is a common practice elsewhere in the world, it is relatively new in the U.S. and only a few bridges have been designed with this system.

Not only was this the first use of a steel extradosed bridge in the U.S., but the project also had to content with heavier-than-usual deadline pressure thanks to Baylor announcing that it would be constructing its new Lane Stadium football facility adjacent to the bridges, with an opening date of August 31, 2014. Nevertheless, the bridges were delivered 4.5 months ahead of the original schedule and opened to traffic that July.

Owner

Texas Department of Transportation, Austin

Designer

AECOM, Glen Allen, Va.

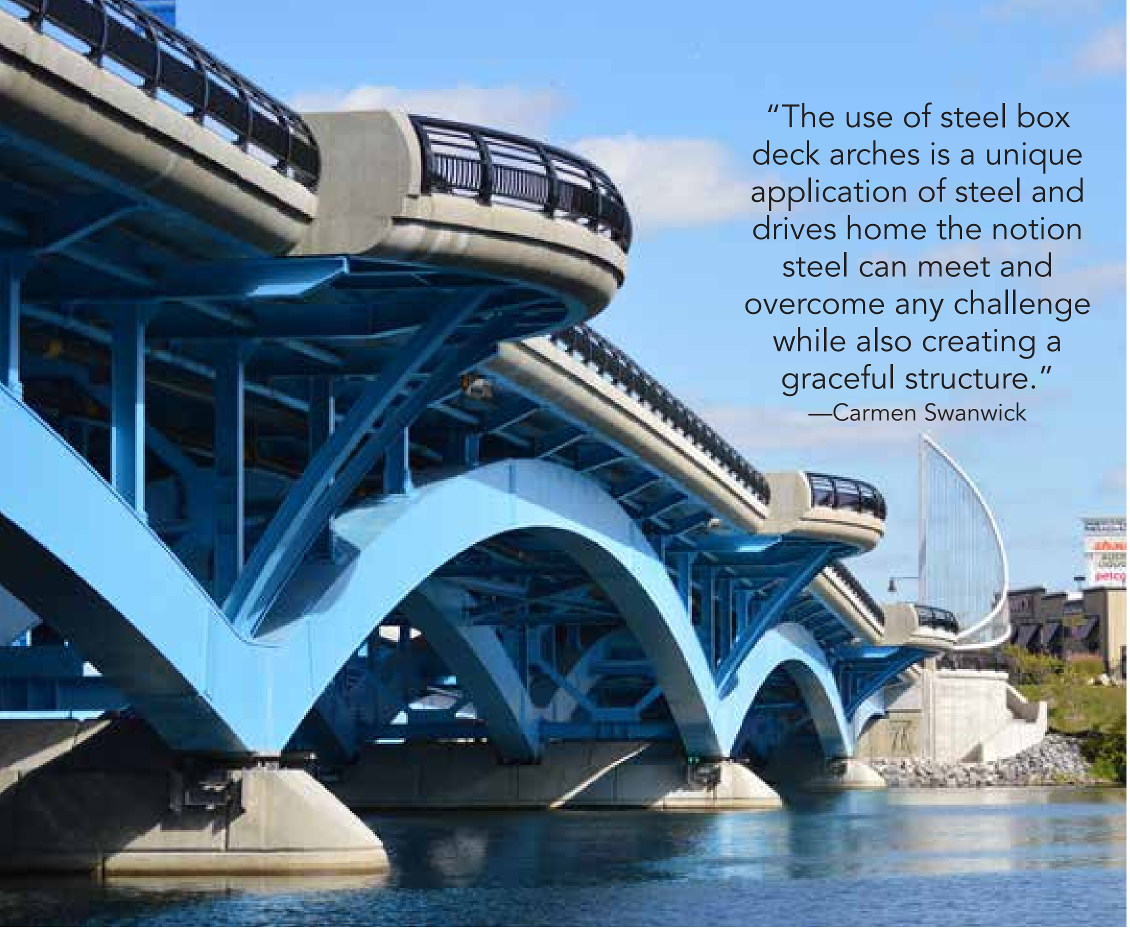
Contractor

The Lane Construction Corporation, Lorena, Texas

Steel Fabricator and Detailer

Hirschfeld Industries, San Angelo, Texas

AISC CERTIFIED FABRICATOR



"The use of steel box deck arches is a unique application of steel and drives home the notion steel can meet and overcome any challenge while also creating a graceful structure."
—Carmen Swanwick

PRIZE WINNER: MEDIUM SPAN

Kenneth F. Burns Memorial Bridge, Worcester/Shrewsbury, Mass.

AFTER NEARLY A CENTURY OF USE, the Kenneth F. Burns Memorial Bridge had run its course.

The multi-span concrete deck arch was an appreciated part of the landscape, but it had become too narrow for modern traffic needs and was deteriorating and due for retirement. Replacing it are two separate bridges, carrying eastbound and westbound traffic, that reflect on the old bridge's grace, but with a modern update using sweeping, sleek, steel box deck arches in place of concrete framing.

Construction staging required maintaining traffic flow on the original bridge, which carries Route 9 over Lake Quinsigamond between Shrewsbury and Worcester, while the new bridge was built around it. The design team developed a unique solution for the new low-rise arch spans: full-bridge-length post-tensioned arch ties. The bridge was designed and constructed as a tied deck arch. Tension ties were placed at the deck level and included full-length bridge post-tensioning, with two ducts per steel box beam. Post-tensioning was performed twice during construction to balance moments and compression forces.

To reduce impacts at the approaches in Worcester and Shrewsbury, vertical grade changes on Route 9 were minimized, which led to a relatively low rise. The resulting arch structures behaved as hybrid arch/continuous beams structures. The team optimized the design by balancing moments, axial compression and tension, using the post-tensioning to reduce maximum moments and carefully coordinating and iterating the analysis with construction staging.

The piers are comprised of steel pipe piles, with a precast soffit and cast-in-place concrete formed above the soffits. The construction of perched piers largely out of the water avoided the need for difficult and expensive sheeting and dredging in Lake Quinsigamond, and improved requirements for environmental permitting in the lake, resulting in better water quality and less disruption for boaters.

The design team developed a complex construction staging model using CSI Bridge, augmented by customized preprocessors and post-processor programs and sheets developed specifically for the project. The staging model matched construction means and methods and was frequently called upon to evaluate conditions in real time during construction. The model was verified during construction by matching predicted deflections with actual measurements at various stages of the work. In addition to the global model, detailed finite element models of complex steel connections were prepared to evaluate special conditions and framing.

Animation was used extensively to evaluate bridge aesthetics. For example, the team was concerned that the post-tensioning ducts on the fascia box beams might look like hanging utility pipes. It was initially thought that shadows from the overhanging decks might minimize the problem, but an animation with sunlight angles estimated from the end of December (with the most direct southern light) clearly showed otherwise. Based on this result, the team moved the ducts up onto the fascias, requiring special steel framing details but greatly improving the appearance of the bridge.

Owner

Massachusetts Department of Transportation, Worcester

Designer

Stantec, Boston (formerly FST)

Contractor

The Middlesex Corp, Littleton, Mass.

Steel Fabricator

Casco Bay Steel Structures, Inc., South Portland, Maine

AISC CERTIFIED FABRICATOR

PRIZE BRIDGE: SHORT SPAN
ACCELERATED BRIDGE CONSTRUCTION COMMENDATION
Wampum Bridge, Lawrence County, Pa.

IN AN ALL-TOO FAMILIAR STORY, a bridge in Wampum Borough of Lawrence County, Pa., had fallen on hard times and wasn't going to get better.

The severely deteriorated existing concrete arch carried SR 288/Main Street over Wampum Run and provided a vital connection for both residents and the local trucking industry. The failing structure had previously been reduced from two lanes to one bidirectional lane, and its weakening condition would have eventually warranted a full closure in the near future, thus requiring a 22-mile detour that was viewed as both costly and extremely inconvenient for local travelers. Either way, the bridge would need to be repaired or replaced.

Conventional phased construction methods for maintenance of traffic were considered but would have required extensive and costly repairs to the arch, thus prompting both the Pennsylvania Department of Transportation (PennDOT) and designer Johnson, Mirmiran and Thompson (JMT) to take the replacement route. Project stakeholders wanted a reduced construction time frame and minimal inconvenience for travelers following the lengthy detour, and JMT and PennDOT agreed that this could be accomplished by using accelerated bridge construction (ABC) techniques.

Preliminary design began with research and discussions with engineering professionals from various states with bridges successfully built using ABC. JMT reviewed these other states' standards and special provisions, and discussed design and construction methods used on their successful ABC projects. As a result of this research, JMT presented a report concluding that a cost-effective structure could be completed in less than a month.

Various superstructure options were considered including multi-girder bridges with full-depth precast concrete decks, partial-depth precast concrete deck panels, adjacent butted beam superstructures, modular prefabricated superstructures and parallel beam superstructures with a conventional deck. PennDOT District 11-0's preference was to avoid post-tensioning and construct a joint-less structure using integral abutments. All stakeholders agreed that the best option was a 78-ft steel beam structure on integral abutments. The pile caps, wing walls, cheek walls, back walls, approach and sleeper slabs were designed to be precast units while the steel beams were to have the deck and barrier cast to them off-site using conventional methods to create three modular units. The initial construction schedule for this structure type was estimated to take 15 days to construct.

The geotechnical findings showed that the piles could be driven, but they would have to be re-struck after 48 hours. Due to the uncertainty of the foundation of the portions of existing arch structure that were left in place, predrilling was required to avoid striking the remnants of the arch during the pile driving operation. Adding predrilling and the waiting period of the re-strike affected the initial schedule, and several production activities were rescheduled to occur during the re-strike waiting period to maintain efficiency. The changes to the schedule increased the allowable timeframe to 17 days. Confident that the bridge could be open to traffic within this time frame, Road User Liquidated Damages (RULDs) were calculated and an incentive/disincentive of $36,000 per day was added to the construction contract.

Due to the accelerated design schedule, coordination with the railroads and limiting impacts to the adjacent railroad property were critical for the project's success. Both CSX and Norfolk Southern have property within the project limits, and the roadway tie-ins were designed to ensure the required right-of-way was minimal on the CSX property and was not necessary on the Norfolk Southern property. Additionally, through coordination with CSX, the necessity for flaggers was eliminated by providing construction fence to prevent the contractor from accessing railroad property.

Another challenge was coordinating the relocation of Columbia Gas Transmission's line in a narrow time window. The existing gas transmission line crossed the roadway less than 15 ft behind the existing abutment, and because the gas line was so close to the structure and the project used integral abutments, it was impossible to avoid impacting the line. It had to be relocated prior to construction and the design had to be expedited in comparison to a typical project due to the condensed design schedule. Through extensive coordination between JMT and Columbia, a relocation route was developed, avoiding the proposed abutments, drainage structures and guiderail posts as well as roadway excavation. The roadway was closed for seven days, the new bridge was constructed in 7 days and the overall project was open to traffic on August 24, 2014, well ahead of the September 21, 2014 milestone date.

Owner

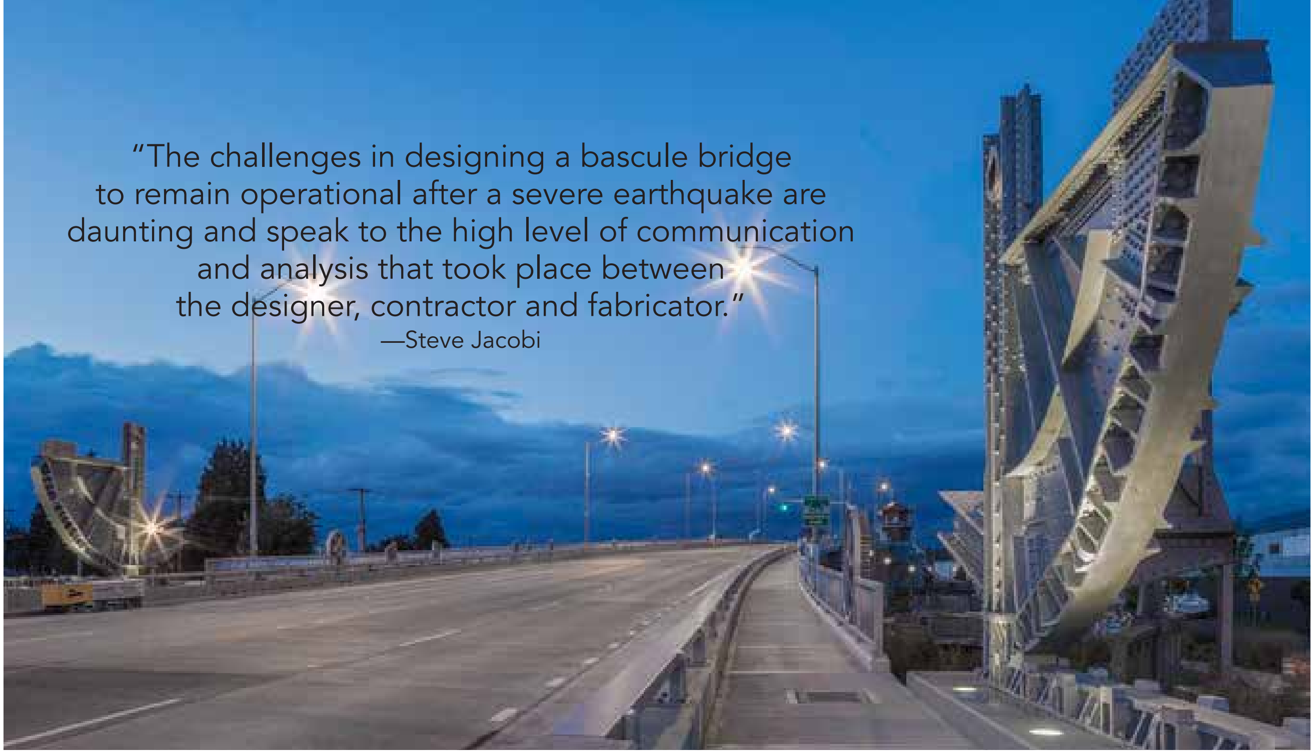
Pennsylvania Department of Transportation, Bridgeville

Designer

Johnson, Mirmiran and Thompson, Inc.,
Moon Township, Pa.

Contractor

Joseph B. Fay Company, Tarentum, Pa.



PRIZE BRIDGE: MOVABLE SPAN

South Park Bascule Bridge Replacement, Seattle

THE SOUTH PARK BRIDGE is a first-of-its-kind "trussed" plate girder design.

Designed and built to survive a major seismic event with minimal damage, the replacement bridge is a community lifeline, improving freight mobility and providing better regional access to downtown Seattle and the adjacent industrial area.

The original bridge was one of the few working examples of a rolling lift bascule Scherzer bridge. There was significant public agency and community interest in preserving its character and significance, as it was listed on the National Register of Historic Places and officially designated a historic landmark by the King County Landmarks Commission. The new bridge was designed to emulate the overall look and feel of the original bridge by incorporating truss-like features in the girders without incorporating the disadvantages of a traditional truss design. The fascia girder treatments off the main span were selected to honor the approach trusses on the original structure and to improve aesthetics. Economy in the design of the girder yielded a shallower structure, providing the desired waterway clearance improvements while minimizing the overall height of the bridge so it did not appear to tower over the surrounding community.

While a beloved community landmark, the original bridge's gusset-plated joints were numerous and sizable. Multiple large plates and fasteners intersecting at various angles created geometrically complex regions at every panel point. These joints acted like pockets, accumulating dirt, debris, moisture, guano and other substances detrimental to the steel bridge's long-term reliability. Designer HNTB's innovative main girder design of a continuous welded plate eliminated the problem-prone areas of traditional gusset-plated joints and two time-consuming steps common in its construction: match-drilling and field installation of thousands of bolts. With those steps gone, the "trussed" plate girder design—the first known use of this girder type—sped fabrication, shop assembly alignment and erection.

The bascule leaves are connected at the tips by span lock bars that will keep the leaves together vertically and transversely during a seismic event. However, there are no longitudinal restraints between the two leaves. During a seismic event, the joint will experience separation and closure of up to 18 in. of total movement. If the leaf superstructure was allowed to collide longitudinally, the impact load would have been very large, and the loads would have been transferred back to the span-supporting trunnion frames, requiring a more robust frame.

HNTB's solution was to design the draw span superstructure with 19 in. of separation and include a collapsible center joint. During a seismic event, only the leaf tip joint assemblies would collide, thus preventing large load transfers back to the trunnion frames. The collapsible joint was detailed so that steel components on tapered shims would shear off when displaced, resulting in damage that would be easily detectable and repairable.

Several solutions were incorporated in order to meet stringent seismic performance requirements, including sunken caisson foundations, isolated trunnion frames and a collapsible center joint on the lift spans. The bridge was designed to remain fully functional in the aftermath of an Operational Earthquake Level (108-year return period), and only moderate, but repairable, damage was permitted as the result of a Design Earthquake Level (975-year return period).

A citizens advisory group of diverse stakeholders met often and conveyed an important public perspective, which was incorporated into the bridge's design during the eight-year environmental documentation phase. One of the more notable action items was the group's request to include many of the original bridge's architectural details in the new bridge's design, as well as to salvage and display more than 100 original bridge parts at the project site. Gears from the operating machinery were artistically incorporated in the sidewalk railing. The track-and-rocker assemblies, the historical features from the original Scherzer bridge were transformed into gateway monuments at each end of the bridge. The decorative light posts, decorative railing panels, cast concrete railing, old bricks, decorative rail posts and deck grating were used to embellish the site around the bridge.

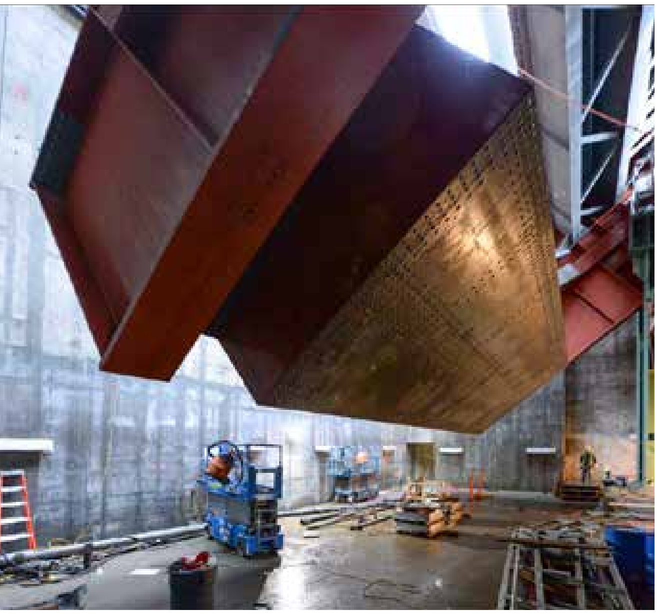
In addition, the design features a decorative rain garden that serves as landscape art while also collecting and naturally treating storm water runoff from the bridge prior to discharging it into the waterway, thus eliminating the need for a huge and expensive underground detention vault. The bridge was also engineered with an energy-efficient drive system that can operate each 1,500-ton draw span with approximately the same horsepower needed to drive a Toyota Prius.

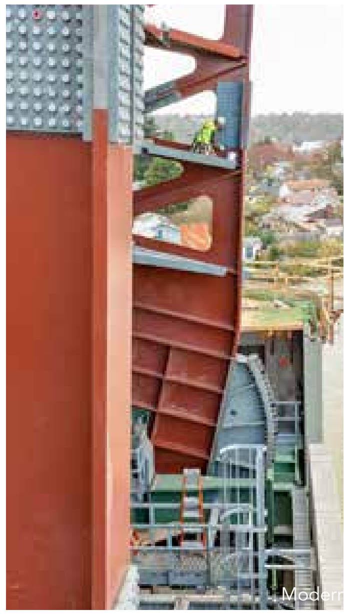
Owner
King County Department of Transportation, Seattle

Designer
HNTB Corporation, Bellevue, Wash.

Contractor
Kiewit-Massman (JV), Federal Way, Wash.

Steel Fabricator and Detailer
Stinger Bridge and Iron, Coolidge, Ariz.





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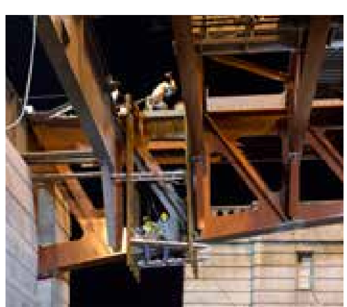


PRIZE BRIDGE: RECONSTRUCTION SPAN
Alexander Hamilton Bridge, New York

ALEXANDER HAMILTON IS NOT ONLY the star of Broadway's current smash hit, but also a star of the New York metro area's transportation infrastructure.

The $413 million Alexander Hamilton Bridge (AHB) Rehabilitation Project rejuvenates a major link in the area, leading to enhanced mobility throughout the region, improved safety and a structure that was designed to endure for generations. The project also restored existing recreational facilities and constructed new ones in the park land within the project to provide safe gathering areas for local communities.

The original AHB consisted of two separate superstructures with a longitudinal open joint along the centerline of bridge. The asymmetrical shifting of traffic lanes and cutting of the existing bridge cantilever brackets required extensive implementation of temporary and permanent bracing between the two superstructures to resist the unbalanced loadings during construction and in the final condition.

The weight of the new widened AHB and the modifications of existing bridge superstructure for the elimination of deck joints (transverse and longitudinal) required the strengthening of the existing four 505-ft-long deep steel-box arch-ribs that span between the Harlem river. Detailed step-by-step procedures were developed and provided in contract document for the pretensioning and installation of the new reinforcing top and bottom cover plates.

The superstructure of the new bridge is composed of a girder-floor beam-stringer system. For the strengthening of existing floor beams and the introduction of new retrofit girders between the existing girders under live loads, detailed analyses were performed and complex details were developed for the safe cutting and temporary support of the existing floor beams. For the new widening, the existing cantilever brackets along the fascia of AHB were replaced with longer (57-ft) cantilever brackets, almost twice the original length. To strengthen and replace corroded sections, temporary support and bracing details were developed and provided in contract documents for the partial disassembled of long and slender composite box-sections under heavy loads.

For more on a different award-winning portion of the Alexander Hamilton Bridge project, see the "Ramp TE Over I-95" write-up in the 2014 Prize Bridge Awards feature in the June 2014 issue, available at ***www.modernsteel.com****.*

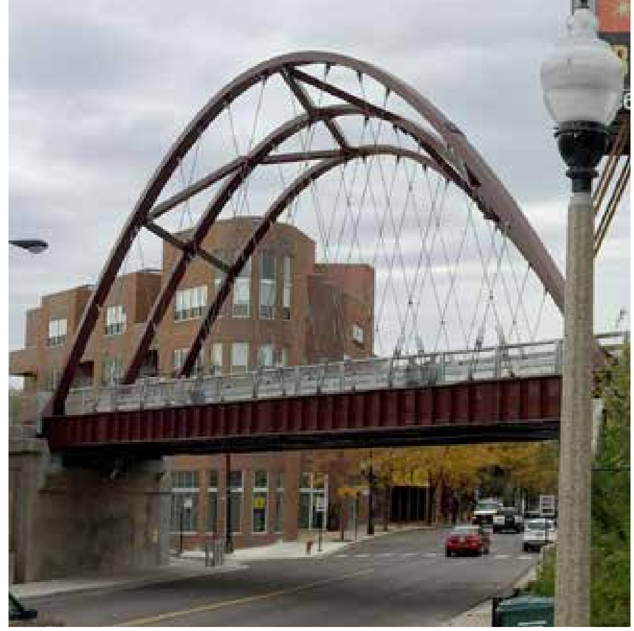
Owner
New York State Department of Transportation, Long Island City

Designer
Jacobs Engineering, New York

Contractor
Halmar International, Nanuet, N.Y.

Steel Fabricator and Detailer
Canam-Bridges, Claremont, N.H.

"This project is as complicated as it gets. Where it was possible, the superstructure was salvaged, resulting in a revitalized link that will carry over 200,000 cars per day for the next 100 years."
—Michael Culmo



PRIZE BRIDGE: SPECIAL PURPOSE
The 606 - Milwaukee Avenue Bridge, Chicago

CHICAGO'S LATEST HIGH-PROFILE PARK rises above it all.

The 606, named for the first three digits of the city's zip codes (and also known as the Bloomingdale Trail), is a 2.7-mile-long former elevated train line that was converted into a new park and pedestrian trail on the city's north side.

Often seen as the centerpiece of the project, one of the park's bridges—over Milwaukee Avenue—was transformed from a four-span, low-clearance structure with three piers that obstructed traffic below, to a single-span tied-arch structure that allowed street traffic to flow unobstructed and with improved sight lines and vertical clearances.

In order to reuse as much of the structure as possible, the team proposed transforming the existing superstructure into a tied-arch bridge while reusing the existing plate girders as the tie girder and installing new curved rectangular HSS arches that provide support to the new single 98-ft, 2-in. span. LUSAS and CSiBridge modeling software were used to analyze the structure, including modal and dynamic analysis. The existing girders were spliced together at the piers for continuity and retrofit steel was added to the existing plate girders where needed due to the deterioration that had taken place over the past 100 years. Lateral earth loads were reduced by the use of geofoam, allowing the existing abutments to be completely reused.

The skew of the bridge provides for unique perspectives of the structure from different vantage points. Motorists below see three staggered arches that appear tall and steep, while train riders above, from a view perpendicular to the arches, see them as long and shallow. Pedestrians passing through the arches see the unique angles and elevation changes of the bracing and arch members provided by the 45° skew. With limited space due to nearby Chicago Transit Authority (CTA) Blue Line elevated train structural support columns, creating access at the west side of Milwaukee Avenue forced the designers to think outside the box. The solution was to have the access ramp cut through the existing retaining wall, starting outside the elevated portion of the trail at Milwaukee Avenue and moving inward and upward until access to the trail was gained. Tied-back steel sheet pile wall was incorporated to accomplish this feat.

Owner
Chicago Department of Transportation, Chicago

Designer
Collins Engineers, Chicago

Contractor
Walsh Construction Company, Chicago

Steel Fabricator
Prospect Steel Company, Little Rock, Ark.

Steel Detailer
Weaver Bridge Corporation, Granville, Ohio

"Retrofitting the existing plate girders as the tie girder for a new arch structure was innovate and could provide a method for retrofitting many of our shorter-span structures where the substructure has deteriorated but the superstructure has retained its load-carrying capacity."
—Carmen Swanwick

MERIT AWARD: MAJOR SPAN

Stan Musial Veterans Memorial Bridge, St. Louis/St. Clair County, Ill.

TOO MANY INTERSTATES ON ONE BRIDGE were causing quite the traffic nightmare over the Mississippi River near downtown St. Louis.

The Poplar Street Bridge, which carries Interstates 55, 64 and 70 as well as U.S. 40, was overburdened with traffic, so the decision was made to build a new crossing for I-70, the Stan Musial Veterans Memorial Bridge. The bridge features two 400-ft towers above the third-longest cable-stayed bridge in the United States.

Currently carrying four lanes, the design allows for the addition of two lanes through re-striping and can accommodate a future adjacent four-lane bridge. Traffic is now able to flow at posted speeds adjacent to downtown St. Louis between Missouri and Illinois, which reduces congestion, enhances air quality and aids in interstate commerce. Designer HNTB co-located with owner and FHWA representatives to solve problems in real time, thus delivering a buildable, economical design in one year—half the typical time for similar bridges.

To optimize materials, the team chose steel floor beams and edge girders composite with precast concrete panels for the superstructure, which made it easier to erect. The design eliminated the tedious job of constructing concrete corbels for the upper cable anchors and incorporated steel anchor boxes to reduce the amount of post-tensioning around pylons. The decision to fabricate the boxes in the shop made them safer and more precise while eliminating significant amounts of work 300 ft or more above the river. The steel anchor boxes incorporated a bolted connection between the anchor beam and anchor box, which allowed the fabricator to precisely locate the anchorage before bolting it permanently into position. The lower cable anchorages were steel weldments bolted to the side of the edge girders. By locating the these anchorages alongside the edge girders as opposed to on top of them, the length needed between the strand anchor and top end of the guide pipe could be obtained such that smaller more compact friction cable dampers could be used. In addition, HPS70W steel was used in the edge girders to reduce the overall weight of the superstructure.

The superstructure was designed to be redundant and able to withstand the loss of any cable without significant damage to the bridge. The cable spacing was optimized to assist with the load transfer in the superstructure under the cable-loss scenario. Various details were incorporated into the design of the bridge to address security measures important in today's world.

Because of the bridge's location in a high-seismic zone, the magnitude of the span and the soft soils and potential for liquefaction during a seismic event, HNTB tapped researchers at the University of Illinois and University of California-Berkeley. They analyzed the design using a conditional mean spectrum (CMS) approach, which had never before been used for bridge design. The approach considers the most expected response spectrum of a structure under different ground motions rather than aggregating multiple ground movements from various potential seismic events. The analysis revealed realistic demands on the bridge and eliminated potential for lateral spreading and the need for any associated ground improvements. To further test its effectiveness, HNTB performed dual-level seismic checks to ensure the bridge would be in service after a 1,000-year event and suffer only minimal damage at the 2,500-year maximum credible event. The process confirmed that the CMS approach reduces costs, and its successful implementation points to future value for the engineering profession.

For more on this project, see "Thinking Inside the Box" in the November 2013 issue, available at ***www.modernsteel.com****.*

Owner

Missouri Department of Transportation, Jefferson City

Designer

HNTB Corporation, Kansas City

Contractor

MTA (JV), Kansas City

Steel Fabricator

W&W | AFCO, Little Rock, Ark. AISC CERTIFIED FABRICATOR





MERIT AWARD: LONG SPAN

(I-270 over) Chain of Rocks Road Canal Bridge, Granite City, Ill./Bellefontaine Neighbors, Mo.

STRUCTURALLY DEFICIENT AND FUNCTIONALLY OBSOLETE (technical terminology for "way past its prime") is the best way to describe the twin truss bridges that carried I-270 over the Chain of Rocks Canal near Granite City, Ill.

Built in 1963, the bridges had served as a major Interstate and St. Louis Area commuter link between Illinois and Missouri, crossing the canal that acts as a Mississippi River Bypass for all barge traffic traveling through St. Louis. Heavy existing traffic—nearly 55,000 vehicles per day—coupled with the regularly required bridge repairs caused significant congestion and delay for users and was a major source of concern and countless complaints, and the decision was made to replace the bridges.

Designer HDR's analysis showed the I-270 trusses were deficient; the structures needed serious help. Determining a remedy for the larger issue of how to design and construct a replacement bridge while keeping I-270 open to traffic quickly moved the project up on the priority list. In addition to managing preliminary engineering and final design services for the bridge replacement, HDR also inspected the bridges annually to ensure that the structural integrity of the existing bridges was sufficient during the design and construction phases. Due to the recent I-35W bridge failure, the inspections and follow-up specifically included gusset plate inspections and ratings to determine and monitor the strength of the connections in the trusses. After assessing the existing condition of the nearly 50-year old bridges, HDR identified rehabilitation requirements to keep the structures serviceable in the near term. Since construction funding was not secured at the time, HDR developed a plan to construct the bridge in phases as funding became available.

The project's Traffic Management Plan (TMP) staged construction to maintain two lanes of traffic in each direction and provided motorists with advanced warning/information of the lane closures and alternative route options, thus minimizing traffic backup lengths and using the most efficient method of construction staging to maximize safety and quality.

Due to the navigable canal and adjacent levee, the question was whether the USACE and the United States Coast Guard would issue permits in and around a levee in the "post-Katrina" environment. HDR's mutually acceptable solution involved placing suitable compacted fill to widen the levee and stabilize the area enough so that the pier location could be allowed. The bridge design could then proceed at speed.

Opened to traffic in 2014, the new I-270 bridge represents the largest steel plate I-girder bridge in Illinois. The five-span crossing, with a total length, of 1,970 ft, includes spans of 350 ft, 440 ft, 490 ft, 440 ft and 250 ft. The span arrangement was dictated by the need to span the canal and adjacent east flood protection levee and in doing so, the bridge was configured with 10 variable-depth steel plate I-girders. Given the amount of steel required, the design strived to achieve economy with regard to material, fabrication and construction. Flange plate thicknesses are repeated throughout the structure as much as possible in an effort to reduce the number of plate sizes required to be procured by the fabricator, Stupp Bridge. For the 18 girder field pieces along each girder line, only six different Grade 50W flange plate thicknesses are used, and only four different HPS70W flange plate thicknesses are used.

For more on this project, see "Increasing Spans and Possibilities" in the March 2014 issue, available at ***www.modernsteel.com****.*

Owner
Illinois Department of Transportation, Collinsville, Ill.

Designer
HDR, Inc., Chicago

Contractor
Walsh Construction, Chicago

Steel Fabricator and Detailer
Stupp Bridge Company, St. Louis AISC CERTIFIED FABRICATOR

MERIT AWARD: MEDIUM SPAN
Falls Flyover Ramp, Wichita, Kan.

FOR MORE THAN TWO DECADES, the City of Wichita, Kan., sought to relieve 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 solution is manifested in the form of two structural steel plate girder bridges—2,273 ft long and 1,690 ft long, respectively—which are part of a new partial interchange with 13th Street and I-235.

Establishing the flyover bridges' span arrangement to fit the project site was challenging due to multiple constraints. The design team had to carefully locate the bridges over 1,000 ft of floodway and around its levees, as well as around I-235, other roadways, a lakeside residential development and a 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. The U.S. Army Corps of Engineers (USACE) required a geotechnical seepage analysis be completed for bridge piers adjacent to the dry side of the levees to demonstrate they would have no substantive impact upon seepage potential through or beneath the levees.

Structural 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, and weathering steel was selected to minimize future maintenance requirements. In all, the bridges use 2,875 tons of structural steel.

Both bridges are 32 ft, 6 in. wide, with four plate girders spaced at 8 ft, 8 in. apart, and the girder webs are 84 in. deep. 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. Vertical bridge profiles were set to provide for an access road on top of the levee at three crossings and an access road adjacent to the dry side of the levee at the fourth crossing.

For more on this project, see "Flying over the Floodway" in the March 2015 issue, available at www.modernsteel.com.

Owner
Kansas Department of Transportation, Wichita, Kan.

Designer
HNTB Corporation, Overland Park, Kan.

Contractor
Dondlinger and Sons Construction Company, Wichita

MERIT AWARD: SHORT SPAN
SUSTAINABILITY COMMENDATION
Mill Creek Bridge, Astoria, Ore.

EVER-INCREASING REHABILITATION needs for corroded steel bridges are one of the Oregon Department of Transportation's (ODOT) biggest ongoing concerns.

While high-performance steel (HPS) is an important step in increasing toughness and corrosion resistance when compared to weathering steel, it is still vulnerable in corrosive and high-humidity environments inherent to the state's coastal areas. The conventional way to accommodate bridge steel corrosion is to apply protective paint coatings and to periodically recoat the bridge during its service life. However, the lifecycle cost of this design choice can be much higher than the initial cost of the bridge. An alternative to weathering steel or HPS and painted steel girders is corrosion-resistant ASTM A1010 Grade 50 steel that needs no corrosion protection coating.

A sample steel plate girder bridge employing A1010 is the 123-ft-long, 42-ft, 8-in.-wide Mill Creek Bridge along Lower Columbia River Hwy. No 2W (U.S. 30), only the second A1010 plate girder bridge for public use in the world. The pre-purchasing contract adopted for the project divided it into two segments: contracting steel fabrication as soon as the steel design and specification was completed followed by the remainder of construction. This type contract gives the fabricator extra time for ordering steel plate, testing plate samples for compliances to the contract requirements and replacing plate that does not meet them, and helps prevent time loss from unforeseen issues that could cause delays.

For the other bridge project using A1010 Grade 50 steel, see the Dodge Creek Bridge item in the 2014 Prize Bridge Awards section (it won the same award and commendation as the Mill Creek Bridge) at ***www.modernsteel.com****.*

Owner and Designer
Oregon Department of Transportation, Salem

Contractor
Oregon State Bridge Construction, Inc., Aumsville, Ore.

Steel Fabricator
Thompson Metal Fab, Inc., Vancouver, Wash.

Steel Detailer
Candraft Detailing, Inc., New Westminster, B.C., Canada

MERIT AWARD: MOVABLE SPAN
Henry G. Gilmerton Bridge,
Chesapeake, Va.

THE HENRY G. GILMERTON BRIDGE, one of five critical bridges connecting the Hampton Roads region in southeastern Virginia, is in one of the world's largest natural harbors, so it's not surprising that the bridge carries approximately one million travelers every month.

But nearly 70 years after the original bascule bridge was constructed, the Virginia Department of Transportation (VDOT) determined that it would need to replace the aging span and thus embarked on a $134 million project. The replacement, which was substantially completed in 2013, was built with the goals of reducing automobile congestion at the bridge and alternate routes, increasing clearance to accommodate marine and motorist traffic with fewer openings and increasing lane width to improve traffic flow and accommodate future widening of Military Highway—all without impacting vehicular or marine traffic, changing the existing alignment of military highway or modifying the navigational channel geometry.

In addition, the close proximity of the Norfolk Southern Railroad line to the bridge posed a significant challenge. Installation of the bridge's eight new 12-ft-diameter drilled-shaft foundations, erection of the superstructure and demolition of the original bridge all needed to be done in such a way that did not disrupt the railroad bridge or its foundations. Complicating the need for increased width is the nearby railroad bridge's right-of-way. A hard bend in the river south of the bridge eliminated the possibility of expanding in that direction, so Norfolk Southern's willingness to yield some of its right-of-way was the only way the wider bridge could be constructed; the 89-ft-wide bridge is one of the widest lift spans ever to be built.

During installation of the new drilled shafts, the team used vibration-monitoring equipment to identify potential settlement impacts to the railroad bridge foundations. Installing the foundations also presented a challenge for the construction and design teams. The Gilmerton Bridge is located in the Great Dismal Swamp, a marshy area on the coastal plains region in southeast Virginia with less than desirable soil conditions. The drilled-shaft foundations were designed to reach 120 ft below ground level, which required special equipment and a team of industry experts. The team employed a massive oscillator to drill the foundations, and the project incorporates some of the largest drilled shafts ever constructed using the oscillating method.

Because rock was too deep to rest the drilled shaft foundations on, the foundations were predicted to experience some settlement with time. Settlement can be an issue for any bridge, but is of particular concern for movable bridges because of the precise tolerances required to ensure operation without binding. Jacking brackets were designed into the towers to allow them to be jacked under full, dead load to compensate for the settlement.

The lift bridge tower legs were positioned outside and behind the existing bascule bridge piers. This allowed the new towers to be built over the existing bridge without impacting the bascule span's ability to open for marine traffic. This required the lower portion of the tower to be designed as an unbraced portal frame. The new steel towers provide the required 135 ft of vertical clearance for the 250-ft lift span. Due to the exceptional bridge width, four 15-ft-diameter sheaves, each carrying twelve 2¼-in.-diameter wire ropes, were required on each tower to support the load of the lift span and counterweights—twice as many as typically necessary.

Using accelerated construction for the lift span required that the span be floated in on barges following construction of the towers. A specially retrofitted barge was needed to carry the nearly 2,500-lb lift span. Removing the old span and floating in the new span not only required continuous collaboration of design and construction teams, but it also required very specific timing around weather patterns and the tide.

Perhaps one of the bridge's most striking features is its turquoise paint. This color was chosen by the City of Chesapeake as part of an ongoing bridge unification initiative, which calls for matching paint coatings for all of their iconic structures. Another, more functional, coating treatment can be found in the machinery room. For safety purposes, movable bridge components are coordinated using bright colors to distinguish between movable, stationary and other bridge parts. Additionally, the silhouette and form of the new Gilmerton Bridge complement the nearby railroad bridge, even when both movable structures are in the open position.

Ultimately, the bridge was built to hold six travel lanes, addressing the original goal of accommodating future growth of Military Highway. Initially, however, both outside lanes will be striped, allowing them to operate as shoulders before the necessary expansion. The new bridge's 35-ft clearance allows smaller craft to traverse under the bridge without impacting vehicular traffic and reduces openings by 40%, as well as wear on bridge mechanical components. The reduction in congestion allows a growing community easier travel, while ensuring the uninterrupted flow of commercial goods by vehicle, rail and boat.

For more on this project, see "Widening the Gap" in the January 2013 issue, available at ***www.modernsteel.com****.*

Owner

Virginia Department of Transportation, Richmond

Designer

Modjeski and Masters, Inc., Mechanicsburg, Pa.

Contractor

PCL Civil Constructions, Inc., Tampa, Fla.

Steel Fabricator and Detailer

Banker Steel Company, AISC CERTIFIED FABRICATOR
Lynchburg, Va.

MERIT AWARD: MOVABLE SPAN
World War I Memorial Bridge, Portsmouth, N.H./Kittery, Maine

SINCE 1923, THE WORLD WAR I MEMORIAL BRIDGE linked Portsmouth, N.H., and Kittery, Maine, providing a multimodal transportation system that enhanced commerce, tourism, community life and the historic and aesthetic character of both communities.

But in recent years, structural deficiencies led to its closing, prompting the need for a replacement crossing. The new bridge is 900 ft long and comprises three spans: two approach spans of 298.75 ft each and a lift span of 302.5 ft, with a width of 49.5 ft to 54.6 ft.

In the designing the truss, contractor Archer Western and designer HNTB hoped to explore new fabrication capabilities to avoid one of the most challenging aspects of truss design: gusset-plate connections. The demise of the existing bridge was due to corrosion and deterioration of gusseted truss connections, which are difficult to inspect, collect debris, corrode and are impossible to remove and replace without underpinning the structure. In addition to avoiding gusset plates, the team elected to fabricate the top and bottom chords much in the same way plate girders are fabricated and to use rolled wide-flange sections for the diagonals to further simplify fabrication.

For the continuous flanges, the team designed the bottom flange of the bottom chord and the top flange of the top chord to be continuous. To add the necessary area, designers made the bottom flange of the bottom chord bigger than the top flange of the bottom chord and vice versa. The result is a monosymmetric I section, where the bottom flange is wider than the top flange. This has several advantages, including: 1) the truss acts as a deeper truss; 2) it forces some of the load to transfer around the web instead of going back into the web; and 3) the bottom chord/bottom flange and the top chord/top flange are wider and heavier. While there are more than 20,000 truss bridges in service across the U.S., the design team knows of no other bridge that has used this strategy to eliminate gusset plates. It is likely that this modified truss design, using plate-girder fabrication technology, is the first of its kind in North America.

*For more information on this project, see "A New Way to Connect" in the April 2014 issue, available at **www.modernsteel.com**.*

Owner
New Hampshire Department of Transportation, Concord
Maine Department of Transportation, Augusta

Designer
HNTB Corporation, Westbrook, Maine

Contractor
Archer Western Contractors, Canton, Maine

Steel Fabricator and Detailer
Canam-Bridges, Claremont, N.H. AISC CERTIFIED FABRICATOR

MERIT AWARD: RECONSTRUCTION

Donald R. Lobaugh Bridge, Freeport, Pa.

SPANNING THE ALLEGHENY RIVER approximately 25 miles northeast of Pittsburgh, the Freeport Bridge, also known as the Donald R. Lobaugh Bridge, carries State Route 0356 and a multiuse path. Built in 1965, the bridge is vital for commerce and serves as a route for tourists and outdoor enthusiasts using the extensive nearby rails-to-trails network, including the Kiskiminetas-Conemaugh water trails.

However, recent inspections by the Pennsylvania Department of Transportation (PennDOT) indicated that emergency attention was necessary to extend the bridge's useful service life and ensure the safety of the traveling public. Several existing conditions contributed to bridge deterioration, including steady chloride-laden runoff passing through a 1-in. open median joint, leaking stringer relief joints and free-fall roadway drainage from slots at the base of the barriers. This deterioration became a prime concern for PennDOT and prompted the need for emergency repairs.

In late 2006, designer Modjeski and Masters provided PennDOT with designs and details for significant emergency repairs to temporarily prevent the bridge from being weight restricted. Had these emergency repairs not been performed, all heavy live loads, including school buses and emergency vehicles, would have been prohibited from crossing the bridge resulting in a 20-mile detour. Steel plate reinforcement of deficient portions of the truss span's floor system was complete; however, to preserve a safe crossing more extensive repairs would be required in the future.

Beyond the need for immediate structural repairs, improvement of the roadway geometry and safety features was also required. Substandard features included inadequate curb-to-curb width, outdated bridge rails, insufficient sidewalk width and lack of pedestrian protection. Design for the modernized bridge needed to address the steel corrosion and section loss issues and bring the bridge's geometric features up to current standards.

The project focus became the rehabilitation and strengthening of the three-span deck truss spans and complete replacement of the north and south approach structures, an ambitious project with an overall length of 2,443 ft from abutment to abutment when completed (a reduction of 671 ft). The deck cross section accommodates four lanes of traffic and one variable-width barrier protected multi-use path.

The first challenge related to meeting PennDOT's requirement to maintain two-way traffic during all construction phases. Due to the existing bridge deck geometry, the deck needed to be temporarily widened for the first phase of construction. This temporary widening included the removal of the existing sidewalk and barrier. The widening required several modifications to maintain structural stability and ensure safety of the construction crews and traveling public.

Due to strength issues, construction staging and PennDOT's desire for a joint-less deck, the two-span stringer units and stress relief joints on the truss spans were replaced with continuous full-length stringers. The new joint-less reinforced concrete deck will extend the service life of the bridge and minimize future maintenance requirements for PennDOT.

Two of the six expansion rocker bearings on the truss-spans were observed to behave abnormally—they were in their expanded position on a cold day—and PennDOT opted to replace rather than to attempt to reset them. Because of the very large vertical loads and the need to move due to thermal expansion and contraction, pot-type high-load multi-rotational bearings were selected. Since the rocker bearings do not introduce eccentricity in the end post and the end post was not designed for such a loading, the replacement bearing configuration needed to replicate this condition. In a normal pot bearing application, the nonmoving component (the pot) is connected to the substructure. In this case, the nonmoving component needed to be connected to the end post and the sliding surface on the substructure, which could resist the additional loading due to the eccentricity. This meant that the bearings needed to be installed upside-down and protected with sheet metal covers. At six other expansion bearing locations on the truss spans, seismic retrofits were installed to improve the connection of the superstructure to the substructure in the event of an earthquake.

Owner

Pennsylvania Department of Transportation, Uniontown

Designer

Modjeski and Masters, Inc., Mechanicsburg, Pa.

Contractor

Brayman Construction Corporation, Saxonburg, Pa.

MERIT AWARD: RECONSTRUCTION
Wells Street Bridge, Chicago

THE WELLS STREET BRIDGE is the longest double-deck, double-leaf, bascule bridge built over the Chicago River, and only one of two remaining bascule bridges in the city of Chicago that carries both automobile and transit (Chicago Transit Authority elevated trains) on two levels.

Recent in-depth inspection and analysis of the 1922-built bridge revealed that substantial structural rehabilitation was required. As the bridge carries an average daily traffic of approximately 12,000 vehicles and serves nearly 4,500 pedestrians a day on the lower level and a two major transit lines carrying 70,000 riders per day on the upper level, the crucial crossing had to be rehabilitated with minimal impact to its users.

In the initial design plan, CTA would only allow weekend shutdowns, which only provided for partial replacement of select members. To accomplish this partial replacement the rail operations would have needed to be suspended for 15 long weekends throughout the year, a situation that was deemed unacceptable. Instead, it was determined that the replacement of the truss "river arm" structure and framing would take place during two nine-day shutdowns of upper-level train traffic (two weekends and one work week).

The main span of the bridge is 345 ft long and 72 ft wide. Dual open-web trusses, as main load carrying members, support both levels of framing. Both levels of framing were entirely replaced along with major rehabilitation to the mechanical and electrical components of the bridge. Bridge houses and bridge pits, including counterweight boxes, received select repairs. Due to the bridge's historic status, most elements such as the railings, bridge houses and major structural components were replaced in-kind to preserve the historic look.

The bridge was rehabilitated one leaf at a time, providing temporary shoring under the counterweight box for the leaf under construction so train traffic could be maintained. Vehicle and pedestrian traffic was safely detoured to other local streets and bridges over the river, and working on only one leaf at a time allowed one leaf to remain operable to accommodate river traffic.

The first shutdown took place in March 2013 and another in April 2013. During each line cut, transit service over the structure was halted on a Friday evening and resumed again by rush hour on the second Monday. As CTA was planning to perform loop track repairs around the same timeframe as the Wells bridge rehabilitation—and these repairs would have required additional weekend shutdowns—the two projects were combined and resulted in minimal impact to users as well as a $500,000 savings for the city.

Construction staging was perhaps the most complex part of the work and the key to the success of the project. In addition to the limited closures for CTA trains, the Coast Guard required that one leaf be operational at all times between March and October. Because the bridge was located over a river in the heart of the city, nearby streets were not available for the staging of the material, and the project relied heavily on marine equipment for staging. Before the project was bid, an early procurement contract was awarded for the river arm structural steel fabrication. The fabricator stored the trusses and assisted the contractor in assembly of the truss/floor beam river arm that was eventually barged to the site.

Achieving bridge balance was another challenge. In order to proceed with work on the north leaf, the south leaf first needed to be operational, which required the latter to be balanced in the interim condition. To balance the bridge for operation, concrete jersey barriers were lashed to the deck toward the nose of the span. The north leaf counterweight was then shored and the construction sequence was repeated for the north leaf.

The Wells Street Bridge project demonstrates that in-situ rehabilitation of moveable structures nearing their useful life can be a viable alternative to replacement. Full-scale replacement of moveable bridges can be a long process often requiring realignment, property acquisition and displacement of people and businesses. The rehabilitation of the Wells Street structure was performed with minimal disruption to local businesses in a congested urban site.

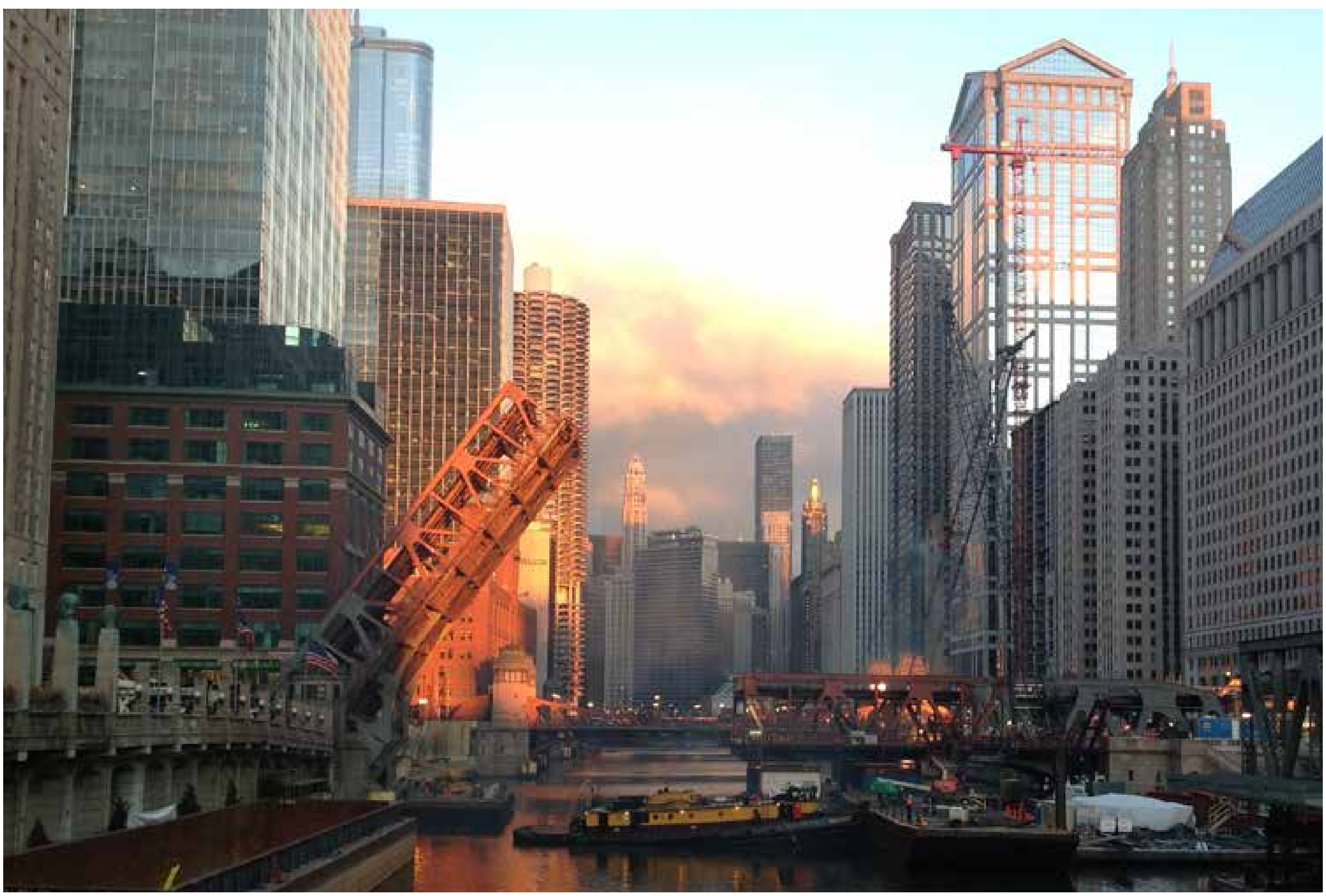
Owner
Chicago Department of Transportation, Chicago

Designer
AECOM, Chicago

Contractor
Walsh Construction and II in One (JV), Chicago

Steel Fabricator
Munster Steel Company, Inc., Hammond, Ind.

Steel Detailer
Candraft Detailing, Inc., New Westminster, B.C., Canada



ACCELERATED BRIDGE CONSTRUCTION COMMENDATION

Milton-Madison Bridge, Milton, Ky./Madison, Ind.

SINCE ITS COMPLETION IN 1929, when America was on the brink of the Great Depression, the original US-421/Milton-Madison Bridge served as a vital link over the Ohio River between Milton, Ky., and Madison, Ind.

A structure that was designed for the occasional Model-A Ford had seen its burden grow to more than 10,000 modern vehicles per day, including semitrailer trucks loaded at full capacity. Although it was historically significant, the aging bridge had become functionally obsolete. A TIGER discretionary grant from the U.S. government became the catalyst to one of the most innovative bridge replacement project endeavors in the nation.

Using the accelerated bridge construction (ABC) method, the project began with the construction of temporary approach ramps, allowing traffic to be rerouted off of the existing approach spans to begin their unobstructed demolition and replacement. While these phasing activities were occurring, sections of the 7,200-ton truss superstructure were being preassembled on barges for the eventual float-in and strand lifting onto temporary piers, which were constructed adjacent to each existing pier stem. The temporary piers were designed to support live traffic on the completed bridge in its temporary alignment, freeing the existing structure for explosive demolition and pier cap widening. The temporary pier caps featured a key design element—the "sliding girders"—which would serve as the pathway for the record-breaking truss slide. The nearly ½-mile long completed bridge, weighing more than 16,000 tons at the time of the slide, was moved 55 ft laterally into place atop the refurbished and widened pier stems of the existing bridge. ■

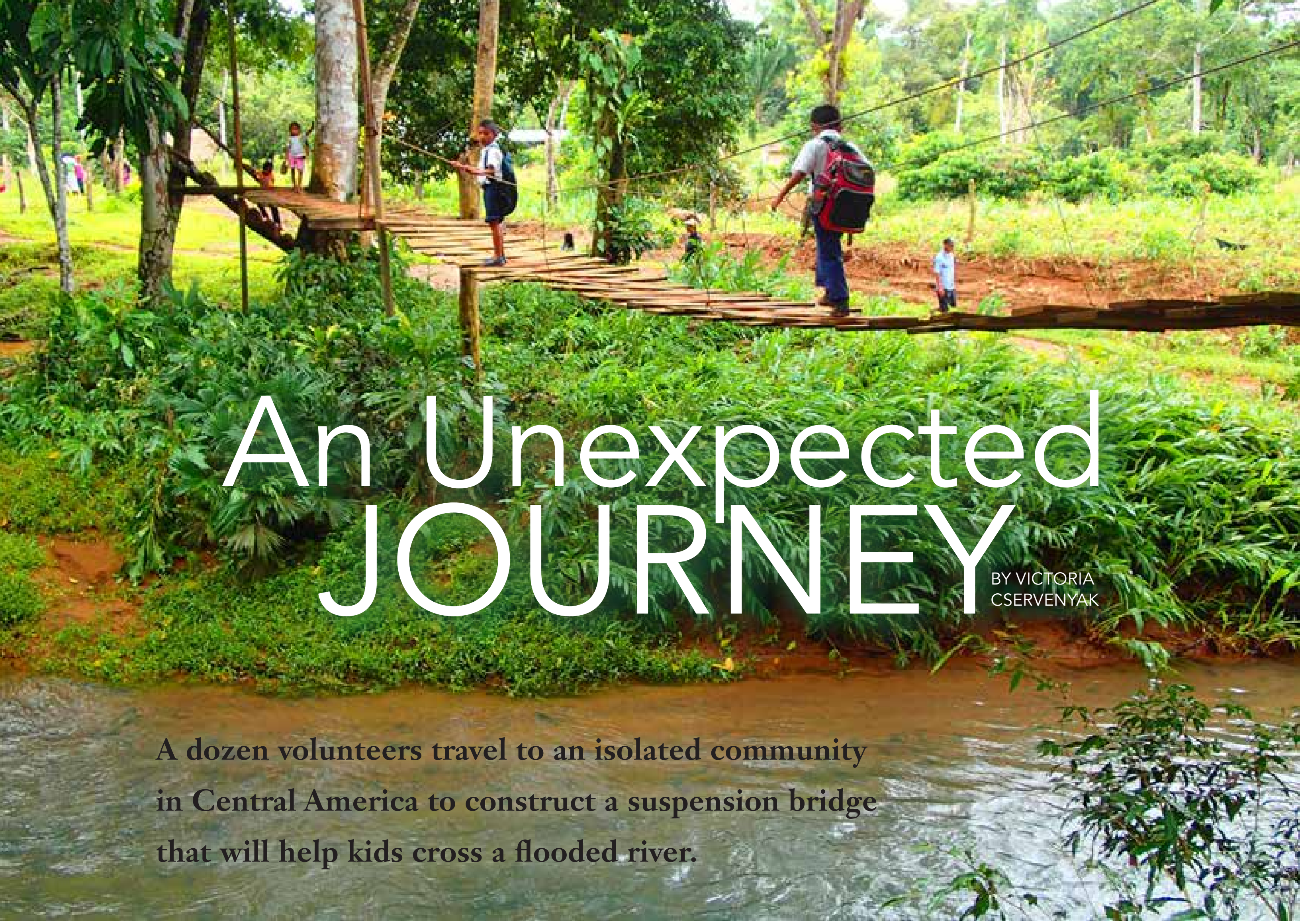
*For more on this project, see "Move that Bridge!" in the February 2012 issue and the item "Biggest-Ever Bridge Slide" in the News section of the August 2014 issue, both available at **www.modernsteel.com**.*

Owner
Indiana Department of Transportation, Indianapolis
Kentucky Transportation Cabinet, Louisville

Designer
Buckland and Taylor, Ltd., North Vancouver, B.C., Canada

Contractor
Walsh Construction, Chicago

Steel Fabricator
High Industries, Lancaster, Pa. AISC CERTIFIED FABRICATOR



An Unexpected JOURNEY

BY VICTORIA CSERVENYAK

A dozen volunteers travel to an isolated community in Central America to construct a suspension bridge that will help kids cross a flooded river.

THIRTY MINUTES AWAY from the closest town, and still 30 minutes away from their destination, the winding, asphalt-paved road carrying 12 volunteers dissolves into a rough, dirt and gravel path.

The loss of cellular phone service is forgotten as succulent greenery surrounds the SUVs carrying the travelers, who chat apprehensively. Outside, it is 95 °F with nearly 100% humidity—an average temperature for mid-April in this part of Panama.

As they continue travelling down the dusty road, small, single-story homes begin to appear every few hundred yards until finally, the travelers reach a valley.

Although there is a language barrier—only one of the group's members can speak Spanish—the curiosity and excitement are mutual as the amiable villagers greet the group and assist in assembling their tents outside the pavilion. Covered but without walls, the pavilion will serve as their base for the next two weeks.

The serene farming community of Lura, in Churuquita Grande, Coclé, Panama is where they will build a 51-m (about 168 ft) suspension bridge across the Lura River to replace the hazardous existing bridge, made even more perilous in the rainy season, which is starting soon.

The travelers comprise a volunteer team of industry professionals from the U.S. representing Bridges to Prosperity (B2P), a Denver-based, nonprofit organization that builds footbridges in isolated communities around the world.

Over the past 18 months, Jeff Carlson, National Steel Bridge Alliance (NSBA) regional director, commissioned a team of 11 other volunteers including Holly Bartelt, (B2P), Rafael Davis, (Arizona DOT), Jonathan Hirschfeld (Hirschfeld Industries), Mike Keever (California DOT), Jessica Martinez (Colorado DOT), Curt McDonald (HDR, Inc.), Theodore "Tad" Molas (WSP | Parsons Brinckerhoff), Adrian Moon (WSP | Parsons Brincherhoff), Nate Neilson (Utah Pacific Bridge and Steel), John Rohner (CH2M HILL) and Josh Sletten (Utah DOT), a group that is unique to B2P's Industry Partnership Programs because of the diversity in the type of organizations involved.

"What is great about this team is that we all came from different backgrounds," said Sletten. "We're all at different levels in our own organizations. When we came to this team, we were all equals."

Unanticipated Progress

In the dense darkness, a rooster begins crowing long before 6 a.m., prompting team members to gradually emerge to divide responsibilities and prep for the day's work—roughly eight hours of physical labor under the tropical sun.

Before this trip, no one on the team had been involved in the manual exertion of bridge building. The B2P team in Panama—consisting of program manager Devin Connell, two masons (David Hernandez and Asuncion "Chon" Sanchez Castro) and

▲ According to the United Nations' Sustainable Development Goals, inadequate infrastructures thwart inhabitants from leaving their community to access agricultural, educational, economic and health-care resources, perpetuating poverty.

► The volunteers slept in tents for their two-week stay.

◄ Basic infrastructure—roads, information and communication technologies, sanitation, electrical power and water—remains scarce in many developing countries.

▲ ▼ The bridge was made of local materials consisting of custom bent rebar suspenders with steel cross-beams, timber decking and safety fencing.

Bridge Corps Fellow (B2P's volunteer division) Kelsey Welch—have already built the anchors, ramps and the pedestals that the tower rests on. Now the team must enact their strategy to complete construction in 12 days.

Unlike the meticulous calculations and planning and incorporation of historical data for bridges in the U.S., the masons determined bridge dimensions by asking villagers to recall the highest level they saw the river at during previous rainy seasons, then add 2 m (about 6.5 ft) of freeboard onto the estimate to elevate the highest point of the bridge, the mid-span, to be 6 m (about 20 ft).

Jonathan Hirschfeld

Holly Bartelt/Brenda Troyo

▲ The team connecting angles to the main tower structure.

▼ The team pulling the main cable through the anchors.

Brenda Troyo

Victoria Cservenyak (**cservenyak@aisc.org**) is AISC's digital communications specialist.

▲ Mike Keever (left) and Josh Sletten (right).

▼ Holly Bartelt (left) and Jeff Carlson (right).

▲▼ When the bridge opened, the entire community walked across.

◄ B2P staff and community members assemble the reinforcing cage for the anchor.

Community residents journey more than an hour to the jobsite; walking and horseback riding are their only form of transportation. Without a safe bridge to cross, the Lura River segregates neighbors on each side. But now with the shallow water levels, they join the team not only to build the bridge but also to learn how to maintain and repair it.

Although the language barrier sometimes slowed progress, the tireless determination of the bridge builders, as well as the village women who cooked all of the meals and even laundered some of the volunteers' clothing, invigorated and united the team and community.

At noon, they briefly exchanged work for lunch and shade. The soup du jour, replete with chicken, potatoes, plantains, rice, root and yucca, introduced exotic flavors to some of the team members. Five hours later, dinner was a combination of the same ingredients, presented in a different form. After the final meal, nonverbal cues and common interests cheerfully connected the villagers with the volunteers as they played baseball, Frisbee and kickball until everyone contentedly retired for the night.

"Being in the Lura community allowed me to focus on just living in the moment and I wasn't worried about the normal things that distract on a daily basis, you're just enjoying the moment," Hirschfeld said.

Aside from occasional trips to Penonomé, the province's capital, for Internet access and more supplies, the team repeated their daily pattern and completed the bridge four days early and under budget.

Overwhelming First Steps

The team decided special supplies were needed to inaugurate the bridge. "No fumar" read the sign when they pulled up to a difficult-to-locate home an hour away from Lura—"No smoking" because the house could easily be consumed by flames: It was full of fireworks. The team purchased 100 fireworks to help with the celebration.

A few short, but sentimental speeches opened the inauguration. The community donated one of their cows for the festive lunch, performed traditional dances and played a three-inning baseball game against the project team, using the home plate backstop the volunteers just constructed on the village's makeshift field.

"Just to see how proud the community was, and to hear that they really want to take ownership of the bridge, was really gratifying," Carlson said.

When the bridge opened, the entire village joyfully walked across it. An older man grabbed Bartelt's arm and with a look of overwhelming gratitude, repeatedly thanked her and conveyed to the translator how he had been waiting for the bridge for years and was elated the team came to build it.

"A lot of my life choices were solidified in this moment," Bartelt said. Nine months before arriving in Panama on her first suspension bridge construction project, Bartelt moved from Indianapolis to Denver to work for B2P, moving out of her home state for the first time in her life. After traveling to Kenya to consult on a bridge project, she realized her purpose: to use her engineering skills to support isolated communities.

▲ Bartelt and the villager who thanked her on inauguration day.

➤ The volunteers purchased 100 fireworks to light at the inauguration.

▲ The team on the completed bridge.

Isolated Community, Connected Commonality

Several of the team members described the trip as "exceeding expectations" and an "overwhelming success." Not only had the project been completed early and within budget, but the team also had no health, safety or weather issues. The volunteers' commitment to create safe access to vital services such as health care, jobs, markets and schools for the community of more than 500 also helped to develop fast friendships amongst the group.

Upon meeting the group on the first day, the Peace Corps volunteer translator, Brenda Troyo, was in awe of their camaraderie and was convinced that they were all longtime friends. "We were able to work together, solve problems, communicate well and then execute on that strategy and build a successful project," said Hirschfeld. "I thought it was pretty impressive. The people within the community and the people that we had on the project are who made it memorable and successful."

And the greatest measure of success is the knowledge that almost 200 children who once walked more than an hour-and-a-half to school each day, without knowing for sure that they could complete their journey over the sometimes flooded old bridge, now had a sure path. ■

Through local engagement, from regional governments to members of each partner community, B2P is committed to a sustainable model that puts the focus on people and the opportunities that make it possible for them to thrive. In 2016, B2P will build 40 new footbridges, increasing the overall total to more than 200 bridges and raising their cumulative impact to one million people worldwide. To learn more about B2P, how you can become a volunteer or industry partner or to support their mission, visit **www.bridgestoprosperity.org/what-you-can-do**.

Highly skewed bridges now have a solution involving halved round HSS for improving fatigue performance, allowing better fit-up and facilitating easier installation of diaphragms and cross frames.

TWO HALVES are Better than None

BY YUAN ZHAO, P.E., PH.D., KARL FRANK, P.E., PH.D., AND JOHN HOLT, P.E.

WHEN IT COMES TO BRIDGES, the best skew is none at all.

Unfortunately, skewed supports for highway grade separation bridges are a reality when the alignment of the crossing roadway(s) cannot be oriented more favorably with regard to that of the bridge. Although highly skewed bridges—those with skew angles greater than 45°—are needed infrequently, they present significant challenges and difficulties to all parties involved with their design and construction.

Of their many challenges, one is the connection of cross frames to girders along the lines of support, such as bents or piers. A skewed stiffener cannot be used when the skew is significant due to the inability to make the weld on the acute angle side. The common approach for these connections on highly skewed bridges is to use a bent plate at each corner of the cross frame, with one leg of the plate bent parallel to the skew, or close to it, and the other leg attached to bearing stiffeners or connection plates installed square to girder webs. These bent plates are typically the most flexible component of a lateral bracing assembly, limiting a cross frame's ability to be fully engaged in girder bracing and introducing additional girder rotations at skewed supports.

A relatively new alternative to this traditional approach uses half of a round hollow structural section (HSS) in place of a conventional bearing stiffener on each side of a girder web. Attached to this round HSS section is a cross frame connection plate, placed in line with the support skew angle. This connection plate is radial to the half HSS regardless of skew and permits a conventional cross frame to be connected to the girder without any bent plates. This alternative solution is known as a split-pipe bearing stiffener (SPBS).

Research

The potential benefits of the SPBS led the Texas Department of Transportation (TxDOT) to fund research at the University of Texas at Austin to investigate cross frame connection details for skewed bridges. The research indicated that the SPBS provides a much stiffer cross frame connection detail than conventional details using bent plates, allowing more effective use of cross frames and providing better resistance to girder end twist.

Additionally, the researchers found that the SPBS provides significant warping restraint at the support bearings, which increases the torsional stiffness, and in turn, the elastic buckling strength of the girder. This allows for larger unbraced lengths, giving designers the opportunity to move the first interior cross frame locations further from the end bearings. Cross frames located close to end bearings on a highly skewed bridge tend to be more highly loaded than those further away. These highly stressed cross frames, which are too close to the end supports, can potentially create or experience fatigue problems later in the bridge's service life. Having the ability to move the first interior cross frames further from highly skewed end bearings helps reduce their forces and minimize future fatigue-related problems. The increased warping restraint provided by the SPBS will also help with girder stability during handling and lifting.

Design

The research report, "Cross-Frame Connection Details for Skewed Bridges" (available at **tinyurl.com/splitpipe**), provides design recommendations and an SPBS design example, including steps in selecting a half round HSS for providing

▲ Steel girders erected over a 60° skew interior bent.

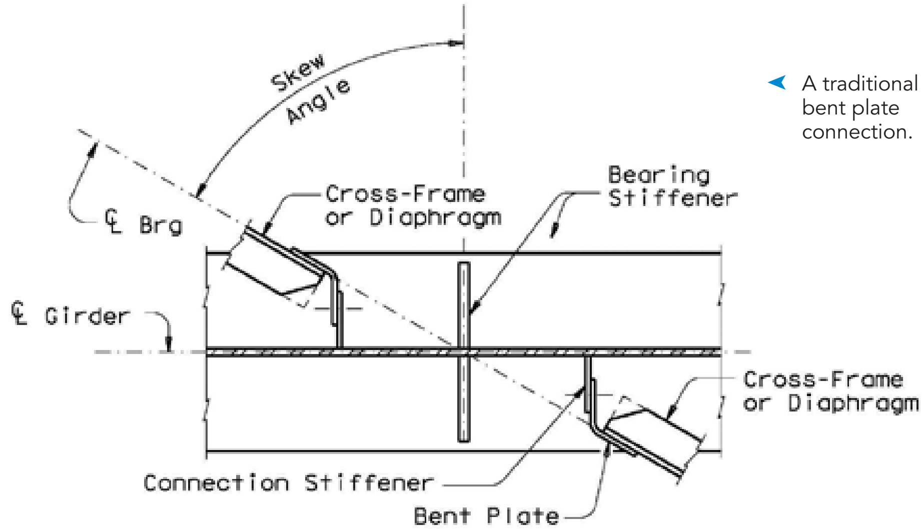
➤ A painted stiffener on a weathering steel girder with a weathering steel connection plate welded to the split pipe.

the required warping restraint based on the required girder buckling capacity.

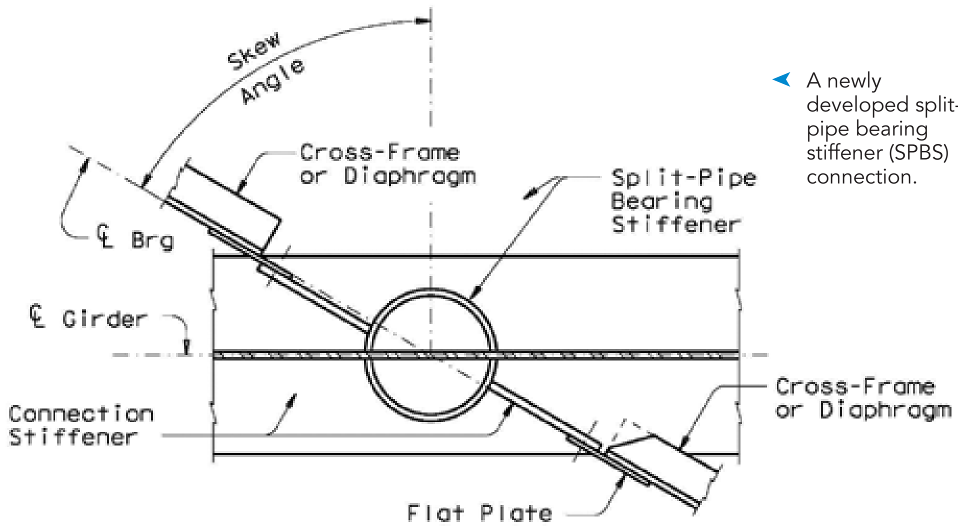
In their design recommendations, the researchers recommended the cross frame connection plate be welded to the HSS only and not welded to the girder flanges. This avoids a fillet weld partially along the length of the flanges, a weld that could not be classified as Category C' like conventional cross frame connection plate welds, which are normal to the web (or slightly skewed, up to about 20°). The lab tests showed that a skewed stiffener weld, partially along the flange's length, experienced a reduced fatigue life compared to that of a normal, Category C' stiffener weld.

However, not connecting the connection plate to the girder flanges contradicts the AASHTO LRFD *Bridge Design Specifications*. These specifications require that cross frame connection plates be connected to girder flanges, which is intended to reduce or eliminate damaging distortion-induced fatigue problems arising from the unstiffened web gaps. The welding of the split pipe to both flange and web eliminates the web gap and provides a stiffer connection between the web and flange than a welded plate stiffener. The stiffener can still be welded to the flange to provide additional restraint particularly for end diaphragm connections where the bending stress in the flange is nominally zero.

Yuan Zhao is a senior structural engineer with Burns and McDonnell (and was previously a bridge design engineer with TxDOT's Bridge Division), **Karl Frank** is a chief engineer with Hirschfeld Industries and **John Holt** is a senior bridge engineer with HDR (and retired as the design section director of TxDOT's Bridge Division).



A traditional bent plate connection.



A newly developed split-pipe bearing stiffener (SPBS) connection.

Based on commonly used girder flange widths for highway bridges—15 in. to 24 in.—most designers will find an SPBS that works by using round HSS of 11 in. to 20 in. in diameter, with wall thickness ranging from 0.50 in. to 0.75 in. Designers should verify that the section selected is available and can meet "Buy America" provisions.

The most oft-recommended material specification for round and rectangular HSS is ASTM A500. Thick-wall extra-strong or double-extra-strong ASTM A53 pipes are generally not available in large diameters or must be bought at a premium; the use of this material is therefore discouraged by fabricators. With the recent advent of the ASTM A1085 specification, improved HSS products are now available for structures subjected to dynamic loading. All HSS produced to A1085 has a minimum yield strength of 50 ksi and is required to meet the equivalent AASHTO Zone 2 CVN requirements for Grade 50 steels (minimum 25 ft-lb at 40 °F). Whenever availability is not an issue and its use can be justified economically, A1085 should be the recommended HSS specification for bridge applications.

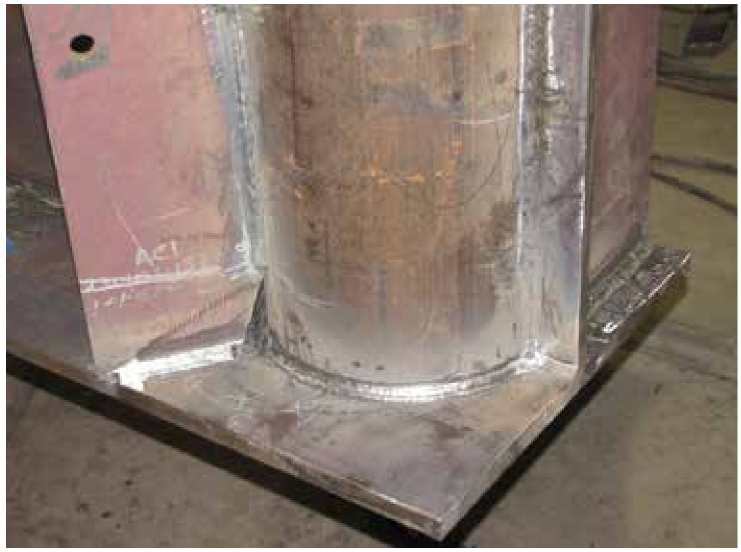
Both A500 and A1085 are suitable for bridges that will be painted. Some bridge owners, however, prefer the aesthetic benefits of weathering steel. Finding a round HSS section meeting ASTM A847 (a weathering steel grade) can prove challenging, especially in the diameters expected for a SPBS system. The quantity of round HSS needed to provide a SPBS system for a typical bridge project could be small, making availability even more elusive. An alternative for weathering steel is to allow for A1085 or A500 material in lieu of A847 and paint these components to provide an appearance similar to weathering steel. The downside to this approach is the inability of the selected color to match the weathering steel's appearance throughout its life. But instead of trying to match the color of weathering steel, this could be seen as an opportunity to enhance aesthetics and paint the component a contrasting color that complements the overall structure aesthetic.

Lastly, a SPBS is a bearing stiffener, so its design needs to satisfy the conventional bearing stiffener design specifications.

Fabrication

An SPBS offers much simpler and less congested cross frame fabrication than the conventional plate bearing stiffener due to the elimination of bent plates. The fit-up between the pipe and the flange, however, can be challenging. Slight distortion of the flange will cause fit of the pipe to be difficult over its full mating surfaces with the flanges. It is difficult to attain the 75% contact normally specified for bearing stiffen-

An SPBS used for a cross frame connection along a highly skewed bent line.



Cross frame connection details at highly skewed support bearings.

A weld detail at a web-to-flange connection (note that weld access is provided on the acute side).

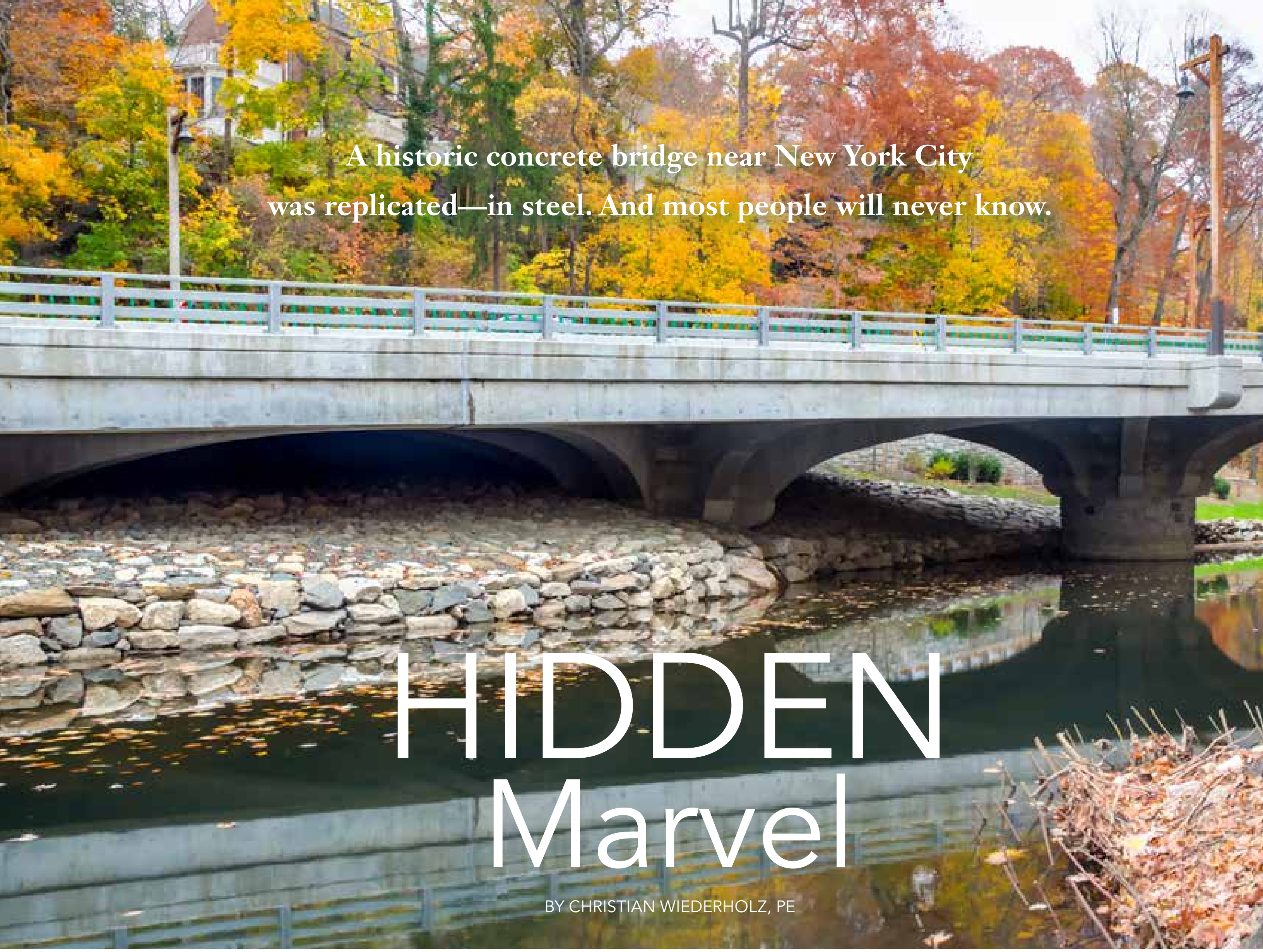
ers. Due to the larger bearing area of the pipe stiffener relative to that of the normal plate stiffener, less area needs to be in contact in the case of the pipe stiffener. A maximum gap of 1⁄16 in., the limit for interior connection plates, can be used instead. This is based on findings of the research lab tests where larger gaps were used without yielding any adverse effect. The split pipe is tacked in place and then the connection plate is tacked to the pipe and flange.

The half-pipe is seal welded with fillet welds on all edges to prevent corrosion on the interior side. These welds will intersect the web-to-flange welds. As a result, it is recommended that the split pipe corners be clipped to clear the flange to web welds with a clearance of no more than 1⁄8 in.

Implementation

Due to its promising advantages, the SPBS system was adopted by bridge designers in several projects even before the University of Texas research was finished. While more jobs are now being designed and fabricated since the 2014 incorporation of SPBS into TxDOT's *Bridge Standards*, the authors are aware of at least four completed SPBS projects in Texas. In all four cases, the construction of all steel units went very smoothly, and no difficulties or challenges were experienced by the contractors in regard to girder erection and cross frame installation.

Based on the research findings and its successful implementation on the aforementioned projects, the SPBS is now incorporated into the TxDOT Bridge Division's standard drawings as a standard cross frame connection detail for future design of highly skewed steel bridges. When used in conjunction with a properly designed superstructure framing layout, the SPBS offers advantages over a simplified and streamlined connection for lateral bracings. With more designers specifying the use of SPBS, the cost of this system is also expected to compare favorably with that of the conventional stiffeners in future applications. ■



A historic concrete bridge near New York City was replicated—in steel. And most people will never know.

HIDDEN Marvel

BY CHRISTIAN WIEDERHOLZ, PE

WHEN YOU HEAR "MUSHROOM BRIDGE," you may very well think of Mario Kart.

But it's also the nickname of a portion of the Crane Road Bridge, a nearly century-old crossing in north suburban New York City.

Comprising two bridges, it carries the Bronx River Parkway through a forest preserve in the Village of Scarsdale and the Town of Greenburgh in Westchester County. The southern bridge is a multi-span concrete bridge (the Mushroom Bridge portion) that crosses the Bronx River, while the northern bridge is a single-span steel through-girder bridge that crosses the two-track Metro-North Railroad Harlem Line (accordingly called the MNR Bridge).

Christian Wiederholz (christian.wiederholz@stantec.com) is an associate and structural engineer based in Stantec's Rochelle Park, N.J., office.

Dating back to 1925, both bridges have been rehabilitated over the years. But when two separate incidents, both of which involved holes punched completely through the concrete deck, resulted in emergency closures and repairs, the Westchester County Department of Public Works and Transportation took immediate action.

An in-depth bridge inspection revealed that the concrete deck slab of the Mushroom Bridge was in a state of advanced deterioration. The reinforcing steel exposed through the spalled concrete had advanced section loss, and the concrete slab around the holes was completely pulverized. The concrete brackets had numerous cracks and spalls, and the overall condition of the deck was rated poor and quickly approaching severe.

Furthermore, traffic safety along this stretch of the Bronx River Parkway continued to be a major concern. Within the project limits, narrow travel lanes with minimum shoulders, sharp curves and limited sight distance had led to an accident rate six times higher than the statewide average. As a result, the County hired a team led by structural engineer Stantec, contractor EE Cruz and construction inspector LKB, Inc., to inspect, repair and design replacement bridges for the aging structures.

Photos: Stantec

Pedestrian use and access to the bridge was improved with the reconstruction.

The new bridge alignment enhances the historic bridge's roadway geometry.

Blending History with Innovation

From the outset, the County and the other agencies involved with funding and approving the project recognized that preserving the unique historic and environmental character of the bridge was a crucial factor. After an exhaustive public screening process and inter-agency coordination, the County selected a replacement alternative of constructing a new bridge that replicated the existing Mushroom Bridge structure.

This alternative provided wider replacement structures immediately south of the existing bridges, allowing for construction phasing such that traffic could continue to use the highway while improving the roadway geometry. The proposed modifications included:

- Replacing the Mushroom Bridge superstructure and substructure with a substantially wider deck
- Replacing the MNR Bridge with a redundant type structure (composite prestressed concrete box beam bridge)
- Replicating the existing Mushroom Bridge piers to maintain the form of the historic pier configuration while relocating them along a similar alignment
- Completing the project's construction within an approximately $39 million budget and three-year schedule

Achieving these vast improvements was no easy task. While the bridge was to be widened, the number and size of the piers were required to remain more or less the same due to flow concerns, which presented obstacles for the structural design. Each existing mushroom superstructure panel was approximately 40 ft by 40 ft, or 1,600 sq. ft. The proposed configuration for each panel was approximately 60 ft by 60 ft, or 3,600 sq. ft, so each panel more than doubled in size. Therefore, the loads that each bracket and pier needed to carry were increased substantially.

The design codes had also changed since the original bridge was built, requiring the replacement structure to carry much heavier traffic loads than the original design. Additionally, each bracket has needed to be significantly longer in order to reach the ends of the enlarged panel. Deflections at the tips of these brackets played a large role in the design of the structure.

The Steel Solution

Most of the Bronx River Parkway bridges are made of concrete, including the existing Mushroom Bridge. Reinforced concrete arch brackets radiated from the central piers, and the center of the deck panel directly over the piers was heavily reinforced with bars from each bracket that converged and overlapped. Since the new brackets are longer and carry more

The partially erected Mushroom Bridge, showing steel core and steel brackets.

The bridge's concrete-encased brackets and core.

Existing bridge in the background; replicated structure in the foreground.

The bridge's uniquely shaped piers gave it its Mushroom Bridge nickname.

load, using reinforced concrete brackets in the new design proved to be impossible due to rebar congestion. In order to address the unique configuration of the superstructure, an innovative steel option was used in which an octagonal core made up of steel plates was fastened to the concrete pier.

The steel core is made up of 1¾-in. steel side plates and 2½-in. top plates. Steel brackets protrude from each face of the core and are spliced to steel ring plates at the top of the core. To provide access for bolting, the top plates were designed as rings, leaving a 3-ft diameter opening in the center. A bottom flange shelf support and web connection, along with with horizontal stiffeners to combat the prying action, complete the bracket connection to the core.

Further complicating the custom steel core were the longitudinal and transverse slopes of the roadway, as well as the curvature of the bridge. As a result, each octagonal core was unique. But while the top of the core was sloped in two directions, the top flanges of the brackets were level, so tapered shims were required to facilitate the bolted connection.

The steel core was secured to the concrete pier via anchor bolts and studs. The built-up plate brackets were designed with varying depths to decrease the weight and allow replication of the existing curved arch brackets. The perimeter of each 60-ft by 60-ft deck panel was lined with rolled steel edge and fascia beams that connect to the tips of the cantilever built-up steel brackets. The entire steel solution was encased in concrete to maintain the overall aesthetic of the existing bridge.

In order to provide for a redundant structure, the bridge was designed to remain in service even after complete failure of one bracket per panel. Additionally, each panel of the superstructure can behave independently, gaining no necessary support from an adjacent panel.

Built to Last

Despite the tight parameters, the project team designed a replacement structure that has vastly improved the crossing's safety. The entire bridge is wider, including all four travel lanes and shoulders. There are also acceleration/deceleration lanes now, which were virtually nonexistent before. In addition, a merging roadway was raised so that drivers on the Bronx River Parkway and the road can easily see one another and merge safely, and the curved alignment was softened to improve sight distances.

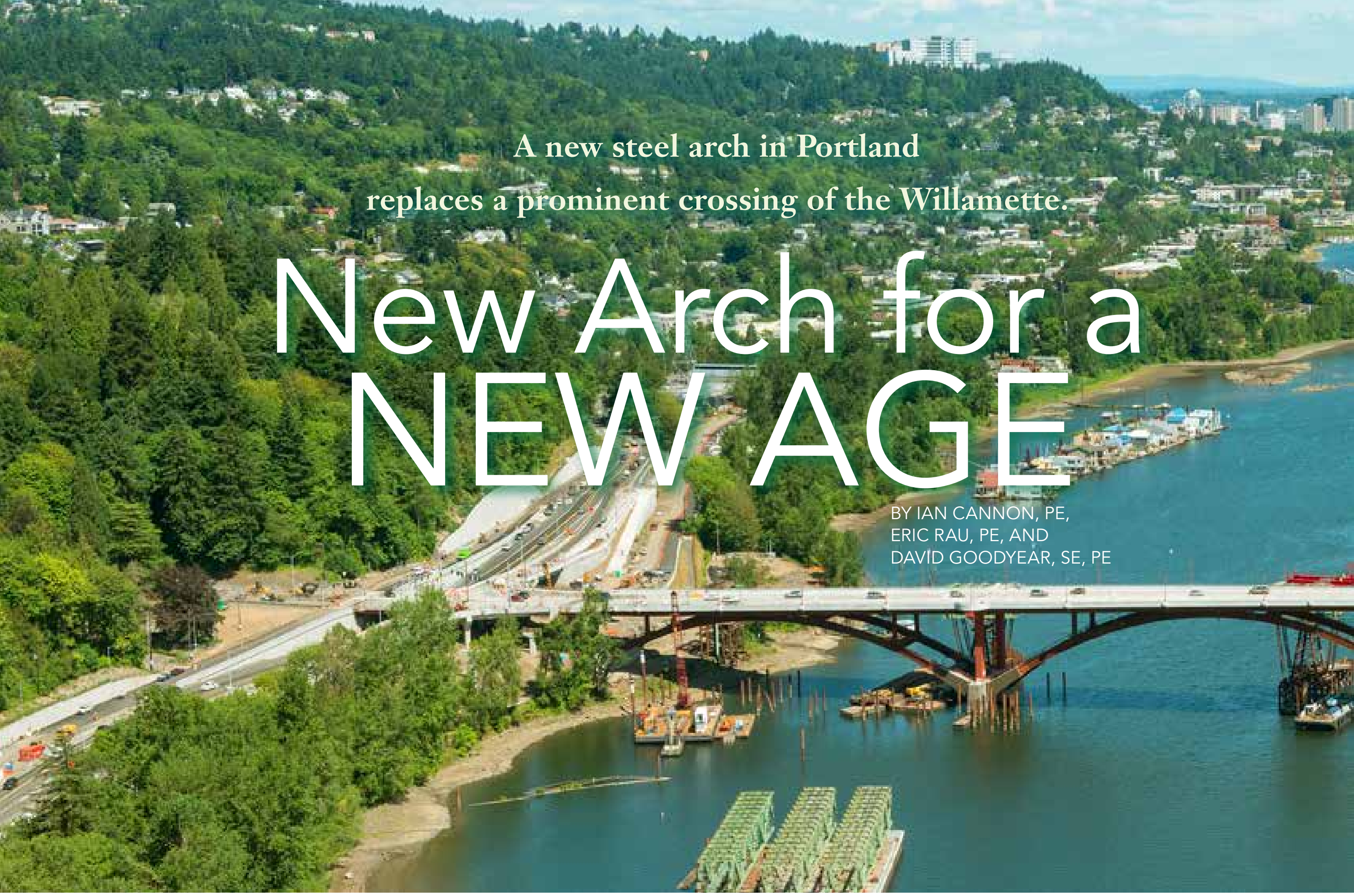
And beneath a facade of salvaged stone facing lies the Crane Road Bridge's most unique and crucial feature: its steel core. While the replacement bridge continues to be a striking and historic fixture of the Bronx River Reservation, the true marvel of its reconstruction will remain hidden from the public eye. ■

Owner
Westchester County Department of Public Works and Transportation

General Contractor
EE Cruz, New York

Structural Engineer
Stantec, Rochelle Park, N.J.

Steel Fabricator
American Bridge, Coraopolis, Pa.



A new steel arch in Portland replaces a prominent crossing of the Willamette.

New Arch for a NEW AGE

BY IAN CANNON, PE, ERIC RAU, PE, AND DAVID GOODYEAR, SE, PE

TWO OUT OF 100.

That was the National Bridge Inventory (NBI) sufficiency rating that the 90-year old Sellwood Bridge received in 2005 after the latest round of engineering studies, emergency repairs and additional load restrictions. Multnomah County, Ore., the owner of the bridge, was keenly aware that shoring up the old bridge was no longer an option.

Constructed in 1925 to replace the Spokane Street Ferry, the Sellwood Bridge spans the Willamette River just south of downtown Portland. It was designed by Gustav Lindenthal, a noted bridge engineer of the time and—along with the nearby Ross Island and Burnside bridges—was built with funds from a $4.5 million local bond measure.

Lindenthal was hired to redesign the Sellwood Bridge as a result of cost overruns on the Burnside Bridge. The result was a unique and efficient four-span continuous steel truss costing a mere $541,000. At 32 ft wide, the bridge was extremely narrow: two lanes, no shoulders or median and one 4-ft-wide sidewalk. It was Portland's first "fixed span" bridge across the Willamette and the first to not be designed for streetcars.

The NBI rating of 2 for the old bridge reflected a number of critical issues ranging from movement of an ancient landslide on the west bank of the Willamette to general deterioration of the 90-year old concrete approach structures.

The County began the NEPA (National Environmental Policy Act) process in 2006, and an engineering team of CH2M and T.Y. Lin International (TYLI) was retained to perform the engineering studies and develop alternatives for a new crossing. The evaluations included rehabilitation and replacement options for the main bridge, a dozen structure types for the main crossing and various alignments and project configurations. The recommendation was replacement on the same alignment, and through an active and meaningful public outreach process, the Community Advisory Committee's (CAC) preferred

Ian Cannon (ian.b.cannon@multco.us) is Multnomah County's transportation director and program manager of the Sellwood Bridge project, **Eric Rau** (eric.rau@tylin.com) is a bridge engineer with TYLI and **David Goodyear** (david.goodyear@tylin.com) is TYLI's chief bridge engineer and the lead bridge engineer for the Sellwood Bridge project.

Oregon Department of Transportation

The new Sellwood Bridge over the Willamette River near downtown Portland, Ore., replaces a more-than-90-year-old span that had become unusable.

alternative—a steel deck arch—was approved by the County Board of Commissioners.

Forming a Team

Multnomah County elected to use the construction manager/general contractor (CM/GC) method of project delivery. A primary advantage of this method is that the contractor, in the role of construction manager, provides direct input to the owner and design team regarding constructability, pricing, scheduling and phasing of the work throughout the design process. In 2009, Multnomah County selected SSJV, a joint venture between Sundt Construction and Slayden Construction, as the CM/GC for the bridge based on a competitive qualification based proposal. In 2010, TYLI and CH2M were selected to develop the final design. To facilitate the CM/GC process, Multnomah County established a collocated project office with full-time staff from the owner, owner's representative David Evans and Associates, the engineering design team and the CM/GC.

The 1,275-ft main structure over the river is flanked on the east by a five-span concrete approach structure extending 500 ft from the riverbank into the adjacent Sellwood neighborhood. On the west side, the structure terminates with a significant interchange connection to Oregon Highway 43, which is composed of approximately 3,600 ft of bridge and retaining wall ramp structures.

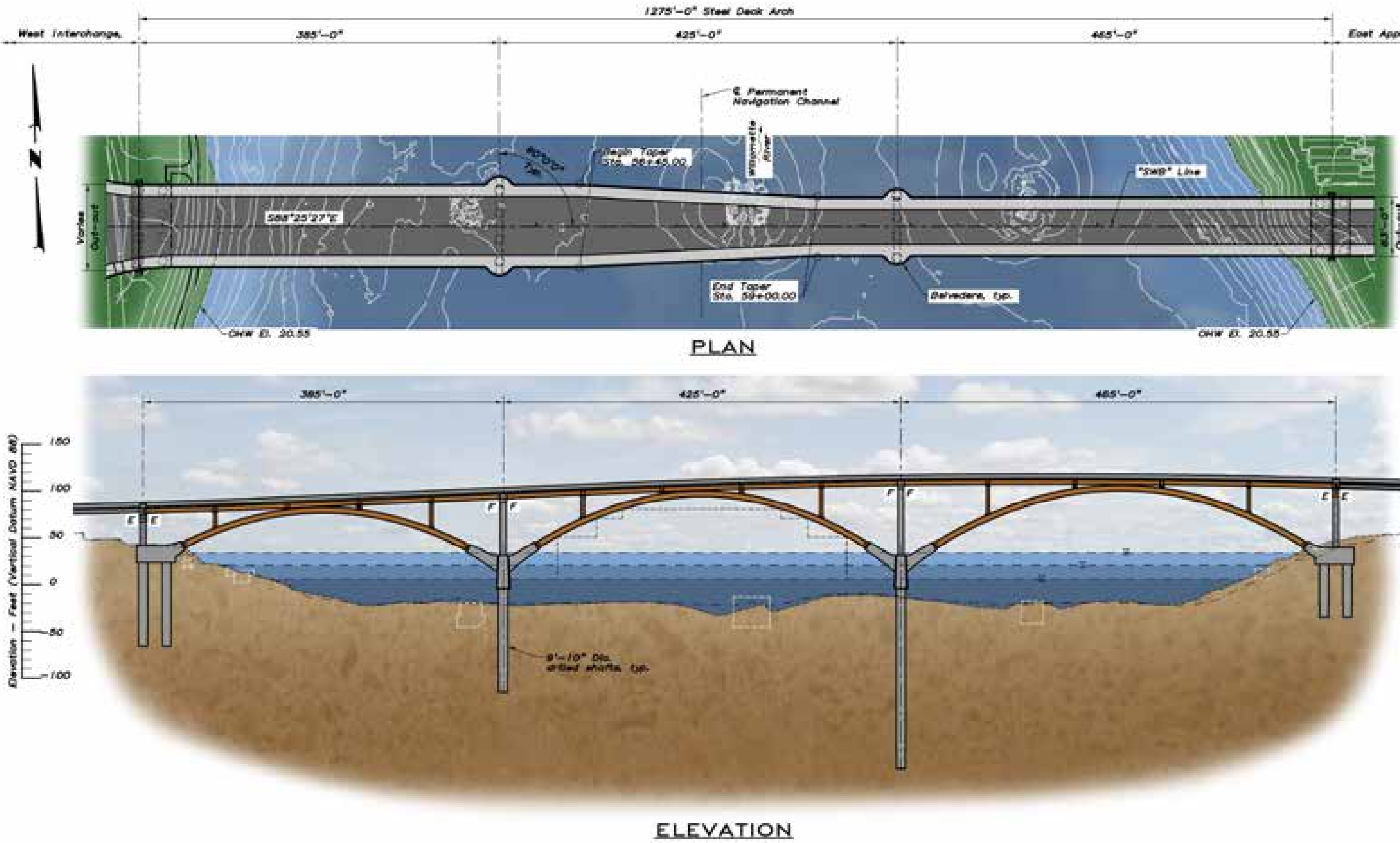
The west side of the project site is located within an ancient landslide, which had moved about 4 ft since the original bridge opened in 1925. To prevent movement during construction and stop chronic seasonal movements in the long term, an anchored shear pile system that spanned the full 500-ft width of the landslide was employed. Consisting of 40 6-ft-diameter drilled shafts connected by a grade beam and 70 ground anchors with loads up to 850 kips per anchor, the system is designed to limit seismic deformation to under 4 in. during a moment magnitude scale (MMS) 9.0 Cascadia Subduction Zone earthquake. The landslide mitigation was bid at a construction cost of $14 million.

Both the original truss bridge and new arch bridge have only two through-traffic lanes. This was the recommended configuration from the environmental impact statement (EIS) stage, driven by the request from the Sellwood neighborhood to restrict traffic to two lanes to match the capacity of Tacoma Street to the east. While the existing bridge had an overall structure width of 32 ft, the new structure provides 6-ft, 6-in. shoulders, designated bike lanes and raised 12-ft sidewalks on each side of the bridge. The result is a pedestrian-friendly structure that has a nominal width of 63 ft. The structure width increases on the western half of the bridge to 90 ft, allowing for additional turn lanes to and from Highway 43. Using 5,000 tons of structural steel, the bridge opened earlier this year.

Steel Deck Arch Structure

The 1,275-ft-long three-span steel deck arch has a span arrangement of 385 ft-425 ft-465 ft, with two arch ribs per span. The progression of span lengths generally follows the rise of the bridge in grade from west to east.

A reinforced concrete Y-arm extends from the pier and footing substructure to meet the steel arch rib at the springing connection in order to keep the steel ribs above the 100-



Plan and elevation drawings of the new bridge.

year flood stage. These extensions are up to 36 ft in length at the river piers and follow the curved geometry of the arch.

The solid-ribbed arches are welded box sections with a constant web depth of 70 in., a flange width of 54 in. and a smooth parabolic curve profile (all steel curving was performed in-house by the project's fabricator, Thompson Metal Fab). Each of the three arch spans has four spandrel columns, which coincide with the location of the portal bracing between the two ribs. Each spandrel column supports a transverse steel cap beam, with longitudinal girders spanning between them.

Both the girders and cap beams have an overall steel depth of 60 in. and are composite with the reinforced concrete deck. The girder system is 15-span continuous over the 1,275-ft arch structure, with five to seven girder lines spaced up to 14 ft, 6 in. Based on pricing feedback from the CM/GC, plate transition splices were eliminated and flange and web plate thickness were held constant for the entire girder system. Flange plate width varied based on structural demand but was held constant within a spandrel span.

Top and bottom girder flanges are connected across the cap beams with a continuity connection plate while the girder web is connected with traditional clip angles. The cap beam has an internal diaphragm at the girder line, and the entire connection is bolted. The CM/GC requested slotted holes at specific girder locations to increase tolerances for fit-up during erection.

The transverse cap beams are built-up box-shape members composed of two welded I-girders with top and bottom cover plates. The entire assembly is bolted to eliminate the possibility of crack propagation across the entire section and is designed for the loss of either I-shape or cover plate.

The spandrel columns are welded box sections with dimensions of 42 in. × 36 in. and plate thicknesses varying between 1.25 in. and 2 in. The connection of the spandrel columns to the arch rib is a bolted end-plated moment connection.

Establishing the articulation of the spandrel columns was an important aspect of the design. Design iterations evaluated various configurations of "pinned," "fixed," and "free" boundary conditions at the 12 column locations, with the primary challenge being to balance structure stiffness and load path during seismic and thermal response.

The final articulation uses unidirectional bearings at the top of spandrel columns in the flanking spans 3 and 5 and fixed end-plate moment connections for the columns in the center span 4 (the middle arch span). These fixed columns function similarly to a closed arch crown, while the deck structure is free to move at the ends.

Engineering Development

Like many replacement projects, local site conditions and the associated built environment imposed a number of engineering challenges. The structural system of the new Sellwood Bridge had to meet the following constraints:

- Provide a horizontal and vertical navigational opening that meets or exceeds that of the existing bridge.
- Provide a span layout that, when combined with the existing bridge, would allow continued navigation throughout construction
- Limit the amount of structure constructed in the waterway to comply with no-net-river-level-rise criteria.
- Provide a similar roadway profile as the existing bridge in order to limit project extents and facilitate construction staging

Meeting the profile grade requirement resulted in limited rise in the west arch. The three arches have a rise-to-span ratio that varies from 1:7.7 (0.13) to 1:6.4 (0.16). The shallow nature of the fixed arches led to increased bending demands compared to the more efficient arching action that could be attained with more ideal geometry.

In order to limit the effects of flexural demands on the size of the arch section, the springing connections were left in a pinned condition during construction from initial rib placement through concrete deck placement. The two-hinged arch freely rotated during construction loading, resulting in "simple span" bending, with zero negative moment at the springing support and increased positive moment at the crown. After deck placement the springing connection was fixed, shifting the flexural response toward "fixed-fixed" beam action for subsequent loading.

The springing connection consists of ten 4-in.-diameter ASTM A354 Gr. BC high-strength steel rods that are embedded up to 15 ft into the concrete substructure. In the temporary hinged condition, the rods are not tightened to the end of the arch ribs. A high-strength (15-ksi) UHMW pin plate was placed at the springing connection to transfer axial thrust while allowing rotation, and was coupled with an external frame support for vertical loads. Upon completion of staged construction, the fixed connection was completed by grouting the pin plate gap and prestressing the anchor rods for service level moments.

Thompson Metal Fab proposed piece-by-piece stick erection, with arch ribs placed on shoring towers instead of a float-in system originally considered for arch erection. Each rib span contained two bolted field splices to match the optimum weights chosen by the CM/GC for fabrication and erection, resulting in three segments per span with lengths up to 148 ft and weights up to 146 tons each. Steel was transported to the site on barges and placed with cranes operating from work bridges and barges.

When the bridge opened to traffic, the crossing immediately jumped to a sufficiency rating of 100. ■

Owner
Multnomah County, Ore.

General Contractor
Slayden/Sundt Joint Venture
Slayden Construction Group, Stayton, Ore.
Sundt Construction, Tempe, Ariz.

Structural Engineer
T.Y. Lin International, Beaverton, Ore.

Architect
Safdie Rabines Architects, San Diego

Steel Team

Fabricator
Thompson Metal Fab, Vancouver, Wash.

Detailer
Candraft Detailing, Inc., New Westminster, B.C., Canada


Nick Garibbo/Nick's Photo Design

The shallow nature of the bridge's fixed arches led to increased bending demands.


Oregon Department of Transportation

Steel was transported to the site on barges and placed with cranes operating from work bridges and barges.


Nick Garibbo/Nick's Photo Design

There are three segments per rib span, with lengths up to 148 ft.

The 1,275-ft-long three-span steel deck arch has a span arrangement of 385 ft-425 ft-465 ft, with two arch ribs per span.


Nick Garibbo/Nick's Photo Design

On the Fast TRACK

BY BEN NEAL

How to plan and prepare for what-ifs when constructing a rapid-replacement rail bridge.

All photos: The Ruhlin Company

Ben Neal (bneal@ruhlin.com) is a superintendent at the Ruhlin Company near Akron, Ohio, where he focuses on renovation and new construction of structural and industrial projects.

NORFOLK SOUTHERN'S RAIL LINE THROUGH Monroe, Mich., is an important, if perhaps unknown, contributor to the U.S. auto industry.

Carrying trains between Toldeo and Detroit, it is a direct conduit for supplying commodities and freight to automakers. But a major component of the line, an existing steel bridge over the Raisin River in Monroe, had reached the end of its useful life. Built in 1894, the three-span ballasted deck through-truss (Baltimore configuration) had deteriorated to the point where replacement was the only solution to keeping this important rout open.

The $10.9 million project posed numerous challenges, including constructing a new steel superstructure and three new piers,

▲ Picking the new span up from the lay-down yard.

◀ Removing an existing span.

▲ Placing an old span on an island downstream to make way for the new spans.

using the same alignment and grade as the existing structure while minimizing impact to railroad traffic. Through proactive problem-solving and teamwork, the construction team preassembled the new spans in a staging area just south of the site while simultaneous construction of the new piers was taking place beneath the existing bridge.

Minimizing Delays

The new bridge is a four-span through-plate girder superstructure. With the new structure continuing along the same alignment and grade as the existing bridge, a traditional construction approach would have required a four-month disruption in train service. With the emphasis on minimizing train delays, Norfolk Southern granted a five-day (120 hours) outage to complete the replacement of the structure. With a tight work schedule and a complicated span erection sequence, safety was a critical factor. Coordination was made more complex by random train movements, which came from both directions throughout the day. Constant communication between work crews and the on-site Norfolk Southern representative ensured that everyone was clear prior to trains coming through the site.

In preparation for on-site construction, the team created a lay-down yard on the southeast corner of the structure while a temporary causeway was constructed across the river. The causeway was constructed using 15,000 tons of stone, 120 tons of temporary steel beams and 160 crane mats. The temporary beams and crane mats were used to create bridges to maintain the flow of the river and allowed for quick removal in times of high water to eliminate the potential for flooding upriver. These temporary bridges had to be designed to withstand the weight of the cranes carrying the new bridge sections as well as maintain the ability to be removed quickly, if the need arose. While the new piers were being constructed beneath the existing bridge—they were constructed to within 1 in. of the existing structure—the project team constructed four new steel spans in the lay-down yard. Each span consisted of approximately 385 pieces of structural steel and 7,500 field bolts, and each incorporates a ¾-in. steel deck that was welded together during assembly to create a solid floor plate 115 ft long. Steel for the entire bridge totalled 780 tons. Using a steel deck also expedited construction by allowing the deck to be installed ahead of time, as other types of deck would likely have had to be installed during the outage. Waterproofing was applied over the entire deck prior to the outage in order to make efficient use of the available work time.

The Importance of Preplanning

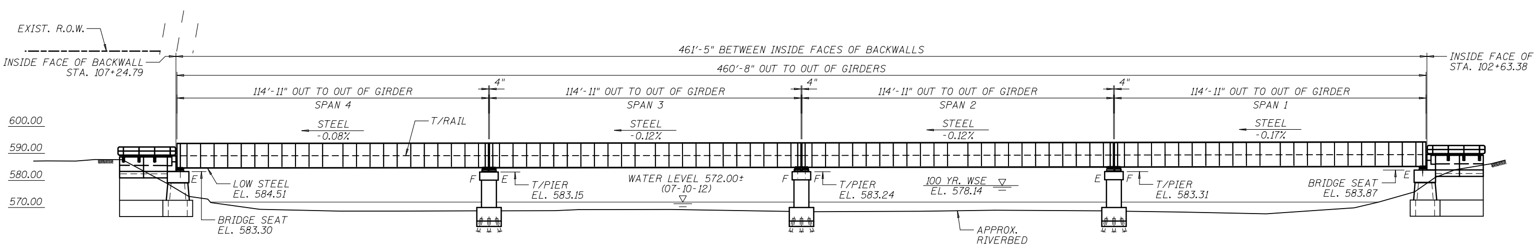
Extensive planning was required prior to the owner granting an extended outage. Several team meetings were held to coordinate not only the work to replace the structure, but also the work required to maintain the signals and remove and replace the tracks themselves. An outage schedule, broken down into half-hour increments, was required by the owner prior to the beginning of the work outage window. When all of the pieces were in place, the outage was scheduled for late winter. However, four days prior to the scheduled outage, the area received 3 in. of rain that caused ice in the river to break and flow downriver. The temporary bridges were removed to prevent potential flooding, and the outage was delayed. Luckily, preplanning during the design phase anticipated the potential for flooding,

Unloading steel in the lay-down yard.

and a contingency plan was built into the project to address possible delays. Further project discussions led to the outage being rescheduled for July 2015. Activities were rearranged so that work that was scheduled to take place after the outage could be performed prior to the outage in order to keep the crews productive. The decision to postpone until July was made to avoid the area's historically wet spring season and the potential for even more flooding. As such, the existing structure would continue to carry train traffic for a few additional months.

During the outage, the existing bridge was dismantled, via torch-cutting, enough to lighten each span for removal. The three existing spans were then picked and placed on temporary stone islands downriver to make way for the new spans. Two

A view, from the deck, of two new spans set onto new piers.



An elevation view of the new bridge.

existing sandstone and concrete piers and the top 4 ft to 5 ft of the existing bridge abutments were also removed during demolition. The removed spans produced 750 tons of steel to be recycled.

Once demolition was completed, bearings were placed on the newly built piers and precast concrete abutment pieces were placed to allow for span erection. The new spans were then rigged and walked into place with a tandem pick by 300-ton Manitowoc 2250 crawler cranes. Each pick was approximately 200 tons and carefully choreographed so that weight remained balanced and the operators of each crane stayed in sync throughout the movement. The spans were carried approximately 1,000 ft across the lay-down area and through the river before being swung into their final positions on the newly installed bearings. Once the four new bridge spans were placed, expansion joints were installed and waterproofing was applied over the joints.

Up and Running

This bridge replacement demonstrates the importance of coordination, planning and execution of complex projects. The challenge was to balance replacing the bridge quickly and avoid an extended outage while also factoring in potential weather- and flood-related delays inherent to the area. Starting with design, the project team rose to that challenge and was able to work together to meet a critical time frame while keeping quality a priority and maintaining the project budget. Crews worked in two 12-hour shifts so that construction continued around the clock for 120 continuous hours, and the project was completed on time, resulting in minimal disruption to the trains and the industry that they serve. ■

Owner
Norfolk Southern, Norfolk, Va.

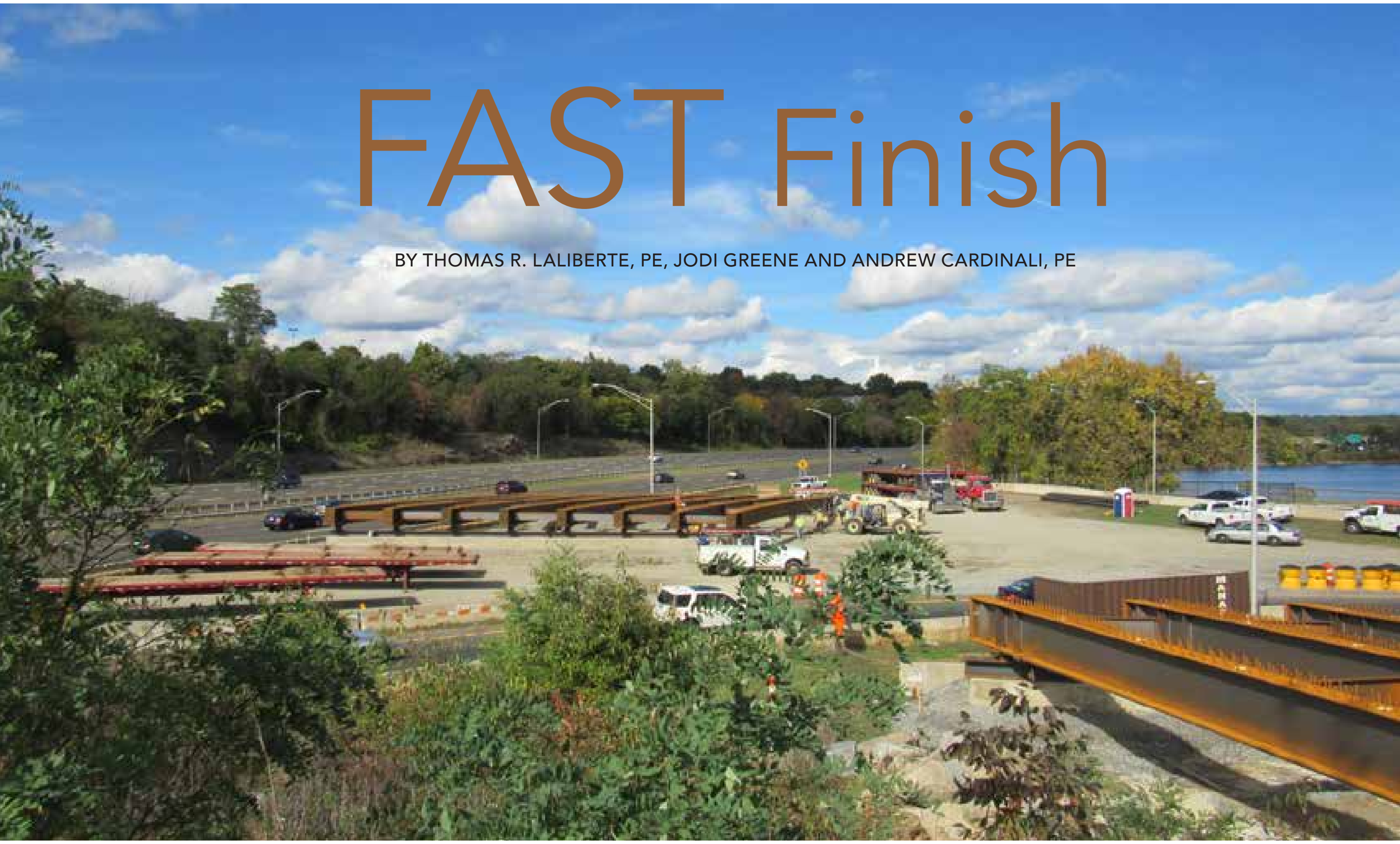
General Contractor
The Ruhlin Company, Akron, Ohio

Structural Engineer
Alfred Benesch and Company, Chicago

Steel Fabricator
Industrial Steel Construction, Inc., Gary, Ind.

AISC CERTIFIED FABRICATOR

The Connecticut DOT makes quick work of its first-ever design-build project.



FAST Finish

BY THOMAS R. LALIBERTE, PE, JODI GREENE AND ANDREW CARDINALI, PE

At the lay-down area, temporary or "mock" abutments were constructed to match existing bridge substructures, which were reused for the proposed bridge.

WITHIN 15 MONTHS from the notice to proceed for design, four bridge superstructures carrying Route 8 over Lindley Street and Capitol Avenue in Bridgeport, Conn., were fully replaced and open to the public.

The bridges were originally built in 1970 with prestressed concrete girders. These bulb tee girders experienced premature deterioration caused by alkali-silica reactivity, and the failed bridge joints created sub-

Thomas R. Laliberte (lalibertet@pbworld.com) is the Connecticut structure lead and the lead bridge engineer for the Route 8 project, and **Jodi Greene** (greenej@pbworld.com) is a bridge engineer; both are with WSP|Parsons Brinckerhoff's Connecticut office. **Andrew Cardinali** is a supervising engineer for CTDOT and was the owner's project manager for the design of the Route 8 project.

structure spalls and bearing damage. The poor superstructure conditions are what prompted this rehabilitation project.

Hoping to complete the project within a shortened period, the Connecticut Department of Transportation (CTDOT) conducted a risk analysis that determined this project was appropriate for its first design-build endeavor. The design-build project delivery method allowed general contractor Manafort Brothers, Inc. (Manafort) and engineer WSP | Parsons Brinckerhoff (WSP | PB) to develop innovative design and construction solutions from the start of design through the end of construction. Design began in April 2015, girders were delivered to the site in October and all bridges were replaced and in service by July 2016.

Crossover

A significant portion of the project involved preparing for two 14-day crossover periods when the superstructures were replaced. Due to the geometry of the existing highway alignment, it was possible to shift both north- and southbound lanes onto the original northbound highway during the crossover period. This method was repeated in the opposite direction as well. When traffic was diverted to one side of the highway, demolition of the existing and construction of the new structures was performed on the opposite side. Various accelerated bridge construction (ABC) techniques were used to replace the superstructures in less than weeks, and one specific technique was the use of prefabricated bridge units (PBUs). The short construction schedule relieved highway users (the average daily traffic volume on the bridges is 89,300 vehicles) and the community of Bridgeport from long-term construction impacts.

The PBUs were designed to be constructed off-line, then erected during the crossover periods. They consisted of two steel plate girders and a concrete deck, and the magnitude of the units made transportation and erection challenging. The PBUs were up to 90 ft long by 17 ft wide and weighed up to 260 kips. Manafort assembled the units at a site less than one mile from the bridges, then they were transported to the project site with 16-axle highly specialized steerable and adjustable trailers, then finally tandem-picked into final position with 300- and 500-ton cranes.

➤ WSP | PB designed prefabricated bridge units for the Route 8 bridge project.


Captain's
A beach in every can.

◄ Traffic was shifted to one bound during each crossover period. The two sides were replaced in under 14 days.

▼ Highly specialized steerable and adjustable trailers were used to transport the PBUs on local roads to the bridge location.

At the assembly area, temporary or "mock" abutments were constructed to match existing bridge substructures, which were reused for the new bridges. Each bridge span included four PBUs that were constructed together full width and length to ensure fit-up during erection. All steel members were erected prior to casting the deck, then the diaphragms between the PBUs at the closure joints were removed when it was time to transport the units. As a result, minimal adjustment occurred at the final site.

Switch to Steel

The bridge design focused on reducing the weight of the superstructures for both final condition and construction purposes. For this reason, steel girders were used instead of the concrete scheme of the original bridges. The overall superstructure weight reduction prevented the need for advanced geotechnical analysis on the existing abutments and piers, which were reused for the new bridges. The switch from concrete to steel girders resulted in lighter PBUs, ultimately saving on the costs associated with larger capacity cranes and transportation devices.

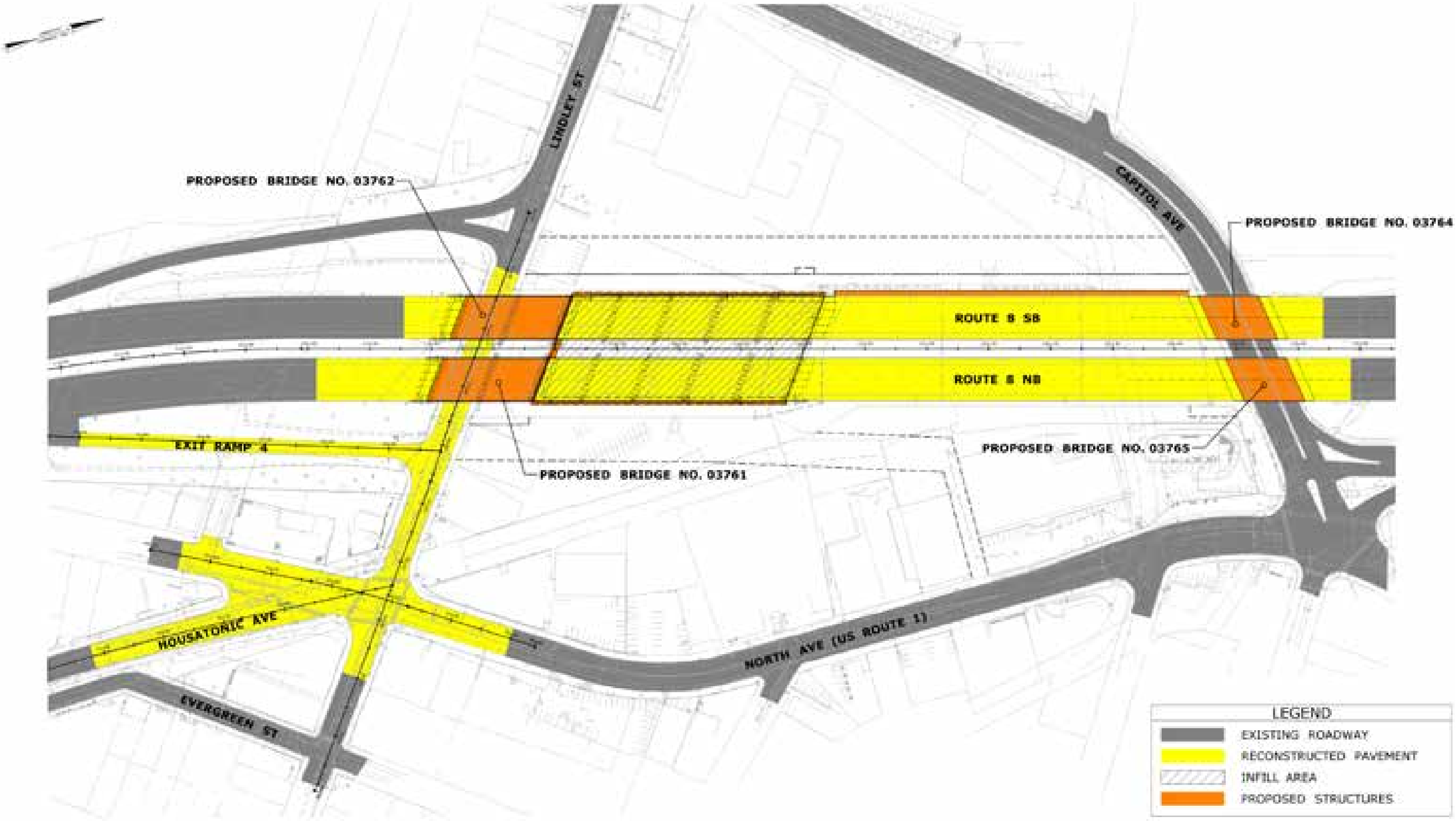
The design-build delivery method prompted innovative use of steel girders that resulted in benefits to safety, schedule and budget. The steel girder design reduced superstructure depths; at controlling locations, minimum vertical clearances increased from 14 ft, 5 in. to 15 ft, 11 in. for local roads below, improving safety to the traveling public. The design also allowed for an alteration that reduced the number of girders per span from twelve to eight, which decreased the number of PBUs and required cast-in-place deck closure pours. The ripple effect of this change saved both cost and time for the required materials, fabrication, erection and construction of each bridge superstructure.

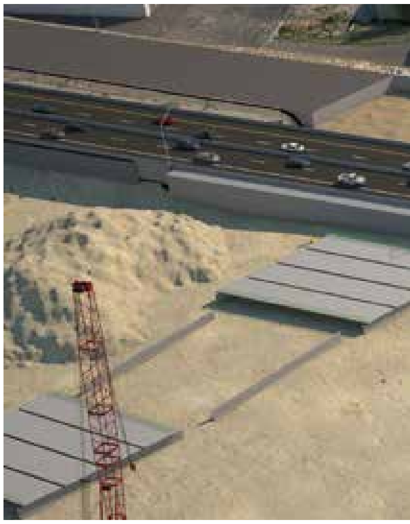
Reduced Maintenance

In the long term, the replacement project will reduce bridge maintenance. The existing bridges over Lindley St. consisted of seven spans, and the new configuration filled in five spans and replaced the final two, resulting in ten fewer spans to inspect and maintain; retaining walls were constructed alongside and below the existing bridge prior to the crossover period to accomplish this. The existing bridges were carefully demolished adjacent to these walls, and fill brought the grade up to match the elevation of the bridge approach.

In addition, the bridge superstructeres are built from weathering steel, which will eliminate the need for future painting. The design incorporates link slabs and semi-integral abutments, which will eliminate bridge joints and associated damage to bearings and concrete, a problem on the original bridge. The existing structures only served for 46 years, and CTDOT was keen on avoiding premature replacement in the future. The new bridges have an expected service life of 75 years.

The two crossover periods required round-the-clock construction activity, with more than 100 employees working to further accelerate the schedule. Manafort completed the closure periods a total of





▲ The design team produced visualizations for public information meetings to communicate the project sequencing to the community.

◄ Construction activities were schedule around the clock during the crossover periods.

► The bridge cross section was composed of four PBUs with closure pours between units.

four days ahead of the 28-day schedule, and one local paper noted that the project was "likely the fastest bridge replacement project ever seen in Fairfield County." ■

An overhead view of the project layout.

Owner
Connecticut Department of Transportation

General Contractor
Manafort Brothers, Inc., Plainville, Conn.

Structural Engineer
WSP | Parsons Brinckerhoff, Glastonbury, Conn.

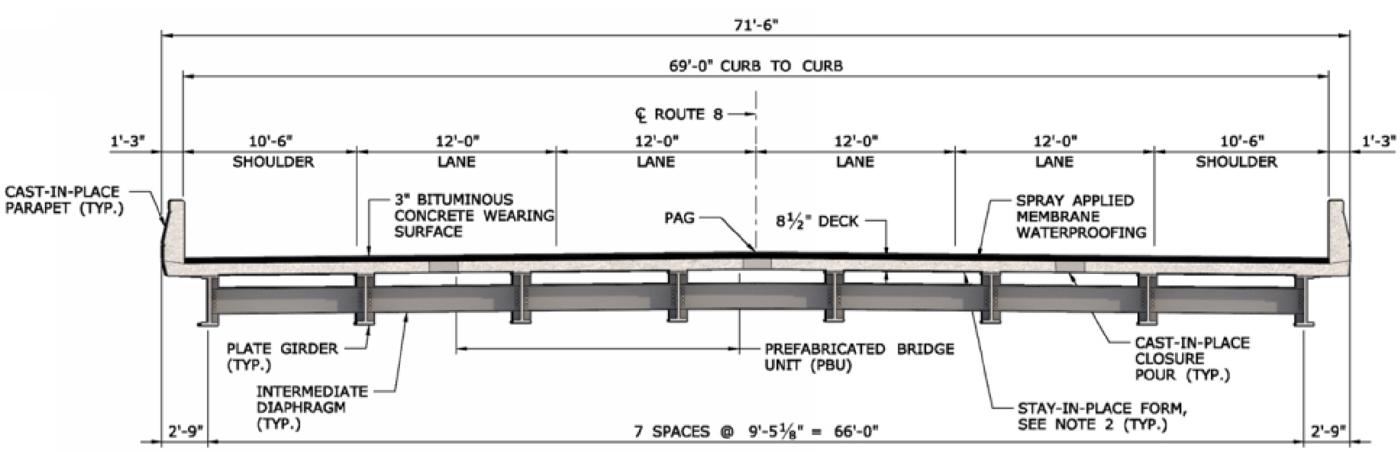
Steel Team

Fabricator
High Steel Structures, LLC, Lancaster, Pa.

Erector
Hartland Building and Restoration Co., East Granby, Conn.

The PBUs were assembled about a mile from the final bridge locations.

Seven spans of the existing bridge were demolished within hours of the start of crossover.



TYPICAL CROSS SECTION

A Utah bridge team turns to structural steel solutions for expanding an Interstate crossing to accommodate high-occupancy vehicle lanes.

HIGH Volume, LOW Impact

BY JASON KLOPHAUS, MICHAEL GOODMAN AND CORIN PIACENTI

Jason Klophaus (**jason@klophausllc.com**) is the owner of Klophaus and Associates. **Michael Goodman** (**goodmanmp@pbworld.com**) and **Corin Piacenti** (**piacentice@pbworld.com**) are senior bridge engineers with WSP|Parsons Brinkerhoff.

The sliding system involved an end diaphragm with the stainless steel slide shoe on Teflon bearing pads.

The permanent abutment between the temporary abutments provided a level sliding path.

AS UTAH'S ONLY north-south Interstate, I-15 is critical to the state's infrastructure.

Average annual daily traffic in 2012 was approximately 65,000 vehicles in each direction and is predicted to rise to over 80,000 by 2040. To brace for this increase and help address the potential for the delays it would likely incur, the Utah Department of Transportation (UDOT) has implemented systematic capacity improvements through the addition of high-occupancy vehicle (HOV) lanes in the Salt Lake City vicinity.

One prominent interchange where HOV lanes were recently implemented is the I-15–Hill Field Road interchange located 25 minutes north of Salt Lake City. A pair of existing three-span bridges carried the I-15 traffic while local traffic moved along Hill Field Road below. The goal of the project was to provide additional structure width for HOV lanes as well as to upgrade the intersection to a single-point urban interchange (SPUI) configuration. This configuration allows large volumes of traffic to move through limited space by providing multiple turning movements below the bridges while high-volume traffic flows on I-15 above.

Using the design-build method, the design and construction team believed that building the new substructure behind the existing bridge abutments, then sliding the superstructures into place would be the most cost-effective and least disruptive approach. The construction would take place in three phases and maintain three lanes of I-15 traffic in each direction by using the southbound bridge as a "shoofly" (a solution in which an existing bridge is temporarily used as a detour in both directions).

The southbound bridge was used as a shoofly while the northbound bridge was slid into its final position.

The new superstructures were built next to the existing ones.

Wing wall construction, following the slide.

The project came with challenging construction requirements:

- The existing bridges were three spans, each 56 ft wide. Clearances over Hill Field Road to the bottom of girders were substandard
- Lowering the Hill Field Road profile would require extensive utility (storm and water line) relocations, create new drainage issues and require more roadway reconstruction
- The RFP allowed only two full 12-hour closures of each direction of I-15. Additional full or partial closures would be charged to the contractor at $20,000 per lane per hour
- Hill Field Road was allowed only 12 off-peak, 12-hour full closures, with ramps used to maintain I-15 traffic during that time. Any additional full or partial closures would be charged to the contractor at $10,000 per lane per hour

Sliding Solution

The team explored multiple accelerated bridge construction (ABC) concepts. Sequential phasing of the superstructures was immediately ruled out due to strict maintenance of traffic (MOT) requirements and closure constraints. Self-propelled modular transporters (SPMTs) were also examined but ended up not being cost-effective or convenient when considering the grading that would need to occur to accommodate the new interchange configuration. However, as the new structures were adjacent and similar in geometry, a third ABC concept—bridge sliding—proved to be a winner and was selected based on estimated completion time, feasibility and cost.

Temporary pile-founded abutments would be driven adjacent to either side of the existing structures and connected to future permanent pile-founded abutments, which were to be built once the existing structure was removed. This long continuous section of temporary and permanent abutments would be the level surface that would support the superstructure during the slide. The wider typical section of the new bridges allowed for part of the permanent abutments to be used in the temporary construction location. The southbound superstructure (in the temporary location) would carry I-15 traffic in a shoofly condition while the existing structures could be removed.

Once the shoofly was in place, the existing bridges could be removed, the northbound bridge would be slid into place and all traffic would move to the northbound structure. Finally, the southbound bridge would be slid into place, approaches would be completed and the remaining civil work could occur. The temporary abutments would then be removed prior to substantial completion.

Slide Shoes

To move the superstructure, the team employed two, 13-ft-long concrete blocks with polished stainless steel surfaces at the bottom of each end diaphragm. These shoes would slide on Teflon bearing pads, with the superstructure pulled by prestressing jacks at each abutment line. The Teflon pads had a lubricated coefficient of friction of about 5% during the slide and were replaced with permanent bearing pads in the final location.

The team initially considered a prestressed concrete solution, but this would have required a two- span bridge and a bent that would be overly expensive to build. This approach would also have increased user cost penalties thanks to the additional lane closures needed to construct a pile cap, columns and bent cap. It would also increase the overall length of the structure to accommodate the SPUI configuration. Overall, an equivalent concrete option for this project would have weighed 437.5 tons more and required a structure that was 3 ft deeper.

All of these considerations led the team to select a single-span steel superstructure. Steel allowed for a shallow girder section that closely matched the existing structure depth. The I-15 northbound and southbound profiles matched the existing profiles at the bridges, which minimized expensive interstate roadway reconstruction. The Hill Field Road profile was lowered only 2 ft; this was 2 ft less than what the RFP concept plans called for and required fewer utility relocations. Plus, the lightweight single-span steel solution was simply easier and quicker to slide. A single span meant one less temporary support, thus reducing traffic interference when shifting lanes under the new and existing structures.

Due to vertical clearance requirements, the northbound bridge was built 2 ft higher than the final condition. This provided temporary clearances that would ensure the new steel gird-

▲ The prestressing resistance anchor block used in the slide.

ers would not be damaged by over-height vehicle traffic below the bridge prior to the bridge slides. Once again, the single-span solution proved beneficial, as it simplified lowering the structures.

Fast Start

The schedule dictated that the project needed to be substantially completed by August 1, 2016, and the design and construction schedules had to be coordinated to allow lead time when ordering materials, particularly the steel girders. The design team provided an early steel package one month after the notice to proceed (NTP) to allow for the 12-week fabrication time. Complete bridge plans were released for construction by mid-July 2015, just three months after the NTP.

Pile driving for the temporary abutments began immediately. The end diaphragms with slide shoes were formed on the temporary abutments, girders were set on these end diaphragms and both superstructures were complete by September. As the southbound bridge functioned as a shoofly, northbound and southbound lane capacity was maintained, avoiding work through the winter.

On March 9, 2016, one year after design began, the I-15 northbound bridge was scheduled to be moved into place. Preparations began several weeks earlier with the removal of the existing structures and placement of permanent piling and pile caps. Several days prior to the slide, prestressed cable was threaded through the end diaphragm blockouts.

The day of the slide, the northbound structure was lowered 2 ft onto sliding pads and Hill Field Road was closed to traffic. The horizontal movements began at midnight and the 1,600-ton bridge—both bridge spans are 178.5 ft long—was slid 74 ft in five hours. (Also, these girders were fabricated and erected at full length, thus avoiding the need for field-bolted splices.) Following the slide, earthwork and approach wingwalls were constructed.

On May 1, the second bridge slide was completed, following the same principles. The team switched to hardwood, instead of the more compressible plywood, to support the Teflon slide bearings. They also tensioned the prestressing strands more uniformly for smoother jack advancement. These improvements, along with the crew's familiarity with the system, resulted in a slightly faster completion time for the second slide. Wing walls, approach slab construction, backfill and civil work associated with the second bridge were delayed due to a wet spring season. Despite the complications, the project opened to traffic by the end of August.

Thanks to the design-build process and the ABC component of bridge sliding, the team was able to work together to optimize construction. And for this project (and many others) steel played a critical role in the ability to design longer spans and minimize reconstruction. ABC, design-build and structural steel were the perfect match for this prominent Interstate overpass. ■

Owner
Utah Department of Transportation

General Contractor
Ames Construction,
West Valley City, Utah

Structural Engineers
WSP | Parsons Brinckerhoff, Murray, Utah
Klophaus and Associates, Salt Lake City

Steel Fabricator and Erector
Utah Pacific Bridge and Steel Corp., Lindon, Utah AISC CERTIFIED FABRICATOR

A new accelerated bridge construction solution makes its commercial debut in a statewide bridge replacement project in Pennsylvania.

A NEW TAKE on Plate Girders

BY TOM STOCKHAUSEN

Tom Stockhausen (thomas.stockhausen@cdrmaguire.com) is president of CDR Bridge Systems, LLC.

THE PENNSYLVANIA Rapid Bridge Replacement Project (PRBRP) is ambitious in scope, to say the least.

The project involves replacing 558 structurally deficient bridges over a three-year span, and the majority of the bridges in the project are short spans proposed to be replaced with concrete structures. Approximately 10% of the bridges restricted the general contractor (Walsh/Granite) to a five-week maximum detour, thereby necessitating accelerated bridge construction (ABC).

As an alternative to concrete, general contractor Walsh/Granite turned to the folded steel plate girder (FSPG) system for multiple restricted-detour bridge projects under the PRBRP. FSPG bridges were first constructed as demonstration projects in Massachusetts and Nebraska using Accelerated Innovation Deployment Grants from the Federal Highway Administration (FHWA). When compared to concrete, they can be erected faster, last longer, require roughly the same level of maintenance and compete in terms of cost. As of publication, four of the seven bridges ordered by Walsh/Granite have been manufactured and two have been erected. The three others are currently being manufactured.

Installing the first section.

The folded steel plate girders are fabricated from a single steel plate of uniform thickness that is bent along multiple lines using a hydraulic metal press brake, forming a trapezoidal-shaped section that is open at the bottom

Concurrent Construction

Conventional construction of a typical precast 60-ft bridge was estimated by Walsh/Granite to take about 11 weeks following design and manufacturing of the bridge beams. In many rural areas where detours are quite lengthy, the 11-week detour placed a severe impact on local residents, in particular emergency vehicles and school buses. However, the FSPG solution moved much of the construction process off-site, where it was performed concurrently with demolition of the old bridge and construction of new abutments—which reduced the typical detour time frame by more than 50% as well as accelerated design and manufacturing. An added benefit was the improved quality of the deck concrete because it was constructed in a controlled environment.

The ability to standardize the FSPG process is part of what significantly reduced the design and manufacturing time. With 11 standard sizes, design basically involves selecting the appropriate size girder for the span length and opening, then detailing the horizontal and vertical geometry of the specified deck. This allowed superstructure design for the Pennsylvania projects to be accomplished in a matter of days.

CDR Bridge, which designs and manufactures the FSPG system, was also able to reduce total production time for the Pennsylvania bridges to as little as 14 weeks, including steel procurement. Manufacturing took place in three steps: forming the steel, galvanizing it and then precasting the deck panels. For this project, the girders were formed by cold bending ½-in. plate steel. Shear studs, sole plates and bearing stiffeners were welded and flange separators were bolted to complete the first step. The steel fabrication process took less than three weeks for four girders of a typical two-lane bridge.

The second step in the process was applying corrosion protection via hot-dip galvanizing, which took only a few days to complete. The galvanizing has a guarantee of 25 years and reduces maintenance over that time frame to a level equivalent to or better than that of concrete. Removable, galvanized bird screens were added to the open bottom of the FSPG girders and allow easy inspection of the girders while still keeping birds and other critters from nesting on the bottom flanges.

The final step in the manufacturing process was precasting the deck panels to form the composite system. Precasting the panels (for the four folded steel plate girders) took about three weeks. Once

Installing the third section.

Folded Steel Plate Girders

What is a folded steel plate girder (FSPG)?

Developed at the University of Nebraska-Lincoln, the girders are fabricated from a single steel plate of uniform thickness that is bent along multiple lines using a hydraulic metal press brake, forming a trapezoidal-shaped section that is open at the bottom. The plate thickness of either ⅜ in. or ½ in. can accommodate all span lengths by simply changing the location of the bends. Only the width of the top and bottom flanges and the depth of the web vary depending on span length. However, the maximum span length for FSPG bridges is currently limited to 60 ft.

The FSPG system eliminates the need for internal and external cross frames due to the large amount of lateral stiffness generated by the design. The absence of cross frames in the system results in less costly details, and the need for welding is significantly reduced. The open bottom geometry of the girders simplifies inspection, and the hot-dip galvanization process is used for corrosion protection.

Two companies—CDR Bridge Systems and HBS—have been granted exclusive distribution rights, and the steel is fabricated by approved fabricators. (You can view standard drawings at **www.cdrbridges.com**.)

the precast decks were completed, the prefabricated units making up the superstructure were ready for shipment to the construction site for erection.

The first of the Pennsylvania FSPG bridges—a two-lane, 50-ft bridge near Bradford, Pa.—was erected this past October in less than three hours. Closure pours (to complete construction of the superstructure) took only a few days, which enabled the bridge to be reopened in 30 calendar days, five days ahead of the accelerated, five-week schedule for the project detour. This is a full month faster than it would have taken using conventional cast construction. The second Pennsylvania FSPG bridge was erected near Lewiston, Pa., just two weeks later—and took only 2½ hours.

Enhanced Accelerated Construction

These first few FSPG units have provided our steel fabrication and precast concrete partners with valuable experience that has led to improvements in their processes and further reductions in manufacturing time for future bridges. It also reinforced the need for communication throughout the process. The team maintained constant communication with the contractor, so that everyone knew precisely where each bridge was in the design and production process. We were able to adjust schedules through the supply chain without interrupting our other team members' business. The responsiveness of the team resulted in the completion of production in advance of scheduled erection for every bridge despite changing erection dates.

In addition, the Walsh/Granite design-build team proactively managed the schedule, thanks to such practices as a weekly discussion of detailed preproduction tasks between the teams. CDR shared changes with its team and coordinated adjustments to its production priorities to meet Walsh/Granite's changing schedule. That proactive coordination enabled the team to meet every erection schedule—especially important on an ongoing project involving multiple bridges spread over a large area.

Another benefit of this proactive approach was that each member of the team brought ideas and energy to the table that improved the process and the system. The engineers aggressively responded to design changes to substructure and roadway design, shop drawings and RFIs. The fabricator procured extra material so that it was prepared for any changes during fabrication. And the design and precast teams worked together to solve an issue with lifting the exterior girders, adding inserts into the barriers to balance the lift and keep the exterior units from rolling during erection.

The entire team worked together to create a better, faster and more constructable product. It also proved that ABC is not just about accelerating the schedule at the job site but rather accelerating the *entire* process of design, manufacturing and construction.

Owner
Plenary Walsh Keystone Partners

General Contractor
Walsh/Granite JV

Architect and Engineer
HDR, Inc.

Superstructure Engineer/Supplier
CDR Bridge Systems, LLC, Pittsburgh

NEW STEEL, Vintage Feel

BY TYLER J. BARILE, CHRISTOPHER J. MENNA, PE, JAMES B. MILLER, PE, CHRISTOPHER J. RENFRO AND MICHAEL J. SCHICKLING

New steel supports an old bridge over a busy transit line in Philadelphia.

PHILADELPHIA IS A city steeped in history, and its bridges are no exception.

The existing Willow Grove Avenue Bridge over Southeastern Pennsylvania Transportation Authority (SEPTA) tracks was a three-span, simply supported, shallow-depth, steel stringer bridge with asphalt deck. The bridge was rehabilitated in 1962 and incorporated the original stone masonry abutments and wing walls that dated to 1884. Located in the Chestnut Hill neighborhood of northwest Philadelphia, the original iron channel and timber superstructure was built by the Edge Moor Bridge Works for the Pennsylvania Railroad—and designed for a live load of horse and carriage.

Built during the industrial revolution, the structure was created to provide grade separation for a street crossing in a very affluent, new neighborhood in Philadelphia. Both the original and rehabilitated bridges featured materials appropriate for the area: timber, metal and Wissahickon Schist stone. Though decidedly inelegant, the structure provided a practical solution to challenging geography in the form of a short hump crest vertical curve nestled between two driveways. Originally spanning over two mainline tracks and a rail siding track to access an ice house, the updated bridge spanned over two electrified tracks between the piers.

Images courtesy of City of Philadelp

◄ "Buddy beams" were installed to strengthen the weakened center-span stringers.

▼ Installation of the sidewalk utility bay.

Emergency Repairs

Prior to 1994, the bridge was on a regular five-year inspection cycle, per Federal Highway Administration (FHWA) standards. However, due to suspected flaws in the 1962 replacement steel (specifically excessive chemical impurities and poor batch casting) along with the constant infiltration of water and snow salts through the asphalt deck, inspection crews discovered in 1995 that deterioration had accelerated aggressively. Strengthening repairs were required, and inspection reporting was increased to a two-year inspection cycle. For the repairs, the City's Bridge Section performed in-depth inspections to develop rehabilitation and reconstruction strategies. The major findings were as follows:

1. Severely corroded steel, including the pier steel bent frames
2. Seized and malfunctioning bearings and expansion dams

Tyler Barile is an engineering specialist, **Christopher Renfro** is an engineering supervisor and **Michael Schickling** is a civil engineer, all with the City of Philadelphia, Streets Department, Bridge Section. **Christopher Menna** is an engineer of design with Jacobs Engineering Group (and was previously with the City of Philadelphia, Bridge Section) and **James Miller** is director of engineering with Michael Baker International.

▲ The completed bridge, from below.

➤ Construction bracing.

▼ The completed bridge, at street level.

3. Abutment and wing wall stonework in need of repointing and/or rebuilding
4. Moderate spalling of the asphalt deck
5. Failure of the superstructure steel coating system
6. Moderate deterioration of the non-composite deck stay-in-place forms

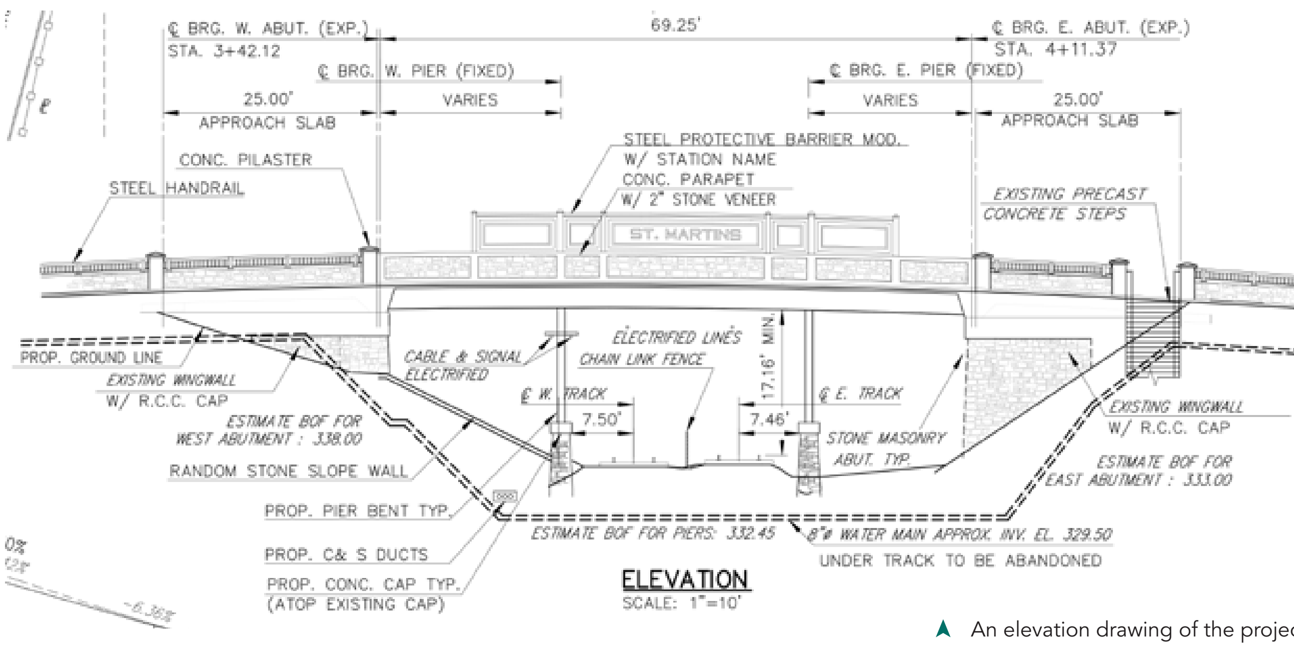
Due to severe steel deterioration under the sidewalk bays, elevated wood boardwalks were furnished and installed temporarily by the City's Bridge Maintenance Unit in 2006 to span weakened areas and provide safe passage of pedestrians. Concrete median barriers were also installed at the curb lines to keep vehicles off the sidewalk, and a truck detour was instituted around the bridge to remain in effect until full reconstruction could take place.

In 2013, the condition and section loss of the stringers became so significant that another emergency repair was performed. Due to deterioration and/or section loss of up to 100% in some places, the bridge was posted for "3 Tons/No Trucks," the minimum allowed by state law. Bridge closure and replacement was imminent unless remedial action was taken. Additional adjacent "buddy beams" were installed to strengthen the weakened center-span stringers, allowing the bridge to remain in service for one more winter without closure and major impact to the commuter rail traffic below. At this point, the structure became the primary design priority for the City's design unit.

New and Improved

The design scope included removal and replacement of the entire superstructure, stabilization and adaptive reuse of the substructure, roadway approach work, reconstruction of a portion of the train station platform stairway and utility work. The engineering and logistical challenges involved with replacing a severely deteriorated, weight-posted, structurally deficient bridge—that was also integrated into an existing SEPTA commuter rail station—were apparent. Moreover, very little information by way of existing plans was available, necessitating extensive survey and substructure probing to verify site conditions. Prior to construction, Verizon, Comcast, City Street Lighting and Philadelphia Electric Company utilities were relocated, and SEPTA Electrical Traction power cables at the west pier were detached. Bridge work would be coordinated with SEPTA, Verizon, Philadelphia Water Department and Philadelphia Gas Works.

Due to the uniqueness and historic setting of this bridge, the design team faced numerous challenges on top of those typically involved with working in a densely populated environment. One of the key questions that had to be answered early on was whether the existing substructure could be reused for a third time. Michael Baker International was contracted as part of the design team, with two main substructure tasks: Verify the existing substructure conditions and formulate an adaptive reuse design plan.



An elevation drawing of the project.

In order to preserve the existing horizontal clearance of the rail, the existing stone masonry piers were maintained. The existing pier foundations consisted of open joint masonry stone walls founded on stepped stone footing. These existing foundations were stabilized by placing a 1-ft.-thick Class C concrete collar around the perimeter to lock in the foundation and solidify the foundation for reuse. The existing stone masonry piers were cleaned and entirely repointed, and a new concrete cap was doweled into the existing stone cap to support new HP12×84 steel columns.

To stabilize the existing abutments and wing walls, existing backfill was carefully removed to the bottom of the existing walls and replaced with Class A concrete immediately behind the masonry structure to form a new gravity abutment, a concept that essentially knit the "old" and "new" structures together. This was done while the existing walls were monitored for excessive movement. The stone masonry of the abutments and wing walls were cleaned, and new caps were provided on each abutment to serve as a seat for the new steel beams.

With the substructure modification plan in place, focus was shifted to stringer design. Numerous beam design runs were performed and compared to the limited available superstructure depth envelope. The design team opted against using plate girders in this application because of the shallow depth available for the new structure. Additionally, the team had committed to providing at least three additional inches of vertical clearance over the electrified railroad. Still, a Public Utility Commission design exception for substandard vertical railroad clearance was required.

The design scheme mimicked the existing configuration of very shallow, closely spaced stringers; however, the existing structure load distribution was improved by converting the arrangement to a three-span continuous structure. Rolled sections, 12 in. deep, were proposed, except for at the fascia, where deeper rolled sections were required in order to accommodate the size and additional weight of utilities. Additional live load deflection calculations were required to gain PennDOT approval of this design scheme. The design team also had to demonstrate that cambering could be done for such a shallow rolled section held down at four points. Cambering feasibility was verified by local fabricators, who noted that the cold cambering method could be successfully implemented in this case.

Due to the bridge's location in the Chestnut Hill Historic District, additional historic requirements had to be met as well. For instance, rivets were used in the existing barrier ironwork that was removed, so the barriers had to be replaced with an historic-looking bolt. Therefore, acorn nuts were selected as the fasteners on the proposed barrier due to their rounded heads that perpetuate the rivet-head appearance. Historic-looking punch rivets were also used for the handrail; when hammered into place, the rivet head went smooth and expanded the bolt to secure the railing.

The project also included multiple interesting construction challenges, which the City's Construction Unit and the contractor, Loftus Construction Company, worked through together. One of the biggest challenges was constructing a bridge over an active operating railroad. Though only the middle span was over active tracks, the bridge's entire footprint had overlapping right-of-way with SEPTA. Much of the work over the tracks could only occur during track and power outages, which were permitted exclusively at night and lasted just three to four hours on average. Numerous night shift outages were needed to complete the demolition and deck reconstruction phases of the project.

Because the bridge was in very poor condition, contract documents included weight restrictions and equipment placement limitations. This challenged the contractor to devise creative means and methods for the demolition phase, mainly using small equipment. The existing deck and sidewalks were removed almost entirely through the use of demolition saws, hand tools and a single mini-excavator, with constant monitoring of the structure for instability. With demolition taking place at night in a residential area, and considering the poor existing condition, hammering of the existing superstructure was prohibited.

Stairs from street level down to the SEPTA tracks.

Due to the shallow interior beams and presence of utilities in the fascia bays, standard diaphragms could not be used. Therefore, the utility supports functioned as braces between the girders. However, the location of the utility supports fell on the bottom portion of the fascia girder, which left the top portion of the fascia girder unbraced. This caused stability issues for the fascia girder during construction due to the exterior overhang support system. To prevent excessive overturning force on the exterior girder, WT sections were bolted to the top of the interior girder at regular intervals. Double angles were then extended between the WT section and the exterior girder to provide sufficient bracing to allow construction operations.

Given the numerous project constraints, this bridge rehabilitation was only possible with the use of structural steel. This versatile material enabled the use of girders shallow enough to provide an additional three inches of vertical railroad clearance while satisfying the necessary roadway vertical curve sight distances and load-carrying requirements. The use of steel girders, protective barriers and pier columns preserved the historical aesthetics of the bridge as well. Overall, the project was a sound investment in the Chestnut Hill neighborhood. The project appropriately restored a bridge, fitting it properly to the area's historical context, and set the standard for future projects in historic areas. This exercise was lauded as an excellent example of context-sensitive design by the community and critics alike, and the design team helped restore grandeur and functionality to a prominent structure. ■

Owner
City of Philadelphia

General Contractor
Loftus Construction Inc., Cinnaminson, N.J.

Structural Engineers
City of Philadelphia, Department of Streets, Bridge Section
Michael Baker International, Philadelphia

Bridge Architecture
KSK Architects Planners Historians, Inc., Philadelphia

Steel Fabricator
Michelman Steel Enterprises, LLC, Allentown, Pa.

Camber Fabricator
Greiner Industries, Inc., Mount Joy, Pa.

The design scheme mimicked the existing configuration of very shallow, closely spaced stringers.

Numerous beam design runs were performed and compared to the limited available superstructure depth envelope.

conference preview

How quickly can a 425-ton bridge move 92 ft? Pretty quickly, actually.

SLIDE-IN SOLUTION

BY THADDEUS KOSMICKI, PE

ONCE CONSTRUCTED, BRIDGES typically stay in a single location. And when it comes to short-span bridges, their movement is generally limited to live load deflections and the effects of thermal forces.

But that's not always the case. Slide-in bridge construction (SIBC), which involves constructing a bridge in one spot then moving the entire assembly into place, is suitable for some projects. In fact, this tactic was recently employed for a multi-bridge project on Interstate 70 in Columbia, Mo. Three of five total bridges (replacing six existing bridges) were designed and built off-alignment in a temporary location—two of them were used for maintenance of traffic (MOT) in their initial location—and were then laterally slid into their final locations to match the existing alignment.

The team employed the design-build project delivery system to replace the six structurally deficient bridges while maintaining traffic on I-70—more than 80,000 vehicles per day—during construction. Although the lateral bridge slide required a short-term traffic diversion, the public experienced limited inconvenience when compared to the extended traffic impacts associated with traditional phased construction. Further, using SIBC and constructing temporary bypasses allowed construction crews to work uninterrupted and away from the traveling public, ensuring safety for all. The project approach also minimized the amount of closure time of local city streets and improved traffic flow through the entire corridor.

The Missouri Department of Transportation's (MoDOT) decision to use the design-build procurement method allowed the engineer and contractor to work in collaborative environment to detail, design and construct the bridges. Along with developing the bridge sliding procedures, there were several distinct structural elements that required close partnership to facilitate the slide as well as address MoDOT's long-term durability requirements.

The three I-70 bridges employing the SIBC technique were located at the Route 763 (Rangeline), Garth Avenue and Business Loop 70 (West Boulevard) interchanges.

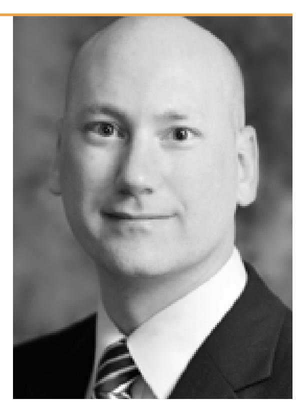
End Bent Considerations

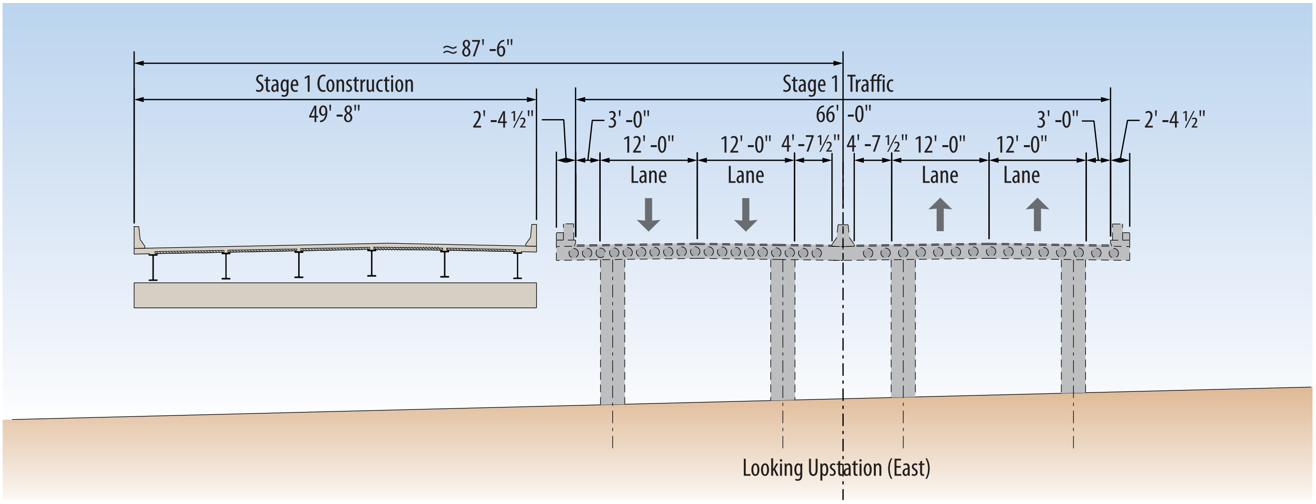
With respect to the substructures for the SIBC bridges, the end bents were designed in a manner that supported the temporary condition, the bridge slide, the final permanent state of the bridge and future widening. Throughout the duration of the slides, the end bent cap experienced a transient loading across the entire length of the cap. The end bent cap design needed to accommodate not only the maximum dead load at any given point, but also any loading attributed to vertically jacking the bridge to install bearings. In addition, the end bent pilings needed to resist the lateral forces transferred through the end bent cap, generated by the hydraulic jack used to slide the bridge into the final position. A concrete anchor block was detailed at the ends of the end bents to provide a structural element to pull or push the bridge, if necessary. An embedded plate in the top of the end bent provided a level surface and a means to restrain the slide bearings and final bearings.

Unique to MoDOT bridges—and vital to the slide—was the incorporation of semi-integral end bents. Providing continuity between the end bent pile caps and superstructure, the semi-integral end bent was the best approach to accommodate sliding the bridge from a temporary location to the final alignment. The solid end diaphragm of the semi-integral end bent provided a large, rigid member to lift the superstructures vertically to install sliding bearings, and it also served as an anchorage point to pull the bridges into their final horizontal alignments. To facilitate vertical jacking and to slide the bridges, additional shear and moment reinforcing, anchorages for high-strength bars and stainless steel shoes (to provide a sliding surface for the lateral move) were all incorporated into the design of the end diaphragms.

In the case of the bridge over Business Loop, the slide itself used two Enerpac hydraulic jacks, which were located at each end bent and attached to the anchor block that pulled the bridge a total of 92 ft to its final location. The total slide time was approximately 11 hours and included the time needed to vertically lift the bridge to install the temporary PTFE (polytetrafluoroethylene) sliding pads as well as the permanent bearing pads once the bridge was in place.

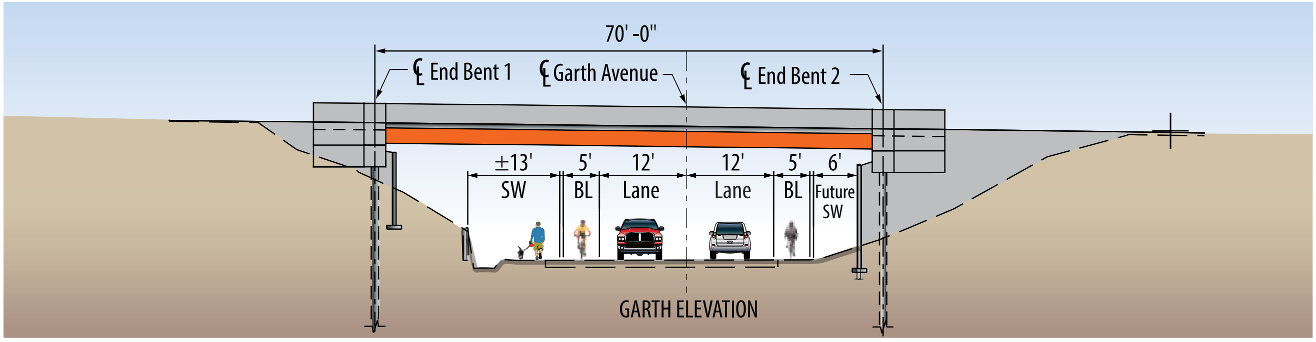
Thaddeus Kosmicki (thad.kosmicki@parsons.com) is a project manager and principal bridge engineer with Parsons and works in Overland Park, Kan. He is engaged in multiple projects throughout Missouri and the U.S.





▲ A typical section of the westbound I-70 bridge over Garth Avenue in the temporary alignment, with existing I-70 bridges shown.

▼ Slide in progress for the westbound I-70 bridge over Rangeline.



The 355-ton westbound I-70 Bridge over Garth Ave. was slid 36 ft.

An aerial view of the I-70 bridge over Business Loop, which replaced two existing bridges.

Weathering the Slide

Not all of the bridges were identical, but the design approach and bridge slides were similar. The superstructures of all of the bridges were comprised of a composite steel and concrete deck system that featured precast concrete deck panels on welded weathering steel plate girders and a partial-depth cast-in-place deck slab. The I-70 bridge over Business Loop, at 89 ft long and 83 ft wide, was the widest and heaviest single-span bridge in the whole project. It replaced two bridges: the east- and westbound structures on I-70. The 11 weathering steel plate girders with 36-in.-deep webs were spaced at 7 ft, 7 in. With a total steel weight of 143 tons, minimizing the dead load of the bridge was key to sliding it effortlessly at approximately 9 ft per hour.

Weathering steel plate girders were selected for the superstructure thanks to the following advantages:

- Economical means to minimize structure depth and improved vertical clearance
- Lightweight structure, which allowed for a faster bridge slide
- Reduced construction costs because erecting the girders only required one crane
- Competitiveness with other methods on a cost basis
- A conventional structure type that provides long-term durability and minimizes necessary maintenance

The entire project—including the I-70 roadway, construction of a "dog bone" roundabout at Route 763/Rangeline and dual-lane roundabouts at the I-70-Business Loop interchange and five new bridges, three of which used SIBC—was completed in 13 months, including design and construction. The combination of SBIC and design-build delivery comes at a most opportune time when our nation's rapidly aging bridges are in urgent need of repair or replacement, and our highways are already congested even before the added strain of road closures. This approach has the potential to be an instrumental part of the solution. ■

*This article is a preview of Session B4 "Long-Span Steel Bridges" at NASCC: The Steel Conference, taking place March 22–24 in San Antonio. Learn more about the conference at **www.aisc.org/nascc**.*

Owner
Missouri Department of Transportation

General Contractor
Emery Sapp and Sons, Inc.

Structural Engineer
Parsons

Steel Fabricator and Detailer
DeLong's, Inc., Jefferson City, Mo. AISC CERTIFIED FABRICATOR

Orthotropic steel deck is a viable option for rehabbing movable bridges working on tight schedules.

ORTHOTROPIC Deck Delivers

BY MARK TORRIE, PE, AND ÉRIC LÉVESQUE, MSc, PEng

AS STATES AND OWNERS consider how to address our nation's infrastructure issues and increase the long-term performance of their assets, their first reaction is typically to consider options in their comfort zone.

However, there are other proven cost-competitive alternatives that can be effective when it comes to re-decking long-span steel bridges. One such option is the use of prefabricated orthotropic steel deck (OSD) panels.

These modular panels employ a lightweight deck to reduce the overall weight of the superstructure and can be designed to sit on existing piers and foundations. The geometry of these deck panels can also be designed to increase the total height of the deck, increasing its moment of inertia in specific areas to reduce deflection. Ultimately, a panel can be installed with a thin, anti-skid wearing surface (with an optional asphalt overcoat) that can provide a service life in excess of 75 years.

The technology was recently employed on a swing bridge in Hastings, Ontario. Built in the early 1950s, the bridge was a key fixture in the community and a major link over the Trent-Severn Waterway. But after years of exposure to de-icing salts and in-service operation, the bridge was nearing the end of its serviceable life and was de-rated based on its condition.

In 2014, Parks Canada Agency, with Public Works and Government Services Canada, retained Associated Engineer-

The new bridge in full-service condition.

The underside of an OSD panel during installation.

The OSD panel is positioned in place prior to bolting the bottom flange of the transverse floor beam to the steel superstructure.

Project Time Line

Fall 2015. The contract is awarded, and fabrication of the steel superstructure begins immediately and continues nonstop on a 24-hour cycle through the end of 2015 and into early 2016. Site work progresses in parallel, as the owner would not allow de-mobilizing the existing bridge until fabrication was sufficiently advanced to ensure the completion date could be met. The fabrication schedule is achieved, and demolition of the existing bridge commences on time.

January 2016. The existing structure is removed, resulting in a 34-mile detour. The contractor immediately starts rehabilitation of the concrete foundations. During the rehabilitation, steel superstructure fabrication is completed for the girders and OSD deck, including all shop welding.

March 2016. The completed assembly is shop-painted and delivered to the site just as rehabilitation work on the existing foundations is completed.

ing, Ltd., to undertake a full inspection, load evaluation, life-cycle analysis, preliminary design and detailed design of a rehabilitation or replacement structure.

The existing span was arranged with a cantilever span of approximately 56 ft and a back counterweight span of 28 ft. The structural system was comprised of two primary steel through-girders, a floor beam system, a central pivot girder, a concrete back-span deck acting as the counterweight and an open-grid deck. The west side of the structure includes

Mark Torrie is a structural engineer with Associated Engineering, and **Éric Lévesque** is a project services manager with Canam-Bridges.

A view of the longitudinal bolted splice between the orthotropic steel deck panels before asphalt was added. The panels were delivered with an anti-skid waterproofing membrane that was preinstalled by the fabricator for durability and worker safety during the wet winter months.

a sidewalk, but an east sidewalk was never installed as it would have conflicted with marine traffic using the canal.

In 2012, Associated Engineering performed an initial site investigation, load evaluation and inspection, which indeed confirmed that the structure was approaching the end of its serviceable life. The existing load posting was confirmed and preliminary design options were evaluated to estimate the rehabilitation cost. The optimal solution called for replacing the superstructure with steel and rehabilitating the substructure, which was anticipated to reduce the overall project schedule by one month.

The original bridge relied on an open-grid deck, a fatal feature that allowed de-icing materials to accumulate on the steel components below, leading to corrosion. To prevent these corrosion issues in the new bridge, the engineer evaluated closed deck solutions to better control the de-icing fluid run-off from the roadway surface. When evaluating a closed-deck solution, the overall weight of the structure is critical, as it impacts the loads imparted on the newly rehabilitated foundation, and both concrete and steel closed deck options were considered, including precast concrete, cast-in-place concrete, concrete-filled open-bar grating and OSD configurations.

Winter Ready

The OSD option was selected, as it was the only one that offered the optimum combination of light weight, durability, stiffness and geometric flexibility. Additionally, for this particular project, one of the biggest advantages of OSD was that the structure could be prefabricated in a temperature-controlled environment, lim-

Updated Weld Penetration Requirements for Rib-to-Deck Welds

Last year, AASHTO voted to modify the weld penetration requirements to 60% minimum penetration and added a requirement for the weld throat to be greater than the rib thickness. The previous requirement of 80% penetration (70% minimum) forced fabricators to employ an expensive balancing act using welding procedures with enough energy to reach the joint root while not burning through the root. This change improves constructability, provides more flexibility and reduces the chances of melt-through or burn-through; it also reduces the potential for hot cracking. The updated weld requirements are based on fatigue test results reported by J.M. Barsom and J.W. Fisher, who demonstrated that if the new requirements are met, fatigue performance was at least equal to the previous 80% requirements. (The study, *Evaluation of Cracking in the Rib-to-Deck Welds of the Bronx-Whitestone Bridge*, will be published in the American Society of Civil Engineers' *Journal of the Structural Division*.)

Even with these improvements, research continues on the rib-to-deck weld. The FHWA recently presented to the AASHTO/NSBA Streel Bridge Collaboration a regression analysis of rib-to-deck welds, correlating weld performance to key parameters such as penetration percentage and fit-up gap. The Collaboration is also developing a proposed experimental evaluation of the weld to establish bounds for fit-up gap tolerance. Each of these studies will further improve understanding of this important weld.

By Duncan Paterson, PE, PhD, Cincinnati Bridge Section Manager, Professional Associate, HDR

A view of the steel superstructure prior to installing the OSD. The transverse floor beams of the OSD were used as a template to drill matching holes in the superstructure above.

iting the amount of winter concrete work while accelerating the delivery schedule.

The waterway is closed to traffic for four months in winter, so all construction to replace the superstructure and to rehabilitate the substructure had to take place within this tight window. This limitation required that the steel superstructure, including the deck, be fabricated, delivered to the site and installed as soon as the substructure work was completed. Movable bridges always require tight tolerances to work effectively, and the steel OSD solution allowed an extremely high level of geometric precision to fit the new structure to the existing roadway profile.

The new superstructure configuration consists of primary girders, floor beams, a central pivot pier, a steel counterweight, OSD, an asphalt wearing surface and a pedestrian sidewalk on the west side of the structure. The new movable swing bridge is nearly 84 ft long and 27 ft wide and has a surface area 2,200 sq. ft.

This configuration posed a challenge because the new superstructure geometry had to fit on existing foundations, resulting in a structure with a center of gravity that was longitudinally and transversely offset from the geometric center of the structure. Based on their past experience, the designers knew that the OSD offered the best solution to address this issue. Thus, the team committed to integrating the OSD into the design from day one, and models and all connection and interface details between the OSD and steel were provided in the initial design stages. Performance specifications for the OSD were developed and included in the final contract specifications; the OSD components were designed to act compositely with the structure's floor beam system both in the open and closed position. The welding specification for the rib-to-deck plate welds required a practical and achievable partial joint penetration. According to the AASHTO *LRFD Bridge Design Specifications* (2012 Edition), a target penetration of 80%—with 70% minimum—was respected (see sidebar at left for more). The deck plate and closed rib details were specified to be designed by the fabricator to ensure economy, stiffness, fatigue life, fabrication quality and system durability.

Splendid Splice

The OSD layout and design were chosen with shop fabrication, shipping and erection issues in mind. With input from the fabricator, this OSD design completely eliminated on-site welding of the deck plate by incorporating a longitudinal bolted joint splice. By bolting this splice instead of welding it, the contractor saved valuable time by limiting delays related to winter conditions that would prevent welding.

The bridge components and steel counterweight units were field-bolted together, then the OSD, with a shop-applied high-performance waterproofing membrane, was field-bolted to the steel superstructure. Next, the assembled unit was swung into place and paved before the contractor performed the final balancing and commissioning. Vehicle traffic returned to the roadway almost a month ahead of schedule, and vessel passage was achieved in time for the opening of boating season on the Trent-Severn system.

The project highlights a successful implementation of an OSD design for a surprisingly complex structure with a compressed winter construction schedule. A large portion of its success is due to the contractor's and fabricator's involvement in the preliminary and final design of the OSD and final pre-shop assembly of the steel superstructure and OSD to confirm geometric acceptance prior to delivery to the site. ■

*Visit **https://youtu.be/R3kgSJV2Xr8** for a short video on the installation of the Hastings Swing Bridge.*

MAKING a Signature Connection

BY NATALIE McCOMBS, SE, PE, AND SARAH LARSON, PE

An attractive steel arch and sophisticated engineering define Little Rock's new Broadway Bridge.

IN 2010, LITTLE ROCK'S BROADWAY BRIDGE was showing its age.

Completed in 1923, the bridge had served the region well as a landmark structure dedicated to veterans of World War I and was a vital connector over the Arkansas River between downtown Little Rock and North Little Rock, Ark. The existing structure consisted of one steel arch span, three concrete deck arch spans and multiple concrete beam spans. However, by 2010 the bridge was carrying 21,000 vehicles per day, becoming costlier to maintain, and was considered structurally deficient.

Steel would play a vital role in the replacement bridge, which is designed to accommodate 34,000 vehicles daily. The new bridge features four 11-ft lanes with 4-ft shoulders and a 16-ft-wide shared-use path to increase access for pedestrian and bicycle users. These needs were addressed with two new bridge-to-ground access ramps—a mix of pedestrian bridges and paved paths supported by mechanically stabilized earth retaining walls that connected to the Arkansas River Trail System. While the $98.4 million project was administered by the Arkansas State Highway and Transportation Department (AHTD), it was made possible by a $20 million contribution from Pulaski County, as county officials wanted to establish the new bridge as an icon for the community.

Natalie McCombs (nmccombs@hntb.com) is senior technical advisor and **Sarah Larson** (sjlarson@hntb.com) is a bridge design engineer, both with HNTB.

Critical Constraints

When determining the structure type and erection method for the main spans, the plan needed to accommodate the limited construction and storage space on the site, as the bridge spans a navigable waterway designated as a connector for the Marine Highway M-40. Due to U.S. Coast Guard requirements, the waterway had to remain open during the construction phase, and closure windows were therefore limited.

Once the location limitations were established, the team determined the span configuration. The bridge would be made up of 440-ft tied arches for the two main arch spans (designed by HNTB) and welded steel plate girder approaches (designed by Garver, LLC, the prime design consultant for the project).

The initial design criteria included provisions to accommodate a future trolley on the bridge, using minimally invasive work with light-duty construction equipment. While this was a relatively simple concept on the plate girder approaches, applying the same concept to the arch spans was more complicated due to the transverse floor beam and longitudinal stringer framing layout. The localized slab depth required to accommodate the trolley was nearly 16 in. thick, which would require an undesirable large haunch or make the floor beams noncomposite with the deck. The team initially decided that the longitudinal stringers would sit on top of the transverse floor beams, making them noncomposite. In addition, the look of the inclined basket-handled arches that were touching at the peak appealed to stakeholders. To meet vertical

HNTB

The new bridge is designed to accommodate 34,000 vehicles a day via four 11-ft-wide lanes with 4-ft-wide shoulders.

clearance on the roadway and have the arch ribs touch at the peak, the initial geometry of the arch ribs was set to tilt inward at a 25° angle from vertical.

However, in order to keep the project within budget, HNTB reevaluated the design criteria for the arch spans to determine the most effective design. Major revisions involved accommodating the future trolley system and increasing the arch rib spacing at the peak from touching to approximately 21 ft. These two modifications changed the angle of the arch ribs from 25° to 18°. These changes allowed for shorter composite floor beams (reduced from 99 ft to 88 ft) and a framed-in stringer floor beam system, efforts that saved approximately 2,500 tons of structural steel as compared to the original plan; total tonnage for the arch spans is 4,100.

Building a Better Bridge

Throughout design and construction, great care was taken to observe the U.S. Federal Highway Administration's strict guidelines for fracture-critical members.

Greg Davis

▲ The arch span uses 4,100 tons of steel in all.

▼ The tie girder cross section consists of a closed parallelogram box girder, made up of two inclined webs and two horizontal flanges.

HNTB

The bridge was made with ASTM A709 Grade 50 steel, which includes the Charpy V-notch Zone 3 requirements for increased toughness. This was important for the tie girder, floor beams and hanger plates, as they are all considered fracture-critical members. For the tie girder, the cross section consists of a closed parallelogram box girder made up of two inclined webs and two horizontal flanges. The web plates are welded to tab plates with a double-fillet weld and are then bolted to the flanges. This bolted connection isolates a potential fracture of one plate without allowing the fracture to propagate throughout the cross section. The resulting three-sided tie girder section was designed to carry the structural demands at an extreme event limit state, and this internal redundancy eliminates the potential of a catastrophic structural failure.

The hangers are 2⅜-in.-diameter ASTM A586 bridge strand and are made with Grade 2 wire as opposed to Grade 1, and a Class C coating was applied to all wires in the strand to enhance durability. The strength of a hanger using Grade 1 wire with Class A coating is 344 tons, while Grade 2 wire with Class C coating throughout provides a capacity of 357 tons and was tested to failure at 395 tons.

A key element to consider in the tied arch design is the elongation of the tie girder that occurs when load is placed on the arch structure. When the slab load is applied, elongation occurs in the tie girder and forces the string-

➤ The project was let in September 2014 and substantially completed this past June.

ers to move along with it. Since the stringers are smaller in area, the stress that would be put on them would be high compared to that of the tie girder. HNTB alleviated this dead load axial stress by using slots in alternating stringer-to-floor beam connections. This allowed for movement to occur, with most of the weight of the slab in place. Closure pours at every other floor beam allowed bolts to be tightened once most the deck concrete was in place. To complete the riding surface, the closure pours were placed after bolts in the stringer-to-floor beam connections were tightened.

Rapid Reconstruction

The construction contract allowed the bridge to be closed to traffic for up to six months during the construction period, and AHTD officials employed an incentive bidding approach incorporating a rate of $80,000 for each day the bridge would be closed. New piers were built under the existing bridge (while traffic was maintained on it) while tied arch spans were simultaneously assembled on barges just downstream of the existing bridge. The existing bridge was then closed to traffic and demolished, the new arch spans were floated into place and the deck was poured. Most of the approach girders were delivered the day they were to be set so they could be picked directly from the truck.

With the new bridge built on the existing alignment, Massman constructed the foundations under the existing bridge prior to the closure period in order to minimize closure time. Using a tied arch superstructure also minimized impact to the traveling public and allowed for rapid reconstruction, as this structure type is stable and can be assembled off-line and transported to the job site via barges.

The float-in of the arches posed a significant risk to the contractor. Because the arches were designed to have a certain amount of free-board, they couldn't be floated in if the water surface was too high or too low. There had to be at a "sweet spot" of sorts. If the surface is too high, water velocity and lowering becomes a concern, and with increased water velocity it becomes too difficult to position the spans; the tug boats must fight the water current to get the arch spans set within a +/-1-in. tolerance. And if the water is too low, there isn't enough freeboard to float over the new piers.

The construction team assessed the river conditions for the right windows, and the first span was floated in in mid-November, followed by the second span in early December 2016. In the end, the crossing only had to be closed for five months instead of the allotted six, and a new steel icon now graces Arkansas' capital city. ■


HNTB


AHTD

▲ The arch ribs are welded plates resembling an H-shape that is 4 ft wide by 3 ft, 10 in. deep. The tie girders are 4 ft wide, 5 ft deep and about 73 ft long. The floor beams are I-shaped welded plate girders that are 5 ft deep at the tie girder and increase in depth to match the profile grade. The final floor beam lengths are approximately 88 ft and the beams are spaced at 36 ft, 8 in.

Owner
Arkansas State Highway and Transportation Department

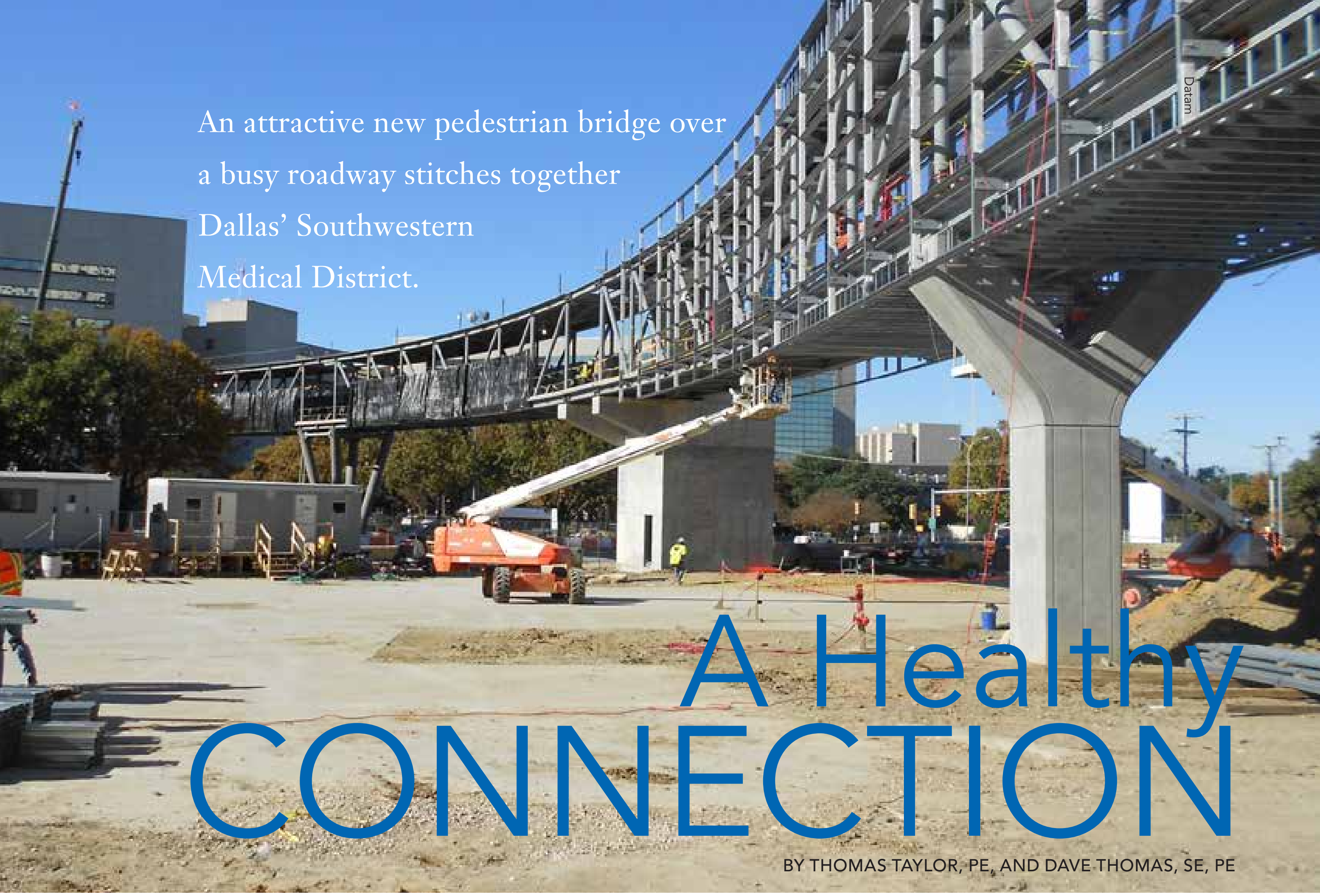
General Contractor
Massman Construction Company, Kansas City

Structural Engineers
Garver, LLC, North Little Rock, Ark.
HNTB, Kansas City

Steel Team

Fabricators
Veritas Steel, Palatka, Fla. (Arch Spans) AISC CERTIFIED FABRICATOR
DeLong's, Inc., Jefferson City, Mo. (South Approach) AISC CERTIFIED FABRICATOR
W&W|AFCO Steel, Little Rock (North Approach) AISC CERTIFIED FABRICATOR

Detailers
Tensor Engineering, Indian Harbour Beach, Fla. (Arch Spans) AISC DETAILER
DeLong's, Inc. (South Approach)
ABS Structural Corporation, Melbourne, Fla. (North Approach) AISC DETAILER



An attractive new pedestrian bridge over a busy roadway stitches together Dallas' Southwestern Medical District.

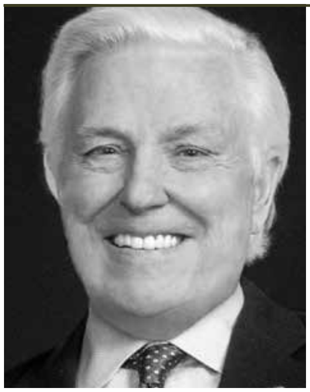
A Healthy CONNECTION

BY THOMAS TAYLOR, PE, AND DAVE THOMAS, SE, PE

THE SOUTHWESTERN MEDICAL DISTRICT— which is actually northwest of downtown Dallas—hosts nearly 2.6 million patient visits per year.

Home to medical facilities including Parkland Memorial Hospital, Children's Medical Center of Dallas and the vast University of Texas Southwestern Medical Center complex, the district grew even larger in the last couple of years with Parkland's addition of a new 2.5-million-sq.-ft, 862-bed hospital.

The new building is located across the busy six-lane Harry Hines Boulevard from the existing facilities. That's where a new 830-ft-long curving pedestrian bridge comes in. The high-visibility location mandated attractive structure that would contribute to the overall architectural expression of the hospital—but as a public hospital, an economical solution was essential and always the primary driving criteria. Structural steel truss framing was chosen as the solution to achieve both goals.



Thomas Taylor is principal design engineer and **Dave Thomas** is an associate, both with Datum Gojer Engineers.

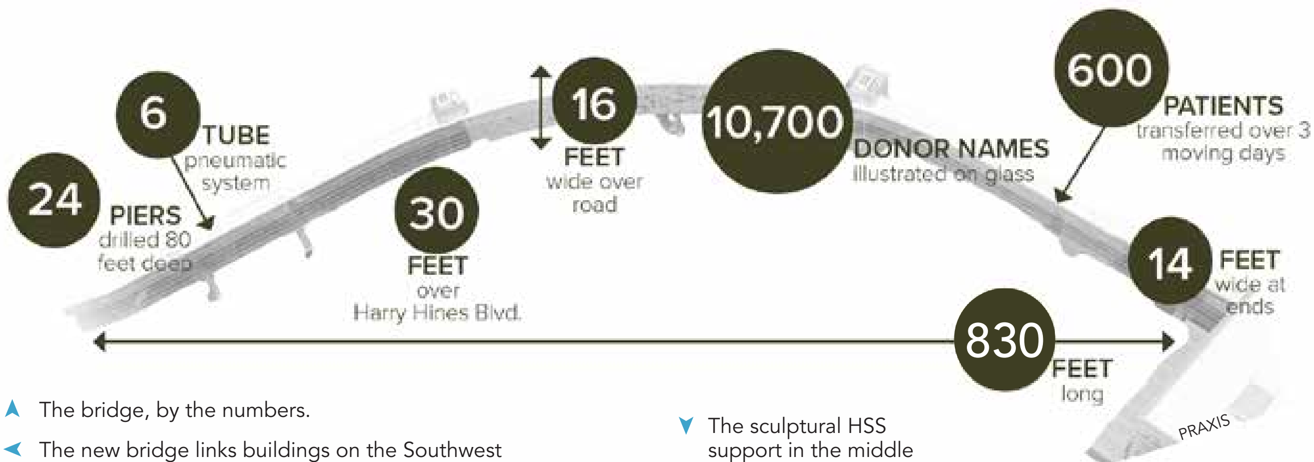
Due to a tight budget and the need to build over a busy roadway, both the fabricator and erector, Irwin Steel and Bosworth Steel Erectors, were added to the team early in the process to advise structural engineer Datum Gojer Engineers on economics and constructability. During this process, the team determined that the most economical construction approach would be to erect the bridge in segments. The original concept was to ship the segments as fully fabricated boxes, but this wasn't possible due to the size of the boxes and the constraints along the delivery path. There are a total of 15 segments of varying lengths, with 60 ft being the most typical. All of the vertical and diagonal truss members are 8-in. round hollow structural sections (HSS) with varying wall thicknesses, providing a uniform appearance along the length of the bridge. The top and bottom chords of the trusses are W14 members.

The floor and roof structures were also trussed. While the floor slab would have laterally braced the floor against wind forces once poured, the structure was exposed to torsional stresses due to the curved sections and lateral erection and wind forces during construction. The horizontal trusses in the floor and roof provided added stiffness during construction and contributed to resisting the torsional stresses in the curved sections.

The solution was to prefabricate the wall truss sections of each bridge segment in the plant and deliver them to the site to

The completed bridge.

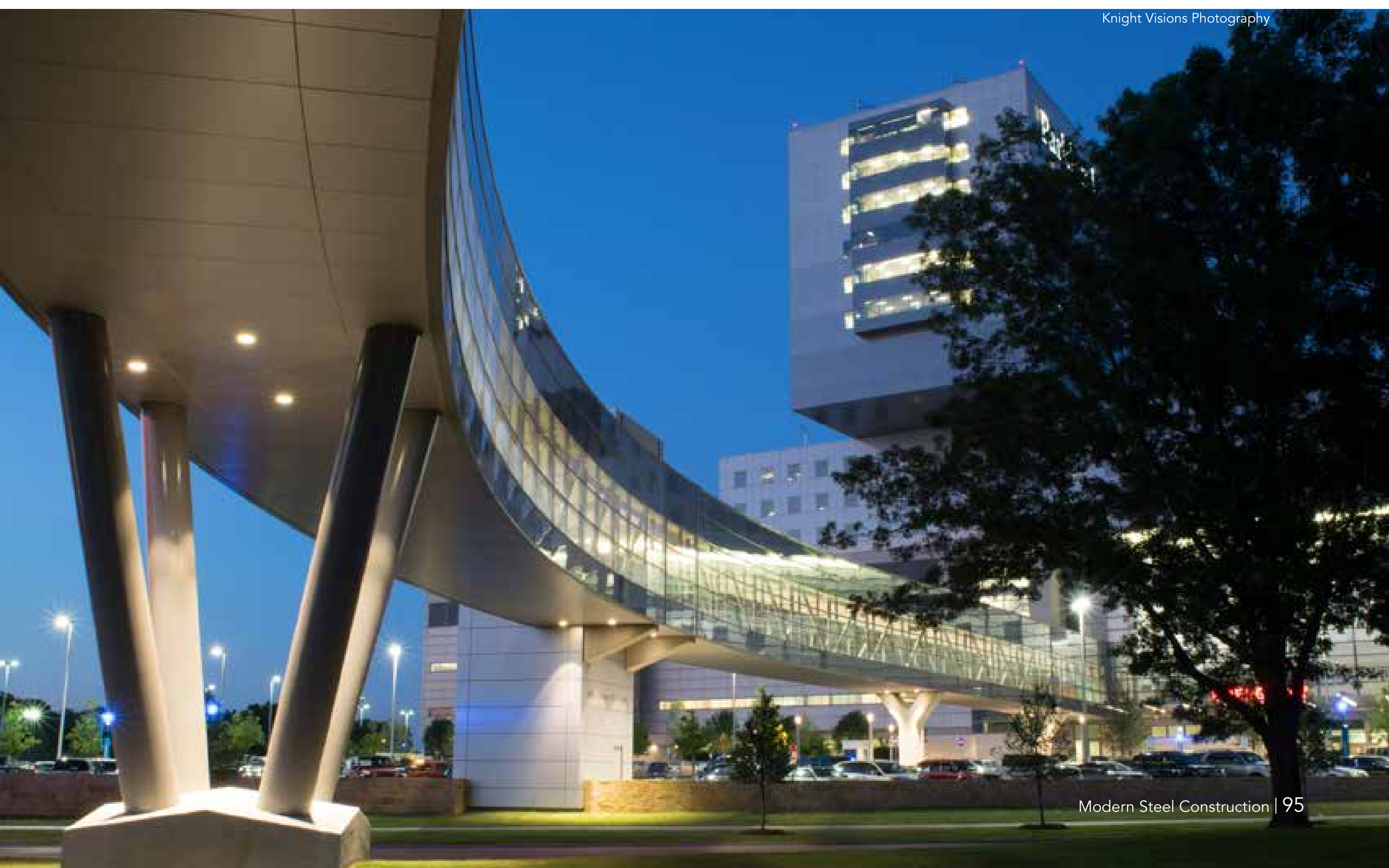
Installing one of the truss segments.



The bridge, by the numbers.

The new bridge links buildings on the Southwest Medical District campus near downtown Dallas.

The sculptural HSS support in the middle of Harry Hines Boulevard.



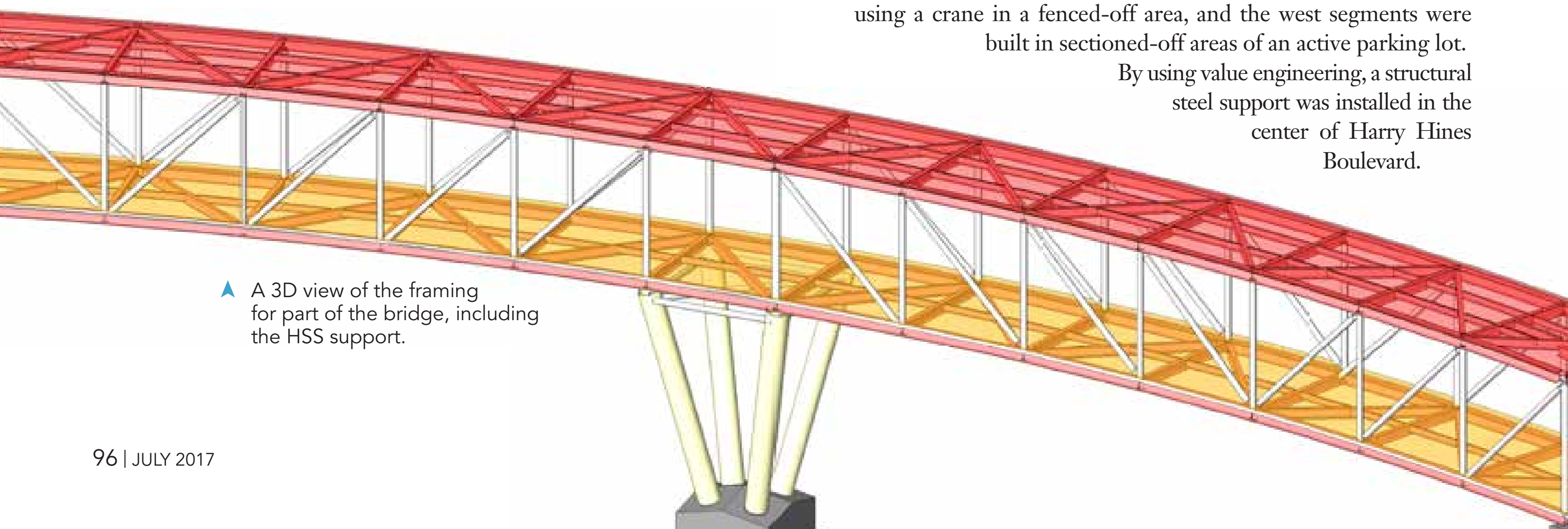
Two truss segments.

Interior glass and exposed trusses.

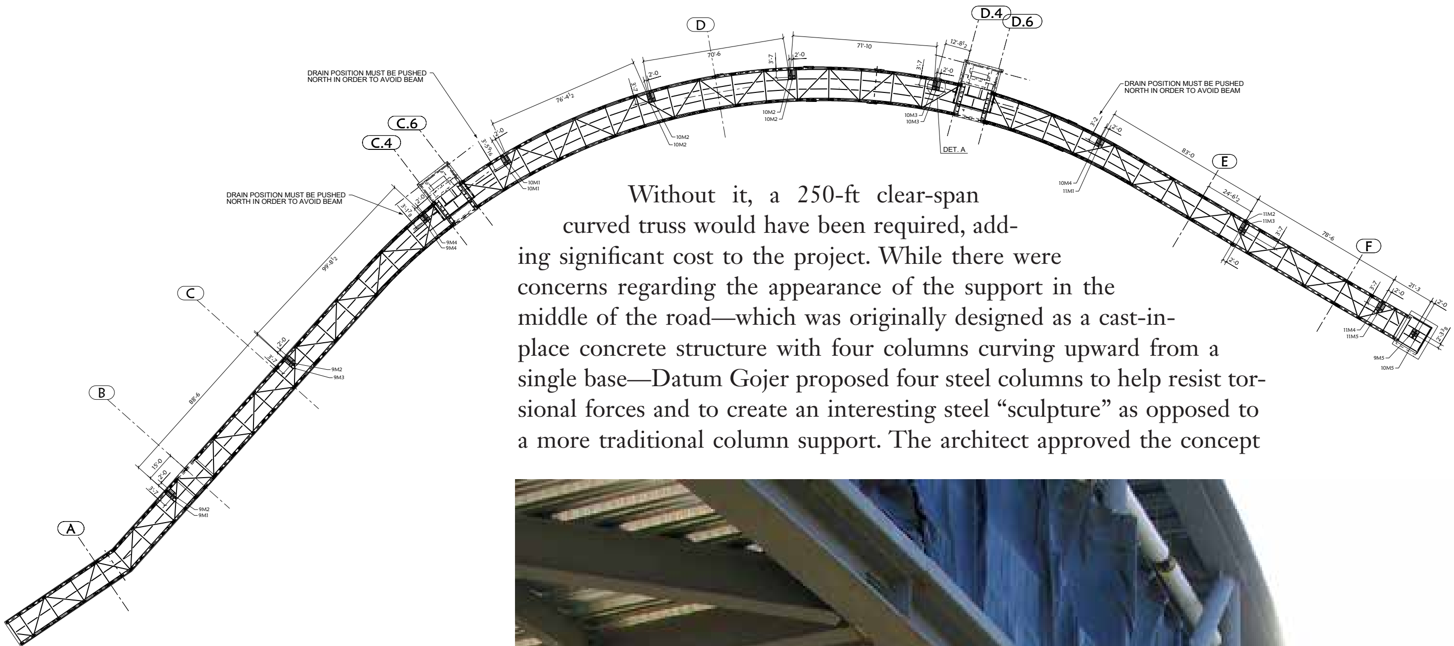
be assembled. The new hospital building was still under construction, but General Contractor Azteca worked with Bosworth to locate an area, intended for future parking, that was close to the bridge site and could be used for the assembly process. Irwin fabricated and shipped the wall trusses to the site, then connected the floor and roof elements to the trusses in this parking lot. Once a segment was completed, Bosworth lifted it into place. Traffic on Harry Hines could not be affected during peak periods, so erection times were closely coordinated with the City of Dallas and the Texas State Highway Department. Bridge segment placement required a full northbound closure/detour over one weekend and a full southbound closure/detour over another weekend. Each time, the work began on a Saturday afternoon and was completed by Sunday afternoon.

Erection was broken into three phases: east, center and west. The east phase was on the new hospital side where the future parking lot could be used to assemble three segments with a crawler crane with minimal restrictions. The center segment was built in the median between the northbound and southbound lanes of Harry Hines using a crane in a fenced-off area, and the west segments were built in sectioned-off areas of an active parking lot.

By using value engineering, a structural steel support was installed in the center of Harry Hines Boulevard.



A 3D view of the framing for part of the bridge, including the HSS support.



Without it, a 250-ft clear-span curved truss would have been required, adding significant cost to the project. While there were concerns regarding the appearance of the support in the middle of the road—which was originally designed as a cast-in-place concrete structure with four columns curving upward from a single base—Datum Gojer proposed four steel columns to help resist torsional forces and to create an interesting steel "sculpture" as opposed to a more traditional column support. The architect approved the concept

The bridge is made of 15 segments of varying lengths, with 60 ft being the most typical length.

Construction Notes

The complexity of installing the trusses over a the major thoroughfare required street closures and constant mindfulness of the traffic flow below. Prior to mobilizing, an investigation of the street and substrate was performed to verify that both were able to support the cranes and loads. From there, we developed a detailed lifting plan.

Safety cables were required on the truss sections prior to and during erection. After this process, the crane was assembled at a nearby laydown area and moved into place after all traffic controls were implemented. Once the crane was secured, the trusses were moved into position, the rigging was completed and the trusses were lifted into place during nighttime hours on preapproved weekends. The trusses remained under load until reaching their final location with permanent connections made and the area secured. At that time, the crane released the load, equipment was demobilized and the street was reopened to traffic.

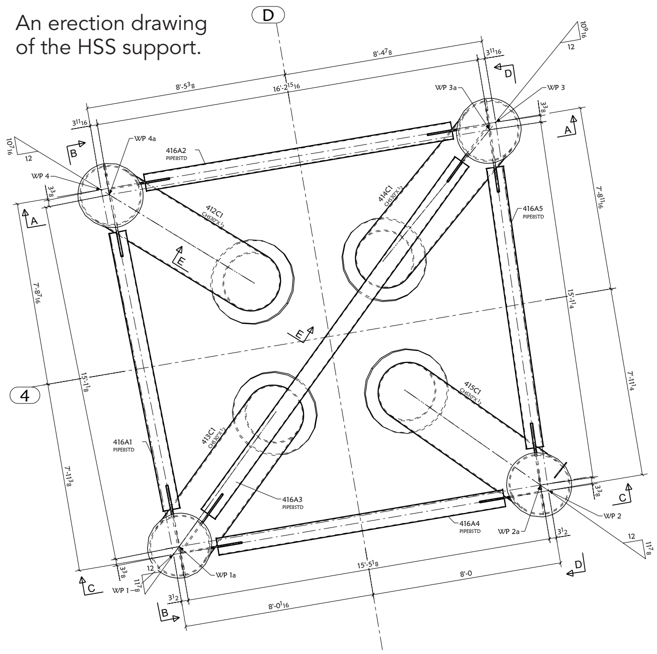
— Contributed by Azteca

Datam

Erection was broken into three phases: east, center and west.

A segment awaiting erection.

An erection drawing of the HSS support.



Erecting one segment during the day...

...and another at night.

▲ An overhead view of the bridge spanning over Harry Hines Boulevard.

▼ An erection drawing of the HSS support.

from an aesthetic standpoint, it was within budget and it solved structural issues in a visually pleasing way. It was also a more constructable option.

By teaming up and conceiving the most economical structure and construction process, the handsome architectural concept was built for $13 million, well under the initial budget of $20 million (the total cost of the hospital project was $1.2 billion). In addition, substantial completion was accomplished two months ahead of the contract date, and the final certificate of occupancy was received weeks ahead of the hospital's planned opening.

The day it opened, patients were transferred from the old hospital into the new one. Donor names, etched into tree patterns, adorn the glass walls of the bridge and mirror tree patterns on the new hospital. The bridge was so well received that one donor even contributed the entire cost of the bridge, which is now named the Mike A. Myers Sky Bridge. ■

Owner
Parkland Health and Hospital System, Dallas

General Contractor
Azteca Enterprises, Dallas

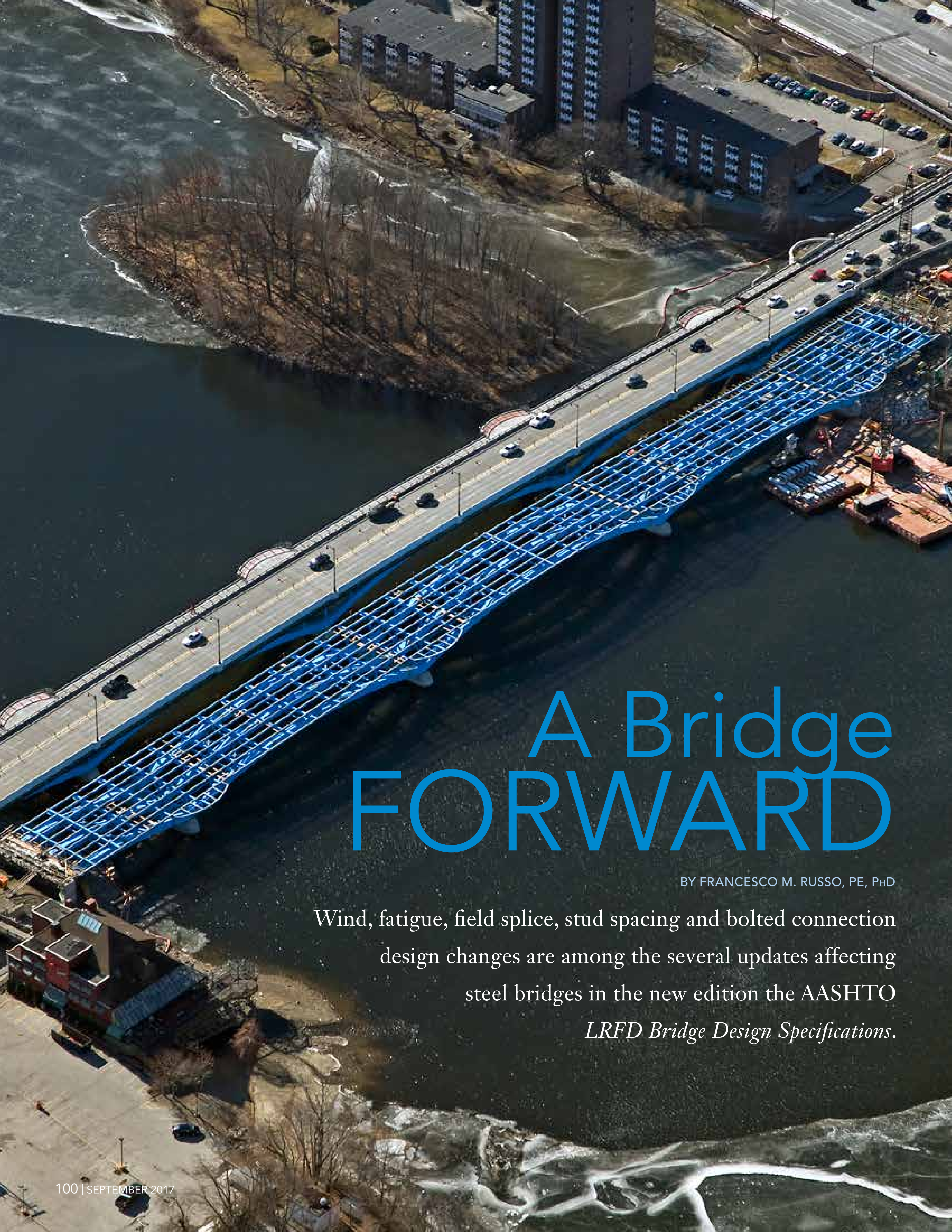
Architects
Moody Nolan, Dallas

Structural Engineer
Datum Gojer Engineers, Dallas

Steel Team

Fabricator
Irwin Steel, Justin, Texas (AISC Certified Fabricator)

Erector
Bosworth Steel Erectors, Inc., Dallas (AISC Associate, AISC Certified Erector)



A Bridge FORWARD

BY FRANCESCO M. RUSSO, PE, PhD

Wind, fatigue, field splice, stud spacing and bolted connection design changes are among the several updates affecting steel bridges in the new edition the AASHTO *LRFD Bridge Design Specifications*.

THE 8TH EDITION of the AASHTO *LRFD Bridge Design Specifications* introduces a number of changes affecting steel bridges.

The majority of these changes appear in Chapter 3 – Loads and Load Factors and Chapter 6 – Steel Structures. In addition, a new AASHTO guide specification, *Guide Specifications For Wind Loads On Bridges During Construction*, introduces tools to evaluate the effects of wind loads on bridges of all types under construction. Here, we'll cover some important changes in the new AASHTO *LRFD Specifications* as well as the new *Guide Specifications* and how they apply to steel bridge design.

Chapter 3

Let's begin with Chapter 3 of the *LRFD Specifications*. A significant change in this chapter affecting steel structures is the introduction of new Fatigue I and II Limit State load factors. The load factors that have been commonly used through the 7th Edition *Specifications*—1.5 for Fatigue I and 0.75 for Fatigue II—are based on prior research on effective truck weights and experimental testing of steel structures. Historically, it has been assumed that the 1.5 and 0.75 load factors were sufficient to represent the effects of maximum and effective fatigue loading. It was also believed that only a single truck in a single lane contributed to the stress range. There were also assumptions of how many cycles of stress were produced by the passage of a truck for simple spans, continuous spans, cantilever structures, floor beams, etc. These rules had not been examined in several decades. As a result, the Transportation Research Board sponsored Project R19B as part of the SHRP2 program and one of the goals of the project was to assess and calibrate the fatigue limit state.

The R19B team, led by Modjeski and Masters, collected weigh-in-motion (WIM) data from around the country in order to quantify actual truck axle weights and spacing. Using approximately 8.7 million records, they were able to simulate the ranges of bending moments in a family of simple- and two-span continuous bridges, and they were able to compare those to the moments produced by the AASHTO fatigue design loading: a three-axle vehicle with a gross weight of 72 kips. (Note that this work specifically focused on moments, a value relatable to stress range, and not simply truck weight.) Prior fatigue studies have generally been based on vehicle weight, but it is obvious that weight is only one factor that, along with axle spacing and relative axle loading, produces the stress range.

Using the statistics of the WIM data, the R19B team was able to determine the effective truck moments using Miner's rule, the probability-based maximum moments and the appropriate load factors for each limit state. Although the R91B project initially recommended load factors of 2.0 for the Fatigue I Limit State and 0.8 for the Fatigue II Limit State, further examination of the data resulted in AASHTO adopting new load factors as follows: a Fatigue I Load Factor of 1.75 and a Fatigue II Load Factor of 0.8. Both are clearly larger than the current practice. Also, note the historic relationship of 2:1 between the Fatigue I and II load factors is no longer valid. This is due to a growing number of vehicles that produce large bending moments in relationship to the effective value. The relationship between the Fatigue I and II load factors is now approximately 2.2—i.e., 1.75/0.8. These changes only affect the loading aspects of fatigue design; the resistances of the various details have not changed as a result of this work.

Other aspects of the calibration of the Fatigue Limit State included determining if a single truck in a single lane is still a valid design approach, as well as determining if the cycles-per-passage table in AASHTO is still applicable. The R19B project confirmed that it is still valid, based on the WIM data, to assume that a single truck in a single lane is the proper loading to produce the design stress range. Although there are occasional passages of trucks in adjacent lanes, it is rare that they are fully correlated in terms of passing time and force effects such that a multi-lane effect needs to be considered. The study also evaluated the AASHTO cycles-per-passage approach and recommended some simplifications. For longitudinal members such as rolled beams or plate girders in a multi-beam cross section, the new recommendations for cycles per passage are as follows:

Table 1: Cycles per Passage for Longitudinal Members

Longitudinal Members		*N*
Simple-span girders		1.0
Continuous girders	Near interior support	1.5
	Elsewhere	1.0

This approach removes the distinction of bridges with spans under and over 40 ft. Recommendations for cantilever spans and floor beams are also found in AASHTO in the revised table.

Chapter 6

Numerous changes to Chapter 6 were also introduced in the new *Specifications*. Some of these are major changes in practice, such as new bolted field splice provisions, new design approaches for compression members and changes in shear stud spacing that will facilitate the use of precast deck panels. Other changes in detailing skewed bridges, longitudinal stiffeners and connection plates and editorial changes to various bolt design provisions (to reflect changes in ASTM designations) are also discussed.

Bolted Field Splices. A major change in the design procedure for bolted field splices was adopted in the new edition, greatly simplifying the design approach. The approach in the 7th Edition, stemming from work to rationally address bolted

Frank Russo (frusso@mbakerintl.com) is associate vice president and senior technical manager – bridges with Michael Baker International. The author would like to acknowledge **Michael Grubb, PE**, of M.A. Grubb and Associates, LLC, who is responsible for creating the many ballot items for AASHTO T-14 each year and is also an AISC consultant.

splices in composite members, has been around for nearly twenty years. Though deemed safe, it was also perceived by some as complex and lacking in clarity. The new approach described below results in similar or slightly larger flange splices with a general lowering of the number of web design bolts and is a substantially simpler process.

To determine whether a new method of splice design could be advanced, a task force was formed, working on behalf of AASHTO T-14, to develop a new design approach for flexural splices. This task force consisted primarily of Michael A. Grubb of M.A. Grubb and Associates, Karl Frank of Hirschfeld Industries, Justin Ocel of the FHWA and the author. The work resulted in a simple approach that requires the engineer to design the splice as follows:

- Provide a web splice to develop the factored shear resistance of the web
- Provide a flange splice that develops the factored strength of the smaller of two abutting flanges at a splice

In following these two simple rules, the capacity of the web in shear is fully developed across the splice as is the capacity of the smaller of each of the abutting top and bottom flanges. If a model that includes only the axial capacity of the flanges is sufficient to resist the factored moments at the point of splice, the design is deemed sufficient. This is demonstrated in Figure 1. This model determines if the capacity of the flanges alone is sufficient to carry the design moments—i.e., there is no need for the web to carry any moment.

Note that there is no longer a requirement for the flexural capacity of the splice to be a function of the strength of the section. The splice must be capable of resisting the factored moments at the point of the splice after proportioning the web and flange as described above. This is a significant change in philosophy in the *Specifications*. The new premise is that if the web is fully spliced for the shear strength of the section and the flange is fully spliced for the capacity of the flanges, those two requirements bound the possible limits for each component. If the moment resistance provided by the flange couple shown in Figure 1 is insufficient to resist the factored moments at the point of splice, an additional horizontal force, H_w, is added to the web as illustrated in Figure 2.

The additional horizontal force added to the web is that required for the design moments to be resisted. The horizontal force is vectorially added to the vertical force on the web splice for purposes of checking the web bolts.

Many splice designs were performed using the 7th Edition and proposed 8th Edition provisions. These splice calculations covered girder spacing from 7.5 ft to 12 ft. and three-span bridges with center spans ranging from 150 ft to 300 ft. There were some instances in which the 8th Edition provisions produced a substantial decrease in the number of web bolts due to the omission of a required moment to be carried by the web. In order to assess if this was a concern with regard to overall performance, a series of nonlinear finite element analyses including nonlinear bolt shear force distribution models were performed. The analyses were conducted on a bolted splice in an approximately 109-in.-deep plate girder to assess the expected safety of these new splices with fewer bolts. The results of the modeling indicated that the forces were easily accommodated in these smaller bolt patterns.

Coinciding with the introduction of this new design approach, AISC has published an annotated design example and an accompanying design spreadsheet (visit **www.steelbridges.org/nsbasplice** to access these resources).

Axial Strength of Compression Members. The provisions for compression member strength have been simplified and reorganized in the 8th Edition. They are similar to the approaches used by AISI and AISC for members with and without slender compression elements. The 7th Edition approach implements the "Q factor" reductions for slender elements and combines slender and non-slender compression members in Article 6.9.4.1.1. Specifically, Table 6.9.4.1.1-1 includes two parallel columns, one in which only "column buckling modes" are applicable—i.e., Q=1—and one for which a blended effect of column buckling and local buckling interact—i.e., Q<1. The 8th Edition does away with the Q factor blending of local and column buckling and instead relies on the unified effective width concept for the treatment of local buckling of slender sections in a revised Article 6.9.4.2 and accompanying sub-articles.

Compression member strength is now treated with a simpler two-step process for members with and without slender compression elements. In the first step, the axial compression strength of the gross section is defined as $P_{cr} = F_{cr}A_g$ where F_{cr}

Figure 1. Positive moment flexural resistance based on flange capacity alone.

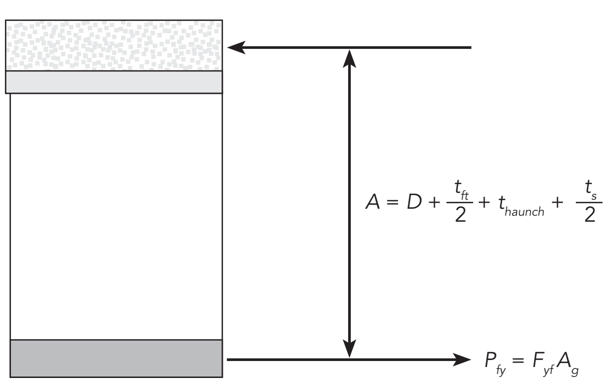
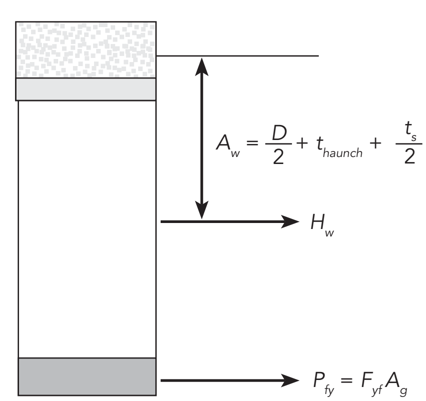


Figure 2. Positive moment flexural resistance relying on a web contribution.



The 8th Edition of the *LRFD Specifications* includes changes in shear stud spacing that will facilitate the use of precast deck panels.

is related to the limit states of flexural, torsional and flexural-torsional buckling of the gross section, assuming local buckling is precluded. For a member with non-slender elements—i.e., *b/t* and *D/t* limits that satisfy non-slender limits of AASHTO 6.9.4.2.1—only the member stability limits apply. Nevertheless, nearly all compression members have their capacity limited by overall member slenderness to some stress, F_{cr}, less than F_y. Thus compression members with and without slender elements are likely to have their capacities limited to less than F_y regardless of the local slenderness.

For a member containing slender elements, the capacity of the section is defined in Section 6.9.4.2.2, but the element slenderness need not be checked against a limit based on F_y; rather its slenderness need only be sufficient to be stable to a level of stress, F_{cr}, that corresponds to the member stability limits. This is a change in prior practice and a substantial benefit in the computed strength for slender elements. Implementation of these unified effective width provisions is an essential part of ongoing work that will replace the current LRFD non-composite box member provisions in the next few years.

Maximum Shear Stud Spacing. Over the course of several research projects, researchers at the University of Texas, George Washington University, the University of Arkansas and the FHWA Turner Fairbanks Laboratory have investigated the maximum shear stud spacing used for composite construction. The 24-in. limit in LRFD is historically linked to work completed by Newmark in the 1940s, which concluded that a 24 in. limit seemed reasonable. With a greater interest in precast concrete deck panels as a means of accelerated bridge construction (ABC), the 24-in. limit has become a constraint. The results of FHWA's tests on steel beams made composite with precast deck panels with pockets spaced 12 in., 24 in., 36 in. and 48 in. on center showed no discernable difference in the moment vs. deflection response of the specimens. All tests were carried out on 24-in.-deep beams.

The George Washington University tests yielded similar results. As a result, the spacing limit has been relaxed. The new provisions of Article 6.10.10.1.2 allow for shear studs to be placed up to 48 in. on center for beam depths of 24 in. or greater. For beams shallower than 24 in., the current 24-in. spacing limit is retained since that limit is consistent with test results from prior researchers.

Steel Detailing for Fit. Continuing with the incremental introduction of fit and detailing considerations into LRFD, various definitions have been added describing terms, such as no load fit (NLF), steel dead load fit (SDLF) and total dead load fit (TDLF) and other terms related to fit, girder, diaphragm and cross-frame detailing. The designer's attention is drawn to the impact of staged construction on girder deflection and fit via changes to Article 6.7.2. One of the more important changes is that Article 6.7.2 now defines a series of conditions for which the contract documents are required to stipulate the anticipated fit condition. Combinations of skew, span length and girder radius are provided for which the fit condition must be provided on the plans. A detailed commentary is provided as is a method to reduce the cross-frame design forces for structures in which a total dead load fit is chosen.

A brief summary and a more comprehensive document addressing the various aspects of girder fit in straight, straight-skewed and curved steel girder bridges can be found at **www.steelbridges.org**.

Gary Prinz

John Rodems

Maximum shear stud spacing has been the subject of several recent research projects.

Cross-Frame Forces in Skewed Bridges. In the new commentary to Article 6.7.4.2, the effects of skew are further explored with respect to the placement of cross frames in highly skewed structures. The commentary builds on recent research conducted at Georgia Tech on the forces in skewed steel bridges. The commentary describes a practice of omitting cross frames near highly skewed corners, staggering cross frames in straight bridges so as to minimize the stiffness of the bridge along transverse lines and providing a recommended offset of the first cross frame from a skewed support in highly skewed structures (Figure 3 provides an example). Note that every other cross frame in the figure is also intentionally omitted within the bays between the interior girders. This is done to reduce the total number of cross frames required within the bridge as well as to reduce the overall transverse stiffness effects.

Constraint-Induced Fracture: Updates on Detailing. Article 6.6.1.2.4 addresses the detailing of structures to minimize the possibility of constraint induced fracture in steel structures. The guidance has been updated to clarify a minimum ½-in. gap between adjacent weld toes and to provide enhanced graphics illustrating the preferred detailing at the intersection of longitudinal stiffeners and lateral connection plates with transverse intermediate stiffeners and bearing stiffeners. Two examples from the updated figures are provided (see Figure 4). The first example demonstrates that in areas of tension or reversal, when a longitudinal and a transverse stiffener intersect, the longitudinal stiffener should be kept continuous to improve the fracture and fatigue performance. The second demonstrates the preferred detailing at the intersection of a bearing stiffener and a lateral connection plate in a region subject to compression only. In this case, since the web is in compression at the connection plate, fracture is precluded and it is acceptable to cope the connection plate to fit around the continuous bearing stiffener.

Global Stability of Narrow I-Girder Bridge Units. The 8th Edition includes

Commentary in the new specification builds on recent research conducted at Georgia Tech on forces in skewed steel bridges.

an equation that serves as an indicator as to when global stability of the spans of two- and three-girder systems may be critical as a failure mode when in their non-composite condition during the deck placement operation. This is found in Article 6.10.3.4.2, which has been renamed "Global Displacement Amplification in Narrow I-Girder Bridge Units." The recommendations in this article, resulting from research at the University of Texas, are intended to avoid excessive amplification of the lateral and vertical displacements of narrow, straight, I-girder bridge units, with no external bracing or flange-level lateral bracing during the deck placement operation or at any other time before the concrete deck has hardened. The global buckling mode in this case refers to buckling of the bridge unit as a structural unit generally between permanent supports, and not buckling of the girders between intermediate braces. The provisions are not intended for application to I-girder bridge spans in their full or partially composite condition, or to I-girder bridge units with more than three girders. The current equation for the elastic global lateral-torsional buckling resistance of the span acting as a system, M_{gs}, is shown below, with the introduction of a C_{bs} factor in the 8th Edition that reflects the moment gradient conditions of the structure:

$$M_{gs} = C_{bs} \frac{(\pi^2 w_g E)}{L^2} \sqrt{(I_{eff} I_x)}$$

The value of C_{bs} is 1.1 for simple-span units and 2.0 for fully erected continuous-span units. For continuous units in the partly erected condition, the 1.1 value for simple spans is conservatively used. In addition to the introduction of the C_{bs} term, the 8th Edition also increases the percentage of this moment that can be applied to the system prior to needing to introduce measures such as lateral bracing systems or resizing the beams to provide a higher degree of stiffness. The new provisions allow the applied factored moment to reach 70% of M_{gs} as a limiting value. Cautionary guidance is given that the behavior of narrow straight girder systems should not assume to apply to narrow curved girder systems; these systems require a more careful examination of displacement and stress amplification when external bracing or flange level lateral bracing is not provided.

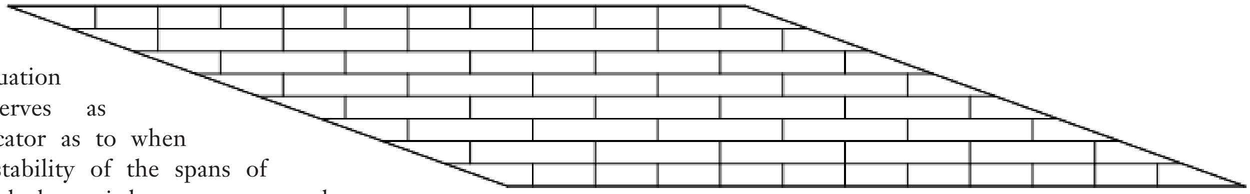


Figure 3. AASHTO Figure C6.7.4.2-1: Beneficial staggered diaphragm or cross-frame arrangement for a straight bridge with parallel skew.

Updates to Bolted Connection Provisions. The shear strength of bolts with threads included and excluded from the shear plane has been increased to reflect a slight increase in the stated value of the ratio of the yield to tensile strength of high-strength bolts (raised from 0.6 to 0.625), as well as to reflect newer information on the non-uniform load sharing in lap splice tension connections (correction raised from 0.8 to 0.9). This results in the common shear strength of a bolt being raised from a traditional value of 0.6 × 0.8 = $0.48A_b F_u$ to a new value of 0.625 × 0.9 = $0.56A_b F_u$—an increase of 16.7% for a typical high-strength bolt with the treads excluded from the shear plane. A similar increase is provided for threads included in the shear plane. However, due to the increase in the non-uniformity factor from 0.8 to 0.9, a revision in the long-connection correction factor was needed. The existing provision that requires an additional 0.8 factor to be applied for lap-splice tension connections longer than 50 in. has been revised to a correction factor of 0.83 for connections longer than 38 in.

Additional changes to the bolted connection provisions include slight changes to the slip coefficient table and the introduction of a new Class D surface condition having a slip coefficient value of 0.45, slightly below the 0.5 Class B value. Some coating systems are not able to meet the 0.5

Steve Percassi, Bergmann Associates

The new specification provides long-needed guidance for contractors and their engineers who need to evaluate strength and stability during critical stages of erection.

The specification includes changes regarding detailing skewed bridges, longitudinal stiffeners and connection plates.

Class B slip coefficient and as a result, were then required to have the bolts designed using the much lower 0.33 coefficient. Introduction of the new Class D surface condition provides a slight reduction in capacity, but reflects the actual performance of these coating systems and their influence on bolt capacity.

A new article on high-strength structural fasteners, 6.4.3.1, is now included to introduce the new ASTM F3125 standard for high-strength bolts, which combines ASTM A325, A325M, A490, A490M, F1852 and F2280. ASTM will no longer maintain the many specifications related to high-strength bolts, nuts, washers, indicators, etc. All bolting components are now included in the new F3125 standard. In terms of specification, what was once called an A325 bolt will now be referred to as ASTM F3125, Grade A325 bolt.

Guide Specification for Wind Loads

In 2015, interim revisions to the 7th Edition of the *Specifications* introduced new wind load provisions based on a

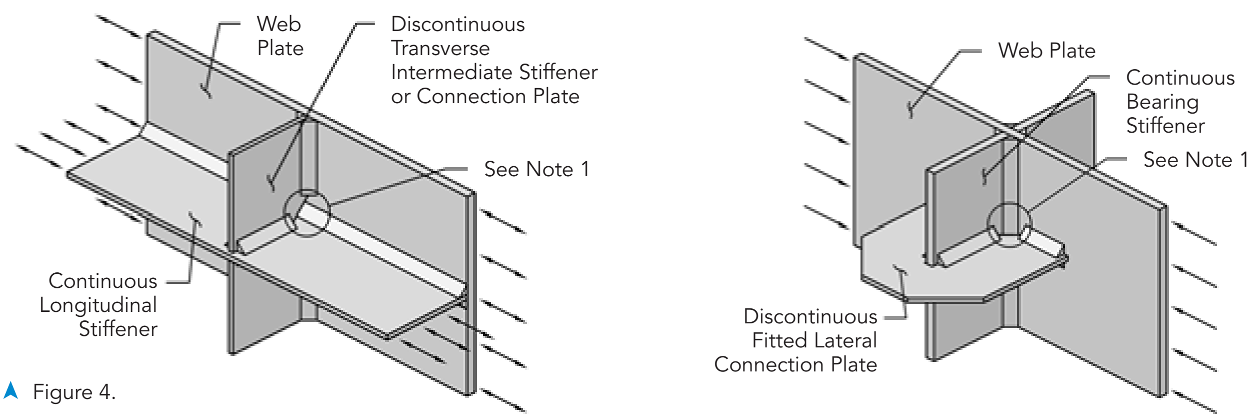


Figure 4.

"three-second-gust" procedure for determining the design wind speeds. This replaced the prior definitions based on the "fastest mile" approach. In parallel, new wind load provisions for temporary loading of bridges during construction were also being prepared. In 2016, these provisions were successfully balloted and have been published as a new *Guide Specification for Wind Loads on Bridges During Construction*. They reflect that the flow of wind around a completed structure is fundamentally different than on an open frame during construction. The exposure period for construction also differs greatly from that for completed bridges. Completed bridges need to be designed for maximum wind loads that they might experience over their lifetime, while the critical construction period for a typical girder bridge might be as short as a few weeks. This correlates to a much different probability of exceedance for short exposure wind loads. All of these factors have been considered in the new *Guide Specification*.

The publication is based on factors that relate to the elevation of the structure, gust factors and drag on open framing systems. Concerning drag, unique loadings are specified for windward, interior and leeward beams in a cross section. The gust factors also reflect the type of girder; for steel bridges, both I-girder and tub girder cross sections are addressed along with a correction for characteristics of girder spacing versus girder depth. This new specification provides long-needed guidance for engineers who design bridges, as well as for contractors and their engineers who need to evaluate strength and stability during critical stages of erection.

These are just a few of the important changes in the 8th Edition of the AASHTO *LRFD Specifications* that will influence the design, detailing and construction of steel bridges. The intent of all of these changes is to integrate the latest research, clarify the provisions for steel structures when needed and provide engineers the most current state of the practice for safe and efficient steel bridges. ■


John Rodems

▲ The *Guide Specifications For Wind Loads On Bridges During Construction* introduces tools to evaluate the effects of wind loads on bridges of all types under construction.


Steve Percassi, Bergmann Associates

▲ ▼ The flow of wind around a completed structure is fundamentally different than it is around an open frame during construction.


Steve Percassi, Bergmann Associates

Collaborative EFFORT

BY BRANDON CHAVEL, PE, PhD

Throughout its 20-year existence, the AASHTO/NSBA Steel Bridge Collaboration has been dedicated to helping steel bridge industry professionals get the most out of their bridges.

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BRIDGE INDUSTRY PROFESSIONALS are faced with many decisions as they journey through the steel bridge superstructure design, fabrication and construction process.

The designer is faced with choosing the right bridge type, understanding the structural behavior, running the appropriate analysis, designing the bracing, creating the appropriate fatigue details and selecting a suitable bearing configuration. Fabricating the steel elements is just as important, encompassing material procurement considerations, web and flange cutting and sizing issues, welding and assembling connections and selecting the desired fit condition if the bridge alignment is curved and/or the supports are skewed.

Furthermore, designers should—and erection engineers must—consider how the bridge may be constructed, including common steel erection methods and possible crane placements, girder shipping lengths and weights and the deck-casting sequence. Fortunately, there are several published references that can be used to aid in the development of efficient and economical steel bridge superstructures from design through construction.

Brandon Chavel (**brandon.chavel@hdrinc.com**) is a senior professional associate and bridge section manager with HDR in Cleveland.

Leading the way is the AASHTO/NSBA Steel Bridge Collaboration, a joint effort between the American Association of State Highway and Transportation Officials (AASHTO) and the National Steel Bridge Alliance (NSBA). This collaboration brings together representatives from state departments of transportation, the Federal Highway Administration (FHWA), academia and various industry groups related to steel bridge design, fabrication and inspection. For the last 20 years, the group's mission has been to provide a forum where professionals can work together to improve and achieve the quality and value of steel bridges through standardization of design, fabrication and erection. Currently, there are 10 working Collaboration Task Groups in which members volunteer their time and resources with the intent of developing resources that provide steel bridge industry professionals with best practices. The Task Groups include:

- TG 1 – Detailing
- TG 2 – Fabrication Specification
- TG 4 – QA/QC
- TG 8 – Coatings
- TG 10 – Erection
- TG 11 – Steel Bridge Design Handbook
- TG 12 – Design for Economy and Constructability
- TG 13 – Analysis of Steel Bridges
- TG 15 – Data Modeling for Interoperability
- TG 16 – Orthotropic Deck Panels

Over the last 20 years, these groups have produced essential resources, in the form of specifications or guidelines, that are available free to the steel bridge community. The specifications are written in standard specification language so they can be adopted in whole as part of the contract document, as applicable, and continue to be a priority for the Collaboration as they provide a means of standardization when adopted as a part of project's contract documents. The guidelines are written as references to be used during the design, fabrication and construction processes and are a consensus of best practices developed by industry. Referencing these specifications and/or guidelines allows for a common language across all stakeholders, including owners, engineers, fabricators and erectors, while also facilitating the standardization of steel bridges.

In some cases, individual specifications and guidelines are written with a specific audience in mind. Here, we'll highlight which documents are applicable to which stakeholder, though it should be noted that anyone involved in the steel bridge industry should be familiar with each of these very important and useful documents.

Bridge Engineers

Several of the specifications and guidelines are "must-have" references for all steel bridge engineers, regardless of level of experience and expertise. These references provide bridge engineers with consensus best practices and necessary information to design efficient and economical steel bridges. They are as follows:

- G1.1-2000 *Shop Drawing Approval Review/Approval Guidelines* provides owners and engineers with typical guidelines, as well as an overall framework of responsibilities for the approval of shop drawings. A checklist of common items that should typically be reviewed is provided.
- G1.2-2003 *Design Drawings Presentation Guidelines* should be referenced for the development of design drawings, as it provides advice on the minimum information required to detail and fabricate a steel structure. Sample drawings illustrating the needed information are provided. While most owners have their own standards, additional drawing details may be warranted, as shown in these guidelines, that will help to facilitate detailing and fabrication of the structure.
- G1.4-2006 *Guidelines for Design Details* is a guideline that provides a collection of sample design details that allow for the economical fabrication and erection of bolted splices, cross frames and stiffeners. When in doubt regarding a specific design detail, this should be the engineer's first reference.
- G9.1-2004 *Steel Bridge Bearing Design and Detailing Guidelines* presents steel bridge bearing details that are cost-effective, functional and durable.
- G12.1-2016 *Guidelines to Design for Constructability* provides engineers with design and detailing recommendations to help make steel girder type bridges more easily fabricated and constructable. Engineers should refer to this guideline for a better understanding of certain details can affect fabrication, as well as for general guidance that will allow the engineer to make better informed decisions during design.
- G13.1-2014 *Guidelines for Steel Girder Bridge Analysis* provides the most comprehensive presentation and discussion regarding analysis techniques associated with steel girder bridges. The guideline includes a discussion on line girder, 2D and 3D analysis methods while also helping engineers determine the appropriate level of analysis based on a bridge's geometric aspects. Other topics include the behavior characteristics of curved and/or skewed steel girder bridges, loading considerations, constructability analyses, consideration of detailing methods, cross frame modeling and necessary analysis considerations for phased construction, re-decking and widenings.

The Collaboration: A History

The AASHTO/NSBA Steel Bridge Collaboration was born in 1997, over dinner one night at the spring meeting of the AWS D1 Committee in Phoenix.

At that dinner, Fred Beckmann, Krishna Verma and I chatted about the lack of standard practices in steel bridges. In our collective experience, we knew of many examples where lack of standardization led to higher costs and longer schedules. Further, the broad variety of special requirements often led to errors; what was demanded on one job might be strictly forbidden on another and expectations could sometimes get mixed up, resulting in errors, headaches and costly rework. Frustrated by the unnecessary waste, we hypothesized about the reasons and possible solutions for such great variation. Wasn't there some way to get everyone, or at least many more folks, on the same page?

Clearly, a lack of standards was a fundamental reason for the lack of standardization. For fabrication requirements, most state DOTs used their own specification (typically a "steel structures" item in their standard specifications book) resulting in a great variety of requirements on the shop floor. Conversely, the AASHTO/AWS D1.5 *Bridge Welding Code* demonstrates what is possible when standards exist. Brought into existence in 1988, 49 of 50 states have adopted it to govern bridge welding—thus achieving some standardization for many aspects of steel bridge fabrication.

AASHTO/NSBA
COLLABORATION
CELEBRATING 20 YEARS

The founding trio felt that many more steel bridge construction activities could be standardized if standards existed, and the potential benefits were obvious. We knew that the standards would be strong if a diverse group, representing every aspect of steel bridge design and construction, collaborated to develop them. Further, we recognized that like the *Bridge Welding Code*, such standards would gain greater acceptance in the bridge community if they were approved by both the public and private sectors.

So, we decided to pursue support from AASHTO and NSBA months later at the annual AASHTO Highway SCOBS (Subcommittee on Bridges and Structures) meeting in Jackson, Wyo. Graciously, Ed Wasserman, then chair of T14 – Steel Design Technical Committee, agreed and put me on the agenda to present the concept publicly. At the same venue, Fred and I presented the concept to the NSBA Executive Council, where it received strong support from Arun Shirole, who was the executive director at the time, as well as Pat Loftus, then chair of the NSBA Executive Committee, and permission was eventually granted to found the Collaboration.

During the summer of 1997, Fred and I worked through the details about how to operate the Collaboration. Wanting to be as inclusive as possible, I sent written invitations (by signed letter, since this was before email was in wide use) to more than 300 industry professionals, including reaching out to every state DOT. To help facilitate travel for some potential participants, the first meeting was scheduled for co-location with the fall 1997 meeting of the AWS Structural Welding Committee in Cincinnati. Speakers were arranged to address a variety of topics to help plant the seeds of success.

The meeting found fertile ground indeed; responses were overwhelmingly in favor of the standardization promise envisioned in the Collaboration. Though many could not travel to the meeting, more than 40 industry professionals attended. Walter Gatti of Tensor Engineering and Bob Kase, then of High Steel, presented recommendations for constructable steel bridge detailing. Kim Roddis, then of the University of Kansas, spoke about a pooled fund study she was leading to automate and standardize shop repair procedure. Krishna spoke about new technologies and the need for associated implementation to advance the steel bridge state-of-the-art. I spoke about the broad and often conflicting variety of shop fabrication requirements. And Lou Triandafilou, then with FHWA, spoke about efforts among in the FHWA Region 3 (now the mid-Atlantic) states to standardize practices regionally—similar to the Collaboration's plans at the national level.

Enthusiasm and energy built and after two days of meetings, action plans were in place. Eight task groups were formed to address the first priorities of the Collaboration. The Collaboration was structured such that the task groups would develop standards, then present them to the Main Committee for approval—and upon approval, the standards would be sent to the NSBA and SCOBS. The Main Committee would operate by consensus: Anyone with an interest in steel bridges could participate and vote because the founding body felt it important that everyone be heard.

Hence, the Collaboration was born, and the initial meeting took place in the fall of 1997. The Collaboration has met twice a year since then, each spring and fall. The past two decades have seen thousands of hours of fruitful discourse among hundreds of professionals in the steel bridge community, as well as the publication of dozens of standards—many now in their second or third edition. Undisputedly, the standards represent the state-of-the-art for the topics they cover. Steel bridges and the needs of the steel bridge community continue to grow and change, but the original mission—improving steel bridge design and construction through the development and implementation of state-of-the-art standards—has remained the same. Here's to the next 20 years!

– By Ronnie Medlock, PE

Fabricators

There are also multiple Collaboration documents geared directly toward fabricators and owners. Several of these are often adopted in whole by owners, or as part of the contract documents for a particular project.

- S2.1-2016 *Steel Bridge Fabrication Guide Specification* provides fabricators with the necessary guidance to better achieve quality and value in the fabrication of steel bridges by providing specification language in regard to material control, workmanship, application of heat and geometric control.
- S4.1-2002 *Steel Bridge Fabrication QC/QA Guide Specification* sets minimum requirements that can be adopted by fabricator's quality control program and by an owner's quality assurance program.
- S8.1-2014 *Guide Specification for Application of Coating Systems* provides consensus procedures for the application of zinc-rich coating systems on steel bridges. It includes requirements with regard to material acceptance, surface preparation and paint application, as well as references to all applicable standards.
- S8.2-2016 *Guide Specification for Application of Thermal Spray Coating (Metallizing) for Steel Bridges* is a specification for shop metalizing practices. With an expected late 2017 publication date, owners can adopt this practice to govern metalizing practices at fabricators. Comprehensively, it addresses fabricator personnel qualifications, quality control manual requirements for metalizing and cleaning and application requirements.
- G1.2-2002 *Shop Detail Drawing Presentation Guidelines* provides a reference for fabricators for the development of shop drawings, while also serving as a reference for owners and engineers approving shop drawings.
- G2.2-2016 *Guidelines for Resolution of Steel Bridge Fabrication Errors* addresses common issues that may occur during the fabrication process and how these errors can be resolved in an ecumenical fashion while also preserving the integrity and resistance of the particular component.
- G4.2-2006 *Recommendations for the Qualifications of Structural Bolting Inspectors* defines the essential factors involved in structural bolting and the qualification of personnel inspecting and monitoring those operations.
- G4.4-2006 *Sample Owners Quality Assurance Manual* provides an example that can be used as a guide by owners or other shop inspection agencies for the development of their own quality assurance procedures.

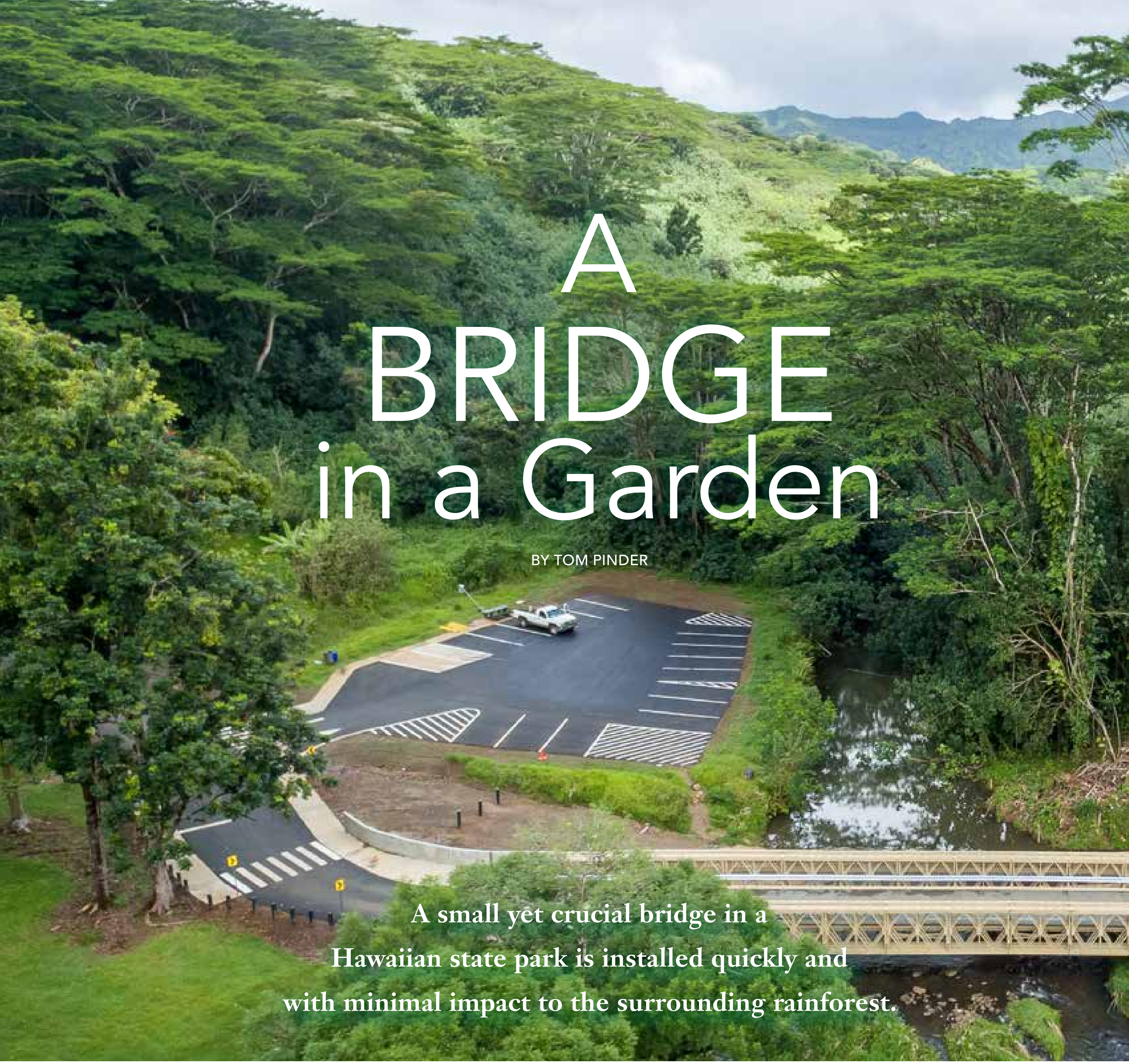
Contractors and Erectors

One collaboration specification directly addresses the erection of steel girder bridges: S10.1-2014 *Steel Bridge Erection Guide Specification*. This document, which can be adopted by owners in whole or in parts, covers all aspects of steel girder bridge erection, including transportation and job site storage of girders, bolted connections, inspection, repairs and guidance for erection engineering computations. It also includes checklists for erection plans and procedures and erection engineering computations that can be used when assembling or reviewing erection plans, procedures and computations.

Ongoing Collaboration Work

The various task groups continue to develop resources for the steel bridge community. For example, TG10 has been working to update the existing S10.1-2014 *Steel Bridge Erection Guide Specification* while also combining efforts with TG15 to develop an information delivery manual for steel bridges that would standardize the software formatting for erection engineers in a bridge information modeling (BrIM) working environment. TG11 is currently developing a much-needed guideline document for cross frame and diaphragm design in steel I-girder bridges. And TG16 is currently working towards a guideline for more easily manufactured orthotropic decks, evaluating recent research results and industry projects and authoring research needs statements.

For the last 20 years, the AASHTO/NSBA Steel Bridge Collaboration has strived to provide the best available resources to the steel bridge community for the efficient and economical construction of steel bridges. The aforementioned specifications and guidelines focus on providing necessary references and guidance to designers, fabricators, owners and erectors and in many cases, these Collaboration documents are the only such references in existence. All Collaboration members have one goal in mind: to develop methods, strategies and standardizations that others can use on a day-to-day basis. Because of the wide range of Collaboration members, the documents produced by its task groups are consensus industry documents that will provide bridge professionals with the ability to achieve high-quality economical solutions for steel bridges for years to come. ■



A BRIDGE in a Garden

BY TOM PINDER

A small yet crucial bridge in a Hawaiian state park is installed quickly and with minimal impact to the surrounding rainforest.

Tom Pinder is Acrow's western regional sales manager.

THE HAWAIIAN ISLAND OF KAUAI is nicknamed "the Garden Isle" thanks to the tropical rainforest covering much of its surface.

A recent bridge preplacement project deep in the heart of this lush landscape was built with the goal of maintaining that green status and addressing the ecological and land management concerns of state regulators and community groups, not to mention the thousands of people who use the area every year. The project's location, the Keahua Arboretum in the Keahua State Forest, is home to native plants and vegetation introduced by the University of Hawaii and is enjoyed by locals and tourists for picnicking, swimming, hiking, biking and horseback riding.

The original route in and out of the preserve, which crossed the Keahua Stream, was a concrete roadway that had sunk into the stream over time and often flooded after heavy downpours, rendering safe passage difficult if not

All photos: Acrow Bridge

A new bridge replaces a a concrete roadway that had sunk into the stream over time and often flooded after heavy downpours.

impossible during the rain-heavy winter and spring months in this remote but highly traveled area. A new bridge was needed to provide a safer route during these high-water events so cars would no longer be stranded if they were on the mountain side of the river or washed out trying to cross the existing ford during flash flooding.

Hawaii's Department of Land and Natural Resources (DLNR) decided to construct a permanent bridge at the site, along with a new roadway alignment and a drainage culvert with an inlet and outlet. The 110-ft-long single-lane bridge also includes a sidewalk on one side, separate from the roadway, creating a safe crossing for pedestrians.

The project was made available for bidding in June 2014; general contractor Mocon Corporation finished negotiating substructure design changes with the DLNR and their engineering consultant, Kai Hawaii, in June 2015; and fabricator Acrow began production immediately after design was finalized, then shipped the bridge components to the job site that fall. However, due to rains and foundation pile supply issues, the bridge sat on the job site for about five months before assembly began in early 2016. The success of the project, which opened this past April, was made possible by several key factors, both structural and logistical:

ABC. The rapid assembly and installation of the bridge, using only a three-man crew, made it a true accelerated bridge construction (ABC) project.

Minimal installation equipment. The simplicity of the structure meant only standard hand tools were required. In addition, because all bolts were non-slip-critical, neither pre-installation verification testing—using a tension calibrator such as a Skidmore-Wilhelm Bolt Testing Measuring Device—nor DTI (direct tension indicator) ASTM F959 washers were required, resulting in a snug-tight approach and lower installation cost compared to concrete or large steel beam bridges.

Modular construction. Modular components allowed for ease of shipping, parts staging and assembly in this remote region with a tight build area. Large cranes are not commonly available on Kauai, but this did not pose a problem since the contractor was able to build the structure in situ on falsework in the shallow stream while requiring minimal space for laydown and construction. The modular approach also reduced the potential of damage to the painted sections. All bridge elements were packed into 40-ft shipping containers at Acrow's facility in New Jersey and shipped directly to Kauai.

Low-to-no maintenance. Maintenance was yet another issue to consider, especially given the project's location in a remote area with a tropical marine environment. The components were hot-dip galvanized to ASTM A123 for a long life (75+ years with less than 5% surface rust in a tropical marine environment) and completely painted for further rust protection and aesthetic purposes; the individual elements of the structure were shop painted prior to shipping, with touch-up performed on site. Special care in handling and packaging

The 110-ft-long single-lane bridge also includes a sidewalk on one side, separate from the roadway, creating a safe crossing for pedestrians.

was taken in order to minimize the paint scratches that might occur in typical shipping practices.

The duplex coating system provides a higher level of corrosion protection, with the galvanized coating protecting the base steel by providing both cathodic and barrier protection, and the paint acting as a barrier for the zinc layer and significantly reducing its corrosion rate. The maintenance cycle for paint over galvanized steel is typically one-and-a-half to two times greater for paint over bare steel, resulting in a significant cost savings over the life of the structure and a virtually maintenance-free bridge.

Reduced foundation costs. Conventional designs would have required larger abutments and, in the case of concrete, longer curing times, increasing the construction schedule and requiring larger cranes—which again, aren't as readily available on Kauai. In addition, longer members would have been more difficult to maneuver over the tight, curving island roads.

Non-fracture-critical design. As the bridge's design uses redundant members, it is not fracture-critical and is on a normal bridge inspection schedule. The mostly visual inspection process is particularly easy to perform since nearly every part of the structure is accessible.

Owner
Hawaii Department of lands and Natural Resources

Designer
Kai Hawaii, Inc., Honolulu

General Contractor
Mocon, Honolulu

Steel Fabricator
Acrow Corporation of America, Parsippany, N.J.

Modular components allowed for ease of shipping, parts staging and assembly in this remote region with a tight build area.

BIG GAIN for the Little River

BY JONATHAN HISEY, PE

A plate-girder scheme was the only feasible option to replace a small county bridge over the Little River in Oklahoma's southeastern-most county.

Jonathan Hisey (jhisey@mkec.com) is a transportation engineer and project manager with MKEC Engineering, Inc.

THE LITTLE RIVER BEGINS in the Ouachita Mountains of southeastern Oklahoma and runs for 130 miles before entering Arkansas and finally the Red River.

It cuts across southern McCurtain County, Okla., and is crossed by seven bridges, three of which are owned by the county. One of these, a structurally deficient single-lane Parker through-truss, was in poor condition, with a weight limit of only three tons and a sufficiency rating of 9.4 out of 100.

Efforts were made to repair the substructure, but the condition of the truss eventually caused the Oklahoma Department of Transportation (ODOT) to close the bridge to all traffic in 2014. Temporary bridge options

▲ The bridge has span lengths of 100 ft, 237 ft and 100 ft.

were investigated, but the locally available bridge was too short and purchasing a temporary bridge was deemed too expensive. The closure created an 18-mile detour that was a major inconvenience to commuters, school buses, and local farmers and ranchers hauling livestock.

As a result, the bridge was slated for replacement as part of the Oklahoma County Improvements for Roads and Bridges (CIRB) program, which is overseen by the Local Government Division of ODOT. The design of the new bridge began with an initial three-span configuration of 120 ft, 160 ft and 120 ft, composed of simple-span plate girders.

Endangered Species and Schedules

However, significant construction challenges were encountered during the preliminary environmental clearance process. The Leopard Darter, an endangered species of fish, and several endangered species of mussels were present in the river, which would complicate and lengthen the time needed to obtain permits from the U.S. Army Corps of Engineers and their consultation with the U.S. Fish and Wildlife Service. Based on previous experience, the consultation process was expected to take two to three years.

The engineer of record, county commissioner and environmental consultant met with ODOT staff to figure out how to accelerate the timetable for replacement. It was determined that

Girder depth was limited to 6 ft to fit within the proposed roadway profile, and also to allow for the low beam elevation to clear the 25-year event water surface elevation.

if all construction activities were performed outside of the Ordinary High Water Mark (OHWM) of the river, the federal nexus of oversight, consultation and review would be greatly reduced—which in turn would reduce the wait from years to months.

In order to move all construction activity outside the OHWM, the piers of the proposed bridge were moved outside the two-year water surface elevation. The end spans were sized to be long enough to create a total bridge length that would meet the hydraulic requirements necessary for the county road. Once the hydraulic design was analyzed, the structural team settled on final span lengths of 100 ft, 237 ft and 100 ft.

Preferred Plate

Most county bridges in Oklahoma are built using standardized prestressed concrete beam designs specifically developed by ODOT for county roads. However, given the span lengths and the remote location, the use of continuous parallel-flange steel plate girders was really the only option for the superstructure. The typical section of the bridge consisted of four girders on 9-ft, 2-in. centers, a 32-ft clear roadway, 8-in.-thick deck and an ODOT TR3 concrete traffic rail. The project, whose total cost was just over $4 million, used nearly 400 tons of structural steel.

The entire bridge was completed by December 2016, and the 2,500 ft of approach roadway was completed in January 2017 and immediately opened for traffic.

Girder depth was limited to 6 ft to fit within the proposed roadway profile that had already been designed, and also to allow for the low beam elevation to clear the 25-year event water surface elevation. This created a span to depth ratio of 39.5, which is greater than the minimum ratio of 37 recommended in Chapter 2 of the AASHTO *LRFD Bridge Design Specifications*; this depth limitation did cause the flanges to be slightly wider than would have occurred with a deeper section. Ultimately, 24-in.-wide flanges with thicknesses ranging from 1 in. to 2½ in. were selected for the design. With a girder-spacing to girder-depth aspect ratio of 1.5, K-type cross-frames were used and spaced at approximately 25-ft centers.

The customary design practice for continuous girders normally uses end span lengths that are approximately 80% of the center span length. The Little River Bridge end span lengths are almost half of that, at only 42% of the center span length. The NSBA *Steel Bridge Design Handbook* (**www.aisc.org/nsba**) states that for bridges using integral abutments, end span lengths of less than 60% of an adjacent interior span are economically feasible with the use of integral abutments.

With this in mind, integral abutments were chosen for three reasons. First, the abutment would act as a counterweight. While there was no dead load uplift, some uplift would occur under the design live load. Second, integral abutments are simpler and less expensive to construct compared to conventional abutments. Third, expansion joints, which are prone to leaking and are maintenance problems, would be eliminated at the abutments.

Since most integral abutment bridges in Oklahoma are typically limited to 300 ft in length, a literature review was done to ensure proper detailing and design of the connection between the girders and the abutment. The girders were analyzed for both simple and fully fixed support conditions at the abutment, and shear connectors were designed for the web at the end of the girders to ensure proper connection to the concrete abutment diaphragm. Holes were also detailed through the web to allow reinforcing steel in the abutment diaphragm to pass through. Bearing assemblies at the abutments consisted of 1-in.-thick plain elastomeric pads with 1⅝-in.-diameter anchor bolts passing through the bottom flange.

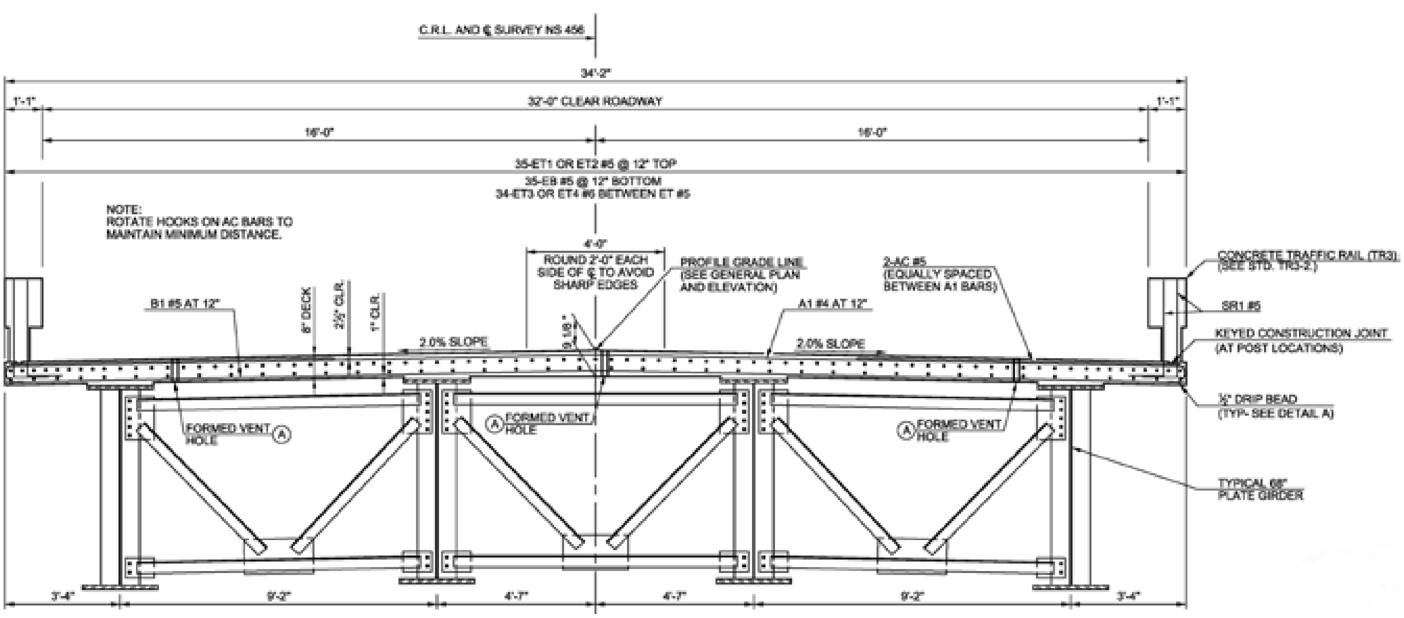
Per traditional continuous girder design practices, field splice locations were located at points of dead load contraflexure. However, due to the small end-span to center-span ratio, the contraflexure location in the end spans were approximately 20 ft from the abutment. Therefore, two field splices were placed in the center span 45 ft from both piers, with optional field splices located 67 ft from each end of the girder. The sole purpose of the optional field splices was to shorten field sections of the girders for shipping.

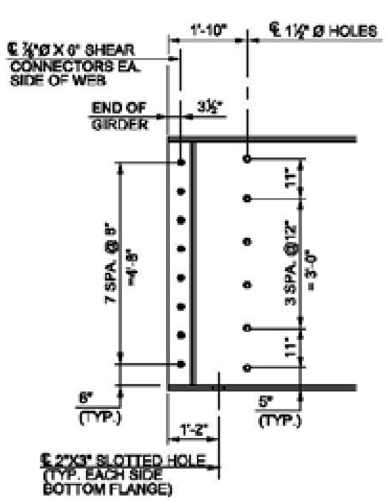
Staying Above Water

Since all construction activity would be prohibited below the OHWM, no cranes or equipment of any kind would be allowed between the piers. Before bridge design had progressed too far, two bridge contractors in Oklahoma were consulted on the feasibility of construction, given these constraints.

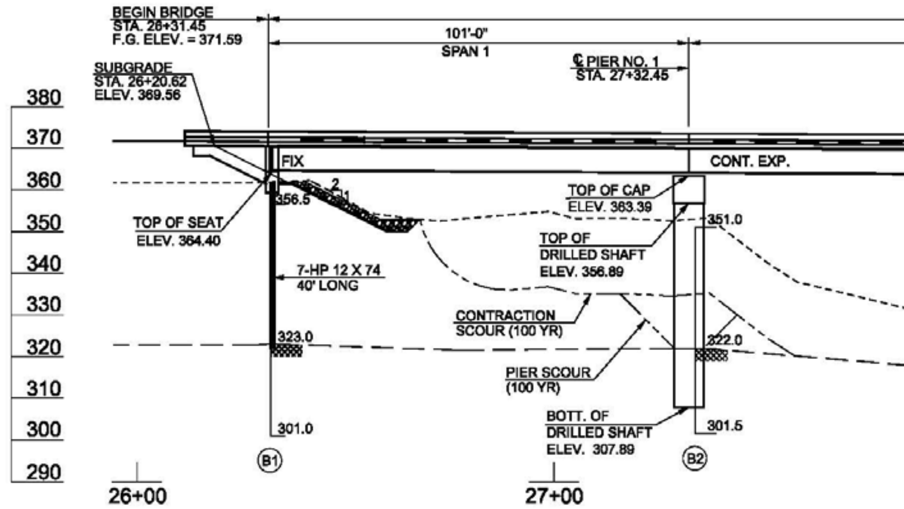
Two options were considered: using one large crane or using a barge in conjunction with several smaller cranes. The barge option was ruled out due to the exclusion of activity below the OHWM, the difficulty of launching a barge within the environmental constraints and the water level of the river. The contractor chose not to use the optional field splices, resulting in three equal 146-ft sections for each girder. While most cranes used in Oklahoma bridge construction range from 100 tons to 250 tons, the larger crane size was needed to place the 30.5-ton center sections while also reaching out nearly 165 ft for the furthest lift. A quote from a local crane service was obtained, and it was estimated that a 500-ton crane with a 140-ft reach would be needed to place the center section of the girder. Construction of the new bridge was awarded to Manhattan Road and Bridge in January 2016, but was slowed by spring rains and significant flooding on river, which delayed girder erection until late summer. To place the center section, Manhattan used an 818-ton Liebherr LTM 1750-9.1 mobile crane with a luffing jib.

A typical cross-section view between the four girders.





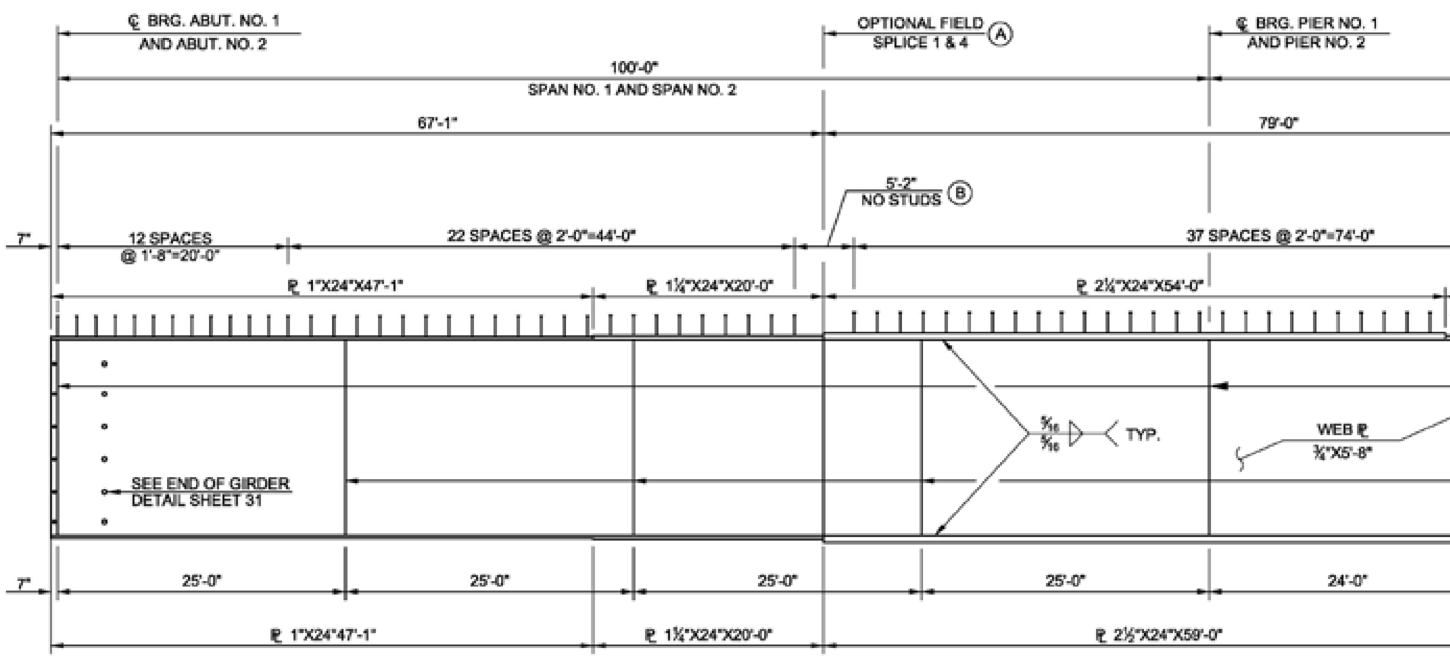
End-of-girder detail.

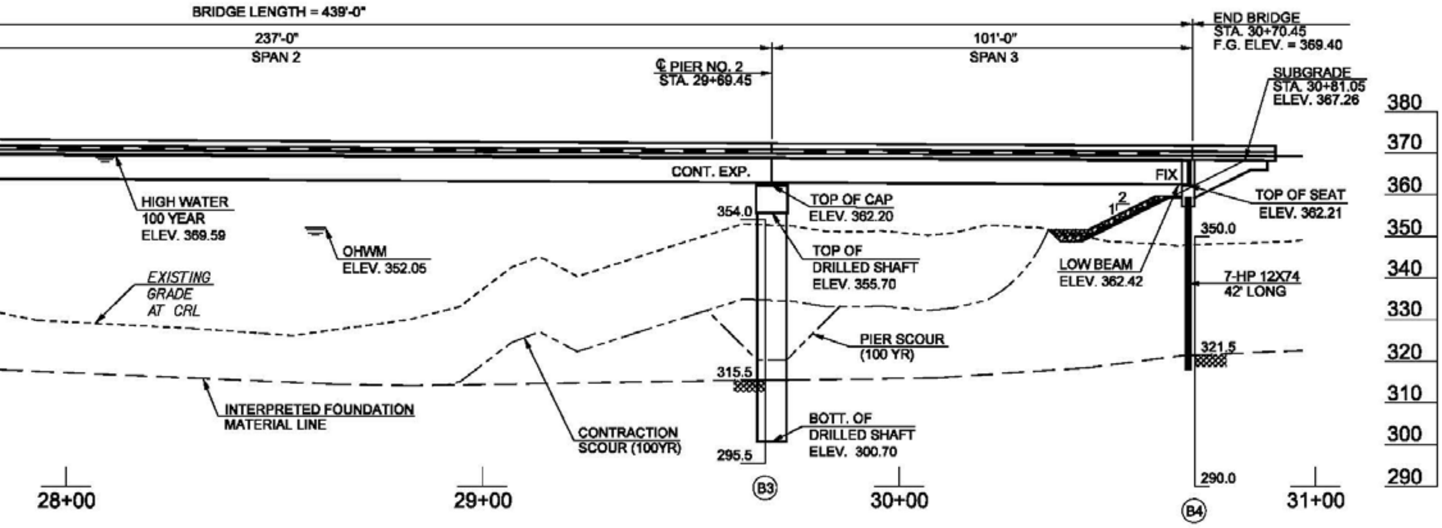


An elevation view of the crossing.

The project uses nearly 400 tons of structural steel.

Jonathan Hisey





A luffing jib, while reducing the ultimate lift capacity, was required to gain the required lifting radius. While placing the furthest girder, the crane was operating at nearly 93% of lifting capacity. This required extensive safety precautions, since a lift exceeding 75% of a crane's capacity is classified as an OSHA Critical Lift.

Girder erection was completed by the beginning of September 2016, and the entire bridge was completed by December. The 2,500 ft of approach roadway was completed in January 2017 and immediately opened for traffic—a big success for this important crossing over the Little River. ■

Owner
McCurtain County, Okla.

General Contractor
Manhattan Road and Bridge, Tulsa, Okla.

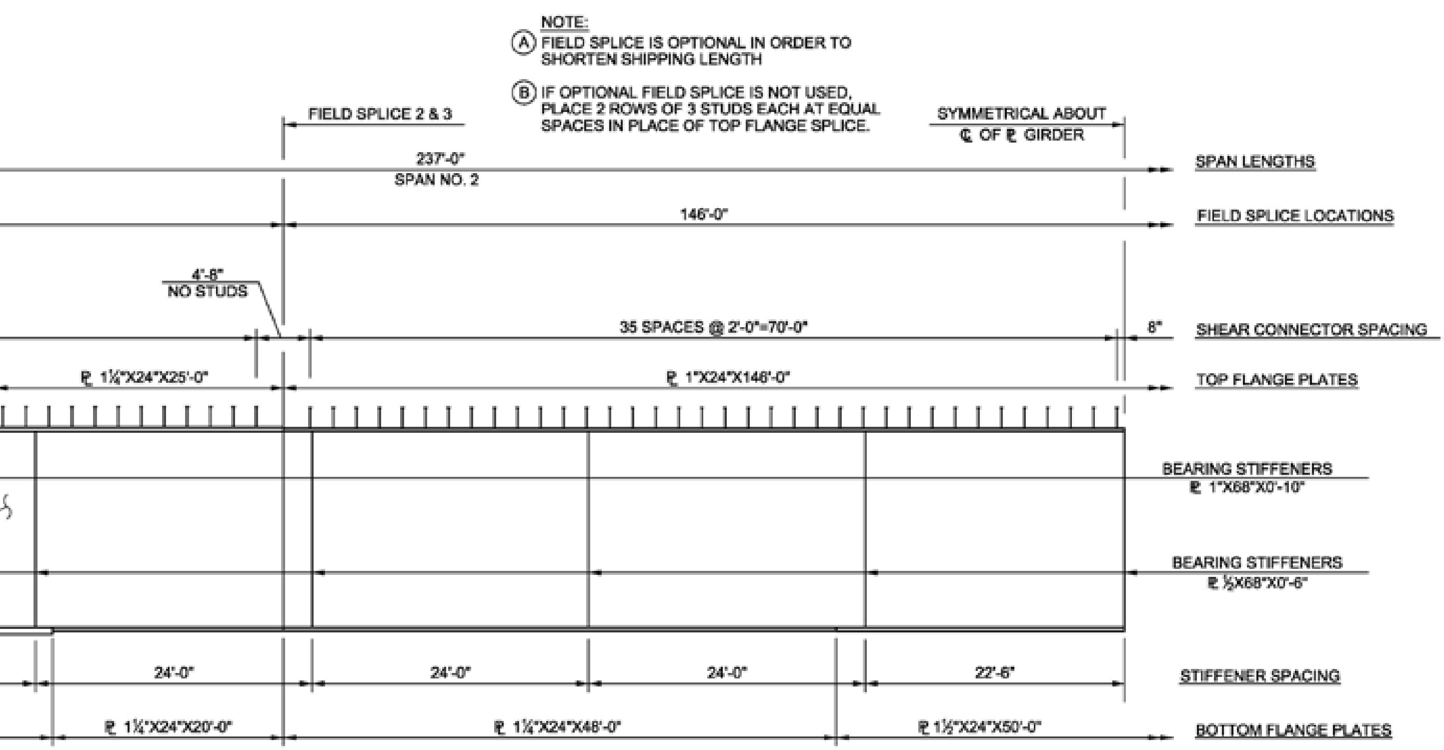
Designer
MKEC Engineering, Inc., Oklahoma City

Steel Team

Fabricator
W&W | AFCO Steel, Little Rock, Ark. AISC CERTIFIED FABRICATOR

Detailer
ABS Structural Corporation, Melbourne, Fla. AISC DETAILER

An elevation view of the plate girders.



conference preview

WHEN AESTHETICS GOVERN

Pedestrian bridges, architecturally exposed structural steel and you.

BY TERRI MEYER BOAKE

The Peace Bridge in Calgary, Alberta, is a highly customized project that used extensive brake forming to shape the steel.

CAN ROAD AND RAIL BRIDGES be attractive—even stunning?

Of course they can. Though for the most part, the more pragmatic concerns of safety, economics and efficiency typically govern their designs.

In recent years, we've seen a surge in the construction of pedestrian and cyclist bridges—and in these types of spans, aesthetics are front and center, playing an enhanced role in the decision-making process.

Terri Meyer Boake (**tboake@uwaterloo.ca**) is a professor of architecture with the University of Waterloo in Cambridge, Ontario, Canada. You can find out more about her and see more of her photography at **www.tboake.com**.

This has resulted in some challenges to the bridge construction industry. Having an architect on the team can sometime bring challenges to a process that doesn't typically involve architects. As a result, the design of such pedestrian bridges falls across the fields of engineering and architecture, having to answer to equally strong pulls between function and aesthetics. This aesthetic drive creates and must solve problems that are very different from the methods previously established for vehicular bridges.

This increase in the expressive use of structure has been enabled by the marked decrease in live loading for pedestrian bridges and has created a paradigm shift in the standard design of the spanning typology normal to bridges. Pedestrian bridges are creating structurally and visually different ways to span, challenging and inspiring the design community with a multitude of one-off designs. Although many of the spanning methods have been derived from existing structural typologies for heavier bridge types, variations have arisen as a direct result of this unique marriage of structure, architecture and art. In fact, many of these

➤ The Gateshead Millennium Bridge in Newcastle, England, must rotate to allow for the passage of ships along this waterway. The low-to-the-water design provides easy access along the river for pedestrians and cyclists.

new bridges are also being seen as public art, adding politics to the already complex processes of design and fabrication.

Beauty on Demand

Given the demand for beauty, as well as the fact that bridge framing is pretty much always exposed to view, architecturally exposed structural steel (AESS) has become the structural material of choice for the majority of these recent footbridges. Structural steel is excellent in addressing many of the particular design and erection concerns of pedestrian bridges, and AESS in particular is able to provide an excellent variety of aesthetic solutions for this new role of pedestrian bridge as public art. Steel also enables:

- ➤ Prefabrication of near-complete elements prior to erection/installation
- ➤ Simplified transportation of large bridge elements to the job site
- ➤ Minimal disruption to traffic flow during erection
- ➤ Durability and ease of inspection for maintenance

The 2016 AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303, **www.aisc.org/specifications**) Section 10 addresses important aspects of designing with AESS as applied to building elements and pedestrian bridges. Many of these bridges typically need to be designed in accordance with three of the five AESS categories: 3, 4 and C, all of which permit all-welded connections and allow for—but do not require—the grinding of welds (categories 1 and 2 will typically not be applicable for most pedestrian bridges). The treatment decisions for welded connections must be carefully negotiated by the team, as a welded and then ground choice can greatly increase the cost of the structure, but might not necessarily be required to achieve the desired aesthetic effect. (For more details on the AESS categories, see this month's "Buyer Be Aware" SteelWise

➤ The Helix Bridge in Singapore was fabricated from duplex stainless steel in order to resist the tough marine climate. Stainless steel provides a very durable solution but extreme care must be taken in its fabrication and detailing for optimum results.

➤ This mast-and-cable-supported bridge creates a lightness of structure that is only possible for this type of lightly loaded footbridge.

The George C. King Bridge in Calgary, Alberta, seems to skip across the Bow River, landing once before reaching the opposite shore. The flattened elliptical looking tubular arches have been fabricated from round HSS that have been joined by rolled plate sections. The welds on the top surface have been fully remediated, providing a bit of mystery regarding the fabrication process.

This pedestrian bridge crosses over a busy roadway in a single span. The black tube and mesh gridded barrier creates a visual contrast with the inclined trapezoidal truss that forms the main spanning system.

article on page 17.) Here are basic descriptions for these three categories:

- AESS 3: Feature elements viewed at a distance less than 20 ft (6 m)
- AESS 4: Showcase elements with special surface and edge treatment beyond fabrication
- AESS C: Custom elements with project-specific requirements that don't fall into the parameters of the other four categories

The primary influence on the generated typology seems to focus on the specific expression of the structure that is supporting the walking surface, in combination with the overall width of the span as a function of available support points. Here are the most common support systems for pedestrian bridges:

- Support from above. The mast and cable system is designed as a variation of a suspension system. The location of the mast is often eccentric or sloped resulting in a very dynamic appearance to the structure. The structural deck that supports the walkway is quite light
- Support through the middle. Tubular trusses have pedestrians walk through and experience the structure
- Support from below. The structural support system for the deck is located beneath the walking surface. It is less apparent when the bridge is crossed but very apparent when viewed from a distance

The Puente de Luz in Toronto used plate rolling to create the custom curved box sections that comprise the ribs. The bridge was divided into large segments to allow for quick site bolting during erection, as it spans over a significant train route into the center of the city.

This pedestrian bridge provides safe passage over a busy divided highway. A central support breaks the span. The deck is suspended from arched elements, and an artistic barrier prevents falls.

Weather and Site Conditions

When AESS is subjected to the elements, as is the case with most footbridges, corrosion protection must be addressed. Painted finishes remain common as there tends to be a desire to include color in the expression. It is critical that painted finishes are paired with a durable undertreatment such as metallization, as simple priming is generally inadequate. As a primary approach that combines corrosion protection and finish, stainless, weathering and galvanized steels are frequently used. However, it is necessary to detail for durability—with a focus on preventing debris from accumulating or water from pooling—as corrosion protection can ultimately fail if detailing is not done correctly. Roosting birds can also damage the structure and finish.

When planning a pedestrian bridge, the site conditions must be carefully considered. Unlike bridges that support vehicular traffic, the site conditions for urban pedestrian bridges are often significantly constrained. Pedestrian bridges are frequently being constructed as improvements to existing urban neighborhoods that are separated by rail, highway and river corridors, and as a result will have to negotiate between the footprints of existing buildings and respond to inflexible abutment conditions. Although some pedestrian bridges do make use of suspension systems, many have minimal access to their abutment or mid-span conditions and may not afford more than a simple span.

Transportation and erection concerns tend to be the drivers in determining the maximum size of prefabricated elements and the type and placement of splices and internal connections. Connection design presents many challenges to the team as they work to address aesthetic choices in regards to member size, shape and complexity. And if welding is preferred over bolting, they need to maximize shop welding and work towards expedient site erection, which lends itself to bolted connections. Site layout will also constrain the design if there is inadequate space for materials, assembly and crane access. A high level of communication amongst the designers, fabricators and erectors is critical to the success of these projects—perhaps even more so than with vehicular bridge projects.

The design choices for pedestrian bridges are virtually endless, particularly if the project is governed by the use of custom sections rather than standard available shapes, and there is a wide range of detailing strategies that seek to simultaneously address the concerns of aesthetics and good engineering practices. Want to know more? Come to our presentation! ■

This article is a preview of Session B9 "Pedestrian Bridges – Invigorate Design Creativity" at NASCC: The Steel Conference, taking place April 11–13 in Baltimore. Learn more about the conference at ***www.aisc.org/nascc****.*

conference preview

A BIG CONNECTION BETWEEN SMALL TOWNS

A new bridge between Iowa and Illinois signals another success for structural steel over the Mississippi River.

BY GREG HASBROUCK, PE

THE TINY TOWN OF SABULA, IOWA (population: circa 550) is located on a tiny island in the Mississippi River, roughly halfway between the larger Iowa river towns of Dubuque and Davenport.

While small in stature, it sits at one end of a significant crossing over the river, which has recently been replaced. The new US 52 Savanna-Sabula Bridge opened to traffic this past November, capping a monumental project for both Sabula and Savanna, Ill., on the other side of the Mississippi. At over 2,400 ft in length, the steel tied arch and plate girder bridge replaced a truss bridge, which was built in 1932. The crossing provides a crucial transportation link for the region, with the nearest alternate route over the Mississippi located 20 miles to the south in Clinton, Iowa. Over the years, a number of repairs had been made to the bridge, and it was rapidly approaching the end of its useful life and in need of replacement.

The new Savanna-Sabula Bridge consists of 12 spans totalling 2,454 ft: a 546-ft main span steel tied arch over the navigation channel flanked by steel girder approach spans. The structure, at a total cost of $80 million, extends from a causeway on the Iowa side in the middle of the Upper Mississippi River Wildlife and Fish Refuge to the high bluffs of the Mississippi Palisades in Illinois.

Greg Hasbrouck (greg.hasbrouck@parsons.com) is a supervising bridge engineer in the Chicago office of Parsons and was the bridge design lead for the US 52 Savanna-Sabula Bridge project.

Cross Section and Type Selection

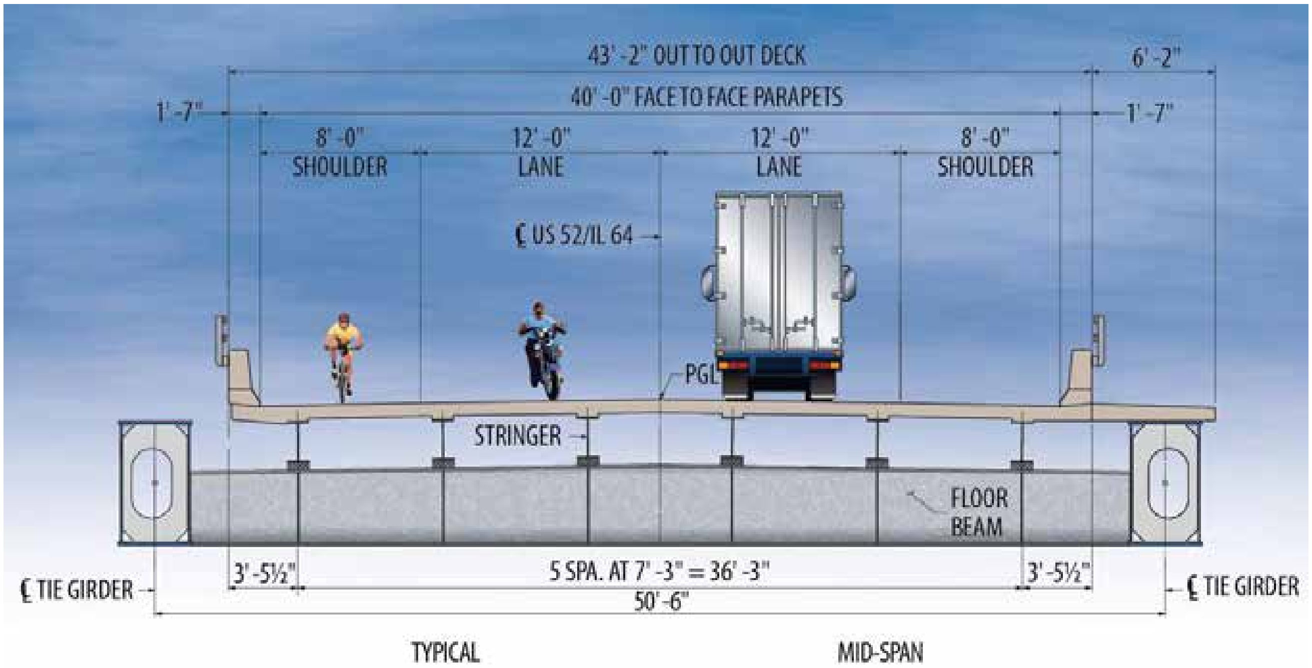
The existing structure consisted of two 10-ft lanes with no shoulders, and while projected traffic volumes for the area did not warrant more capacity, improving the cross section to meet policy standards was a priority for enhancing safety and mobility. The new cross section consists of two 12-ft lanes with 8-ft shoulders on each side to provide more clearance and accommodate bicyclists on the shoulders.

The original navigation channel span closer to the Illinois riverbank provided a 508-ft horizontal clearance and a 64.6-ft vertical clearance above the normal pool elevation. As part of the studies to improve the geometric configuration of the new bridge and roadway, the desire was to minimize the vertical profile grades on the bridge. To help achieve this goal, the design team approached the U.S. Coast Guard and facilitated the coordination of a 150-ft shift of the navigation channel toward the center of the river to bring down the grade and allow for a 7.5-ft superstructure depth over the channel.

Deep girder spans were eliminated from consideration due to the minimal structure depth, leaving tied arch and cable-stayed structure type options for the main span. After a type evaluation, a steel tied arch was selected for the main span due to its slightly lower overall cost, perceived advantages in constructability and DOT familiarity with maintenance and potential future deck replacement options.

New Tied Arch

The new bridge structure consists of an eight-span 1,420-ft steel girder approach structure on the Iowa side, a 546-ft main span steel tied arch over the navigation channel and a three-span 488-ft steel girder approach structure on the Illinois side, including a span over the BNSF rail road. The cross section consists of a 6-girder layout with girders spaced at 7 ft, 3 in. Using six girders allowed for the bridge deck to be replaced one half at a time while maintaining bidirectional controlled traffic on one lane, and three girders to maintain a redundant struc-



A cross-section view at the arch.

The $80 million bridge consists of 12 spans totalling 2,454 ft: a 546-ft main span steel tied arch over the navigation channel flanked by steel girder approach spans.

ture and eliminate the need for total closure for future maintenance and repairs.

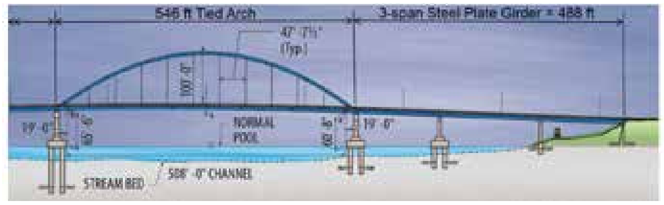
A welded steel box section arch rib and bolted built-up box tie girder were selected to provide both an efficient section for the arch in compression and redundancy for the tie girder in tension, with continuity of the force flow through the web plates in the knuckle to simplify connection details. High-performance Grade HPS50W steel was specified for the tie girder and knuckle plates for improved toughness, and the tie girder is designed for the loss of any single plate (web or flange) and the resulting eccentric loading on the remaining section.

Two vertical structural strand hangers support the tie and floor system and are offset from the floor beam and upper lateral bracing connections to simplify load paths, detailing and fabrication at these critical connections. The dual-strand system also provides redundancy at the hanger connection in the event of hanger loss.

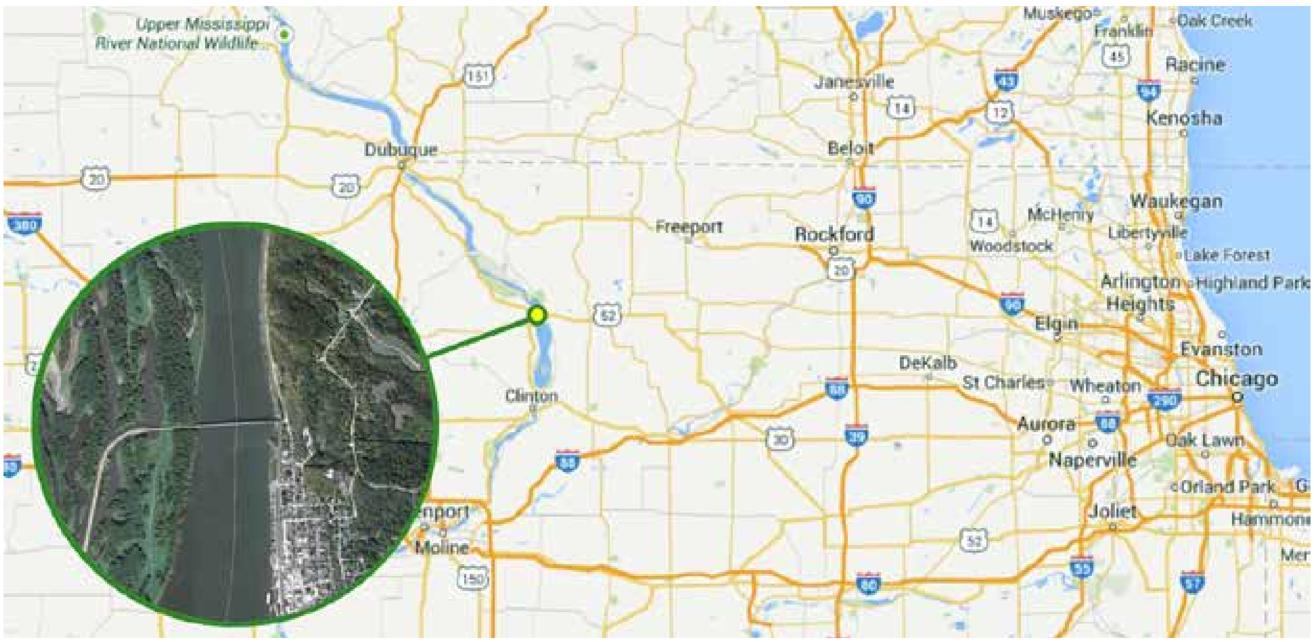
A floating deck system, with deck supported by six lines of continuous stringers spanning over top the floor beams, was selected to accommodate future deck replacement. The stringers are fixed at the center two floor beams and rest on elastomeric bearings over the remainder of the floor beams to allow for any differential movements of the arch and floor system during service. Relative movements during erection were taken through slotted holes in the connections, with the bolts tightened after pouring the deck.

Erection of the cantilever arch.

▲ The apan layout of the new bridge.



▼ The bridge's location on the Mississippi River between Iowa and Illinois.



At midspan of the arch, the deck is extended and connected to the tie girders to transfer longitudinal loads from the deck directly to the arch system through diaphragm action of the deck. The floor beams, stringers and lower lateral bracing were all detailed with lengths under 60 ft to permit hot-dipping in local galvanizing tanks for additional corrosion protection of the floor system.

Upper lateral X-bracing was chosen to provide a modern, light and efficient bracing system using simpler pin-connected tension and compression truss elements made from square box sections. These sections are galvanized to provide a protective coating on both the inside and outside surfaces of the members.

Contractor and steel erector Kraemer North America elected to construct the arch through cantilever erection, with stay towers erected on top of the main river piers and tied back to the approach superstructure steel girders two piers away. The tension in the back stays was resisted by compression in the approach girders back to the main piers, creating a balanced system with the compression occurring in the arch during erection. ■

This article is a preview of Session B3 "Major Spans–Part 1" at NASCC: The Steel Conference, taking place April 11-13 in Baltimore. Learn more about the conference at ***www.aisc.org/nascc****. In addition to featuring the US 52 Savanna-Sabula Bridge project mentioned above, the session will also focus on Little Rock's Broadway Bridge replacement project, which was featured in "Making a Signature Connection" in the July 2017 issue, available at* ***www.modernsteel.com****.*

Owner
Illinois Department of Transportation

Structural Engineer
Parsons, Chicago

Steel Team

Fabricator
Veritas Steel, Eau Claire and Wausau, Wis.

Erector and General Contractor
Kraemer North America, Plain, Wis.

Detailer
Tensor Engineering, Indian Harbour Beach, Fla.

conference preview

Bridging the information exchange chasm.

BIM FOR BRIDGES AND STRUCTURES

BY AARON COSTIN, PhD, AND
JASON STITH, SE, PE, PhD

WE ARE ARGUABLY living the greatest age of information and technology.

In the past decade, there has been an explosion of information-producing technology and software. Even more so, we are witnessing mass use of that information. Google's and Facebook's—two of the largest companies in the world by market capitalization—greatest asset is their ability to own, manage and maintain information. And we are experiencing the same evolution in the architecture, engineering, construction and operations industries, with the maturity of building information modeling (BIM) and the development of smart cities.

However, the transportation infrastructure has been slow to adopt these technologies, mainly due to the non-interoperability (e.g., sharing capabilities) of the various software options. There is a great need to bridge the chasm of non-interoperable software in order to reap the full benefits of information production, use and sharing for the life cycle of bridges and other transportation structures. To meet that need, we must look to adopt the proven means and methods of BIM seen in the building domain.

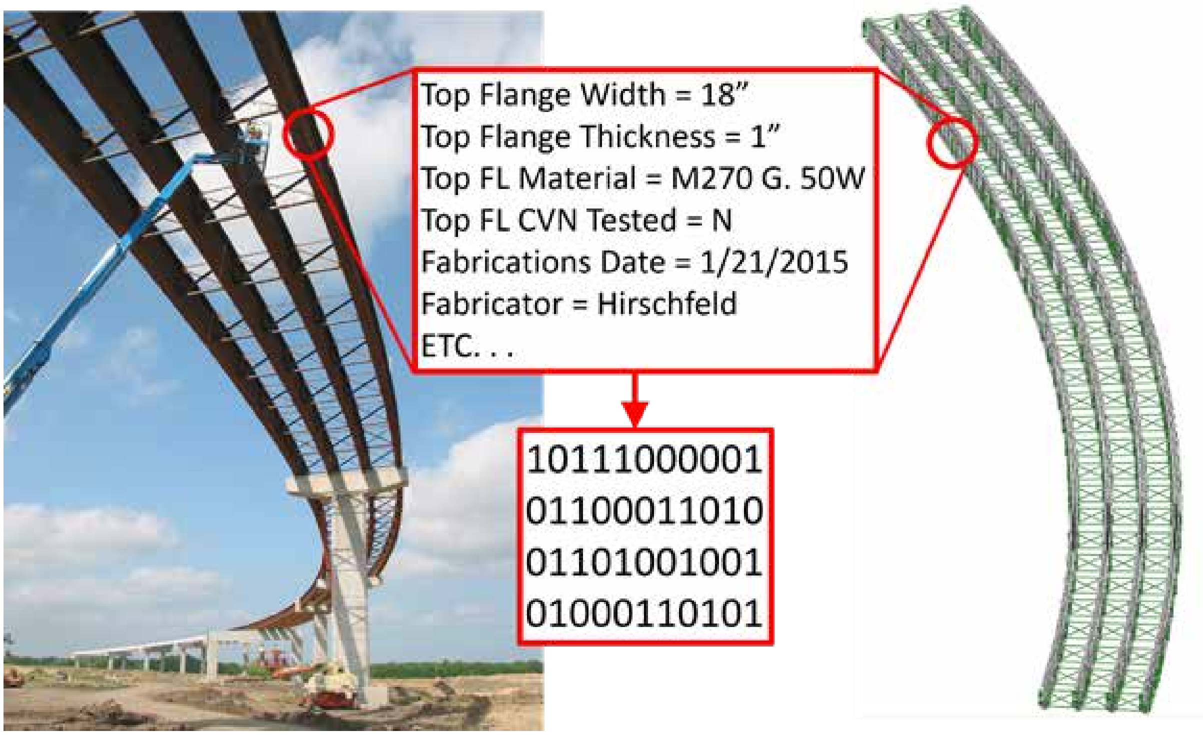
The concept of BIM for bridges and structures will be integral to the management of bridge projects in the future, and state departments of transportation (DOTs) and other owners are already beginning to see the potential asset management advantages of using BIM. BIM is not just a pretty 3D picture of a structure that one can fly through and use for marketing purposes. Rather, it encompasses the information that describes a structure, from conception through operation and beyond. Essentially, BIM *is* information. Behind the scenes of the representations of the model is data. To a computer, data is just bits and bytes—1s and 0s. Importance is placed on the *data that describes* the geometry, material properties, section properties, fabricator changes, coating systems, field changes, etc. Being able to use the information in a stand-alone fashion (e.g., structural design) can be useful.

However, an enormous potential exists to link that information to other stakeholders, such as designers to fabricators to contractors to maintenance/asset management tools. This sharing of information is known as information exchange, and the free and effortless exchange of information from one software to another is called interoperability. For the bridge industry to capitalize on interoperability to enhance asset management systems, a standardized scheme and method needs to be developed and adopted. As the National BIM Standard and the industry foundation classes (IFC) (**www.nationalbimstandard.org/**) provide the standards and methods for information exchanges in the building domain, so too must such standards and methods be developed for bridges. In order adopt a neutral format like IFC, the bridge industry must first develop standardized exchange requirements.

AASHTO/NSBA Efforts

The National Steel Bridge Alliance (NSBA) is leading the way for standardization of steel bridges by developing exchange requirements for these structures. The effort to develop a standard has been going on for over a decade. Several years ago, AASHTO/NSBA began a task group (TG15) formed to focus on Data Modeling for Interoperability, headed by Dr. Stuart Chen. This group started to build a data set library and develop a graphical representation of the

Aaron Costin (**aaron.costin@ufl.edu**) is an assistant professor at the University of Florida's M.E. Rinker, Sr. School of Construction Management, and **Jason Stith** (**jason.stith@mbakerintl.com**) is a bridge technical manager for Michael Baker International. Both are chairs of AASHTO/NSBA Steel Bridge Collaboration task groups.



Bridge as binary data.

bridge life-cycle (process map). Formed as a pilot study, the TG10/TG15 subcommittee worked with erection engineers to determine a model for this standardization process. Over the course of two years, the authors of this article led the subcommittee of dedicated volunteers, including Ron Crockett, Steve Percassi, Jon Stratton, Rob McKenna, Jon Gast, Ronnie Medlock, Hanjin Hu and others. The group developed an information delivery manual for steel bridge erection engineering that identified the erection engineering exchanges needed for interoperability. Currently, the AASHTO/NSBA database has grown to more than 2,000 unique entities that can be specified for any given exchange. This bottom-up approach to BIM standardization is an important distinction that uses bridge industry experts rather than BIM experts to define the necessary information to be exchanged. Several lessons were identified, including detailed assumption and standardized formats, which would enable future work to be completed faster and more purposefully. Currently, TG15, chaired by Samy Elsayed, is modifying final deliverables per comments provided by AASHTO/NSBA members.

Latest FHWA Push

Since standardization of the data scheme needs to be at the national level, and not state DOT-specific, the Federal Highway Administration (FHWA) has an integral role to play. One of the most notable voices in the realm of BIM for bridges and structures is Brian M. Kozy, principal bridge engineer at FHWA. Kozy has been a staunch advocate in moving the industry toward a BIM-based project development and asset management approach. In a recent discussion with Kozy, he stated that there are two global benefits in adopting BIM for bridges and structures:

1. "When engineers produce and maintain a BIM model, this is fundamentally providing a product that has much greater value to the owner and other stakeholders downstream. Anyone who has need of information about the bridge can benefit when a 3D model has been used, from engineer to fabricator to contractor to owner to inspector and beyond."
2. "BIM-based workflow fundamentally advances the way that engineering is done for the bridge. Engineers and other stakeholders can invest more of their time on developing the optimal solution for the project rather than wasting time on the data management and other book-keeping aspects of the project."

FHWA, along with 13 state DOTs, has pledged to finance a pooled-fund study lead by Ahmad Abu-Hawash of IowaDOT and backed by SCOBS T-19 (chaired by Scot Becker of WisDOT) (**www.pooledfund.org/details/solicitation/1450**). With a current total of $1.24 million pledged, the FHWA and these DOTs are committed to moving the practice forward by taking the recommendation from the recently completed NCHRP 20-07, Task 377, led by Michael Baker International, which outlined the steps to develop and implement BIM within the bridge industry.

Looking Toward the Future

As this transition to a digital, model-based workflow transpires, the members of the AASHTO/NSBA committee will be advising and providing recommendations from lessons learned on behalf of the steel bridge industry. In the not-too-distant future, when BIM for bridges and structures is realized to its full potential, much of the information transferred via engineering drawings will be exchanged electronically.

What does this mean for engineers? The designers will provide the camber information, which fabricators will be able to transfer into

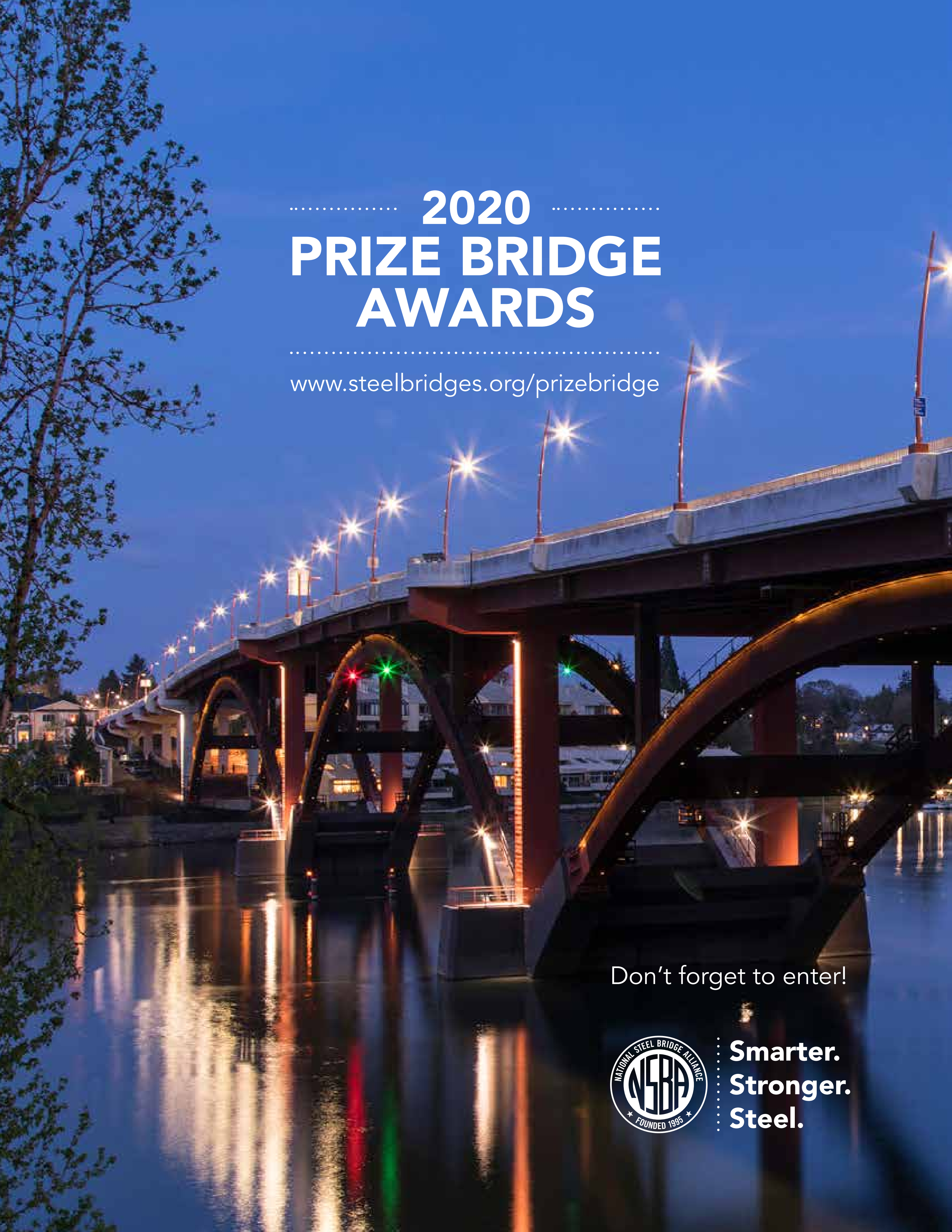
C.Y. Yong

Jason Stith

detailing software without the risk of typos or time-consuming data entry. Another area fabricators have identified where BIM will assist is the bill of material sheet necessary for ordering steel. Identifying and keying in this information takes time and resources and introduces a risk for error. CNC machines need information about the bridge geometry that could originate from the designer and be refined by the detailer. Quick and efficient data exchanges would reduce cost and provide meaningful advancement over current practices. Later, the bridge lifecycle load raters and consideration for overload permits need much of the same information as the original designers, in addition to as-built and bridge inspection conditions, which, if stored electronically in a standardized format, would expedite accurate ratings. The list of possible improvements is vast, but it is fair to say that perpetually scarce transportation funding and resources will necessitate this kind of innovation.

Significantly, tangible benefits via a model-based approach have been proven in the building domain, such as reduced errors, shortened schedules, decreased projects costs and increased profits. However, in order to realize the full potential of BIM for bridges and structures, it's essential for all stakeholders to band together to provide collaboration and alignment. BIM has been successful because it has been driven by the building industry, and those of us in the bridge industry must drive the effort for our projects as well. ■

This article is a preview of Session B8 "Reducing Errors in Bridge Drawings—What You Can do Today and Look to in the Future" at NASCC: The Steel Conference, taking place April 11-13 in Baltimore. Learn more about the conference at ***www.aisc.org/nascc****.*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2018 PRIZE BRIDGE AWARDS

AMERICA'S BEST STEEL BRIDGES have been honored in this year's Prize Bridge Awards competition. Conducted every two years by the National Steel Bridge Alliance (NSBA), the program honors outstanding and innovative steel bridges constructed in the U.S. The awards are presented in several categories: major span, long span, medium span, short span, movable bridge, reconstructed, technological advancement, integrated project delivery, special purpose, accelerated bridge construction and sustainability. This year's 16 winners, divided into Prize and Merit winners, include a first-of-its-kind tub girder solution in Ohio, a remote crossing in the frigid far north of Alaska and a modest double-counterweight design in a coastal Maine town. Winning bridge projects were selected based on innovation, aesthetics and design and engineering solutions by a jury of five bridge professionals.

This year's competition included a variety of bridge structure types and construction methods. All structures were required to have opened to traffic between May 1, 2015 and September 30, 2017.

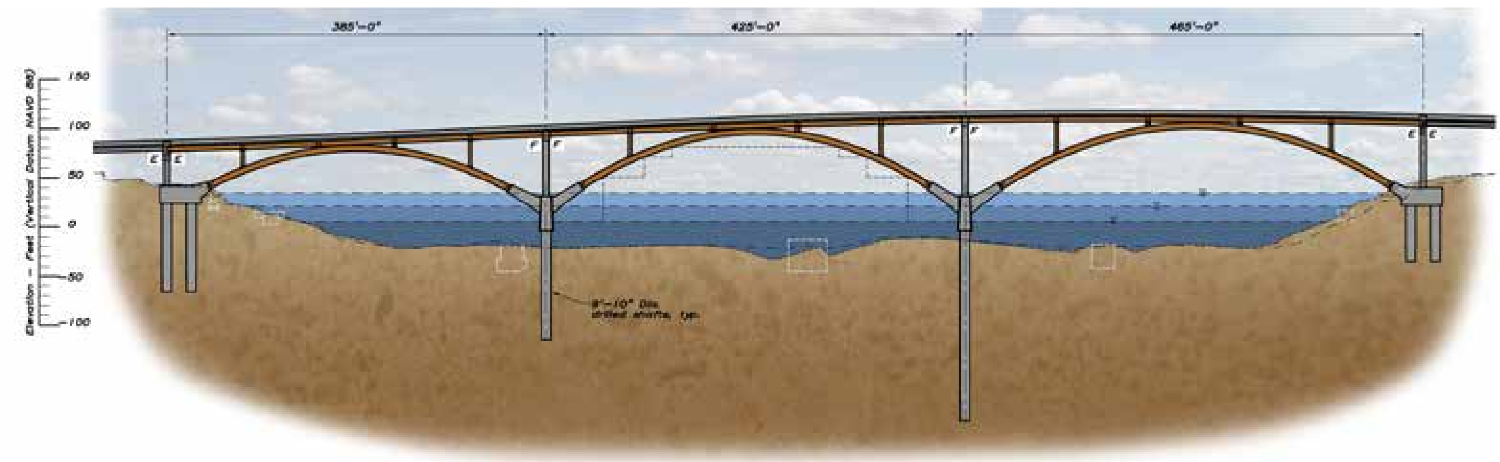
The competition originated in 1928, with the Sixth Street Bridge in Pittsburgh taking first place, and over the years more than 300 bridges have won in a variety of categories. Between 1928 and 1977, the Prize Bridge Competition was held annually, and since then has been held every other year, with the winners being announced at NSBA's World Steel Bridge Symposium. The following pages highlight this year's winners. Congratulations to all of the winning teams!

And check out past winners in the NSBA archives at **www.steelbridges.org**.

Judges

Amber Blanchard, PE, Minnesota Department of Transportation
Michael Culmo, PE, CME Engineering
Tony Hunley, PE, PhD, Stantec
Frank Russo, PE, PhD, Michael Baker International
Dominique Shannon, PE, Kansas Department of Transportation

A look at the innovative, efficient and eye-catching winners of this year's Prize Bridge Awards competition.



ELEVATION

MAJOR SPAN | NATIONAL AWARD

Sellwood Bridge, Portland, Ore.

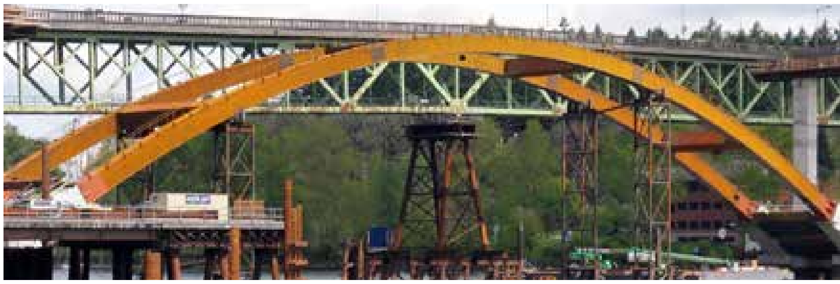
IN A CITY known for its bridges, Portland, Oregon's Sellwood Bridge over the Willamette River stands out amongst the rest for its elegant, flowing lines and low profile. It is also the city's first steel deck arch structure and currently the only structure crossing the Willamette River that meets modern seismic requirements.

The original Sellwood Bridge opened in 1925 and became the busiest two-lane bridge in the state. In 2005, it received a National Bridge Inventory sufficiency rating of 2 out of a possible 100. This low rating was attributed to critical issues such as deterioration of the approach structures and extensive cracking in the concrete girders, which was caused by a nearby hillside that was slowly sliding into the river, thus exerting pressure on the west end of the bridge.

Using 5,000 tons of structural steel, the new 1,976-ft-long replacement bridge—with 1,275 ft comprising the steel deck arch—features three arch spans that support the 63-ft-wide to 90-ft-wide deck of the main river spans. The bridge carries two 12-ft-wide vehicular lanes, two 6.5-ft-wide bike lanes/emergency shoulders and two 12-ft-wide shared-use sidewalks, and will also accommodate future streetcar service. The project included the replacement bridge, modernization of the Highway 43 interchange and stabilization of the hillside located west of the bridge and interchange.

As the profile for the new bridge ascended from west to east, the arch spans could not be in balance geometrically, which affected the differential thrust across the interior piers. The three arches have a rise-to-span ratio that varies from 1:7.7 (0.13) to 1:6.4 (0.16). The arch framing was scaled to trend with the profile, allowing the necessary balance at the piers. This same geometry for a deck arch resulted in rather shallow arches—shallower than structurally optimum for a deck arch. Bending moments from dead load for fixed arches would have dominated the arch size, creating a heavier arch profile. Through an innovative use of advanced ultra-high-molecular-weight (UHMW) plastic hinges, engineer T.Y. Lin designed the arch ribs to be pinned during construction, allowing rotation of the rib springing. Connections were designed to allow grouting and bolting of the springing at completion of the bridge for the desired continuity needed for live and seismic loadings.

Another challenge was how to replace a major transportation route with minimal traffic disruption. The original plan was to build the replacement bridge in two halves, using the existing



The bridge is a great blend of functionality and aesthetics. The three-span deck arch superstructure creates a signature structure for Portland while also working to provide cross-river access in the event of a natural disaster. —Tony Hunley

span for traffic while the southern half of the new bridge was built first. Once the southern half was completed, traffic would shift onto it, allowing demolition of the old bridge and construction to begin on the northern half of the new bridge. However, this plan had its drawbacks, including the need for extensive staging to keep traffic moving and increased construction time and costs. As such, an alternative solution was developed. Once temporary steel piers were erected north of the existing bridge piers, the entire 1,100-ft-long steel deck truss of the old bridge, comprising four continuous deck truss spans, was slid over on rails to a detour alignment using hydraulic jacks. The horizontal distance of the move was 66 ft at the west end and 33 ft at the east end. Temporary approach spans at both ends of the truss were also constructed. This solution enabled the replacement bridge to be built in one phase on the original bridge footprint. Benefits included eliminating the need for staged construction; reducing the number of arch ribs from four to two; freeing up the existing alignment for work crews; enhancing public safety by removing traffic from the construction zone; and reducing bridge closure time to only six days.

Multnomah County also saved up to a year in construction time and as much as $10 million in project costs with some help from steel fabricator, Thompson Metal Fab. The company proposed piece-by-piece stick erection for the steel deck arch, with arch ribs placed on shoring towers rather than the float-in system originally considered for arch erection. Each rib span contained two bolted field splices to match the optimum weights chosen by general contractor Sundt Construction for fabrication and erection. This resulted in three segments per span, with segment lengths of up to 148 ft and weights of up to 146 tons each. Steel was transported to the site on barges and placed with cranes operating from work bridges and these barges.

Owner
Multnomah County, Ore.

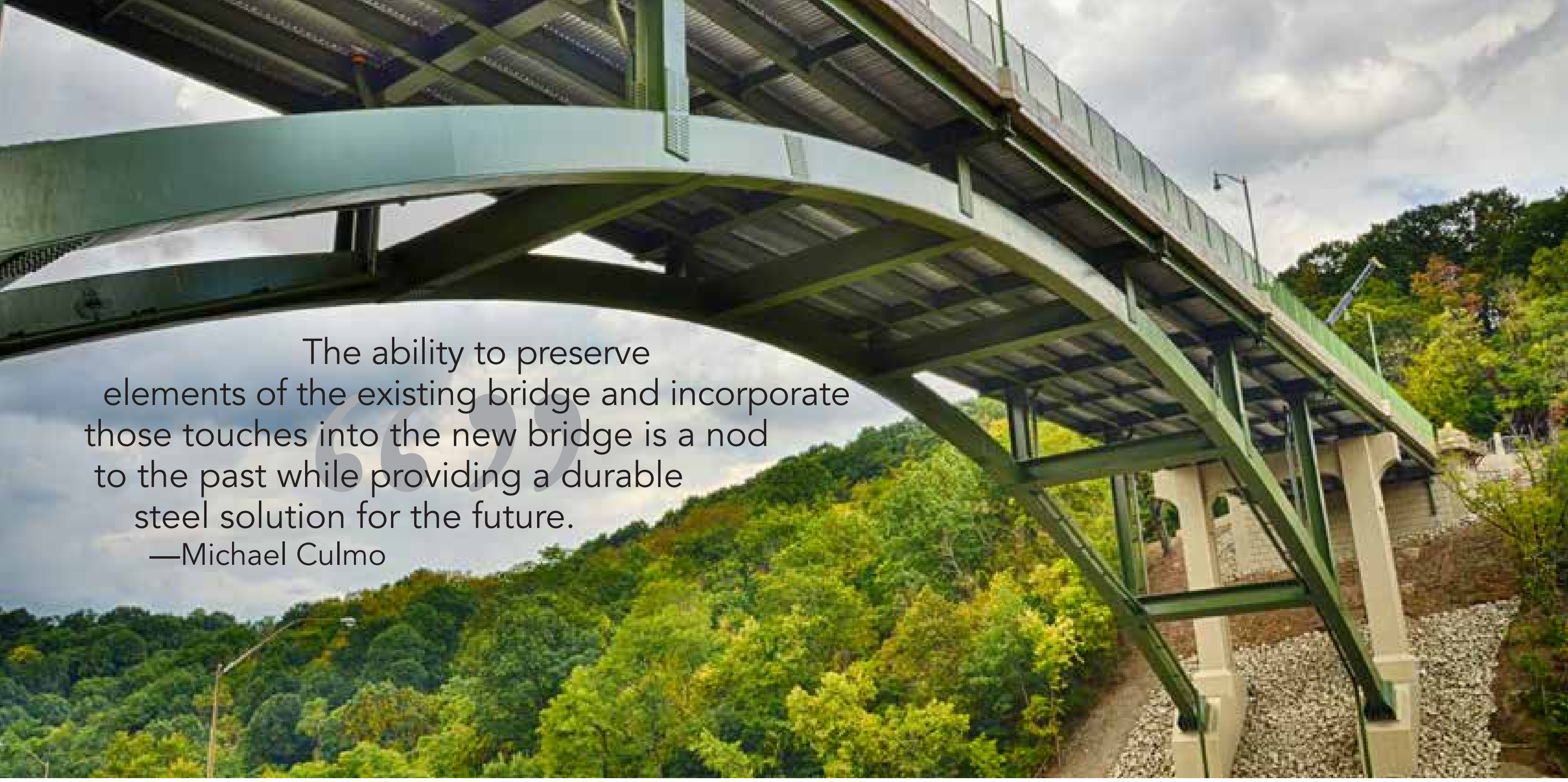
General Contractor
Sundt Construction, LLC

Structural Engineer
T.Y. Lin International

Steel Team

Fabricator
Thompson Metal Fab, Inc.

Detailer
Candraft AISC DETAILER



> The ability to preserve elements of the existing bridge and incorporate those touches into the new bridge is a nod to the past while providing a durable steel solution for the future.
> —Michael Culmo

LONG SPAN | NATIONAL AWARD

Greenfield Arch Bridge, Pittsburgh

THE GREENFIELD ARCH BRIDGE functions as a gateway arch to the greater Pittsburgh area for the more than 85,000 cars that pass under it each day. The structure takes numerous design cues from an existing structure and captures the historic nature of the area.

The existing reinforced concrete open-spandrel arch connected Greenfield to neighboring Schenley Park since 1921. Formally known as the Beechwood Boulevard Bridge and originally just spanning over a neighborhood street and stream, the structure eventually carried traffic over one of the busiest stretches of Interstate in Western Pennsylvania by the 1950s. Interstate 376, known locally as the "Parkway East," would grow under the bridge to become the main artery linking downtown Pittsburgh to all points east, and the bridge would become known as the city's gateway to the east.

In 2012, after years of study and consideration, it became apparent that it was time for a new structure and gateway. With an overarching goal to minimize impacts to the underlying Parkway, steel was selected for the new arch as it would simplify and expedite erection and minimize closure time of the important roadway.

The new bridge consists of a 287-ft open-spandrel arch span, with each rib of the arch consisting of just three field pieces of steel. Vierendeel bracing was used for the ribs to minimize members and connections. Additionally, the depths of the floor beams and stringers were matched to allow for simplified connections splicing the stringer flanges over the floor beam flanges. These design choices would allow the vast majority of the structural steel to be erected in just one weekend closure of the Parkway.

The piers were used to significantly reduce the effects of longitudinal loads such as braking and wind on the steel arch. Due to geometric and aesthetic requirements, the piers have significant strength and stiffness. By fixing the stringer/floor beam floor system to the piers and using expansion disc bearings at the shortest spandrel columns, the demands on the spandrel columns and associated connections were significantly reduced.

For a steel arch, the floor beams are traditionally defined as fracture-critical members (FCMs) requiring increased material testing and frequent in-depth inspections over the life of the structure. To eliminate the fracture-critical designation and eliminate the need for costly in-depth FCM inspections, the floor beams were detailed and designed as system-redundant members (SRMs); redundant members are those whose failure will not cause instability of the overall structure or loss of function.

The redundancy of the floor beams was achieved and verified through a combination of robust line girder analyses and detailed 3D finite element analyses (FEAs) accounting for the dynamic effects of a floor beam fracture. Line girder analyses were used to verify the ability of the stringers and their splices to effectively span two floor beam bays and also confirm the strength of the remaining floor beams to support the increased loads. The connections of the stringers to the floor beams were detailed to resist the high loads. In the event of a floor beam fracture, the stringer connection effectively becomes a splice at mid-span. The line girder analyses ignore the beneficial resistance of the remaining portions of the fractured floor beam and 3D load sharing of live load across the structure. In addition to the line girder analyses, 3D FEA simulations were used to capture the behavior of the structure during a complete fracture event.

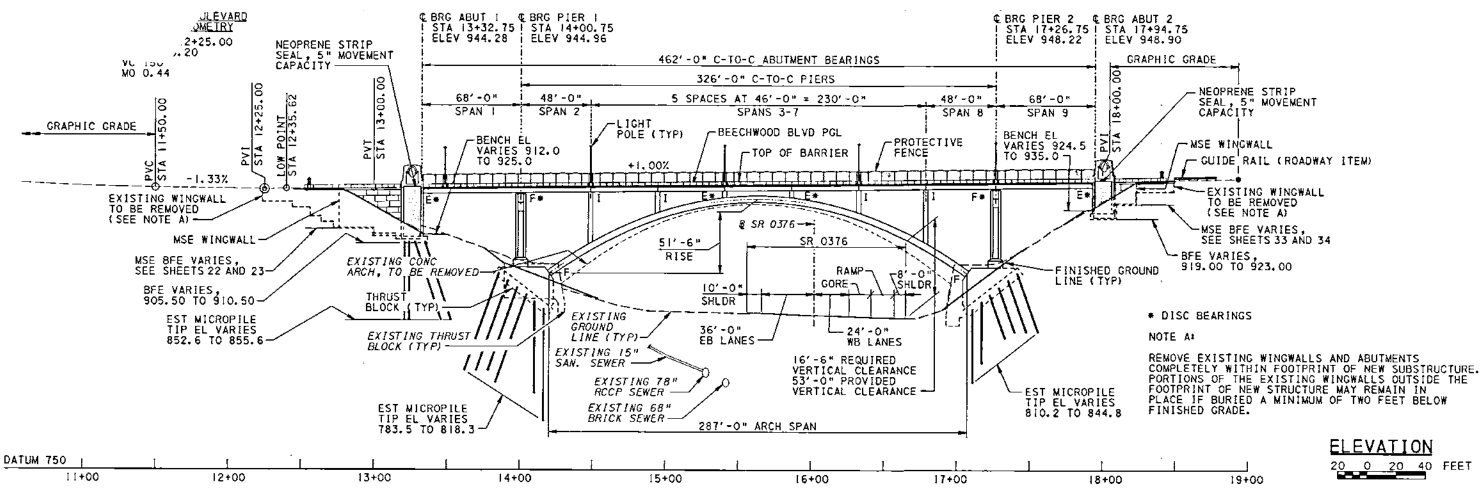
The analyses considered different levels of structural damping and fracture locations (floor beam mid-span and spandrel support). These parameters were varied to determine the appropriate increase in dead loads to account for dynamic effects. The parametric 3D FEA indicated that the simplified line girder analyses resulted in a design that provided more than adequate resistance during a fracture event.

Owner
City of Pittsburgh

General Contractor
Mosites Construction Company

Structural Engineer
HDR, Inc.

Steel Fabricator and Detailer
High Steel Structures, LLC



MEDIUM SPAN | NATIONAL AWARD

Colville River Nigliq Bridge, Nuiqsut, Alaska

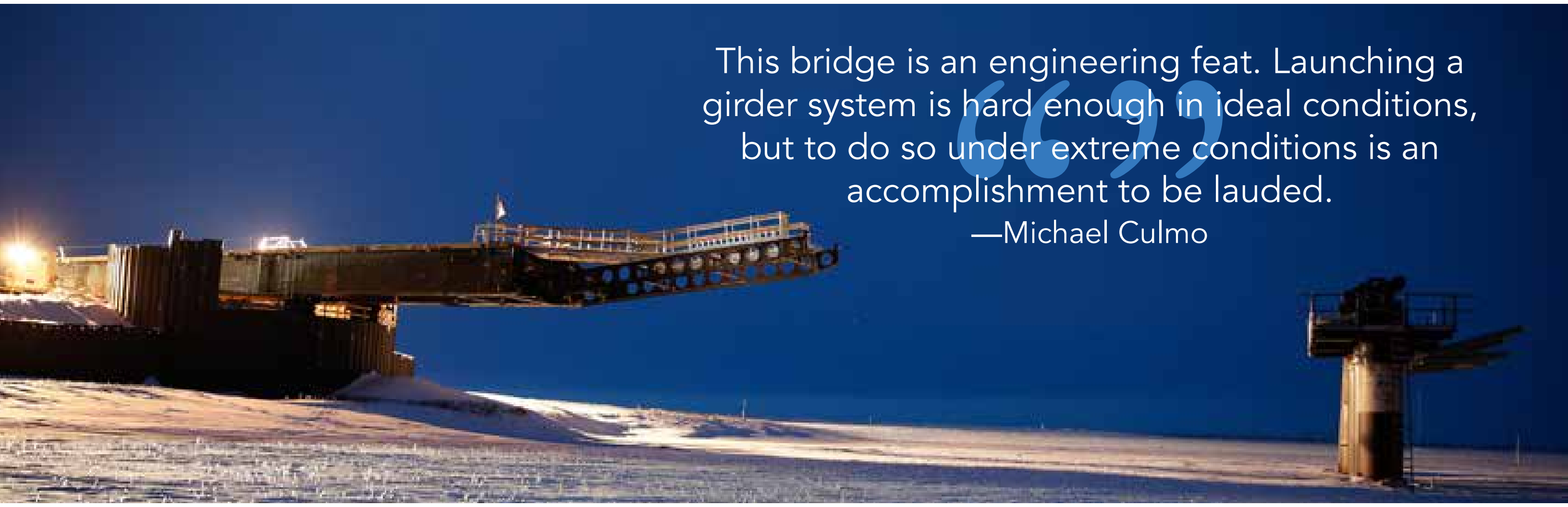
THE COLVILLE RIVER NIGLIQ BRIDGE is located almost as far north as you can go and still be in the United States, in an area where mammoth ice floes are no big thing.

Located on Alaska's remote north slope, the bridge provides daily transportation access for trucks carrying thousands of barrels of oil, and crossing the Nigliq Channel of the Colville River was one of the project's greatest challenges. The resulting bridge consists of a 1,421-ft steel box girder and a total of eight spans varying between 162 ft to 200 ft, and provides access to oil field service vehicles weighing up to 175 tons. The design accommodated these vehicles by using a 115-ton gravel hauler as the AASHTO Strength I design vehicle loading configuration and a 175-ton drilling services vehicle as an AASHTO Strength II loading condition. Additionally, an 87.5-ton fatigue vehicle was considered in the analysis in recognition of the high frequency of heavy loads that can result in premature fatigue-related issues.

The span-depth ratio of the bridge girders (almost 28:1) was optimized to accommodate the navigation requirement on the Nigliq Channel while achieving an efficient section both in service and during the launching operation. The bridge was engineered with twin fully enclosed steel box girders to support a total of six pipelines along the entire length on the downstream side, in protection against 6-ft ice floes measuring over 200 ft wide.

The bridge substructure was designed with sloped ice breakers that reduced horizontal ice loads from 2,600 kips (crushing) to 370 kips (bending). This design approach passes the large ice floes more efficiently than vertical piles by breaking up the ice in bending and has the added benefit of providing an appealing look to the upstream side of the bridge. The substructure of the bridge consists of 48-in.-diameter steel pipe piles constructed over varying degrees of permafrost and fully thawed geotechnical conditions.

Fully enclosed steel box girders were ultimately selected for the span because they efficiently supported the bridge superstructure as well as the pipelines, which imposed a torsional load from the cantilever of up to 22 ft off the downstream side of the bridge. The box girders provided a stiff and stable system for the launching operation as well as redundancy for the two-girder system. While atwo-girder system is typically considered non-redundant and, thus, fracture-critical, the project's owner—ConocoPhillips Alaska—wanted the system to avoid operational issues associated with fracture-critical systems. Therefore, an in-depth analysis was performed to demonstrate system redundancy through the use of torsional capacity of the individual girders and the ability of the moment-connected diaphragms to transfer the loads under a "fracture" scenario. Using this analysis, the majority of the structure was demonstrated to have load path redundancy, with only a cou-



> This bridge is an engineering feat. Launching a girder system is hard enough in ideal conditions, but to do so under extreme conditions is an accomplishment to be lauded.
> —Michael Culmo

ple of locations at the girder ends and expansion joints requiring fracture-critical designations.

The bridge abutments used the Open Cell sheet pile system to contain the gravel approaches and protect the vertical abutment piles. The system is constructed of flat sheet piles in a circular shape that does not close on the back side, providing for much more efficient construction than traditional closed-cell systems. Similar to closed cellular structures, horizontal hoop stresses are developed from the lateral soil pressure, but these stresses are resisted solely by soil friction along the rear sheet piles buried beneath the roadway.

Construction on Alaska's North Slope is bounded by a short 90- to 110-day window—commonly called ice road season—during the depths of winter when ice roads offer access to remote locations across the frozen tundra. In order to support the subsequent launching operation, the various foundation systems for the bridge had to be installed within this three-month window in which ice roads were passable and river ice was thick enough for heavy equipment operations. As a consequence of working in the arctic environment in the depths of winter, numerous challenges and complications were faced. Among these were workforce limitations due to the remote site, billeting constraints at accessible camps, maintaining production under extremely low temperatures (-45 °F, with wind chills reaching -65 °F), blizzards, limited infrastructure, equipment limitations due to extreme climate, extended darkness and limited access due to ice road restrictions. As such, extreme planning and contingencies were incorporated into the project to ensure the successful completion within the allotted window.

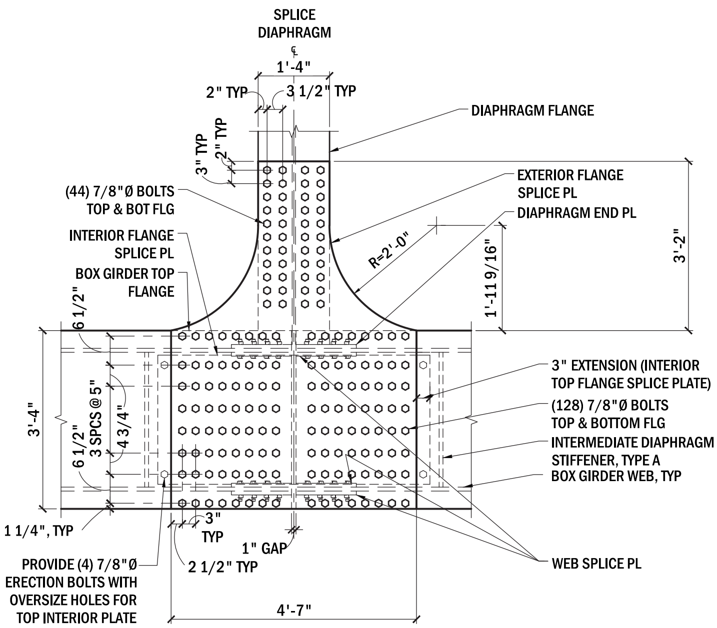
Due to the limited site access, the entire bridge superstructure was designed as prefabricated sectional steel spliced box girders for easy field assembly and launched to greatly reduce the project's environmental footprint and shorten the construction time frame. Bridge superstructure elements were brought onto the site during the first year ice road access season, during which the substructure elements were constructed. Most bridge segments were constructed in transportable units of less than 65 ft in length, with 14 segments fabricated in longer lengths and trucked using dollies. All other bridge components were transported on standard trailers. A "multi-season" insulated ice pad was then constructed to last through summer months as a storage site for these materials. The bridge girders and structural components were then assembled using bolted connections (a total of 10,000 bolts for the project) at the east abutment, incrementally launched on a custom hydraulic launch bed and lowered onto the bridge bearings.

Owner
ConocoPhillips Alaska, Inc.

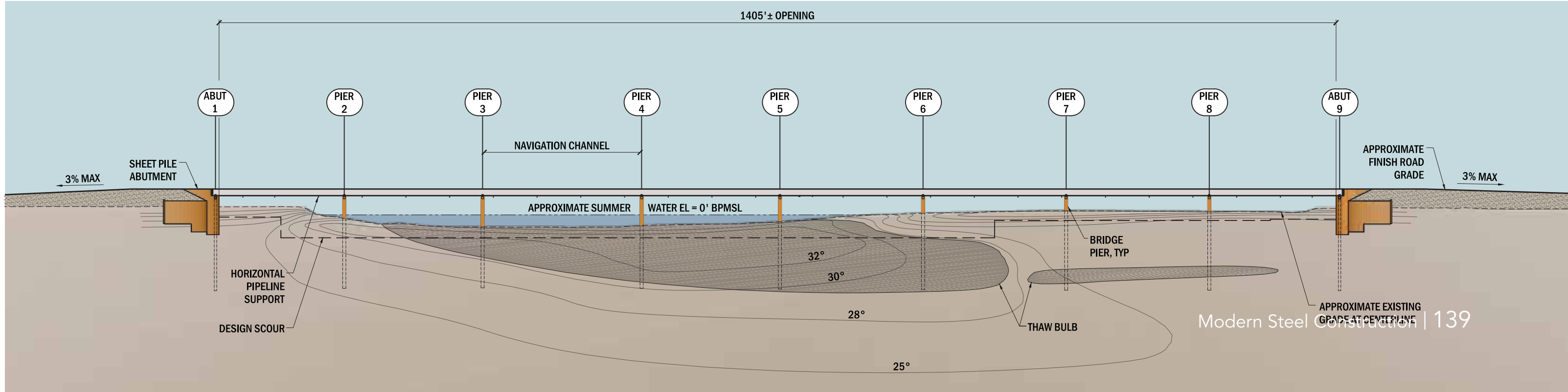
General Contractor
PCL Civil Constructors, Inc.

Structural Engineer
PND Engineers, Inc.

Steel Fabricator and Detailer
Jesse Co. AISC CERTIFIED FABRICATOR



DIAPHRAGM & FLANGE SPLICE PLAN



SHORT SPAN | NATIONAL AWARD

US Route 340, Waynesboro, Va.

THE ROUTE 340 BRIDGE was the first highway bridge in the United States to be constructed completely out of stainless steel, including ASTM A1010 Grade 50 (A1010) steel for the girders and cross frames and stainless steel fastener assemblies for the bolted splices.

A1010, a martensitic/ferritic stainless steel, was selected based on two concerns. The first was that the bridge is located within sight of an industrial plant, which caused the surrounding soil to be contaminated with mercury and potentially created a highly corrosive environment. The second reason was that the bottom flange of the bridge is located only approximately 9 ft above the South River, which the bridge spans. Both the proximity of an industrial plant and the presence of a low-level water crossing are conditions in which the use of uncoated weathering steel is discouraged. Since A1010 has a considerably slower corrosion rate when compared to traditional bridge steels, the bridge is expected to be low-maintenance throughout its entire service life, which translates into significant maintenance savings.

Although other A1010 bridges have been constructed in the United States, some of the innovative features of the Route 340 Bridge were the first of their kind in the nation. Previous A1010 bridges have used either ASTM A709 Grade 50W or galvanized Grade 50 steel cross frames, and ASTM F3125 Grade A325 Type 3, galvanized Grade A325 or ASTM A193 stainless steel bolts and accompanying fastener assemblies. The Virginia Department of Transportation (VDOT), the owner and designer, elected to use stainless steel for the cross frames and fasteners so that the entire bridge would be uniformly low-maintenance. Since structural shapes are currently not available in A1010, VDOT offered fabricator High Steel the opportunity to select either welded, built-up shapes or bent plates mimicking rolled shapes.

High Steel elected to fabricate all of the angles and channels used for the cross frames with bent plate. Prior to the Route 340 Bridge, A1010 steel had been bent on one other highway bridge project (0.16-in.-thick plates were used on a

"A1010 will provide the owner with a superior corrosion protection system while reducing their long-term maintenance costs."
—Dominique Shannon

bridge in Colusa County, Calif.). However, 0.5-in.-thick plates were required for the cross frames of the Route 340 Bridge. Since this was to be the thickest A1010 steel plate to be bent for a highway bridge project, the bends were examined using nondestructive testing (NDT) techniques, with successful results. As part of the haunched steel girder design, the bends on the flange plates were also tested and achieved passing NDT results.

The bridge is also the first to use stainless steel fasteners in a structural bolted splice application. To determine which types of stainless steel were most suitable for this bridge, a testing program was conducted on four types of stainless steel bolted fastener assemblies. Based on the test results, VDOT selected ASTM A193 Grade B8 bolts, ASTM A194 nuts and Grade 303 washers to be used for all bolted connections. The selection of the A193 bolts and accompanying fastener assemblies was finalized after a modified tightening procedure was developed to account for differences in mechanical behavior between these bolts and the Grade A325 bolts traditionally used on steel bridges.

In addition, a new type of welding consumable was also used on the Route 340 Bridge. Previous A1010 bridges had used 309L consumables, and VDOT planned to do the same. But due to an unforeseen business transition for the consumable supplier, VDOT ultimately identified and used a 309L-C consumable that complied with federal Buy America regulations. In order to approve the 309L-C consumables, additional NDT and weld procedure qualification record (PQR) tests were required, and passing results were obtained.

After the bridge was completed, ASTM A1010 was incorporated into the ASTM A709 specifications as Grade 50CR steel. VDOT is developing a special provision to provide guidance on using Grade 50CR steel for both new designs and repair strategies for steel bridges moving forward.

Owner
Virginia Department of Transportation

General Contractor
Fairfield-Echols, LLC

Structural Engineer
Virginia Department of Transportation–Staunton District

Steel Fabricator and Detailer
High Steel Structures, LLC AISC CERTIFIED FABRICATOR

MOVABLE BRIDGE | NATIONAL AWARD

Gut Bridge, South Bristol, Maine

WORKING ON A PROJECT SITE about the size of two football fields was just one of the challenges for the team working on the Gut Bridge. Fitting within an approximately 300-ft-diameter circle, the project site was constrained by three buildings occupying the northwest, south west and southeast corners of the site. Thankfully, a double-counterweight design allowed the structure to fit within the site while blending seamlessly into the surrounding town.

The project replaced a failing 80-year old Bobtail Swing Bridge with a new cable-stayed bascule bridge across the Gut, a small channel that separates Rutherford Island from Bristol, Maine, and serves as a passage for boats hoping to avoid circumnavigating the island. The Bobtail Bridge had undergone multiple mechanical failures and the process of replacement had been underway for almost a decade. The bridge opens more than 8,000 times per year for the heavy boat traffic, so reliable operation is a necessity. The final bridge concept for a new crossing was developed with the local community's input and includes the bridge replacement (including foundations), construction of an operator's house, new traffic warning systems, approach work and a temporary runaround.

The Gut Bridge superstructure used a combination of innovative details that formed a new type of bascule span. The cable-stayed superstructure allowed the span to be efficiently counterweighted, resulting in a low bridge profile and reduced foundation costs, and using steel orthotropic deck minimized structure depth and weight (the roadway profile is low to the waterline, so a shallow superstructure was a necessity) thus reducing fabrication and operational costs.

It was challenging to develop a concept for the new bridge that provided dependable operation while also being aesthetically pleasing. The bascule configuration was selected to facilitate reliable operation, as it reduces demands on machinery. It functions as a simply span when seated for vehicular traffic and opens quickly for navigation traffic—important because again, it sees more than 8,000 openings per year. The cable-stayed configuration for the superstructure allowed the span to be efficiently counterweighted (balanced) and eliminated the need for an overhead cross counterweight, leaving an aesthetically pleasing appearance. The upper stays contain balance materials along with the back leg of the girder, and the rear portion of the bridge (stays and back legs) opens into slotted pits. The cables provide additional support to the forward leg of the girders, reducing the overall size of the superstructure, and pay homage to an early bridge at the site that was also cable-stayed. The height of the stays fits well to adjacent building structures, and drivers have a clear view between the stays because there is no overhead counterweight. And the slanted cables add a dynamic element to the bridge even when it is seated.

As South Bristol Harbor is busiest during the summer months, navigation through the Gut could not be restricted during this high season, and construction was primarily limited to two winters. Despite these restrictions, construction was completed on time.

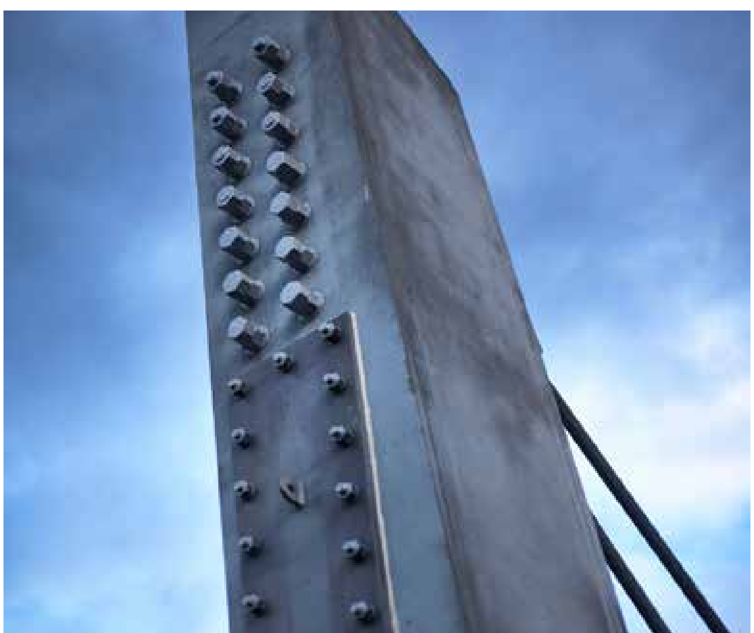
Owner
Maine Department of Transportation

General Contractor
Cianbro Corporation

Structural Engineer
Hardesty & Hanover

Steel Fabricator and Detailer
Steward Machine Co., Inc. AISC CERTIFIED FABRICATOR

Using a double-counterweight was a very creative way to save space and provide a solution that fits the site while also providing functionality.
—Frank Russo



RECONSTRUCTED BRIDGE | NATIONAL AWARD

RFK Bridge—Manhattan Approach Ramps, New York

WHEN BUILDING A NEW BRIDGE proved too costly, steel's ability to be retrofitted provided the economical solution for the RFK Bridge Manhattan approach ramps. Aside from localized deterioration, the steel superstructure of these approach ramps was in good structural condition but did not meet modern seismic requirements. Because of the bridge's importance as an evacuation route, the steel superstructure was retrofitted to include a seismic isolation system, thus extending the useful life of the bridge dramatically.

Prior to this project, no major rehabilitation had been performed on the Manhattan approach ramps to the RFK Bridge (officially known as the Robert F. Kennedy Harlem River Lift Bridge) since its original construction in 1936. Due the effects of chronic deterioration, primarily at the locations of leaking expansion joints, a comprehensive rehabilitation effort of the steel portion of the structure was required. A number of unique and innovative methods were necessary to successfully complete this project within the congested urban environment—all while maintaining traffic on these critical ramps, which carry in excess of 85,000 vehicles per day.

The original structure consisted of concrete "cellular" spans near grade transitioning to steel rigid-frame spans for the elevated portions. While the concrete spans had significantly deteriorated and were in need of replacement, the steel spans were in relatively good condition (aside from localized deterioration near the expansion joints) making them a good candidate for rehabilitation. The steel structure consists of a non-redundant, two-girder system with typical span lengths of 60 ft framing directly into steel rigid-frame pier bents. The original structure does not use expansion joints; expansion and contraction is instead accommodated by flexure of the steel columns (which are oriented for weak-axis bending in the longitudinal direction of the bridge).

Due to this structure's importance as an emergency evacuation route, it is classified as a "critical" structure for purposes of seismic analysis and must be able to resist a 2,500-year return period event with minimal damage. "Extreme event" loads of this magnitude were not explicitly considered during the original design of the structure. As expected, the results of initial analyses indicated that numerous structural elements had inadequate capacity

Floating a new superstructure above the existing bridge was a solution that could only be provided by steel, and saved the owner hundreds of millions of dollars while creating a lifeline bridge that would function in the event of a natural disaster.
—Amber Blanchard

to resist these loads. These initial analyses showed that the existing steel rigid-frame pier columns benefited from the fact that they are rather flexible under lateral loading, resulting in relatively low seismic demands. However, there were certain vulnerable regions, specifically the column-to-girder and column-to-floor-beam connections, that were overstressed during the modeled seismic event. Due to details that are relatively common for structures of this era (built-up members with riveted construction, etc.), strengthening of these vulnerable locations was found to be problematic and cost-prohibitive (if not altogether unfeasible) due to tight access and the labor-intensive nature of the work. Since the existing concrete deck was slated to be replaced due to the advanced level of deterioration present, a "floating deck" isolation system was developed to reduce seismic demands by isolating the new deck and floor system from the existing steel rigid-frame substructure below. The isolation scheme that was developed resulted in a reduction in seismic demands such that only a handful of strengthening retrofits were required—all of which were located at regions that were relatively easy to access.

Unlike most isolation systems—which typically use conventional lead-core or elastomeric isolation bearings—for this very flexible structure it was necessary to develop a hybrid system using both sliding bearings (typical PTFE-stainless steel interfaces) and elastomeric bearings. In this design, the sliding bearings carry all vertical loads and dissipate energy through friction, while only a small number of isolation bearings are needed, solely to provide the required restoring force. At the service load level, the friction developed at the sliding bearings resists the lateral design forces of wind, live load braking and live load centrifugal force (where applicable on the curved section of the on ramp). The details of the floating deck seismic isolation system also resulted in the removal of the majority of the expansion joints, thereby reducing the major source of deterioration (namely chloride-laden water infiltrating the steel) and creating a more maintenance-free structure.

The original replacement cost was estimated by some to be as high as $200 million, and such a project would have been more disruptive to traffic and the local community in this highly congested area of Manhattan. But the rehabilitation solution developed by engineer Modjeski and Masters cost roughly $70 million, of which only a small percentage went to the cost of isolation bearings and installation of the additional seismic retrofits.

Owner
Triborough Bridge and Tunnel Authority

General Contractor
DeFoe Corporation

Structural Engineer
Modjeski and Masters, Inc.

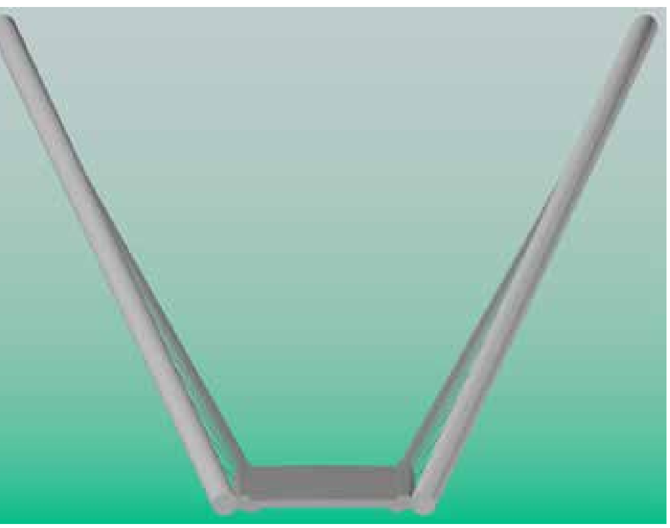
Peter Courtney Minto Island Bicycle and Pedestrian Bridge, Salem, Ore.

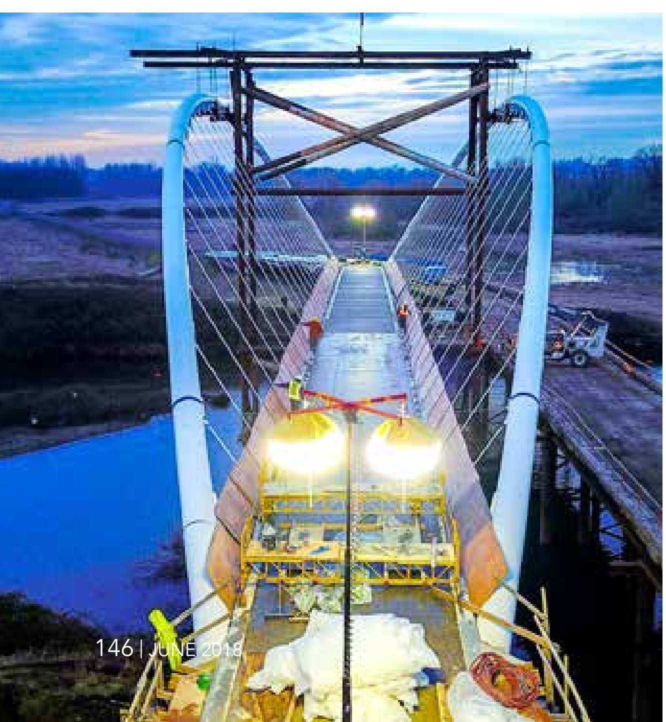
THE PETER COURTNEY MINTO ISLAND Pedestrian bridge is a 300-ft-long, tied-arch structure that links Riverfront Park with Minto Island in Salem, Ore. The structure blends seamlessly into the surrounding environment while providing trail access for outdoor enthusiasts.

The overall configuration is a five-span bridge with a main span steel tied arch of 304.5 ft at the spring-line chord and thin cast-in-place post-tensioned haunched slab approach spans, three at 50 ft on the island and one at 35 ft in Riverfront Park. The arches are a pair of 30-in.-diameter by 1-in.-wall round tube ribs, each bent to a hyperbola and tilted away from the deck at 25° from vertical and with no added lateral bracing.

The pair of tied arches supports a precast panel stress-ribbon deck. The precast panels are positioned on the arch ties, which are sequentially harped by the progressive panel placement sequence. Upon completion, the panels lie on a parabolic vertical curve profile. The precast panels also function as stay-in-place forms for a CIP post-tensioned topping slab. Full-length deck post-tensioning through the approach spans and the topping slab of the arch span provides capacity to the approaches and precludes live load tension at the arch span precast panel joints. Cast-in-place haunched panels connect the precast panels to the arch piers at the arch span ends. The arch-span deck is suspended from the arch ribs by 1.5-in.-diameter high-strength rods. These are arranged such that when projected onto the rib plane-of-curvature, they are radial to a point 490 ft above an arc passing through the ends of the arch spring-line chord. The completed arch-span deck, while very slender, is resistant to pedestrian-induced resonant vibrations and wind-induced aeroelastic flutter.

Meeting the combination of geometric, physical and administrative constraints necessitated an atypical design solution in a unique integration of multiple bridge engineering technologies. A pair of steel tied-arches on minimal size piers with a main span floor system depth of 15.4 in. from deck soffit to finish grade addressed the flood clearance, short approach length available to match grades in Riverfront Park, ADA maximum grades and the





The inclined arches are a unique solution to a long-span tied arch. The innovative approach to the arch ties provided a slender, aesthetically pleasing structure.
—Michael Culmo

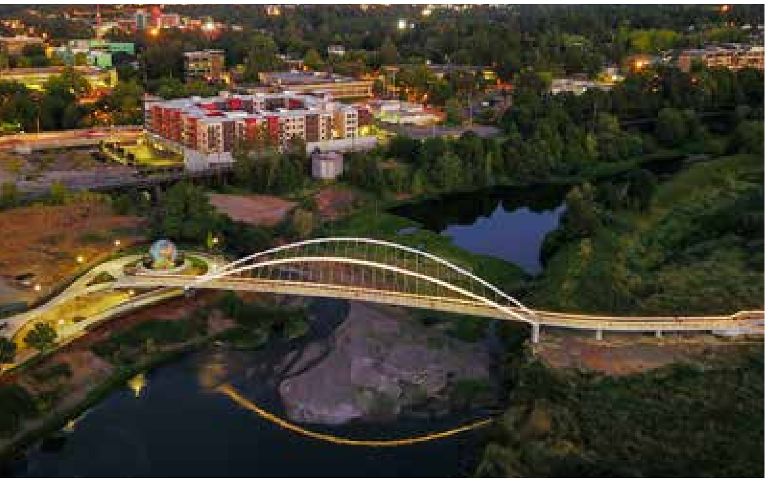
main span contribution to the FEMA "no rise" requirement. Each arch rib tilts outward from the bridge centerline 25° from plumb, with no cross bracing, to provide unlimited vertical clearance and an open feeling for bridge users. For the piers at the arch-span ends, the pier edges match the 25° tilt angle of the arches to form inverted delta piers that converge on the single shaft. The balance of forces between the self-weight of the rib plus the vertical component of suspender tension and the horizontal component of the suspenders lying slightly out-of-plane to the rib plane yields braced behavior that would not occur with the rib plane-of-curvature being oriented vertically.

The arches were fabricated off-site from A572 Gr. 50 1-in. plate formed and welded into tube with single-seam, double submerged arc welds in conformance to ASTM A252 Grade 3. The tubes were heat-bent to their specified hyperbolic shape, which follows the thrust line of the rib self-weight and the normal and tangential force components of the radial suspenders projected onto the rib plane-of-curvature. The radial suspenders impart a longitudinal force in the precast panels toward mid-span, which facilitated sequential placement of the panels outward from mid-span. For the center panel at mid-span, the suspender angles are mirrored on opposite ends of the panels for zero net horizontal force. Suspenders connect to the arch ribs via truncated angular plates, with the base against the rib fitting the skewed cross-sectional curvature. Because the rib is both curved and tilted, the suspender connection plates are attached at variable angles to the rib axis as tilt relative to the rib axis in the plane-of-curvature, rotation about the rib axis and twist relative to the rib plane. The fabricator created an independent 3D model for preparing shop drawings, with data for the connection plate fabrication and attachment angles in agreement with the design model. To verify that the plates were attached as intended and the rib segments matched the hyperbolic curvature for their positions along the arch, 3D laser scans were conducted in the fabrication shop. As part of the inspection and acceptance process, the scans were sent to the design engineer for comparison to the design model.

Owner
City of Salem, Ore.

General Contractor
Legacy Contracting, Inc.

Structural Engineer
OBEC Consulting Engineers



INTEGRATED PROJECT DELIVERY SPECIAL AWARD

Rouchleau Mine Bridge, Virginia, Minn.

IN PREPARING FOR the expiration of a land agreement between the state of Minnesota and a local mining company, set to expire in 2017, the Minnesota Department of Transportation (MnDOT) began plans for a new bridge. The new 1,100-ft weathering steel plate girder Rouchleau Mine Bridge sits 180 ft above the floor of the Rouchleau Mine pit and connects the mining towns of Virginia and Eveleth, Minn., on State Highway 53. Due to challenging site conditions, the three-span structure features a 480-ft main span and haunched girders with depths ranging from 7 ft, 9 in. at mid-span to 14 ft, 6 in. over the piers. Pier depth was limited to facilitate shipping to the rural area, and the girders were shipped horizontally. All steel was GR50 weathering steel.

To meet the project deadline, the bridge was designed and built under an accelerated schedule using the construction manager/general contractor (CMGC) delivery method. In addition to the fast-track schedule, the project was also challenged with near-vertical rock walls, mine waste rubble to depths of 120 ft below the pit floor, potential water elevation changes greater than 100 ft, the need to accommodate future mine blasting operations of the reactivated mine and construction over a lake that serves as the drinking water supply for the town of Virginia.

The design team worked collaboratively with MnDOT and the CMGC to validate the structure type and develop delivery strategies to directly address the project risks associated with schedule, the northern Minnesota weather and the unique terrain of the open pit mine. The project was delivered in two packages: an early steel package and a substructure/deck package. Structural engineer Parsons included detailer Tensor Engineering as a design team member to provide draft shop drawings as part of the bid package to minimize bid risks, facilitate mill orders and ultimately expedite fabrication. Within 52 days of Notice to Proceed, Parsons delivered the complete plans for the 5,200-ton superstructure. The remainder of the bridge package was delivered within the overall seven-month schedule, and the bridge opened this past summer.

Owner
Minnesota Department of Transportation

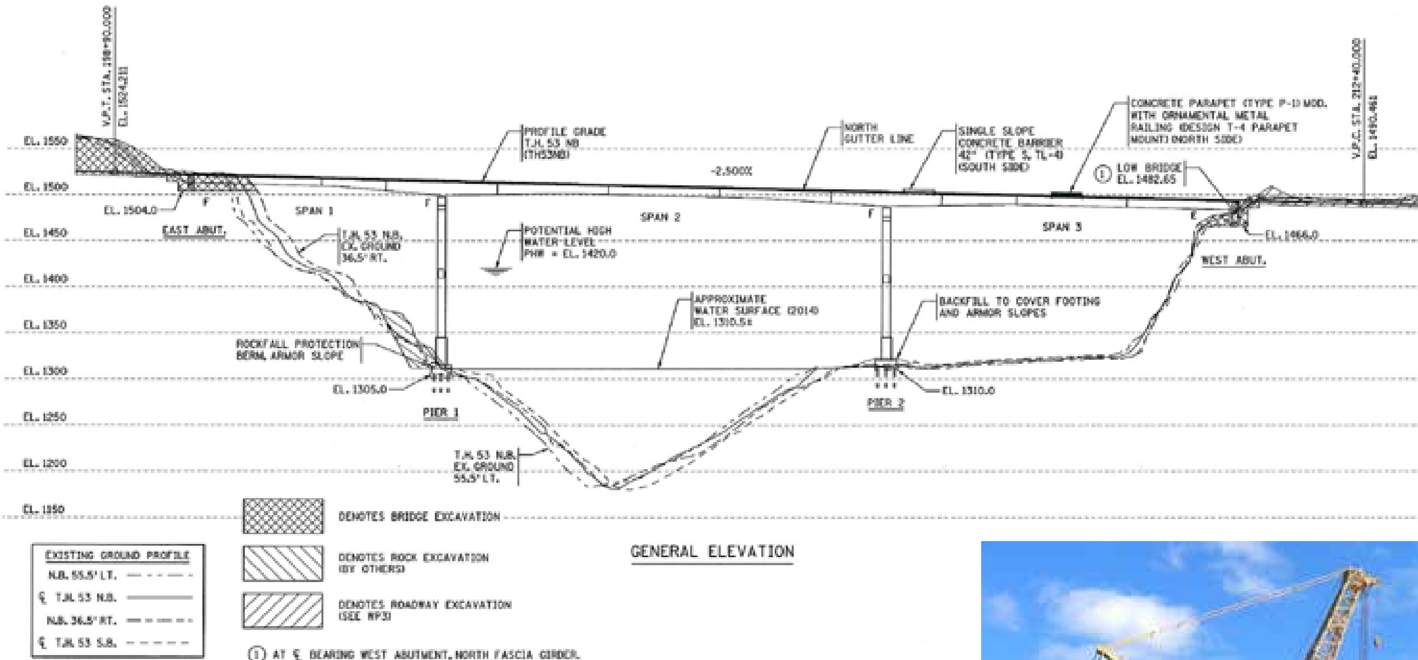
General Contractor
Kiewit Construction

Structural Engineer
Parsons Corporation

Steel Team

Fabricator
Veritas Steel AISC CERTIFIED FABRICATOR

Detailer
Tensor Engineering AISC DETAILER



When it comes to the CMGC project delivery process, steel is a real winner because of its ability to be delivered quickly and reliably.

—Tony Hunley

> Using innovative construction practices to beat out conventional construction methods goes to show that engineers shouldn't accept the status quo.
> —Frank Russo

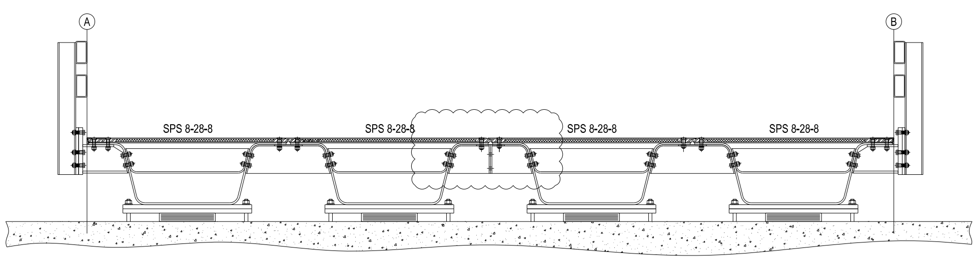
TECHNOLOGICAL ADVANCEMENT | SPECIAL AWARD

Folded Steel Plate Girder Bridge, Muskingum County, Ohio

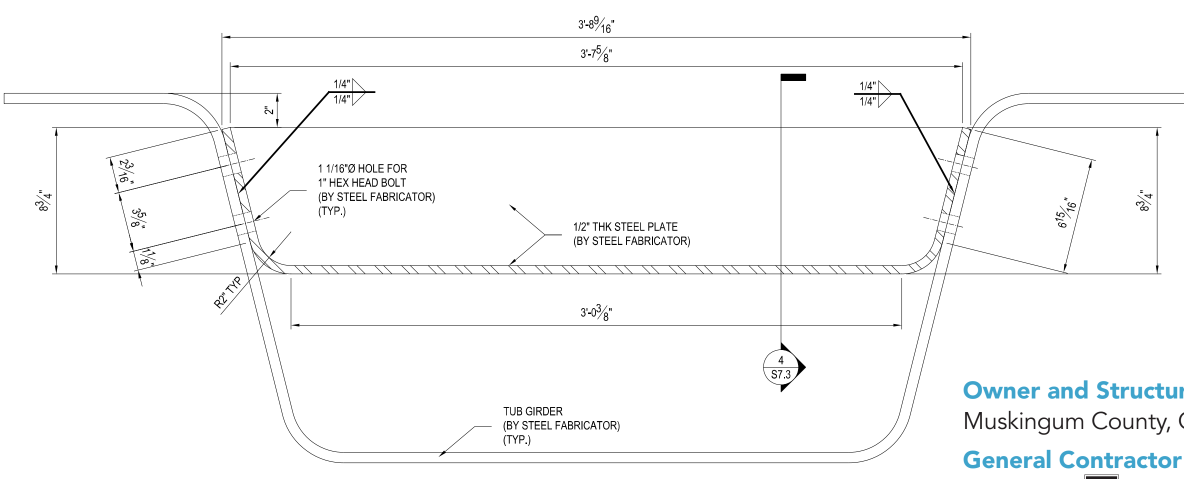
MUSKINGUM COUNTY, OHIO, broke new ground in the short-span bridge market with the first press-brake steel tub girder structure to use a steel Sandwich Plate System (SPS) deck. The system uses two steel plates that are bonded to a compact polyurethane elastomer core. The elastomer, as a two-part liquid, is injected into closed cavities formed by the steel face plates and perimeter bars. This approach effectively lowers dead loads and eliminates the time required to cure a concrete deck. The all-steel solution saved the community from a lengthy closure, while the shallow superstructure and lack of a horizontal surface is a big benefit to the owner because of the structure's location over a stream that is prone to flooding.

As the bridge superstructure is one-of-a-kind, there were no traditional shop drawings or specifications to refer to. Hand drawings were used and CAD drawings were ironed out to make these new details available to future designers. The bridge was preassembled in two halves and set and bolted in place in one day; the whole project was completed in 26 days. This included removal of the existing structure and a H-pile foundation with spread footings and concrete cast-in-place abutment walls. Once the substructure was completed, the superstructure was set in place in just 20 minutes.

The short duration of the project eliminated the need for traffic to endure an eight-mile detour around the road closure. In addition, the design of the tub girder, using a smooth bottom, prevents stream debris from getting caught on the superstructure during frequent storms that cause local flooding in the area.



3 S8.1 ASSEMBLY ARRANGEMENT - COMPLETE BRIDGE
SCALE: 1/4" = 1'-0"



1 S7.3 TYPICAL INTERNAL TUB GIRDER STIFFENER
SCALE: 1 1/2" = 1'-0"

Owner and Structural Engineer
Muskingum County, Ohio, Engineer's Office

General Contractor and Steel Detailer
US Bridge

FOR THE STUDENT STEEL BRIDGE COMPETITION, each student team develops a concept for a scale-model steel bridge to span approximately 20 feet and to carry 2,500 pounds according to competition rules. This is likely the first time that students will use structural analysis software for a real project. They must determine how to fabricate their bridge and then plan for an efficient assembly under timed construction at the competition. Bridges are then load tested and weighed and are also judged on aesthetics at the event. For many students, seeing this project come to fruition is the pinnacle of their academic careers; a project that despite challenges and sometimes failures, provides them with valuable technical, practical and soft skills to carry forward into their professional careers.

Our Vision

Empower students to acquire, demonstrate, and value the knowledge and skills that they will use, as the future generation of design professionals, to contribute to the structural steel design community and construction industry in the United States.

Our Mission

Challenge students to extend their classroom knowledge to a practical and hands-on steel-design project that grows their interpersonal and professional skills, encourages innovation, and fosters impactful relationships between students and industry professionals.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION AISC FOUNDED 1921

Student Steel Bridge Competition

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- Become a national sponsor
- Sponsor a team
- Attend an event

www.aisc.org/ssbc

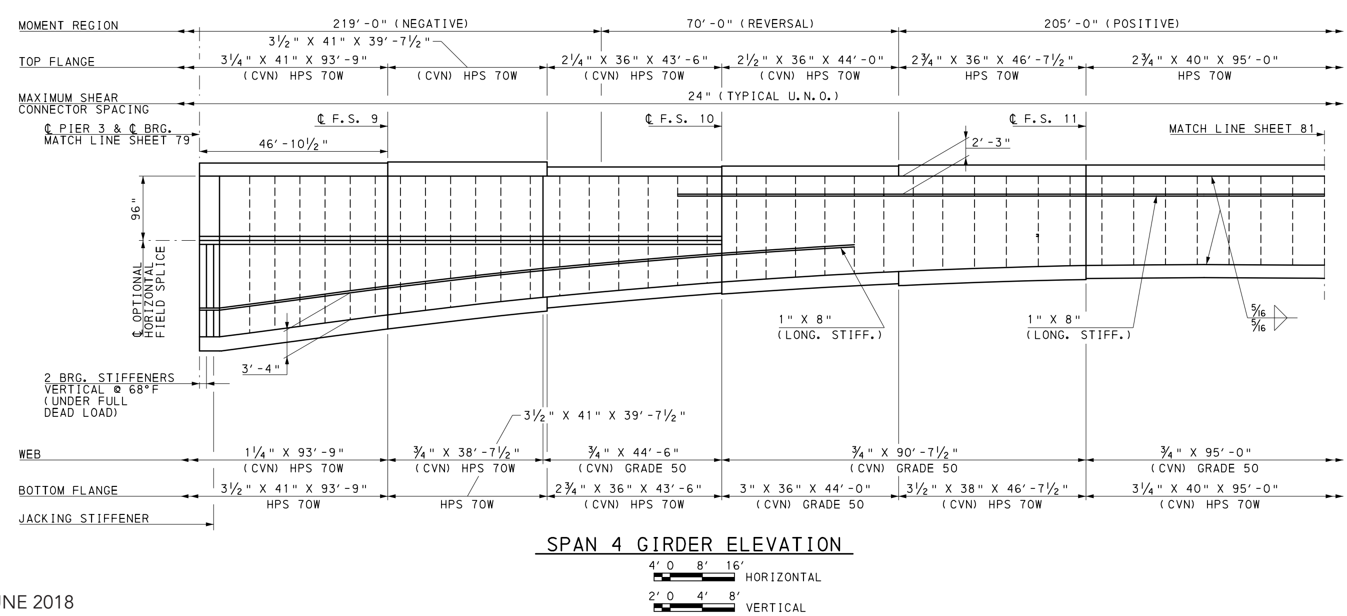
MAJOR SPAN | MERIT AWARD

Hulton Bridge, Pittsburgh

A WESTERN PENNSYLVANIA LANDMARK located 13 miles east of Pittsburgh, the Hulton Bridge provides a vital regional connection to the Pennsylvania Turnpike and State Route 28/910 interchange. With a main span of over 450 ft, the Hulton Bridge is proving that steel plate girders can be the economical choice for major-span bridges.

With average daily traffic counts increasing yearly, the condition and functionality of the original Hulton Bridge, a Parker Pratt Through Truss built in 1908, did not meet growing community traffic demands and was structurally deficient. To enhance the traffic level of service and meet regional mobility goals, PennDOT contracted Gannett Fleming and Brayman Construction Corporation to design and construct a new $65 million bridge. A signature achievement built to last well into the 22nd century, the enhanced 1,633-ft-long, multi-span, steel haunched girder structure improves safety, traffic flow and functionality.

The new bridge consists of a 100-ft single span over the Norfolk Southern Railroad and a four-span continuous section over the Allegheny River with span lengths of 274 ft, 479 ft, 500 ft and 275 ft. The railroad span was supported on a splayed substructure configuration to accommodate both the alignment of the railroad and skew of the river. This geometry required varying beam lengths for the eight constant-depth steel girders. The four-span superstructure was comprised of five haunched steel girder lines with varying web depths from 9 ft at the end supports to a maximum of 20 ft over the piers. The five girder lines spaced nearly 15 ft apart allow for four traffic lanes on the new structure compared to the existing two-lane bridge. The girders used a hybrid configuration by incorporating both 50-ksi steel and high-performance 70-ksi steel to optimize the design. Due to girder depths in excess of 14 ft, horizontal splices were designed and detailed to facilitate fabrication, delivery and erection. Additionally, longitudinal stiffeners were used to minimize web plate thickness for material efficiency. Top chord lateral bracing was provided between fascia and first interior girders to accommodate wind loading during construction.



The bridge is proportioned so well it masks the fact that the main span is 500 ft long. Coupled with an innovative strand jacking method of main-span construction, steel bridges again show their flexibility to adapt to any site. —Frank Russo

Preliminary studies considered five potential crossing locations covering a 3.5-mile stretch of the Allegheny River from Blawnox, Pa., to the State Route 910 interchange with State Route 28. The best solution to maintain the existing roadway network and connectivity across the Allegheny River was a new structure just upstream of the existing bridge on a skewed alignment from the original structure. The skewed configuration allowed the entire bridge superstructure to be constructed in one phase, which optimized the construction duration and minimized impacts to right-of-way and utilities.

The project team conducted studies to evaluate traffic capacity issues, pedestrian access, bicycle movements, accident data and master planning. As a result, the new upstream structure design features four 11-ft lanes, two 6-ft shoulders, a 4-ft median, a 5-ft sidewalk, an eastbound turn lane onto Allegheny Avenue and westbound right and left turn lanes onto Freeport Road that create more efficient traffic movements. The improvement in multimodal transportation mobility is a dramatic safety upgrade. While roadway safety features now meet current industry standards, future maintenance measures were also incorporated into the bridge design. For example, the bridge width and five girder lines allowed traffic to be maintained in both directions during re-decking. Construction crews also had enough room to safely perform maintenance as the bridge remained open to traffic. The new structure includes steel underdeck inspection walkways within the interior superstructure bays, as well as direct access to gas, electric and communication utilities supported by the superstructure.

Because of commercial traffic on the Allegheny River, the U.S. Coast Guard prohibited using temporary towers in the navigational channel and limited the channel closure to a maximum of 72 hours. The restrictions required an innovative approach to erect the 1,200 ton, 280-ft-long, 60-ft-wide closure section of the steel superstructure over the channel in less than three days. To meet this challenge, engineer Gannett Fleming proposed a strand jacking erection technique that had never before been used by the Pennsylvania Department of Transportation (PennDOT) on a steel girder bridge. (Strand jacks are hydraulic devices that use multiple steel cables, or strands, to lift large loads.) Lifting the navigational channel span into place required four 600-ton capacity strand jacks. The five-girder system was preassembled on barges and floated into place the night before the lift. At 10:00 a.m. the following day, the strand jacks, each using 36 ¾-in.-diameter strands, began lifting the span into place inches at a time. The segment was raised approximately 50 ft to its final elevation at 6:00 p.m. Once in final position, girders were secured so that the strand jacks could be removed and the channel reopened.

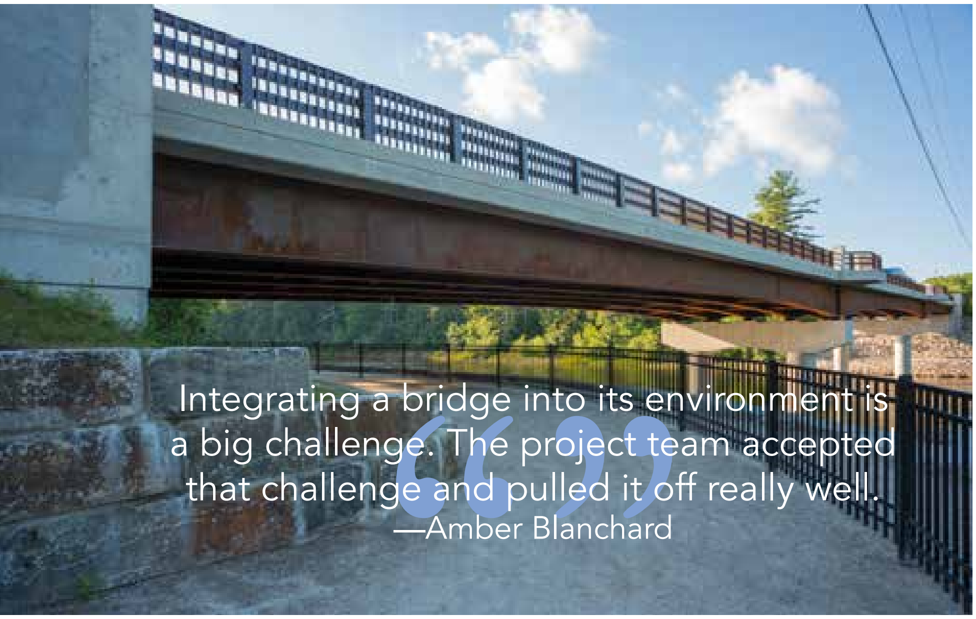
To safely open the navigational channel, 25% of the girders' 10,000 field splice bolts needed to be in place and fully torqued, and 50% were required to release and remove the strand jacks. All five girders were erected simultaneously, with only a 48-hour channel closure.

Owner
Pennsylvania Department of Transportation

General Contractor
Brayman Construction Corporation

Structural Engineer
Gannett Fleming

Steel Fabricator
High Steel
Structures, LLC AISC CERTIFIED FABRICATOR



MEDIUM SPAN | MERIT AWARD

Sewalls Falls Bridge, Concord, N.H.

CROSSING THE MERRIMACK RIVER, the Sewalls Falls Bridge in Concord, N.H., is a vital link to the local four-season recreational area. Designed for the next 100 years, the bridge pays homage to the 100-year-old truss bridge that it replaced by repurposing a number of bridge truss sections into the new Heritage Park.

Due to increased traffic volume and heavier truck weights, it was determined that the existing bridge had reached the end of its functional service life and needed to be replaced. The design team worked with the owner and many other project stakeholders to develop a replacement design that honored the history of the existing bridge while also contributing to the natural landscape and park-like setting.

The new bridge is a modern three-span structure using steel plate girders with variable-depth (haunched) girder profiles at each pier. Structural steel girders were selected based on economic and aesthetic considerations, and they also pay tribute to the century of service life provided by the original steel truss bridge at the same location. The two-column piers each consist of two 6-ft-diameter drilled shafts socketed into bedrock, while the abutments are founded on steel H-piles driven into bedrock. The new bridge accommodates a 32-ft-wide roadway for vehicles and bicyclists, along with a 5-ft sidewalk on one side of the bridge for pedestrians. A powder-coated steel rail system was selected for its openness, and deck overlooks were provided along the bridge allowing pedestrians to enjoy the scenic landscape. Five continuous steel plate girders support the roadway, which are designed as composite with a reinforced concrete deck slab. Partial-depth (3½ in.) precast concrete deck panels were used as stay-in-place elements for the slab.

The structural steel is ASTM A709 Grade 50 weathering steel, which was used to reduce future maintenance and eliminate the need for painting. The girders feature ⅝-in.-thick web plates with web depths varying from 48 in. to 80 in. in order to create the haunched girder profile while also providing a slender and visually appealing beam system. Maximum flange plate sizes over the piers are 20 in. by 1¾ in. Bolted field splices are located near points of dead load contraflexure for design efficiency and to balance girder section weights for improved constructability. Maximum shipping lengths were less than 90 ft, and the maximum steel weight for each girder section was less than 15 tons.

Owner
City of Concord, N.H.

General Contractor
E.D. Swett, Inc.

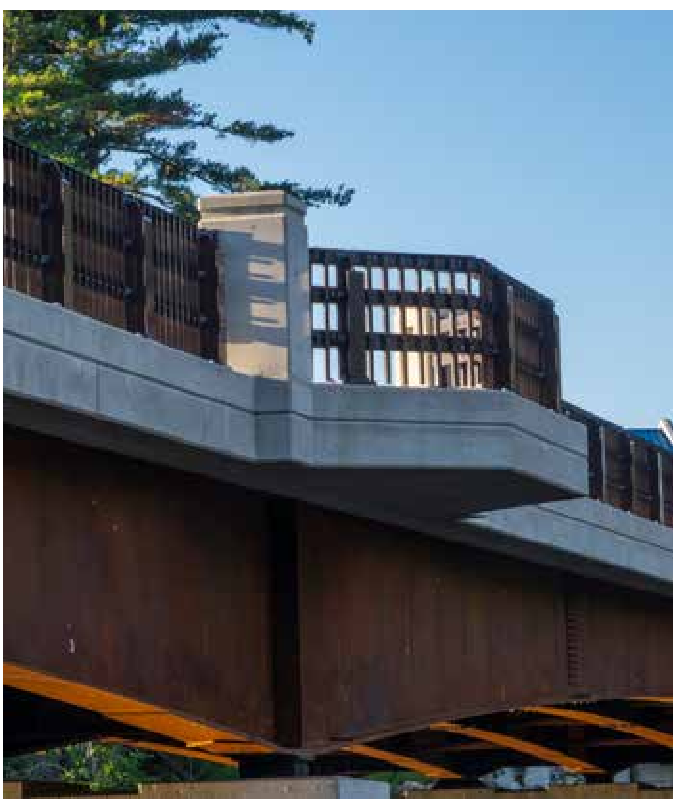
Structural Engineer
McFarland Johnson

Steel Team

Fabricator
Casco Bay Steel Structures, Inc.

Detailer
Tensor Engineering

The design team worked closely with local historians and the Department of Historic Resources to preserve the historical integrity of the existing bridge, which was designed by John W. Storrs. A notable bridge engineer who retired in 1933, Storrs was credited with having carried out or overseen the design and construction of more bridges in New Hampshire than any other individual. Following retirement from professional practice, Storrs later served as a two-term mayor of Concord. When complete replacement of the bridge was selected as the preferred alternative, the design team was determined to develop a project that honored the local history of the original bridge ad well as its preeminent designer. A small park was designed and built next to the bridge along one of the existing recreational trails. Portions of the historic metal truss bridge were repurposed and used as a trellis within this newly created Heritage Park, which included the original commemorative bridge plaque as well as new interpretive sign panels explaining the history of the original bridge. In addition, the granite stone masonry blocks from the original bridge piers were repurposed on-site to serve as retaining walls and benches within the park.



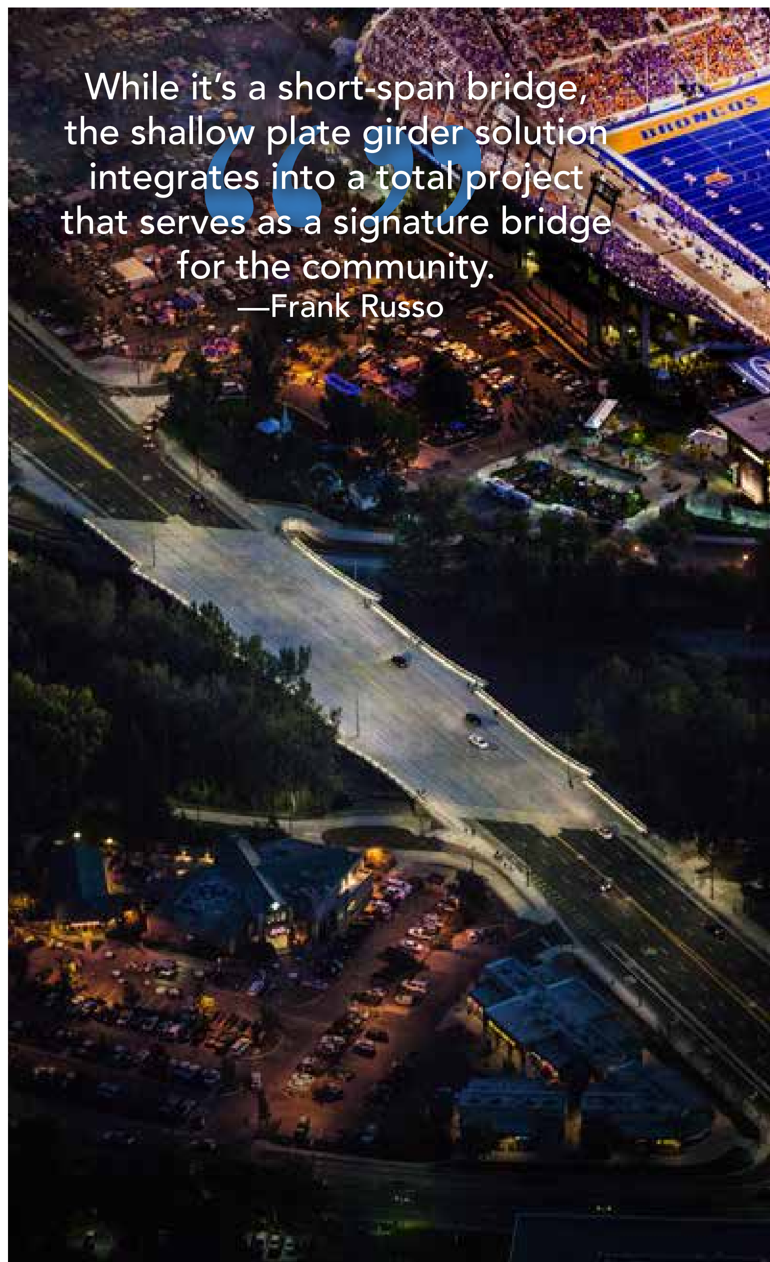
Broadway Avenue Bridge, Boise, Idaho

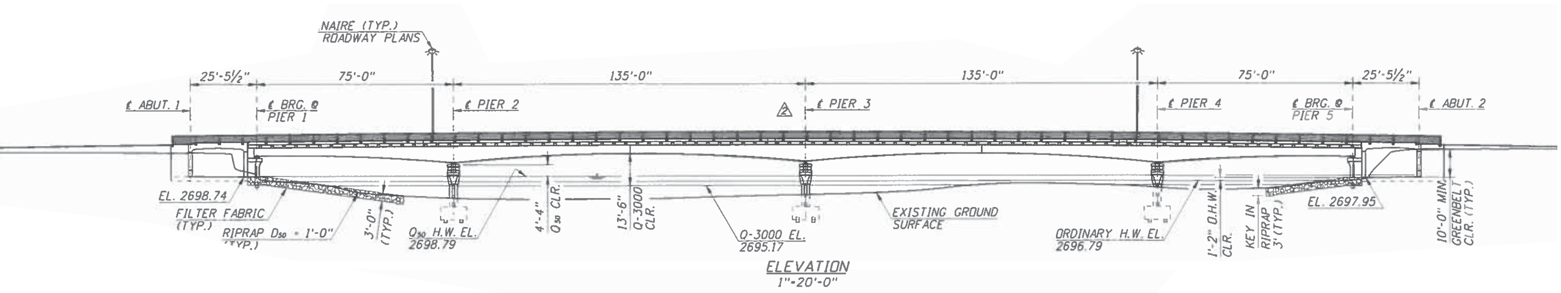
THE BROADWAY AVENUE BRIDGE connects the communities of Boise, Idaho, by spanning the Boise River with a single-span, parabolic haunched plate girder system. The bridge is the focal point of the greenbelt paths that run along the river through downtown Boise.

The new bridge includes beautiful tunnels under both ends, 10-ft-wide sidewalks with flared belvederes on both sides of the bridge complementing the steel girder curvature, design for future streetcar/light rail transportation, dedicated bike paths on both sides and six travel lanes (three in each direction). The bridge also incorporates design for future thermal utility water lines for potential expansion of the current system used by the city and university for heating needs. City planners and architectural designers assisted with aesthetically pleasing design features such as lighted steel pedestrian railings, pier and tunnel wall patterns/finishes and sidewalk saw-cut patterns and coloring.

The four-span, continuous steel girders include a parabolic haunched design for pleasing visual effects and higher clearances for rescue personnel during high flows. The bridge deck included modillions under the overhangs for supporting the larger loads due to the flared belvederes. The steel railing on the sidewalks, paths, tunnels and stairs adds some flare to the steel bridge, and the wave design of the steel railing and belvederes reflects the waves of the Boise River.

Collaboration with steel detailers allowed for efficient stiffener details and diaphragm details as well as efficient flange and web sizing. Initially, the design team considered using half-pipe attached to the web with the stiffener, believing this would handle the 45° skew for the cross frames at the piers and abutments. The assumed degree of difficulty for welding these stiffeners proved false, allowing the team to detail normal stiffeners at a 45° angle. Additionally, the efficient use of bent plates was encouraged for some of the diaphragms. Confirming the availability of $\frac{11}{16}$-in. web plate sizes gave the team comfort in steel availability going into final design. To accommodate emergency rescue personnel, given the high recreational use of the Boise River, the parabolic





haunched girders allow for additional clearance in excess of the 2-ft requirement over Q50 river flow.

One of the biggest challenges for this project was minimizing impact on the traveling public as much as possible. The team decided that a full closure of less than one year provided the least amount of impact. This allowed a short window for all work in or near the water to be accomplished. Additionally, the city restricted work activities at night. The work schedule required several crews working on different foundations and piers at the same time. Some girders were placed on completed piers while other piers were still being constructed. To improve success with the accelerated schedule and ensure that the project could be out of the water in early spring, the team put out a separate contract to the steel industry to have the girders built prior to the bridge project being bid. Another challenge was ensuring that the greenbelt recreational path elevations were above the high water mark and allowed clearances for maintenance vehicles. This required the use of jump spans (tunnels) to meet the required elevations.

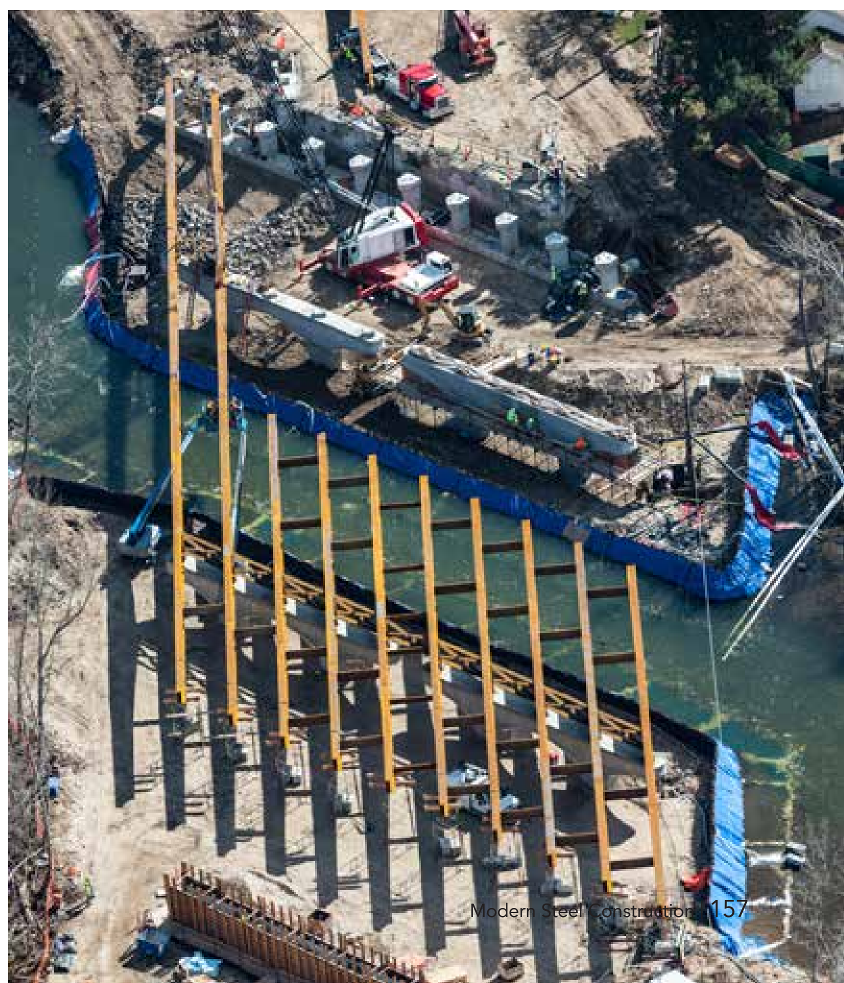
Due to the large skew, length and 108-ft width of the bridge, the design team worked closely with the contractor to make sure the girders were plumb after placing the deck and directing the use of two staggered screeds during deck placement. This involved explaining why the girders were detailed out-of-plumb prior to placement, due to the 45° skew, and giving guidance during fit-up of the girders on-site. It also required guidance on the importance of the pour sequence, allowing for rotations of the girders to take place and eliminating some permanent stress in the girders.

Another innovation enabled by using steel girders was the type of forming used to support the large overhangs from the sidewalk belvederes. An elaborate steel formwork was designed and attached to several girder flanges to support the overhangs and facilitate successful construction of the modillions and flared belvederes.

Owner and Structural Engineer
Idaho Transportation Department

General Contractor
Knife River Corporation Northwest

Steel Fabricator and Detailer
Utah Pacific Bridge & Steel AISC CERTIFIED FABRICATOR



MOVABLE BRIDGE | MERIT AWARD

Fort Street Bridge, Detroit

A RECORD-SETTING, single-leaf bascule bridge proves innovation can also be economical. The 176-ft bascule span Fort Street Bridge in Detroit is the heaviest bascule leaf in the world and second-largest by deck area. It is also one of the most efficient to open due to the rolling-lift design.

The original Fort Street Bridge had been an important connection over the Rouge River since the 1920s. The bridge was built in 1922 as a result of the dredging and enlargement of the Rouge River (to twice its width and depth) for the Ford River Rouge Plant. By the early years of the 21st century, the existing bascule bridge was nearing the end of its useful life and required replacement. The Michigan Department of Transportation (MDOT) engaged engineer Hardesty & Hanover to study alternatives and then take the most feasible option through final design. There were several critical goals for the new bridge. It had to increase the navigation channel from 118 ft to 135 ft, accommodate five 12-ft traffic lanes and two pedestrian/bikeways, minimize right-of-way impacts, avoid two utility tunnels and existing sub-piers and provide a striking visual enhancement to the community.

The new bridge is the heaviest bascule leaf in the world at 4,100 tons and the second-largest by deck area at over 15,000 sq. ft. Due to the efficient rolling-lift design, it requires minimal power to operate the bascule span under normal conditions. However, the two 150-HP motors move the bascule span during high wind and heavy ice conditions. Stringers, floor beams and two 13-ft-deep pony trusses support the steel grid-reinforced lightweight concrete deck.

The two segmental girders support the entire dead load of the bascule leaf in all positions. Each segmental girder is a heavily stiffened three-plate weldment. The web is 3 in. thick by 96 in. deep. The curved top flange is 32 in. wide by 3 in. thick, and the curved bottom flange is 32 in. wide by 3½ in. thick, after finishing. The steel for these girders is Grade HPS70W.

An enhanced design was developed for the segmental treads and tracks. These elements are high-strength steel castings, ASTM A148, Grade 130-115, with very robust cross sections to support the 2,050 ton loads on them. They are I-shaped with 10-in.-thick webs and 30-in.-wide flanges. Both edges of the tread bottom flanges have protrusions (lugs or teeth) that extend down into corresponding pockets in the tracks. These are spaced longitudinally at 25-in. centers. Locating the protrusions on the treads instead of the tracks, as is historically done, prevents debris from being trapped against the vertical surfaces of the protrusions.

> The project sets a record for a single-leaf bascule bridge while the counterweight structure creates a unique and visually striking bridge for the community.
> —Dominique Shannon

The counterweight structure is a six-panel trapezoidal box truss, 25 ft, 10 in. deep by 18 ft, 6 in. wide at the top and around 9 ft, 6 in. wide at the bottom. The bottom of the front truss is supported by the upper ends of the segmental girders, and there are cross frames at each panel point. The back truss is sloped so that it is parallel to the roadway when the bascule is fully open. Sloping the back truss provided the most efficient counterweight, which is sheathed in stainless steel panels to provide an enhanced aesthetic appearance and reduce future maintenance needs. The stainless sheathing on the roof of the counterweight and machinery room is underlain with a heating system to limit the amount of snow and ice that can accumulate and potentially fall onto the roadway.

Due to the very poor soil conditions in the immediate area of the bridge, an extensive geotechnical investigation was conducted. It was determined that concrete slabs borne on steel piles would provide an efficient structure to support fill, road and live loads and adequately address the soil instability concerns. These pile-supported slabs are located behind each abutment and each is supported by 43 steel HP12×74 piles. The bascule pier is located over two utility tunnels and the four 12-sq.-ft sub-piers of the existing bridge. The new foundation had to avoid these obstructions while supporting the 9,000-ton weight of the pier and bascule structure as well as the plus/minus 162,000 ft-kip overturning moment. Bedrock is located about 80 ft below the footing. Various foundation systems were investigated, and the ideal solution was to use the new heavy steel piles, HP18×204, which provide very high capacity with a relatively small individual footprint. A total of 12,745 ft of these sections was used, and this is the first MDOT project to employ these new steel sections.

Owner
Michigan Department of Transportation

General Contractor
Toebe Construction, LLC

Structural Engineer
Hardesty & Hanover

Steel Team

Fabricator
Steward Machine Co., Inc. AISC CERTIFIED FABRICATOR

Detailer
Tenca Steel Detailing, Inc. AISC DETAILER

RECONSTRUCTED | MERIT AWARD

BNSF Railway Company, Bridge 482.1, West Memphis, Ark.

BNSF'S RAIL BRIDGE 482.1 proves accelerated bridge construction (ABC) isn't always about building new structures. Simultaneously sliding in a new superstructure while removing an existing 339-ft truss span was one of many challenges overcome by using ABC techniques.

HNTB Corporation provided design, permitting and construction engineering and inspection services for replacement of the 125-year-old bridge's west approach. The single-track bridge, which lies between West Memphis, Ark., and Memphis, Tenn., carries 90 million gross tons of intermodal and coal traffic over the Mississippi River annually and is essential to the BNSF Kansas City-to-Birmingham corridor. Reconstruction of the approach was split into four phases, with the final spans of the bridge replaced in August 2017.

As part of BNSF's $6 billion capital expenditure plan, the 2,712-ft-long existing west approach—consisting of fracture-critical, open-deck approach spans supported on steel towers—was replaced with new ballasted deck plate-girder spans supported on hybrid drilled shaft-micropile foundations. All structural steel used on the project was unpainted weathering steel, which will greatly reduce BNSF's future maintenance costs.

To keep the existing bridge in service during construction, HNTB designed a phased construction schedule with minimal track closure windows. Intermediate jump spans were designed to transition the existing open deck bridge to the new ballast deck bridge. The ABC approach allowed 2,712 ft of bridge to be changed out during four track-closure windows over a 10-month period, with the final spans replaced and traffic resuming in August 2017. The four track-closure windows ranged in duration from 36 hours to 52 hours, and the new approach consists of 27 ballasted deck plate-girder spans ranging in length from 72.5 ft to 191 ft.

The Phase I change-out replaced the first seven spans on November 10-11, 2016. This change-out was complex due to the approach requiring two 176.5-ft spans to cross over the

Big River Crossing pedestrian bridge and a county road. With the weight of these spans greater than the capacity of the on-site cranes, both spans were erected on extended pier caps and rolled into place during the change-out. Close coordination between the project team and the Memphis engineering firm in charge of the Big River Crossing trail was important throughout the design and construction phases. The opening of the Big River Crossing trail was scheduled three weeks before the Phase I change-out in 2016, and timing to install a canopy below the BNSF bridge was critical due to interference between the canopy and the bottom of the new deck plate-girder span prior to the final roll-in. The canopy was successfully installed two days after completion of the Phase I change-out.

Next, 708 ft of the approach was replaced during both the Phase II change-out on February 20-21, 2017, and the Phase III change-out on April 3-4. All spans in these two phases were 88.2 ft long and were shipped to the site in two shop-assembled units, which greatly reduced field assembly time. During the Phase II and III change-outs, each span was erected in place with one on-site crane after the existing structure was removed.

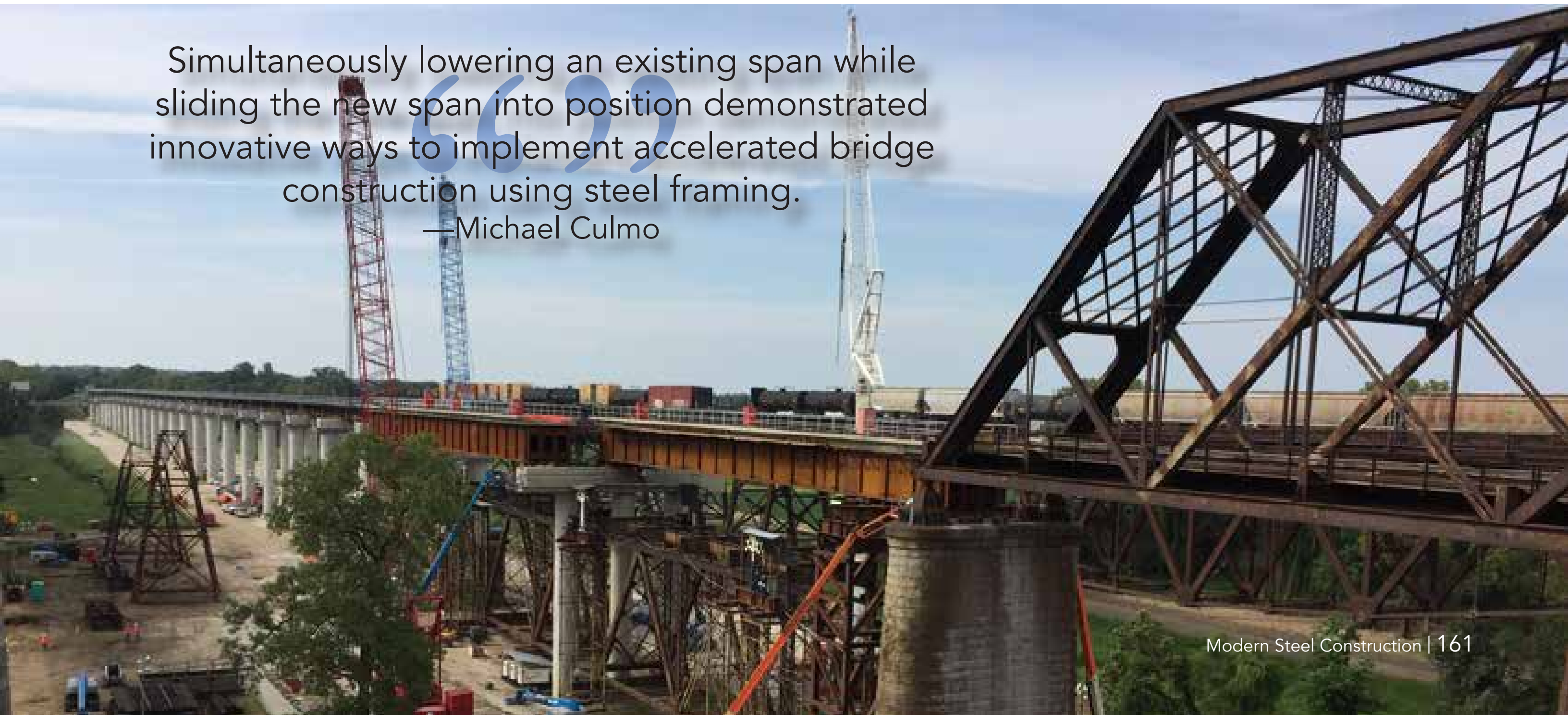
The Phase IV change-out, on August 28-30, replaced the final 548 ft of approach. Construction activities leading up to this change-out included constructing a pier cap and column through the existing 339-ft deck truss and erecting extensive falsework and shoring towers to aid with removal of the deck truss. Managing this fourth and most complex phase involved lowering a 339-ft deck truss using strand jacks, rolling a 191-ft span transversely into place and rolling a 178-ft span both transversely and longitudinally. Prior to lowering the deck truss, the north and south ends of the truss were removed to clear the existing piers, and portions of the truss surrounding the center pier were removed to permit the truss to be lowered around the pier.

Owner
BNSF Railway Company

General Contractor
Kraemer North America

Structural Engineer
HNTB Corporation

Steel Fabricator and Detailer
Delong's, Inc. AISC CERTIFIED FABRICATOR



"Simultaneously lowering an existing span while sliding the new span into position demonstrated innovative ways to implement accelerated bridge construction using steel framing."
—Michael Culmo

SPECIAL PURPOSE | MERIT AWARD

Moose Run Golf Course Bridge, Anchorage, Alaska

A NEW STRIKINGLY beautiful bridge serves as a signature crossing for golfers on the Moose Run Golf Course. The Moose Run Golf Course Bridge suspension bridge used a unique approach that saved construction costs while mitigating environmental impacts to the surrounding area. To support the bridge deck, a system of horizontal and vertical cables spans between two vertical towers.

Suspension bridges, while materially efficient, are often some of the most difficult to build because they require unusual and complex means of construction and may require extensive temporary aerial catwalks and gantries. The most typical superstructure for a 200-ft-long clear-span pedestrian bridge such as the Moose Run Bridge would be a heavy through-truss or heavy girder design. Construction of this type of bridge would generally require mobilizing a large crane or multiple cranes to set the superstructure, adding considerable cost to the project and significant impact to the environment.

Alternatively, PND Engineers designed the bridge as a Strand Bridge, a type of suspension system that uses vertically harped and horizontal strands to form the bridge support, provide for bridge erection and form the final hand rails. This bridge type, codeveloped by PND and contractor Swalling Construction Company, is launched instead of using conventional construction methods, permitting the use of smaller construction equipment and requiring no significant temporary structures, further reducing costs over classic suspension bridges.

After construction of the foundations and towers, horizontal strands are placed and intermediate U-frames and minimal decking are launched from one side of the bridge. These horizontal strands also function as hand railing for the completed bridge. Once the U-frames are in place, harped suspension strands are installed and tensioned to provide a flat bridge camber, after which the bull rail and the remainder of superstructure elements are installed. The bridge is then cambered to its final position and load tested.

Using the Strand Bridge method, the Moose Run Bridge's superstructure was launched from one side of the bridge, rather than being craned into place. The design used high-strength, low-relaxation, galvanized pre-stressing strands. Ultimately, the bridge is sturdy enough to support use by pedestrians, golf carts and groundskeeping equipment (load tested with an 8-ton H-configuration truck) and to accommodate a 90-psf uniform load in accordance with AASHTO. The uniform load easily accommodates the local design snow load of 57 psf.

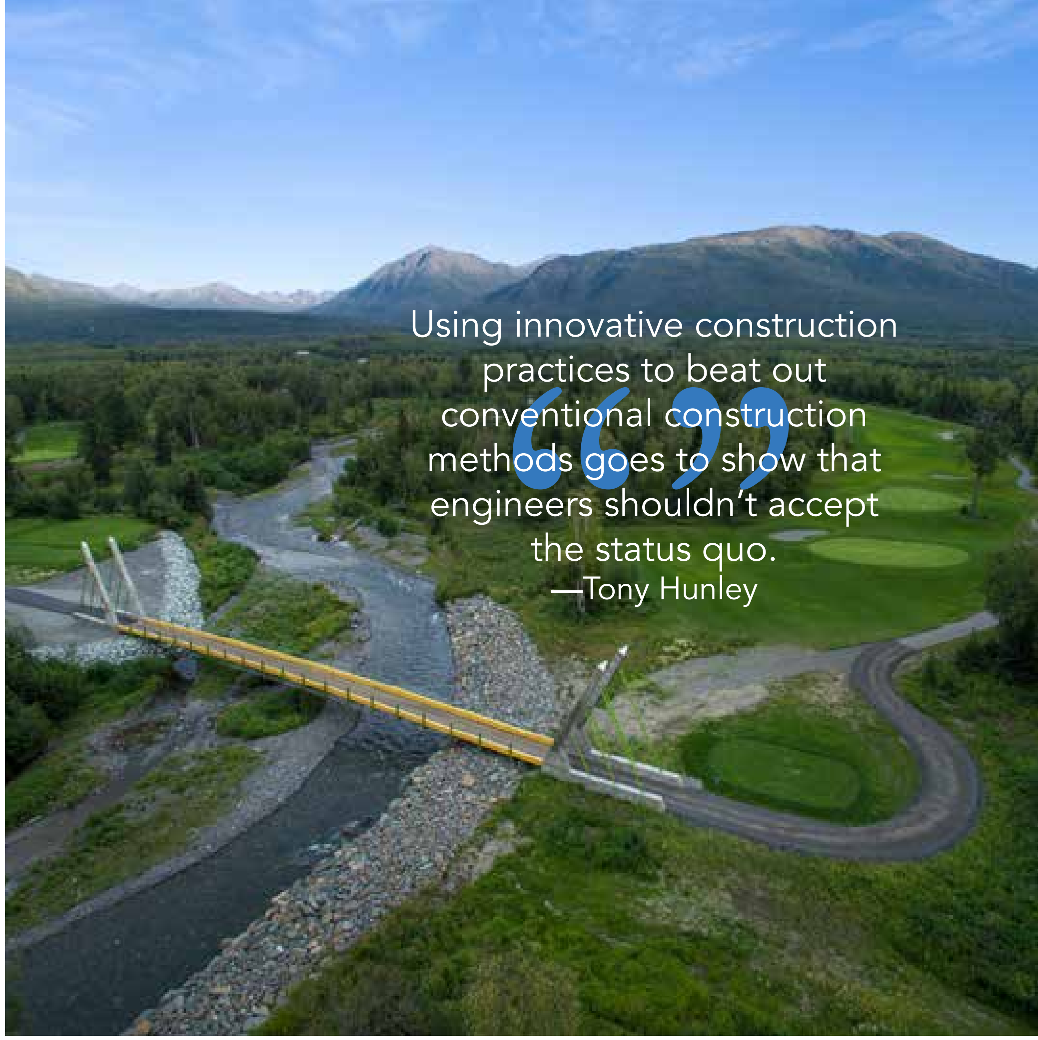
In function, the post-tensioning strands provide vertical global support for the 200-ft span, and the rectangular steel HSS box beams efficiently provide local vertical and torsional stiffness as well as function as a vehicle guard rail. Because these types of light bridges are prone to vibrational amplitudes and frequencies that pedestrians are often sensitive to, and because these characteristics are sometimes difficult to accurately predict, this bridge was designed with several important considerations. It was detailed to readily accept mass-dampers if the bridge dynamics were shown to be a problem. The HSS beams were designed to provide vertical, lateral and torsional stiffness to the system, which in turn mobilizes more mass from the structure, thereby helping to mitigate the localized pedestrian dynamic loads. Post-construction, the bridge was monitored with velocity transducers to determine its vibration amplitude and frequency characteristics. Although the bridge is relatively light, its amplitudes have not been considered objectionable.

Owner
Moose Run Golf Course at Joint Base Elmendorf-Richardson

General Contractor
Swalling Construction Company

Structural Engineer
PND Engineers, Inc.

Steel Fabricator
Jesse Co.



> Using innovative construction practices to beat out conventional construction methods goes to show that engineers shouldn't accept the status quo.
> —Tony Hunley




CAT

SUSTAINABILITY COMMENDATION

Neponset River Greenway, Milton, Mass.

THE NEPONSET BRIDGE in Milton, Mass., blends seamlessly into the canopy of the surrounding trees. Designed and fabricated to limit material waste and using a durable coating system will limit the life-cycle costs associated with the bridge. When it comes to designing for sustainability, this project checks all of NSBA's boxes.

This structure is a single-span steel arch bridge that spans the Neponset River. The preassembled weight was approximately 62.5 tons and originally required a 660-ton crane for the pick. Directly under the laydown area for the crane was a 48-in. sewer line that had to be protected. A crane of this size would have created a large impact on the site and involved the removal of numerous trees in the wetland area as well as posed a risk to the sewer line. General contractor S&R opted to erect the structure by using a tandem pick with two cranes: a Liebherr LR-1200 crawler crane on the north side of the river and a Liebherr LTM-1400 hydraulic crane on the south side. This method also involved creating a 65-ft-long temporary crane "runway" bridge, allowing the LR-1200 to crawl into position while completely avoiding the sewer main. This method of erection ultimately resulted in a time and cost savings as well as protected the extremely sensitive environmental areas adjacent to the project.

All structural steel was prefabricated and preassembled to the fullest extent possible off-site, resulting in minimal cutting and assembly in the field. Concrete forms also were fabricated off-site, resulting in minimal waste. All Ipe wood decking was cut to length before being delivered, and all custom millwork of Ipe handrails was done at a sawmill before being delivered to the job site. This technique of prefabricating material allowed S&R Corporation to have minimal waste, while most projects of this magnitude would generate a much larger amount of waste and leave a much larger impact on the local landfills.

The project was heavily focused on aesthetics and involved many unique and creative applications. The railing system on all of the structures involved steel posts coated with Tnemec paint, square tube steel railing frames, DecorCable stainless steel mesh infill panels and custom-milled Ipe wood. This was a complex application due to the intricacies for the 100+ different shapes of steel frames and DecorCable infill panels, with tolerances as low as 3⁄16 in.

In addition, the bridge structure is fitted with a unique and impressive lighting system. There are 10 lighting fixtures located within the structural steel arch (using the steel framing as conduit) shining down onto the Ipe deck and illuminating the walking surface. In addition, 30 lighting fixtures were recessed into the deck to shine upwards, illuminating the steel arch.

To complete this uniquely aesthetically pleasing project, the bridge is fitted with steel tree silhouettes and acrylic leaves set in a specific, detailed pattern to assume the appearance of seasonal trees with decorative foliage. S&R used full-size renderings of the designed pattern to ensure that more than 2,700 acrylic leaves were placed specifically to capture the beautiful outcome envisioned by the designers. ■

Owner
Massachusetts Department of Conservation and Recreation

General Contractor
S&R Corporation

Structural Engineer
Crosby Schlessinger Smallridge, LLC

A significant steel Interstate crossing comes together over a large rail yard in Montana with minimal impact.

Over the Rails

BY DUSTIN HIROSE, PE

Photos: HDR and MDT

ONE OF THE FINAL PIECES of an Interstate puzzle has been put in place in Montana's capital city.

Administered by the Montana Department of Transportation (MDT), the project included reconstruction of a section of Interstate 15 (to increase capacity) and replacement of a pair of functionally obsolete and seismically deficient bridges that span the Montana Rail Link (MRL) rail yard, which includes 14 active tracks.

The new bridges over this busy rail yard needed to be built with minimal impact to its operations. Traffic maintenance during construction was another important element. The bridges are centered between the Cedar Street Interchange, located at the north end of the project, and Capitol Interchange at the south end of the project. This section of Interstate exhibits high volumes of traffic along with weaving movements between the closely spaced interchanges, which are less than a mile apart. With no acceptable detour routes, traffic had to be maintained on the existing bridges during construction. Furthermore, the project would take two full construction seasons to build, and it was imperative that one of the new bridges be built in the first construction season so that the interstate could be restored to two-lane, two-way traffic during the winter shutdown period.

Planning and Development

The Capitol-Cedar Interchange project is one of the final segments of an initiative that began in 2003, when an environmental impact statement was completed for the I-15 corridor through Helena. The statement documented the need for additional capacity and safety improvements throughout the corridor, resulting in opposite: The Interstate spans a major rail yard. below: The new bridges were built wide enough to accommodate a future fourth lane each.

several projects along the corridor. This segment consisted of many complexities that required a different approach to project delivery.

The roadway is on a steep grade in order to provide clearance over the rail yard in the short distance between the interchanges. Weaving movements between the interchanges, along with the relatively narrow 28-ft-wide bridges, resulted in traffic accident clusters, specifically in winter months when driving conditions were poor.

The environmental document identified the need to replace the functionally obsolete bridges and widen the roadway to add an auxiliary lane in each direction to reduce the weaving movements between the interchanges. The immediate goal was to add an auxiliary lane in each direction. However, long-term planning identified the need for an additional through lane along the corridor and within the service life of the new bridges. Therefore, the new bridges over the rail yard were built wide enough to accommodate a future fourth lane each, and the roadway drainage infrastructure was also designed and built with additional capacity to accommodate a future through lane.

Dustin Hirose (dustin.hirose@hdrinc.com) is an engineer with HDR in Missoula, Mont. **Stephanie Brandenberger, PE**, of MDT also contributed to this article.

The team performed a comprehensive evaluation of the various bridge types and span arrangements and how they would impact the rail yard.

Placing deck reinforcement.

The chosen bridge span arrangement required three railroad tracks to be relocated in advance of construction.

In addition to traffic maintenance and minimizing the impact to the active rail yard, contaminated soils, oversized loads, storm water, City of Helena coordination, noise impacts and utilities routing also needed to be addressed. As such, MDT required a different approach that would serve to identify risks early on and develop strategies for mitigating them ahead of construction.

In 2010, MDT selected HDR to lead the design effort for the project, which began with a comprehensive bridge Type, Size and Location Study. Out of 23 different bridge options that were considered in all, two options stood out as providing the best fit concept for this site, a concrete option and a steel plate girder option, both calling for four spans—180-ft – 212-ft – 212-ft – 180-ft (784 ft total)—for both northbound and southbound bridges.

HDR continued with developing the project final design through a custom project schedule that allowed for an accelerated delivery due to the significant planning done during the bridge study. In the end, the steel option prevailed thanks to cost and speed. Considering the bridge skew, width of the structures, erection over the rail yard and aggressive construction schedule, the cost of the steel portion was well below the design team's original estimates (final erected cost was roughly $1.10 per lb). The final design uses around 2,000 tons of steel in all for both bridges, with the girders being approximately 6.5 ft deep and made from grade 50 weathering steel. By 2015, the project final design and right of way acquisition was complete, and the project was let to construction in 2016.

Railroad Coordination

A proactive approach to engaging the railroad early in project development was needed in order to develop a bridge design that could accommodate the needs of the highway above yet be practically built in the busy rail yard.

As part of the bridge study, the design team performed a comprehensive evaluation of the various bridge types and span arrangements and how they would impact the rail yard, and clear spanning all of the tracks was not a practical option. Intermediate bents would be necessary and properly locating them required an understanding of rail yard operations, so in addition to referencing standard railroad guidelines, input from local railroad personnel was needed to help establish final clearances between bridge elements and the railroad tracks. In addition, temporary clearances during construction, permissible track closures and acceptable construction work windows were important variables to consider when evaluating possible span configurations and were dependent on the specific operations within the rail yard.

Site access was another important consideration. Access from one end of the bridge site to the other during construction would require the contractor to cross the railroad tracks, move equipment and materials around a lengthy detour or mix with the travelling public through the Interstate construction zone, which would be restricted to two-lane, two-way traffic. On top of that, speed of construction was another crucial aspect of the project, and having the ability to stockpile materials and equipment on both ends of the rail yard would be important to avoid the inefficiencies of having to frequently cross the tracks or travel through the construction zone with equipment and materials.

The chosen bridge span arrangement required three railroad tracks to be relocated in advance of construction. It was more economical to move the tracks than to increase girder lengths and span over them. As a result, the preferred bridge alternate included a balanced and symmetrical span arrangement that reduced material costs and was easier to erect. Without the extensive early coordination with MRL to identify this option, a bridge alternate with significantly longer spans, and higher cost, would have been necessary.

In the end, the project design team developed a partnership with MRL to design the project. Many of the items typically left for the contractor to resolve were addressed early on during the design phase, resulting in reduced risk for all parties involved.

Construction Sequencing

As the Interstate corridor is located in an urban environment, options to shift the alignment were not feasible considering the impacts to properties adjacent to the highway right of way. Even if adjacent property impacts could be justified and afforded, the geometric constraints of the closely spaced interchanges made an alignment shift impractical. Therefore, the only possible way to build the project was to sequence construction such that work on one side of the Interstate could be completed while traffic was maintained on the opposite side.

Extensive traffic analysis was performed to verify that the anticipated traffic volumes could be maintained through the construction zone along with merging traffic from the interchanges without causing significant disruptions elsewhere in the system. Although the anticipated level of service during construction was not ideal, the proposed plan of having two-lane, two-way traffic during the first construction season would function. During the following season, the newly completed bridge, which is significantly wider than the previous bridges, could maintain at least one additional lane during construction. Understanding that there would be head-to-head traffic on the narrow, 28-ft wide, existing bridge during the first season, an emergency detour plan was developed in the event an accident occurred on the existing bridge.

A critical part of the project sequencing was the requirement that the first new bridge be built in the first construction season. With this requirement, the Interstate could be restored to the four-lane configuration during the icy winter months. Having traffic negotiate crossovers and traveling in a head-to-head configuration on the narrow existing bridge during the winter was not acceptable.

Considering the short, seven-month construction season in Montana along with the importance of having the first bridge complete in the first season, a detailed constructability review of the project was needed. HDR used a team of construction engineers to evaluate the project from the viewpoint of a contractor. One of the goals of the review was to understand if the bridges could be built by conventional methods within the needed timeframe, or if some type of accelerated bridge construction (ABC) method would be necessary. Although there was merit in using ABC, the cost impacts did not appear to offset the user cost benefit, and the final determination was that it would be more cost-effective to use additional equipment and workforce to complete the project using conventional methods.

Temporary shoring to support the roadway near the proposed south abutment.

Crews had to deal with the area's relatively short seven-month construction season.

A critical part of the project sequencing was the requirement that the first new bridge be built during the first construction season.

The original northbound and southbound bridges.

right: Driven steel piling was determined to be an adequate foundation type.

MDT maintains a library of historical bid prices, which are typically used to help estimate project costs. For this project, a more detailed evaluation of construction costs was performed to account for the additional equipment and work crews that were anticipated. HDR developed the cost estimate from the perspective of a contractor considering materials, equipment mobilization, labor classifications, indirect expenses and applied escalation factors for construction elements that were subject to higher risk. In the end, this exercise helped to better define the project cost. This project required a large share of MDT's construction program funding in a given fiscal year, and it was important to have a good understanding of construction cost prior to bidding the project.

Pile Test Program

Building foundations adjacent to railroad tracks typically present challenges. To name a few, there are minimum clearances to maintain during construction, requirements for shoring excavations (which can be significant if subject to surcharge loading from trains) and limited work windows available to complete the foundation construction.

HDR worked with geotechnical engineer Tetra Tech, MDT, and MRL to obtain geotechnical borings within the rail yard during the Bridge TSL work to develop options for the bridge foundations as part of evaluating various bridge alternates. Alternates with longer spans had the advantage of fewer foundation units, but generally required a larger foundation footprint compared to alternates with shorter spans.

Several soil types were encountered at the site, and a very dense matrix of cobbles and boulders was identified roughly 30 ft below the surface. The material above this layer consisted of loose fill and clay that was not ideal to support a bridge foundation. The material below this layer was relatively consistent and extended to the bottom of the geotechnical borings, which were advanced between 100 ft and 150 ft below the surface depending on the location.

Spread footings were eliminated as a practical foundation type, since the temporary shoring would be impractical to construct given the excavation depths needed to reach the dense cobble/boulder/ash soil elevation. Additionally, the bridge site is located in a moderate seismic zone, so lateral loading controlled the design of the bridge foundations. The required footprint for a spread footing, if founded at a higher elevation, was not feasible considering the close proximity of the railroad tracks. Driven steel piling were a good foundation choice considering the axial capacity that could be achieved in the cobble/boulder/ash matrix. However, there was some concern that the piling would refuse in that layer prior to obtaining enough penetration to obtain lateral fixity and the uplift capacity needed to resist seismic loading. Therefore, initial recommendations were to use drilled shafts

above: Span construction.

below: Typical intermediate cross frames.

since they could be advanced deep enough to obtain the needed capacity. The downside of using drilled shafts was that they were the most expensive foundation option, and if any defects were found during construction, they would be very difficult to correct and have significant schedule implications.

The design team recognized some significant advantages associated with a pile foundation if the piles could obtain the needed lateral capacity at the shallow depth. In addition to a significant savings in construction cost, the construction schedule could be reduced by about a month per season with a pile foundation. With this in mind, the team moved forward with a pile test program very early in the design phase of the project.

Five steel test piles were installed at the project site, and both H-piles and cylindrical piles were installed to compared drivability, capacity and penetration. As expected, most of the piles refused with minimal penetration into the cobble/boulder/ash matrix. The axial capacity obtained at this elevation was plenty adequate for the anticipated locating, and a lateral load test was performed to determine if the piles could obtain fixity and to help calibrate soil data used for analyzing the piles under lateral loading. Uplift testing was also performed for the same purpose of verifying a pile foundation would be adequate for the anticipated seismic loading. It was ultimately concluded that driven steel piling would be an adequate foundation type. The pile testing program also served to identify what equipment would be needed to install the piling during bridge construction, solidify the pile tip elevations and provide more certainty on the total length of piling needed. Additionally, the preliminary pile footprint and number of piles were reduced due to the additional capacity that was identified by the pile test program.

The program cost about $200,000 to install the test piles and perform the engineering and testing to verify the adequacy of the piles. However, compared to drilled shafts, the use of piling resulted in about $3 million in construction cost savings (total construction cost was roughly $27 million) in addition to reducing the overall construction schedule. Construction was completed last year, and the contractor, Sletten Construction, received full incentive for completing the work within the schedule requirements of the contract. ■

Owner
Montana Department of Transportation

General Contractor
Sletten Construction

Structural Engineer
HDR

Steel Fabricator
TrueNorth Steel AISC CERTIFIED FABRICATOR

Narrow Margin

BY KEN SAINDON, SE, PE, AND ALEX WHITNEY, PE

It's a tight—but successful—squeeze for a replacement steel span in a remote Idaho canyon.

Ken Saindon (**kens@estinc.com**) is Colorado Bridge Group manager with EST and served as the technical lead and engineer of record for this project. **Alex Whitney** (**alexander.whitney@hdrinc.com**) is senior bridge project manager with HDR and served as consultant project manager. Both were formerly with Atkins.

WHILE CERTAINLY SCENIC, the steep nature of a V-shaped canyon near Riggins, Idaho, created quite the challenge for the designers of a replacement bridge over the Salmon River.

The original Manning Crevice Bridge carried Salmon River Road over the river at this location, providing access to residences, resorts and commercial rafting ventures and acting as a main artery for recreational users of the river and surrounding forest lands.

By 2010, the bridge (built in 1938) had reached the end of its service life, and the decision was made to replace it. But this would be no easy feat. The site, located in a steep canyon, had limited access for trucks and limited space available to stage construction equipment and materials, not to mention sharp bends in the road. The choice of steel for temporary and permanent works was crucial to developing a feasible erection scheme on this difficult site and addressed the following requirements for the replacement project:

- A bridge deck clear width of 16 ft for a single lane
- A minimum vertical clearance of 18 ft
- A minimum load capacity of AASHTO HL-93 and a 45-ton logging vehicle
- Roadway curvature at the bridge ends had to be able to accommodate a logging truck crossing the bridge
- No permanent construction could be placed within the 100-year flood plain
- Traffic had to be maintained on the existing bridge during construction
- The river had to remain open to rafters during construction
- Construction equipment was not allowed in the river

Not-So-Easy Access

After evaluating six different structural configurations, a single-tower, asymmetric suspension bridge scheme was chosen. Competent bedrock at the site provided ample capacity for anchoring large horizontal forces, thus favoring arch and suspension bridge types over cable-stayed structures. Given the limited access for construction equipment, cable suspension was judged to be more constructable than an arch because of the light weight and flexibility of steel cables. The bridge span length is 300 ft and with a cable sag of 18.5 ft at mid-span, the resulting sag ratio (span/sag) of 16.2 is much flatter than the classical suspension bridge sag ratio of 10. The bridge uses a total of 180 tons of structural steel.

The new bridge spans 300 ft.

Roadway curvature at both ends was required to allow a logging truck to cross the bridge.

A number of factors led to the single-tower configuration. For one, the rock face adjacent to the north tower of the existing bridge required a minimum tower height of at least 60 ft to place anchorages on favorable rock geometry. A large debris flow zone and a continual water seep on the south hillside made this an unfavorable location for a new tower and anchorage. Finally, the size of crane that could be placed on the south side of the river was highly uncertain given that the only two access routes to the south side are either over an unpaved high mountain pass with very tight switchbacks or across the existing bridge, which had neither the geometry nor load capacity to handle a large crane. (Note that the CM/GC was able to deliver a large lattice crane over the high mountain pass to the south side of the structure.) As such, a tower on the south side of the river would not be feasible.

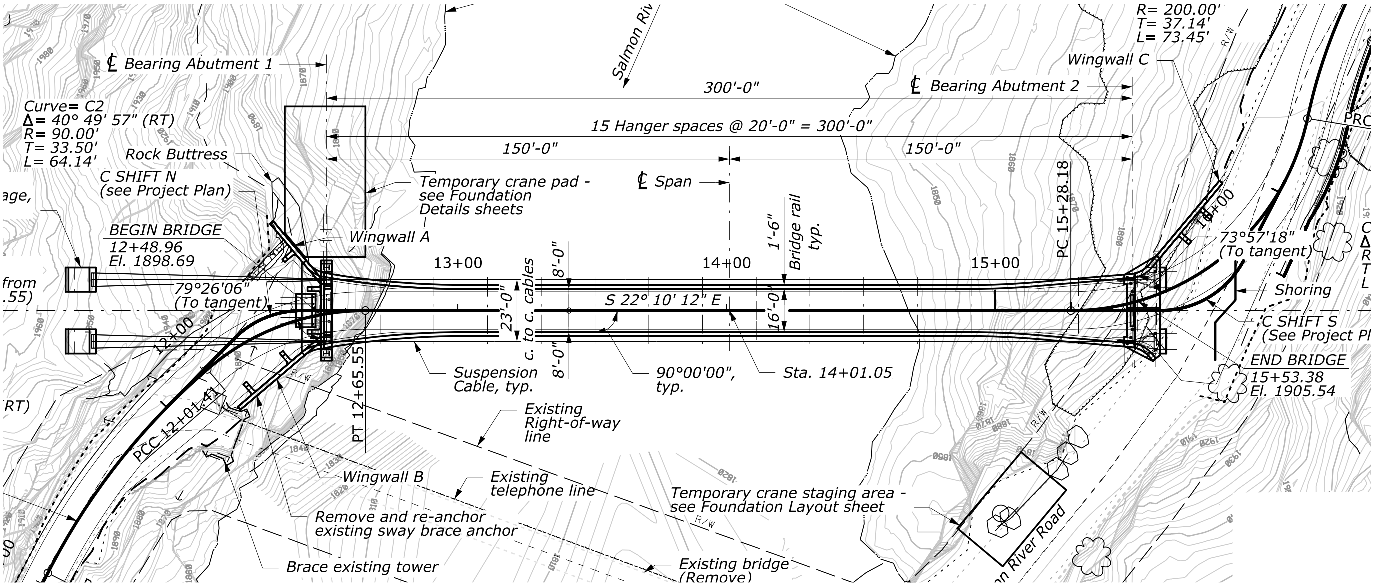
Orienting the new bridge was a balance between providing roadway alignment geometry to allow a WB-62 vehicle to negotiate the approaches, providing the shortest overall bridge length, maintaining the existing bridge in operation during construction and choosing a favorable tower and anchorage location on the north side of the river. The south abutment and anchorage were placed close to the river and, being below the road surface, has protection from hillside debris flows. The south abutment and anchorage placement also struck a balance between keeping all permanent construction outside the 100-year floodplain and providing sufficient room beyond the anchorage to allow traffic to pass during construction.

The site features a narrow shelf road with steep drop-offs in hard rock terrain. Standard construction techniques for such steep sites typically involve temporary benching. However, the hard rock site and pristine canyon location made benching both cost-prohibitive and inappropriate at the north abutment. Luckily, the presence of soil overburden on the south river bank allowed a cost-effective cut bench to be used at the south abutment. During the design phase, a temporary crane platform was located on the north side of the river for erection of the tower and cable anchorages. Additional temporary platforms were also used for construction at the north anchorage and behind the tower base. The existing south-side roadway bench was wide enough to accommodate a crane for erection and still allow vehicles to pass, and all construction materials were staged and delivered from Riggins to the north end of the bridge.

Steel Simplifies Erection Scheme

Helically wound galvanized wire (ASTM A586) was used for the main cables and hangers. The main cable and hanger cable connections consist of heavy steel castings with molten zinc spelter sockets, and the cable system saddles consist of 1-in.-thick steel plates and steel castings with groove and fillet welds throughout. The tower consists of welded I-sections for the battered legs and rolled W-shapes for the diagonal bracing. The superstructure framing was designed for simplicity and economy, and all members are rolled steel sections with W-shapes for the stiffening girders and floor beams and WT shapes for the lateral bracing. The stacked superstructure framing configuration was conceived to permit easy assembly from the bottom up, starting with the floor beams followed by the lateral bracing and then the stiffening girders. High-strength bolts were used in all field connections.

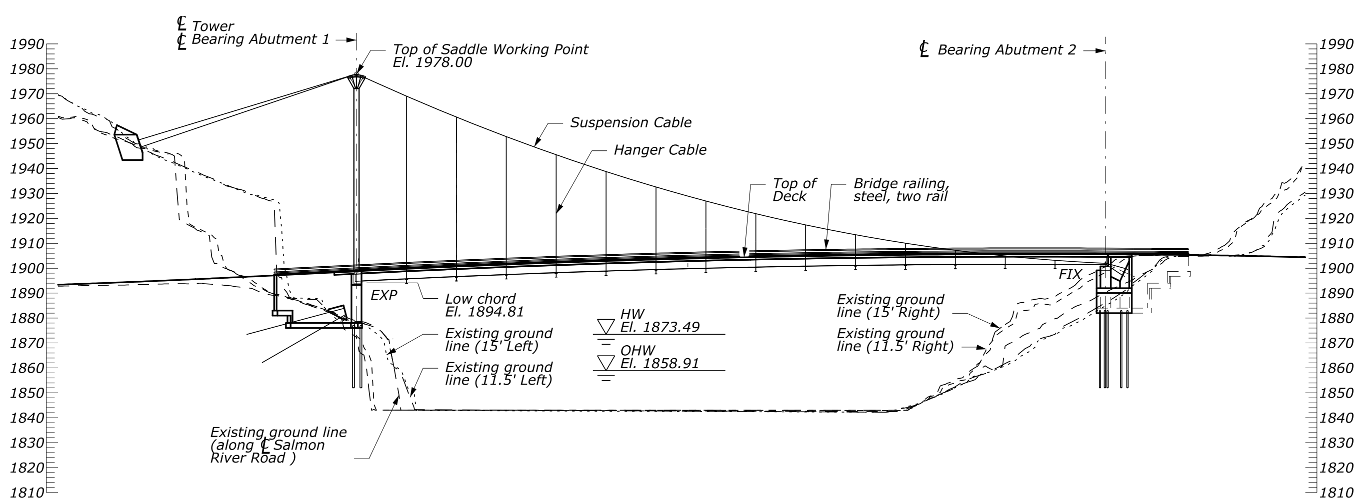
Tower erection was a breeze given the small reach and piece weights of about 9.5 tons. The main cables were erected using a cableway accordion sling (designed and patent-pending by Inland Crane) to support each strand at regular intervals on the temporary cables as it was pulled across the river from the tower to the south abutment. Erecting the cable hangers and bridge superstructure framing from the two fixed crane locations required crane reaches of up to 160 ft at mid-span. Hangers, floor beams and lateral bracing had piece weights of 2.25 tons or less,



An overhead view of the tight project site and sharply turning roadway.

A view (looking west) of logistics on the south bank of the river, with the existing bridge in background.

Cable installation from the tower to the south abutment anchorage.



An elevation drawing of the new bridge.

A minimum tower height of at least 60 ft was required in order to place anchorages on favorable rock geometry.

Erecting the tower from a temporary crane platform.

Cable installation.

so the long crane reach was not a problem for these items. The stiffening girder piece weights varied with the exterior 50-ft-long sections weighing around 5.5 tons and the interior 40-ft-long sections weighing 4.5 tons. Splice locations and piece weights were designed to reduce the demands on the cranes, and superstructure erection was completed in less than three weeks.

The new single-tower bridge opened this past June, bringing a touch of uniqueness to the canyon and respecting the constraints of the site with its force layout. With longevity in mind, especially considering the winter climate, Class C galvanizing was specified for the steel cables, and Grade 50 weathering steel was used for the towers and superstructure—not only for corrosion resistance but also to reduce visual contrast with the weathered granite prevalent at the site. The project's reception by the community has been overwhelmingly positive, and it is anticipated to last well beyond the century mark. ■

Owner
FHWA-Western Federal Lands, Vancouver, Wash.

Construction Manager/General Contractor
Record Steel Construction, Inc., Boise

Structural Engineer
Atkins, Denver

Steel Team

Fabricator
Rule Steel, Caldwell, Idaho (AISC Certified Fabricator)

Detailer
ABS Structural, Melbourne, Fla. (AISC Detailer)

A view of the fixed crane positions for superstructure erection.

Temporary erection platforms on the north side of the river.

Polyaspartic Coatings

BY AHREN OLSON, TODD WILLIAMS AND RONNIE MEDLOCK, PE

Reducing the cost of shop-painted steel bridges by improving painting efficiency.

Ahren Olson (ahren.olson@covestro.com) is the segment manager for corrosion protection and **Todd Williams** (todd.williams@covestro.com) is the protective and marine lab manager, both with Covestro, LLC. **Ronnie Medlock** (rmedlock@high.net) is vice president of technical services with High Steel Structures, LLC.

PROTECTIVE COATINGS HAVE been used to mitigate corrosion on steel bridges for more than a century.

The state-of-the-art for the past several decades now has been a three-layer system consisting of an organic or inorganic zinc-rich primer, an epoxy intermediate coat and a polyurethane finish coat (commonly abbreviated as ZEU). Each layer provides specific protection mechanisms working in unity to prevent corrosion:

1. The zinc-rich primer provides galvanic protection, with the zinc preferentially "sacrificing" itself to protect the steel.
2. The epoxy layer provides barrier properties by reducing the permeability of water, oxygen and salts through the coating.
3. The polyurethane topcoat's main function is to protect the underlying coatings from the sun's ultraviolet rays while also providing abrasion and chemical resistance.

Economics and schedule impacts have driven multiple state and local departments of transportation (DOTs) to apply all three coats in the shop for new steel bridges. This has shifted the painting responsibility to steel fabricators or blast and paint shops. For fabricators, painting provides value-added work but can also create additional scheduling complications.

Applying three coats of paint is a time-intensive process. Each layer of paint has a minimum recoat time, which is the minimum amount of time before another layer can be applied. The recoat time is dependent on product chemistry and the degree of cure required before subsequent coatings can be applied. Environmental conditions also have a significant impact on recoat time. For instance, inorganic zinc-rich primers can

Bridge #5160, which carries Main St. over the Little Madawaska River in Stockholm, Maine, was repainted with a PAS system.

require more than 24 hours at low humidity to cure before subsequent coats can be applied, thus reducing productivity. In addition, the total time to apply a ZEU system in a shop setting can vary significantly depending on the available shop space and number of painting shifts per day. The longer the recoat time, the longer the product takes up space waiting, resulting in less product that is able to be handled. Depending on work load and scheduling, a fabricator may subcontract out painting due to the bottleneck that applying multi-layer coating creates in the paint shop.

Polyaspartic Solution

Advancements in coating resin technology have improved painting efficiency. More than 20 years ago, polyaspartic (PAS) coating resins were invented by Covestro. This new coating resin replaces the "polyol" or paint resin in the "A-side" of two-component polyurethanes.

PAS coatings bring two important application and physical property advantages:

- In general, PAS coatings offer fast curing with a reasonable pot life (useable time to apply the coating). Typically, these coatings are dry-to-handle in one to two hours at 75 °F and 50% relative humidity, while having a pot life between two and three hours. By comparison, polyurethane coatings are dry-to-handle in six to eight hours, with a two- to four-hour pot life.
- They can be applied at higher dry film thicknesses (6-10 mils), which is much higher than polyurethanes (2-5 mils). The larger film build tolerance of PAS coatings allows for more forgiving application when painting complex geometries, as well as a reduction in the number of coats needed to provide corrosion protection. For instance, a ZEU three-coat system can be replaced by a two-coat system of zinc-rich primer with a PAS topcoat at the same overall film thickness.

PAS coatings are applied by the same means and methods as polyurethane coatings: spray, brush and roll. Their color and gloss retention is equivalent to polyurethanes, but they deliver better edge retention and cure significantly faster. These application and physical property advantages have been documented to increase painting productivity while reducing proj-

The Maine bridge project is a simple-span design with four steel girders spanning about 100 ft.

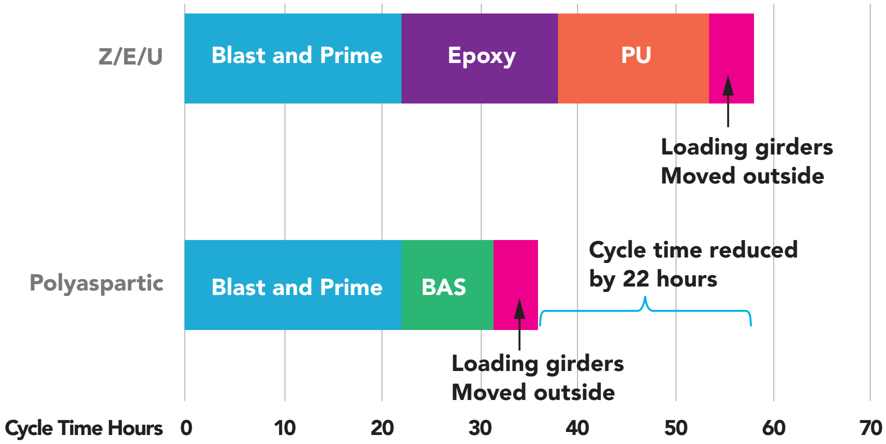


Figure 1. A graphical representation of the cycle time difference between the PAS system that was used on the Maine bridge project and a typical ZEU system.

ect costs without sacrificing corrosion protection. PAS coatings have become common in a number of different markets that shop-paint steel, including oil and gas, stadiums, railcars and structural steel.

PAS coatings have also been used in the steel bridge market for more than 15 years, and many of these applications have been in field maintenance painting. Since the early 2000s, a number of state DOTs have used PAS two-coat systems in this manner—e.g., Virginia, Maine, Connecticut, Michigan, Maryland, Pennsylvania, North Carolina and Kentucky—many of whom use salt liberally in the winter. In terms of total structures painted with PAS coatings, the Virginia DOT currently has the largest number for any one state, with more than 150 bridges.

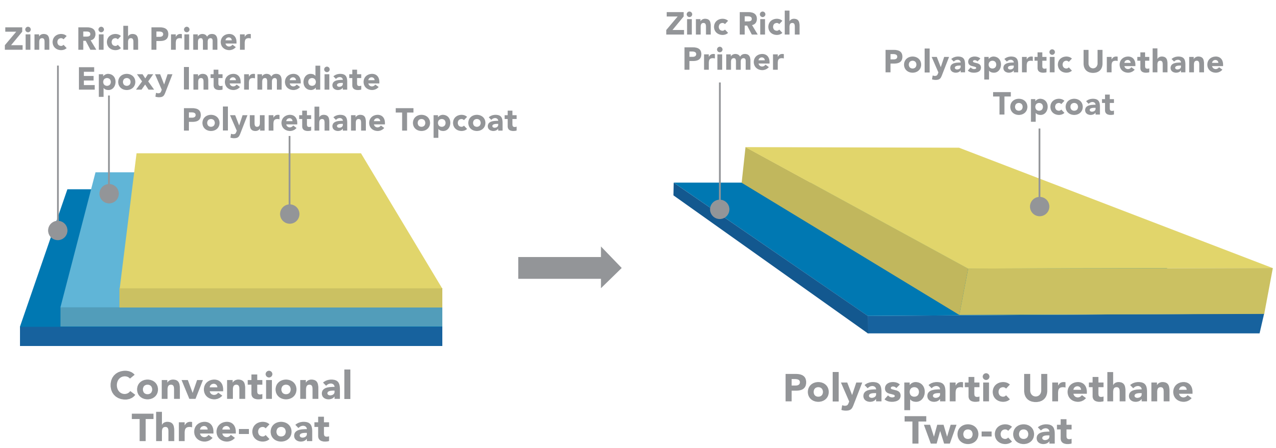
The system has proven itself. The Connecticut DOT quantified the cost benefit for field applications of PAS coatings to show a cost reduction of up to 20% and a greater than 30% improvement in maintenance painting efficiency when compared to traditional ZEU systems. In addition, the long-term corrosion resistance of PAS coatings on steel bridges has been documented to show corrosion resistance equivalent to ZEU systems.

A Virginia DOT project—I-64 over Simpson Creek in Clifton Forge—was repainted with a PAS system in 2005 and has experienced miminal rusting after 12 years in service (above photos and bottom-left photo on opposite page).

Spanning Main Street in Maine

While PAS coatings have predominantly been used for maintenance painting on steel bridges thus far, they are starting to see more use on new steel structures. One of these is bridge #5160, which carries Main St. over the Little Madawaska River in Stockholm, Maine, and was recently replaced with a new steel crossing (designed by HNTB and fabricated and detailed by NSBA member and AISC certified fabricator High Steel Structures). The design for the replacement structure is a simple-span bridge using four steel girders and spanning roughly 100 ft. The bridge was constructed with weathering steel girders with painted beam ends approximately 5 ft from both abutments. While the coating system was initially planned to be ZEU, the Maine DOT showed interest in PAS coatings after successfully using the technology for field maintenance painting, and as a result allowed a change order for the coating system. A two-coat system consisting of an organic zinc-rich primer with a PAS topcoat was eventually specified.

Beam ends were blasted to SSPC-SP 10 prior to primer application. Following surface preparation, the zinc-rich primer was applied per manufacturer requirements at 3-5 mils dry film



Layers of a standard three-coat ZEU system and a PAS two-coat system. Both systems have total dry film thicknesses ranging from 9 mils to 14 mils.

A close-up view of one of the painted beam ends on the Main St. bridge project.

A Michigan DOT project—West Road over I-75 in Woodhaven, Mich.—was repainted with a PAS system in 2017.

thickness. After the primer was applied and inspection was complete, the PAS finish coat was applied using a single-component airless pump. The final inspection on the finish coat began four hours after completion of the application. After final inspection, the beams were loaded and moved outside to the lay-down yard. The total cycle time for blasting, painting and moving the finished product outside was 36 hours.

In order to provide a comparison between the two-coat PAS system and the traditional ZEU, a second timeline was put forward based on years of experience with ZEU systems. Both timelines assume the paint bay has three shifts. The total cycle time for the ZEU system for the same beam end project would be 58 hours (see Figure 1 on p. 40 for a graphical comparison of the time cycles between the PAS and ZEU systems). This timeline for the ZEU system also assumes ideal environmental conditions (temperature and humidity). Using the two-coat PAS system reduced the cycle time by 22 hours compared to the ZEU system. This 61% increase in throughput is attributed to reduced curing time and one less coating layer. The PAS system has a combined approximately six hours of curing "downtime" while ZEU has around 26 hours of curing downtime. One less layer for the PAS system also requires one less inspection, saving an additional two hours or so of cycle time. The PAS systems enables a significant improvement in the throughput and painting efficiency of the paint shop, essentially increasing a fabricator's painting capacity without having to add additional shop space or resources. In periods of high demand, PAS coatings can improve scheduling as well as require less painting work to be subcontracted out to third parties.

Reducing the number of paint layers improves the throughput and also generates cost savings through a reduction in coating application and steel handling costs in the painting process. While the material cost of a PAS system can be double that of a ZEU system, coating application and handling costs can be greatly reduced since, again, only two layers need to be applied versus three. In the case of the Maine project, the PAS system generated a 28% savings in coating application and steel handing in the painting operations. Considering both raw material cost increase and the coating application and steel handling savings, the

A Connecticut DOT project—I-75 over Starr Ave. in Danbury—was repainted in 2002 with a PAS system. After 15 years in service, minimal rust has been experienced.

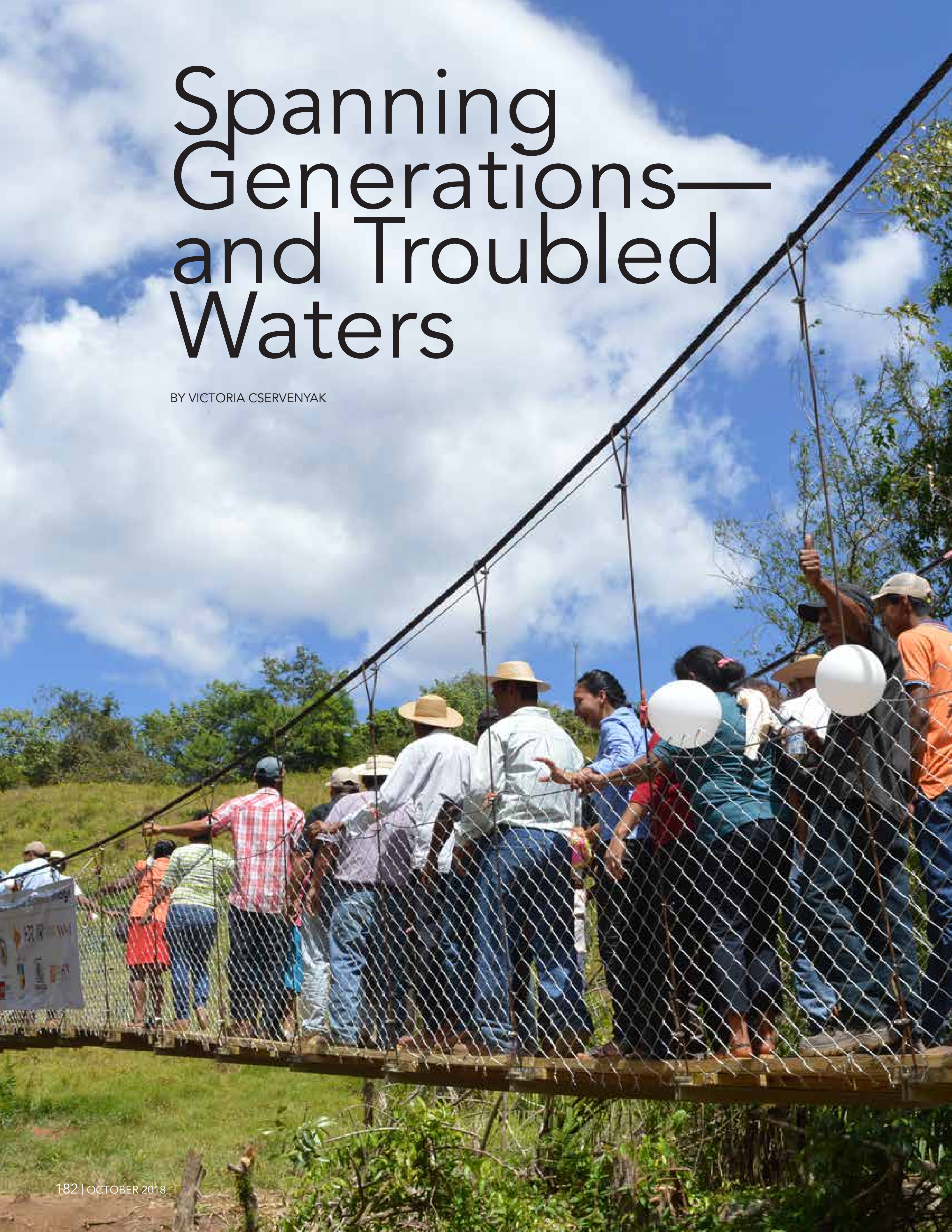
PAS system created an overall cost reduction for painting of 14%, which factored to a 2% reduction in the total cost of the new fabricated and painted steel girders.

As the trend to shop-apply all coats of paint for new steel bridges continues, PAS coatings offer an option to deliver significant value to both fabricators and bridge owners requiring shop painting of new steel bridges. By reducing cycle time using PAS, steel bridge fabricators can gain additional painting capacity, and this can be very significant in periods of high painting demand. Ultimately, this will lead to time and cost savings for owners who can leverage the advantages of PAS systems into solutions for new steel bridges without having to sacrifice long-term corrosion resistance.

This article is a summary of Session B25 "Advanced Coating Systems" from the 2018 NASCC: The Steel Conference/World Steel Bridge Symposium in Baltimore. Next year's conference takes place April 3-5 in St. Louis. Learn more at ***www.aisc.org/nascc****.*

Spanning Generations—and Troubled Waters

BY VICTORIA CSERVENYAK



A team of bridge professionals connects a remote Panamanian village to nearby communities with a new bridge over a treacherous river.

IN AN ISOLATED jungle community 30 minutes from the nearest town, an 80-year-old woman hesitantly crosses a suspension bridge over the Tuancle River nearly 100 ft below to her family on the other side.

Following behind, two children laugh and romp along the bridge, gliding their little hands across the chain-link fences on the sides.

At the foot of the bridge, a man speaks about how going forward, this day will have a happy meaning for him and his family, who several years earlier had a son die on the same date.

The 100-ft-long El Macho Puente (puente is Spanish for bridge) opened this past spring and the nearly 200 residents of the village—for which the bridge is named—are celebrating their newfound freedom of safely traveling to and from their Panamanian hamlet over the river.

New Bridge, New Hope

But less than a week before this celebration, no bridge existed.

Spring is the dry season in El Macho, which lies about 200 miles west of Panama City. So the river, which geographically quarantines the remote area from larger

Victoria Cservenyak (cservenyak@aisc.org) is AISC's digital communications manager.

left: El Macho residents test their new bridge.

above: Building the bridge over the Tuancle River.

Carter Bearden and Gary Kinchen installing decking near the middle of the bridge span.

Children excitedly skipping across the bridge for the first time.

towns, is only about 1 ft deep and even becomes a dry riverbed in some places.

But it's a different story during the rainy season, when villagers are sometimes forced to traverse the rocky riverbed to reach nearby communities—and are sometimes simply unable to make it. During the rainy season when flash floods are common, the water can surge to more than 6 ft high, making the river impassable. Recently, a few men were carrying a sick friend on a hammock, attempting to take him to the doctor. When they arrived at the river, the water was too high and before it could recede enough to cross, the man died.

Six months before the first bridge tower was installed, plans commenced to build a footbridge in El Macho across the river. And over the course of three months prior to the bridge's opening, Maria Rodriguez, the Panama country manager; Daniel Magallon, Bridges to Prosperity (B2P) mason; Chase Luckey, B2P fellow (volunteer); and the El Macho community worked to create the foundation.

Camaraderie and Colleagues

Since leading his first volunteer trip with B2P in 2016, Jeff Carlson, NSBA's director of market development, has been enthusiastically committed to the organization's mission. Whenever he meets with the AEC community, he evangelizes about the need for footbridges in rural areas throughout the world—which is how the El Macho team formed. In addition to Carlson, team members included Carter Bearden (HDR); John Hastings (Tennessee DOT); Marne Helbing (Tennessee

Patrick Montgomery and team putting together scaffolding.

John Hastings and Jeff Carlson installing hanger assemblies.

The B2P team celebrating the completion of the El Macho Bridge—and holding up Jeff Carlson.

DOT); Gary Kinchen (New Mexico DOT); Patrick Montgomery (Fought and Company, an AISC member and certified fabricator); Carlos Ramirez (WSP); Michelle Romage-Chambers (Texas DOT); and Scott Wilson (Palmer Engineering).

When the team began their trip, they were not familiar with each other, yet bonded as they worked seamlessly without construction issues or personality clashes.

"The most memorable part of the bridge for me was twofold," said Carlson. "First, everyone on the team worked well with one another. They were all respectful of their fellow teammates, the B2P staff and the local community. Second, I was impressed by how organized the B2P Panama staff was for our project."

"Our group had a lot of camaraderie," added Wilson. "We could have fun and at the same time all work hard towards the same goal, which was a benefit I didn't expect."

The group's gregariousness helped each team member to adroitly and quickly discover how to best use their individual skills to benefit the group as a whole. As a fluent Spanish-speaker, Ramirez harmoniously coordinated the community members and Kinchen cheerfully supervised the fabrication and rebar cutting on the ground, while the other team members constructed the towers and assembled the remaining pieces. Montgomery, as a fabricator, attempted not to heckle his team members, who were adjusting from their usual computer work to onerous manual labor.

"Most bridge designers are not used to hands-on experience, and to suddenly take a concept on a piece of paper and translate that into an actual built structure was a challenge at first," Kinchen explained.

However, Montgomery was happily astounded by his teammates' enthusiasm. "All the engineers were down-to-earth and ready to go to work," he said. "You hear that engineers are going to be finger-pointers. But every single one of them wanted to get their hands dirty. And they did."

Kind-Hearted Community

With both the temperature and humidity in the 90s (degrees and percent, respectively) the team spent the first day acclimating to the steamy climate, then dove in to work side by side with the El Macho residents to construct the bridge. It was essential to the B2P team that the community members take an active role in construction so they would know how to make future repairs to the bridge as necessary. Throughout the week, between 15 and 25 community members assisted with construction, spanning from 12-year-olds to octogenarians, and even more inhabitants made the American team feel extremely welcome. Each morning, two women walked for two hours to cook breakfast over a fire. Families invited the group into their concrete-walled, dirt-floor homes for lunch, and other community members cooked them dinner at the campsite at night. The villagers even built a hut made out of palm branches for the workers to take a reprieve from the blistering sun.

Once the crew arrived in El Macho, two weeks were allotted for building the bridge and despite a few minor injuries, they completed the project in six days.

"It's so gratifying to do something else that goes along with the skills that you have, especially in places where they're desperately needed," Ramirez said.

In addition to the gratifying work, the friendships formed also made the trip an unforgettable experience. Getting to know the community was the highlight of everyone's trip.

"It's really a neat relationship that you gain working with them and working with a lot of people you don't know; you get to know them well over that two-week period," Hastings said. "Everybody was wonderful. The whole experience was wonderful."

Although they started off as strangers, the team members were so invigorated by their journey to Panama that they have already begun to plan the next B2P opportunity.

Building Bridges

Through local engagement, from regional governments to members of each partner community, Bridges to Prosperity (B2P) is committed to a sustainable model that puts the focus on people and the opportunities that make it possible for them to thrive. In 2018, B2P will complete 39 new footbridges, increasing its overall total to 279 bridges and impacting more than 1,000,000 people since 2001.

To learn more about B2P, how you can become a volunteer or industry partner or to support its mission, visit www.bridgestoprosperity.org.

The completed El Macho Bridge before the inauguration ceremony.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