Steel Bridges 2019–2020

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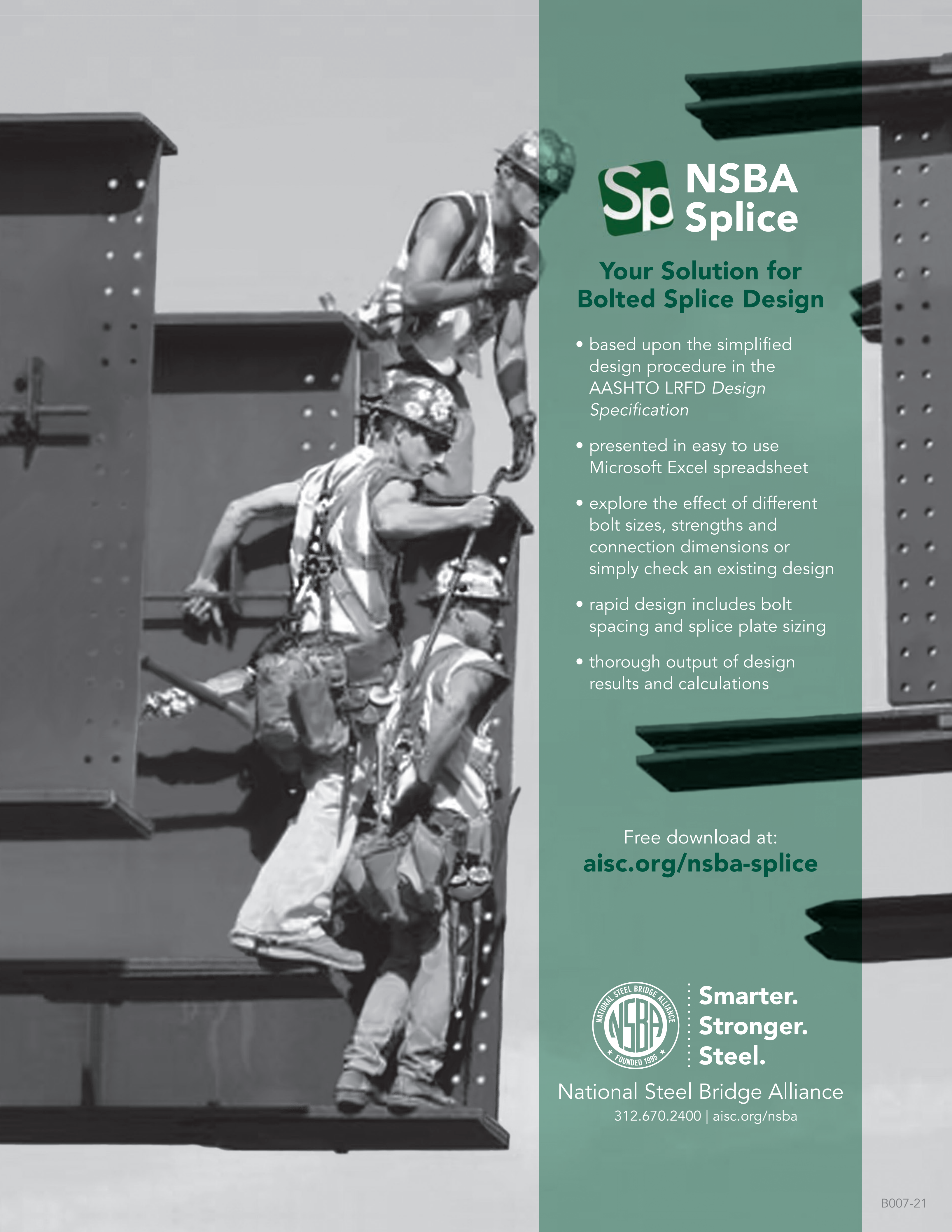
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Greg Davis, courtesy of Massman

2019

2020

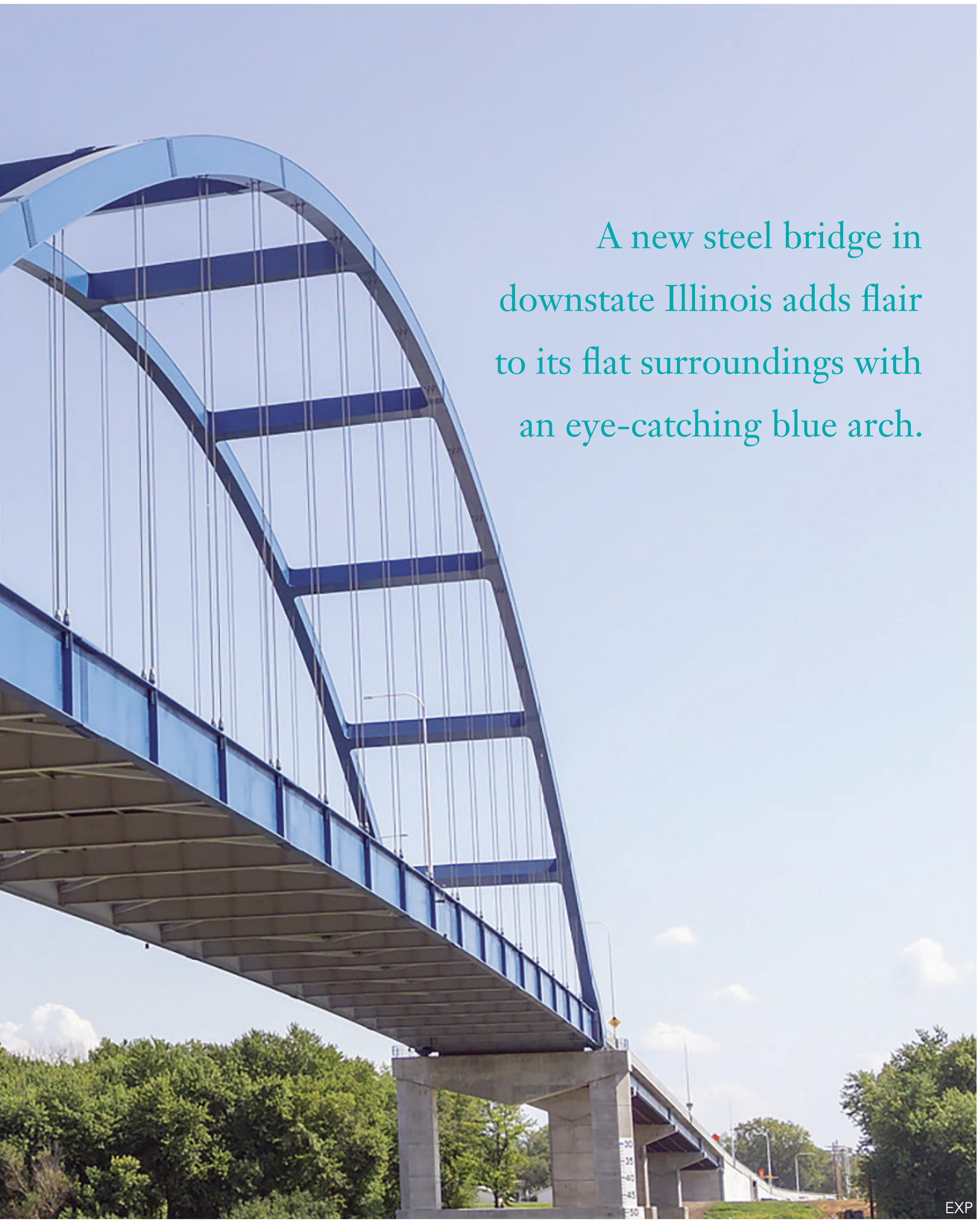
Artistic Arch

BY SHANKAR NAIR, SE, PE, PHD,
VINOD PATEL, SE, PE, NADIA ABOU,
AND SARAH WILKINSON

IN THE VILLAGE of Meredosia, Ill., small-town charm is a staple of everyday life. Located roughly 45 miles west of the capital city of Springfield, the town's 1,000 residents relied on an existing crossing, built in 1936, of Illinois Route 104 (IL-104) over the Illinois River. By 2007, the bridge was deemed structurally deficient and the following year, design firm EXP was authorized to begin planning for its replacement.

As Meredosia and the region awaited a new bridge crossing the Illinois River, the big questions became where and whether a river crossing would be available during the bridge replacement. The bridge served as a backbone to the region's mobility and transportation network, and its closure would have required an extensive detour and greatly impacted the region's commercial traffic and many farmers. A new bridge bypassing the village would not be good for the local businesses. Finding an optimum solution for the bridge's replacement required creativity and close collaboration with the community and several agencies. An extensive environmental assessment and a context-sensitive solutions study were completed, evaluating multiple alignments and bridge types in search of a solution that would be efficient and aesthetic while still keeping the traffic flowing through the village.

The study determined the ideal location of the replacement bridge to be approximately 255 ft north of existing bridge, as it met the project's primary purpose of providing a reliable and safe river crossing as well as the local and regional economic needs. After selecting the alignment, a bridge-type study evaluated truss, tied-arch and cable-stayed configurations for the new bridge. A tied-arch bridge was selected for ease of inspection and maintenance as well as the Illinois Department of Transportation's (IDOT) experience and familiarity with tied arch bridges. The new 2,125-ft-long bridge features a 590-ft-long tied-arch main navigation span and nine welded plate girder approach spans ranging from 142 ft to 200 ft, requiring approximately 3,360 tons of steel in all. The new bridge carries two 12-ft lanes and two 10-ft shoulders and provides nearly 74 ft of vertical navigation clearance above normal pool. The bridge's design required complex analysis and used state-of-the-art programs to determine loads and perform design checks. With the arch rising nearly 200 ft above the water, the new crossing opened this past summer to an excited crowd and will serve as an attractive gateway to the region for the next 100 years.



EXP

A Tied Arch Seen from the Distance

Part of this attractiveness is due to the paint job. Now known by residents and onlookers for its piercing blue arch, and serving in stark juxtaposition to its surroundings, the new bridge features a simple and direct approach to design and steel detailing. Form followed function in design, aesthetics and implementation, a strategy that allowed the bridge to take its distinctive presence.

During the construction phase, the previous bridge remained open to traffic to reduce mobility restrictions and economic impacts to the village. The contractor elected to erect the arch through cantilevered erection, with stay towers erected on top of the main river piers and inclined back-stays anchored to the approach superstructure near the adjacent approach pier on both sides of the arch span. The uplift component of the tension in the back-stays was resisted by vertical hold-down cables anchored in to the approach pier footing and the horizontal component was balanced by the thrust at the arch knuckles, all through the approach superstructures. This allowed for the completion of the 590-ft span without falsework towers in the river and an unrestricted navigation channel for the river traffic. The fully constructed arch span, including the concrete deck, was designed for prestressed assembly to counteract flexure in the ribs and ties due to dead load. The bridge was also designed for additional load combinations beyond those required in AASHTO LRFD, as arches can be particularly sensitive to certain unbalanced load patterns.

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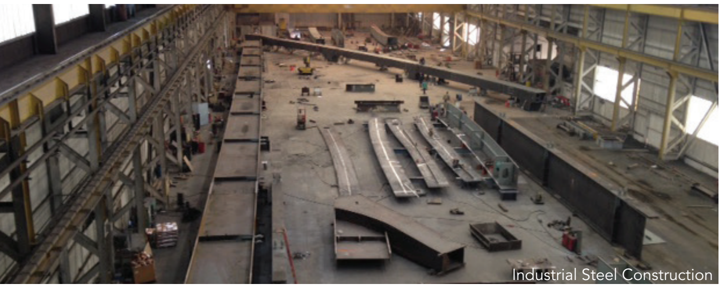
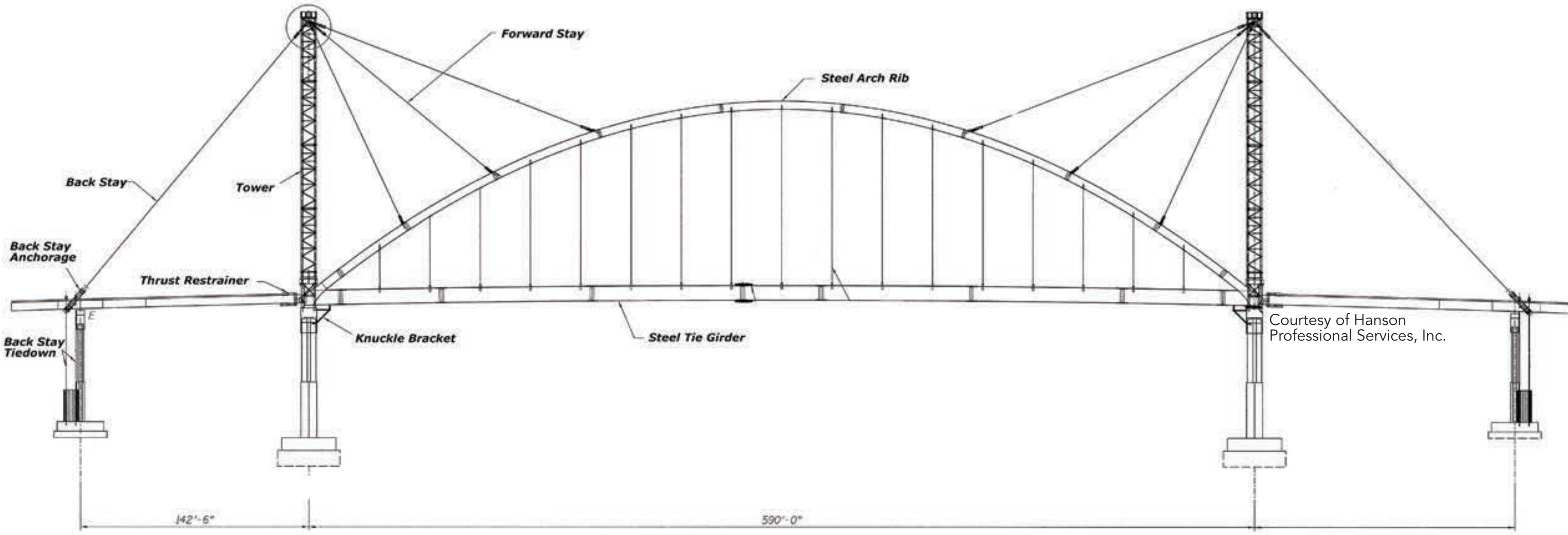
The blue arch creates a new structural icon for the area.



Fabrication of the new 2,125-ft-long bridge, which features a 590-ft-long tied-arch main navigation span and nine welded plate girder approach spans ranging from 142 ft to 200 ft, requiring approximately 3,360 tons of steel in all.



Courtesy of Hanson Professional Services, Inc.

An elevation view of the new bridge during erection.

The main span of the bridge, coming together in the middle.

above and below: The new bridge carries two 12-ft lanes and two 10-ft shoulders and provides nearly 74 ft of vertical navigation clearance above normal pool. The bridge's design required complex analysis and used state-of-the-art programs to determine loads and perform design checks.

The substructures consist of stub-type abutments and architecturally enhanced hammer-head approach piers and dual-column portals for the main span, all founded on steel H-pile foundations. The foundations were designed with all vertical H-piles counting on the soil-structure interaction to resist lateral loads. Using all vertical H-piles as opposed to conventional batter piles resulted in a simple and constructable pier and cofferdam system, and helped reduce foundation costs by 40%.

The tied arch span uses 9-ft-deep I-shaped tie girders that greatly reduce fracture-critical and long-term inspection and maintenance concerns. The curved arch ribs (box sections) were all fabricated by Industrial Steel Construction, including bending the top and bottom flanges of the rib box sections, then welding the flanges and the side (vertical) web plates together to form the curved rib segments. Nicholas Petkus, the company's vice president of sales and estimating, worked closely with the EXP design team and comments, "The I-Girder design of the ties proved to be very economical, in both cost and time, compared to the welded or bolted boxes that we typically see. This helped, not just throughout the fabrication process, but also in the standup assembly of the deck floor system. When it came time to drill the connections, it was not necessary to have our people working inside the boxes; this was a major time saver. The girders were easier to handle compared to boxes. This switch was the most interesting on the project."

The uniqueness of the I-shaped tie resulted in an unusual but efficient rib to tie connection at the arch knuckle. Careful detailing of features such as simple floor-beam-to-tie-girder connections, Vierendeel arch rib bracing with large strut spacing and struts offset from the hanger locations made the structure more efficient and easier to construct.

The new IL-104 bridge required extensive coordination amongst the owner, agencies, designer, contractor, steel fabricator and community to create a transportation landmark to replace an obsolete and structurally deficient span. Innovative design methods, coordination and precise steel fabrication resulted in the enhancement of a traditional tied arch bridge, yielding a striking new gateway to let visitors and locals access Meredosia's small-town charm. ■

Owner
Illinois Department of Transportation (IDOT)

Structural Engineer
EXP, Chicago

Contractor
Halverson Construction, Springfield, Ill.

Construction Engineering
Hanson Professional Services Inc., Springfield

Steel Team

Fabricator
Industrial Steel Construction, Gary, Ind.

Detailer
Graphics for Steel Structures, Inc., Hicksville, N.Y.

A new steel span provides safe pedestrian passage over a busy street to a prominent gathering spot along the Charles River in Boston.

Take Me to the River

BY WILLIAM GOULET, SE, AND MARIAN BARTH, PE

William Goulet (william.goulet@stvinc.com) is a senior structural engineer and **Marian Barth** (marian.barth@stvinc.com) is a project manager, both with STV, Inc.

THE NEW FRANCES "FANNY" APPLETON BRIDGE, named for the second wife of Henry Wadsworth Longfellow, is appropriately adjacent to Boston's historic Longfellow Bridge.

The bridge, which opened this past summer, replaces an existing decaying pedestrian bridge that, due to narrow switchbacks, did not meet current accessibility standards and could not accommodate the mixed use of people on foot and bicycles. The bridge crosses Storrow Drive in Boston and provides an important connection for pedestrians and cyclists from the adjacent Longfellow Bridge and Charles Circle to the Esplanade parkland that extends along the Charles River. The Esplanade is the location of the 4th of July fireworks and Boston Pops concert, attracting hundreds of thousands of people each year.

Undertaken as part of the Longfellow Bridge Rehabilitation Design-Build Project by the Massachusetts Department of Transportation–Highway Division, the new steel bridge—consisting of 280 tons of steel, all metalized and painted with a mid-coat and a topcoat—provides a modern Vierendeel arch structure that contrasts with the traditional arches of the historic Longfellow Bridge. The coating requirements were changed from galvanizing to thermal sprayed metalizing, which eased shop assembly of the larger parts.

Sporting a ribbon-like appearance, the 550-ft-long bridge superstructure, with 100-ft ramp abutment structures at either end, runs through the existing park, weaving in between the trees. The main span is a slender arch whose geometry was primarily determined by site constraints, roadway clearances and the need to maintain accessibility standards. The arch, spandrel columns and approach piers all use hollow structural sections (HSS) while a pair of continuous tub girders run from abutment to abutment. The bridge also incorporates Y-shaped piers that branch out to support the two longitudinal tub girders. Steel castings designed and supplied by Cast Connex Corporation (an AISC associate member) were used to connect the vertical columns of the piers to the angled supports.

right: The point where the tub girders split at the east stair.

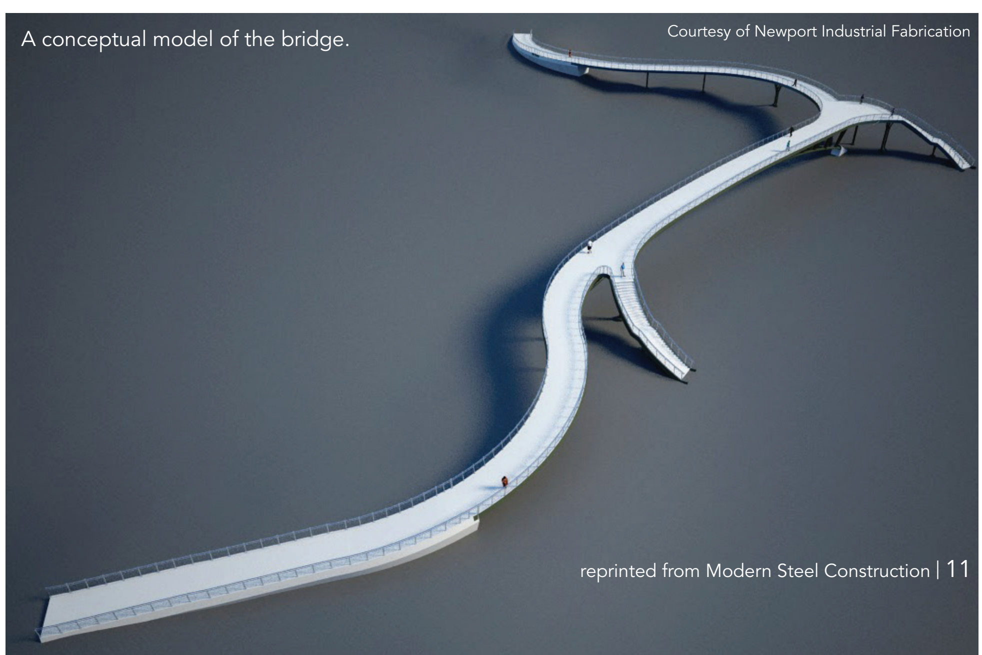
below: An elevation view of the Fanny Appleton Bridge's main span and east stair.

Courtesy of STV

Slender yet Complex

The slender design, complex geometry and site constraints required a design that could be fabricated and erected with minimal disruption to the area while providing the desired aesthetic and expected performance of a modern pedestrian bridge. The continuous fascia plate of the girder that produces the ribbon appearance was held to dimensional tolerances one-half of those typically used for fabrication, with the intent of minimizing horizontal or vertical waves in the fascia plate that would be noticeable to the public. The revised tolerance criteria were also applied to the tub girders that support the fascia plate, as variations in the tub girder would be reflected in the plate. All connections were detailed to be unobtrusive and were welded wherever possible. Connections that needed to be bolted for constructability purposes used splice plates on the interior of the tub girders and had bolts orientated so only the heads of the bolts are visible. Weld access holes at the girder shop splices were also

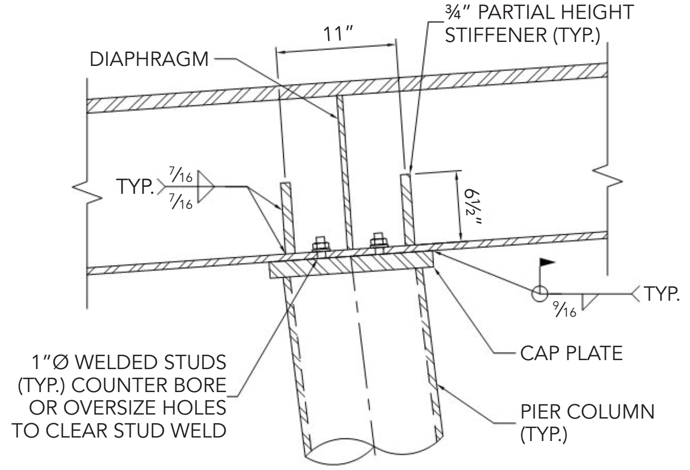
Courtesy of STV



A conceptual model of the bridge.

Courtesy of Newport Industrial Fabrication

Courtesy of Cast Connex Corp.



left: A casting for the Y support connections, made by Cast Connex.

above: Welded studs for alignment at the piers.

The most extreme forming condition involved a reversing *R*=2-ft curve with a 15° off-plumb profile at a grade change.

Courtesy of Newport Industrial Fabrication

filled with custom 3D printed plastic plugs to maintain an uninterrupted visual appearance. This provided an economical solution that maintained the desired appearance while not incurring the cost of plug welds and the associated grinding.

Engineer of record STV's designers recognized early on that a slender main span and flexible pier system could provide the potential for pedestrian-induced vibrations. Multiple publications that address vibration, including AISC's Design Guide 11: *Floor Vibrations Due to Human Activity* (Second Edition, available at **aisc.org/dg**), the AASHTO *LRFD Guide Specifications for the Design of Pedestrian Bridges* and the SETRA guide *Assessment of Vibrational Behavior of Footbridges under Pedestrian Loading* were used to evaluate the bridge at different stages in the design process and to evaluate the as-built structure. The various guides provide different analysis methods and varying loading assumptions, and it was determined that the group loading assumptions in SETRA were most appropriate given the size and anticipated use of the bridge. The allowable accelerations from the other guides were also compared to the allowable limits provided by SETRA to confirm that the appropriate limits were being used.

Modifications to the structure were made to improve the dynamic performance of the bridge. However, providing strength and stiffness needed to be balanced with the fact that a continuous curvilinear structure would require a certain amount of flexibility to accommodate thermal movements. Some elements, such as the spandrel columns, were used to improve dynamic performance with only minor impact on the influence of thermal forces. The spandrel columns were varied to provide added stiffness in areas of the Vierendeel arch that were determined to be most critical, while at the same time maintaining a

Courtesy of Newport Industrial Fabrication

above: Transporting massive curved sections to the site.

Courtesy of Newport Industrial Fabrication

above: Cutting 18-in. HSS used for the support columns.
left: Assembling a portion of the superstructure in the shop.
below: A view of the bridge and its position related to the Longfellow Bridge to its left.

Courtesy of Newport Industrial Fabrication

Courtesy of STV

lighter structure where the spandrel columns had little influence. The spandrel column connections to the tub girders were welded moment connections and were provided with internal stiffeners at all locations. Although some spandrel columns did not require stiffeners to provide adequate strength at the connections, this lack of stiffeners would provide a significantly more flexible connection due to the width of the tub girder bottom flange—and this added flexibility would have been detrimental to the dynamic performance of the bridge.

Piers and Tub Girders

The approach pier structures required a more balanced approach for dealing with vibration and thermal forces. The piers were connected to the continuous tub girders using welded connections similar to the spandrel columns, so an increase in pier stiffness would also result in increased moments due to thermal expansion of the approaches. The foundation-soil interaction became a critical part of the structural design for two main reasons. First, if foundation stiffnesses were overestimated, the steel structure would still be able to accommodate all of the design forces—but the vibration analysis of the structure would prove unconservative. Second, if the foundation stiffness were underestimated, the vibration analysis would be conservative, but pier forces might exceed those used in design. To deal with these competing issues, both the upper and lower bounds of the foundation stiffnesses were determined. In addition to the multiple stiffnesses used in the design, the team also investigated the possibility of the ground freezing and imparting additional forces into the piers due to the added restraint. Analyses included running a portion of thermal loads with the typical foundation springs and a portion with

Lifting a long bridge assembly at Newport's facility.

Courtesy of Newport Industrial Fabrication

Courtesy of STV

above and below: Erecting the brige over Storrow Drive.

Courtesy of Newport Industrial Fabrication

fixed supports, and applying a service live load since the design load of 120 psf would be unlikely at a temperature of -30 °F.

The continuous tub girders at the deck level presented multiple challenges for laying out the framework of the bridge. Curved staircases frame into the main span from either side and provide a horizontal restraint to the main span. As the framing for the stairs diverges from the main span steel, the tub girders needed to be split so that the exterior appearance would remain consistent with the fascia plate while maintaining the same relative location to the tub girder. The tub sections start with two webs then split, ending with four webs. In working with fabricator Newport Industrial Fabrication, the design team decided that the flanges would be cut to a shape that would provide a radiused transition at the splits. The webs of the girders would have to blend into the main girder web if they were to follow the shape of the bottom flange and would require welding the plates at a sharp angle. To avoid this sharp-angled weld and an abrupt end of the plate at the weld location, it was decided to curve the web plate so that it would return perpendicular to the main tub web. Internal diaphragms and cross frames were located to aid in transferring shear from the incoming webs.

Challenging Curves

The curved tub-girder and stair members were fabricated as built-up sections with each plate cut to shape and were formed in-house by Newport as needed prior to assembly of the section. The 1-in. fascia plate was challenging to form, as the slope in the fascia plates and fascia girders mathematically created a "warped surface." While software exists that can perform the necessary flattening function to create blanks, it did not provide adequate forming data. As such, Newport developed custom numeric modeling software to flatten the surface, which allowed shop engineers to program bump scribes on the CNC plasma table and provide the press brake operator with the specific location and direction of each press strike.

Courtesy of Newport Industrial Fabrication

above: The superstructure, taking shape at Newport's facility.

below: The completed bridge at the west end, near the Charles River's edge.

Courtesy of STV

Knowing the bump frequency, the angular deflection required to approximate the curve was calculated, and the press brake operator used these two pieces of information to build a highly accurate part on the very first try. The most extreme forming condition involved a reversing 2-ft-radius curve with a 15° off-plumb profile at a grade change. Newport began by burning two blanks, assuming it would take at least two attempts to achieve the proper shape. The forming methodology was so reliable that the first attempt was successful and the second blank unnecessary. When it came to the steel arches, the 18-in. × 1.375-in. pipe used for these elements was too large for Newport's equipment, and bender-roller Kottler Metal Products was employed to take on this bending work.

The three main steel sections of the bridge required careful coordination for delivery, due to their length and width, as they were transported through Charles Circle. While delivering such large components was tricky, it allowed Newport to maintain control of the geometry in the shop and greatly reduced field work. The laydown site consisted of a small area between the Charles Circle off ramp, Storrow Drive and the Longfellow Bridge since all of the pathways on the Esplanade side were required to remain open to the public during construction.

Working closely with the contractor and fabricator from the beginning of the project, the team was able to achieve the design goals, fabricate to the stringent and complex geometries and achieve the required erection tolerances. As a result, Boston now has an attractive new pedestrian route to the Charles River waterfront. ■

The Fanny Appleton Bridge is featured in the presentation "Pedestrian Bridges: Unique Design and Analysis" at the 2019 NASCC: The Steel Conference, taking place April 3-5 in St. Louis. For more information, visit ***aisc.org/nascc,*** *where you can also view a recording of the presentation approximately 45 days after the conference.*

Owner

Massachusetts Department of Conservation and Recreation

General Contractor

WSC-Joint Venture of J.F. White Contracting Co., Skanska and Consigli Construction Co., Boston

Structural Engineer

STV, Incorporated, Boston

Steel Team

Fabricator and Detailer

Newport Industrial Fabrication AISC CERTIFIED FABRICATOR, Newport, Maine

Erector

Saugus Construction Corp. AISC ASSOCIATE AISC CERTIFIED ERECTOR, Georgetown, Mass.

Bender-Roller

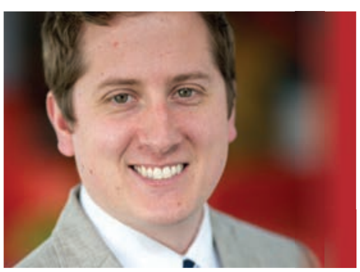
Kottler Metal Products AISC ASSOCIATE, Willoughby, Ohio

steelwise

OPPOSITES ATTRACT

BY CHRISTOPHER HEWITT, SE, PE, PENG, ALAN HUMPHREYS, PE, PHD AND ERIC TWOMEY, SE, PE

A primer on galvanic corrosion of dissimilar metals.



Christopher Hewitt (**cmhewitt@sgh.com**) is a senior project manager, **Alan Humphreys** (**aohumphreys@sgh.com**) is a senior metallurgist, and **Eric Twomey** (**ejtwomey@sgh.com**) is a senior structural engineer, all with Simpson Gumpertz and Heger (SGH).

ENGINEERS AND ARCHITECTS often encounter conditions where dissimilar metals are in contact.

When these situations arise, it is appropriate to consider the corrosion susceptibility of the materials. Although the term corrosion can be broadly applied to any degradation of a metal and can have a variety of causes, this article will discuss the main drivers of galvanic corrosion, which occurs between two dissimilar metals, and offer some strategies to help engineers assess and mitigate its effect in structural applications.

What is Galvanic Corrosion?

Galvanic corrosion typically occurs when dissimilar metals are in electrical contact with each other in wet or humid conditions. It is caused by an electrochemical reaction between the two metals where the transfer of electrons from one metal to the other causes one metal to be oxidized (corroded) at the expense of the other. The reaction occurs when the materials have been connected in a galvanic cell, which is made up of four essential elements (see Figure 1 for an illustration of the relationship between these elements):

1. An **electrolyte** is a conductive liquid or gel that allows the transfer of electrons between the two metals—e.g., water.
2. An **anode** is the negative terminal of a galvanic cell, from which electrons are transferred, resulting in oxidation and section loss of the metal. The anodic metal has the greater negative electrical potential of the two metals in contact.
3. A **cathode** is the positive terminal of a galvanic cell, to which electrons are transferred. No degradation occurs at the cathode. The cathodic metal has the lesser negative electrical potential compared to the anode.
4. An **electrical connection between the anode and cathode** that allows the electrochemical reaction to occur. In immersed conditions, the electrolyte may produce the electrical connection.

If an electrolyte is not present or if the metal ions don't have a mechanism that will allow them to transfer between the materials (i.e., the metals are not in contact) then galvanic corrosion cannot occur. Surface wetting, which can act as an electrolyte, typically occurs when the relative humidity of the environment is greater than 80%. In a marine environment where chloride contamination of a surface has occurred, surface wetting can occur with a relative humidity as low as 30%, which in practice results in the permanent presence of an electrolyte on the surface of a structure. In addition, the rate of galvanic corrosion increases with the conductivity of the electrolyte, so rapid galvanic corrosion can occur in marine environments.

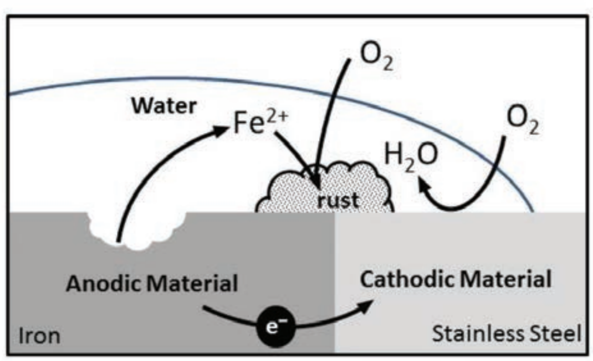
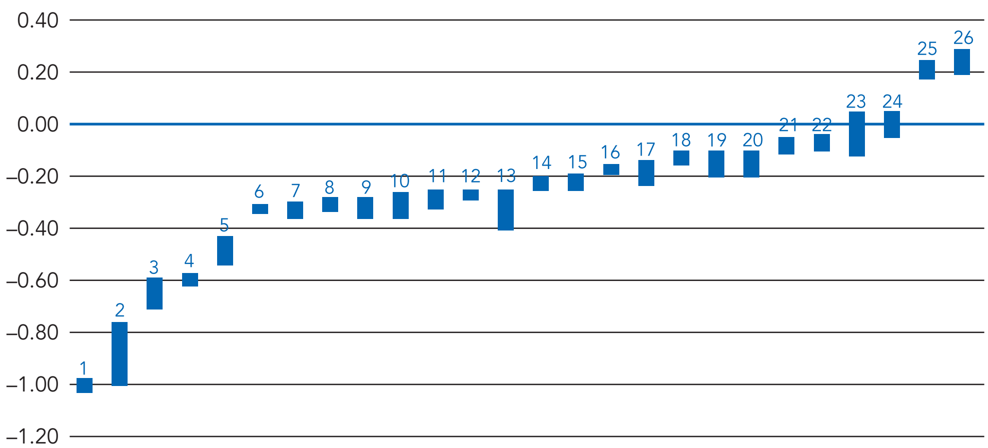


Fig. 1. An example of a galvanic cell.



1. Zinc
2. Aluminum alloys
3. Low-carbon steel, cast iron
4. Low-alloy steel
5. Austenitic nickel cast iron
6. Tin
7. Copper
8. Manganize bronze
9. 505n-50Pb solder
10. Stainless steel (AISI types 410, 416) – passive
11. Tin bronze
12. Silicon bronze
13. Brass
14. Stainless steel (AISI types 430) – passive
15. Lead
16. Inconel 600 – passive
17. Nickel-aluminum bronze
18. Silver
19. Nickel
20. Silver brazing alloys
21. Stainless steel (AISI types 302, 304, 321, 347) – passive
22. Stainless steel (types 316, 317) – passive
23. Titanium
24. Alloy 20 stainless steels – passive
25. Platinum
26. Graphite

Fig. 2. Galvanic series of common metals: potential energy (in volts) when in saltwater.

In many cases, the effects of galvanic corrosion are negligible. The severity of galvanic corrosion depends on the difference in the electrical potential between the contacting metals. A chart showing the relative potentials of different metals, known as the galvanic series, is shown in Figure 2. The galvanic series is given in the figure based on saltwater conditions. However, the actual electrical potential of a metal will vary based on the electrolyte. For more exotic metal alloys, the electrical potential can be measured in a test laboratory according to ASTM G82: *Standard Guide for Development and Use of a Galvanic Series for Predicting Galvanic Corrosion Performance*. Note that stainless steels have different potentials according to whether they are passive or active (corroding) due to a breakdown of their passivating layer. The galvanic series shown in Figure 2 provides the electrical potential for stainless steels in their passive (normal) state rather than their active state, as this condition of stainless steel has an electrical potential that is further away from that of carbon steel.

In general, galvanic corrosion will not occur if the difference in potential is less than 200mV, but in an aggressive environment dissimilar metal corrosion can occur with a potential difference as small as a few tens of millivolts.

Galvanic corrosion is also affected by the relative surface areas of the metals in contact. If the surface area of the anode is much smaller than the surface area of the cathode, then the flow of electrons will have a high current density in the anode, resulting in rapid corrosion of the anodic metal. If the surface area of the anode is much greater than the area of the cathode, then the current density at the anode will be low and the corrosion will typically be negligible. As a rule of thumb, a cathode-to-anode surface ratio of at least 10:1 is optimal for minimizing galvanic corrosion.

The severity of galvanic corrosion can be predicted for construction materials of different potentials and surface areas using design charts—and of course, fasteners have a smaller surface area than structural members. See Figure 3 for a chart listing the corrosion susceptibility of different fastener/member metal couples.

Fig. 3. Galvanic corrosion potential between steel and common construction metals.

Galvanic Corrosion of Dissimilar Metals		Fastener					
		Stainless Steel	Copper	Brass	Carbon Steel/ Iron	Aluminum Alloys	Galvanized Steel (Zinc)
Approximate Electrical Potential, Measured in Volts*		-0.05 to -0.25	-0.36	-0.25 to -0.4	-0.61	-0.80	-1.00
Member	Carbon Steel/Iron	Member may corrode	Member may corrode	Member may corrode	—	Fastener may corrode	No significant corrosion**
Member	Galvanized Steel	Member may corrode**	Member may corrode**	Member may corrode**	No significant corrosion**	No significant corrosion	—
Member	Stainless Steel	—	No significant corrosion	No significant corrosion	Fastener likely to corrode	Fastener likely to corrode	Fastener likely to corrode

* Volts in saltwater. Note that compatibility of materials should be assessed based on a galvanic series that is applicable to the exposure environment. For example, the difference in electrical potential between aluminum and stainless steel is typically negligible in general atmospheric conditions but is more pronounced in saltwater environments.

** Zinc coating is likely to corrode but is sacrificial.

Condition	Level of Concern
A carbon steel bolt is used in a stainless steel beam within a sealed plenum space of an office building.	Because there is no electrolyte (water or humidity) present in the space, a galvanic cell will not be expected to form, and galvanic action is typically not a concern.
A stainless steel walking surface is supported on a carbon steel pedestrian bridge with stainless steel bolts.	In an outdoor environment, a galvanic cell may form between these materials. The stainless steel deck should be isolated from the carbon steel structure. The stainless steel bolts will also tend to cause corrosion of the steel structure, but the amount of carbon steel will most typically be much greater than the amount of stainless steel, and the section loss in the carbon steel structure may be tolerable. If not, insolation kits should be used on the bolts to prevent section loss on the carbon steel members.
An aluminum cable tray is supported from a galvanized steel beam in an electrical switchyard.	If there is potential for moisture in this condition and chlorides are present, the aluminum may experience some section loss. If this is of concern, the materials should be isolated. In general atmosphere, galvanic corrosion between these materials will not typically occur.
A zinc-coated, carbon steel Unistrut member is welded to a stainless steel member to support a bin that will be exposed to seawater.	The Unistrut has a much more negative electrical potential and will likely corrode in this environment.
Aluminum fasteners are used in a steel bridge.	If chlorides from road salt are present, the aluminum fasteners will corrode significantly faster than the steel members, potentially resulting in premature collapse.
A carbon steel pipe is attached by a flange connection to a bronze pump body in a city water system with a high chloride content.	The carbon steel pipe will corrode and leak due to galvanic corrosion. A dielectric coupling should be used at the joint to prevent this from occurring.
A galvanized light-gauge carbon steel sloped cover plate is specified to prevent birds from perching on the bottom flange of an exterior beam. The plate is to be attached to a carbon steel beam with stainless steel fasteners.	As this situation occurs in an outdoor environment, the stainless steel fasteners will tend to cause section loss in the light-gauge galvanized steel plate. As the light-gauge material is thin, the amount of section loss may not be tolerable. Using a galvanized fastener will reduce the likelihood of galvanic corrosion.

Preventing Galvanic Corrosion

The simplest mitigation strategy is to remove any one of the four components from the galvanic cell. This may involve one or more of the following:

Elimination of the electrolyte (such as water) from connecting materials. Materials wholly enclosed inside of buildings with controlled environments are typically not susceptible to galvanic corrosion. Exceptions can occur on surfaces where condensation can form, or where the internal humidity is high. Coatings can also be used to keep an electrolyte from a surface, but the coating must be durable and free of defects to provide reliable protection.

Electrical isolation of the anodic and cathodic metals using an electrically insulative material. If there is no electrical connection between materials, the galvanic cell cannot form and galvanic corrosion cannot occur. Effective electrical insulators are materials that have high dielectric strength and low capacity for water absorption. Various isolation kits are available in the market, which often include materials manufactured from neoprene, mylar, nylon, PTFE (Teflon), or similar insulators. For bolted connections, an isolation kit may include plastic washers, bolt sleeves, and shims to isolate the dissimilar metals. When selecting alternative washers and shims that are to be used in a bolted joint, the washer and shim strength and stiffness must be evaluated for compatibility with the loading condition—e.g., if the bolt is loaded in tension, the washer must provide adequate strength and stiffness to transfer and distribute the load to the connected parts.

The designer should also note that *RCSC Specification* Section 3.1 does not allow compressible materials to be included within the grip of a high-strength bolt assembly and states that "any materials that are used under the head or nut shall be steel." Since an insulator is necessary under the head and nut to maintain electrical isolation, the engineer cannot rely on the installation procedures from the *RCSC Specification* for these bolts and must evaluate the effect of the compressible material on the joint—while recognizing that the joint may not develop pretension and will need to be kept from loosening in service.

Using materials of similar electrical potential. If possible, the difference in electrical potential between contacting metals should be limited, and/or the design should ensure that the anodic metal has a much greater surface area than the cathodic metal. The environment should also be considered, as any surface contamination with chlorides (such as in a marine environment or surfaces exposed to deicing salts) can result in very aggressive galvanic corrosion.

When Corrosion is a Good Thing

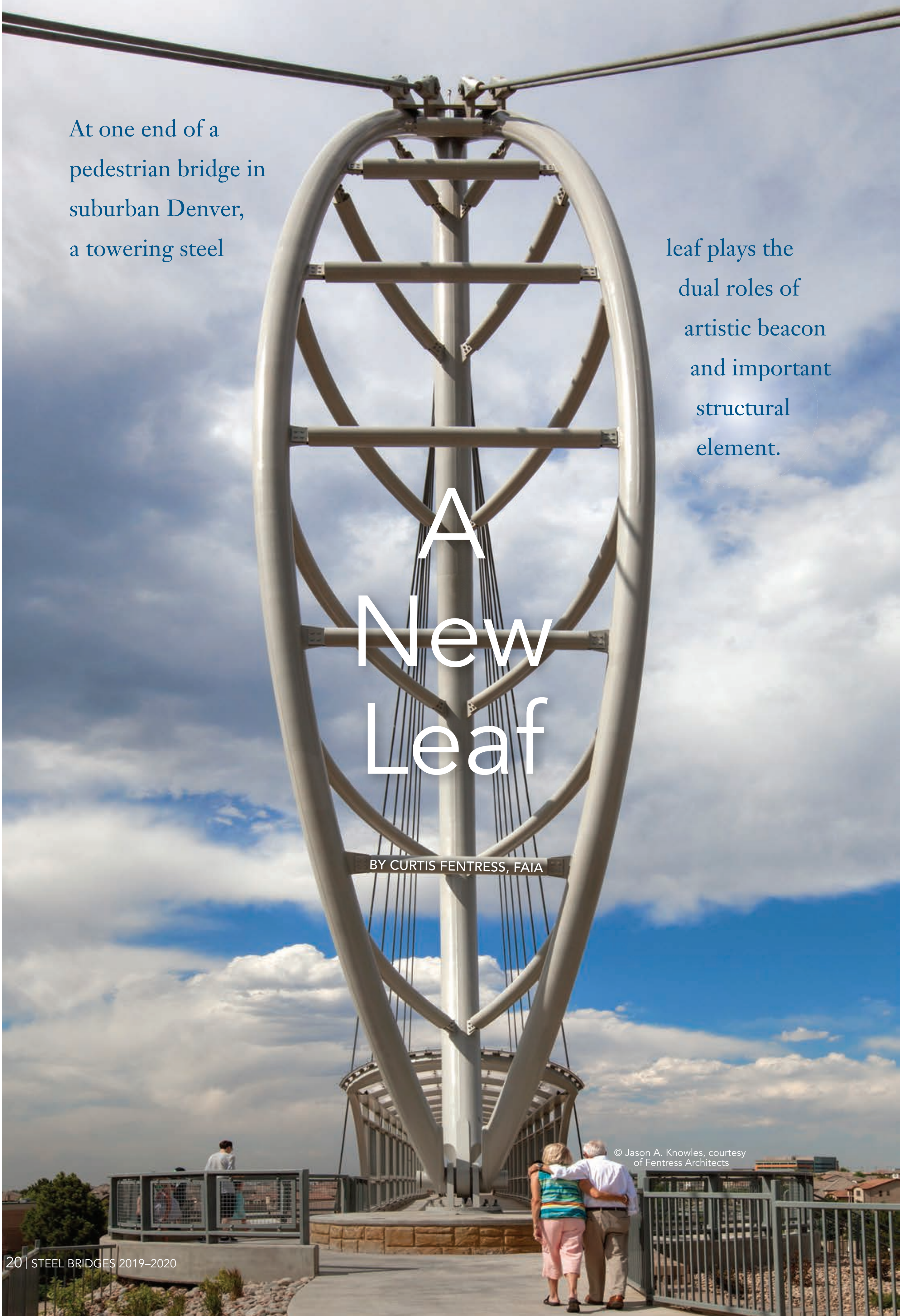
Is corrosion always a bad thing? No! Although not strictly a galvanic form of corrosion, two common corrosion protection methods use the concept of corrosion to your advantage:

Hot-dip galvanizing. In some cases, sacrificial anodes are coupled to structural materials to protect them from corrosion. The most common example of this is a zinc coating used to galvanize carbon steel, where the zinc is not relied upon to perform a structural function and the zinc material is slowly consumed to protect the underlying steel. Although not technically considered to be galvanic corrosion, the science behind this process is similar in that an anode (zinc) is designed to corrode over time and force the structural load carrying element (carbon steel) to be the cathode.

Cathodic protection. Cathodic protection systems come in two types: sacrificial anodes and impressed current systems. Both apply the concept of a galvanic cell with the protected material acting as the cathode (positive pole of the cell). These systems are typically most effective in buried or immersed conditions where the soil or water is available to act as an electrolyte. Sacrificial anodes without an external power source are often used for small structures such as buried tanks, piles, etc. These systems are low-maintenance and easy to install. For larger structures such as pipelines and wharves, an impressed current system may be more cost-effective—though the designer should consider that these systems have the potential to be turned off during a structure's life, resulting in a loss of protection. In above-grade conditions, cathodic protection systems are not typically effective for steel frames as there is not an electrolyte readily present.

The table at right provides a few typical conditions of dissimilar metals in contact that a structural engineer may be asked to consider.

We hope this discussion has given you a better understanding of galvanic corrosion as it relates to structural design. For more nuanced conditions such as enclosure design, conditions involving water flow, or materials subjected to unusual exposures, please consider additional reading or consulting a building scientist or metallurgist for assistance. ■



At one end of a pedestrian bridge in suburban Denver, a towering steel leaf plays the dual roles of artistic beacon and important structural element.

A New Leaf

BY CURTIS FENTRESS, FAIA

© Jason A. Knowles, courtesy of Fentress Architects left and below: The design team refined the bridge's pylon leaf design while retaining structural integrity. From the leaf, six pairs of cables extend to the north to support the bridge. Soaring 100 ft above Lincoln Avenue, the bridge's unique form is seen from many vantage points throughout the city of Lone Tree, Colo., and creates a memorable image for those traveling on and over the busy thoroughfare.

© Michael Tamburello, courtesy of Fentress Architects

LINCOLN AVENUE, an east-west corridor in Lone Tree, Colo., sees a traffic load of 90,000 cars per day.

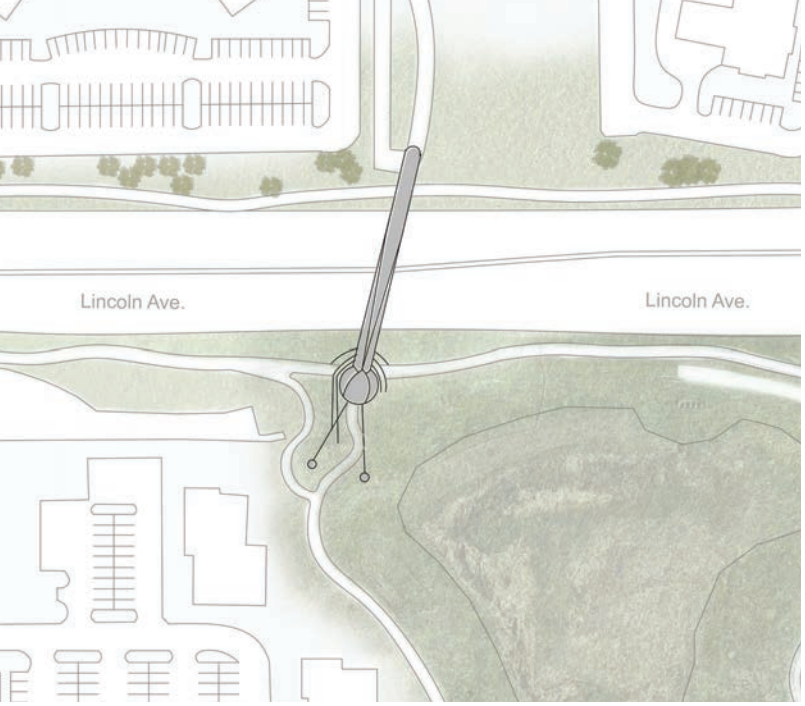
The busiest thoroughfare in this southern suburb of Denver, it divides Lone Tree Elementary School and residential communities on the north from local retail, workplaces, and parkland on the south. Given the area's growth and Lincoln Avenue's significant traffic load, it became increasingly treacherous for pedestrians and cyclists to cross at streel level.

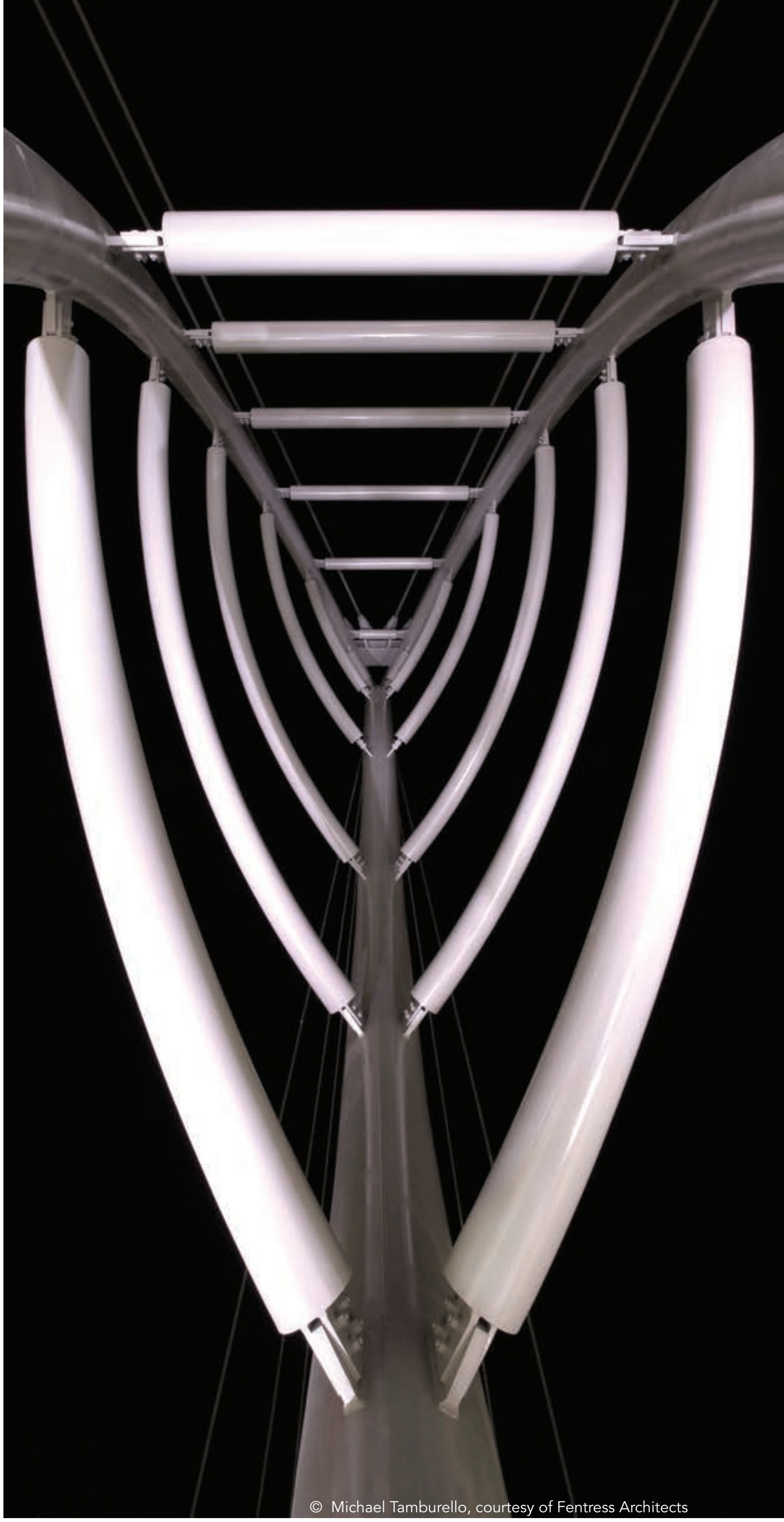
But a new pedestrian bridge at a strategic crossing point—near vibrant commercial strips and the school—now allows children from the elementary school to avoid Lincoln Avenue's traffic and safely reach open space parkland for scientific and ecology education. It also lets workers on the north side easily get to eateries on the south side and provides runners and cyclists with unhindered access to a network of recreational trails that the road interrupts.

Symbolic Design

Completed last year, the new 170-ft-long, steel-framed Lone Tree Pedestrian Bridge achieves these goals rather stylishly. In the preliminary stages of the project, the city challenged Fentress Architects to design not only a pedestrian bridge, but one with flair, an icon that would reference the city itself. Lone Tree's logo is a tree and the Lone Tree Art

Curtis Fentress (**fentress@fentressarchitects.com**) is president and principal in charge of design at Fentress Architects.

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above: The bridge provides safe pedestrian passage over a major road with a traffic load of 90,000 vehicles per day.

right: The pylon is a 3D lattice truss constructed of steel members, with a twist in the geometry to create a sculptural form.

below: Stainless steel mesh on the sides and an ETFE roof on top protect bridge users from severe weather while enabling them to enjoy the elements on pleasant days.


© Jason A. Knowles, courtesy of Fentress Architects

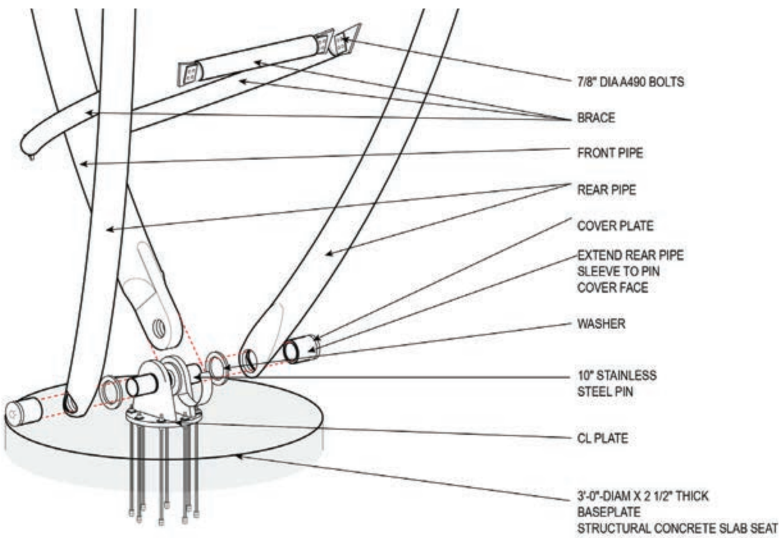
Center's (near the bridge on the south side of Lincoln) is a leaf, and the city wanted this foliage theme incorporated in the design.

Fentress and structural engineer Thornton Tomasetti delivered. The steel main pylon, in the shape of a leaf, rises 100 ft above the road, creating a highly visible landmark. From the leaf mast, six pairs of cables extend down to support the bridge deck. The deck is defined by an in-plane steel truss created by longitudinal edge beams, cross beams, and diagonal bracing—all using conventional rolled steel members, mostly W18s (fabricated by King Fabrication)—below the slab to provide lateral stiffness and stability to the span. The bridge deck consists of a 3-in.-thick reinforced concrete topping slab over 3.5-in.-thick precast concrete panels. The topping slab was poured onto the steel beam between the panels, providing a structural connection between the slab and the beams. In addition, a steel "knuckle" at the base of the leaf pylon incorporates a 10-in.-diameter by 4-ft-long stainless steel pin inside of a steel sleeve, resulting in a true pinned base with no rotational restraint at the base of the pylon.

While Fentress has worked on pedestrian bridges that connect offices, laboratories, and airport buildings, this was the firm's first major pedestrian bridge over a busy roadway. The design team studied a number of structural systems, initially presenting a box truss option and a suspension option along with the chosen cable-stay option. The cable-stay format was eventually chosen as it was a lighter than a deep-girder bridge while also being stiffer than a suspension bridge. It also allowed much of the structure to be located at the pylon rather than within the span, creating a thinner profile for the span

above: Looking down on Lone Tree. The bridge, at center in the image, provides a vital link in an extensive pedestrian and cyclist trail system.

left: Transporting the completed leaf pylon to the job site.



above and left: A steel knuckle at the base of the leaf pylon incorporates a 10-in.-diameter by 4-ft-long stainless steel pin inside a steel sleeve.

itself—minimizing the impact on views to the Colorado mountains and downtown skyline and also meeting the clearance requirement of 17 ft, 6 in. for pedestrian bridges that cross major arteries—and reducing the structure's weight as well as material costs.

Light as a Leaf

The bridge uses roughly 100 tons of structural steel in all, coated with a zinc-rich epoxy primer topped with fluoropolymer urethane. The leaf is formed from thick-walled pipe, 24-in.-diameter for the primary and 18-in.-diameter for the secondary members, with 10-in.-diameter "veins" in between. The thick walls allowed the sections to have a thinner profile while still achieving the required structural properties.

The leaf was delivered to Lone Tree in one piece, and the spans were delivered in two pieces, all from King's shop in Houston. Thanks to the bridge essentially arriving at the job site in only three major components, it was erected with only one weekend closure of Lincoln Avenue. Once the road closure began (8:00 p.m. on a Friday), the leaf, including the pin at the base, was driven to the road beneath its pedestal. Two cranes were used to lift it, one at the base (300-ton) and one at the tip (500-ton). Once lifted horizontally, the base crane slowly lowered the bottom, then released it. With the installers guiding it, the leaf was lowered onto its pedestal, and adjustments were made to rotate it so that it would align with the anchor bolts. Once the leaf pylon was in place, the backstay cables were connected while the other crane was still holding it erect.

above and below: As a critical connection for local pedestrian traffic, the new bridge spans across the rapidly growing area's busiest east-west road.

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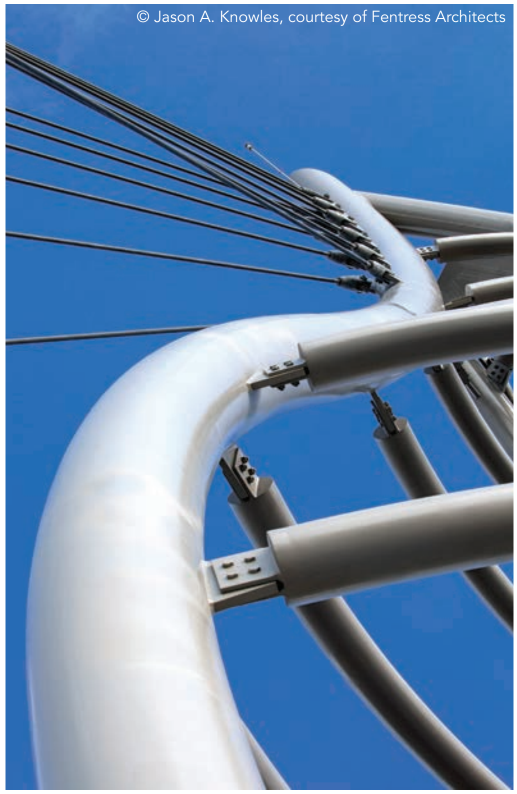
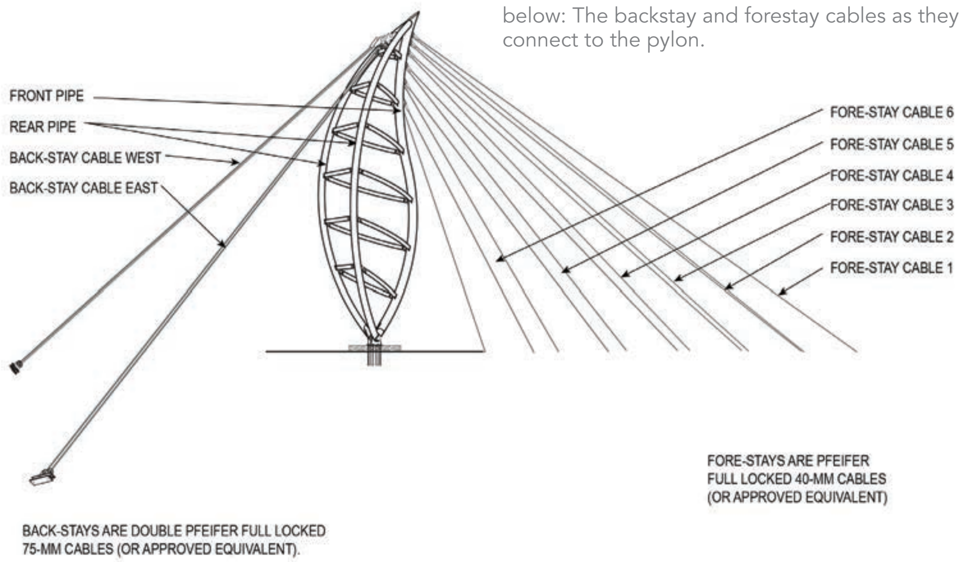


© Jason A. Knowles, courtesy of Fentress Architects

above: For the leaf pylon, the team focused on the sensitivities of the cable and pylon geometry to harmonize steel's efficiency with an artistic design.

below: The backstay and forestay cables as they connect to the pylon.



The bridge was erected in only one weekend closure of Lincoln Avenue.

© Jason A. Knowles, courtesy of Fentress Architects

While the leaf was going up, another crew was installing a shoring tower at mid-span. As work continued on the leaf pylon, the two spans were installed and joined at mid-span. Next came the 1½-in.-diameter forestay cables, which connected to the leaf and the bridge spans. The forestays are a fixed length, so the 3-in.-diameter back-stays were jacked in order to tension them. The cables are connected to the pylon at their tops and to the slab at their bottoms via large tension-cable sockets, providing field adjustability during installation and allowing for rotation of the cables during construction. The backstay cables are connected with similar sockets to long steel rod anchors that transfer the bridge cable forces into the earth.

The roof of the bridge consists of a thin ethylene tetrafluoroethylene (ETFE) membrane stretched between pretensioned steel cables anchored at each end. The ETFE system keeps the bridge roof light yet still stiff enough to support snow loads of the roof without significant sagging, while also allowing light through into the covered span. A simple portal frame supported on the main span deck, featuring mesh panels down either side, provides the infrastructure for the enclosure. Ramps on each end of the bridge facilitate accessible design, multimodal access and connect students, residents, and workers to the amenities on both sides.

The new Lone Tree Pedestrian Bridge celebrates the unification, both socially and physically, of Lone Tree's north- and south-side communities. The structure's memorable form creates a major landmark for the city and establishes a model of its vision for the future: a more easily accessible community with safe passage for pedestrians. ■

Owner
City of Lone Tree, Colo.

General Contractor
Hamon Infrastructure, Denver

Architect
Fentress Architects, Denver

Structural Engineer
Thornton Tomasetti, Denver

Steel Team

Fabricators
King Fabrication, LLC (AISC Certified Fabricator), Houston (also Detailer)
FabriTec Structures (AISC), Dallas (ETFE Framing)

Bender-Roller
Bendco (AISC Associate), Pasadena, Texas

Both waterways and vehicular traffic were kept free of interruption during a major steel bridge replacement over the Missouri River.

Open Channels

BY BRUCE A. BURT, PE | PHOTOS COURTESY OF MoDOT

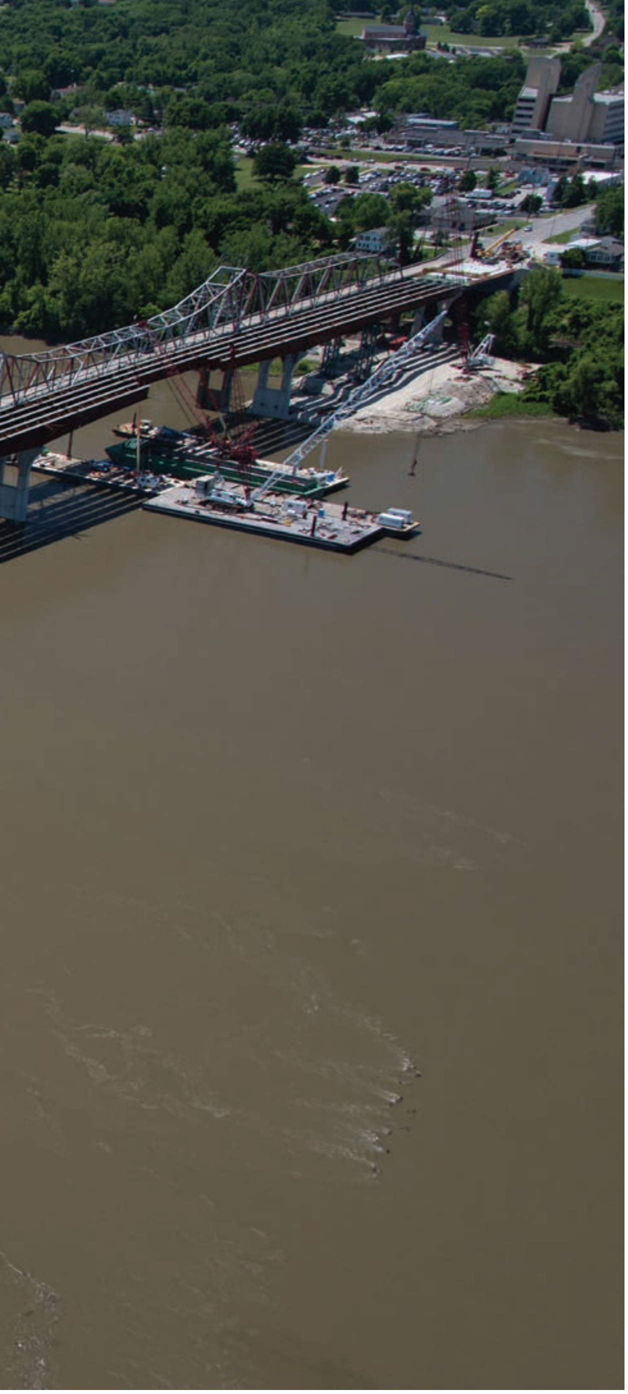
Bruce Burt
(**bburt@rubyandassociates.com**) is a principal with Ruby+Associates, Inc.

SINCE OPENING IN 1936, the two-lane Route 47 Bridge in Washington, Mo., has provided a prominent vehicular crossing over the Missouri River roughly 50 miles west of downtown St. Louis. Unfortunately, the historic two-lane steel truss bridge had become functionally obsolete in recent years and required replacement.

Just 15 ft upstream from where it once stood is its brand-new structural steel replacement (the distance between the new and old bridge centerlines is 60 ft), which provides a 52-ft vertical clearance for river traffic. The new 1,770-ft-long main span bridge, which opened to traffic in December 2018, features 12-ft lanes, 10-ft shoulders, and a protected 10-ft bicycle/pedestrian path. Structurally, the new bridge consists of five concrete 160-ft approach spans and four steel main spans. The main spans are each comprised of five steel plate girders whose depths range from 18 ft at the support piers to 10 ft, 6 in. at their midspans. The two outer spans are 385 ft long, and the two central spans are 500 ft long. In total, 5,800 tons of structural steel, including 75 girder sections, 346 cross-frames, 336 lateral braces, and 24 diaphragms, were used in its construction.

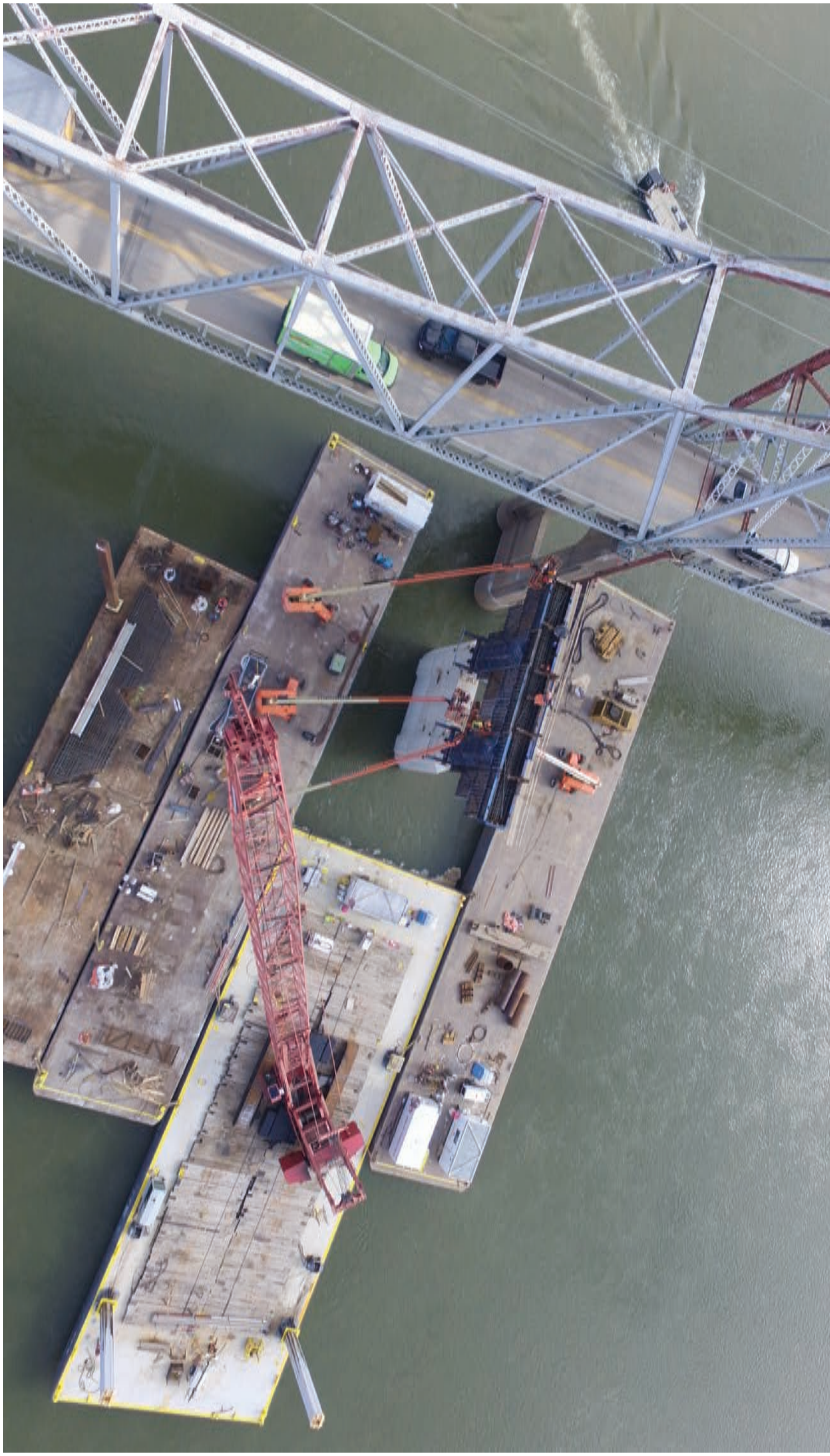
Minimizing Falsework

Close collaboration between general contractor Alberici Constructors and erection engineer Ruby+Associates resulted in an efficient erection plan that minimized falsework



left:The superstructure for the new bridge, which uses 5,800 tons of stuctural steel in all, nearing completion.

below: A bird's-eye view of the temporary shoring towers, which were founded on compacted gravel and timber crane mats. The original bridge remained open to traffic during construction of the new bridge.



and temporary shoring for the bridge. A flexible support bracket system was developed to reduce the length of girder cantilevers during erection. The bracket system was designed to move from pier to pier, resulting in a significant weight savings in steel falsework and river-based shoring.

Thanks to a comprehensive stability analysis and careful planning, the team was able to reduce the number of shoring locations to just three, with one shore placed on a temporary causeway and two within the waterway. A hybrid shoring system was developed for the two shoring bents located in the river. Salvaged 42-in.-diameter steel pipe was driven in the riverbed in four-pile groups, then cut to elevation above river high water datum. Temporary shoring towers were attached to the pile groups using conical transition pieces, allowing positional adjustability to account for piledriving tolerances. A

above: Girders were raised using a pair of crawler cranes.

left: The out-to-out distance between the new and old bridges is 15 ft.

below: Lifting one of the massive plate girders from the ground.

temporary support girder spanned between adjacent tower sections, and each hybrid tower—consisting of pipe piles, transition elements, and rented tower sections—was designed as a cantilevered column to eliminate the need for custom-fabricated, field-installed bracing between the towers.

To eliminate the need for piledriving at the third shoring location, a temporary causeway was extended and the temporary shoring towers were founded on compacted gravel and timber crane mats. To provide lateral stability of the shoring system without the need for expensive field bracing between tower sections, one shoring tower was designed as a cantilevered column and the adjacent tower was designed to lean on the cantilevered tower. Base fixity for the cantilevered tower was achieved by mounting it on outrigger beams and casting concrete blocks on the outriggers to provide overturning and sliding resistance.

To further reduce shoring requirements as well as the number of "air splices," the ironworker crews needed to assemble two girder sections—one 100 ft long and the other 114 ft long—for the outer two bridge spans end-to-end on the ground. These preassembled girders, which combined to form 214-ft-long girder sections weighing more than 100 tons apiece, were raised using a pair of crawler cranes working in tandem. A midspan support was required to provide lateral stability for the first preassembled girder section, and a lightweight shoring tower was repurposed from a previous Alberici project to provide the necessary support. Subsequent girder assemblies were erected using the same two-crane lift method, then laced back to the previously erected girder before being released from the crane. Due to the increased lateral strength of the now interlaced girders, intermediate shoring was not required beneath the remaining girders.

The temporary pier brackets mounted to the piers provided stability to the double-cantilever girder sections and were designed to deflect in order to accommodate deformations induced during erection as additional girder sections were added. This support bracket system was mounted to one concrete pier, then demounted and reused at subsequent piers, and spacers of varying depths were placed at the tips of girder brackets to account for different girder profiles at each bridge pier.

Erection Sequence Proves Critical

One of the project's main challenges required maintaining the 400-ft-wide shipping lane in the Missouri River, necessitating the erection of the 500-ft-long central bridge span above the shipping lane without the use of shoring or other waterway obstructions. A

The original bridge was demolished via synchronized demolition charges. The new bridge is fully visible in the third photo.

sequenced erection plan was developed to ensure the stability of the long cantilevers that resulted from the un-shored method, and a procedure was developed for installing the final "keystone" girder sections (called this because they are at the center of the middle 500-ft span).

In order to safely install the long cantilevers required to partially close the 500-ft span, the adjacent bridge spans had to be completed first. This entailed erecting the double-cantilever girder section on the concrete pier, using the temporary support bracket for stability, erecting girder sections at the opposite end of the span using the shoring towers for temporary support, and then completing the span with infill girder sections. Once the adjacent spans were installed, girders were erected from the double-cantilever girder sections to form 180-ft-long cantilevers projecting from the piers.

The shoring towers in the adjacent spans were equipped with hydraulic jacks that could raise or lower the adjacent bridge spans, which in turn affected the elevations of the ends of the cantilevers. This jacking system allowed precise elevation adjustment of the ends of the cantilevers to ensure the girder ends were properly aligned for the installation of the 140-ft-long keystone girder sections. The girders were installed slightly offset longitudinally from their final position to leave a gap for installing

Overcoming Fabrication Complexity

General contractor Alberici Constructors partnered with fabricator Industrial Steel Construction (ISC) on the project for three key strategic and economic reasons: 1) ISC is AISC certified, 2) it can perform large girder line assemblies under roof (thanks to its 900-ft-long shop bay and 100-ton-capacity crane) to ensure proper fit-up, and 3) its location in Gary, Ind., gives it access to Lake Michigan, which facilitates barge shipping.

With 112-ft-long haunch girders over the piers varying in depth from 10 ft, 6 in. to 18 ft, ISC not only employed vertical butt splices but also horizontal butt splices in the web for 45 out of the 75 girders on this complex project. Butt splicing is typically used to join two steel plates together, but for this project four separate 1-in.-thick steel plates had to be joined together to make a web plate for one girder. The longest girder line assembly involved five girder segments with a total length of 612 ft, which ISC accomplished under roof. This moved the schedule forward by at least three months, since these assemblies could be achieved inside during the winter months.

In addition to the plate girders, the project also involved several secondary steel members, such as 346 cross frames, 336 lateral bracings, and 24 diaphragms that had to be fabricated in tandem with the plate girders in order to deliver the steel to the site on time. Luckily, ISC was able to dedicate another entire shop bay to process these members.

ISC shipped 50 girders by barge, from Lake Michigan to the Illinois river to the Mississippi river to the Missouri river, and finally to the job site, which helped reduce land transportation costs. A total of 10 barges was sent to the job site with approximately five girders loaded on each barge, and the girders were erected directly from the barges. The other 25 girders and the secondary steel members were transported by truck and erected from rock causeways on either side of the river. The transportation costs incurred by the materials transported via land were offset by the lower cost of erecting from land-based cranes in lieu of barge-mounted cranes, which typically takes twice as long.

—Ankit Shah, Senior Project Manager, Industrial Steel Construction

Rigorous Analysis

Fine-tuning the erection sequence and minimizing falsework required significant preplanning by Alberici and sophisticated analysis from Ruby's engineering team, which performed a staged erection analysis using LARSA 4D bridge design software. This analysis allowed deflections of the steel to be accurately determined at each stage of construction, essential for ensuring proper fit-up of the steel during erection, developing the means for dimensional control, and allowing for critical girder stability checks. Due to their excessive weight—over 100 tons each—the steel plate girders were installed one at a time instead of in a more stable paired configuration. And the long cantilevers that resulted from un-shored erection of the 500-ft span also necessitated a rigorous stability review. Once confirmed in the staged erection model, girder stability was verified via empirical methods. In addition, the team used RISA 3D to design the flexible pier brackets, the hybrid shoring system, and the midspan girder support shores, and also used UT Bridge—a 3D finite analysis program developed by the University of Texas—to perform a validation review of girder stability.

The secondary structural steel contained 346 cross frames, 336 lateral bracings, and 24 diaphragms.

The contrasting framing schemes of the old and new bridges.

Children span the bridge's width on its opening day.

above and left: The bridge's superstructure is made up of 75 individual plate girders whose lengths vary from 100 ft to 140 ft and whose depths range from 18 ft at the support piers to 10 ft, 6 in. at the midspans.

each keystone piece. The girders were placed on temporary low-friction polytetrafluoroethylene (PTFE) slide pads to facilitate the required longitudinal movement. With the necessary preparations complete, each keystone girder was lifted in place early in the morning. As the temperature of the steel increased throughout the morning, thermal expansion closed the gap left between girders, allowing ironworkers to bolt the girders together.

The entire project was performed without interrupting river or vehicular traffic, as the original bridge remained open during construction, and the new crossing accommodates a daily traffic volume of 13,000 vehicles. In April, the original bridge was demolished via synchronized demolition charges and its steel salvaged for reuse, perhaps as a future iconic steel structure. ■

Owner
Missouri Department of Transportation

General Contractor
Alberici Constructors, Inc., St. Louis

Bridge Designer
HDR Engineering, Inc., St. Louis

Erection Engineer
Ruby+Associates, Inc., Bingham Farms, Mich.

Steel Fabricator
Industrial Steel Construction, Inc. AISC CERTIFIED FABRICATOR
Gary, Ind.

A Bridge Replacement in Four Parts

BY JAKE WILLIAMS, PE, AND CHRIS KELLEY, PE

Steel construction makes quick work of a quartet of neighboring bridge structures requiring replacement or repair in Memphis.

Jake Williams (jwilliams@benesch.com) is a senior project manager and **Chris Kelley** (ckelley@benesch.com) is a project engineer, both with Alfred Benesch and Company.

ASK ANY MEMPHIS COMMUTER to describe I-240, and "traffic nightmare" would be a typical response.

Over the past six years, this corridor, which curls around the eastern side of the city and connects motorists to the Memphis International Airport, has been under constant construction. When the Tennessee Department of Transportation (TDOT) was faced with the urgent need to replace or repair four deficient structures all spanning I-240 within a quarter-mile of each other, shutting the Interstate down for weeks at a time to make it happen simply wasn't an option. With traffic levels of approximately 180,000 vehicles per day, TDOT wanted this critical project, dubbed MemFix 4, completed quickly and with minimal impact to travelers—and structural steel was at the center of the solution.

The high seismic demands of the region, which is near the New Madrid Fault Line, had increased the urgency to replace or repair the four 58-year-old structures, including two bridges at the busy Poplar Avenue interchange; a Norfolk Southern Railroad (NSR) bridge that serves as a critical east-west connector over the Mississippi River; and the concrete Park Avenue bridge. Due to the complexity of this $54 million endeavor, TDOT opted to use the construction manager/general contractor (CM/GC) project delivery method, which is designed to maximize efficiency and enable close collaboration between the owner, design team, and contractor during design and construction.

MemFix 4 is only the second project in Tennessee to be delivered using the CM/GC method. More traditional design delivery methods, such as design-bid-build and the use of concrete, would have required three years to construct. CM/GC, coupled with the use of structural steel and accelerated bridge construction (ABC) techniques, including new substructures constructed under traffic and modular bridge superstructures, made it possible to complete the project in just 19 months. These methods also reduced the impact to vehicular and rail traffic and resulted in minimal change orders at a significantly lower level than the industry average—especially on a project of this size.

Again, implementing a steel solution was integral to the success and constructability of the project. Limited space and weight constraints made concrete an unsuitable option. And with an increased chance of impact from earthquakes and other seismic activity, steel was the ideal choice because of its lightweight, slim nature and structural integrity.

Tight Space

For the NSR bridge replacement portion, the key challenge was overcoming tight spatial constraints. The railroad's *Public Projects Manual* listed five acceptable concrete superstructure options for the bridge—none of which could accommodate the site constraints, which included limits on how high the bridge could be raised due to the adjacent profile as well as clearance requirements for the highway bridge beneath (i.e., it could not be lowered). Conventional concrete beams would not have been shallow enough to fit, making steel the optimum choice. Structural steel framing—in the form of welded steel plate girders with bolted diaphragms, walkway brackets, and steel floor plates, with a total steel weight of 950 tons and a maximum span of 88 ft—allowed the designers to meet the railroad's span length and height criteria while reducing bridge mass for seismic design, further minimizing demands on the supporting bridge components and thus improving cost-effectiveness. These inherent design efficiencies of using steel reduced construction costs when compared to concrete girders.

The NSR corridor, which has both a mainline and a siding track, required continuous operation of both tracks, allowing only two 12-hour interruptions to a single track at a time. To replace this bridge, a temporary shoofly structure was constructed adjacent to the existing bridge, comprised

above: The MemFix 4 project involved repairing or updating four deficient bridges over Interstate 240 on the east side of Memphis.

below: A temporary shoofly structure was constructed adjacent to the existing NSR bridge, then the permanent steel superstructure supporting a ballasted track was erected on the shoofly alignment.

All images credited to Alfred Benesch & Company except as noted.

above: Sliding in the second NSR bridge. This replacement railroad crossing comprised a total steel weight of 950 tons and a maximum span of 88 ft.

left: The "bridge farm" where the Poplar Avenue bridges were built along the side of I-240.

above: Following construction at the bridge farm, the Poplar Avenue structures were rolled two miles down I-240 to the project site on self-propelled modular transporters (SPMTs).

below: A close-up look at the ends of one of the Poplar Avenue modules. A total of 900 tons of weathering steel was used to create low-profile superstructures stretching to a maximum span of 150 ft.

of temporary concrete piers supported by micropile foundations to minimize ground disturbance, then the permanent steel superstructure supporting a ballasted track was erected on the shoofly alignment. With trains traveling on the shoofly structure, the old bridge was demolished and the new substructures built. The two new, 1,100-ton superstructure sections were then slid 35 ft into place, one track at a time, during two weekend Interstate closures.

Two of a Kind

When it came to the two concrete Poplar Avenue structures (one with four spans and other with five), their condition was poor enough that repair wasn't an option—and again, concrete wasn't viable as a replacement option. Instead, a two-span steel girder design for both bridges satisfied current seismic codes and significantly improved the long-term reliability and flexibility of the corridor. A total of 900 tons of weathering steel was used to create low-profile superstructures stretching to a maximum span of 150 ft, meeting severe grade modification restrictions and accommodating a widened corridor. On average, opting for steel instead of concrete reduced the depth of each structure by about 30% and the weight by more than half.

The superstructure for each bridge was split into four modular units and constructed off-site two miles down the road on temporary substructures at ground level in an open section of roadside right-of-way known as a bridge farm. The units were then rolled down I-240 itself on self-propelled modular transporters (SPMTs). Using steel not only made the bridges structurally feasible, but steel also required fewer, lighter pieces to assemble and move. The roll-ins necessitated only two 56-hour closures of I-240, significantly less time than would have been required had the bridges been constructed in place using concrete. In addition, custom steel bearings eliminated the need to adjust the existing pier cap elevations while also transmitting significant seismic loads, and additional lateral framing elements mitigated lateral load path discontinuities imposed by the superstructure construction techniques. The existing bridges were successfully demolished and the modular superstructure units were slid into place, the heaviest unit being six girders wide, 150 ft long, and 550 tons.

Critical Casings

The fourth structure on the MemFix 4 project was the Park Avenue Bridge, adjacent to the NSR bridge. While the bridge has a concrete superstructure, it was preserved using a novel steel foundation retrofit design to optimize its seismic behavior. Essentially, all 16 existing concrete columns were retrofitted with 3-ft, 9-in.-diameter, ⅜-in.-thick steel casings to improve their ductility and bring them into conformance with current design standards without adding weight or rigidity to the structure.

By addressing the area's seismic design criteria, further complicated by tight spatial constraints, steel was in the driver's seat for successfully completing the quartet of critical infrastructure components that made up MemFix4. Fully completed in July, the project successfully transformed I-240's aging infrastructure with minimal impact to the travelling public and improved the highway's—and Memphis'—mobility for years to come. ■

Owner
Tennessee Department of Transportation

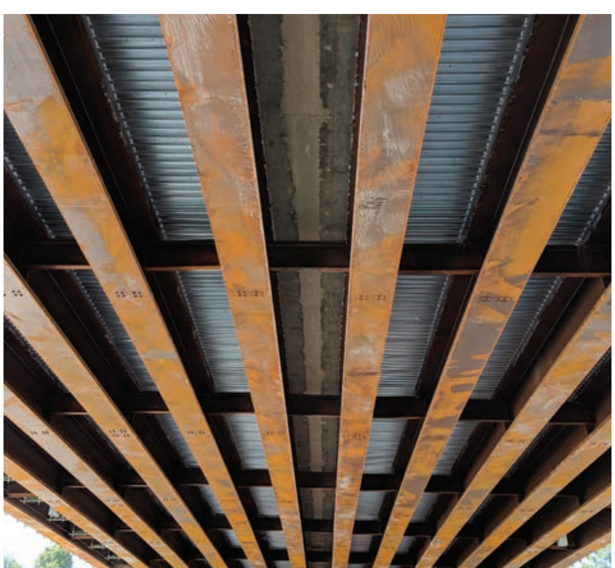
Structural Engineer
Alfred Benesch and Company

Construction Manager and General Contractor
Kiewit Infrastructure South Co.

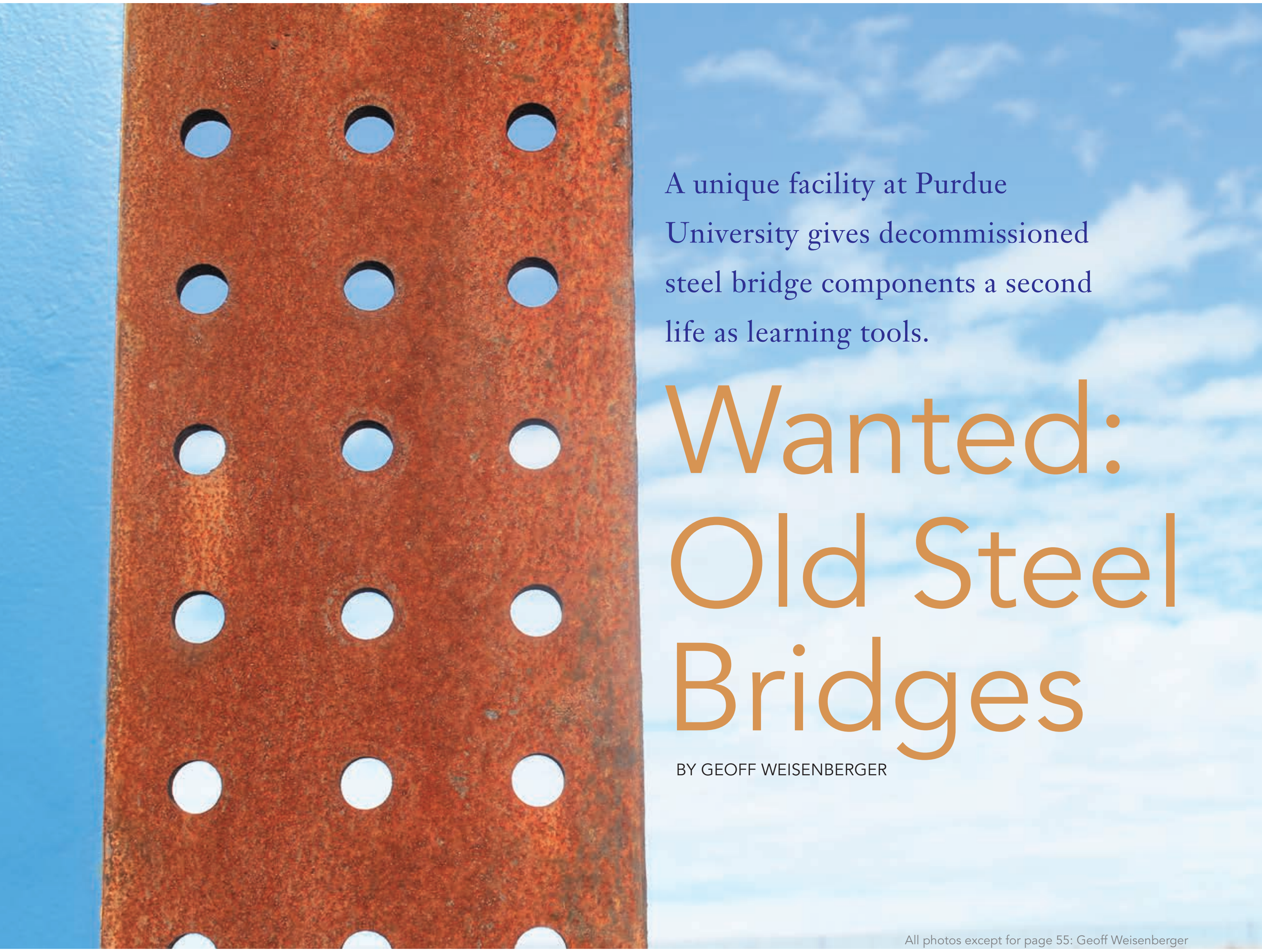
Steel Fabricator and Detailer
W&W/AFCO Steel AISC CERTIFIED FABRICATOR, Little Rock, Ark.

below: On average, opting for steel instead of concrete reduced the depth of the Poplar Avenue structures by about 30% and the weight by more than half.

right: After demolishing the existing bridges, the modular superstructure units—with the heaviest being six girders wide, 150 ft long, and 550 tons—were slid into place.



above and below: The Poplar Avenue roll-ins required only two 56-hour closures of I-240, significantly less time than would have been required had the bridges been built in place using concrete.



A unique facility at Purdue University gives decommissioned steel bridge components a second life as learning tools.

Wanted: Old Steel Bridges

BY GEOFF WEISENBERGER

All photos except for page 55: Geoff Weisenberger

Geoff Weisenberger (**weisenberger@aisc.org**) is senior editor of *Modern Steel Construction*.

JUST A COUPLE OF MILES to the southwest of Purdue University's main campus in West Lafayette, Ind., is an open-air site sporting an impressive array of complete steel bridges and bridge components.

Part training facility, part teaching and research lab, and part antique steel bridge museum, this is the school's Center for Aging Infrastructure (CAI), a 22-acre site focused on studying and improving the country's infrastructure, especially as much of it is reaching an advanced age and in need of rehabilitation and repair. By far the facility's largest user is another Purdue initiative, the Steel Bridge Research Inspection Training and Engineering Center, or S-BRITE.

The name says it all. While S-BRITE serves as a hands-on, real-world lab of sorts for Purdue graduate and undergraduate engineering students alike, it also exists to provide training for bridge inspectors. Open to the elements, students and others can study the bridges and bridge sections in their "natural habitat" (though no longer having to endure vehicular or train traffic) during daylight hours and at night, in any weather, and in all seasons.

So how did these various bridge assemblies get here? They didn't just fall out of the sky.

Robert Connor, Purdue's Jack and Kay Hockema Professor of civil engineering and director of CAI and S-BRITE, notes that all have been donated and transported by various departments of transportation (DOTs).

"It's all word of mouth," he explains. "When we hear about a bridge being taken out of service and think it would be a suitable addition, we contact the DOT and work to

above: The first statistically significant probability of detection (POD) study for visual inspection of fatigue cracks was conducted at S-BRITE. Weathered painted specimens are shown on the left, and uncoated weathered specimens are on the right.

below: A portion of a former fascia girder over an Interstate now illustrates how a bolted girder splice can be used to repair a fracture.

This is a portion of an all-welded 1956 railway bridge from the Pensylvannia Railroad in New Jersey. The girder sustained a brittle fracture at a detail now know to be susceptible to constraint-induced fracture (CIF). The girder sports a complex bolted repair. Though unrelated to the fracture, holes were drilled to arrest the fatigue cracks at the top of the vertical stiffener.

get it here." (Connor is also AISC's 2018 T.R. Higgins Lectureship Award winner. To see his Higgins Lecture "Towards an Integrated Fracture-Control Plan for Steel Bridges" from the 2018 NASCC: The Steel Conference in Baltimore, visit **aisc.org/2018nascconline**.)

Portions of steel bridges—and three complete ones (the longest is 91 ft!)—are positioned throughout the site on gravel pads and concrete slabs, with roughly 75 pieces in all. A complete 65-ft span railroad bridge here, a section of a highway bridge there, the effect is almost that of a steel bridge sculpture garden. And there's plenty of room for more, with new specimens being donated periodically. S-BRITE's most recent acquisition is a complete 1930s riveted railroad bridge donated by Norfolk Southern that used to carry rail traffic for the Wabash Railroad.

"It has cracks, weld repairs, and quite a bit of corrosion," says Connor. "A great specimen for us."

Other recent finds include multiple plate-girder assemblies. One is a bay of two girders that are still connected via X-bracing, a second is a portion of a multi-span bridge, and yet another comprises two units complete with pins and hangers. There are plans to place a concrete deck with some defects on the latter to train inspectors in sounding a deck. In addition, the center has also acquired failed joints from the collapsed I-35W bridge in Minneapolis—and in fact, S-BRITE is the holder of the only remaining major components from the bridge.

And in addition to the new specimens, the site has recently gained a new building intended to house specimens that should be kept out of the weather, such as fractured girders and a U10 joint from the I-35W bridge. Researchers at Purdue built a steel frame from which the U10 joint is suspended, in several pieces, to illustrate the relative positions of the components prior to the collapse. This greatly aids when explaining to students and visitors the mechanics of the failure and gives insight into forensic investigations.

Asked if there's anything of particular interest that Connor is keeping an eye out for, he says, "We could really use a few very large gusset-plated joints. While we have a complete truss and a few joints, some very large, even shingled, truss joints would be wonderful to have. We are also looking to obtain about 600 ft of railroad track. We have erected signals and hope to install the track to show students how block signals work and track bed is constructed, and many other aspects related to railway engineering."

A 90-ft-long pony truss obtained from China Township, Mich. The structure contains considerable corrosion damage and includes many welded repairs.

below: A portion of a BNSF bridge originally built over the Mississippi River at Burlington, Iowa, in 1893. This floor beam joint contains many repair welds and strengthening members.

left: A group of engineers is dwarfed by 23-ft-deep girder sections from the Dresbach Bridge—provided by the Minnesota DOT and Ames Construction—which carried I-90 across the Mississippi River between La Crosse, Wis., and Dresbach, Minn.

below: The Indiana Railroad provided several components, including this pin-connected lower chord section, when the White River railroad bridge near Elnora, Ind., built in 1899, was replaced in 2015.

There isn't a "typical" day for site. Connor notes that he uses it for his graduate steel design and fatigue and fracture classes and also in undergraduate steel design curriculum. With this usage and that of other professors and other training, the site might host visitors a few times a month to every day for three or four weeks at a time.

When it comes to professional training, the most typical is geared toward inspecting bridges for fatigue; in fact, the center just wrapped up a course in early August. It's been valuable for inspection training not only in terms of helping inspectors develop a keen eye for defects—"We have a treasure map for the cracks," laughs Connor—but also in teaching more general lessons.

"Inspection is not an exact science as people think," he notes. "If you talk to three different inspectors, you will likely get three different answers." In addition, he stresses that inspectors also learn that inspection shouldn't just be performed on a routine schedule.

S-BRITE has also worked with the Army Corps of Engineers—a partner with the center—for the last two years. As a matter of fact, the Corps also plans to do all of its fracture-critical training at S-BRITE in the future.

"One of the main Corps personnel, Phil Sauser, had a great comment," recalls Connor. "He said, 'You could spend 20 years inspecting steel bridges and not see all the details and defects that you could see at S-BRITE in two hours.'"

Besides actual steel specimens, S-BRITE also provides living, breathing expertise in the form of the Distributed Expertise Network (DEN), a "Jedi Council" of 11 bridge experts—some at Purdue but most elsewhere—who have extensive knowledge in bridge-related topics ranging from coatings and corrosion to non-destructive testing to field instrumentation and monitoring and much more. The idea is to provide "on call" expertise as needs or questions arise and establish S-BRITE as a go-to resource for complex issues related to steel bridges. (For a full list of the DEN personnel and their areas of expertise, visit **engineering.purdue.edu/cai/sbrite**.)

How did S-BRITE become a reality? It started in the early 2010s with a collaborative effort between the Indiana Department of Transportation's Joint Transportation Research Program and Purdue, with both entities recognizing that the concept of an outdoor research and education facility could have tremendous benefits to and a positive impact on bridge design, construction, and inspection. The idea caught on and today, S-BRITE is supported by several states through Transportation Pooled Fund Project TPF-5 (281). Civil engineering graduate students helped design and build the first portions of the facility, and later a consultant was hired to perform the full site design, with a general contractor completing the final construction.

In the mid- to long term, Connor hopes to eventually incorporate classroom and laboratory space for on-site training. Further down the road, he anticipates that other users will expand the breadth of the research and training to include buildings and façade systems. But for now, he says, bridges are a full-time job and the next step for the center is to form a corporate sponsor/advisory panel, noting that the industry has expressed interest and has been very supportive in recognizing the value of the program.

"You don't want to just find a bridge in service, take a bunch of photos, then throw it away," he says. "Instead, why not seize the opportunity for training and research using the real thing?"

above: One of the more unique specimens at S-BRITE is this large nested roller bearing from one of the main river piers of the I-35W bridge.

left: A close-up of the bolted splice used when S-BRITE engineers re-erected the Indian Trail Bridge.

below: S-BRITE also contains many miscellaneous bridge components, such as these from the I-35W collapse.

Purdue University

Failed joints from the collapsed I-35W bridge in Minneapolis on display in a new building intended to house specimens that need to be kept out of the weather.

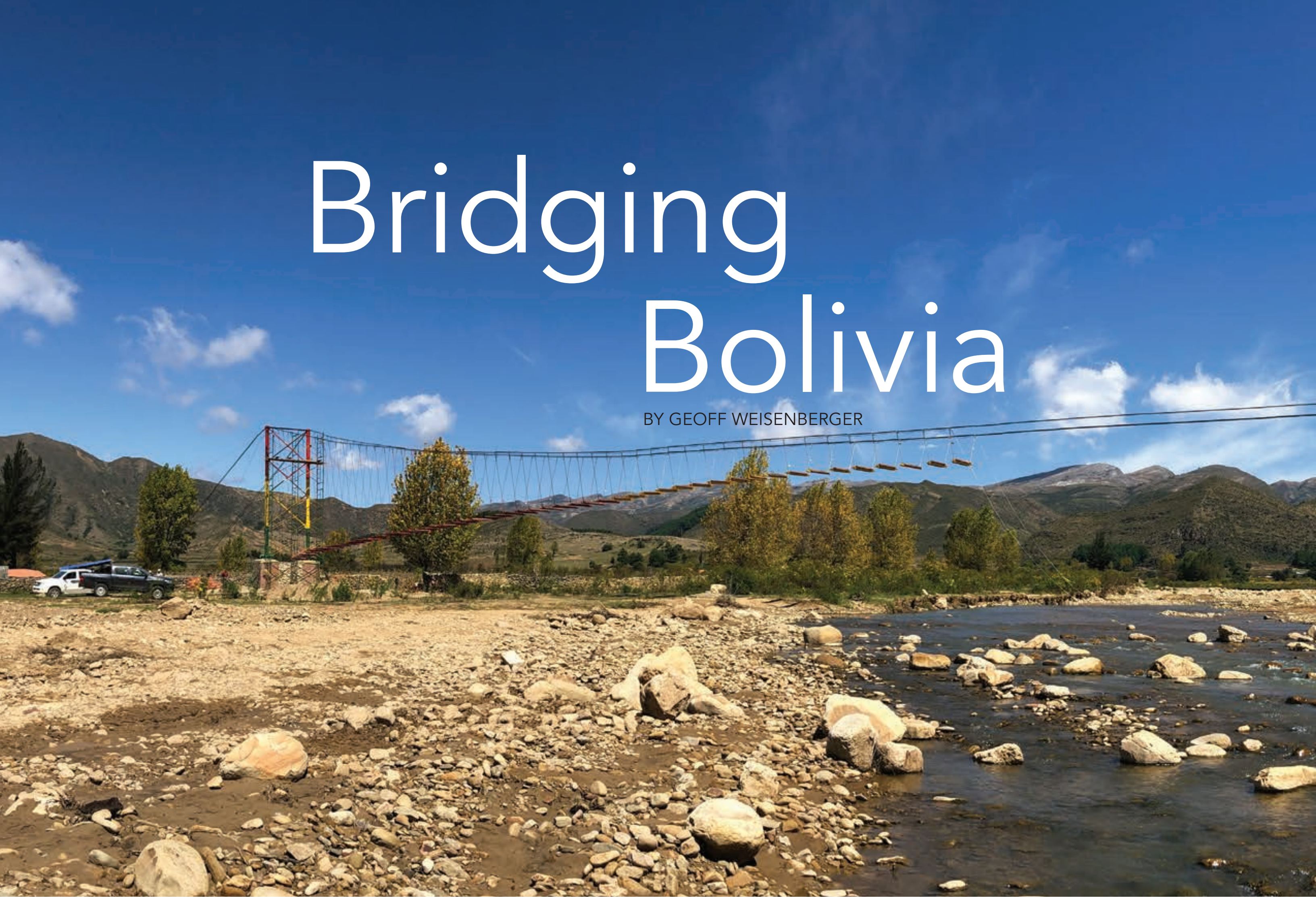
The B-Team

S-BRITE also makes house calls (bridge calls, actually) thanks to its field vehicle.

Remember the A-Team van? Think of that but newer and sporting Purdue black and gold instead of black with a red stripe. And instead of a weapons cache and other strike team-type equipment, the 2008 Dodge Sprinter 3500 acts as a mobile lab outfitted with data acquisition equipment, tools, and materials to allow the A-Team—er, a rotating team of research engineers assisted by Purdue civil engineering students—to perform research, testing, and investigation of steel bridges across the country. The lab on wheels, which has performed work in more than a dozen states, provides space for planning, discussion, and preliminary analysis at the site or on the way.

"The vehicle has been to several states for many, many projects," says Connor. "The furthest project was in California and the closest bridge was here in Lafayette, on I-65. The most recent long-haul trip was to Wyoming for a project to monitor the vibration of four high-mast lighting towers throughout the state over a period of about two years."

The experience is mutually beneficial as it exposes the students to real-world field testing and monitoring. And in some cases, the field work is directly related to a particular student's research project. While many of the projects are planned well in advance, the vehicle is also equipped for rapid and emergency response and can potentially mobilize within hours of being notified. In one such case, Milton-Madison Bridge over the Ohio River experienced a bearing failure during construction. The vehicle and its team were on-site within 12 hours at the request of the engineer. In another scenario, an I-465 bridge was exposed to a propane tanker explosion. The team mobilized the night of the accident to inspect the bridge to ensure public safety.



Bridging Bolivia

BY GEOFF WEISENBERGER

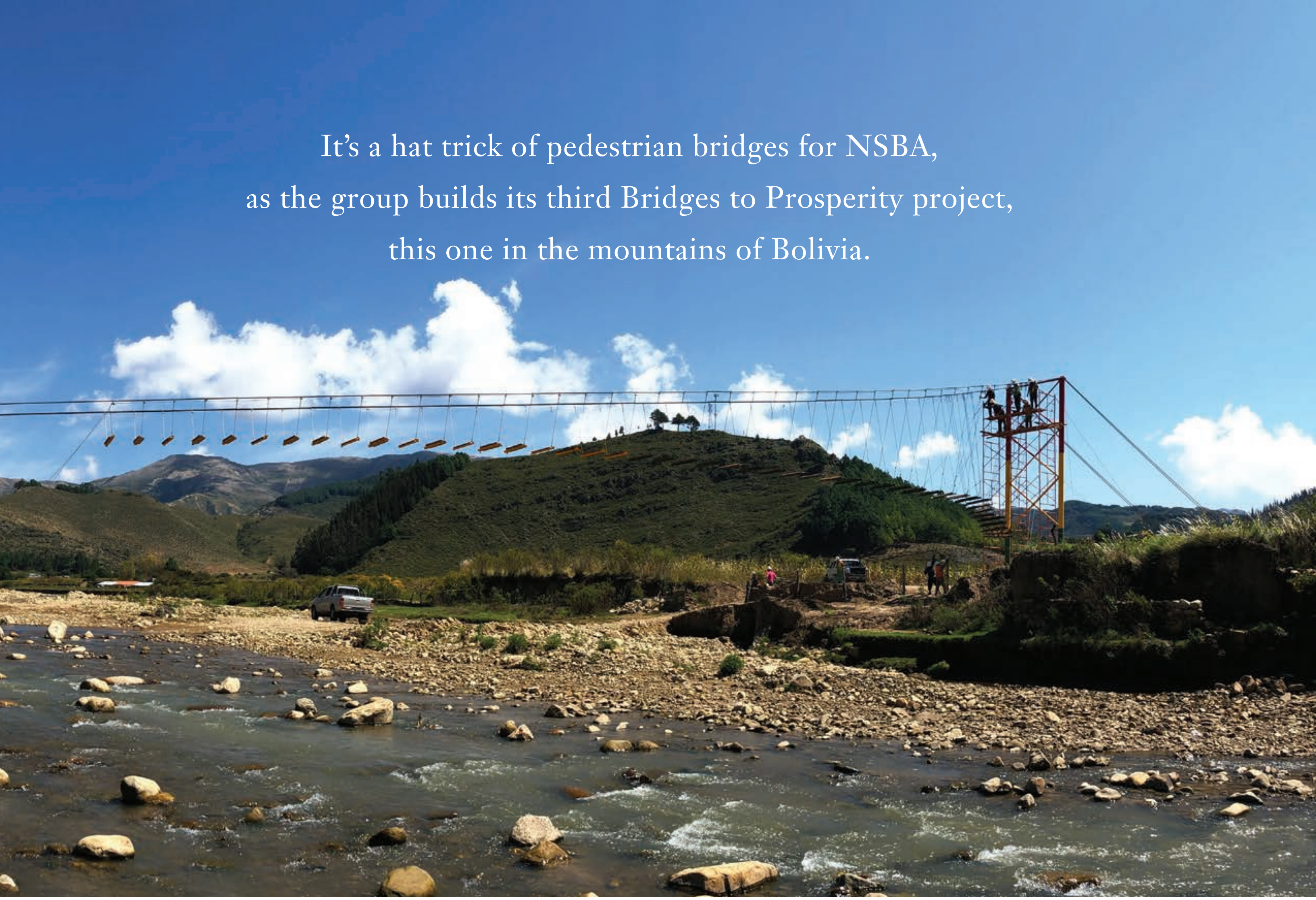
FOR YEARS, the National Steel Bridge Alliance (NSBA), AISC's bridge division, has promoted the proliferation of structural steel bridges across the U.S.

And more recently, it has been involved in bridge building on a much smaller—but nevertheless equally important—scale. Since 2016, it has partnered with Bridges to Prosperity (B2P)—a nonprofit organization whose mission is to build footbridges for isolated, typically poor communities around the world—on three separate occasions to provide Central and South American villages with safe passage over treacherous waterways.

While the first two bridge projects (in 2016 and 2018) were in Panama, NSBA's most recent project with B2P, which was built this past April, took the team to the highlands of Bolivia. There, near the town of Azurduy at an elevation of around 8,000 ft, ten volunteers and three B2P staff built the 361-ft-long, 3-ft, 7-in.-wide La Marca Suspension Bridge. The new span provides pedestrian access across the La Marca River to healthcare and schools, and also gives residents a route to bring their goods to markets on the other side. The NSBA team was comprised of David Alameda (Fought and Company, Inc.), Brad Dillman (High Steel Structures, LLC), Curt Duncan (Tennessee DOT), John Hastings (NSBA), Larry O'Connell (Stupp Bridge Company), Anthony Schoenecker (Modjeski and Masters), Craig Smart (HDR), Craig Stevens (Delware DOT), Scott Walls (Delaware DOT), and Jackie Wong (a volunteer from California).

The team began their two-week Bolivian adventure in the city of Sucre, which sits at roughly 9,200 ft above sea level, then headed to Azurduy, which is the closest town to the bridge site, roughly a 15-minute drive away. As the crow flies, the two towns are less than 100 miles apart but thanks to the winding mountain roads, the drive takes about seven

It's a hat trick of pedestrian bridges for NSBA, as the group builds its third Bridges to Prosperity project, this one in the mountains of Bolivia.



hours. There, the team helped build the bridge along with members of the community, working 11 straight days. During their time off, they did some sightseeing and enjoyed the local culture—e.g., cheering at local football (soccer) games in the evenings.

NSBA's John Hastings led the team, and the Bolivia project was his second with B2P (though his first as an NSBA employee, as he represented the Tennessee Department of Transportation on last year's trip).

"I enjoy volunteering and building things, so this was an opportunity to do both of those and use my engineering skills to help a community have access to the basics," he said. "It was also an opportunity to build relationships with a diverse group of individuals in the bridge industry."

"I was honored to help provide a community with a piece of infrastructure that we take for granted here in the U.S.," noted Stevens. "The gratitude the community showed for the bridge we built was awesome and humbling."

"While working with community members, as they learned some of the skills we had to share, they were also happy to teach us their language and to share their culture and way of life with us," said Schoenecker. "This experience made the difference in that we didn't just build a bridge to cross a river, but that we built a bridge to connect a community."

"In our profession, we work on building bridges every day in some way, and this was that in its most basic form," recalled Wong. "It was a good reminder of why we do what we do: help move people and goods."

The La Marca Suspension Bridge is NSBA and B2P's longest bridge yet together, and the third-longest for B2P overall. The first trip, to Lura, Panama, in 2016, culminated in a 167-ft bridge. The second project took place last year in El Macho, Panama, and was nearly 100 ft long. Together, the three bridges serve more than 1,300 people. NSBA will sponsor another B2P project next year, also in Bolivia.

Following is a "slide show" from the trip. You can also view a video of the trip and project at **youtube.com/user/AISCSteelTV**. ■

Workers installing suspenders and cross beams from both sides of the 361-ft-long La Marca Suspension Bridge near Azurduy, Bolivia.

Geoff Weisenberger (**weisenberger@aisc.org**) is senior editor of *Modern Steel Construction*.

above: A local resident crossing the La Marca River during the dry season. During the rainy season, the river will flow at capacity for several months and will be impassable for days at a time.

right: Craig Stevens and John Hastings installing cross frames on the towers. The nearly 33-ft-tall towers were made from 10¾-in.-OD pipe and L3×3×½ cross frames, all connected with ¾-in. A325 bolts.

above: Larry O'Connell locates the bracket to stop the columns once they are raised.

left: The tower, in position to be raised, with lifting devices installed on the scaffolding.

It's cable day once again! ("It seemed like it was every day," observed Hastings and Wong.) These are the wind guy cables (above). With help from the local community, the team members install the main cables (below).

left: The team pulls the main cables as tight as possible before using a winch to set the final tension.

above: All of the suspenders and crossbeams are installed. Next up: the decking.

below: Craig Smart, John Hastings, and a B2P representative working the decking toward the middle of the bridge.

Built to Serve

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 2019, B2P will complete 29 new footbridges, increasing its overall total to 314 bridges and impacting more than 1,149,000 people since 2001. To learn more about B2P, how you can become a volunteer or industry partner, or to support the mission, visit **www.bridgestoprosperity.org**.

left: Crews working on decking from both sides of the bridge toward the middle.

above: With the decking finished, the crew begins to place protective fencing.

above: The colors for the bridge represent the stripes of the Bolivian flag.

right: The team gathers on the bridge on its inauguration day.

steelwise

BRINGING BRIDGES BACK FROM THE BRINK

BY DAN McCAFFREY, PE

Aging steel bridges can be often be refurbished to extend their service life. Here are some considerations for bridge repair and rehabilitation.

Dan McCaffrey (**demccaffrey@modjeski.com**) is a structural project manager with Modjeski and Masters.

AMERICA'S BRIDGE INFRASTRUCTURE is vast. And much of it is past its prime.

Of the roughly 600,000 bridges in the U.S., approximately 40% are over 50 years old. And while there's much talk by politicians about the need to inject more money and effort into building new bridges and replacing old ones, the approach has been piecemeal to date and the sheer volume of work required is overwhelming.

Luckily, in many cases, full replacement isn't necessary and steel bridges can see their lives extended through rehabilitation of certain areas or components. Here, I'll present some considerations and advice on rehabilitating steel bridges to bring them back to full strength and keep them that way for as long as possible.

Cast or Band-Aid?

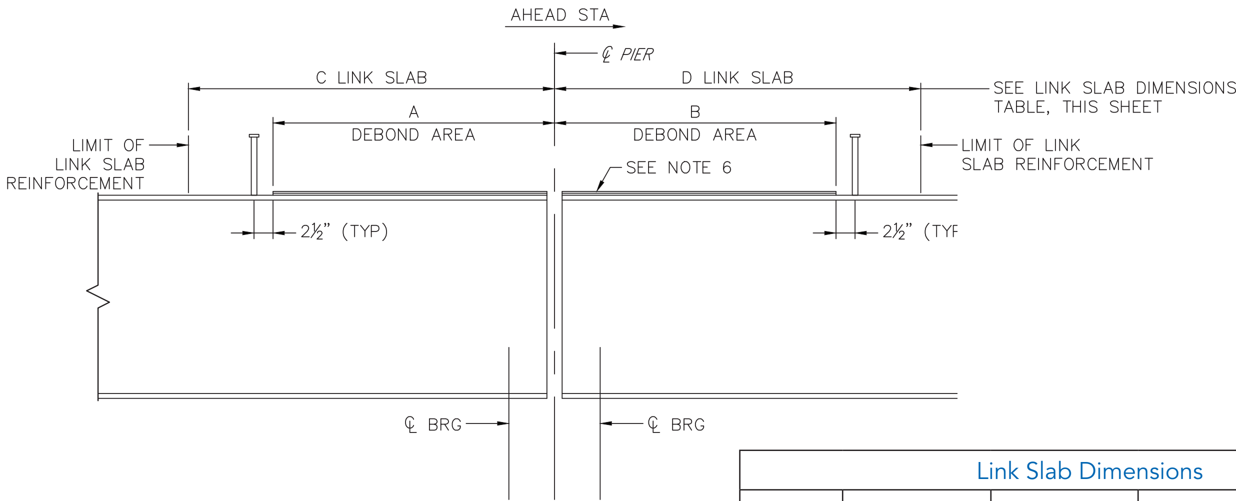
Rehabilitation becomes necessary when even our best efforts of maintenance and preservation are not enough to win the battle against nature, or it may become required simply due to increased weight and traffic volume over time. When considering rehabilitation, the first question to ask is if this will be a long-term rehab or a temporary solution until replacement is possible. Knowing what your expectations are for a bridge will keep you from taking unnecessary steps and incurring additional expenses or minimizing the risk of having to implement a series of Band-aid solutions. This requires an evaluation of not just the bridge in question, but also the remainder of the system using the bridge. If a replacement is inevitable for other reasons in 15 to 20 years, then the scope for the rehab project should be adjusted accordingly. Similarly, if the desired timeline for additional bridge life is 40 to 50 years or even beyond (100?) do the best with what is known, but keep in mind that technology and material science will be advancing during the upcoming decades.

Rehabilitations often arise due to low load ratings, but sometimes there are also deterioration issues that don't show up in a rating because they are not in the direct load-carrying path. This shouldn't minimize the need to address these items, and it's often best to take care of these serviceability-related issues before they lead to a strength-related problem, and to include them in the rehab scope of work.

Eliminating Deck Joints

One of the most successful changes in the design of new bridges is the elimination or reduction of deck joints. Many older bridges were built as a series of simple spans, often with deck joints at each pier. While some of these older bridges are now being replaced, others don't warrant that type of investment. Instead, they are being re-decked and rehabilitated, and as part of these projects, some owners have incorporated the concept of eliminating deck joints. Although there are several ways of accommodating the elimination of a joint, one way is through the use of link slabs. A link slab causes the deck to be continuous, while allowing the steel superstructure to continue to act as a simple span.

A bridge with a link slab is different than one using continuous girders. Essentially, the moment in the girder still drops down to zero at the support, because the adjacent spans are still allowed to rotate relative to one another. This is accomplished by ensuring that at



A typical link slab detail and link slab dimensions for the 15th Street Bridge in Philadelphia, which underwent re-decking. The rehabilitation project involved replacing the deck with a new composite concrete deck, replacing the bearings, and making repairs to the abutments and piers.

Link Slab Dimensions				
Location	Dimension A	Dimension B	Dimension C	Dimension D
Pier 1	1ft 11½in.	3ft 6½in.	5ft ½in.	6ft 7½in.
Pier 2	3ft 5in.	3ft 5in.	6ft 0in.	6ft 0in.
Pier 3S	3ft 6½in.	10½in.	6ft 7½in.	3ft 11½in.
Pier 3N	10in.	2ft 2in.	3ft 11in.	5ft 3in.
Bent 4	2ft 0in.	1ft 4½in.	5ft 1in.	4ft 5½in.

least one of the bearing lines at each pier is not fixed in the longitudinal direction. Even though the positions of the top flanges of the girders are locked, as long as the bottom flanges can move relative to one another, a force couple cannot be developed. In other words, instead of the hinging location being at the bearings and having a deck joint that opens and closes, the hinge point is now the deck, and the space between the bottom flanges opens and closes. This hinging action of the deck of course puts moment in the deck, and additional steel reinforcement is necessary. However, this is a deformation-induced stress, not a load-induced deformation, and even if small cracks do develop, they won't affect the load-carrying characteristics of the bridge.

At Modjeski and Masters, we've employed link slabs in several recent rehabilitation projects, one of which was the re-decking of the 15th Street Bridge over Callowhill Cut in Philadelphia. This steel bridge included six simple spans with a non-composite concrete deck and four total deck joints (three of the piers had a deck construction joint, not a real movement joint). This project involved replacing the deck with a new composite concrete deck, replacing the bearings, and making repairs to the abutments and piers. Link slabs were used at all pier locations and the final configuration only had two deck joints, one at each abutment. As is typical for this type of project, the joints at the abutments needed to be larger to accommodate the increased thermal movement that was shifted here from the joints at the piers. During the bearing replacement design, some fixity configurations were rearranged to ensure that no pier had two lines of fixed bearings. Also, the new shear studs were omitted over the last several feet of the beam, and a bond breaker was added on top of the flange to enable the deck and beams to act more independently. An additional benefit of the fully connected bridge is that it is expected to act more favorably during a seismic event. (For more on eliminating joints and incorporating link slabs, see "Piece by Piece" in the September 2014 issue, available at **www.modernsteel.com**.)

Coating Considerations

Once the rehab scope of work is determined, planning how best to perform the work is the next step. This usually depends on a lot of factors that aren't strength related, but professional engineers need to be well versed in all aspects of the project, not just what is necessary from a load-carrying standpoint. Some of those factors are environmental, safety, impact to the travelling public, or schedule. Nearly always, decreasing the duration of a project or task is beneficial for all the above factors, and "get in and get out" is the new rehab theme. For a steel bridge rehabilitation project, where a procedure requires existing steel to be uncovered and prime painted prior to new material being added, choosing the right faying surface primer can make all the difference.

Many owners have approved product lists that include coating systems, some of which have required cure times of over 150 hours to develop the Class B slip coefficient. By specifying a typical coating without considering schedule impacts, or worse, not providing any information, the project schedule can be extended, while the contractor literally waits for paint to dry. Alternatively, many paint manufacturers have compatible primers with much shorter cure times. The catch is that these primers sometimes have additional restrictions on application conditions, such as temperature and/or surface preparation. However, depending on the project, it may be worthwhile to accommodate these additional conditions to expedite construction.

Ideally, the potential advantage of an alternate primer would be identified during the plans, specifications, and estimate (PS&E) development phase, discussed with the owner, and incorporated into the special provisions. Many owners have already taken the step of including alternate primers in their approved product listing, and it is just a matter of specifying the alternate. Consulting the coating manufacturer for specific recommendations is also a good practice.

A primer with a cure time of 19 hours was used on a steel railing replacement project on the Ambassador Bridge between Detroit and Windsor, Canada.

For example, when it came to select steel repairs that were part of a recent railing replacement project on the Ambassador Bridge in Detroit, we applied a Carbozinc 11 HS primer (with a cure time of 19 hours) in lieu of the project's typical Carbozinc 859 (100-hour cure time) because the contractor had to first remove concrete deck, blast clean existing steel, and fasten new strengthening material prior to forming and placing new concrete. Going with the alternate primer enabled the contractor to maintain a linear progression of repair steps, remain in the same general area of work until complete, and not hold up subsequent rehabilitation work. For a different repair to wind bracing elements, we specified the Carbozinc 11 WB primer, which only requires a 4-hour cure time before bolt-up but can only be applied in warmer temperatures. This enabled the repair at each location to be performed in one work shift, which was important due to wind restrictions during disassembly, and the unreliability of wind forecasting beyond the present day. (For more on corrosion resistance, visit **aisc.org/nsba/nsba-publications** and peruse the "Corrosion Protection of Steel Bridges" portion of the *Steel Bridge Design Handbook*.)

Verifying Loads

Although engineering may appear to be an exact science to the public, those in the industry know that it involves a lot of probability considerations, conservatism, and judgement. Like the load and resistance factor design (LRFD) method, we manage risks based on the likelihood of the existence and magnitude of loads, combined with potential variations in material properties and geometry.

Oftentimes, rehabilitation projects are born out of a structure's rating that indicates a less-than-desirable factor of safety. If the engineer suspects that over-conservatism is leading to a ballooning of the scope of work, it may make sense to perform field testing to verify actual structural responses under known live loading conditions. There are usually more load paths than accounted for in design, and actual stresses are often lower or more evenly distributed. This is one reason why there can be bridges with a calculated rating of zero that show no signs of distress. Strain gauges, used with controlled loads, can be used to determine actual stresses and force distributions and to refine the analyses used in the ratings. Bottom line, better information will lead to better rehabilitation decisions.

Similarly, existing dead loads can be different than expected, as can be seen on projects that include lifting the existing structure. In our experience, the actual lifting load is often up to 120% of the weight that would be conventionally calculated, so we take this into account when sizing jacks and temporary supports. Other times, depending on the structure type (especially if the structure is statically indeterminate or cable supported) the actual required jacking load can be lower than anticipated. If temporary supports are a significant portion of the construction effort, it may make sense to verify the anticipated jacking load using a test lift as a way to reduce the design requirements for those temporary supports.

On the rocker link replacement project at the Ambassador Bridge, we recommended the use of a test lift to verify the dead load present at each of the four corners. On this cable-supported bridge, geometry and suspender rope tensions play a big part in the link reactions. The conceptual temporary support was to be cantilevered from the tower to avoid interference with the actual work. However, for the test lift, a relatively small jacking assembly was easily placed in line with the existing link. The resulting dead loads were then combined with the calculated live load, wind and temperature effects. In the end, the temporary link and supports were designed and detailed for a maximum load of 500 kips, significantly less than the 1,020 kips that the permanent links were designed for. Considering that the temporary link was supported on a bracket that was cantilevered from the tower, this reduction in design load had a significant impact on the size, complexity, time to construct, and cost of the temporary support—and no temporary strengthening of the tower was required.

Pulling Ideas from Elsewhere

One thing to remember when attempting to solve complex rehabilitation problems is that you are not alone. If you seem to be stuck finding a solution, look outside your sandbox. It may be as simple as looking to what was done on other bridge types or even looking to other industries for material, detail and process ideas.

A recent example is the anchor link replacement project we conducted for the Blue Water Bridge in Port Huron, Mich. One of the biggest challenges was that the original tension link was entirely inside of the box tower leg, with the end of the truss penetrating the leg to produce a nearly vertical link, severely limiting access. Replacing the existing link with a new link in the same position required a temporary support that was located outside of the tower and out of the way of the work. Trusses should only be loaded at their panel points, not within members, and fortunately a portion of the end gusset plate extended outside of the tower, which could be connected to. The load was large enough that many fasteners were needed, but the existing gusset plate fasteners were already carrying the truss load and couldn't simply be removed to connect a temporary support; so we chose a method that has been used in a small number of cases for gusset plate strengthening.

The challenge is that although all fasteners cannot be removed at the same time to install new material, they can be removed and replaced one at a time. The trick is that they are replaced with new bolts with extra-long threading, and this extra stick-thru can be used to attach the new material. A special fill plate, or "cheese" plate, is placed over the first set of nuts, providing a flat surface to install new material against. To fasten the new material, a second nut is added to the bolt's shank and fully tightened, and now the whole stack of plates acts in unison. The original material will have locked-in stresses from its current state, while the new material will be at a zero-stress state.

For the Blue Water Bridge, this concept was used to attach a temporary "knuckle" plate to the gusset, which supported a temporary link via a 9.5-in.-diameter pin. Once the new permanent link was installed and the load transferred, the installation process for the temporary connection was reversed to remove the knuckle plate and cheese plate and the fasteners were once again replaced one at a time, this time with permanent bolts of normal length. Compared to other temporary support options, such as extending 80 ft to the ground, connecting to the gusset plate in this manner enabled the use of a more compact temporary support that was controllable and predictable, used less material, and reduced contractor access requirements.


M&M

above and below: The rocker link replacement for the Ambassador Bridge. A test lift was used to verify the dead load present at each of the four corners.


M&M

below and right: The Blue Water Bridge in Port Huron, Mich., underwent an anchor link replacement, where the fasteners were replaced with new bolts with extra-long threading.

OHM Consultants

Embrace Innovation

In a world where knowledge and technology are advancing rapidly, it's easy to be left behind. Due to the very nature of our profession of minimizing and managing risks to provide dependable infrastructure, we tend to rely heavily on tradition and what's been proven. But if we're going to reach our industry's 100-year-bridge-life goal (and beyond), we need to look beyond conventional solutions. Being open to unusual or new ideas, flexible, and willing to think outside of the box (even in simple ways) can help the project and the entire industry advance. Ultimately, when we embrace innovation, share ideas, and improve the state of the art, we all win. ■

PCL Constructio

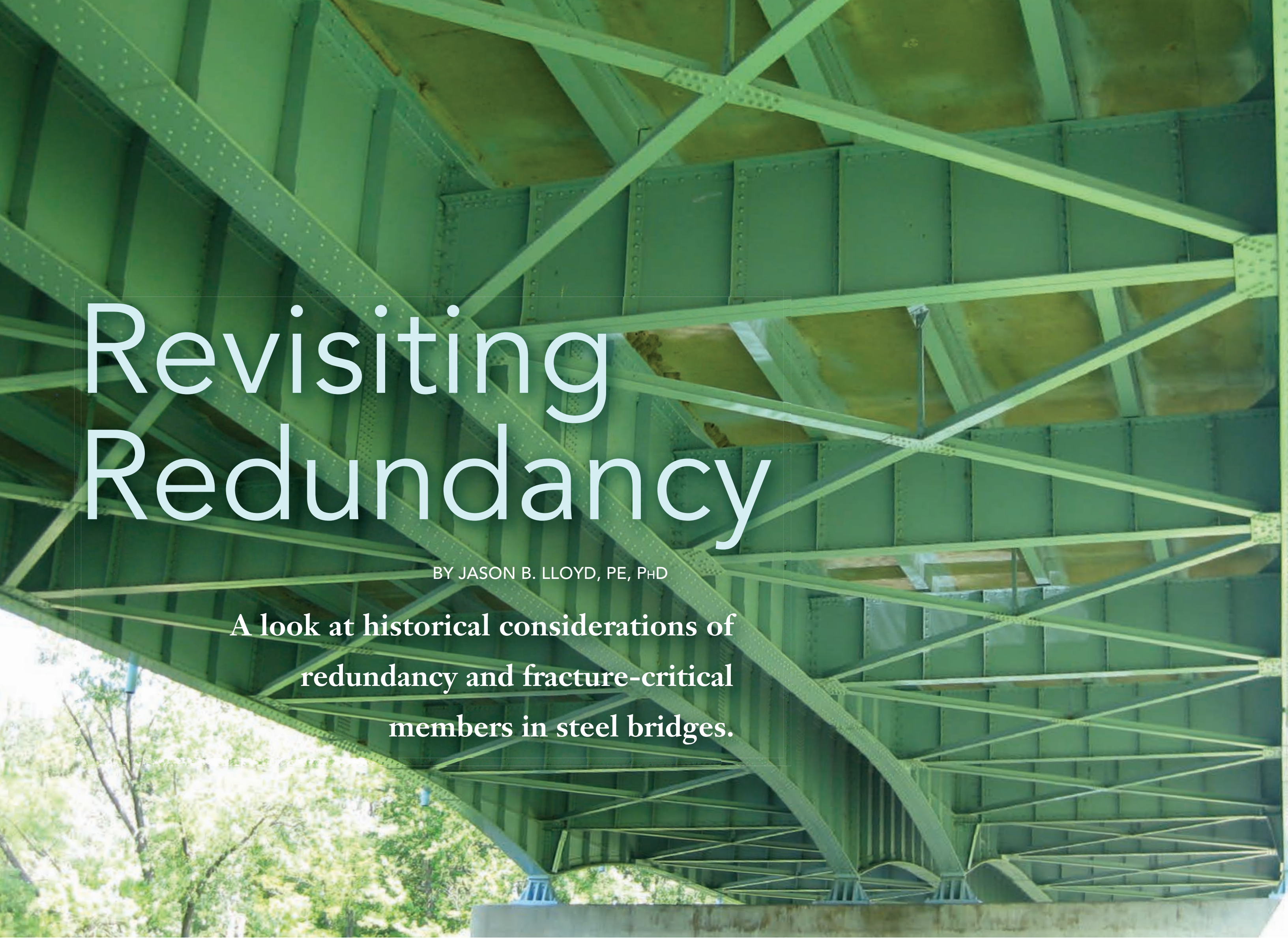
M&M

above: An outboard view of the knuckle plate on the Blue Water Bridge.

Getting Its Bearings

Seismic upgrades are the impetus for some steel rehabilitation projects. As an example, the steel-framed approach ramps for the RFK Bridge in Manhattan, while in good structural condition, weren't up to current seismic standards, so the steel superstructure was retrofitted to include a seismic isolation system. A "floating deck" isolation system was developed that isolated the new deck and floor system from the existing steel rigid-frame substructure below. The scheme 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.

The design team at Modjeski and Masters implemented a hybrid system, using both sliding bearings and elastomeric bearings, with the sliding bearings carrying all vertical loads and dissipating energy through friction, and only a small number of isolation bearings being needed 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 system also resulted in most of the expansion joints being removed, thereby reducing the major source of deterioration (namely chloride-laden water infiltrating the steel) and creating a more maintenance-free structure. (The project is a 2018 NSBA Prize Bridge Award winner; see the June 2018 issue at **www.modernsteel.com** to learn more about it.)



Revisiting Redundancy

BY JASON B. LLOYD, PE, PhD

A look at historical considerations of redundancy and fracture-critical members in steel bridges.

Twin, built-up riveted, two-girder bridges carrying US-41 over the White River in southern Indiana.

THE CHALLENGE of engineering innovation is often balancing the tradeoffs between different performance characteristics—and of course, cost.

Take planes, for example, where innovation has been driven by a desire to improve powered flight, increasing the performance of the aircraft in terms of weight, lift, thrust, and drag in order to produce the fastest and highest-flying airplane possible. Early developments in aviation engineering often produced planes using a stacked-wing configuration, allowing them to achieve more lift with less engine power and without the weight of the wing becoming prohibitive.

One such example is the Sopwith Triplane, manufactured by the Sopwith Aviation Company, which was introduced during World War I. Some advantages of the triplane were a shorter and lighter wing that provided more lift, a wider field of view for the pilot, and improved elevator response (vertical pitch), enhancing maneuverability—a clear advantage for a fighter plane.

Triplanes and biplanes were contrasted by monoplanes (planes having a single fixed wing), which came with their own advantages. These included reduced drag (via eliminating the exposed bracing between wings and internally carrying all of the wing forces) as well as superior aerodynamic efficiency, allowing faster flight. However, they also required higher-powered engines to fly the heavier frames, whereas multi-wing planes possessed superior structural efficiency, allowing smaller and lighter wings, lower-powered engines, and slower stall speeds. As aviation innovation continued, thanks to significant advancements in aerodynamics-related knowledge, engines became more powerful while wing materials became lighter and stronger, moving the aircraft industry almost exclusively toward the monoplane designs that we see today.

Perhaps you're wondering how this relates to steel bridges. Some time ago, a good friend and mentor facetiously made a comparison between a fracture-critical member (FCM) in

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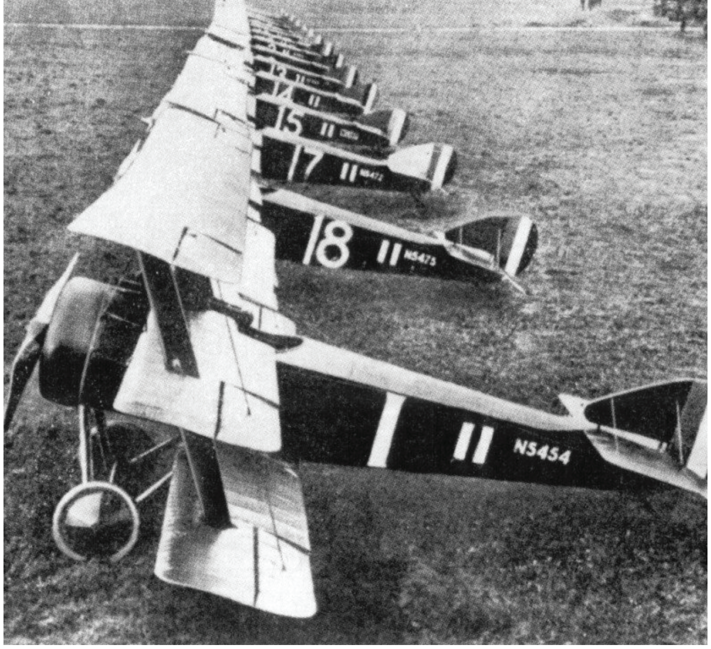

left: More isn't always better—especially when it comes to wings. Thanks to advancements in aviation technology, the only places we see aircraft such as the Sopwith Triplane these days are museums and air shows.

below: The built-up riveted floor truss of the Golden Gate Bridge in San Francisco.

a bridge (typically classified as such through engineering judgment for being non-load-path redundant) and the wing of a monoplane. On occasion, we would encounter engineers who were uncomfortable with non-load-path redundant members in steel bridges, but who didn't seem to have a problem with flying on a monoplane. It begged a somewhat humorous question: Why wouldn't that engineer also insist on flying on a multi-wing plane, or a plane with multiple sets of landing gear, in order to have load-path redundancy during their 36,000-ft commute? (If you want to hear more about this comparison straight from the source, check out Rob Connor's 2018 AISC T.R. Higgins Lecture "Towards an Integrated Fracture-Control Plan for Steel Bridges" at **aisc.org/2018nascconline**.)

Clearly, the aviation industry is motivated to use reliable and redundant structures. So why don't they use multi-wing planes for the case of catastrophic wing failure? Wing failures have occurred in the past in older planes. The simple answer is that they have developed alternative methods to design, fabricate, inspect, and maintain critical elements of their air frames by exploiting forms of redundancy other than load-path redundancy, such as fail-safe and damage-tolerant design methods. These methods recognize that structures must withstand service loads even when damaged or cracked until reliable inspection methods can identify the damage. For example, the wing structure of the plane might possess multiple load paths internal to the wing, mechanically fastened composite layered structures that offer strength and crack arrest capability, other crack arrest detailing, experimental fatigue testing to develop life-prediction models, and inspection programs that are linked to the design, fabrication, fatigue life, and probability of detecting defects.

When it comes to steel bridge design, can we borrow a chapter from the aviation industry's book? Can we exploit other modes of redundancy in steel bridges that might allow for more economical design options? And can we integrate the fracture-control plan (FCP) and link material, design, fabrication, and field inspection frequency to damage tolerance? The answer to all of these questions is Yes!

Historical Context

First, we should understand how we, as an industry, arrived at current practices and policies for bridge redundancy and FCMs. Following the infamous collapse of the Silver Bridge over the Ohio River in 1967, the Federal-Aid Highway Act of 1968 originated a requirement for the Secretary of Transportation to establish the National Bridge Inspection Standards (NBIS) to help ensure the safety of the nation's bridges. The NBIS is overseen by the Federal Highway Administration (FHWA) and is defined by the *Code of Federal Regulations*. Later, the Federal-Aid Highway Act of 1970 limited the NBIS to bridges on the Federal-Aid highway system.

However, the Surface Transportation Assistance Act of 1978 extended the NBIS requirements to all bridges greater than 20 ft on public roads. Then, the Surface Transportation and Uniform Relocation Assistance Act of 1987 expanded the scope of bridge inspection programs to identify FCMs and establish inspection procedures for them. This was possibly motivated by the partial failure of the Mianus River Bridge in 1983 (which was not caused by fracture). Currently, the inspection period for bridges containing FCMs in the United States is mandated at a maximum of 24 months and inspection of FCMs must be performed at "arms-length." This inspection frequency was first defined in the NBIS beginning in 1988. It was based on expert consensus, not necessarily on scientific research or statistical modeling.

In parallel with development of the abovementioned statutes, research was conducted to address concerns related to steel bridge members subjected to tension, specifically as related to the fatigue and fracture limit states. The research resulted in significant additions to the 1974 American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications, including Charpy V-notch (CVN) testing requirements to ensure a minimum toughness (i.e., resistance to fracture in the presence of a crack) at the lowest anticipated service temperature of the non-load path redundant member. Also, the first comprehensive fatigue design provisions were added, introducing the fatigue categories and their respective fatigue resistances.

In 1978, AASHTO published the first edition of the *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*, becoming known as the "AASHTO Fracture-Control Plan." This was the document that introduced the term "fracture critical" and implemented reduced fatigue stress range limits and improved fabrication quality control measures for FCMs. Eventually, the 1978 *Guide Specifications* were abandoned when the FCM requirements were incorporated into ASTM A709 *Standard Specification for Structural Steel for Bridges*, the AASHTO *Bridge Design Specifications*, and AASHTO/AWS D1.5 *Bridge Welding Code* (Clause 12).

While legislation and research helped to shape policy for FCMs, including frequency and depth of inspection, it remained incumbent upon the engineer of record (EOR) to identify FCMs in new design and upon inspectors in existing bridges. The *Code of Federal Regulations* Title 23, Part 650, defined an FCM as a "steel member in tension, or with a tension element, whose failure would probably cause a portion

Built-up riveted bascule bridges carrying vehicular traffic over the Chicago River in downtown Chicago.

of or the entire bridge to collapse." However, without further guidance, it became state-of-practice to designate any tension member that appeared to not be load-path-redundant, as fracture-critical (such as in a two-girder bridge). But the authors of NCHRP (National Cooperative Highway Research Program) Synthesis 354 pointed out that this designation was not applied consistently by owners.

After several decades, the end result is that many bridge engineers are now accustomed to determining redundancy through engineering judgment that is married to a single approach: load path (or number of girder lines). And as an industry, we became comfortable with many girder lines and uncertain, or even afraid, of anything less. That uncertainty was perhaps reinforced for some by the tragic collapse of the I-35W Bridge in Minneapolis in 2007. However, the collapse was actually caused by a design error that resulted in a buckling-induced failure mode. It was not a result of fracture, nor was it related to FCMs. Yet prominent documents such as the *Bridge Inspectors Reference Manual (BIRM)* and countless fracture-related papers and presentations continue to incorrectly promulgate it as an FCM-related collapse.

An Outdated Approach?

Adding girder lines is not an exclusive approach to increasing reliability and in some cases may not be the most efficient design approach either. According to an international scan of other industrialized countries (*Steel Bridge Fabrication Technologies in Europe and Japan*, Report FHWA-PL-01-018) the U.S. appears to be unique in its view of non-load-path-redundant structures. The report suggests that the U.S. design philosophy for non-redundant bridges should be reconsidered. This speaks to a need to revisit outdated practices as well as redundancy in order to allow for design optimization.

We should ask ourselves this: When it comes to redundancy, are we still designing the bridge equivalent of a triplane in some ways? Can we reduce the drag of outdated design philosophies to soar to new heights through innovations that still produce reliable and redundant steel bridges? Reliability of our structures is not load-path-dependent. It can also be achieved through improved materials, design and detailing methods, and fabrication practices. This is anecdotally supported by the fact that there have been no known fractures of FCMs designed and fabricated to FCP standards since its implementation over 40 years ago (for more information, see the fourth quarter 2019 AISC *Engineering Journal* article "Simplified Transformative Approaches for Evaluating the Criticality of Fracture in Steel Members" via **aisc.org/ej**). And innovation continues to power the steel bridge industry forward in areas such as corrosion resistance, material toughness, material strength, welding processes, non-destructive testing, and infinite fatigue life.

These innovations make reliable bridges possible with alternate modes of redundancy, such as system redundancy and internal member redundancy. System-level redundancy prevents the partial or full collapse of a bridge following failure of a system-redundant member (SRM) by redistribution of load through the interconnected system of primary and secondary members and the deck. Member-level redundancy prevents the partial or full collapse of a bridge following failure of a single component within an internally redundant member (IRM) by redistribution of load into adjacent mechanically fastened components of the member itself. System redundancy and member-level redundancy following failure of FCMs (that were built prior to the FCP) have been observed several times over many decades. The empirical evidence demonstrating these forms of redundancy, combined with advancements in fracture control and structural analysis, left leaders in the steel bridge industry asking good questions, like:

- In the absence of load-path redundancy, how can we identify what is an FCM?
- What load case(s) is appropriate and what level of analysis should be required?
- If a member is found to be an SRM or an IRM, how do we link the damage tolerance and the inspection interval?

The basis of these questions was recently researched at Purdue University under state pooled-fund and NCHRP research grants. Researchers studied the fracture resistance, after-fracture load redistribution behavior, and after-fracture fatigue life of members that would have traditionally been considered non-redundant members or FCMs. The research to date has resulted in two newly published AASHTO *Guide Specifications*: the AASHTO *Guide Specifications for Identification of Fracture Critical and System Redundant Members* and the AASHTO *Guide Specifications for Internal Redundancy of Mechanically-fastened Built-up Steel Members*. These new publications offer forward progress in innovative thinking for redundancy in the steel bridge industry. We'll provide more detailed discussions of each *Guide Specification* in upcoming issues of *Modern Steel Construction*.

Crossing the Creek

BY ROBERT ANDERSON, SE, PE,TREVOR KIRKPATRICK, PE, AND KEVIN SWEAT, PE

AECOM

A new steel bridge comes together over a downtown waterway in Texas' capital city thanks to well-planned and executed design and construction.

SHOAL CREEK in downtown Austin might be a fairly modest waterway, but it's seen some big changes in recent years.

The area has been transformed by several projects: the decommissioning of the Green Water Treatment Plant (GWTP) site, stabilization of the east bank of the creek itself, construction of the new Central Library to the west, construction of high-rise condominiums with retail space and restaurants to the east, and the extension of 2nd Street between San Antonio Street and West Avenue.

The latter aspect was the genesis of the 2nd Street Bridge, a new crossing that will provide a vital link for vehicles and pedestrians over Shoal Creek between the new library to the west and the residential/retail areas to the east.

The new bridge is designed, proportioned, and detailed to offer an elegant solution to connect the two sides of the 2nd Street over Shoal Creek with an iconic structure that is and integrated with the future vision of the booming area. Through a series of meetings and design charrettes, AECOM developed a tiered process to elicit input and obtain decisions from key stakeholders. During those workshops the team analyzed and evaluated multiple possible structural steel forms for the bridge, including traditional girder, cable-stayed, and arch schemes.

The chosen bridge type was a canted arch spanning approximately 160 ft. The overdeck supporting elements of the bridge are a pair of trapezoidal shaped steel ribs, each with a network arrangement of galvanized wire rope hangers connected above the deck to the girder framing. A central utility corridor between the box girders accommodates the multiple utility lines that cross the bridge, and the bottom soffit of the corridor is screened by a metal deck bar grating. Outrigger beams carry a curved pedestrian sidewalk that ranges from 12 ft to 14 ft wide. The thrust of the arch ribs is resisted by a foundation system with 6-ft-diameter drilled shafts anchored to bedrock.

Every component of the structure was examined to fit the needs of the project. For example, a gap was created between the sidewalk slab and the bridge's traffic deck. This opening allows light to pass through to Shoal Creek below and creates a feeling of lightness to the bridge. To keep the outriggers from collecting dirt, a stainless steel "hat" section was added to keep the tops of the outriggers clean.

The mantra of "form follows function" was certainly achieved in the bridge's design. The exterior webs of the box girders are canted at a 15° angle to connect to supporting cables to the arch ribs. Transverse framing transfers load from the interior of the girder system to the outside webs, and the tub girders also work to carry load longitudinally to the bearing supports. The result is a highly redundant structural system that is also non-fracture critical, thus reducing future inspection costs for the City of Austin. In addition, the network cable provides longitudinal restraint to the superstructure and also reduces thrust created by the arch rib.

Steel Components

The 160-ft single-span canted parabolic arch bridge varies in width from 63 ft at the abutment to 73 ft at mid-span. The superstructure consists of two steel box girders joined by cross-frames with a composite deck slab, providing two 12-ft-wide traffic lanes. Each

A gap between the sidewalk slab and the traffic deck allows light to pass through to Shoal Creek below and makes the crossing appear lighter and more open to pedestrian and bicycle traffic along the creek.

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Touchstone Architecture

above: The arch ribs are attached to thrust blocks on either end of the bridge via base plates and anchor bolts.

right: The arches were lifted and set using a 400-ton crane with help from a 100-ft main spreader beam and two 30-ft spreader beams.

below: Attaching an outrigger to a superstructure tub girder at the fabrication shop.


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City of Austin

of the two arch ribs is canted (sloped) outwards 15° from the vertical plane, matching the angle of the box girder framing, and rises some 31 ft above the roadway.

Each rib comprises a trapezoidal steel box section 3 ft deep with a width varying from 2 ft at the bottom to 3 ft at the top. One refinement made during final design was the selection of painted weathering steel for the arch rib (it was realized that painting the interior of the arch rib after fabrication would be impractical), and a second refinement was the decision to field-weld the center connection of two of the rib sections.

The thrust arch system resists the compressive forces produced by the arch rib with the foundation elements, versus a tied arch, which resists the arch rib forces with a bottom chord tie. The ends of each arch rib are supported at concrete thrust blocks connected to large concrete footings at each end of the bridge. These footings also support some of the weight of the bridge deck superstructure carried by the two longitudinal steel box beams. Each footing is supported by six 6-ft-diameter drilled shafts that are socketed into the underlying bedrock. The deck roadway surface comprises a 9-in.-thick reinforced concrete slab acting compositely with the steel box beams.

A network of 20 2-in. galvanized wire rope hangers along each arch rib supports the bridge deck structure below. Each hanger is sloped ~45° in the direction of traffic, resulting in a diamond pattern. The tops of the hangers are connected to the arch rib using a forked pin-and-clevis system, and the bottoms are connected to the top of the longitudinal girders using a bolted anchor assembly. The hangers were stressed to approximately offset the tributary load of each panel and thus minimize the longitudinal girder moment. With one or the other bearing longitu-

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above: The bridge is at the epicenter of downtown Austin's construction boom.

left and below: A network of 20 galvanized wire rope hangers along each arch rib support the bridge deck structure below. The tops of the hangers are connected to the arch rib using a forked pin-and-clevis system, and the bottoms are connected to the top of the longitudinal girders using a bolted anchor assembly.

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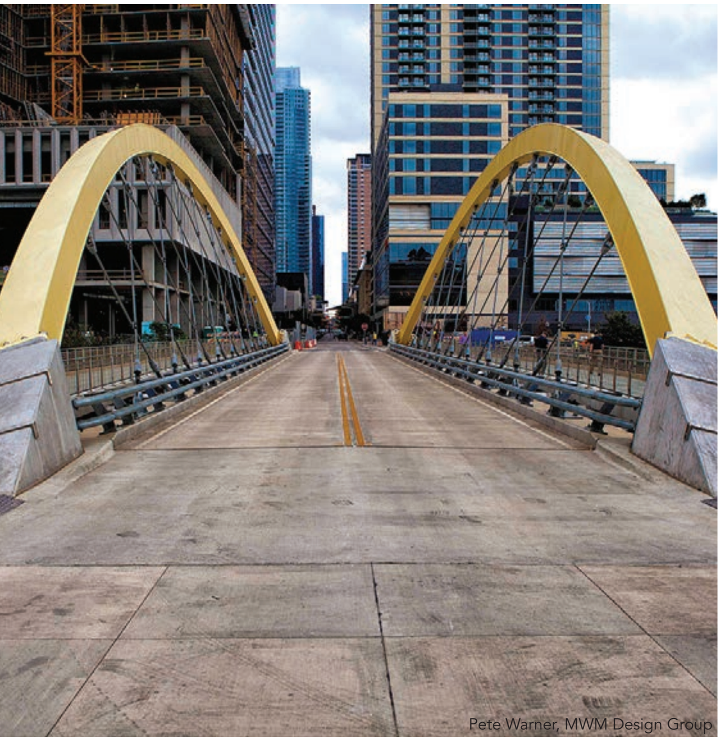
dinally engaged, it became apparent that the short hanger cables would draw too much force due to thermal loads. Therefore, the bridge was released at both ends for longitudinal movement. Longitudinal restraint is provided by the hanger cable network, which transmits longitudinal loads to each thrust block and abutment.

Ten steel "outrigger" I-beams spaced at 14 ft, 6½ in. along the length of the bridge extend outward from each longitudinal steel box beam to support a sidewalk. Each outrigger beam varies in depth from ~6 ft at the steel box beam to ~2 ft, 6 in. at the free end. Like the traffic deck, the sidewalk also consists of a 9-in.-thick reinforced concrete slab—again, with a varying width of 12 ft at the abutment to 14 ft at mid-span. A precast fascia beam attached to the ends of the outrigger beams provides a clean line at the outside edge of the bridge. The width of the gap between the roadway and sidewalk slabs ranges from 3 ft to 6 ft. The combination of the varying gap width and varying sidewalk width creates a curved edge beam in plan, with a depth that varies from 2 ft, 6 in. at mid-span to ~3 ft, 4 in. at the abutment.

Construction

The structural steel components, including the longitudinal steel box girders and the outrigger beams, were shipped to the site via truck. To develop a proposed construction sequence for the bridge, careful consideration was given to the presence of existing overhead power lines located over the east end of the bridge site. A shorter, lighter section of the longitudinal girder was detailed beneath the overhead power lines, enabling a smaller, low-head crane to pick up and place the girder section. The erection sequence presented in the plans was used to construct the bridge, and the contractor supplemented the right: Shipping a completed half of an arch-rib girder from the fabrication facility.

below: The pedestrian and vehicle bridge carries 2nd Street over Austin's Shoal Creek.

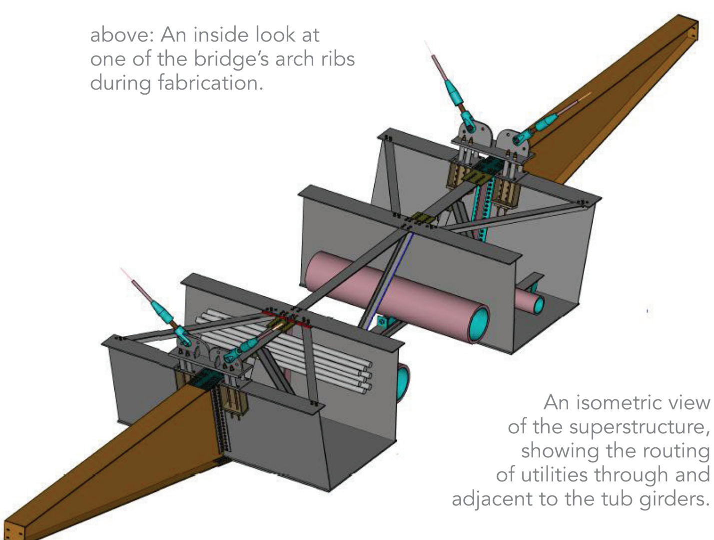


Pete Warner, MWM Design Group

Florida Structural Steel/Tampa Tank

AECOM

above: An inside look at one of the bridge's arch ribs during fabrication.



An isometric view of the superstructure, showing the routing of utilities through and adjacent to the tub girders.

erection sequence with erection plans developed by the erection engineer. The general steps are as follows:

- Stage 1 – Install the foundations. Install the girder temporary shoring in creek.
- Stage 2 – Erect and splice/bolt longitudinal girder sections at the east end.
- Stage 3 – Erect and splice short longitudinal girder section beneath the overhead power lines at the west end.
- Stage 4 – Erect the cross frames, diaphragms, and outrigger beams.
- Stage 5 – Cast the roadway deck and sidewalk concrete.
- Stage 6 – Install the arch rib falsework and erect the arch rib.
- Stage 7 – Remove the arch rib falsework and install and stress the hangers.
- Stage 8 – Remove the girder temporary shoring.
- Stage 9 – Install and complete utilities and finishing works.

The longitudinal girders were lifted and set using a 600-ton crane positioned near the southeast corner of the bridge. Each of the twin box girders were set in two lifts. The first lift comprised the east and central field sections, which were spliced together prior to lifting. The second lift comprised the shorter west field section and was made continuous by splicing the sections in the air. Although an allowance was made for a smaller crane to set the west field section to avoid power lines over the west abutment, the contractor used the larger crane by working with the utility company to temporarily de-energize the lines.

The arch ribs were shipped to the site in halves, and field welding was used to join the halves of the arch rib on the ground. The full length of the arch ribs was lifted and set using a 400-ton crane. A 100-ft main spreader beam in tandem with two 30-ft spreader beams and varying length cables were also used. After grouting the arch rib base plate and stressing the anchor bolts at the connection to the thrust block, the arch rib temporary tower was removed.

left and below: The new 160-ft-long bridge is designed to offer an elegant solution to connect 2nd Street over Shoal Creek with an iconic structure that is friendly to both vehicles and pedestrians.

The cable hangers were fabricated off-site to the lengths specified in the contract documents. Each of the hanger cables was initially connected to the upper and lower pin plate with slack in the cable. The cables were then sequentially stressed to the target jacking forces provided by the erection engineer.

A jacking assembly fabricated by the contractor used hydraulic jacks bearing on the cable anchor and attached to high-strength steel rods to tension the cables. The steel rods were fixed to the lower anchorage assembly by jacking holes provided in the lower pin plate.

The force in each cable was confirmed with a lift-off test. Fine adjustments were made to the cable forces based on the result of this test. With the superstructure in place, miscellaneous components and finishing works were then installed, including railings and utilities. ■

Owner
City of Austin, Texas

Construction Manager
Hensel Phelps Construction Co., Austin

Architect
Touchstone Architecture, Miramar Beach, Fla.

Landscape Architect
MWM DesignGroup, Austin

Structural Engineer
AECOM, Tampa, Fla.

Erection Engineering
Stone Structural Engineering, Beeville, Texas
McElhanney Consulting Services, Inc., Tampa

Steel Team

Fabricators
W&W/AFCO Steel AISC CERTIFIED FABRICATOR, Little Rock, Ark. (Prime)
Florida Structural Steel/Tampa Tank AISC CERTIFIED FABRICATOR,
Tampa, Fla. (Subcontractor)

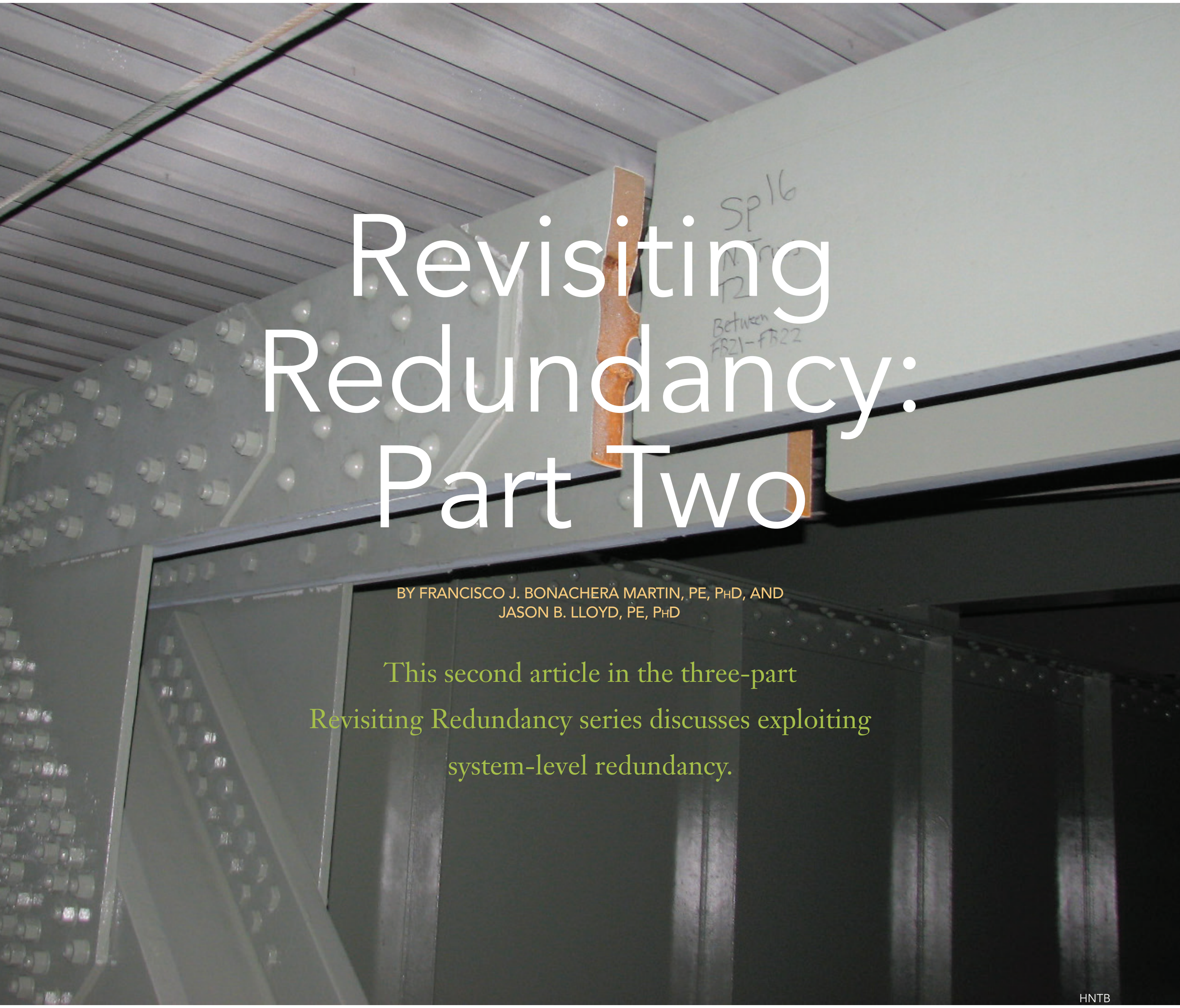
Detailer
Dowco Consultants, Ltd. AISC DETAILER, Langley, B.C.

Charrette Mindset

Design charrettes helped inform decisions on steel design schemes and other site considerations for the bridge. At the first charrette, five bridge concepts were developed that considered discussions from the kickoff meeting: circular arch, trapezoidal arch, canted/butterfly arch, single-plane cable-stay arch, and dual-plane cable stay arch. The arch concepts presented used a lower arch-rib profile to lessen the vertical height impacts on the above power lines. The charrette participants stated a preference for a canted (butterfly) arch (vs. vertical) with an arch rib having a trapezoidal cross section (vs. circular). Avoiding struts, with the use of outriggers, was thought to be less busy and ended up being the preferred option. Additionally, a network arch with crisscrossing hangers was favored over vertical hangers.

The second charrette meeting focused on decisions related to more specific design features of the preferred canted arch structure type, such as hangers and coating system. Several types of wire rope hanger arrangements were presented and discussed. The topics ranged from girder connection type (bottom vs. top) to the crossing angle. The preference was stated for a ~45° crossing angle and a minimalistic above-deck anchorage connection. The top arch rib connection of the hangers was envisioned as a forked pin-and-clevis system.

At the third charrette, general discussion was undertaken regarding bridge finishes, included painting, color schemes, galvanizing, and weathering steel. Regarding unpainted weathering steel, it was removed from further consideration due to its staining potential for the adjacent concrete components. While the life-cycle cost, low maintenance, and durability advantages of galvanizing were attractive, the initial cost and non-painting ability to repair graffiti ruled out this option for the major bridge components like the arch rib and the girders (though galvanizing was felt to be appropriate for secondary steel components such as the traffic and pedestrian rails and the hanger connections at the deck level). For the main steel components, the initial color chosen was a sage green. However, the final color was determined to be yellow after consultation with City of Austin representatives and bridge architect Touchstone, as it provided more "pop" visually.



HNTB

Revisiting Redundancy: Part Two

BY FRANCISCO J. BONACHERA MARTIN, PE, PhD, AND
JASON B. LLOYD, PE, PhD

This second article in the three-part Revisiting Redundancy series discusses exploiting system-level redundancy.

DO MOST STEEL BRIDGES have post-failure load-carrying potential?

The answer is a resounding yes.

While certain bridge collapses, such as the Silver Bridge and the Mianus River Bridge—both of which collapsed due to failures of truly non-redundant tension members—suggest the contrary, the reality is that there are far more cases where steel bridges were able to operate in the faulted condition. This applies even to bridges that have traditionally been considered to have no system-level redundancy. (And of course, damaged structures still need to be repaired and inspection should be performed on all members, regardless of criticality.)

One example of a bridge that withstood the failure of a fracture-critical member (FCM) is the Lafayette Bridge, a two-girder steel bridge in which a fracture rendered a girder unable to carry any significant portion of the load. This scenario would have led to collapse if the bridge was, in fact, nonredundant—but it wasn't and it didn't. Similar scenarios include the Hoan Bridge, the U.S. 422 Bridge over the Schuylkill River, the Green River Bridge, the Diefenbaker Bridge, the Delaware River Turnpike Bridge, and countless others.

Were these structures designed to operate in the faulted state? No. Was system performance in the faulted state considered in the design? Again, no. The reality is that all of these structures, despite being designed in different eras, shared the same overall design philosophy and principles in which post-failure capacity was not considered. In all these cases, system-level redundancy was unplanned, most likely the product of typical conservatism in design. But the fact that it was unintentional does not mean that it cannot be exploited.

AASHTO's *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (referred to hereafter as the SRM Guide Spec) is a tool that allows engineers to

HNTB

above and opposite page: A fracture of a fracture-critical-designated member on the Delaware River Turnpike Bridge, which continued to carry service loads until the fracture was discovered and repaired.

take advantage of previously unexploited system-level redundancy, and owners to efficiently allocate resources to provide better infrastructural solutions to the public.

Released in 2018 and available at **www.aashto.org**, the SRM Guide Spec tackles a complex problem: characterizing the demand and capacity of a structure in which a primary steel tension member has failed. For a system to be considered redundant, two fundamental concepts regarding load were followed: First, the bridge cannot be expected to operate as reliably in the faulted condition as in the pristine condition. Second, the bridge must be able to survive the failure event and provide service in the faulted state.

The first fundamental concept is clear but leaves a question to be answered: What is an acceptable reliability level in the faulted state? To answer this question, let's take a look at the overall failure rate. Current load and resistance factor design (LRFD) bridge design provisions are based on allowing a nominal failure rate that applies to the structure in its pristine state. For the faulted state, the same nominal failure rate can be maintained by acknowledging that it is the product of the failure rate in the faulted state and the rate at which primary tension member failure occurs. In other words, by conservatively establishing how likely it is for a member designated as FCM to fail, a lower target failure rate can be calculated for the faulted state.

So why not calculate the load that causes the member to fracture instead? If a primary steel tension member fractures, load isn't the only culprit. There are also the factors of temperature, material toughness, and quality of fabrication. On top of that, fracture—caused by, say, vehicle impact—isn't always the culprit when a primary steel tension member fails.

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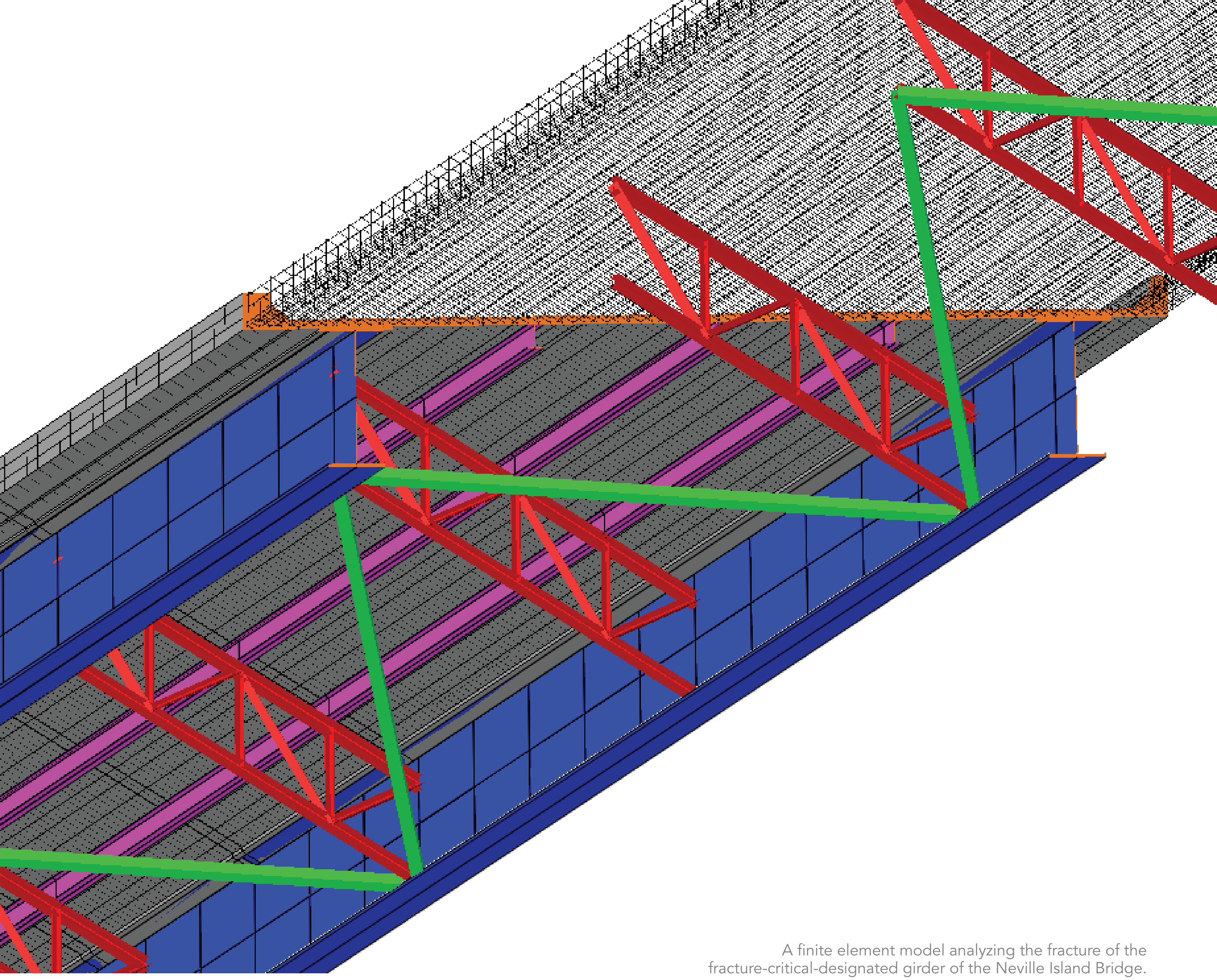

A finite element model analyzing the fracture of the fracture-critical-designated girder of the Neville Island Bridge.

Once an acceptable target failure rate, or level of reliability, for the faulted state is calculated, it is applied to the development of two new load combinations: Redundancy I and Redundancy II. Redundancy I characterizes the loads experienced by the structure during the failure event, which is assumed to be sudden fracture of a primary steel tension member. This load combination is analogous to an extreme event load combination in which the event load includes the dynamic amplification of load due to the inertial effects of the member failure. Redundancy II basically warranties strength in the faulted condition against normal use until the member failure is detected. The need for both load combinations becomes clear when considering several failure cases. For example, in the case of the Neville Island Bridge, fracture of the fascia girder was discovered by a tug boat captain passing underneath the bridge! Meanwhile, the bridge continued carrying traffic and no significant deflections were observed. Based on this case, it is evident that if a member fails and a bridge has adequate capacity against the member failure, traffic will continue to load the bridge.

As previously mentioned, the SRM Guide Spec contains guidelines to calculate, via non-linear, detailed finite element models, the capacity of a steel bridge after the hypothetical failure of a primary tension member. (Typical analysis procedures are not capable of reliably capturing the mechanisms that lead to redundancy without being overly conservative, so finite element analysis is needed to simultaneously consider and evaluate various load paths.) In developing the SRM Guide Spec, much effort was devoted to benchmarking the computational analysis framework against available data from large-scale experimental studies and field data of structures in which a primary steel tension member failed.

The resulting provisions guide engineers through the entire modeling process. Here's how it works: A screening process is used to assess whether the structure is a candidate for the analysis, in order to avoid including structures for which the overall approach would not work—e.g., a suspension bridge—or characteristics that are not reliably implementable in a finite element model, such as pin and hanger assemblies. Then the finite element analysis methodology is explained, including software requirements, analytical procedures, failure scenarios to be modeled for different structure types, and application of loads for the Redundancy I and Redundancy II load combinations.

The guide includes all necessary information for conducting a detailed finite element analysis, including material models for concrete and steel, meshing requirements, application of boundary conditions, and interactions and constraint modeling, as well as detailed provisions to model shear stud behavior. Finally, the guide also includes failure criteria intended to prevent the need for integrating stress data from a finite element analysis with sectional forces and moments.

The SRM Guide Spec opens opportunities for bridge engineers to think outside the box and potentially optimize bridge designs in ways that have been avoided for decades due to a lack of understanding and codified guidance. Furthermore, it provides advantage to owners to more efficiently manage limited resources while maintaining reliability and safety of our infrastructure. ■

*Part One of this series appeared in the November 2019 issue (**www.modernsteel.com**) and discussed historical considerations of redundancy and FCMs. Part Three, which will appear in the April issue, will take a closer look at member-level redundancy.*

above: A close-up of a fracture-critical-designated girder on the U.S. 422 Bridge over Schuylkill River. The bridge continued to carry service loads in the failed condition before the fracture was discovered and repaired.

below: A close-up of a constraint-induced fracture on the former Pennsylvania Railroad two-girder bridge, which is now located at Purdue University's S-BRITE Center. (For more on S-BRITE, see "Wanted: Old Steel Bridges" in the October 2019 issue at **www.modernsteel.com**.)

steelwise

DIPPING DETAILS

BY ALANA FOSSA

A brief look at resources and advice on detailing for hot-dip galvanizing applications.

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GALVANIZING WORKS.

Hot-dip galvanized steel has helped combat steel corrosion in aggressive environments for more than a century, but it continues to evolve as markets emerge and change. Over time, improvements in design and detailing practices for batch hot-dip galvanizing have allowed for superior corrosion protection, optimized aesthetics, lower initial cost, and increased longevity. Reviewing and understanding the most up-to-date steel details and best design practices will help improve the quality and performance of hot-dip galvanized coatings whether specified for long-term corrosion protection, painting or powder coating after hot-dip galvanizing, architecturally exposed structural steel (AESS), fireproofing, and more.

Optimal corrosion protection is primarily achieved by referring to the recommendations provided in the specification ASTM A385 *Standard Practice for Providing High-Quality Zinc Coatings (Hot-Dip)*. This specification outlines recommendations for steel selection along with a variety of design details and fabrication best practices to optimize quality. Because trace elements in the steel chemistry affect the structure and appearance of the galvanized coating, recommended ranges are provided for silicon, phosphorus, carbon, and manganese to achieve a coating of typical appearance and thickness. Steels containing elements outside these ranges (known as "reactive steels") are also successfully galvanized, but produce thick, dark, rough, and/or brittle coatings. The specification also identifies design issues such as overlapping surfaces, different thickness of material in an assembly, moving parts within an assembly, and through-holes, which require special attention if the galvanizing is to deliver a coating according to expectations. Additionally, all designs must consider the need for venting and drainage details such as holes and cropped corners on gusset plates to accommodate the free flow of pretreatment solutions, air, and zinc to achieve a smooth and uniform coating.

Beyond the recommendations in ASTM A385, recent industry research has influenced the design and specification of hot-dip galvanized structural connections. In the 8th Edition of the AASHTO *LRFD Bridge Design Specifications* for Class C slip-critical connections, the requirement to wire brush galvanized faying surfaces is no longer required. This is presently being evaluated for inclusion in the *RCSC Specification for High Strength Bolts*. If needed, slip testing and tension creep testing of zinc-rich paints applied over galvanized faying surfaces have been performed through the American Galvanizers Association (AGA) in accordance with Appendix A of the *RCSC Specification* to achieve improved slip coefficients of 0.45 and 0.50 without impact to corrosion resistance. In the past, there was some concern that galvanizing a connection would cause a standard hole to become small enough that it would be impossible to insert a bolt. The actual zinc coating thickness on a galvanized member can often range from 3 mils to 8 mils. If a member is galvanized the hole may get smaller by up to 16 mils. The standard hole clearance of 1/16 in. is equivalent to 62.5 mils, which is a large allowance for these coatings. AISC's *Specification for Structural Steel Buildings* (ANSI/AISC 360-16, **aisc.org/specifications**) and the *LRFD Bridge Design Specifications* (8th Edition) both include increased standard hole dimensions for nominal


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bolt sizes 1 in. and larger, which will alleviate this perceived concern for bolt hole clearance when galvanizing.

In addition to current industry standards, additional steel details and elevated quality standards are required for AESS to be galvanized. There is a common misconception that it is not possible to obtain AESS-quality galvanized steel because many surface conditions normally acceptable in the primary galvanizing standards (i.e., runs, skimmings, roughness, excess zinc) are not acceptable for showcase or feature elements. To address these concerns, AGA provides supplemental guidance when using the AESS Custom (C) category to facilitate communication regarding additional steel details required to maximize aesthetics for hot-dip galvanized AESS members (for details on the various AESS categories, see "Maximum Exposure" in the November 2017 issue, available at **www.modernsteel.com**). These recommendations include but are not limited to: optimize steel selection with favorable chemistry, use low-silicon welding electrodes, grind thermally cut edges up to $1/16$ in., increase and/or optimize vent and drain hole placement, and provide designated lift points for galvanizing.

above: Improvements in design and detailing practices for batch hot-dip galvanizing have allowed for superior corrosion protection, optimized aesthetics, lower initial cost, and increased longevity.

below: Venting is a crucial step for steel elements that will be put through the galvanizing process, particularly hollow pieces. When moisture trapped inside an element becomes super-heated, it can generate 3,800 psi of pressure and blow a steel piece apart. Galvanizers typically check steel for proper venting before putting it through the process. And in cases where steel isn't vented properly, they contact the fabricator and either have them add venting holes or perform the work themselves on-site using torching or drilling, charging the fabricator accordingly.


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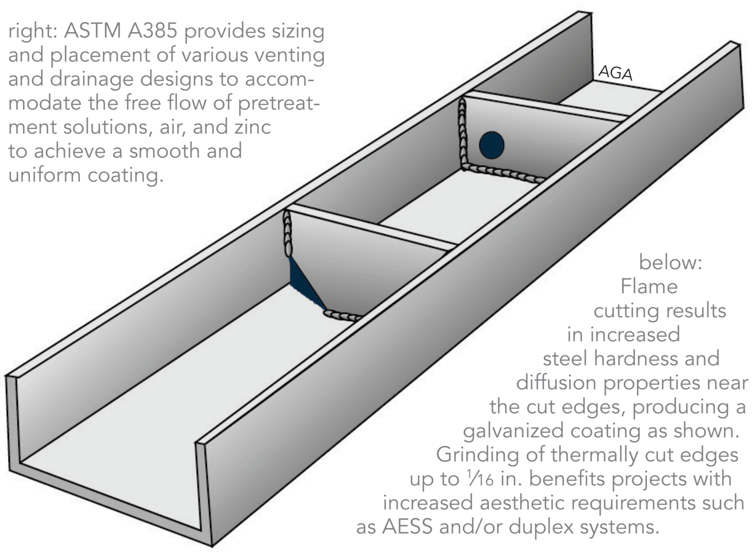
Yann Weymouth, Salvador Dali Museum, 2011

Attention to design details and best practices for duplex systems provided the Salvador Dali Museum in St. Petersburg, Fla., with elevated aesthetics and enhanced measures to protect the steel framing of the outer, artistic glass structure from corrosion.



AGA

right: ASTM A385 provides sizing and placement of various venting and drainage designs to accommodate the free flow of pretreatment solutions, air, and zinc to achieve a smooth and uniform coating.

below: Flame cutting results in increased steel hardness and diffusion properties near the cut edges, producing a galvanized coating as shown. Grinding of thermally cut edges up to 1⁄16 in. benefits projects with increased aesthetic requirements such as AESS and/or duplex systems.

AGA

To achieve a desired color or aesthetic, many projects involving hot-dip galvanized AESS also specify a duplex system, where paint or a powder coating is applied over the zinc coating. ASTM D6386: *Preparation of Zinc (Hot-Dip Galvanized) Coated Iron and Steel Product and Hardware Surfaces for Painting* provides the necessary practices to prepare galvanized surfaces for painting, while ASTM D7803: *Preparation of Zinc (Hot-Dip Galvanized) Coated Iron and Steel Product and Hardware Surfaces for Powder Coating* contains similar practices for powder coating. Many of the same details required for AESS members such as optimized steel selection and venting/drainage details will apply in order to avoid surface conditions that present challenges to coating adhesion.

In a similar fashion, additional details and surface preparations are often required prior to the application of passive fireproofing materials. Where intumescent fire-resistive materials (IFRMs) require a specific primer to promote adhesion over galvanizing, the surface should be prepared identically to a duplex system. On the other hand, when spray-applied fire-resistive materials (SFRMs) are applied over galvanizing, bonding agents or mechanically fastened galvanized metal lath may be required.

Incorporating the above steel details and best practices for hot-dip galvanizing applications can go a long way to ensuring that a coating meets project expectations. In the meantime, industry updates continue to improve the specification and detailing of hot-dip galvanized steel for a variety of industries and uses. ■

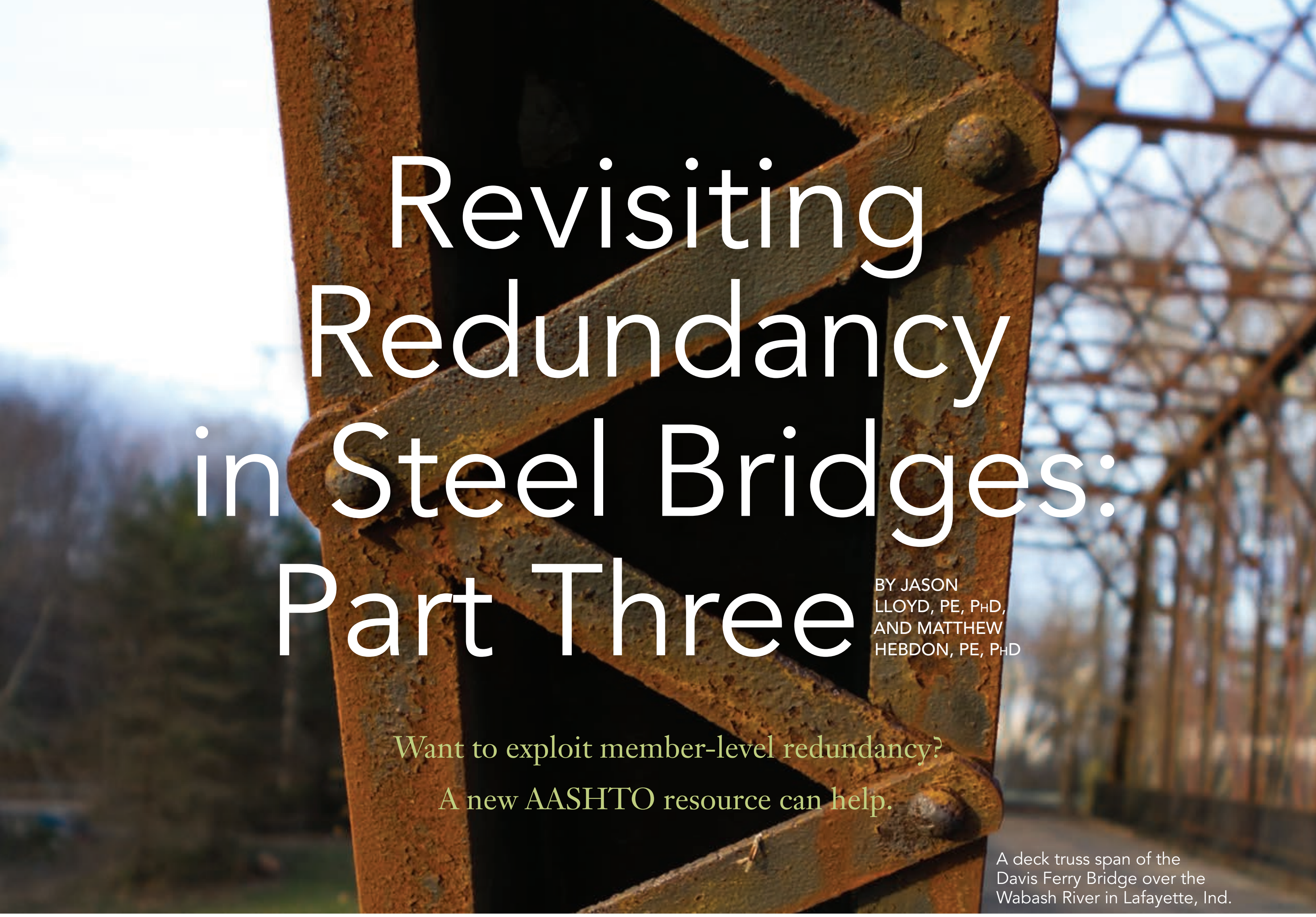
This information will be covered in the presentation "Successful Detailing for Hot-Dip Galvanizing" at the 2020 NASCC: The Steel Conference, taking place April 22–24 in Atlanta. For more information and to register, visit ***aisc.org/nascc****.*

And for more on the hot-dip galvanizing process, see "Galvanizing Illustrated" in the August 2014 issue, available in the Archives section at ***www.modernsteel.com****.*

Revisiting Redundancy in Steel Bridges: Part Three

BY JASON LLOYD, PE, PHD, AND MATTHEW HEBDON, PE, PHD

Want to exploit member-level redundancy? A new AASHTO resource can help.



A deck truss span of the Davis Ferry Bridge over the Wabash River in Lafayette, Ind.

BUILT-UP STEEL BRIDGES have a long history.

Built-up member bridge construction practices using wrought iron can be traced back as far as the late 18th century. From the 1840s onward, construction of long-span wrought-iron bridges in the U.K. continued the advancement of riveted connections and use of built-up member construction. The dawn of rolled steel mills in the late 19th century and early 20th century further advanced the use of built-up construction, making it the most widely used form of building and bridge construction at that time. Hot-driven rivets were predominantly used to fasten together multiple components, such as plates and angles, until the late 1950s and early 1960s, when high-strength bolts and welding processes became preferred methods of construction.

Today, thousands of bridges possessing built-up members continue to serve the highway and railway industries, and many are more than a century old! They remain a vital part of U.S. and international infrastructure, and in many cases have become historic and iconic structures. While built-up construction may not be the most economical design option in current markets, some applications, such as built-up through-girders or built-up steel bents, may be tactically advisable to take advantage of internal redundancy to prevent catastrophic failure.

The new AASHTO *Guide Specification for Internal Redundancy of Mechanically Fastened Built-Up Steel Members* (referred to hereafter as the *IRM Guide Spec*; visit **www.transportation.org**) is a tool to help engineers better understand and leverage internal redundancy when it comes to built-up member structures, and exploit their strength advantages and resistance to failure. The document brings a fresh perspective on how internal redundancy might be exploited in new designs and also provides the industry with a quantitative analysis method for the purpose of showing redundancy and establishing rational inspection intervals for built-up members. The guidelines are realistic about what can be reliably found during inspections and for what duration undiscovered damage may be safely tolerated.

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An approach deck truss span of the former Milton-Madison Bridge over the Ohio River in Madison, Ind.

This, however, must be kept in context when discussing new steel bridges that are designed and built to the standards of the facture-control plan (FCP) which has resulted in *zero* fractures in the past 40 years. In addition, for the past 60 years, no built-up members classified as fracture-critical members (FCMs) are known to have failed due to the fracture of a single member component. Three known cases of component failure in built-up FCMs are the Hastings Tied Arch Bridge (two separate fractures), Milton-Madison Bridge (experimentally fractured for research), and the North Fork Molalla River Bridge. In all three cases, the FCMs did not fail, and the bridges continued to carry service loads until repairs were made. Keep in mind that risk is the product of both likelihood and consequence. The *IRM Guide Spec* helps evaluate the consequence of a member component fracture, conservatively ignoring likelihood and linking damage tolerance to rational inspection requirements.

Designating an FCM is left to the designer or inspector/owner and is currently decided through engineering judgement based on the number of girder lines. This implies that the decision to define a member as an FCM has largely been performed without regard to internal redundancy. The mechanical separation of components within an IRM (internally redundant member) produces an inherent fracture resistance at the component boundaries known as cross-boundary fracture resistance (CBFR). Full-scale experimental research indicates that mechanically fastened built-up members possess CBFR independently of the toughness of the steel. This is a beneficial outcome for owners because a majority of built-up members were fabricated long before the FCP of 1978 began requiring a minimum toughness level. The *IRM Guide Spec* equally applies to new designs and existing members, including all built-up flexural and axially loaded members, as well as members subjected to a combination of flexural and axial loading (e.g., tension ties). Simplified solutions allowing for hand calculations have been developed for a majority of the member types to date, and more will be added in 2020.

The basic steps for analyzing a built-up member for internal redundancy using the *IRM Guide Spec* are:

- Screening criteria such as condition and remaining fatigue life
- Strength limit checks in the assumed faulted condition
- Fatigue life check in the assumed faulted condition
- Selection of a special inspection interval based on fatigue crack growth rates

The provisions of the *IRM Guide Spec* first require the member intended for evaluation to be screened for certain conditions, such as the presence of damage and remaining fatigue life, to ensure a high likelihood of reliable, long-term performance. New and existing members must also meet specified proportioning limits to qualify for this evaluation. Existing members that do not pass the screening criteria should be automatically excluded from further evaluation. The *IRM Guide Spec* is not intended to justify leaving a severed member component in service once discovered, or any other damage that is believed to prohibit reliable service.

Next, the factored load is calculated using the new reliability-based load combination called "Redundancy II" (described in Part Two of this series in the February 2020 issue) and detailed in NCHRP Report 883. Researchers used the same reliability-based procedures to develop Redundancy II that were used to establish the various load combinations of the AASHTO *LRFD Bridge Design Specifications*. Factored loads are used to compute after-fracture stresses for strength and fatigue, assuming that a single component within a member has suddenly failed. For flexural members, the outer cover plate is generally assumed to fail. For axial members, this process is iterated considering failure of a different component each time to find the controlling case, taking advantage of member cross-sectional symmetry. Gross and net section properties are checked for remaining strength in the assumed faulted condition. Laboratory testing and finite element parametric studies have demonstrated that when a member component is severed, localized stress amplifications occur in the adjacent component(s) as a result of load redistribution into and out of the adjacent component. Stress amplification factors are provided in the *IRM Guide Spec* to account for the local stress effects of shear lag and localized bending. For the strength limit state checks, the local amplification has little impact on the global strength. However, because local yielding, slip, and load redistribution allow the section to fully develop the cross section, amplification factors are set to unity. Factored demands are then compared to factored resistance, identical to a typical strength check made during design. If it is found that the member possesses sufficient strength in the faulted condition, the analysis may continue. If not, the member is removed from further analysis (for existing members), or for new designs the member cross section is simply adjusted and reevaluated.

Following strength limit checks, the third step in the analysis is evaluation of fatigue life in the faulted condition. Unlike with strength checks, localized stress amplification of the live load stress ranges must be taken into account when considering the fatigue limit states for the faulted condition. The *IRM Guide Spec* provides simple equations and tables with illustrative cross-section types to help the user determine amplification factors to apply for each case. Fatigue detail categories for members in the faulted condition are provided as well. These were established through full-scale experimental testing of members following failure of a single component. If the member possesses positive fatigue life in the faulted condition, then it has satisfied the provisions of the *IRM Guide Spec*.

The final step is calculation of the special inspection interval. The special inspection process is similar in rigor but replaces the arms-length FCM inspection without changing requirements for routine inspection. The *IRM Guide Spec* includes a methodology to establish the interval for special inspections intended to focus on identifying any tension component that has possibly failed. This inspection of IRMs is referred to as a "Special Inspection," as defined in the *Code of Federal Regulations*, and must be of sufficient depth to reliably detect a severed component. Conceptually, this is a significant departure from the arbitrary, calendar-based, two-year interval intended to find fatigue cracks. The reality is, however, that internal redundancy has been serving us well in our built-up members for well over 100 years.

The *IRM Guide Spec* provides a helpful new tool for built-up steel bridge design and analysis. In addition, NSBA has recently developed a spreadsheet tool that performs the IRM analysis for multi-component axial members. A similar tool is under development for built-up flexural members. These IRM evaluation tools, and many other free and practical design resources, can be found at **aisc.org/nsba/design-resources**. ■

Part One of this series appeared in the November 2019 issue and Part Two appeared in the February 2020 issue. Both are available at ***www.modernsteel.com***.

above: A riveted built-up connection on the Mathews Bridge over the St. Johns River in Jacksonville, Fla., showing built-up tension members.

below: A riveted built-up approach span of the Liberty Bridge in Pittsburgh.

A new report showcases the development of economical and efficient shallow press brake-formed tub girder bridges.

Tub Time


Barry Hatfield II, WVDOH

BY KARL E. BARTH, PHD, GREGORY K. MICHAELSON, PHD, ROBERT M. TENNANT, AND ADAM D. ROH

IN 2009, the Federal Highway Administration (FHWA) challenged the North American steel industry to develop a "cost-effective short-span steel bridge with modular components, which could be placed into the mainstream and meet the needs of today's bridge owners, including accelerated bridge construction (ABC)."

And the Short Span Steel Bridge Alliance (SSSBA) delivered. SSSBA is a group of bridge and buried soil structure industry leaders who have joined together to provide educational information on the design and construction of short-span steel bridges in installations up to 140 ft in length. The group took up the challenge and initiated research into an alternative to prestressed concrete beams for short-span bridge applications. SSSBA's technical working group—consisting of 30 organizations including the American Iron and Steel Institute (AISI), AISC's National Steel Bridge Alliance (NSBA), National Association of County Engineers, steel bridge fabricators, university faculty members, steel manufacturers, government organizations, and bridge owners—developed a solution: a modular, shallow press brake-formed steel tub girder (PBTG). The girder's design is shown in Figure 1.

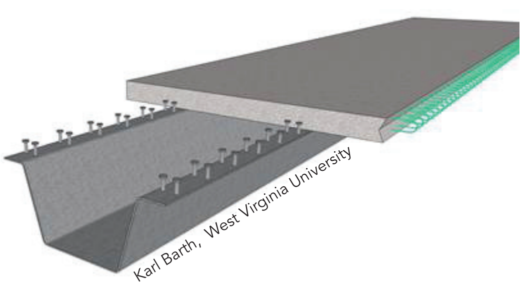
Karl Barth, West Virginia University

Fig. 1. A model of a composite press brake-formed tub girder (PBTG).

The comprehensive research, development, and proof-of-concept efforts were led by West Virginia University and Marshall University. And the complete research study is available in a six-volume report, available at **www.shortspansteelbridges.org**. Following are brief summaries of each volume.

Volume I—Development and Feasibility Assessment of Shallow Press-Brake-Formed Steel Tub Girders for Short Span Bridge Applications. Design of the modular tub girder system was completed in two stages. First, a spreadsheet was developed to compute the section properties of any tub girder configuration. Next, design iterations were performed based on conservative estimates

opposite page and above: Installing the Fourteen Mile Bridge in Lincoln County (District 2) near East Lynn, W.V., a PBTG bridge. Comprehensive research, development, and proof-of-concept efforts for the PBTG design were led by West Virginia University and Marshall University.

of press brake tub girder capacity, limiting the capacity of the composite girders to the yield moment.

In order to verify the performance and capacity of this newly developed modular tub girder, physical testing was conducted at the Major Units Laboratory at West Virginia University. Flexural testing was conducted on simply supported composite and non-composite press brake tub girder specimens in three-point bending. The test load was applied at mid-span using a servo-hydraulic actuator which was mounted to a large structural reaction frame.

Next, two separate analytical tools using nonlinear finite element methods and strain-compatibility procedures were developed and benchmarked against experimental data. Results demonstrate the proposed system is an economically competitive alternative for the short span bridge market.

Volume II—Experimental Evaluation of Non-Composite Shallow Press-Brake-Formed Steel Tub Girders. The originally proposed system consisted of a reinforced concrete deck cast on the girder in the fabrication shop, forming a composite modular

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The Amish Sawmill Bridge in Buchanan County, Iowa, was the first bridge designed, constructed, and opened to traffic using the PBTG concept.

unit once cured. The composite unit would then be shipped to the construction site to be installed. However, the option of implementing a cast-in-place deck was also explored. A critical design stage for these girders occurs during the pouring of the concrete deck, when the non-composite steel section must support the construction load, including the weight of the wet concrete.

Flexural testing was performed on two non-composite specimens to assess the ultimate capacity of the system. Both specimens failed from global lateral torsional buckling. It was also observed that the non-composite girders may be susceptible to torsional amplification due to geometric imperfections. External bracing configurations, which are not required with modular composite units, were recommended for cast-in-place construction. Available system capacity equations agreed with experimental results.

Volume III—Evaluation of Modular Press-Brake-Formed Tub Girders with UHPC Joints. The use of prefabricated bridge elements and systems has led to the recognition that durable connections are the key components in this type of construction. Ultra-high-performance concrete (UHPC), which is a steel fiber reinforced, Portland cement-based product with advantageous fresh and hardened properties, is used for creating robust connections between the prefabricated components. The use of the UHPC as a joint media is becoming more popular during bridge construction.

A model of a bridge system comprised of two composite modular PBTGs connected with a UHPC joint was proposed and evaluated. This was accomplished by constructing two modular units and joining them with a UHPC joint. The system was then fatigue loaded simulating 75-year rural traffic conditions. Experimental results were used to evaluate the reliability of the longitudinal UHPC joint in a composite tub girder system. Results demonstrate the performance of the joint was consistent throughout the test.

Volume IV—Field Performance Assessment of Press-Brake-Formed Steel Tub Girder Superstructures. After several years of lab testing at West Virginia University, the Amish Sawmill Bridge in Buchanan County, Iowa, was the first bridge designed, constructed, and opened to traffic using the PBTG concept. Upon the completion of this bridge, researchers from West Virginia University and Marshall University traveled to Iowa to perform a live load field test.

Live load distribution factors (LLDFs) calculated for each method were nearly identical and displayed how the composite system transferred the various loading between the four girders. Based on the results and conclusions drawn from this research, PBTG bridges exhibit consistent performance and are a practical option in the short-span bridge industry, especially when paired with ABC methods.

Volume V—Fatigue Performance of Uncoated and Galvanized Composite Press-Brake-Formed Tub Girders. The cold-bending of the steel plate into the desired tub girder shape creates residual stresses in the bends of the girder. It was unknown if the high heat of galvanization would affect the residual stresses in the bends of the tub girder.

Laboratory testing was conducted to determine if hot-dip galvanization affects the fatigue performance of a cold-bent shallow PBTG. Two composite steel tub girders were constructed, one composed of an uncoated steel tub and the other composed of a galvanized steel tub. The composite system was fatigue loaded simulating a 75-year life in a rural environment. Experimental results were used to evaluate any difference in the performance of the different steels used in the composite tub girder system. Results demonstrated galvanization did not influence the fatigue performance of the girders and is therefore the recommended means of corrosion protection.

Volume VI—Field Performance and Rating Evaluation of a Modular Press-Brake-Formed Steel Tub Girder with a Steel

Sandwich Plate Deck. The Cannelville Road Bridge in Muskingum County, Ohio, is the second PBTG bridge to be installed in the field. The structure is composed of two modular tub girder and sandwich plate steel (SPS) deck units that were constructed off-site and erected using ABC methods. The main superstructure elements of this bridge were installed in just over 22 minutes. The research team also conducted live load field testing of this structure. (For more on this project, see its description in the 2018 Prize Bridge Awards coverage in the June 2018 issue. And be sure to see the upcoming July 2020 issue, which will feature this year's Prize Bridge Award winners. All issues of *Modern Steel Construction* are available at **www.modernsteel.com**.)

The results of the live load field test and finite element analysis were used to generate bottom flange bending stress, LLDFs, and interior and exterior girder ratings. These values, experimental and analytical, were then compared with equivalent LLDFs, and live-load girder ratings were computed referencing AASHTO LRFD *Specifications*. The results of this testing demonstrated current AASHTO LRFD *Specifications* for analyzing shallow PBTGs are conservative, with field performance exceeding calculated performance.

In addition to high performance, tub girders are practical in ABC applications and compatible with various deck designs as modular units. With a growing demand and need for rapid infrastructure replacement, shallow PBTGs have proven to be an effective application in response to the growing industry demand. They are cost-effective, can remain in service for up to 100 years, and can be installed in far less time than conventional bridges due to the precast nature of the composite deck. ■

above and below: The Cannelville Road Bridge in Muskingum County, Ohio, is the second press brake-formed steel tub girder bridge to be installed. The bridge is a 2018 NSBA Prize Bridge Award winner.

PRIZE BRIDGE AWARDS 2020

2020 Prize Bridge Awards

The art of designing and building beautiful, efficient, economical, and sustainable steel bridges has been practiced for more than a century. AISC announced the first Prize Bridge award in 1928 as a way to showcase the beauty of steel bridges.

2020
PRIZE BRIDGE
AWARDS

AISC AND THE NATIONAL STEEL BRIDGE ALLIANCE (NSBA) are proud to announce the winners of the 2020 Prize Bridge Awards.

The winners span everything from a rugged section of Lake Tahoe's shoreline to a tight Idaho Canyon to a wide stretch of railroad tracks along Chicago's lakefront to a high-profile expressway in Philadelphia's Center City to the Hudson River's massive Tappan Zee. All have made an enormous impact on the lives of the people they serve—some in particularly dramatic ways. For example, the Pfeiffer Canyon Bridge reconnected a California community after a landslide damaged a concrete bridge beyond repair (so much so that groceries and fuel had to be brought in by helicopter!).

"These projects are tributes to the creativity of the designers and the skills of the constructors who collaborated to make them reality," said AISC's president, Charlie Carter. "Steel shines and soars on their talents, and we celebrate the accomplishments these projects represent."

Since Pittsburgh's Sixth Street Bridge won the first competition in 1928, more than 600 bridges of all sizes from all across the United States have received a Prize Bridge Award. Some, such as the Wabash Railroad Bridge in Wayne County, Mich., which won a prize in 1941 and still carries railroad traffic more than 70 years later, have actually outlasted the companies that built them.

Read on to learn about all of the winners. They're also featured in a video at **aisc.org/nsba/prize-bridge-awards**.

Judges

AISC and NSBA would like to thank the 2020 Prize Bridge Award judges for their time and enthusiasm:

- Richard Marchione, deputy chief engineer (ret.), New York Department of Transportation
- Shane W.R. Kuhlman, state bridge engineer, New Mexico Department of Transportation Bridge Bureau
- Frank Russo, vice president and technical director, bridge engineering, Michael Baker International
- Rob Richardson, west region bridge leader, associate vice president, HDR
- Dennis Golabek, GEC-FDOT Structures Design office, WSP

These dedicated judges considered every entry's merits in terms of innovation, economics, aesthetics, design, and engineering solutions.

John Baer Building Images Photography

NATIONAL AWARD Short Span

Vine Street Expressway (I-676) Reconstruction Project—18th to 22nd Streets, Philadelphia

THE VINE STREET EXPRESSWAY is well-known to Philadelphia commuters.

The nearly two-mile stretch of Interstate 676 in the City of Brotherly Love's downtown (aka Center City) is critical to the area's transportation network. But in recent years, six bridges carrying local roads over the expressway were aging and suffering from significant deterioration. The Pennsylvania Department of Transportation (PennDOT) decided to replace these two-span prestressed concrete non-composite adjacent box-beam bridges with single-span welded-plate-girder steel bridges. The project considered vertical clearance issues, reuse of existing bridge abutments, relocation of several utilities supported by the bridges, and high aesthetic standards, including extensive landscaped areas and streetscape finishes atop the new structures.

Each bridge had its own challenges and unique aspects. For example, the deck for the new Family Court pedestrian bridge, located between the 18th and 19th Street bridges, is now a park for the community. This new configuration required that the bridge carry a heavier load to support trees, additional sidewalks and seating areas, and a lawn--a task for thicker flanges. But it still had to be able to flex on the bearing pads on the existing abutment and expand and contract smoothly with temperature changes. Steel was pivotal for supporting the new loads that came with these features while maintaining the clearance needed below the bridge, providing the necessary strength in a shallow profile.

The 19th Street Bridge and the four bays of utilities it supports presented a different challenge. The team prepared a steel design and construction schedule that would allow the utilities to remain in service throughout construction. The utilities were moved to temporary supports while the bridge was removed around them, then the newly fabricated beams were set in place and the utilities were relocated to the new beams while the remainder of the new bridge was built. This reduced the need for outages to move critical utilities and kept them in working order throughout the construction.

Challenging geometry drove the design of the new bridge that would combine the existing 20th Street, Ben Franklin Parkway, and Free Library Bridges into one structure: the 20th/BFP/FL Bridge. Given the sharply skewed geometry (35°) of the Parkway across the bridge, the team investigated whether the design for vehicular live loads could produce larger girder moments and shears running along the sharp skew as opposed to the typical live load configuration of vehicles traveling parallel to the girders. The team developed a 3D finite element model, which confirmed that the skewed live loading condition did not produce effects greater than the standard design vehicular loads running parallel to the girders. The resulting design yielded girders with 24-in.-deep webs and maximum 24-in.-wide by 3.5-in.-thick bottom flanges.

The 22nd Street Bridge posed particular challenges. The clearance below the bridge was too low. There was a pump station behind one of the existing abutments that could not be removed, and the bridge would have numerous existing and proposed utilities. Implementing shallow steel beams eliminated the center pier, raising the profile to the minimum 14 ft, 6 in. without exceeding the capacity of the existing abutments.

The existing concrete 18th Street Bridge carried a heavy 22-in. steam pipe below the deck. The design team worked with the local utility to employ a lighter pipe using less insulation so that the new steel span would be able to not only carry it but also fit it between the bridge beams.

Finally, the 21st Street Bridge had the longest span of all the bridge replacements due to the presence of on/off ramps below the structure, meaning that the abutments had opposing skews of up to 10° from the girder span. As such, each steel girder on this span was unique, resulting in more extensive detailing.

Steel Fabricator and Detailer
High Steel Structures LLC AISC CERTIFIED FABRICATOR, Lancaster, Pa.

Structural Engineer
Pennoni, Philadelphia

General Contractor
Buckley and Company, Inc., Philadelphia

Owner
Pennsylvania Department of Transportation, Harrisburg, Pa.

Bridge Stats

Opened to traffic: November 1, 2018

Span lengths: 18th Street: 95 ft, 2 in.
Family Court: 95 ft, 5 in.
19th Street: 95 ft, 2 in.
20th Street/Benjamin Franklin Parkway/ Free Library: 95 ft, 8 in.
21st Street: 119 ft, 5½ to 133 ft, 10 in.
22nd Street:106 ft, 5 in.

Total lengths: 18th Street: 97 ft, 10 in.
Family Court: 98 ft
19th Street: 97 ft, 10 in.
20th Street/Benjamin Franklin Parkway/ Free Library. 98 ft, 6 in.
21st Street: 120 ft, 3½ in. to 135 ft, 6⅞ in.
22nd Street: 108 ft, 11 in.

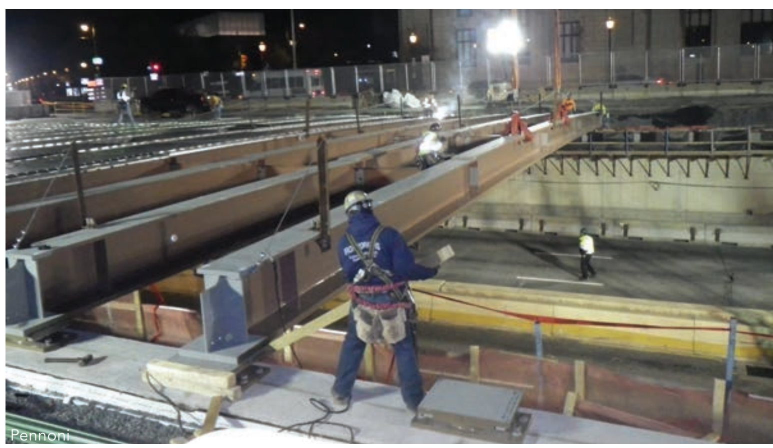
Average widths: 18th Street: 69 ft, 10½ in.
Family Court: 120 ft
19th Street: 64 ft, 11 in.
20th Street/Benjamin Franklin Parkway/ Free Library: 643 ft
21st Street: 67 ft
22nd Street: 83 ft, 6 in.

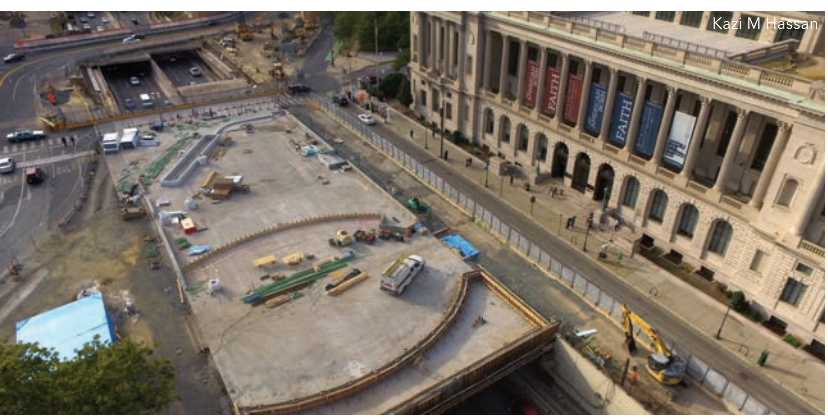
Total structural steel: 2,846 tons

Cost: $65.4 million for entire project

Coating/protection: Three-coat system consisting of an inorganic zinc primer, urethane intermediate coat, and aliphatic urethane finish coat


Pennoni


Pennoni


Kazi M Hassan


Pennoni

TEG Engineering

MERIT AWARD Short Span

Anchor Bay Drive, St. Clair County, Mich.

ANCHOR BAY DRIVE is a scenic road along Lake St. Clair in Clay, Mich., that carries fishing boats and yachts to the marina at the end of the road. Three bridges along the route provide access to the hundreds of homes that take advantage of the spectacular views of the lake and lagoon.

County engineers recently determined that these crossings—prestressed concrete box-beam superstructures with only a 30-year service life—had become either structurally deficient or functionally obsolete. New galvanized steel press-brake-formed tub girder (PBFTG) bridges with a life expectancy two-and-a-half times as long replaced the existing structures. Combined with reinforced precast concrete deck panels, this steel solution provides a cost-effective replacement option at an accelerated construction schedule with a service life expectancy exceeding 75 years.

The St. Clair County Road Commission was able to bundle these three bridges into a collective, successful superstructure replacement project. However, the bridges provide the only point of access to the far reaches of Anchor Bay Drive, rendering a complete tear-down and rebuild impossible. In addition, space around the bridges is extremely tight, with houses packed in close to the roadway and very little dry land to maneuver on.

Luckily, the chosen PBFTG option, TEG Engineering's Con-Struct Bridge System, addressed these issues. The original bridge abutments were in good shape and would not require replacement, and the Con-Struct system can be installed on top of existing substructures. In addition, the system can be delivered two ways: with the precast concrete deck pre-attached to the tub girders, or with it separated. For this project, the team did not want the girders and deck to be attached, due to the space limitations at the installation site.

The county demolished and installed the bridge one side at a time to ensure that traffic flow could continue unhindered. The installation was much quicker than other available options due to the system's modular design. Both the galvanized steel tub girders and the decking took about half a day to set in place. The county's own crew and equipment easily managed installation without additional equipment rentals or labor, saving the county even more time and money.

Fabricator and Detailer

Valmont Industries AISC CERTIFIED FABRICATOR, Valley, Neb.

Structural Engineer

TEG Engineering, Wyoming, Mich.

Owner and General Contractor

St. Clair County Road Commission, St. Clair, Mich.

TEG Engineering

TEG Engineering

TEG Engineering

Bridge Stats

Opened to traffic: July 2, 2019

Span/total length: 57 ft

Average width: 30 ft

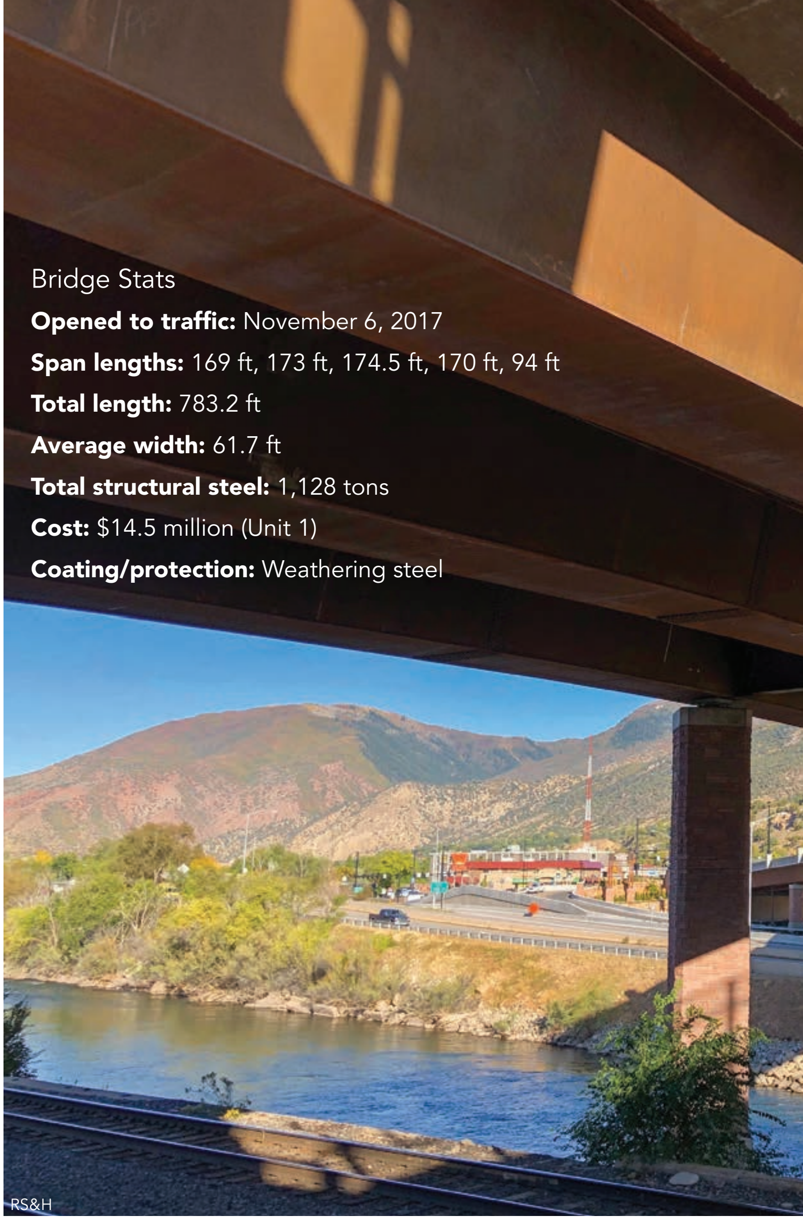
Total structural steel: 58 tons

Cost: $220,000 per bridge superstructure

Coating/protection: Galvanizing

RS&H

RS&H



RS&H

NATIONAL AWARD Medium Span

Grand Avenue Bridge, Glenwood Springs, Colo.

GRAND AVENUE IN GLENWOOD SPRINGS, COLO., has a grand new thoroughfare.

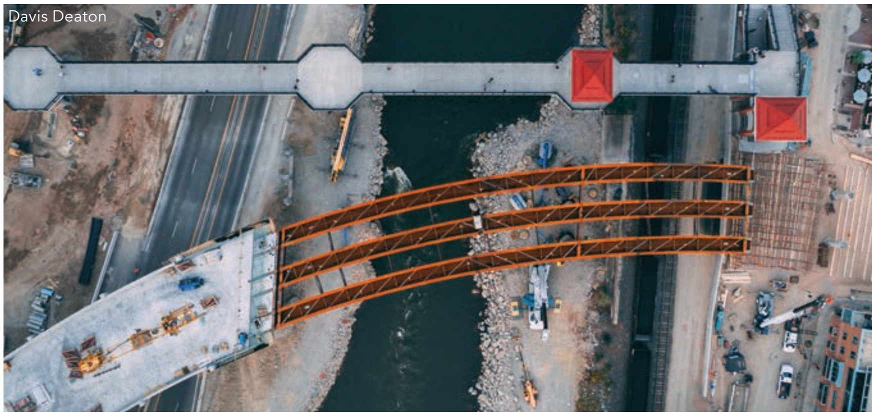
It was driven by the need to replace an aging and functionally obsolete bridge, a nine-span, 676-ft-long steel plate girder bridge constructed in 1953. The bridge carries SH82 over Interstate 70, the Colorado River, and Union Pacific Railroad (UPRR) lines before descending into the historic downtown business district of Glenwood Springs. It is one of only two crossings serving Glenwood Springs as well as other communities along the Roaring Fork River valley, including Aspen to the south.

The existing bridge had four roughly 9-ft-wide lanes that had effectively become a bottleneck to traffic flow. Widening the existing bridge was considered, but the structural capacity didn't meet current codes and there was limited service life remaining, thus making replacement the prudent choice. In addition, the Colorado Department of Transportation (CDOT) was unsuccessful in a previous attempt to replace the bridge due to opposition groups that were ultimately successful in shutting the project down. This time, CDOT made a concerted effort to improve the process by involving the designer, contractor, and public early in the design.

The project included changing the SH82 alignment over the bridge from straight to curved with a 625-ft radius. The new alignment and proposed intersections at the north end improved traffic flow at the SH82/I-70 interchange but made the new bridge geometrically challenging. The horizontal curvature resulted in the bridge crossing I-70, the river, and the railroad at varying degrees of skew. The north end of the bridge was tangent and required a flaring deck width to accommodate the changing lane requirements near the SH82/I-70 interchange. The profile also required a sharp vertical curve to get up and over the UPRR and then immediately begin the descent into downtown.

The new bridge had two distinct regions with significant variation in the required structure depths. A deeper structure of approximately 7 ft was required for the longer spans over the highway, river, and railroad. A shallower structure of approximately 3 ft was required for the shorter downtown spans to allow adequate headroom for a planned pedestrian plaza under the bridge.

For the deeper, steel portion of the bridge (Unit 1), which included the main spans over the Glenwood Hot Springs Pool parking lot, a Frontage Road, I-70, the Colorado River, and UPRR, a five-span



trapezoidal steel tub girder bridge using 6-ft deep girders was selected. A tub shape with sloped sides was the preferred aesthetic for its clean look, while also paying homage to the many steel and concrete tub/box girder structures supporting I-70 in nearby Glenwood Canyon. Tub girders also provided excellent torsional properties to efficiently handle the sharp curvature of the bridge. The tub girder section was optimized by using a narrower bottom flange (5 ft, 7 in. web to web) than had typically been used in Colorado. This, combined with recent enhancements in AASHTO regarding local flange buckling, helped achieve practical bottom flange thicknesses of 2 in. or less without requiring longitudinal bottom flange stiffeners. The increased web-to-web spacing between adjacent tub girders did affect the deck design, but this proved to be relatively inconsequential as compared to an even web spacing.

The number of tub girder lines was reduced from the four girders originally conceived down to three, resulting in fewer members to fabricate and erect and a maximum web-to-web spacing of 18.6 ft at the flared north end of the bridge. In addition, a refined deck analysis resulted in a reasonable deck thickness and reinforcement in this region. The contractor attached a temporary floor beam/stringer system from the tub girders to form the widest deck spans in the flared region. This proved more cost-effective than adding another girder line, which would have been required to accommodate standard deck forming systems in the flared region.

The reduction in girder lines also resulted in increased top flange lateral bracing demands, especially in the flared region. A study comparing Warren and Pratt truss layouts led to the selection of the Pratt truss as most optimal for this bridge. The Warren Truss design would have resulted in larger diagonal member forces in compression, which would have required larger diagonal members and the use of gusset plates at the flange connections. By comparison, the Pratt truss allowed strategic changes in diagonal member orientation to balance the member forces in either compression or tension while mitigating the magnitude of the diagonal member connection forces. The result was reasonable diagonal member sizes and direct connections to the top flange, and no gusset plates were needed.

Steel Team

Fabricator

W&W | AFCO Steel AISC CERTIFIED FABRICATOR, San Angelo, Texas

Detailer

ABS Structural AISC DETAILER, Melbourne, Fla.

Erector

Pioneer Steel, Inc. AISC CERTIFIED ERECTOR, New Castle, Colo.

Structural Engineer

RS&H, Inc., Greenwood Village, Colo.

General Contractor

Granite/RLW Joint Venture, Glenwood Springs, Colo.

Owner

Colorado Department of Transportation – Region 3, Grand Junction, Colo.

MERIT AWARD Medium Span

Williams Creek (Shoup) Bridge, Salmon, Idaho

THE TWO-LANE WILLIAMS CREEK (SHOUP) BRIDGE proves that two is sometimes better than one, as it replaced an existing single-lane river crossing in Salmon, Idaho, with an attractive two-lane bridge.

The original span was a flat compression-loaded bridge that sat on two concrete piers with sheet metal guard rails, and its replacement was architecturally finessed with arched beams for the main frame and tension-loaded with cross cables. The design team performed a fair amount of graphical design work to render the different bridge alternatives it was considering in order to facilitate engaged open houses and public meetings, and the team solicited local residents and business owners for their feedback on the various bridge types and looks. Modeling the different stages of steel erection, deck placement, deck curing, temporary support removals, and cable tensioning was a very involved and detail-oriented process, which allowed the team to accurately capture the cable tension and elastic lengthening and account for all of that elastic deformation in the design of the steel members—so that when everything was completed and all of the loads were on the bridge, the arch resulted in a nice, rounded shape and the roadway profile was at the proper elevation.

The team essentially had to start its analysis with the final product and work its way backwards to determine what shape the arch ribs and tie girders needed to be before they were erected and loaded. "The member lengths and shape of the arch in the final configuration are not the same as the lengths and shapes that get fabricated," noted one project engineer. "For me, that was the most complex part: the level of detail involved in the finite element model we built to determine all of the different loads and deflections anticipated for various support conditions throughout the entire fabrication to erection process."

During the construction phase, increased spring runoff flooded the Salmon River, and general contractor RSCI implemented progressively adaptive construction methods by shifting schedules for in-water work to meet the changing and unexpected water levels and fish

Thompson Metal Fab


Thompson Metal Fab


Don Perkins

spawning seasons. The allowable in-water work windows were tight and because of the historically high-water flows and ice dams, RSCI came up with alternate ways and times to set coffer dams, diversion barriers, and other elements, avoiding excusable schedule delays.

The team employed an Acrow temporary bridge structure for traffic during demolition and construction of the new bridge. The old bridge superstructure was demolished and the new single-span bridge was built using the existing bridge piers as temporary support structures; the piers were later demolished after traffic patterns were redirected onto the newly constructed bridge. This option was provided as a no-cost change order that eliminated the need to completely shut down traffic over the bridge for a period of 48 hours, providing continued use of the bridge during the contracted bridge slide. This method also minimized environmental impact to the river by eliminating the need to install and remove temporary piers required to support construction of the new bridge.

In similar fashion, RSCI implemented an alternate approach for structural steel erection that provided environmental and schedule benefits to the project. This involved designing, installing, and working from a platform that was built directly onto the permanent bridge girders and diaphragms. The work platform was constructed in modular units in the construction lay-down yard and erected along with the girders, allowing immediate use of the structurally supported working area once the substructure steel was installed. This working structure allowed for the use of aerial lifts, materials staging, and manpower to access parts of the bridge that would have otherwise required an additional work platform to be constructed adjacent to the bridge using a pile system, and thus disrupting more of the highly protected Salmon River.

Steel Fabricator
Thompson Metal Fab, Inc. AISC CERTIFIED FABRICATOR, Vancouver, Wash.

Structural Engineer
WSP|Parsons Brinckerhoff, Portland, Ore.

General Contractor
RSCI Group, Boise, Idaho

Owners
U.S. Department of Transportation Federal Highway Administration, Vancouver, Wash.
Lemhi County, Salmon, Idaho

Bridge Stats

Opened to traffic: November 17, 2017

Span/total length: 224 ft

Average width: 32 ft

Total structural steel: 173 tons

Cost: $6.5 million

Coating/protection: Weathering steel

U.S. DOT Federal Highway Administration

Linda Ulery

Don Perkins

NATIONAL AWARD Long Span
Manning Crevice Bridge, Riggins, Idaho

THE MANNING CREVICE BRIDGE carries Salmon River Road across the Salmon River in a picturesque, V-shaped canyon 14 miles upstream from Riggins, Idaho.

Salmon River Road provides access to residences, resorts, commercial rafting ventures and is a main artery for recreational users of the river and forest lands. The existing bridge, built in 1938, had reached the end of its service life and required replacement. The location is remarkable not only due to its beauty but also its limited access and very limited space available to stage construction equipment and materials. The choice of steel for temporary and permanent works was key to developing a feasible erection scheme on this difficult site.

A single-tower, asymmetric suspension bridge was chosen after evaluating six different structure configurations. Competent bedrock at the site provided ample capacity for anchoring large horizontal forces, thus favoring arch and suspension bridge types over cable-stayed options. Given the limited access for construction equipment, a suspension option 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 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, but the hard rock site and pristine canyon location made benching both cost-prohibitive and inappropriate. During design, 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. All construction materials were staged and delivered from Riggins to the north end of the bridge.

Project requirements for the bridge replacement included:

- 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 must allow a logging truck to approach the bridge
- No permanent construction within the 100-year flood plain
- Traffic must be maintained on the existing bridge during construction
- The river must remain open to rafters during construction
- Construction equipment is not allowed in the river
- Reduce the visual contrast of the bridge within the context of the river canyon

Structural steel was integral to the success of the project, especially with regard to treading lightly on the site. The robustness of the erection equipment and temporary crane platform at the north abutment are directly proportional to the piece weights to be erected at mid-span over the river. The light weight of the structural steel sections, combined with the ease of connecting them using high-strength bolted splices, allowed for an erection scheme using only two fixed crane positions with reaches up to 160 ft.


FHWA-WFLHD


FHWA-WFLHD


Ken Saindon

FHWA-WFLHD

FHWA-WFLHD

Bridge Stats

Opened to traffic: January 22, 2018

Span/total length: 300 ft

Average width: 20.1 ft

Total structural steel: 188 tons

Cost: $7,912,900

Coating/protection: Weathering steel

FHWA-WFLHD

Project representatives from the National Park Service were instrumental in identifying key aesthetic concerns, and the bridge deck overlay was designed as an ultra-thin bonded wearing course, with aggregate color that blends with the canyon setting. The bridge deck was cast-in-place concrete using integrally colored, internally cured concrete to enhance long-term durability and reduce visual contrast by providing a color that mimics dark appearance of the weathered granite rock outcrops adjacent to the bridge. The abutments and wind walls were given a surface stain to accomplish the same objective.

The completed structure should last more than 100 years, thanks to its protection scheme. Class C galvanizing was specified for the steel cables, and Grade 50 weathering steel was used for the towers and superstructure, both for corrosion resistance and the aesthetic considerations mentioned above.

The project has been overwhelmingly received by the community, both in terms of local residents and river user groups. The bridge officially opened June 5th, 2018 with a ribbon-cutting ceremony, and many attendees at the ceremony commented on how well the weathering steel finish complements the natural beauty of the canyon. The new single-tower bridge adds a touch of uniqueness to the canyon, with a force layout that reflects the constraints of the site.

For more on the Manning Crevice Bridge, see "Narrow Margin" in the October 2018 issue of Modern Steel Construction, *available at* **www.modernsteel.com**.

Steel Team

Fabricator

Rule Steel, Caldwell, Idaho

Detailer

ABS Structural, Melbourne, Fla.

Erector

Donahue McNamara Steel, Hailey, Idaho

Engineers

Atkins, Denver (structural design and project management)
Horrocks Engineers, Meridian, Idaho (CM/GC advisor and roadway design)
Shannon and Wilson, Denver (geotechnical design)

General Contractors

RSCI Group, Boise, Idaho (also construction manager)
Inland Foundation Specialties, Boise, Idaho (ground anchors and micropiles)

Owners

U.S. Department of Transportation Federal Highway Administration, Vancouver, Wash.
Idaho Transportation Department, Boise, Idaho
Idaho County, Grangeville, Idaho


all photos in this spread courtesy of Caltrans

MERIT AWARD Long Span

Pfeiffer Canyon Bridge, Big Sur, Calif.

RECORD RAINFALL IN THE WINTER of 2016/2017 in Monterey County, Calif., caused several landslides on the scenic coastal State Route 1, which closed the highway.

One of these landslides undermined a support for the Pfeiffer Canyon Bridge and caused severe damage that was beyond repair. The bridge was closed to traffic on February 15, 2017, and its loss devastated a portion of the Big Sur, which effectively became an island between the closed bridge on the north and a large landslide to the south. Groceries and fuel had to be helicoptered into the area. Children were no longer able to attend school located on the other side of the deep canyon. The community, whose main source of income was based on the tourist industry, now had lost its revenue source with State Route 1 closed on either side.

Caltrans immediately contracted with Golden State Bridge to demolish and construct a new bridge, designed by Caltrans, under an emergency force account (EFA). It was quickly determined that a temporary bridge was not feasible at this narrow mountainous site, since there was no room for both it and the permanent bridge as well as the required equipment and staging areas, making the design and construction of the new bridge even more urgent.

A single 310-ft-long composite welded-steel-plate-girder bridge was quickly determined to be the best solution for the replacement of the existing three-span concrete box-girder bridge. Plans for the steel plate girders were provided to the Golden State Bridge in just under two weeks after the damaged bridge was closed to traffic. The plans included two options for the girders: 1) hybrid girders consisting of Grade 50 steel for the top flanges and webs, and Grade 70 steel for the bottom flanges and 2) all Grade 50 steel girders. The latter option was chosen as it involved the quickest delivery when it came to all evaluated bid packages.

The girders were designed to have unstiffened webs to simplify and speed up their fabrication, and the webs were 1¼-in. thick to meet this criterion. The thicker unstiffened webs were also a benefit for launching since the shear resistance of the webs would be constant and not dependent on locations of the transverse stiffeners.

The new bridge width is 40 ft, incorporating three girder lines, and the total structure depth is 14 ft (the steel girders alone are just under 13 ft deep). Each girder line was fabricated in five segments for transport to the site and required four bolted field splices. The largest transported segment was 63 ft long and weighed 56.6 tons, and the girders were shipped to the site laying on their sides and required special Highway Patrol escort due to the width of the load on the narrow two-lane highway leading to the site.

Early on, Golden State Bridge decided it wanted to launch the girders across the canyon, since the girders could not be delivered to the south side of the canyon and erecting all girders from the north side would require a temporary trestle halfway across the deep canyon with an active landslide. Also, some of the temporary erection towers at the girder field splices would have to be located on the landslide.

The girder plans incorporated several details to accommodate the launching. To keep the bottom surface of the bottom flange level and flush for the rollers, the web plate height was varied depending on the flange plate thickness (instead of constant web plate height). Also, the lower field splice plates were redesigned to be three separate plates instead of a single plate so that the middle plate could be left off during launching to allow the rollers to pass though the splice. The existing bridge was on a horizontal curve, and the highway alignment for the new bridge was straightened to simplify the girder details to save design and fabrication time and allow for the girder launching.

To facilitate the launch, temporary pipe supports were constructed on each abutment extending from the seat to just above the back walls, and a central temporary tower was also constructed in the canyon at mid-span. This temporary tower consisted of multiple WACO shoring towers founded on a temporary concrete footing supported by cast-in-drilled hole piles. The approximately 75-ft-tall towers were also guyed at the top. A jacking frame was constructed on the south bank to pull the girders across the canyon using prestressing strands and two 235-kip hydraulic jacks.

All the girders were assembled on the north side of the canyon with a launching nose, and timber soffit formwork for the concrete deck and overhangs was added to the girders while they were being assembled on the launching bed; the catwalks were also installed while the girders were on the launching bed.

The launching plan involved a 14-stage process that included vertical alignment changes to raise the nose up and over the central tower and south abutment supports. The launch took three days following the very controlled and methodical launch plans. As each hydraulic strand jack piston cycled, the girder assembly was pulled in 12-in. to 18-in. increments. After each pull, measurements were taken to check for deflection and alignment to ensure the process was proceeding correctly. This process was repeated again and again until the assembled girders reached the south abutment—and marked the state's first bridge launch.

After the launch was completed, the top portion of the central temporary tower was removed along with the supporting rollers and guides. The girders were then lowered approximately 14 ft onto the abutment seats. The concrete deck was poured and then the see-through bridge railing was constructed. The new bridge opened to traffic on October 13, 2017, just eight months after the existing bridge was closed, reestablishing this vital link to Big Sur and the surrounding communities.

Steel Team

Fabricator

XKT Engineering, Inc. AISC CERTIFIED FABRICATOR, Vallejo, Calif.

Erector and General Contractor

Golden State Bridge AISC ASSOCIATE, Benicia, Calif.

Structural Engineer

Caltrans Structure Design, Sacramento, Calif.

Owner

Caltrans District 5, San Luis Obispo, Calif.

Bridge Stats

Opened to traffic: October 13, 2017

Span length: 310 ft

Total length: 315 ft

Average width: 40 ft

Total structural steel: 809 tons

Cost: $21.7 million

Coating/protection: Inorganic zinc primer undercoat with latex paint finish coat

NATIONAL AWARD Major Span

Governor Mario M. Cuomo Bridge, Westchester/Rockland Counties, N.Y.

THE NEW NY BRIDGE PROJECT produced a crossing of rather epic proportions.

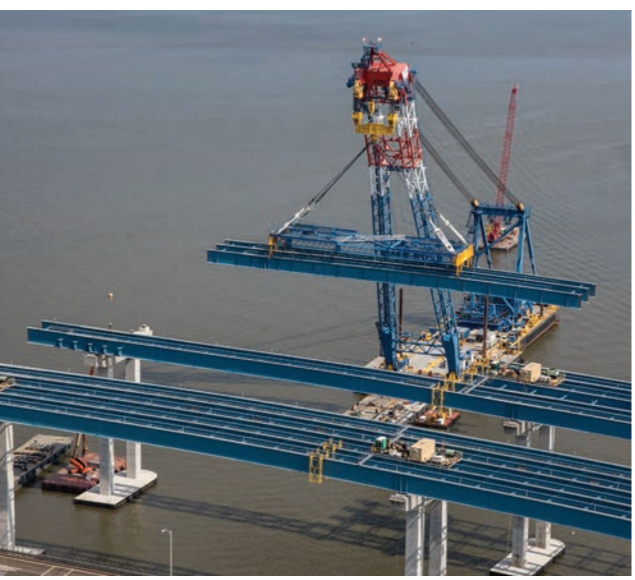
The $3.98 billion undertaking replaced the old Tappan Zee Bridge with the new 3.1-mile-long twin-span Governor Mario M. Cuomo Bridge over the Hudson River, located approximately 20 miles north of New York City. One of the largest-ever transportation design-build contracts in the United States, it is designed for a 100-year service life and carries a new enhanced regional bus service in addition to typical road traffic, and the foundations are designed to carry future commuter/light rail tracks on structures erected between the two spans. The largest bridge project in New York history provides greater traffic capacity while improving operations and safety for motorists crossing one of the widest parts of the Hudson River.

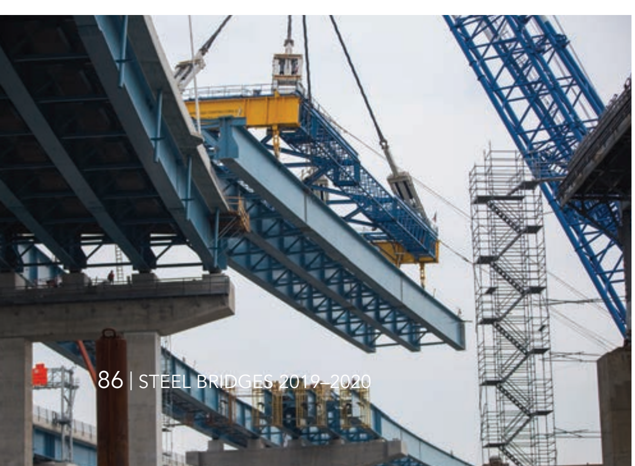
The new bridge features parallel 3.1-mile-long structures, each with a 2,230-ft cable-stayed main span and ten 1,750-ft five-span continuous approach units comprised of 350-ft steel girder spans. It provides eight general traffic lanes, plus dedicated bus lanes and shoulders for emergency access. The design team selected structure types with proven service life and efficiency in order to maximize span lengths and minimize foundation demands while engaging local trade expertise. The approach structure design maximized span lengths using a long-span steel girder sub-stringer system with an average span length of 350 ft, resulting in fewer foundations needed. In the deep clay area, the highest-capacity friction piles (2,100 tons) ever used in these types of soils were implemented and have proven to be successful.

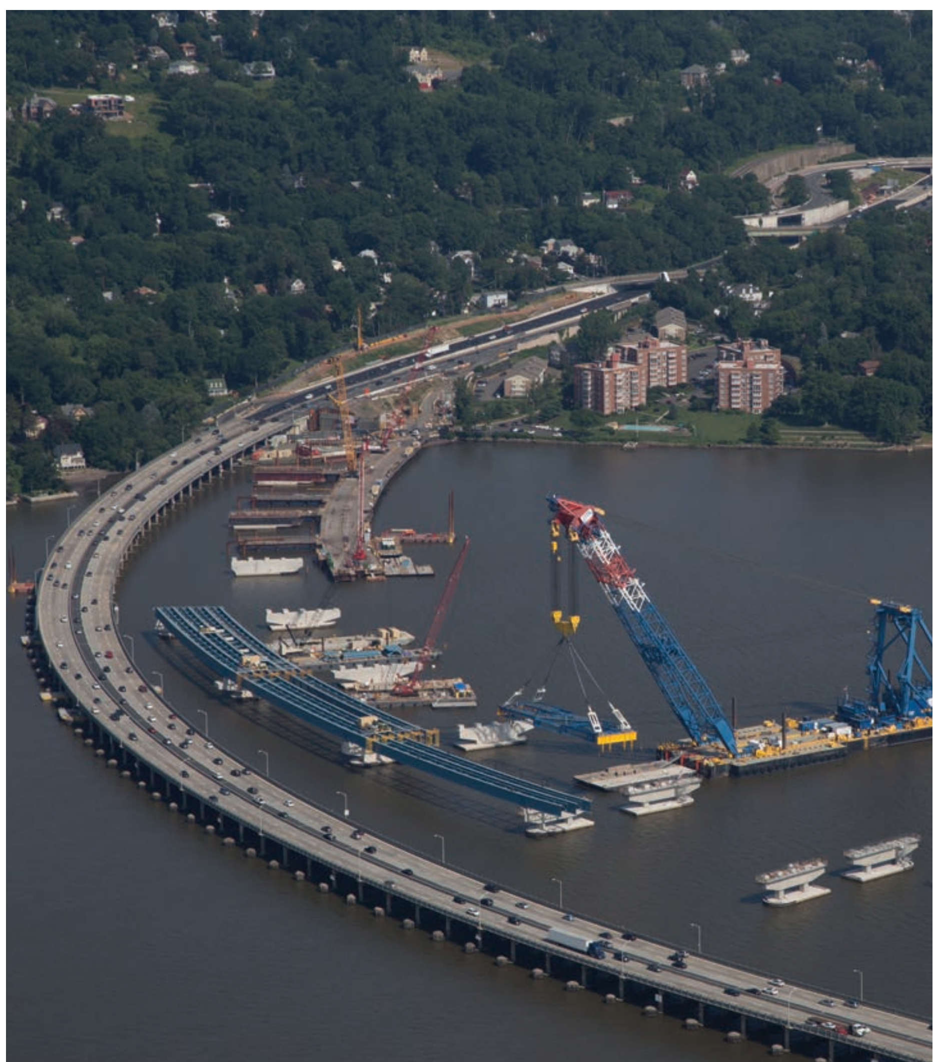
As the lead designer, HDR analyzed, designed, and detailed the approach structure steel girder sub-stringer system, which included composite steel girder design, sub-stringer design, and cross-frame design in accordance with AASHTO LRFD Bridge Design Specifications. 3D finite element models were created to analyze the steel system as a whole and to develop demands for design. Half of the units were located on a curved alignment, which required the design of continuous curved steel girders in which the effects of torsion were considered in both the temporary and permanent state.

Design of the approach spans was based primarily on five-span continuous units. The steel framing supporting each roadway deck included five main girders and four substringers to minimize foundation loads. Overall, 110,000 tons of fabricated structural steel went into the project.

Steel allowed much of the superstructure construction to be modularized. Large picks were made possible by the relatively light superstructure, saving time, minimizing the number of construction activities that needed to occur at elevation, and providing a safer construction process. The light steel superstructure also allowed the team to optimize the pier and foundation designs. Besides minimizing the gravity loads, the seismic demands were minimized by the reduced mass and increased flexibility of the superstructure when compared with other considered structure types. Most of the approach structures are founded on either 3-ft- or 4-ft-diameter steel pipe piles, and the towers, anchor piers, and approach piers adjacent to the anchor piers are founded on 6-ft-diameter steel pipe piles.







The flexibility of a steel superstructure was also highlighted in a portion of the site over Metro-North Railroad tracks where crane access was limited. HDR worked with the contractor to develop a steel girder system that could be launched from the Westchester abutment in multiple phases overnight during track outages. The designer worked hand-in-hand with the erection engineer up front to ensure that the design could accommodate variations in loading during launching activities, which minimized changes during the fabrication process.

Steel Team

Fabricators

High Steel Structures LLC AISC CERTIFIED FABRICATOR & ERECTOR, Lancaster, Pa.
(approach unit superstructure, also detailer)
W&W | AFCO Steel AISC CERTIFIED FABRICATOR, Greensboro, N.C.
(approach unit superstructure, also detailer)
Canam-Bridges, Point of Rocks, Md.
(main span superstructure)
L&M Fabrication and Machine, Inc. AISC CERTIFIED FABRICATOR, Bath, Pa.
(main span superstructure)

Additional Detailer

Tenca Steel Detailing, Inc. AISC DETAILER, Quebec

Structural Engineer

HDR, New York

Design/Builder

Tappan Zee Constructors, LLC, a joint venture of:
Fluor, American Bridge Company AISC ASSOCIATE AISC CERTIFIED ERECTOR,
Granite Construction Northeast, and Traylor Bros., Inc. AISC ASSOCIATE

Owner

New York State Thruway Authority, Albany, N.Y.

Bridge Stats

Opened to traffic: September 1, 2018

Span lengths: Two parallel three-mile structures, each with:

- Unit 11 WB/EB: 2,230-ft cable-stayed unit comprised of a 1,200-ft main span and two 515-ft anchor spans
- Unit 1 WB/EB: 388-ft two-span simply supported approach unit comprised of 116-ft and 272-ft spans, respectively
- Unit 2 WB/EB: 1,000-ft three-span continuous approach unit comprised of spans varying between 309 ft and 350 ft
- Unit 3 WB/EB through Unit 8 WB/EB: Six 1,750-ft five-span continuous approach units comprised of 350-ft steel girder spans
- Unit 9 WB: 1,075-ft three-span continuous approach unit with spans varying between 345 ft and 365 ft
- Unit 9 EB: 1,666-ft five-span continuous approach unit with spans varying from 301 ft to 354 ft with a simple 224-ft jump span at the end
- Unit 10 WB: 745-ft three-span continuous approach unit with spans varying from 235 ft to 262 ft

Total length: 3.1 miles (16,368 ft) per bound

Average width: Westbound: 96 ft; Eastbound: 87 ft

Total structural steel: 110,000 tons (including steel pipe piles)

Cost: $3.98 billion

Coating: Painted weathering steel for the superstructure, galvanized rebar and specific coatings and overlay for the concrete deck

photos in this spread courtesy of New York State Thruway Authority

MERIT AWARD Major Span

Broadway Bridge Over the Arkansas River, Little Rock/North Little Rock, Ark.

THE ORIGINAL BROADWAY BRIDGE served the communities of Little Rock and North Little Rock, Ark., for over 90 years as both a vital crossing and a signature tribute to World War I veterans.

Built in 1922, the bridge carried nearly 24,500 vehicles into the downtown area every day. However, with the continuing trend of residential redevelopment in the two cities' downtown areas, the increasing need for safe and efficient crossings of the river became more apparent. In 2010, the Arkansas Department of Transportation (ARDOT) made the decision to replace this functionally obsolete bridge due to it being structurally unsound as well as the lack of mobility it provided for the growing population in the area. The team of HNTB Corporation and Garver, LLC, was chosen to design the replacement bridge in 2011.

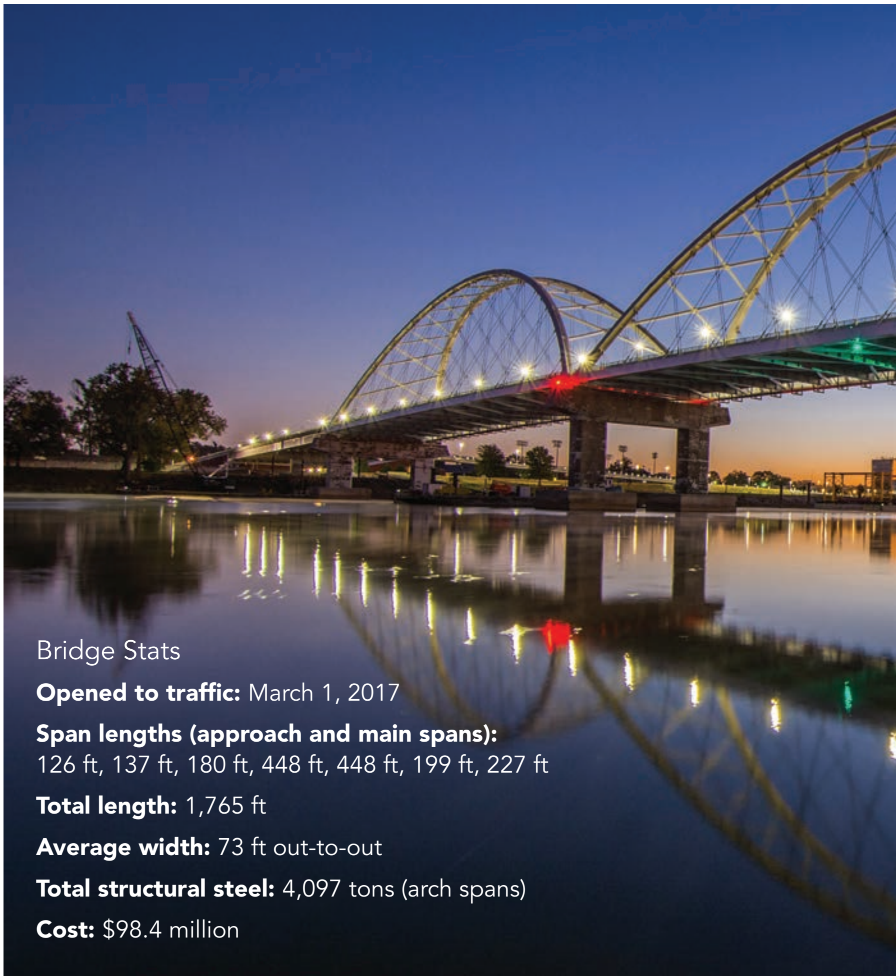
Garver developed a new layout to address the current traffic needs while increasing safety for the traveling public. Garver was responsible for improving sight distances, as well as separating motorists and pedestrians through the addition of a 16-ft-wide shared-use path, two new pedestrian-only ramps connecting the trails directly to this path, and MSE walls to reduce right-of-way impacts and overall bridge length.

Pulaski County leaders wanted the bridge to serve as a unique and pleasing experience for pedestrians and cyclists by enhancing the aesthetics of the bridge, and they contributed $20 million of the $98 million total project cost to be spent toward two signature spans over the river. These funds allowed the design to possess an enhanced aesthetic form constructed in an accelerated fashion and using a limited budget to satisfy the current and future needs of the community.

The HNTB-designed main spans of the Broadway Bridge are composed of two 448-ft network tied-arch spans with steel plate girder approaches. The lengths of the five approach spans vary from 126 ft to 227 ft. The final design consists of inclined basket-handle arches with a framed-in floor system, which lowered costs. The tied arches allowed a signature structure to be constructed on the existing alignment ahead of the anticipated 180-day bridge closure by using an accelerated bridge construction (ABC) technique to float the arches into place.

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

Trey Cambern, courtesy of HNTB



Bridge Stats

Opened to traffic: March 1, 2017

Span lengths (approach and main spans): 126 ft, 137 ft, 180 ft, 448 ft, 448 ft, 199 ft, 227 ft

Total length: 1,765 ft

Average width: 73 ft out-to-out

Total structural steel: 4,097 tons (arch spans)

Cost: $98.4 million

HNTB


HNTB


HNTB


Greg Davis, courtesy of Massman


Greg Davis, courtesy of Massman

The web plates are welded to tab plates with a double-fillet weld and 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 for a catastrophic structural failure.

The construction of the arches took place on falsework floating in the river moored to the north bank of the Arkansas River. This technique provided extra space for the contractor to work within a limited construction footprint for such a large urban project. To minimize the closure period during construction, the bridge's new foundations were strategically placed to provide clearance from the existing foundations. This allowed the contractor to use specialized equipment to construct the new drilled shafts and waterline footings beneath the existing bridge while the bridge remained open to traffic. The new tied-arch structure was floated into place once the primary structural steel framing was erected. This ABC process required only two 24-hour river closures.

Using these techniques, the team was able to open the $98 million structure to vehicular traffic after 2.5 years of construction on March 1, 2017, having removed 28 days from the anticipated 180-day closure period.

For more on the Broadway Bridge, see "Making a Signature Connection" in the July 2017 issue of Modern Steel Construction, *available at* **www.modernsteel.com**.

Steel Team

Fabricators

Veritas Steel, Palatka, Fla.
W&W | AFCO Steel, Little Rock, Ark.
Delong's, Inc., Jefferson City, Mo.
(also detailer, south approach)

Detailers

Tensor Engineering, Indian Harbour Beach, Fla. (arch spans)
ABS Structural, Melbourne, Fla.
(north approach)

Structural Engineers

HNTB, Kansas City, Mo.
Garver, North Little Rock, Ark.

Prime Contractor

Massman Construction Co., Overland Park, Kan.

Owner

Arkansas Department of Transportation,
Little Rock, Ark.

Bridge Stats

Opened to traffic: December 11, 2017

Span length: 483 ft

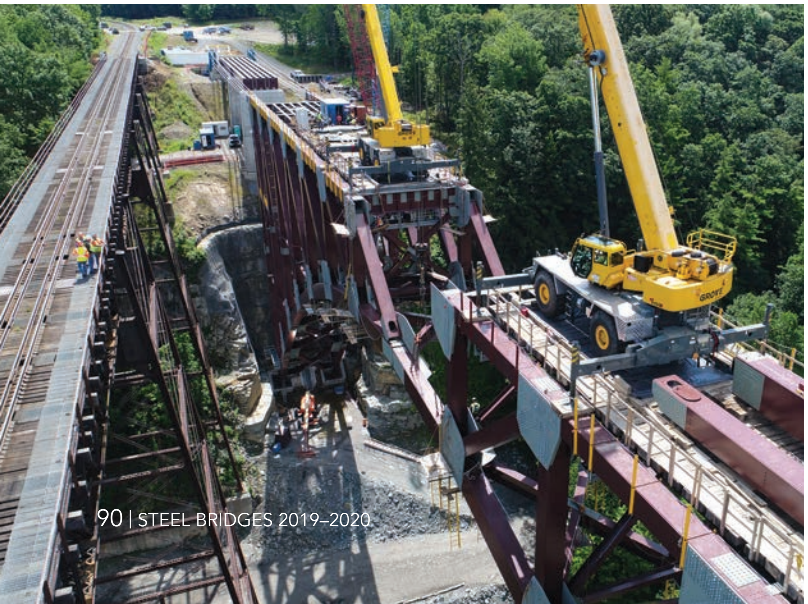
Total length: 963 ft

Average width: 22 ft

Total structural steel: 4,300 tons

Cost: $68 million

Coating/protection: Paint



all photos and graphics in this spread courtesy of Modjeski and Masters

MERIT AWARD Major Span

Portageville Bridge Replacement, Portageville, N.Y.

THE NEW PORTAGEVILLE BRIDGE had big shoes to fill, so to speak.

The original bridge crossed the scenic Genesee River Gorge, known as the "Grand Canyon of the East," in Letchworth State Park in Portageville, N.Y., which hosts more than a million visitors a year thanks to its stunning scenery, including three large waterfalls. The new bridge, adjacent to where its predecessor once stood, is located directly above the Upper Falls.

Built in 1875, the old viaduct bridge was considered iconic within the Park and it was expected that a new bridge would need to be as well. After nearly a decade of public meetings, stakeholder input, environmental study, and engineering analysis, the team determined that the new bridge would be a spandrel-braced arch. Nine different options went through an evaluation process defined by New York's State Environmental Quality Review Act, which considered the project objectives and the site's unique characteristics. Ultimately, the team concluded that removing the existing bridge and building a new bridge on a parallel alignment would be the best option.

The selected design is the first true arch bridge built for the rail industry since the late 1940s. Modjeski and Masters (M&M) led the structural design of the new 483-ft-long arch. The arch is flanked on both sides by three 80-ft-long welded girder spans, and the track is supported across the bridge with a 20-ft-wide concrete ballast deck. The welded girder spans are supported on reinforced concrete piers and abutments that are founded on micropiles.

The bridge's span exceeded the guidance provided by the American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering, which is primarily used on simple-span bridges less than 400 ft in length, and thus required project-specific design criteria. The arch was erected in two halves, from the east and west skewback foundations, using the cantilever method. An "arch tieback system" was designed to support each arch half during cantilever erection up until arch closure. Each tieback system tied into the gusset plate at the end of top chord of the arch, and then anchored into a guy tower and backstay system with 12 cables. The guy towers transferred cable demands to a series of back stay members and directed the vertical components into the permanent approach span abutment. The backstays were connected to a grillage system anchored by 140-ft-long pretensioned rock anchors.

Each individual cable was connected to a tensioning device equipped with a jacking rod and center-hole jack, which was used to adjust the cable lengths and thus the arch geometry during erection and arch closure. The deflection of the arch and the tension in the tieback system cables were monitored throughout cantilever erection stages. Field-recorded values were compared to theoretical values obtained from a staged construction analytical model to ensure the arch closure geometry was eventually achieved. At the arch closure stage, the geometry for each half was fine-tuned using the tieback system until the bolt holes in the lower center panel point were aligned.

The gorge walls had an irregular shape and were not easily accessible. The difficult terrain would have made conventional surveying methods difficult, so the team used lidar scanning to make a preconstruction survey of the gorge walls. This preconstruction survey was used for placement of cranes and the determination of lifting radius. An additional lidar scan verified excavated quantities after the gorge pockets were completed.

The AREMA guideline for spacing trusses at 1/15 of span length was not followed, due to the unnecessary width that would be added due to the long span. The structure was proportioned such that no load combination produced uplift, except for a few combinations during construction staging. Plate thicknesses of box members were sized to preclude the need for longitudinal stiffeners. The main members were designed including in-plane and out-of-plane bending moments. As many of the applied loads can be multi-directional and thus cause moments to change direction, a conservative assumption was made to combine them in an additive manner and match the polarity of the axial loading under investigation.

A memorandum of agreement between the Federal Highway Administration; Norfolk Southern; New York State Department of Transportation; the New York State Office of Parks, Recreation, and Historic Preservation; the National Park Service; and various Indian Nations was created to produce a mutually agreed plan to avoid, minimize, or mitigate the impacts on various historic and cultural resources. The agreement stipulated that portions of the existing bridge would be salvaged and displayed to mitigate the removal of the bridge. A construction protection plan avoided impacts on other historical resources, and additional plans protected endangered species, such as northern long-eared bats, timber rattlesnakes, and bald eagles.

Steel Team

Fabricators

Canam-Bridges, Point of Rocks, Md. (arch bridge)
Veritas Steel, LLC, Eau Claire, Wis. (approach deck girder steel spans)

Detailer

DBM Vircon Services, Port Coquitlam, B.C., Canada

Steel Erector and General Contractor

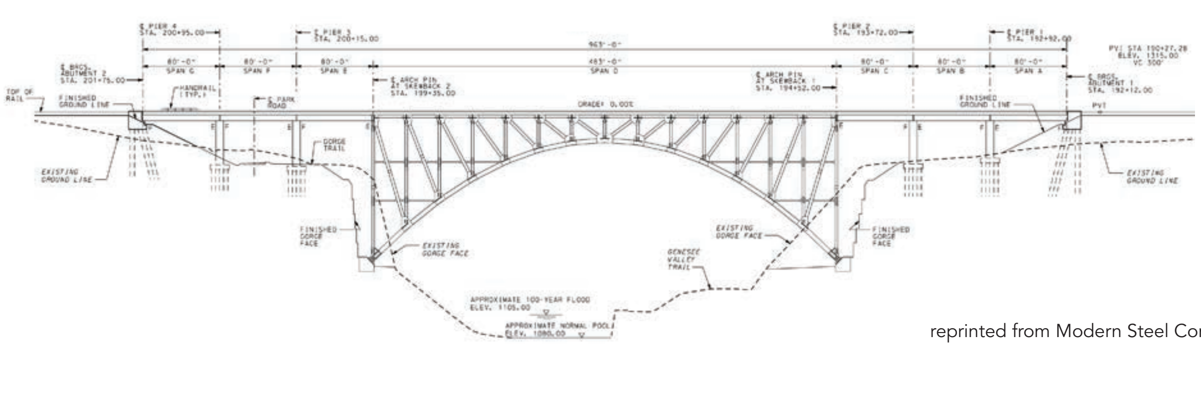
American Bridge Company, Coraopolis, Pa.

Structural Engineer

Modjeski and Masters, Mechanicsburg, Pa.

Owner

Norfolk Southern Corporation, Atlanta



NATIONAL AWARD Movable Span

Sarah Mildred Long Bridge, Kittery, Maine/Portsmouth, N.H.

THE NEW SARAH MILDRED LONG BRIDGE across the Piscataqua River between Portsmouth, N.H., and Kittery, Maine, replaces an existing span built in 1940.

Where the original bridge involved a bi-level lift span and approach bridge format, the new incarnation is a single-level lift span with bi-level approach spans. Both new and existing structures were designed to carry vehicular traffic (on the upper level) and rail traffic (on the lower level), with the new single-level lift span lowering for rail traffic and raising for maritime vessels.

The project is a complete bridge replacement including foundations, an operator's room, new traffic warning systems, a new 300-ft-long steel box girder lift span, and precast post-tensioned towers and vehicular and railroad approach segments. The team contended with several challenges, such as minimizing construction costs and construction time, a swift tidal channel with a current of approximately 5 knots and a tidal change of 8 ft, and a design vessel collision force of 6,000 tons.

On the lift span itself, the rail and roadway are on the same level, with the tracks are embedded in the median. Dual seating positions (vehicular and rail) allow the single-level lift span to match the bi-level approaches. Because the new bridge has a 56-ft vertical clearance when in its "resting" position (an increase in vertical clearance from the original configuration) there will be 68% fewer bridge openings than with the old bridge, significantly reducing the number of traffic delays. The lift span is simply lowered down to match up with the railroad bridge approaches on the relatively rare occasion when trains travel across the river.

The lift span superstructure uses a traditional twin steel tub girder design with a continuous top plate to facilitate shipping to the site by truck. This allowed the final configuration of the lift span to be fabricated at local inland facilities then assembled on-site, reducing the construction schedule and planned existing bridge closures.

The lift span girder is a multi-box steel structure with a composite concrete deck. Based on the length-to-width ratio of the structure, the entire cross section is effective in resisting global forces. Two main boxes with separate bottom flanges, two fascia box beams, and a composite concrete deck are the primary longitudinal load carrying members. In addition to contributing to the overall cross section, the composite deck is designed to transmit local loads transversely to the main longitudinal elements. Longitudinal elements are braced at discrete points along the length of the span at 12-ft increments. Transverse elements include cantilever brackets between fascia boxes and main boxes, internal box bracing, and intermediate diaphragms along the centerline of the span between main boxes, and the lift span girder is supported at each end by transverse lifting girders.

The main boxes are aligned such that the interior webs are located directly below each rail track. The track is embedded

all photos in this spread courtesy of Maine DOT

Bridge Stats

Opened to traffic: March 30, 2018

Span length: 300 ft

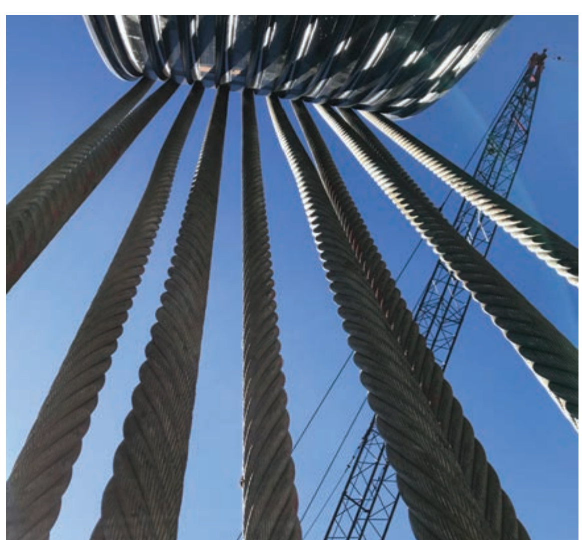
Total length: 2,800 ft

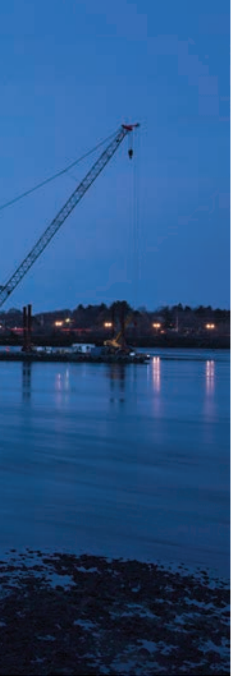
Average width: 42 ft, 7 in.

Total structural steel: 1,235 tons (lift span)

Cost: $163 million

Coating/protection: Metallized





within the concrete deck, with minimal cover to the top of the steel, and the design team implemented a direct load path into the box section. In addition to providing a predictable load path, this alignment eliminated the need for supplemental track support structures and ultimately reduced the span weight.

An innovative retractable support system was developed to support the lift span at the mid-level roadway position and move out of the way to allow the lift span to lower to the rail position. Tapered steel columns founded on spherical bearings at the rail level and cylindrical bearings at the electrical room under the roadway level rotate to allow for the dual seating of the lift span.

The fatigue critical areas of the structure are primarily located along the top flange plate when subjected to transverse loading. Fatigue analysis of the deck plate required an increased plate size along the centerline of the span, below the track and extending beyond the interior web plates. Deck plate details in the longitudinal direction are not a fatigue concern, as the flange always remains in compression.

Placing the operating machinery at the base of the tower is an innovation that is relatively recent to the movable bridge industry—and one that was implemented on the new Sarah Mildred Long Bridge. The lifting machinery, mechanical systems, and electrical systems could all be installed before completing tower erection and lift span float-in because they are placed lower in the tower. This provided for quicker construction, reduced initial costs, and allows easier access for future maintenance.

The lift span box girders and other lift span steel components were fabricated at Casco Bay Steel Structures in South Portland, Maine before being sent by rail to a waterfront facility and barged to the bridge site. Float-in was a complex operation that required a fixed guide barge, an adjacent push barge with two tugs, and a lift span overhanging barge. Several important steps followed the float-in, including deck placement, joint installation, finger joints, mitre rail, span guides, access, and rope connection.

The bridge was designed with long open spans, using 11 fewer piers than the old bridge. This span layout not only enhances vistas for residents and motorists, but it also enabled the new bridge to cross Market Street without a pier in the median. The new bridge serves as a gateway entrance into historic downtown Portsmouth.

Steel Team

Fabricator

Casco Bay Steel Structures, Inc. AISC CERTIFIED FABRICATOR,
South Portland, Maine

Detailer

Tensor Engineering AISC DETAILER, Indian Harbour Beach, Fla.

Steel Erector and General Contractor

Cianbro AISC ASSOCIATE AISC CERTIFIED ERECTOR, Pittsfield, Maine

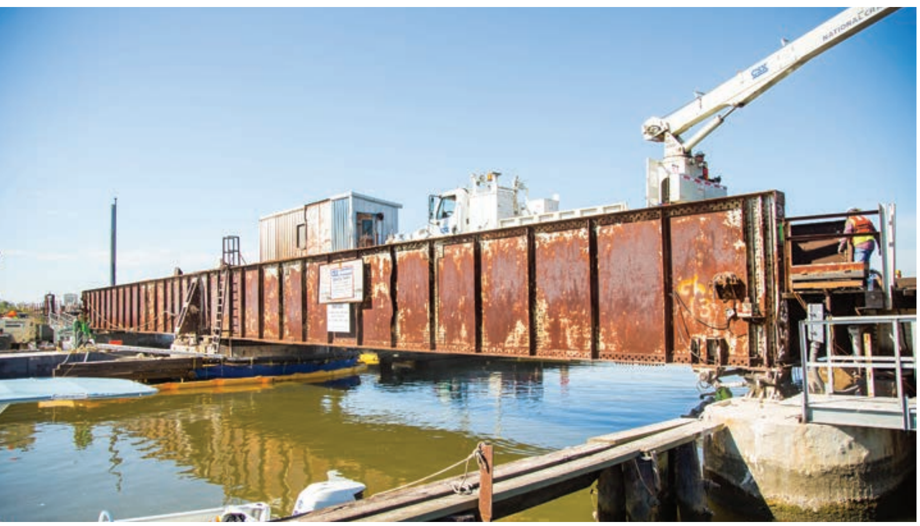
Structural Engineer

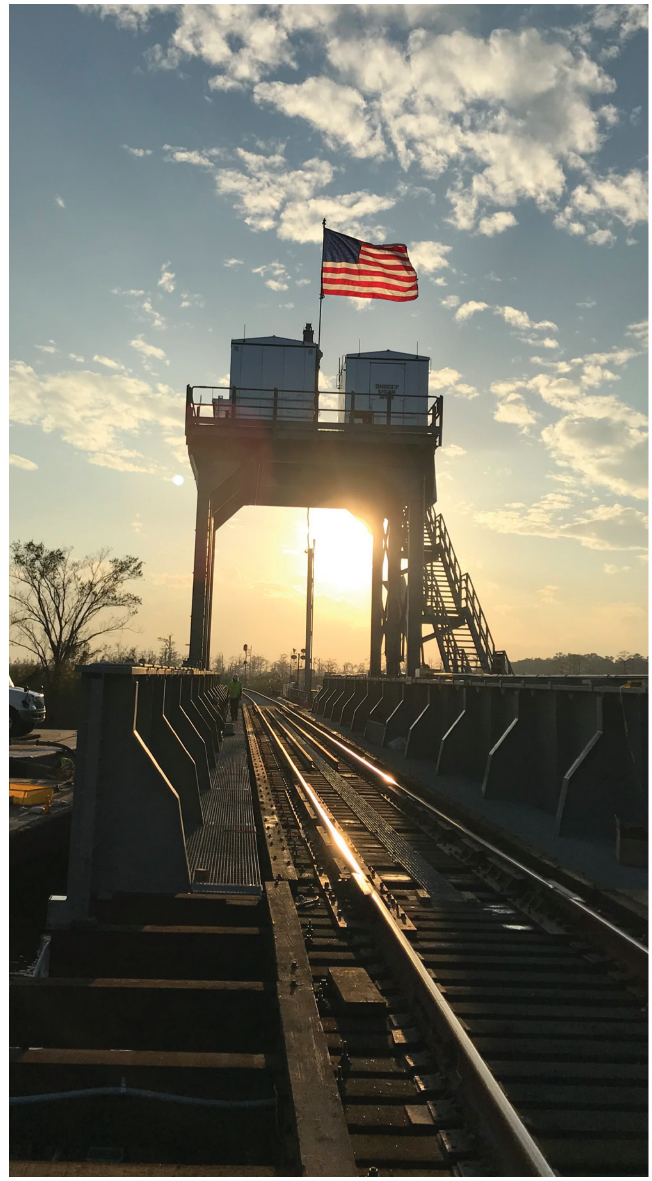
Hardesty & Hanover, LLC, New York

Owners

Maine Department of Transportation, Augusta, Maine
New Hampshire Department of Transportation,
Concord, N.H.

all photos in this spread courtesy of HDR





MERIT AWARD Movable Span

Bayou Sara Swing Bridge, Mobile County, Ala.

CSX'S SINGLE-TRACK, 163-FT-LONG Bayou Sara Swing Bridge is one of the rail transportation company's 47 movable bridges.

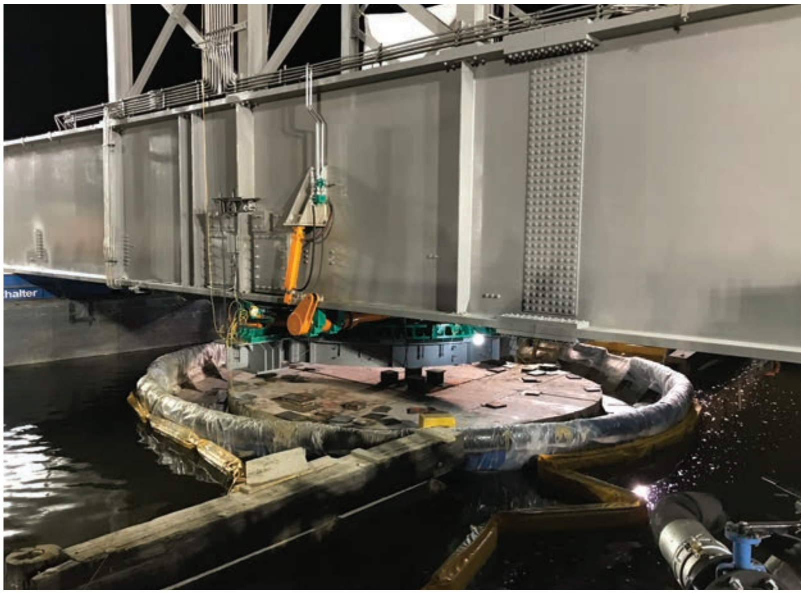
While the approach spans had been recently replaced, the swing span was over 90 years old and was scheduled to be replaced as part of a program to upgrade all of CSX's movable bridges. To replace this critical link on the company's Mobile Bay line, CSX turned to HDR to design a durable replacement with remote operation, minimized maintenance, and limited rail service interruption during construction. An in-kind replacement allowed the team to reuse the substructure, simplifing construction, speeding up the schedule, and reducing permitting requirements and track outages.

During hurricanes or lunar high tide, it was common for the water to rise above the bottom flange of the girders of the old bridge, inundating the bridge machinery with brackish coastal water. Because the bridge approaches could only be raised minimally, the replacement bridge incorporated features that mitigated the effects of high water inundating the lower part of the bridge.

The team placed the electrical components, hydraulic eqipment, and control systems on a gantry 28 ft above the track to remain above the water even during the worst of storms. An outboard walkway and stairway provide access to the platform, away from the track. In addition to improved security and environmental resilience, the platform allowed the team to rebalance a crucial counterweight.

The mass of a swing span must be balanced for proper operation. The control houses for many swing spans, including the old Bayou Sara Bridge, are mounted to a platform along the span edge, near the pivot. This requires a counterweight on the opposite girder to transversely balance the span. Adding the platform to the design allowed the team to reduce the counterweight steel by 20 tons.

Given the challenges, collaboration was critical to project success. The decision to proceed with the grillage concept was ultimately made in September 2017, just two months prior to the target float-in date. This limited the schedule for detailed design, procurement, fabrication, and assembly. When the grillage concept was first discussed,



general contractor Brasfield and Gorrie immediately contacted the steel fabricator, Steward Machine, to discuss constructability and material availability. Steward provided feedback on available structural shapes, which were approved. This collaborative effort expedited shop drawing development and engineering review, which was crucial to procuring the grillage in time for installation prior to the float-in.

From the beginning of the project, CSX's freight rail operations team allowed a 48-hour rail outage, which is a challenging window for removing a movable bridge span and installing a new one. During the construction phase, the team developed a plan to swap out the spans within this time frame, using a precast concrete pier cap to simplify construction and replace the deteriorated concrete cap.

However, as the planned outage drew near, CSX asked if the outage could be reduced so as to avoid delaying trains. The team considered several options, including temporary piles, which would have added significant costs to the project. In the end, the collaborative efforts between the owner, contractor, and engineering teams concluded that the most cost-effective solution was a structural steel support frame (grillage) suspended from the new swing span with pre-mounted rack, wedges, and pivot bearings. This steel grillage took the place of the top portion of the pivot pier, which was removed during construction. The grillage allowed the bridge machinery and bearings to be aligned and locked in their final position prior to float-in. It also provided support for all dead and live loads applied to the pivot pier, permitting rail traffic to pass almost immediately after the span float-in. The outage for marine navigation was longer than for railway traffic. This gave the team time to cast the surrounding concrete in place after the float-in phase, prior to operating the swing span.

Careful planning and pre-work paid off in the form of an accelerated swap-out of the swing spans, reducing the required track outage to only 14 hours.

Steel Team

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

Erector and General Contractor
Brasfield and Gorrie, Birmingham, Ala.

Structural Engineer
HDR, Newark, N.J.

Owner
CSX Corporation, Jacksonville, Fla.

NATIONAL AWARD Special Purpose

Frances Appleton Pedestrian Bridge, Boston

THE FRANCES APPLETON PEDESTRIAN BRIDGE project achieves visual transparency and lightness through a carefully selected structural steel system as it connects Boston's Beacon Hill neighborhood to the Charles River Esplanade.

Designers had to balance the slenderness of the bridge against creating a structure that would potentially have issues with pedestrian-induced vibrations. During the design process, multiple iterations of the structural system were evaluated to achieve the maximum comfort range for pedestrians while eliminating the need for future supplemental measures, such as installing tuned mass dampers. The final design includes the creative use of a lightweight concrete deck with foam-filled, stay-in-place forms and appropriate foundation details.

The 750-ft-long multiuse walkway, adjacent to the historic landmark Longfellow Bridge, consists of a contemporary tubular steel arch with a span of approximately 226 ft over a parkway. The steel superstructure, approximately 550 ft in length, is continuous, without any joints, and its shape in plan follows a curvilinear alignment in two directions. The arch and approach spans employ a distinct architectural theme of slender steel piers and struts for visual consistency and aesthetic appeal.

The new crossing replaced an existing bridge that was too narrow and had inadequate access stairs; conflicts between pedestrians and bicyclists were common. The placement and overall geometry of the new bridge were carefully selected to comply with the ADA maximum slope requirements and avoid impacting large trees in the parkland as much as possible—and its width of 14 ft doubles that of the original bridge. Several entry points and connections to the existing network of walkways along the Esplanade are integrated into the design of the new bridge.

The major challenge of this unique bridge was the fabrication of the steel structure and its overall constructability. Its design included complex curves and welded connections. The elegant steel superstructure consists of steel girders branching into two curved staircases and a scenic overlook plaza near the river. The bridge's steel fit-up required careful planning during the final design phase, as construction over a busy arterial road necessitated a detailed erection plan and sequencing. Stresses were evaluated in all structural members during both fabrication and erection.

The main steel arch has a unique shape, being wider at the crown and narrower at the abutments, which helped minimize the size of the anchoring abutments at the park level. The arch also includes


Juan Navarro


Christopher McIntosh


Newport Industrial Fabrication

Bridge Stats

Opened to traffic: August 31, 2018

Span lengths: 43 ft, 36 ft, 49 ft, 49 ft, 23 ft, 226 ft, 16 ft, 21 ft, 25 ft, 30 ft, 30 ft

Total length: 548 ft

Average width: 14 ft

Total structural steel: 308 tons

Cost: $12.5 million

Coating/protection: Metallized

Carlos Arazaga

David Desroches

David Desroches

a series of inclined struts, creating a unique aesthetic truss effect. It is the longest bridge span over Storrow Drive, connecting the city to the riverfront. The crossing is also higher than any other existing bridge along the highway corridor, opening views and incorporating appropriate vertical clearances.

The arch was brought to the site in pieces and assembled during overnight hours to reduce traffic impacts, and it was welded in place in order to avoid using visible bolted connections. The bridge approaches include Y-shaped piers, which visually match the main architectural theme creating a visually unified structural system. Aesthetic lighting is also included to increase the sense of safety and appeal at night. The sinuous crossing is perfectly integrated into the landscape thanks to its transparency and lightness.

The new signature pedestrian bridge has quickly become a source of pride for the community due to its technical ingenuity, elegant detailing, and context-sensitive design, which perfectly integrates into Boston's landscape and historic riverfront.

For more on the Frances Appleton Pedestrian Bridge, see "Take Me to the River" in the April 2019 issue of Modern Steel Construction, *available at* ***www.modernsteel.com*****.**

Steel Team

Fabricator and Detailer
Newport Industrial Fabrication, Newport, Maine

Erector
Saugus Construction Corp., Georgetown, Mass.

Bender-Roller
Kottler Metal Products, Willoughby, Ohio

Castings
Cast Connex Corporation, Toronto

Structural Engineer
STV, Boston

Designer
Rosales + Partners, Boston

General Contractor
White/Skanska/Consigli, JV, Framingham, Mass.

Owner
Massachusetts Department of Conservation and Recreation, Boston


Kenny Flowers


Kenny Flowers

Illinois DOT

Bridge Stats

Opened to traffic: December 20, 2018

Span length: 750 ft

Total length: 1,500 ft

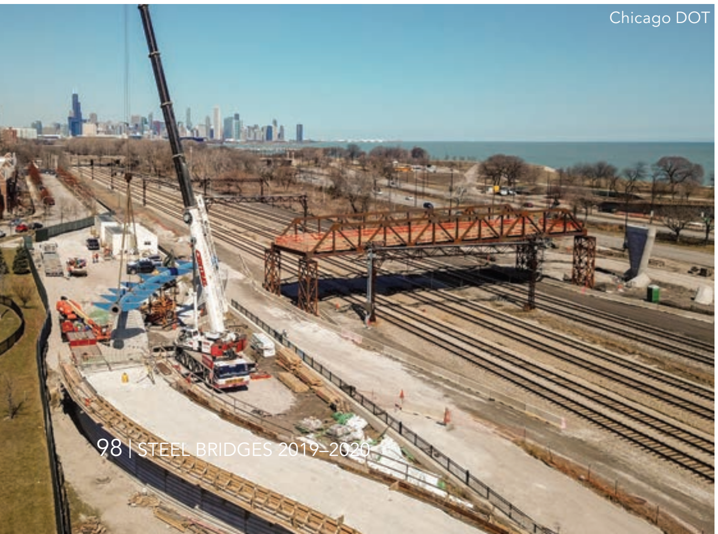
Average width: 24 ft

Total structural steel: 676 tons

Cost: $29 million

Coating/protection: PPG 68HS primer, Amercoat 399 intermediate coat, Amercoat 450H final coat (Blue Oasis)


Chicago DOT


Chicago DOT


Chicago DOT

Chicago DOT

MERIT AWARD Special Purpose

41st Street Pedestrian Bridge, Chicago

CHICAGO'S 41ST STREET PEDESTRIAN BRIDGE design was an award winner right from the get-go.

The design team's curving, arch-supported steel concept won an international design competition to create the bridge. The resulting span connects the city's Bronzeville neighborhood with the trail system that runs along Lake Michigan. The bridge provides pedestrians with safe passage over Lake Shore Drive as well as the Metra Electric/CN Railroads, both of which had to stay in operation during construction. The railway sees approximately 263 trains per day while Lake Shore Drive carries approximately 100,000 vehicles per day.

Two main component round sections (36-in. and 48-in. OD induction bent pipe) tied together with built-up box girders form the main span of the pedestrian bridge. The pipe and bridge have both sweep and camber, so the pipe had to be carefully bent in order to induce both elements simultaneously. The process of induction-bending the pipe was particularly challenging, given that the actual diameter, ovality, and pipe shrinkage had to be taken into consideration prior to fabrication to ensure all of the subcomponents that tie into the pipe fit correctly. The bridge was progressively preassembled in the shop in order to ensure proper geometry and fit-up, which was especially challenging due to the large sweeping and curving geometry that required much preplanning and lots of shop floor space.

The team also had to figure out the logistics of shipping the large sections of the bridge from two fabrication shops to the project site. The bridge components were shop-welded to their fullest extent, resulting in extremely long, wide, and heavy permit loads that required significant preplanning and coordination. The largest structural piece was 62 ft long, 24 ft, 4 in. wide, and 38.3 tons, with the heaviest structural piece being just over 42 tons. The bridge was shipped to the job site in 14 built-up sections, including six approach single-pipe spine assemblies and eight main span double-pipe assemblies; the main span assemblies were more than 24 ft wide.

The arches use bolted splices as well as field welds for aesthetic purposes. The design team chose to use the end-plate bolted connection option to save time and cost during erection. Prior to delivery to the site, the structural steel was blasted and painted with a three-coat paint system in the shop.

The project came in under budget and opened six months ahead of the original contract completion date.

Steel Team

Fabricators

Hillsdale Fabricators AISC CERTIFIED FABRICATOR, St. Louis

Metal Pros, LLC AISC CERTIFIED FABRICATOR, Wichita, Kan. (handrails)

Erector

S&J Construction Co., Inc. AISC ASSOCIATE AISC CERTIFIED ERECTOR, Oak Forest, Ill.

Detailer

Esskay Structures, Inc. AISC DETAILER, Vienna, Va.

Bender-Roller

BendTec Inc. AISC CERTIFIED FABRICATOR, Duluth, Minn.
(also additional fabrication)

Designer/Structural Engineer

AECOM, Chicago

General Contractor

F.H. Paschen, S.N. Nielsen and Associates LLC, Chicago

Construction Manager

TranSystems, Chicago

Owner

Chicago Department of Transportation, Chicago


Mike Okimoto

MERIT AWARD Special Purpose

East Shore Bridge, Lake Tahoe, Nev.

THE THREE-MILE STRETCH BETWEEN Incline Village and Sand Harbor State Park on the east shore of Lake Tahoe in Nevada is, in a word, stunning. And a series of new steel-framed bridges is now an integral part of this scenic multiuse path.

The owner, the Nevada Department of Transportation (NDOT), used the construction-manager-at-risk (CMAR) delivery method for this $40 million trail project. The team faced an accelerated delivery schedule, challenging subsurface conditions and terrain, high seismicity, limited construction access, and an environmentally sensitive project location.

The three miles of new multiuse path was installed on a steep side slope between the existing State Route 28 and Lake Tahoe. The path comprises five steel bridges, totaling 809 ft. To create a structural system that could be installed with minimal disruption to traffic on the heavily used SR-28 adjacent to the trail alignment, the team designed prefabricated bridge spans composed of weathering steel girders that supported lightweight fiber-reinforced polymer (FRP) deck units. Composite Advantage manufactured the 50-ft-long pre-fabricated deck units with steel supplied by fabricator Cox Brothers Machining. The deck units were shipped to the site and placed by contractor Granite Construction during short-term road closures.

The various regulatory agencies that have jurisdiction over the area were focused on aesthetics.The project is highly visible from the lake, and it was very important to minimize visual impacts on the terrain. The steel girders and hand railings use weathering steel to minimize long-term maintenance costs associated with painted steel and to provide a surface finish that blends in with the natural terrain. The steel pipe sections used for the columns at the piers were galvanized and then coated with Natina to provide a finish that matches the weathering steel stringers.

Steel Fabricators

Stinger Bridge and Iron (AISC Certified Fabricator), Coolidge, Ariz.
(substructure elements)

Cox Brothers Machining, Inc. (AISC Certified Fabricator), Jackson, Mich.
(steel stringers and diaphragms)

Steel Erector and General Contractor

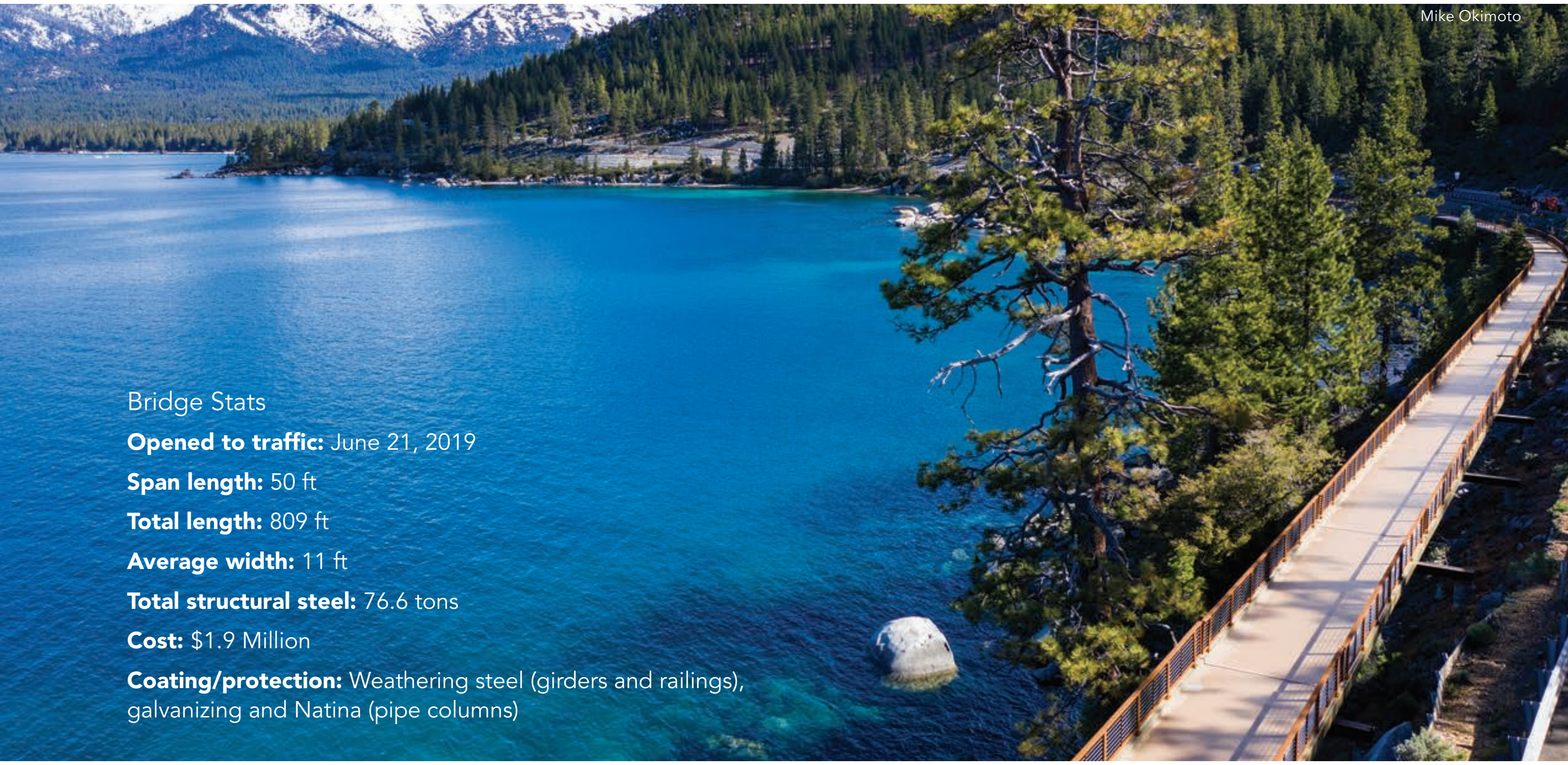
Granite Construction Inc., Sparks, Nev.

Structural Engineer

Jacobs, Sacramento, Calif.

Owner

Nevada Department of Transportation, Carson City, Nev.



Bridge Stats

Opened to traffic: June 21, 2019

Span length: 50 ft

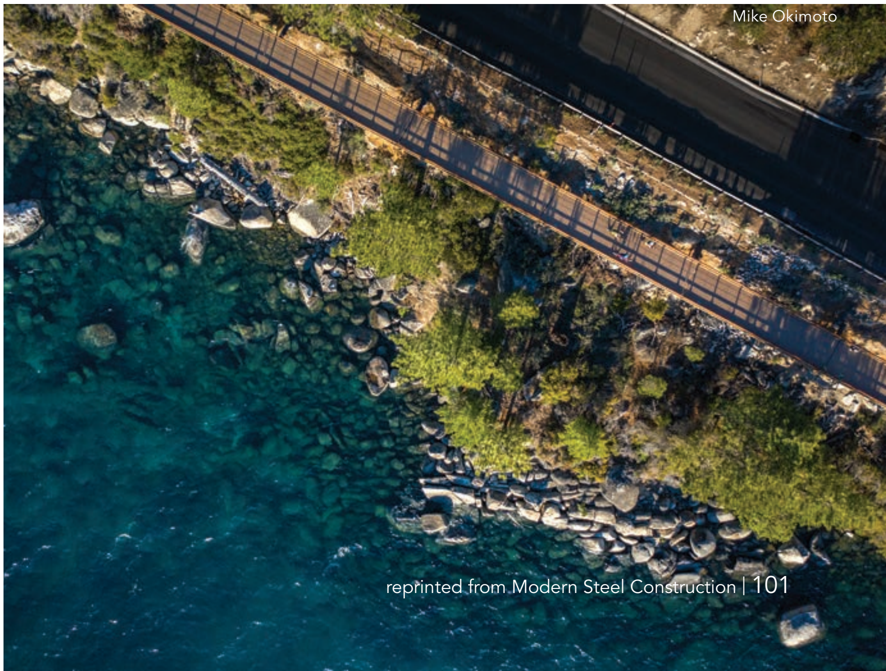
Total length: 809 ft

Average width: 11 ft

Total structural steel: 76.6 tons

Cost: $1.9 Million

Coating/protection: Weathering steel (girders and railings), galvanizing and Natina (pipe columns)




Aaron Colorito, PE (Michael Baker International)


Aaron Colorito, PE (Michael Baker International)


Aaron Colorito, PE (Michael Baker International)

NATIONAL AWARD Rehabilitation
Andy Warhol (Seventh Street) Bridge, Pittsburgh

THE ANDY WARHOL (SEVENTH STREET) BRIDGE, an eye-bar-chain, self-anchored suspension bridge, carries Seventh Street over the Allegheny River, the Tenth Street Bypass, and the Three Rivers Heritage Trail in downtown Pittsburgh.

Named for the famed artist who hailed from Steel City, it is one of the "Three Sisters" bridges constructed from 1924 to 1928—the only trio of identical, side-by-side bridges in the world—and is the first self-anchored suspension span constructed in the United States.

The bridge required rehabilitation due to accelerating age-related deterioration. The project involved replacing the bridge deck, totally repainting the superstructure, performing structural steel substructure repairs, and applying scour protection. The Allegheny County Department of Public Works chose Michael Baker International to perform analysis and design of the rehabilitation. The design team combined recognition of historical significance with modern engineering practices to complete a structurally superior, sustainable rehabilitation that was also aesthetically relevant and pleasing.

The bridge was analyzed for the first time using a fully 3D finite element model to examine the effects of unbalanced loading and modern vehicles on the structure. Completing the rehabilitation required numerous materials that are not normally used in new bridge construction, like post-tensioned tie-down anchorages, forged steel bridge pins and nuts, permanently lubricated bronze bushings and washers, and bronze dedication plaques cast to replace missing plaques. Workers used electric shear wrenches to install thousands of ASTM F3125 Grade F1852 high-strength bolts with button heads, to mimic the look of rivets, thus improving structural capacity while being sensitive to appearance. New bridge lighting on sidewalks and pylon rooms replicates the style of the original lighting fixtures. The new roadway curb boxes are designed to be as unobtrusive as possible while still

Bridge Stats

Opened to traffic: November 17, 2017

Span lengths: 72.80 ft, 221.36 ft, 442.08 ft, 221.36 ft, 41.95 ft, 61.45 ft

Total length: 1,061 ft

Average width: 66 ft out-to-out

Cost: $25,425,000

Coating/protection: Three-coat organic zinc-epoxy-urethane (Aztec Gold)

David Briskey

Aaron Colorito, PE (Michael Baker International)

Aaron Colorito, PE (Michael Baker International)

allowing water to drain and prevent salt and debris from sitting on and corroding the stiffening girders.

The complex rehabilitation was performed as a conventional design-bid-build construction project and concurrent with road work on I-279/HOV lanes/North Shore Expressway. This necessitated well-organized traffic control for nearby PNC Park and Heinz Field (homes to the Pittsburgh Pirates and Steelers, respectively) events, maintenance of pedestrian crossings at the adjacent streets, and sustained access to riverside trails and adjacent businesses.

The bridge also had to act as its own lay-down yard, resulting in tight site conditions. Temporary underdeck shielding and coordination with the U.S. Coast Guard and local river users allowed safe river access. Notice was broadcast daily to mariners, and a monitored phone number and radio channels were established for large vessels. Temporary Duquesne Light (electrical) conduit enabled work on sidewalk brackets and replacement of electric conduits and supports. Temporary conduit in plastic corrugated pipe was placed on the sidewalk to maintain safe working conditions around energized lines, as well as to maintain a major power supply for downtown Pittsburgh.

The team used a variety of other construction innovations, including vibro-screed (air screed) and pump trucks to place the concrete deck, over-pouring the deck by ¼ in., subsequent grinding to provide correct cross slopes and longitudinal smoothness, and employing a temporary hold-down system using permanent post-tensioning rods. The new reinforced concrete deck is fully structural, using channel-type shear connectors to make the deck composite. The existing buckle plates, once the structural part of the deck, now remain as stay-in-place forms.

Steel Fabricator and Erector

Advantage Steel and Construction, Saxonburg, Pa.

Structural Engineer

Michael Baker International, Moon Township, Pa.

General Contractor

Brayman Construction, Saxonburg, Pa.

Owner

Allegheny County Department of Public Works, Pittsburgh

MERIT AWARD Rehabilitation

Winona Bridge, Winona, Minn.

THE THREE-SPAN steel riveted through-truss Winona Bridge across the Mississippi River stands as a beloved landmark and vital thoroughfare for motorists traveling between Wisconsin and Minnesota. Built in 1942, it is the only pre-1946 cantilever through-truss bridge in the latter state and played a central role in sustaining the economy of Winona and facilitating the flow of defense materials during World War II.

The 2007 collapse of Minneapolis's I-35W bridge threatened that history. Following the collapse, the Minnesota legislature provided funding and required MnDOT to develop an ambitious 10-year bridge replacement program, with a focus on fracture-critical bridges. MnDOT's inspection team discovered corrosion and section loss on multiple truss members, resulting in a load posting that restricted heavier commercial vehicles and closed the bridge for more than a week. Immediate repairs provided a short-term solution, but they highlighted the structure's continued importance: Wisconsinites who depended on Winona's first-call ambulance services found their link to the town severed. Local businesses took a hit during the shutdown. Nearly 12,000 motorists per day were forced to make detours of 60 miles roundtrip to other crossings over the Mississippi.

In 2014, MnDOT engaged Michael Baker International as prime consultant and Ames Construction as prime contractor—the department's first use of the construction manager/general contractor (CM/GC) approach—to work together to ensure the long-term reliability of the structure. Tearing down the bridge had already been ruled out; it was eligible for listing on the National Register of Historic Places and had become an iconic asset for the region, even appearing on a postage stamp celebrating the state's sesquicentennial. So the team aimed for an ambitious goal: completely rehabilitating the bridge to resist modern permit loads, reconstructing the approach spans, rebuilding the deck, and adding internal redundancy to comply with the intent of the state statutes, all while avoiding any adverse effects as determined by the State Historic Preservation Office. By modernizing the structure, the team would establish the first through-truss bridge in the Midwest to have internal redundancy added to all its fracture-critical elements.

Accomplishing all this required creative problem-solving and complex coordination. Completing a historic bridge rehabilitation is an intricate undertaking wherever the work occurs, but doing it on budget in Minnesota's harsh climate is a whole other matter. Long winters and road salting had fueled deterioration, making it possible the contractor would uncover even more corrosion in the field. Lead paint had to be removed, section-loss measurements taken, and the entire structure repainted. High-strength bolts and new steel plates had to be installed over tens of thousands of rivets, which had not always been installed according to the original plans. The team also had to replace the aging bridge deck and patch spalled piers to blend with the bridge's concrete color. After analyzing the structure's timber piles, the team encountered

Bridge Stats

Opened to traffic: July 1, 2019

Span lengths: 47 ft, 119 ft, 123 ft, 134 ft, 134 ft, 130 ft, 130 ft, 242 ft, 450 ft, 242 ft, 130 ft, 130 ft, 130 ft, 130 ft

Total length: 2,291 ft

Average width: 33 ft

Total structural steel: 710 tons

Coating/protection: Inorganic zinc-rich three-coat paint system

Kent Zinn

Keith Molnau

Nicholas Sovell

Kent Zinn

another dilemma: The piles would not stand up to the impact of a modern barge collision and would have to be strengthened as well.

Every step of the way, Michael Baker's team worked with the project historian and MnDOT's Bridge Office and Cultural Resources Unit (CRU) to evaluate each engineering improvement for compliance with the National Historic Preservation Act of 1966 and Minnesota's State Historic Preservation Office. This called for extensive, detail-oriented work and intense coordination.

The CM/GC team began work on the Winona Bridge in 2014. It first generated complex 3D finite element models to analyze the fracture-critical components of the structure and formulate plans for strength and internal redundancy retrofits. These designs relied on steel plates and post-tensioning bars that strengthened the bridge and extended its service life by 50 years.

Owing to the age of the structure and the parameters for historic designation, the team faced numerous obstacles during the rehabilitation. It solved the issues posed by the bridge's timber piles by implementing a scour-protection system, which consisted of geobags and rip rap. Additionally, an innovative underwater strut system was designed, essentially linking the original structure to the new parallel bridge. In doing this, the team ensured that both structures would share the impact of any barge collision, distributing the force and bolstering the older bridge's timber-pile foundations.

To rebuild the approach spans, the team installed six new steel deck truss spans and constructed 15 prestressed concrete girder spans. For the main through-truss spans, 148 truss members were reinforced with steel plates and 76 with high-strength rods. The team replaced nine concrete piers from the original design by using longer, prestressed girder approach spans, which were less expensive to fabricate and construct.

Ultimately, the CM/GC approach proved to be a massive success, providing expert oversight, comprehensive coordination, and state-of-the-art solutions. What's more, it delivered these innovative designs with great cost certainty prior to construction and no construction cost growth, opening the bridge to traffic six months ahead of schedule.

Steel Team

Fabricator and Detailer

LeJeune Steel Company, Minneapolis

Erector

Danny's Construction Company, Shakopee, Minn.

Additional Detailer

DBM Vircon Services, Port Coquitlam, B.C., Canada

Structural Engineer

Michael Baker International, Chicago

General Contractor

Ames Construction, Burnsville, Minn.

Owner

Minnesota Department of Transportation, Rochester, Minn.

Jeff Jobe

NATIONAL AWARD Reconstruction

BNSF Wind River Bridge, Skamania County, Wash.

WITH AN EXPECTED LIFESPAN OF A CENTURY, the newly reconstructed BNSF Wind River Bridge serves as a critical connector on BNSF's Fallbridge Subdivision, enabling the safe and reliable crossing of both freight and passenger traffic over the mouth of the Wind River in the Columbia River Gorge in Washington State.

HNTB provided design, permitting, and construction management services for the steel bridge's reconstruction. The new bridge consists of a 260-ft-long, single-track truss span with precast double cell box beam approaches supported on concrete pier caps with drilled shaft and driven pile foundations. The project site is located in a national scenic area between State Highway 14 and the Columbia River, resulting in limited available site access for the contractor and the need for strict environmental compliance during construction.

Because the bridge carries a large amount of freight and passenger traffic, minimizing track closures remained a priority throughout the project. An accelerated bridge construction (ABC) technique, float-in/out, provided two distinct advantages to the project. First, it reduced the need for temporary work bridge piles, which were required to be installed and removed within a dedicated in-water work window. Secondly, it minimized impacts to railroad operations by limiting the time required to remove the existing span and install the new truss span on the existing bridge alignment.

Addressing the challenges associated with the float-in/out operation was one of the greatest challenges faced during the project, due to the number of associated variables. Because the truss span was erected in Portland, Ore., roughly 60 miles west of the project site, it was critical that the contractor's plan to float the erected truss span down the Columbia River be fully vetted. To this end, BNSF and HNTB worked with the contractor to review their proposed maritime procedure and engineering and developed a plan to coordinate water levels with the Bonneville Dam to control the pool elevations during the bridge change-out.

Because the bridge is located in the Columbia River Gorge National Scenic Area, it was critical that the aesthetics of the new structure not disturb the existing view for the public. To address this concern, BNSF and HNTB worked with the applicable regulatory agencies to review proposed span types and bridge colors. The new main span used a Warren-type truss with weathering steel to closely match the feel of the existing Pratt-style truss and its weathered patina. Concrete pier caps and approach spans were also stained with a charcoal color to better blend in with the existing landscape. The team carefully selected materials to fulfill the project's specific aesthetic requirements while also ensuring the integrity of the new bridge's 100-year lifespan.

In addition to meeting a variety of requirements, the bridge design also needed to be adaptable. The bridge can accommodate the heavy live loads of current freight and passenger trains, and it is also robust enough to meet demands imposed by enhanced future railroad loading.

Steel Team

Fabricator

Fought and Company, Inc., Tigard, Ore.

Detailer

Graphics for Steel Structures, Hicksville, N.Y.

Structural Engineer

HNTB, St. Louis

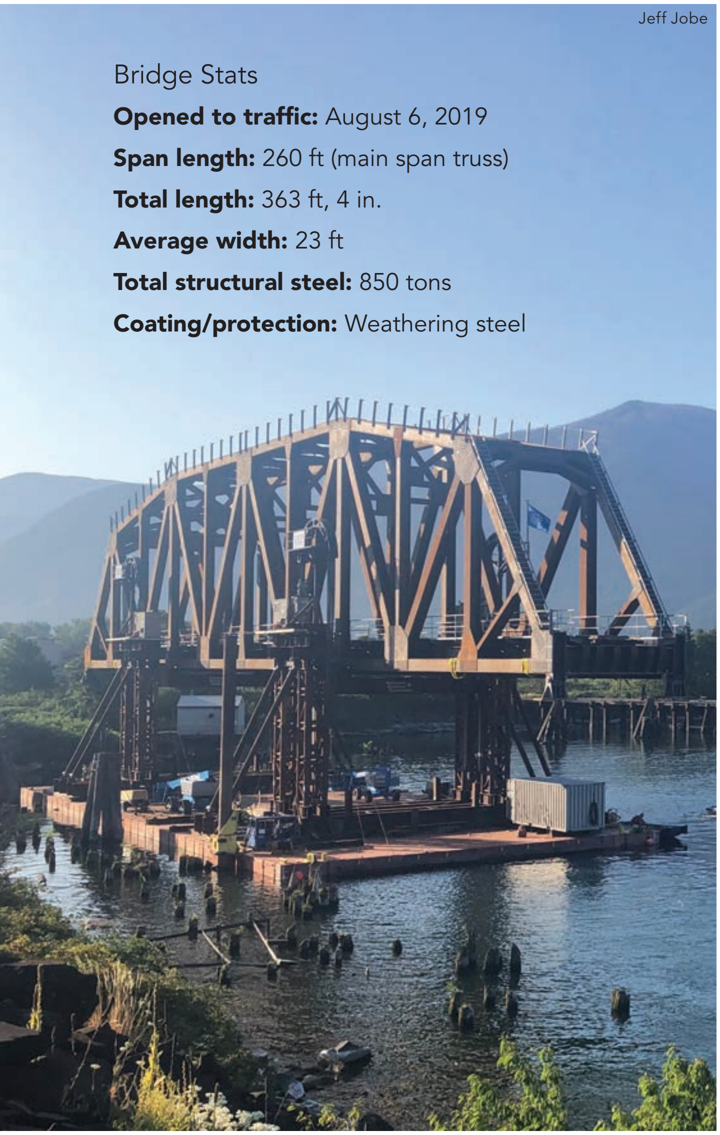
General Contractor

Hamilton Construction Company, Portland, Ore.

Owner

BNSF Railway, Kansas City, Kan.


Kyle Izatt


Jeff Jobe

Bridge Stats

Opened to traffic: August 6, 2019

Span length: 260 ft (main span truss)

Total length: 363 ft, 4 in.

Average width: 23 ft

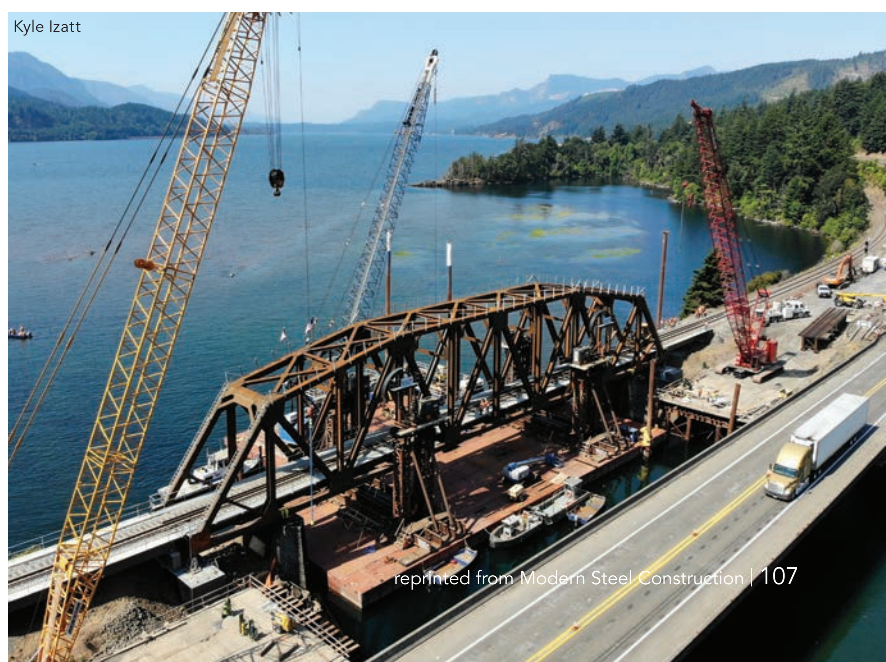
Total structural steel: 850 tons

Coating/protection: Weathering steel


Jeff Jobe

Jeff Jobe


Charlie Woods


Kyle Izatt

MERIT AWARD Reconstruction

I-240 MemFix4, Memphis, Tenn.

WHEN THE TENNESEE DEPARTMENT OF TRANSPORTATION (TDOT) faced the urgent need to replace or repair four deficient structures over I-240 in Memphis, subjecting roadway users to another long-term construction project simply wasn't an option. With traffic levels of approximately 180,000 vehicles per day, TDOT wanted this critical project completed quickly, with minimal impact to travelers.

The four bridges in the project, dubbed MemFix4, are two new Poplar Interchange bridges; a new Norfolk Southern Railroad (NSR) bridge; and rehabilitation of the concrete Park Avenue bridge. This $54 million project was delivered under the CM/GC delivery method—the second-ever CM/GC transportation project in the state of Tennessee. TDOT, Benesch, and Kiewit worked together in the design phase to develop innovative ideas to address the numerous site challenges and project needs while maintaining the ability to meet the project's aggressive schedule.

The WB and EB Poplar Avenue bridge replacements required multiple innovative prefabricated bridge elements. The constructed Poplar Ave. bridges consist of a 263-ft, two-span bridge for WB Poplar and a 222-ft, two-span bridge for EB Poplar. For the replacement of these structures, extensive modeling and structural analysis was required to address high seismic conditions. The team developed several custom elements. These included custom steel bearings and framing, over 13,000 linear ft of micropiles, new substructures constructed under traffic, and modular bridge superstructures—all of which addressed site challenges while completing the project in just 18 months.

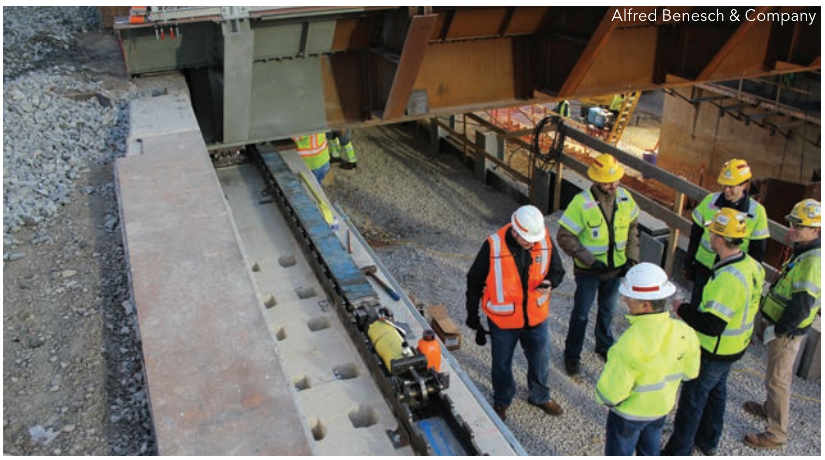
The project team used accelerated bridge construction (ABC) methods to address site constraints and the necessity for minimal impacts to traffic. This led to the Poplar Avenue bridges being built off-site at a "bridge farm," rolled to the site using self-propelled modular transporters (SPMTs), and then lifted into place using large crawler cranes. Once the bridges were constructed, Kiewit was able to complete the planned widening of I-240 to alleviate the lane drop that the entrance ramps required.

Because the existing piers for the Norfolk Southern (NS) Rail Bridge were founded on spread footings, it was not cost-efficient to upgrade the existing bridge's substructures to meet current seismic design standards. TDOT realized that the next project needed to replace the structures while minimizing impacts to the thousands of vehicular travelers through this interchange and the nearly 20 trains per day on the NS/I-240 overpass.

To replace this bridge, a temporary shoofly structure was constructed just inches away from the existing bridge. It was composed of temporary concrete piers supported by a foundation of over 6,000 linear ft of micropiles. Leaving train traffic largely uninterrupted during construction, the permanent steel superstructure supporting a ballasted track was erected on the shoofly alignment and trains were switched onto this alignment. With trains traveling on the shoofly

Alfred Benesch & Company

Kiewit Infrastructure South Co.



Alfred Benesch & Company

Alfred Benesch & Company

structure, the old bridge was demolished and the new substructures were built. The two new 1,100-ton superstructure sections were then laterally slid 35 ft into place, one track at a time, during two weekend Interstate closures.

The Memphis area is located in the influence zone of the New Madrid Fault, which in 1811 and 1812 produced four of the most powerful earthquakes east of the Rocky Mountains in recorded history. The team spent significant effort during the design phase to ensure that solutions could be constructable while still meeting the seismic demands. Designers focused on the impacts of time during the construction phase, especially when it came to key elements that would be built during weekend closures. Benesch used finite element modeling to precisely design elements such as the bearing anchors to minimize the materials and labor required while still meeting the design requirements.

For more on the I-240 MemFix4 project, see "A Bridge Replacement in Four Parts" in the October 2019 issue of Modern Steel Construction, *available at* ***www.modernsteel.com***.

Steel Team

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

Erector and General Contractor
Kiewit Infrastructure Co., Brentwood, Tenn.

Additional Detailer
CRC Steel Detailing, LLC, Worth, Texas

Structural Engineer
Benesch, Nashville, Tenn.

Owner
Tennessee Department of Transportation, Nashville, Tenn.

Bridge Stats

Opened to traffic: June 30, 2019

Span lengths:
WB Poplar Ave.: 150.5 ft, 113.08 ft
EB Poplar Ave.: 88.17 ft, 134.17 ft
Norfolk Southern Railroad Bridge: 50.83 ft, 73.5 ft, 73.5 ft, 87.5 ft, 50.83 ft

Total lengths:
WB Poplar Ave.: 222 ft
EB Poplar Ave.: 263 ft
Norfolk Southern Railroad Bridge: 338 ft

Average width:
WB Poplar Ave.: 65 ft
EB Poplar Ave.: 72 ft
Norfolk Southern Railroad Bridge: 36 ft

Total structural steel:
WB Poplar Ave.: 614 tons
EB Poplar Ave.: 287 tons
Norfolk Southern Railroad Bridge: 948 tons
All bridges: 1,849 tons

Cost: $28.4 million (combined structures cost)

Coating/protection: Weathering steel (WB and EB Poplar Ave.), weathering and painted steel (Norfolk Southern Railroad Bridge)

SPECIAL AWARD FOR RESILIENCE

Liberty Bridge, Pittsburgh

THE LIBERTY BRIDGE has been a landmark structure and Pittsburgh icon since it opened in 1928. A recent construction mishap made it an icon for the resilience of steel, too.

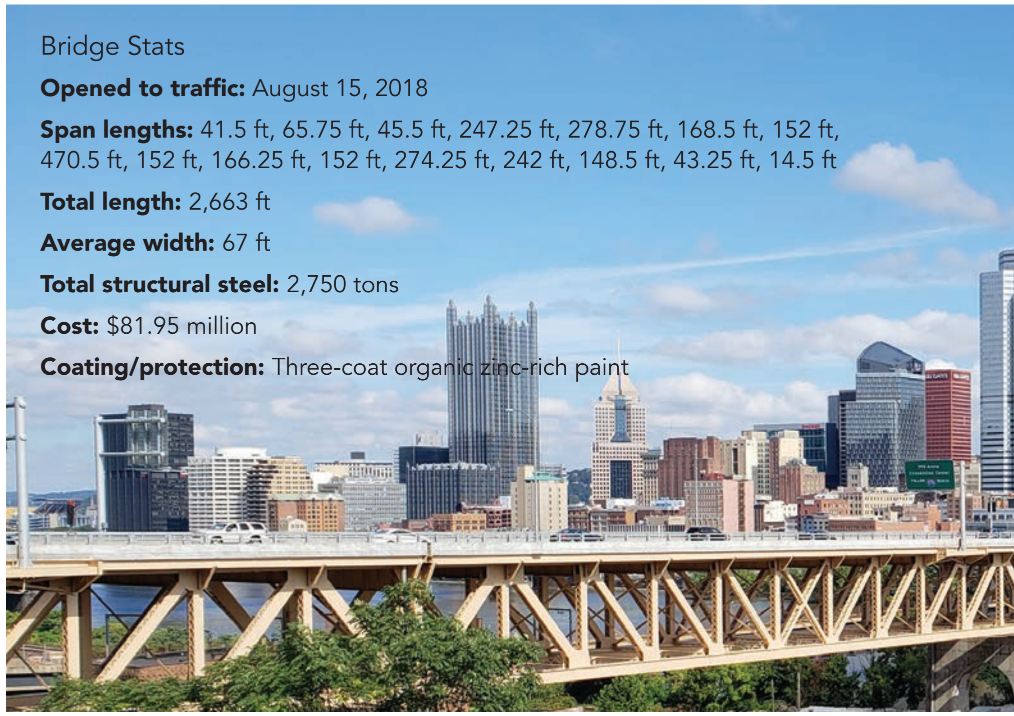
In the years following its five-mile-long opening parade, this bridge created the modern suburbs and quadrupled property values south of Pittsburgh. However, by 2014 the bridge, which carried 55,000 vehicles per day, was in poor condition. It could no longer carry trucks and had become a poster-child for America's infrastructure crisis, featuring prominently in a 60 Minutes profile of America's neglected infrastructure. Referring to Liberty Bridge and others like it, Ray LaHood, United States Secretary of Transportation, said plainly: "Our infrastructure is on life support right now."

PennDOT and HDR responded with a rehabilitation project that preserved the structure while meeting current engineering and accessibility standards. PennDOT's main goals in this rehabilitation were to remove the load posting on the bridge, ensure the bridge was accessible and safe per current codes, and secure 40 more years of use from this historic truss.

The first steel Exodermic grid deck used in Pennsylvania reduced impacts to the bridge's thousands of daily users while a deck the size of three football fields was replaced. Sections of this deck were prefabricated in panels that could be installed during weekend closures and connected together with high-strength concrete. A custom rapid-set concrete mix was created for this project, which allowed traffic to use new deck sections just a few hours after the concrete was placed. The new deck combines the strength of steel T-beams with reinforced concrete on top, making it strong, light, and easy to overlay in the future.

The deck innovations were planned in advance, but the greatest innovations are often unplanned. When an accidental construction fire warped and buckled a main truss compression chord, forcing an immediate bridge closure, the team raced to develop a solution to fix the bridge and reopen this critical urban link. The bridge was in a perilous state; no one knew how badly the structure might be overstressed or if collapse was imminent. To assess and fix the bridge, teams of engineers worked many days and nights until the bridge reopened.


Nicholas Burdette, PE



Bridge Stats

Opened to traffic: August 15, 2018

Span lengths: 41.5 ft, 65.75 ft, 45.5 ft, 247.25 ft, 278.75 ft, 168.5 ft, 152 ft, 470.5 ft, 152 ft, 166.25 ft, 152 ft, 274.25 ft, 242 ft, 148.5 ft, 43.25 ft, 14.5 ft

Total length: 2,663 ft

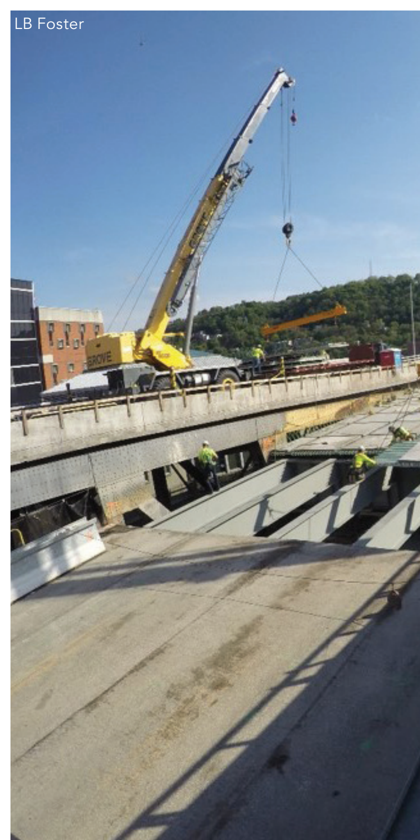
Average width: 67 ft

Total structural steel: 2,750 tons

Cost: $81.95 million

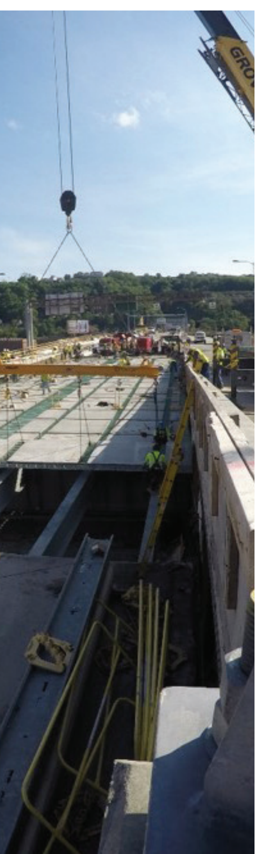
Coating/protection: Three-coat organic zinc-rich paint


Nicholas Burdette, PE


LB Foster

Christine Shiring

Christine Shiring



Christine Shiring

The team used a 3D analysis model to assess the crippled structure, including both trusses, every bracing member, and the partially removed deck. Using hand-drafted documents from the 1920s, hundreds of unique truss and bracing members were modeled. The day following the closure, the new model showed that most of the 1,000 tons carried by the damaged chord shed into the undamaged sister truss through wind bracing. The 3D steel truss and bracing system proved redundant. No member was overstressed from the bridge dead load. This finding gave authorities confidence to open the river below the structure to commercial traffic, preventing further economic impact to river commerce.

Without a historical precedent to go by, engineers developed a steel jacking frame concept to fix the buckled member that same day. This frame would attach to the member and 2,000 tons of force could be applied with huge jacks to straighten the buckled steel. The contractor adopted the concept and their design team developed it further. The member was repaired through a combination of jacking and heat straightening only 24 days after the fire, and traffic was restored on the bridge—a momentous day for Pittsburgh commuters.

Trucks can now use the structure, with its new bridge deck and supporting stringers and after hundreds of unique steel repairs on beams, truss members, and connection plates. Replacing the bridge deck was crucial in order to preserve the bridge and allow it to function safely for another 40 years. The new deck, with modern bridge joints and drainage, provides a robust and waterproof "roof" to keep the steel below dry and corrosion-free. In addition, replacing the old stringers along with the deck eliminated many poor details that are prone to cracking over time. Holes, cuts, and welds in these beams did not meet current fatigue requirements. As years of exposure to traffic mounted, these details were a long-term liability requiring detailed documentation for each inspection. Replacing all stringers with new, properly fabricated beams, eliminated this liability.

Steel Fabricators
Hall Industries, Inc. AISC CERTIFIED FABRICATOR, Ellwood City, Pa.
L.B. Foster Company, Pittsburgh

Structural Engineer
HDR, Pittsburgh

General Contractor
Fay, an i+iconUSA Company, Pittsburgh

Owner
PennDOT, Engineering District 11,
Bridgeville, Pa.

A Minneapolis bridge project takes advantage of an innovative launching process to remove an existing Warren truss and install its replacement.

Ready for Launch

BY MARK MAVES, PE, AND MARTIN FURRER, SE, PE

Mark Maves (**mmaves@sehinc.com**) is the leader of Short Elliott Hendrickson's (SEH) bridge group in St. Paul. **Martin Furrer** (**martin.furrer@parsons.com**) is Program Director, Bridge with Parsons.

THE ST. ANTHONY PARKWAY BRIDGE is at a nexus of sorts.

The bridge is located over the BNSF Northtown rail yard in Minneapolis, one of the most heavily used rail yards in the Midwest, with approximately 20% of all BNSF rail traffic in the United States passing through it. An average of 14 trains of 100 or more cars are assembled here each day, and an additional 60-plus trains operate daily on two main line tracks along the western portion of the yard. At the crossing location, the bridge spans 23 tracks.

Built in 1925, the original bridge at this crossing was a 535-ft, five-span steel Warren truss bridge that had become structurally and geometrically deficient, containing fracture-critical members (FCMs) as well as narrow travel lanes, poor bike and pedestrian trails, and substandard vertical clearances.

Since the bridge was eligible for listing on the National Register of Historic Places, every effort was made to save it. But in 2012, after a series of many meetings and ongoing consultation between the City of Minneapolis, Federal Highway Administration (FHWA), and Minnesota State Historical Preservation Office (MnSHPO), these entities agreed that the bridge was too badly deteriorated and needed to be replaced. The existing bridge was eventually closed to traffic prior to replacement due to vehicles violating the posted weight limits.

St. Anthony Parkway crosses the Northtown yard at a 27° skew, and the approach roadways to the bridge are on significant grades with roadway intersections near each abutment. These constraints significantly limited the ability to modify bridge skew and vertical roadway profiles. In addition, existing horizontal clearances between tracks and in-yard piers did not meet current code requirements, causing concern for rail yard worker safety. The locations of the four piers in the yard also acted as pinch points and

Total Structure Length:	560 ft, 6 in.
Span Lengths:	305 ft (main); 127 ft and 127 ft (approaches)
Average Width:	58 ft
Total Steel Tonnage:	1,094 tons
Coating/Protection System:	Weathering steel

The new St. Anthony Parkway bridge used more than 1.000 tons of steel to cross the BNSF Northtown rail yard in Minneapolis, one of the busiest rail yards in the Midwest.

All photos: Mark Maves

severely impeded BNSF operations and flexibility for future expansion. Due to these issues, BNSF requested a bridge replacement design that reduced the number of piers in the yard to two.

Uninterrupted service through the yard and beneath the bridge is essential to BNSF's operation, especially along the two main line tracks. BNSF understood that lateral clearance on the ground would have to be provided for the duration of this job in order to provide safe conditions for both bridge and rail workers. As the area is already constrained, providing these needed clearances during bridge construction would result in a disruption to normal rail service. In addition, rerouting trains to adjacent tracks within to perform construction had to be scheduled during tight windows, as BNSF would not allow its operations to be shut down for extended periods of time. That meant the project could take much longer to complete, require a less conventional method of removal and erection, and potentially be more expensive. In addition, per the Minnesota Department of Transportation (MnDOT) and FHWA, the replacement bridge needed to be redundant and minimize or eliminate the use of FCMs.

When it came to the new bridge's design, community feedback during public outreach meetings pushed for an above-deck structure visually similar to the existing steel Warren trusses and that the aesthetics should maintain an urban industrial feel. As a result, the bridge's main span consists of a redundant steel truss structure, incorporating unique load path and internal redundancy measures including eliminating fracture critical steel truss members and gusset plates and using a post-tensioned concrete bottom chord. The approach spans consist of conventional steel girders, and all three spans incorporate a full-depth cast-in-place concrete deck constructed with stay-in-place metal deck forms in order to improve safety and minimize construction impacts to the rail yard below.

The design consists of a 305-ft-long steel Warren truss structure with two 127-ft-long conventional steel girder approach spans. The following strategies were employed to eliminate any FCMs from the truss structure:

- Split steel member tension verticals and diagonals (two T-sections), individually bolted to adjacent members, were used, and the members and connections were designed for the fracture of the accompanying twin member and included in the plans as a system-redundant member (SRM).
- The bottom chord is a post-tensioned concrete member encased with a U-shaped steel shell, where the shell provides the tension chord for erection and launching before providing the permanent formwork and steel fascia for the post-tensioned concrete.
- Transverse steel floor beams are spaced at roughly 10 ft and made composite with the concrete deck, where the deck is shown to be capable of sustaining the loss of a floor beam by spanning between adjacent floor beams and included in the plans as an SRM.

above: Removing the trusses from the original bridge, which had been used to cross the rail yard for nearly a century.

above and below: Fabricator Industrial Steel Construction, Inc. (ISC) first assembled the bridge trusses in its Gary, Ind., shop to ensure that everything fit together. The steel was then disassembled and trucked to the bridge site.

The design solution was reviewed with FHWA to confirm that eliminating FCMs in lieu of SRMs negates the bridge requiring biannual hands-on inspections. The use of unpainted weathering steel and practical detailing minimize future maintenance work over the railroad tracks while providing the community with the desired urban industrial feel and the familiarity of the truss structural shape that has provided this railroad crossing for nearly a century. This minimization of future inspection and maintenance reduces the City's life-cycle cost for the crossing, and eliminating two yard piers minimizes impacts to BNSF's operation.

Removing the existing bridge trusses and erecting the new main span truss and approach spans were integral considerations during design, and the design team evaluated constructability and erection approaches that could be achieved in the track shutdown windows that were acceptable to BNSF. It was determined that longitudinally launching the existing trusses out and the new main span truss in using a set of launching beams was the most likely method a contractor would want to use, and a suggested launching scheme was included in the construction plans and specifications.

Selected contractor Lunda Construction Company's erection approach closely followed the launching approach envisioned by the design team with one notable change: The three easternmost truss spans were removed conventionally in the yard, using 10-day closure windows for the yard tracks. The launching system was sized for the roughly 800-ton new truss structure but was first used to remove the two existing westernmost steel trusses, including the concrete deck system, over the BNSF's main line tracks to the western approach embankment for demolition.

The new bridge trusses were installed and the old ones removed via a set of launching beams.

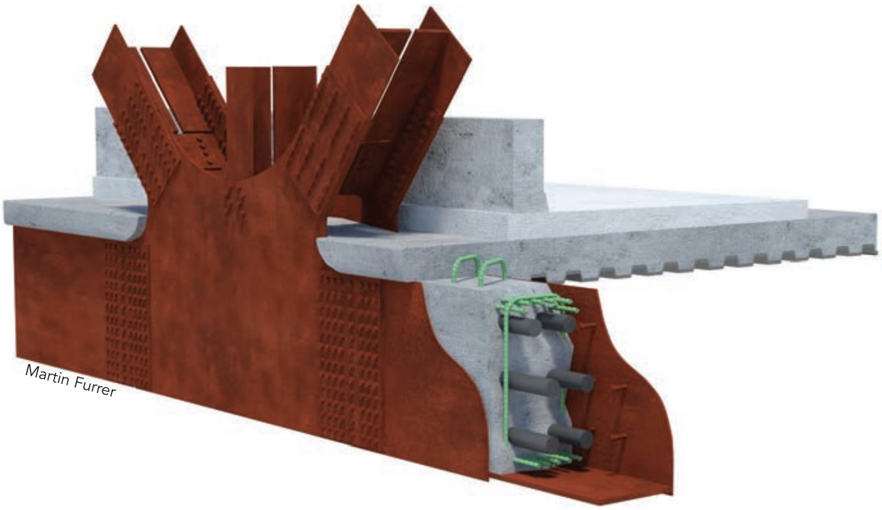
The launching assembly consisted of twin plate girders bolted together with cross bracing near each truss. Lunda selected Hilman rollers as the moving vehicle that traveled in channel sections acting as tracks and welded to the top of the launching beams. Transverse beams spanned between the launching assemblies located on each side of the truss. The transverse beams were connected to the truss that was being launched with post-tensioning bars located at each corner of the truss. These post-tensioning bars were used to raise and lower the truss with hydraulic jacks.

The longitudinal jacking setup consisted of a series of post-tensioning bars coupled together and supported on wood blocking spanning between the top flanges of the twin launching beams. Two jacks located at the end of the launching beams pushed against the transversely spanning jacking beam that in turn is connected to the leading post-tensioning bars. As the selected system was a pull-only system, it was set up on the western approach embankment for the truss removals and then moved east of the new eastern truss pier for the launch of the new

right and below: The launching system was sized for the new roughly 800-ton truss structure but was first used to remove the two existing westernmost steel trusses over the BNSF's main tracks to the western approach embankment for demolition.

above and below: The bottom chord is a post-tensioned concrete member encased in a U-shaped steel shell.

right: An upper chord node with split tension members and connections.



truss. The launched removal of the 130-ft-long Truss 5 (farthest to the west) took two four-hour windows—including the learning curve for the crew members, which were performing this type of operation for the first time. When it came time to remove the 240-ft-long Truss 4, the team was able to do the job in a single six-hour window.

When it came to building the new truss, fabricator Industrial Steel Construction, Inc. (ISC) first assembled it (as well as the end portal system) in its Gary, Ind., shop to ensure that everything fit together. The steel was then disassembled and trucked to the bridge site. Full under-roof shop assembly allowed optimum alignment, eliminating thermal distortions caused by weather, and on-site workers installed more than 27,000 bolts without field drilling or reaming.

Field assembly on the western approach embankment progressed from the east to west with a crane supplying the stick-built truss elements. After completing steel assembly, the team removed the intermediate blocking, and then the stay-in-place metal decking was installed to act as a working platform for rebar, post-tensioning, concrete installation, and as a protective shielding for railroad traffic. The new truss was then launched 310 ft, via the same method that was used to remove the original trusses, during two four-hour windows. Once in its final plan position, the truss was set down on steel columns attached to the west abutment and the eastern truss pier so that the launching beams could be removed. From there, the truss was lowered onto the permanent bearings using the vertical jacking system of post-tensioning bars and hydraulic jacks. The lower chord post-tensioning conduits, hardware, and reinforcing steel were then installed in the steel shell. After the bottom chord concrete was poured and cured, the post-tensioning tendons were stressed, and deck reinforcement was installed before the deck concrete was poured. Finally, the sidewalks, railing, and fencing were installed.

In order to pay homage to the historic structure, the project includes an interpretive plaza adjacent to the west approach that describes the previous bridge crossings, the history of the neighborhood, and the technical aspects of the new bridge. Portions of the steel from the removed bridge were even incorporated into elements such as planter boxes to pay tribute to the historic structure. ■

Owner
City of Minneapolis – Public Works

General Contractor
Lunda Construction Company

Structural Engineers
Short Elliott Hendrickson, Inc. (SEH), St. Paul, Minn. (EOR)
Parsons Corporation, Minneapolis and Chicago (EOR, truss span)

Steel Team

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

Erector
Danny's Construction Company, Inc., AISC Associate, AISC Certified Erector, Shakopee, Minn.

Detailer
Tenca Steel Detailing, Inc., AISC Detailer, Quebec, Canada

above: Erecting one of the two 127-ft-long approach spans, which are supported by conventional plate girders.

below: The west end of the new bridge is highlighted by planter boxes made from steel from the original bridge.

Long-Term Analysis for Short-Span Bridges

BY MICHAEL G. BARKER, PE, PHD

A recent life-cycle cost analysis compares steel and concrete short-span bridges.

Michael G. Barker (**barker@uwyo.edu**) is a professor of civil and architectural engineering at the University of Wyoming. He works with the Short Span Steel Bridge Alliance (SSSBA) to educate bridge owners, engineers, designers, and students on the design and construction of short-span steel bridges.

THERE HAS HISTORICALLY been a healthy competition between material types for new bridge construction.

In personal discussions over the years with officials from state departments of transportation and local county engineers on effective and economical bridge construction, a frequent question that arises is the difference in life-cycle costs (LCC) between steel and concrete girder bridges. Both the concrete industry and the steel industry cite various anecdotal LCC advantages using their assumptions on cost and maintenance for their materials. Even though owners want to consider LCC in bridge design decisions, they are unconvinced with anecdotal discussions—they want evidence.

This is where a life-cycle cost analysis (LCCA) comes in. An LCCA is an economical method to compare design alternatives over the entire life of the structure. It considers not only initial costs, but also future costs, their timing, and the service life of the bridge. An LCCA determines the "true cost" of bridge alternatives, considering the time value of money, for an equivalent monetary comparison.

For instance, if one alternative has a higher initial cost and no future costs, an LCCA can compare this to an alternative that has a lower initial cost and costly rehabilitation in the future, discounting future costs to equivalent today costs for a direct economic comparison.

When addressing the question of steel versus concrete, again, there are many assumptions but a lack of hard evidence, so the Short Span Steel Bridge Alliance (SSSBA) initiated a study to develop useful owner information on historical LCCs for typical bridges. The study included a subset of the bridge inventory from the Pennsylvania Department of Transportation (PennDOT). It was narrowed down to five types of bridges: simple- and multi-span steel rolled beam, steel plate girder, concrete box adjacent, concrete box spread, and concrete I-beam bridges. Here, we'll explore the results. (The full report, "Historical Life Cycle Costs of Steel and Concrete Girder Bridges"—including a detailed explanation of the criteria, calculations, and results—is available at **www.shortspansteelbridges.org**.)

The final LCC database, which serves as the basis for the study, consists of 1,186 state bridges out of the 6,587 built between 1960 (modern era for prestressed concrete and steel construction techniques) and 2010—i.e., 18% of the PennDOT inventory.

Table 1: Life Cycle Costs

Bridge Type	All Bridges			Bridge Length 140 ft or Less			
	Bridges in Database	Deterioration Rate	Avg Life (years)	Bridges in Database	Initial Cost ($/ft²)	Capitalized Costs ($/ft²)	Avg Life (years)
Steel Rolled Beam	54	-7.11%	81.6	27	222.08	266.24	82.5
Steel Plate Girder	144	-8.14%	80.0	18	257.19	311.26	81.3
P/S Box – Adjacent	282	-8.13%	73.8	240	235.03	292.38	74.0
P/S Box – Spread	397	-7.99%	79.5	325	225.14	272.20	80.8
P/S I Beam	309	-8.38%	73.3	98	231.20	281.64	77.2

The initial costs, LCCs, and future costs of the 1,186 bridges in the database were examined with respect to variability in bridge type, bridge length, number of spans, and bridge life. Calculations to compare the five types of bridges in the study included:

- **Historical bridge initial and maintenance costs.** These were converted to present-day dollars using construction cost indices. Future costs were discounted at a rate of 2.3%. The LCC analyses used the perpetual present value cost, or capitalized cost, of bridge alternatives for an equivalent comparison between each bridge of the bridge types. Capitalized cost is the present value cost of continuing the bridge into perpetuity.
- **Deterioration rate.** To model the deterioration rate, it was assumed the superstructure condition rating decreased linearly over time based on the average deterioration rates of the 6,587 bridges in the PennDOT inventory for each bridge type.
- **Bridge life.** To estimate the remaining life for each bridge, it was assumed the bridge would be replaced when the superstructure condition rating reached 3 given the current condition and the deterioration rate.

Research Results

Careful analysis of the data demonstrated that:

- Steel I-beams have the lowest average deterioration rate (Table 1) with a deterioration rate of 7.11% of a condition rating per year.
- Steel I-beams have the longest average expected life (over 81 years). A useful method to analyze bridge life is to consider the probability that a bridge will last at least 75 years, the expected life for a bridge. Figure 1 shows the cumulative density function for bridge life for all of the bridges in the database, assuming the life is normally distributed. There is a 73% probability that a steel rolled beam bridge will last at least 75 years.
- Steel I-beams have the lowest average initial and capitalized costs (Table 1) for short-span bridges (defined as up to 140 ft). Steel plate girder bridges have the highest average costs, but this would be expected for these short spans since plate girder bridges are not as economical below about 80 ft.
- All five types of bridges are competitive for initial costs, future costs, life-cycle costs, and bridge life. Figure 2 shows the capitalized cost probability density function for the statistical properties of all of the bridges in the database for the five types of bridges assuming the costs are normally distributed. With the relative average costs for a given bridge project, any of the five types may result in the lowest LCC.

Overall, the results show that steel performs well and is a competitive and economical option in the short-span market and that owners should consider steel alternatives for short-span bridges.

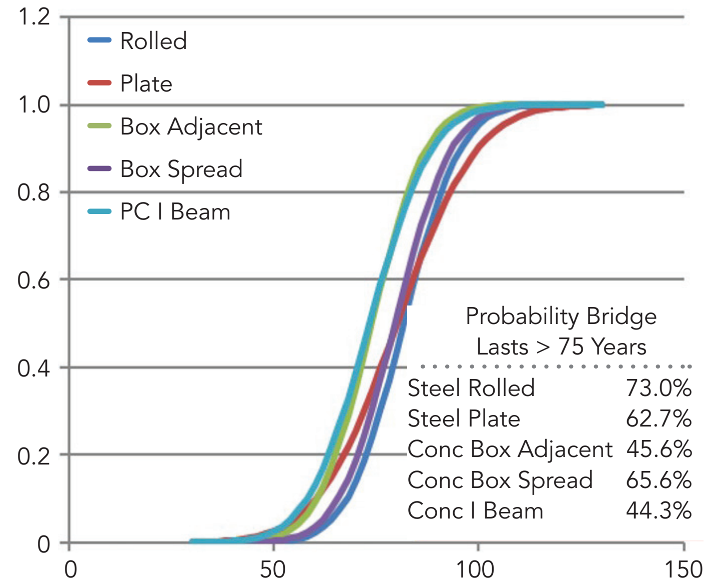


Fig. 1. Cumulative density function for bridge life (all bridges).

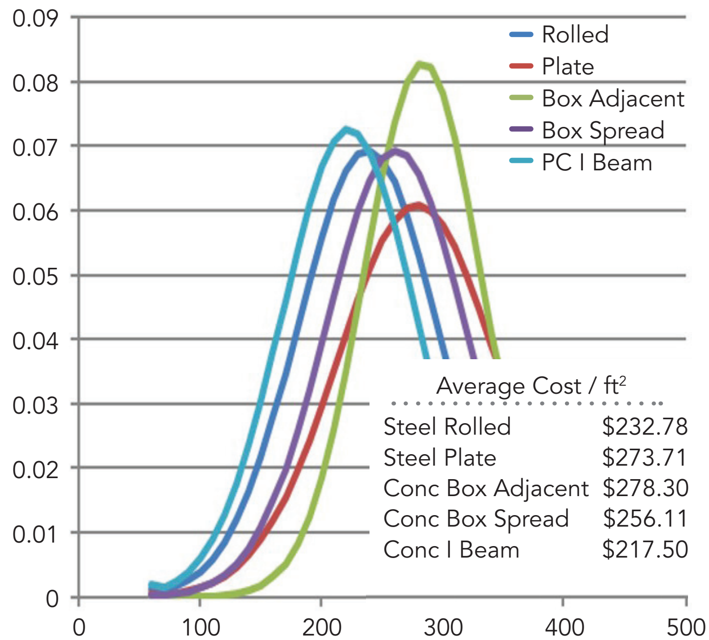


Fig. 2. Probability density function for capitalized costs (all bridges).

Pennoni

above: One of the six bridges in Philadelphia's Vine Street Expressway (I-676) Reconstruction Project.

below: An Anchor Bay Drive bridge in St. Clair County, Mich., one of three galvanized steel press-brake-formed tub girder bridges bundled for the project.

Both short-span steel bridge projects were 2020 NSBA Prize Bridge Award winners. Read about all the winners in the July 2020 issue at **www.modernsteel.com**.


TEG Engineering

Significance

Again, for years assumptions and anecdotes have served as the primary sources of information (or misinformation) on the LCC of steel and concrete bridges, especially short-span bridges—which happen to comprise most of the bridge inventory in the United States. County engineers and officials from state departments of transportation continue to struggle with balancing limited funding and an increased demand to replace the country's aging bridge infrastructure. They are also challenged with incorporating sustainable and cost-effective design and engineering practices into their projects.

The results presented in this study provide them with a tool to assist in making their bridge material choices. Importantly, they are no longer dependent on anecdotes, but now have data to back up their analyses. The need exists for a more comprehensive database of costs for different types of bridges over a diverse set of circumstances, but the research summarized here provides a valid first step—and again, verifies steel as an excellent choice for short-span bridge projects. ■

Advice on classifying system-redundant members.


Trey Cambern/Courtesy of HNTB

That's Not Fracture-Critical!

BY BRANDON CHAVEL, PE, PHD, AND JASON LLOYD, PE, PHD

The Broadway Bridge over the Arkansas River in Little Rock, Ark., a 2020 NSBA Prize Bridge Award winner (to read about it, see the article from the July 2020 issue of *Modern Steel Construction* and on page 88 of this reprint).

THE STEEL BRIDGES we design and build today are not your parents' bridges—so let's stop treating them that way.

What do we mean by that? To put it simply, we don't need to overuse fracture-critical designations.

Bridge engineers and owners now have the resources available to them to remove FCM designations for in-service inspection and recognize system redundancy, allowing them to more efficiently manage resources for steel bridge inspections. Additionally, advances in analysis tools can enable engineers to assess a bridge as a full 3D system, allowing them to consider redundancy and fracture control in a much more integrated fashion.

A Brief History of Fracture Control

To understand where we are going, let's first look at where we've been as it relates to design, fabrication, and in-service inspection of FCMs. Research in the 1970s related to the fatigue and fracture limit states resulted in significant additions to the 1974 American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications, including Charpy V-notch (CVN) testing requirements to ensure a minimum toughness (i.e., resistance to fracture in the presence of a crack) at the lowest anticipated service temperature of the non-load-path redundant member. Also in 1974, the first comprehensive fatigue design provisions were added to the AASHTO bridge design specifications, introducing the fatigue categories and their respective fatigue resistances. As such, steel used in bridges designed prior to 1974 was not subjected to the CVN requirements or the fatigue provisions we design for today.

In 1978, AASHTO published the first edition of the *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*, and this became known as the AASHTO Fracture Control Plan (FCP). These guide specifications introduced the term "fracture-critical" and further distinguished such members to have more stringent CVN requirements than were published in AASHTO M270/ASTM A709; reduced the permissible fatigue stress ranges for fracture-critical members; and introduced more

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Courtesy of Alfred Benesch & Company

The 130th Street and Torrence Avenue Railroad Truss Bridge in Chicago, a 2014 NSBA Prize Bridge Award winner (to read about it, see the July 2014 issue of *Modern Steel Construction*).

The Speer Boulevard Bridge over the South Platte River in Denver, a 2001 NSBA Prize Bridge Award

stringent fabrication and weld quality requirements. These guide specifications are no longer published by AASHTO, as the provisions have been fully integrated into ASTM A709, the AASHTO *LRFD Bridge Design Specifications*, and Clause 12 of the AASHTO/AWS D1.5 *Bridge Welding Code*.

The Surface Transportation and Uniform Relocation Assistance Act of 1987 expanded the scope of bridge inspection programs to identify FCMs and establish inspection procedures for them. In 1988, a maximum in-service inspection frequency of 24 months for FCMs was defined in the NBIS, as well as the "hands-on" (arm's length) inspection requirement. This inspection frequency was only based on expert consensus, not necessarily on scientific research or statistical modeling. The hands-on requirement for inspection and its frequency can be time-consuming and costly to bridge owners, often requiring traffic closures and disruptions. Furthermore, while hands-on inspection of FCMs was intended to improve public safety, a study carried out for Indiana Interstates revealed that overall congested crash rates were 24.1 times higher than uncongested rates with traffic queues of five minutes or more (for more information, see *Characterizing Interstate Crash Rates Based on Traffic Congestion Using Probe Vehicle Data*, at **tinyurl.com/istatecrashrate**). This queue level can result from several things, including closed lanes on bridges for arms-length inspections.

While legislation and research shaped policy for FCMs, including frequency and depth of inspection, it remains incumbent upon the engineer of record (EOR) to identify FCMs in new design, as well as inspectors in existing bridges. Article 6.6.2 of the AASHTO *LRFD Bridge Design Specifications* states that the engineer "shall have the responsibility for determining which, if any, component is a fracture-critical member."

Several industry improvements have occurred since the establishment of the FCP in 1978, as well as following the definition of FCM inspection requirements in 1988. These include improved materials, design and detailing methods, and fabrication practices, along with the advances in the analysis tools that engineers can employ to consider 3D system behavior. In fact, since the implementation of FCP standards, there have been no known fractures of FCMs designed and fabricated to these FCP standards (for more information, see the fourth quarter 2019 AISC *Engineering Journal* article "Simplified Transformative Approaches for Evaluating the Criticality of Fracture in Steel Members" at **aisc.org/ej**). As such, the industry has realized that these improvements and advances can provide a way to better define FCMs for new designs and to reevaluate past FCMs designations.

Enter System-Redundant Members

In June 2012, FHWA issued a Memorandum, *Clarification of Requirements for Fracture Critical Members* (**fhwa.dot.gov/bridge/120620.cfm**), which introduced the new member classification of system-redundant members (SRMs). The FHWA Memorandum defines an SRM as a member that receives fabrication according to the AWS FCP, but does not need to be considered an FCM for in-service inspection. SRMs are to be designated on the design plans with a note indicating that they shall be fabricated in accordance with AWS D1.5 *Bridge Welding Code* Clause 12 and using steel meeting fracture-critical toughness requirements. With this memo, the FHWA has provided bridge owners a means for removing fracture-critical member inspection requirements from certain non-load path redundant structures, allowing a better allocation of inspection resources and reducing life cycle inspection costs of the bridge.

In the Memorandum, the FHWA recognizes that currently available refined analysis techniques can provide a means to more accurately classify FCMs for new designs and to reevaluate existing bridge members that were previously classified as fracture-critical on the record design documents. When a refined analysis demonstrates that a structure has adequate strength and stability sufficient to avoid partial or total collapse and carry traffic in the presence of a completely fractured member (by structural redundancy), the member does not need to be considered fracture-critical for in-service inspection protocol and can be designated as an SRM. The criteria and procedures for the refined analysis and subsequent evaluation should be agreed upon between the engineer and owner. The assumptions and analyses conducted to support this determination need to become part of the permanent inspection records or bridge file so that it can be revisited and adjusted as necessary to reflect changes in bridge conditions or loadings. Additionally, the owner must verify and document that the materials and fabrication specifications of any existing bridge assessed for structural redundancy would meet the FCP. Again, an SRM is a member that must be fabricated according to AWS D1.5 Clause 12 (FCP requirement) but once in-service, it will not need to be routinely inspected at arms-length because it is not an FCM.

However, it should be noted that non-load-path redundant tension members in existing bridges that were not fabricated to meet the modern FCP introduced in 1978 are not eligible for consideration as SRMs at this time.

winner (to read about it, see the October 2001 issue of *Modern Steel Construction*).

Milwaukee's Marquette Interchange twin-tub girder bridge project (to read about it, see "Steel Bridge News" in the March 2007 issue of *Modern Steel Construction*).

In order to obtain the SRM classification, the owner has to demonstrate that the structure has adequate strength and stability sufficient to avoid partial or total collapse and carry traffic in the presence of a fractured member. Once this is done, the owner must submit the detailed analysis and evaluation criteria that are used to conduct the study for review by FHWA Office of Bridges and Structures. The submittal is to be sent through the local FHWA Division Office, who will then forward it to the FHWA Office of Bridges and Structures. Once reviewed and FHWA Office of Bridges and Structures indicates their agreement with the criteria, these criteria can then be employed by the owner systematically on their inventory.

The FHWA Memorandum provides a path to design new steel bridges, and evaluate existing steel bridges, that have non-load-path redundant tension members and adequate system-level redundancy such that the bridge will not collapse and can safely support live load.

Determining SRMs

So where can a bridge owner or engineer get help on determining SRMs? AASHTO's *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (**tinyurl.com/specanaid**) is a tool that allows owners to take advantage of previously unexploited system-level redundancy and efficiently allocate resources to provide better infrastructural solutions to the public.

Released in 2018 and available at **www.aashto.org**, this new *Guide Specifications* document tackles a complex problem: characterizing the demand and capacity of a structure in which a primary steel tension member is assumed to have failed. For a system to be considered redundant, two fundamental concepts regarding load were followed. First, the bridge cannot be expected to operate as reliably in the faulted condition as in the pristine condition. Second, the bridge must be able to survive the failure event and provide service in the faulted state. A February 2020 *Modern Steel Construction* article, "Revisiting Redundancy: Part Two" (**www.modernsteel.com**), further explains the new *Guide Specifications*.

Non-load-path redundant tension members evaluated and meeting the criteria of the *Guide Specifications* will be deemed acceptable for consideration as SRMs in accordance with the 2012 FHWA Memorandum. However, as noted previously, the owner is still required to submit the detailed analysis and evaluation conducted per the *Guide Specifications* for review by the FHWA Office of Bridges. While there may be an additional design cost associated with the required analysis and evaluation, a life cycle cost savings can be realized by the owner as SRMs do not need the calendar-based hands-on in-service inspections required for FCMs.

Alternative Path to SRM Classification

The 2012 FHWA Memorandum does allow bridge owners to choose an appropriate analysis and evaluation method on their own for SRM classification and are not necessarily bound to the *Guide Specifications*. Of course, the chosen analysis and evaluation methods should be founded in suitable research and investigation. Two bridge owners have developed their own methodology (that have been accepted by the FHWA Office of Bridges and Structures) and have been codified within the owner's bridge design specifications for future use. Two examples of how an owner can develop their own methodology and obtain FHWA acceptance are provided in the following two articles. The first involves the Texas Department of Transportation (TxDOT) working in conjunction with the University of Texas at Austin to develop and implement a methodology to design and evaluate twin-tub (trapezoidal box) girder bridges, that provides adequate system redundancy in the unlikely event of bottom tension flange and web fracture. In the second example, the Wisconsin Department of Transportation (WisDOT), working with Purdue University, has developed an approach to evaluate system redundancy in existing and new twin-tub girder bridge systems.

It is very important to note that other bridge owners can adopt either of these methods, or other FHWA-approved SRM methodologies, as their own, thereby avoiding much of the administrative or technical criteria required of the initial owner. The bridge owner will need to obtain formal approval of the chosen method from the FHWA Office of Bridges and Structures in order to evaluate the owner's particular bridge or set of bridges.

The steel bridge industry has a long, proven history of reliability, durability, and sustainability. Decades of research have resulted in further improvements to materials, detailing practices, analysis tools, and fabrication processes that can be integrated with an in-service inspection program that supports even more efficient and reliable steel bridges. The classification of SRMs provides many advantages to owners allowing them to optimize designs, more efficiently manage resources for in-service inspections, improve inspection worker safety, and further increase safety for the traveling public. ■

Again, see the following two articles for two examples of how owners can develop their own methodology and obtain FHWA acceptance.

Redundancy Made Simple

BY CEM KORKMAZ, PhD, AND ROBERT CONNOR, PhD

Milwaukee's "Zoo Interchange." Mulitple twin-tub girder bridges were analzyed for the Purdue/WisDOT study.

Image from Google Maps

STEEL TWIN-TUB-GIRDER BRIDGES have become a popular choice for curved bridges, owing to their high torsional rigidity.

Currently, all two-girder bridges are classified as having fracture-critical members (FCMs), thus potentially subjecting them to expensive arm's-length biennial field inspections.

However, recent research performed by Purdue University has resulted in a new, simplified approach for designing twin-tub girder bridges as having structurally redundant members (SRMs) without the necessity of explicitly modeling fracture in a finite element analysis (FEA). This approach was developed using the procedures, loading criteria, and failure criteria included in the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members* (hereafter referred to as the AASHTO *SRM Guide Specifications*), meaning that bridges designed using this simplified approach will satisfy this document. The research showed that twin-tub girder bridges often possess significant reserve capacity even when one girder is completely severed.

Eighteen multi-span twin-tub girder bridge units (a total of 2.4 miles and 70 spans) located in the state of Wisconsin DOT (WisDOT) were primarily used to develop this proposed simplified guidance along with knowledge gained by analyzing bridges located in other states. Guide limitations were imposed on a number of geometric characteristics to ensure future designs exhibit similar behavior in the faulted state as the multi-span twin-tub girder bridge units analyzed for the state of Wisconsin. The ratio of the length of the span (where the fracture is assumed to occur) to the pre-fracture dead load displacement (of that span) was found to heavily influence the overall load redistribution characteristics of the bridge.

Bottom line, if the simplified guidance is met, future twin-tub girders can be automatically classified as having SRMs without the necessity of explicitly modeling fracture via FEA. Thus, if a bridge is designed and detailed to meet the proposed criteria, acceptable post-fracture behavior is ensured.

Guide Limitations

Geometric limitations based on the types of bridges analyzed were developed to ensure the desired post-fracture behavior would be achieved, and are as follows:

- Minimum of a two-span continuous composite bridge with properly detailed shear studs (there is no upper limit on the number of continuous spans)
- Total deck width shall be no more than 50 ft and a maximum of three design lanes.
- Maximum center to center girder spacing is 25 ft
- Web vertical height must be between 60 in. and 90 in.
- Interior span lengths must be between 70 ft and 250 ft, and exterior lengths shall be between 100 ft and 200 ft
- Ratio of adjacent span length to assumed-fractured span length must be between 0.60 and 1.70
- The radius of curvature over the longest span length is no more than 1.85
- The bridge supports must all have a skew angle of less than 10°

The ratio of span length (L_F) to pre-fracture (unfactored) dead load displacement (D_F) has been found to be a useful predictor in providing insight into the expected post-fracture behavior. If the displacement is high compared to span length, there will likely be moderate to significant inelastic behavior, and the methodology may not be able to accurately estimate the behavior. Based on the overall observed behavior, it is apparent that as the flexibility of the bridge in the *unfaulted* stated increases, so does the level of damage in the *faulted* state. In fact, the authors believe this is somewhat intuitive. In order to ensure acceptable performance, a limit was selected based on the worst (i.e., most flexible) performing bridge while adding some conservatism. Hence, using the limit of $L_F/D_F \geq 300$, it has been determined that this methodology can be applied. This same limit (i.e., $L_F/D_F \geq 300$) can be conservatively applied to interior spans as well.

A simplified approach for designing system-redundant members in composite continuous twin-tub girder bridges.

Proposed Design Guidance

An attractive feature of this approach is that it simply uses the pre-fracture resistance capacities under the AASHTO LRFD Strength I load combination and does not require the engineer to explicitly model the fracture or identify the location that would be critical. This was considered during the development of the procedure and is effectively "built into" the approach. The discussion below will show how post-fracture demands (i.e., those due to Redundancy I and II in the *faulted* state required by the AASHTO *SRM Guide Specifications*) are satisfied by setting additional limitations on the demand/capacity ratios associated with the Strength I loading in the *unfaulted* state. As stated, this included 18 multi-span twin-tub girder bridge units in the state of Wisconsin. The FE analysis results were used to obtain the post-fracture demand/capacity ratios under the Redundancy I and II load combinations. These ratios were compared to the demand/capacity ratios under the familiar Strength I load combination.

In many cases, the demand/capacity ratio in the *faulted* state under the Redundancy load combinations were very low. In addition, in many cases, the demand/capacity ratio in the *faulted* state under the Redundancy load combination was almost always less than it was under Strength I in the *unfaulted* state. In a few isolated cases, the ratio in the *faulted* state exceeded the ratio in the *unfaulted* state, but only by a few percent. Hence, as will be shown, many failure modes listed below will not need to be considered under Redundancy load factors in the *faulted* state. The demand/capacity ratios under the Strength I load combination do provide some insight into the outcome following a fracture. However, they cannot be used directly. In other words, one cannot simply assume acceptable behavior if the Strength I demand/capacity ratios are less than 1.0 in the *unfaulted* state. After a detailed evaluation of the data and all failure modes in the bridges, it was found that setting additional limits on the Strength I demand/capacity ratios in the *unfaulted* state resulted in acceptable performance (e.g., limiting $D/C \leq 0.8$ for some limit state during design). The proposed guidance explicitly addresses all the failure modes defined in the AASHTO *SRM Guide Specifications* though they are handled "behind the scenes" to the user.

All the twin-tub girder bridges analyzed in Phase I have multiple full-depth & full-width intermediate diaphragms and continuous spans. These features provide additional load paths and help to make the bridges redundant, thereby avoiding many failure modes that simple-span bridges and continuous bridges without full-depth and full-width intermediate diaphragms are likely to experience following the fracture of a tub girder. Minimum section details and the locations and number of intermediate diaphragms needed to ensure adequate load transfer in the *faulted* state are stipulated in the methodology.

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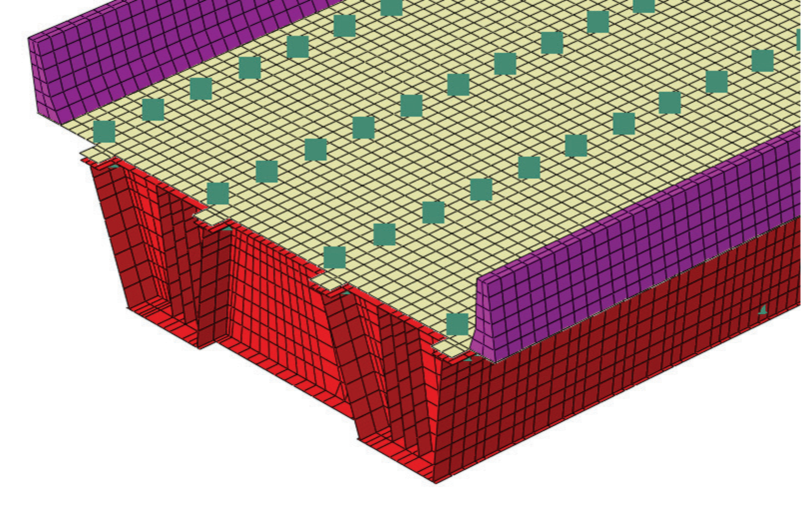
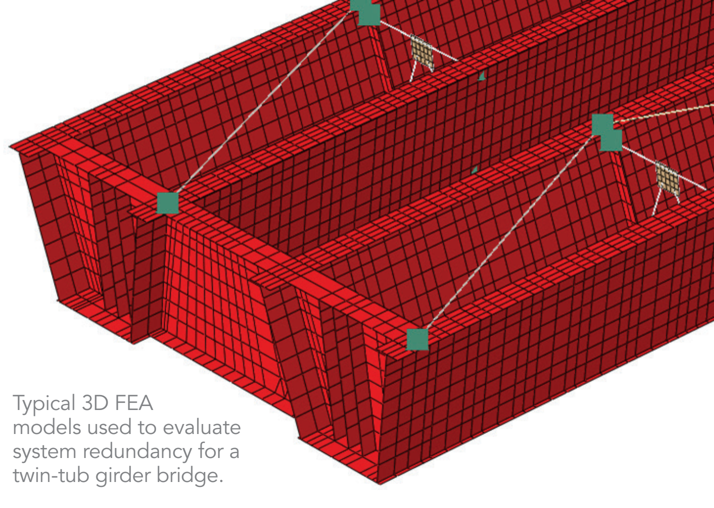




Typical 3D FEA models used to evaluate system redundancy for a twin-tub girder bridge.

Further, the following twin-tub girder members and/or components shall be designed as a minimum to satisfy:

1. The shear stud provisions
2. The provisions for intermediate diaphragms
3. The bottom flange buckling resistance provisions
4. The maximum positive moment flexural resistance

The study revealed that the following failure modes need not be explicitly considered under the Redundancy load factors in the faulted state if the above are satisfied:

1. Web shear buckling
2. Deck related failure modes due to flexure, shear, and torsion
3. Support bearing failure due to excessive reactions and excessive horizontal displacements
4. Excessive vertical displacement in the faulted state
5. Brace failures

Shear Stud Provision

Properly designed and detailed studs have also been shown to be critical in the post-fracture performance of twin-tub girder bridges. The superior ability of composite steel bridges to transfer load is documented in "Modeling the Response of Fracture Critical Steel Box-Girder Bridges," Report No. FHWA/TX-10/9-5498-1, which was based on full-scale experiments in a simple-span twin-tub girder bridge that underwent failure of the bottom flange and webs of one of the tub girders. In order to increase ductility for concrete breakout capacity, shear studs shall extend a minimum of 2 in. above the bottom layer of reinforcement. The proposed methodology specifies the required placement and geometry of intermediate diaphragms to avoid shear stud pull-out. It is also noted that the behavior of the shear studs (i.e., the tension demand) was found to be directly affected by the pre-fracture (unfactored) dead load displacement at the location where the first intermediate diaphragm is located. There were no other failures in any of the bridges evaluated when the first diaphragm was located where the dead deflection was less than L/500. Therefore, it was proposed that the first diaphragm be placed as close as practical to the location where the pre-fracture dead load deflection is less than L/500 to avoid shear stud failure.

It was found that when all other criteria contained in these proposed guidelines are satisfied, the normal AASHTO shear stud design will ensure adequate performance in the faulted state. Since the greatest longitudinal spacing that was included in the study was 22 in., this was selected as an upper limit when three shear studs are used transversely. In cases where two studs are to be placed transversely, it is proposed to simply use a maximum longitudinal spacing for two studs that is ⅔ of the maximum longitudinal spacing used for three studs, or 14 in. (⅔ × 22 in. = approximately 14 in.). Based on the AASHTO *SRM Guide Specifications*, the minimum distance between the outermost stud and the haunch edge should be 1.5 in.

Provisions for Intermediate Diaphragms

The load after fracture is primarily redistributed from the *faulted* girder to the intact girder through the intermediate diaphragms. The diaphragms were capable of transferring both shear and moment during post-fracture behavior, and in most cases had substantial reserve strength themselves. Further, the FEA also confirmed these diaphragms also possessed adequate stiffness to transfer the load to the intact girder. Results indicated that the top and bottom flanges of the diaphragms should be at least the same as the smallest top flange used in the longer exterior span girder. While this is conservative, it will provide adequate stiffness and hence load distribution. Similarly, it is also proposed that the web sections of the diaphragms be equal to the minimum web section of the longer exterior span exterior girder. The connections should be designed using normal AASHTO procedures.

The optimal number and location of the diaphragms in a span were studied to understand how to 1. distribute the loads between the intact and fractured spans; 2. reduce the post-fracture moment at the pier; 3. minimize the damage to the deck; and 4. eliminate shear stud pull-out failures. It is also very important to note that the number and locations of the diaphragms have a significant influence on the distribution of the negative moment transferred to the pier between the girders. In short, the deck alone is not capable of reliably distributing the moments between the fractured and intact girder when considering the negative moment at the pier. (While the deck may possess strength through yield line analysis, it does not provide enough stiffness to transfer the load to the intact girder as the thin deck is far less stiff than the tub girders themselves.)

The parametric study has confirmed that properly spaced and detailed diaphragms are required in multi-span bridges. It was found that the placement and quantity of the intermediate diaphragms can be easily determined in relation to the pre-fracture dead load deflection. For exterior spans, if the dead load deflection at 30% of the span length (0.3L) from the abutment is less than or equal to L/500, two intermediate diaphragms are recommended. The first diaphragm should be placed between 0.3L and 0.4L and should not be located beyond the location where the displacement is equivalent to L/500. The second diaphragm should be placed symmetrically within the same span. If the deflection at 30% of the span length (0.3L) is more than L/500, the study found that a minimum of three intermediate diaphragms should be placed in the span. The first diaphragm should be placed as close as practical to the location where the deflection is L/500. The second diaphragm should be placed at mid-span. The third diaphragm should be placed symmetrically with the first diaphragm within the span. For interior spans, two intermediate diaphragms should be placed as close as is practical to the third points of the span. The intermediate diaphragms of interior spans should possess the same cross-section as the exterior span diaphragms.

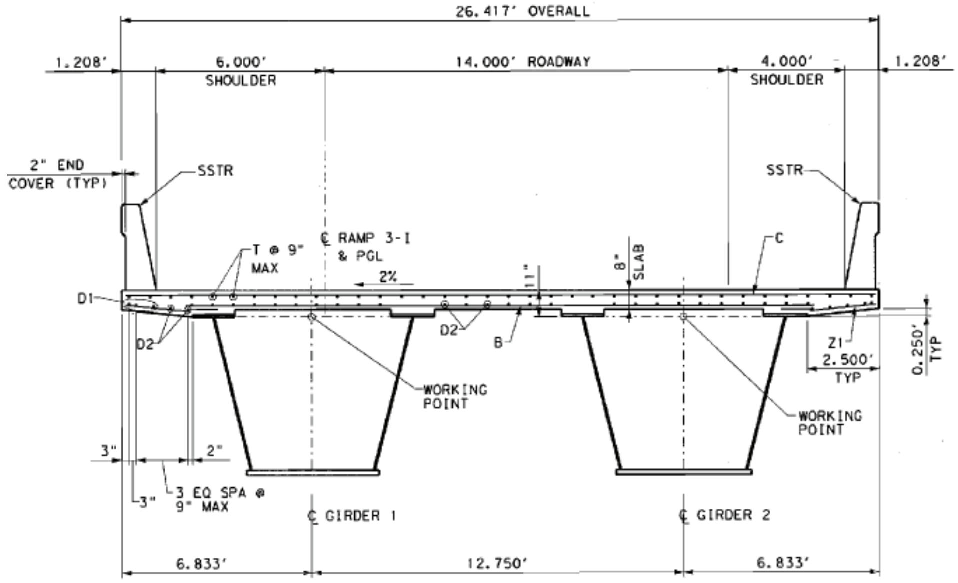
Bottom Flange Buckling Resistance in Negative Moment Region

The parametric study has demonstrated that bottom flange local buckling in the negative moment region is the most likely failure mode in the faulted state due to the redistribution of positive moment (to the negative moment region) in the span that contained the assumed fracture. It is also important to note that the most critical section is wherever the bottom flange section changes, such as at a flange transition to a thinner section away from the pier, as shown in Figure 1 (next page). Obviously, the thinner section's capacity needs to be sufficient to avoid local bottom flange buckling in the post-fracture behavior. In order to eliminate this form of failure, locations of flange thickness changes are recommended as a function of span length in the approach. Thus, using a very simple criterion, this failure mode can be prevented.

It has also been observed that the maximum pre-fracture dead load displacement is a strong indicator of the potential for bottom flange buckling in the *faulted* state. According to the AASHTO Report *A Simplified Approach for Designing SRMs in Composite Continuous Twin-Tub Girder Bridges*, there is really no need to check the sections between the pier for a distance of 0.2L away from this pier, since all of the demand/capacity ratios under Strength I are generally higher than those produced in the *faulted* state using the Redundancy load combinations. Thus, no additional criteria appear needed in this region. However, to avoid the high demand/capacity ratios in the faulted state at a flange transition between 0.2L and 0.3L away from a pier, the pre-fracture demand/capacity ratio should be less than 0.7 for the Strength I load combination. The sections more than 0.3L away from a pier do not need to be checked. Additional FE analysis was evaluated on the criticality of buckling in the negative moment region when a fracture is assumed to occur within an interior span. Due to the double cantilever behavior at an interior span, the effects were found to be insignificant and do not need to be considered. In summary, it was observed that fracture in an end or exterior span was more critical than a fracture within an interior span.

Flexural Yielding in Positive Moment Region of Flanges in Intact Girder

After fracture occurs, a significant amount of the load is redistributed from the fractured girder to the intact girder. In the fractured span, the positive moment flexural resistance of the intact girder should be checked. The most critical location for this check is at the maximum positive moment closest to the assumed fracture. When the intact girder substantially exceeds its elastic moment capacity, the post-fracture moment redistribution is difficult to estimate with simplified methods. For example, a considerable amount of plasticity in the positive moment region causes more moment to be redistributed to the cross sections close to the pier. The overall method developed herein ensures there will be little-to-no yielding in the positive moment region of the intact girder. When the pre-fracture demand/capacity ratio in the exterior girder under Strength I load combinations is less than 0.8, no plasticity was observed in intact girder for post-fracture behavior. It is therefore proposed to limit the pre-fracture demand/capacity ratio to less than or equal to 0.8 for both girders.



A sample twin-tub girder bridge cross section.

Easy, Reliable Design

The simplified guideline and associated design checks will ensure that newly designed twin-tub girder bridges meet the requirements of AASHTO *SRM Guide Specifications* without the need for full non-linear FEA. The updated method was developed using these specifications and approved by FHWA for analysis and design of twin-tub girder bridges. Thus, the simple guidance in this project is sufficient to classify continuous composite twin-tub girder bridges within the above stated geometric limitations as having SRMs.

The methodology requires future twin-tub girder bridges to have intermediate diaphragms in order to be redundant. The full-depth intermediate diaphragms used by WisDOT also appear to reduce the likelihood of shear stud failures, bottom flange buckling at (or close to) support, deck and parapet crushing, deck reinforcement yielding, lateral brace failing, and torsional buckling in the intact girders. These diaphragms were shown to be very effective in transferring load in the *faulted* condition and significantly contributed to the excellent system performance of the bridges in the Wisconsin inventory.

The guideline provided in Appendix-A of the AASHTO Report (see *A Simplified Approach for Designing SRMs in Composite Continuous Twin-Tub Girder Bridges* for more information) presents a method on how twin-tub girder bridges can be easily and reliably designed as redundant structures. ■

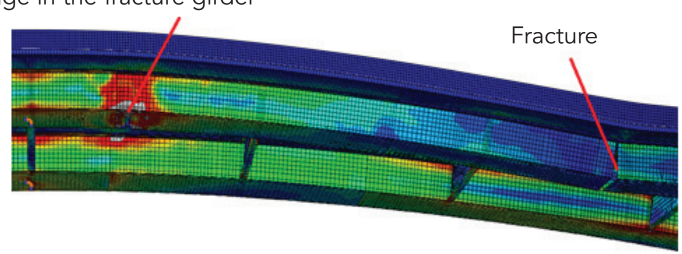
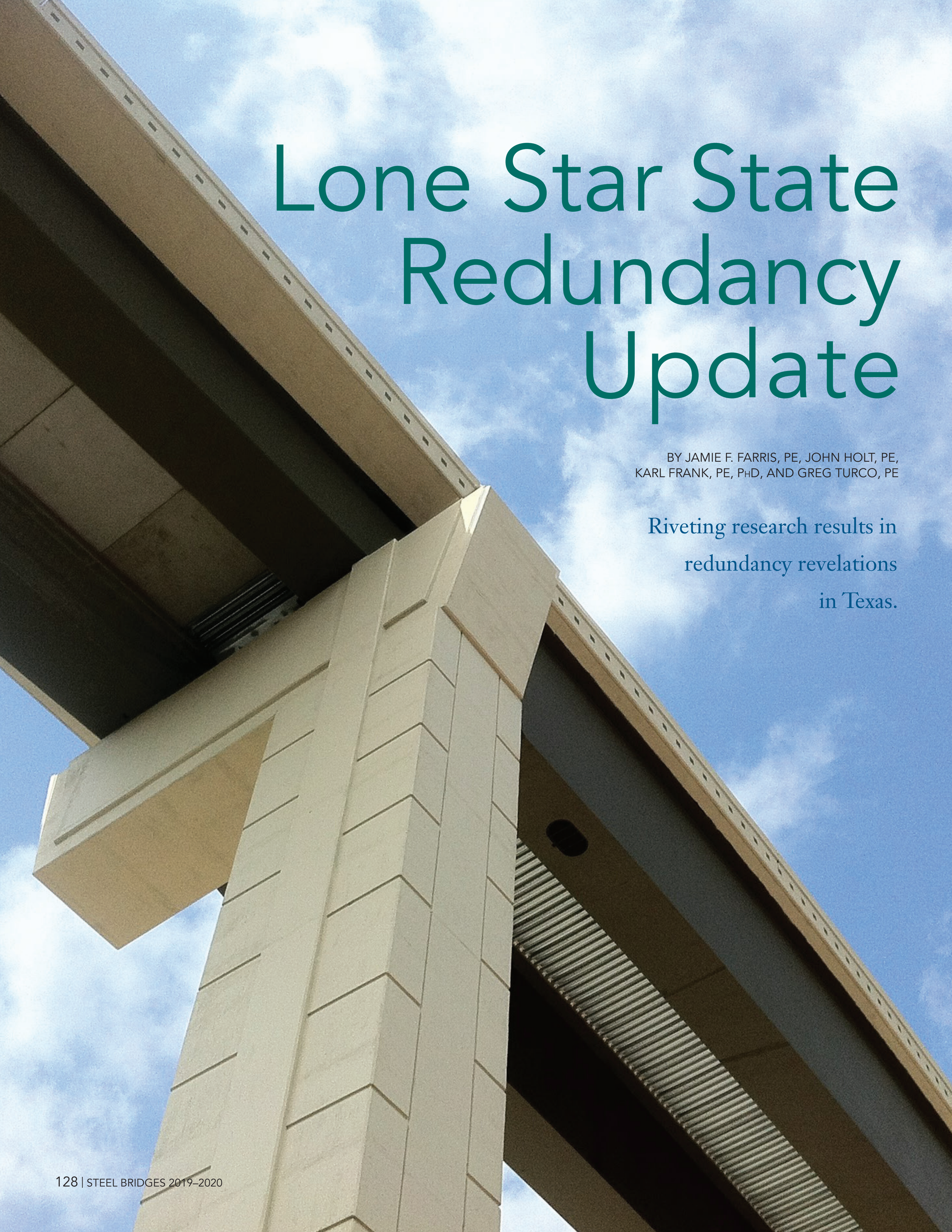


Fig. 1. A thinner section local bottom flange buckling at the section change in a fractured span.



Lone Star State Redundancy Update

BY JAMIE F. FARRIS, PE, JOHN HOLT, PE,
KARL FRANK, PE, PhD, AND GREG TURCO, PE

Riveting research results in redundancy revelations in Texas.

A COMMON BRIDGE TYPE historically assumed to include fracture-critical members (FCMs), based on a simplistic load-path redundancy assessment, is the steel twin-tub girder bridge.

These bridges consist of two steel box girders, frequently trapezoidal-shaped, and a concrete deck and are a very effective solution for curved ramps and connectors in multi-level interchanges. The two bottom flanges and webs of a steel twin-tub-girder bridge are considered to be fracture-critical elements in the positive bending moment region.

Texas has more than 480 existing twin-tub girder spans currently in use, and the Texas Department of Transportation (TxDOT) spends more than $2.3 million every two years inspecting twin-tub girder spans—not including traffic control costs, which can be up to $2,000 per span per day for a fracture-critical bridge (FCB) inspection. This added expense of the field inspections limits the use of what is a very efficient structural system.

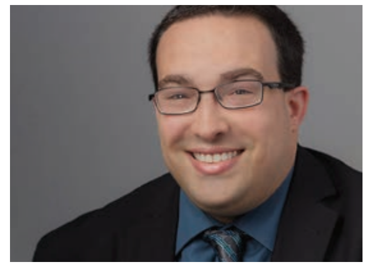
Thanks to recent research, a simplified method for evaluating system redundancy in two-tub girder span bridges has been added to the state's bridge design policy. The *TxDOT Bridge Design Manual–LRFD* now presents an LRFD-based methodology to design spans with two tub girders in cross section such that the span will continue to safely carry traffic after the fracture of one of the girders. The probability of such a fracture for tub girders designed for infinite fatigue life is considered exceedingly small in comparison to the bridge's design life. Therefore, the Texas method addresses the design of a simulated fracture as an extreme event limit state.

UT Twin-Tub Research

Several historical events have shown that severe damage can occur to a bridge without resulting in its collapse. Early research into multiple incidents, including a full-depth fracture of in-service fracture-critical bridges (FCB), suggests that in some cases, a redundant load path does exist for some FCBs. To address concerns that current provisions do not reflect the performance of steel twin-tub girders during a fracture event, TxDOT and the Federal Highway Administration (FHWA) sponsored a research project at the University of Texas at Austin to characterize the level of redundancy that exists in a steel twin-tub girder bridge. The main goal of Research Project 0-5498: "Modeling the Response of Fracture Critical Steel Box Girder Bridges" was to develop guidelines for modeling a bridge's behavior in the event that a critical bottom tension flange fractures. The research included a combination of laboratory testing, experimental evaluation of a full-scale tub girder bridge, and detailed structural analysis.

The tested bridge was taken out of service and reconstructed at the Ferguson Structural Engineering Laboratory (FSEL) at UT to evaluate the redundancy after a series of tests. The experimental bridge's geometry represented a worst-case scenario, as it was a 120-ft-long horizontally curved simple-span bridge with no external diaphragms. The first test included using a linear shape-charge explosive to rapidly cut through the entire width of the bottom flange of the outside girder, simulating a fracture of the flange, with the equivalent of an HS-20 truckload positioned at the most

To address concerns that current provisions do not reflect the performance of steel twin-tub girders during a fracture event, TxDOT and the FHWA sponsored a research project at UT Austin to characterize the level of redundancy that exists in a steel twin-tub girder bridge.



Jamie F. Farris is the Bridge Division deputy director and **Greg Turco** is a Bridge Design Group leader, both with the Texas Department of Transportation. **John Holt** is a senior project manager/regional director with Modjeski and Masters, and **Karl Frank** is a professor emeritus at the University of Texas at Austin, and a consultant.

Fracture-critical inspection of a steel twin-tub girder bridge at night.

severe location over the fracture. A second test on the bridge included a simulated full-depth fracture of the outside girder webs. The bridge was lifted to its original position and temporarily supported by using an external jack system. The equivalent HS-20 truckload was positioned to have the highest possible bending moment acting at the fracture location, and the webs of the damaged girder were then cut using a torch. The tie-rods of the jack system were rapidly cut using explosives, which released the energy stored in the jacks instantaneously. In its damaged state, the mid-span of the girder deflected 7 in., while the deck had a maximum deflection of 3.8 in. The final, third test was conducted to define the ultimate load that the bridge could sustain in the damaged state. The bridge was incrementally statically loaded until it collapsed—after 182 tons of weight was placed on the deck.

The three tests on the experimental bridge clearly demonstrated system redundancy. Data gathered from the experimental testing program were used to validate nonlinear finite element models and develop a simplified procedure to assess the redundancy of steel twin-tub girder bridges in Texas. (For more details on the tests, see the expanded version of this article in the digital edition of this issue at **www.modernsteel.com**.)

Redundancy Case Studies

Following a memorandum that introduced the new member classification of system redundancy member (SRMs) (see the article "That's Not Fracture-Critical" on page 121), TxDOT met with FHWA to discuss a path to move forward addressing steel twin-tub girder bridges using the proposed analytical modeling methods proposed in the 0-5498 research. The research includes a simple method, which assumes a full-depth fracture in one of the two girders, for analyzing steel twin-tub girder bridges. TxDOT bridge design engineers analyzed three existing steel twin-tub bridges using the simplified modeling procedure developed in the 0-5498 research. The three TxDOT case studies represented typical highway flyover steel twin-tub girder bridge configurations. Table 1 summarizes the bridge geometry for each bridge.

The three TxDOT case studies represented typical highway flyover steel twin-tub girder bridge configurations. The results of the

A test simulating a full-depth fracture of the outside girder webs of a twin-tub girder bridge.

Table 1 Bridge Geometries for Case Studies

	Year Designed	Span Lengths (ft)	Overall Deck Width (ft)	Girder Depth (ft)	Centerline Structure Radius (ft)
Case 1	1998	148 – 265 – 190	30	6.5	716
Case 2	2007	199 – 243 – 179	28	6	1,033

redundancy case studies indicated that the intact girders had sufficient bending capacity as well as adequate deck shear strength and shear stud tensile capacity. In all cases, the intact girder failed in combined torsion and shear at the supports, but the margin of inadequacy was small. Compared to the 0-5498 experimental bridge, the three TxDOT study cases had longer span lengths and a sharper horizontal curvature, which lead to a greater dead load, larger eccentricity, and therefore higher torsion and shear at support locations. In addition, none of the three cases passed the simplified method criteria at the support girder section's shear capacity check due to the large torque and shear forces. However, none of the cases failed due to a lack of shear stud tensile capacity. This led TxDOT to believe that new steel twin-tub girder bridges could be designed and detailed for system redundancy by accounting for the large torque and shear forces. Thus, they would not be considered fracture-critical.

Texas Steel Quality Council Task Group

In September of 2016, the Texas Steel Quality Council (TSQC) instituted a Twin-Tub Task Group to develop LRFD-based design specifications that would govern the analysis and design of non-fracture-critical steel twin-tub girder spans. The task group membership reflected the overall structure of the TSQC, with 17 members participating. The TSQC was originally established in 1995 and is a joint owner-industry forum made up of TxDOT inspectors, designers, fabrication, erection engineers, consultant engineers, FHWA bridge engineers, academics, steel bridge fabricators, detailers, trade association representatives, and steel mill representatives. Through the effort of the task group, an AASHTO Ballot Item was developed and presented at several industry and AASHTO meetings around the nation. In the end, AASHTO was not ready to put language in the specifications specifically for twin-tub girder bridges, which led TxDOT to develop language for its own bridge design policy manual and submit to FHWA for approval. Through several conversations and correspondence with TxDOT and FHWA, the TxDOT design methodology to design twin-tub girders for system redundancy was approved by FHWA in late 2019. The FHWA approval means a steel twin-tub girder bridge designed according to TxDOT's design methodology and submitted to TxDOT for approval is recognized as system-redundant by FHWA.

TxDOT Design Methodology

The *TxDOT Bridge Design Manual—LRFD* presents an LRFD-based methodology to design spans with two tub girders in cross section such that the span will not collapse if one of the girders fractures.

The bridge is designed as it normally would be, using the following limit states and exceptions:

- Design for Strength Limit State using a Redundancy Factor, $\eta_R = 1.05$
- Design for Service Limit State
- Design for Infinite Fatigue life for Fatigue and Fracture Limit State

Next, the bridge is designed for member failure. The bottom flange in tension of the critical girder and the webs attached to that flange are assumed to be fully fractured at the location of the maximum factored tensile stress in the bottom flange determined using Strength I load combination. In order to create the worst-case loading scenario, the girder assumed to be fractured is chosen based on its position in the cross section relative to the traffic lanes and its eccentricity to the deck and railing. If the span under consideration is horizontally curved, the girder with the largest radius is assumed to be the fractured girder, and the investigation for system redundancy is limited to end spans of continuous units and all simple spans.

The probability of such a fracture for tub girders designed for infinite fatigue life is considered exceedingly small compared to the bridge's design life. Therefore, the TxDOT method addresses the design of a

The study's third test, in which a bridge was incrementally loaded until it collapsed—after 182 tons of weight was placed on the deck.

One of Texas' nearly 500 twin-tub girder bridges currently in service.

Table 2 Supplement to AASHTO Table 3.4.1-1 to Include Extreme Event III

Load Combination Limit State	DC DD DW EH EV ES PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time				
										EQ	BL	IC	CT	CV
Extreme Event III	γ_p	1.10	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00

simulated fracture with the extreme event limit state. TxDOT revises the AASHTO definition of Extreme Event Limit State to include structural member or component failure. A new load combination is introduced as Extreme Event III, which is defined as a load combination relating to a structural or component failure. Tables 2 and 3 supplement AASHTO Table 3.4.1-1 and Table 3.4.1-2, respectively:

Table 3 Supplement to AASHTO Table 3.4.1-2 to Include Load Factors for Extreme Event III

Type of Load, Foundation Type, and Method Used to Calculate Downdrag	Load Factor	
	Maximum	Minimum
DC: Components and Attachments for the evaluation of system redundancy as specified in the *TxDOT Bridge Design Manual–LRFD*, for Extreme Event III only	1.10	0.90

All load effects during an assumed fracture event due to both permanent and assumed transient loads are then amplified by a factor of 1.20 to simulate the dynamic effects of a fracture on the twin tub girder span(s).

Two types of analysis can be used to evaluate Extreme Event III:

- Approximate structural analysis, as described in Research Report 5498-1: *Modeling the Response of Fracture Critical Steel Box-Girder Bridges*, and the simplified method, as described in the *TxDOT Bridge Design Guide*, for two tub girder bridges are permitted when:
    - Spans do not exceed 250 ft
    - Supports are skewed no more than 20°
    - Horizontal curvature greater than 700 ft
    - The engineer ascertains that the use of an approximate analysis method is adequate

For the approximate analysis to be permitted for spans satisfying the conditions specified above, the entire self-weight of the span under consideration and the entire live load is assumed to be carried by the intact girder after the assumed fracture event. It is assumed that prior to fracture, the fractured girder was carrying 50% of the total dead load and the entire live load on the bridge, and thus it is assumed that the bridge slab must transfer this load from the fractured girder to the intact girder.

- Refined structural analysis, as described in Research Report 5498-1, accounts for the capacity of the intact girder as well as portions of the fractured girder that can still provide structural resistance, such as interior support locations. The load distribution between the intact girder and the fractured girder is realistically modeled. A table of live load distribution coefficients for extreme force effects in each span is not required when evaluating system redundancy, as specified in the *TxDOT Bridge Design Manual*.

A structurally continuous railing, barrier, or median barrier, acting compositely with the supporting components, may be considered structurally active at Extreme Limit State III when evaluating system redundancy as specified in the *TxDOT Bridge Design Manual*.

Under Extreme Event III, live load includes both truck and lane load. The truck is positioned on the bridge deck directly above the presumed fracture location to cause the most severe internal stresses to develop in the assumed intact girder. Consistent with the experimental testing program described in Research Report 5498-1, the number, width, and location of design lanes are taken as the number, width, and location of striped traffic lanes on the bridge. If the future lane configuration is known at the time of design, it should also be considered when evaluating redundancy. It is considered overly conservative to place additional live load in a striped shoulder to represent a parked or disabled truck when evaluating system redundancy. The 1.10 live load factor in the Extreme Event III limit state is considered appropriate for determining system redundancy because of the very low probability of fracture of one steel tub girder in a twin-tub girder superstructure cross section that has been designed for infinite fatigue life.

The intact tub girder and portions of the fractured girder that can still resist load are checked for adequate flexural and shear resistance after the assumed fracture event under Extreme Event III load combination, according to the provisions of the AASHTO Articles. The flexural resistance of the intact girder in regions of positive and negative flexure needs to be checked after the assumed fracture event to ensure that the girder can sustain the load transferred from the fractured girder in conjunction with the self-weight of the intact composite girder. For shear, St. Venant torsional shears are included in the calculation of V_u, where applicable. The concrete deck is also checked for adequate shear resistance to resist the shear due to torsion after the assumed fracture event under the Extreme Event III load combination. Figure 1 depicts the deflected shape of the concrete deck and bending moment diagram, assuming that the shear studs have adequate tensile capacity. The bridge deck is a vital link in the transfer of load from the fractured girder to the intact girder, and the shear studs connecting the deck to the fractured girder must also have sufficient tension capacity. The use of empirical deck design is prohibited due to a lack of research on the behavior of this type of deck design and system redundancy of steel twin-tub girder bridges.

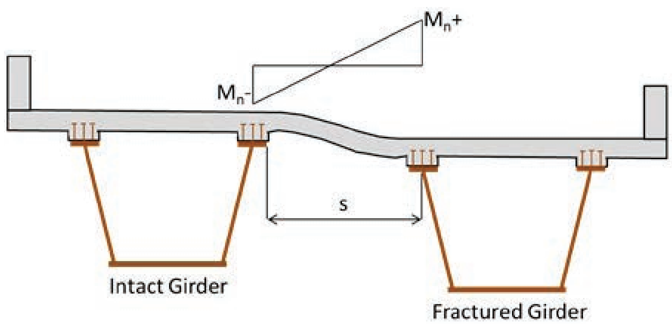


Fig 1. Deflected shape and moment diagram before any failure of shear studs.

End diaphragms and their connection to both tub girders are also checked to ensure adequate resistance to the torque applied to the intact girder after the assumed fracture event under Extreme Event III load combination. Stud shear connectors connecting the deck to the assumed fractured girder are designed to have sufficient tension capacity to develop the plastic beam mechanism in the bridge deck after the assumed fracture event. All shear connectors are detailed to extend above the bottom mat of deck reinforcement.

The radius of curvature must be considered for the intact tub girder. A decrease in the radius of curvature increases the torsion on the bridge, which must be resisted by the intact girder in the event of a fracture of a critical tension flange. Under such conditions, the eccentricity should be computed as the distance from the center of gravity of the loads to the line of the intact girder interior supports. The center of gravity for non-prismatic girders can be determined by using equations in *Guidance for Erection and Construction of Curved I-Girder Bridges* (Technical Report FHWA/TX-10/0-5574-1) modified for the case of tub girders. This applied torque is resisted by a couple generated by the bearings of the two girders—i.e., bearing reactions. The reaction at the bearing of the fractured girder is equal to the torque applied to the intact girder divided by the distance between the bearings of the two girders. If two bearings per girder are used, then the torque applied to the intact girder could be distributed to its two bearings.

Diaphragms

TxDOT requires steel twin-tub girder bridges to include internal and external diaphragms at all supports. The diaphragms and connections must be designed to resist the torsional moment in the assumed intact girder, and also to transmit vertical and lateral forces to the bearings during and after an assumed fracture event. In addition, they must be designed to act compositely with the slab with shear connectors. Also, at least two permanent external intermediate diaphragms, designed according to AASHTO and Extreme Event III, must be provided on each side of the location of maximum factored tensile stress in the bottom flange in the span under consideration determined using Strength I load combination. This is intended to enhance system redundancy by providing additional load paths on each side of the assumed fracture location. In Texas, external intermediate bracing elements are sometimes removed after the deck placement for aesthetic purposes, but with the new requirements they must permanently remain in the structure to provide additional load paths in the event of a fracture.

Detailing

TxDOT also requires several detailing criteria when designing steel twin-tub girder bridges for system redundancy. All details on both tub girders, except for drain holes in the bottom flange and details on the bracing members, are detailed to have a fatigue resistance based on Detail Category C' or higher. Drain holes in the bottom flange (Category D) are detailed to be located at least 20 ft from the location of the maximum tensile stress in the flange determined using the Strength I load combination. Positive restraint and adequate support lengths are provided to keep the superstructure on the substructure after the assumed fracture event. Bearings do not need to be evaluated for this limit state. Finally, structurally continuous barrier railings at least 32 in. in height must be provided and should be considered to be structurally active for the analysis at the Extreme Event III limit state.

Fabrication and Inspection

Twin-tub girder spans satisfying the system redundancy requirements of the *TxDOT Bridge Design Manual—LRFD* are assumed to possess adequate system redundancy at Extreme Event III Limit State. Members or portions within spans that would otherwise have been classified as fracture-critical, when evaluated based on load path redundancy alone, are instead designated in the contract documents as SRMs. They are also not subject to the hands-on in-service inspection protocol for FCMs described in 23 CFR 650. The SRMs are fabricated according to the American Welding Society (AWS) D1.5: *Bridge Welding Code* fracture-control plan (FCP).

Moving Forward

Future twin-tub spans will be designed with the updated methodology and classified as SRMs. TxDOT is currently developing in-house spreadsheet tools to allow for the simple application of the approximate analysis method per research project 0-5498. In addition, prototype models are under development to provide guidance for future redundancy evaluations. A future goal is to have all existing twin-tub girder spans evaluated for redundancy using this methodology. The implementation of this methodology will result in twin-tub girder bridges that are more economical, as the life-cycle costs of future inspections are reduced with the SRM classification.

steelwise

KEEPING CROSS-FRAMES IN CHECK

BY DEVIN ALTMAN, PE AND BRANDON CHAVEL, PE, PHD

When in doubt, don't just make your cross-frames stout.

Devin Altman (**altman@aisc.org**) is a bridge steel specialist and **Brandon Chavel** (**chavel@aisc.org**) is director of market development, both with AISC's National Steel Bridge Alliance.

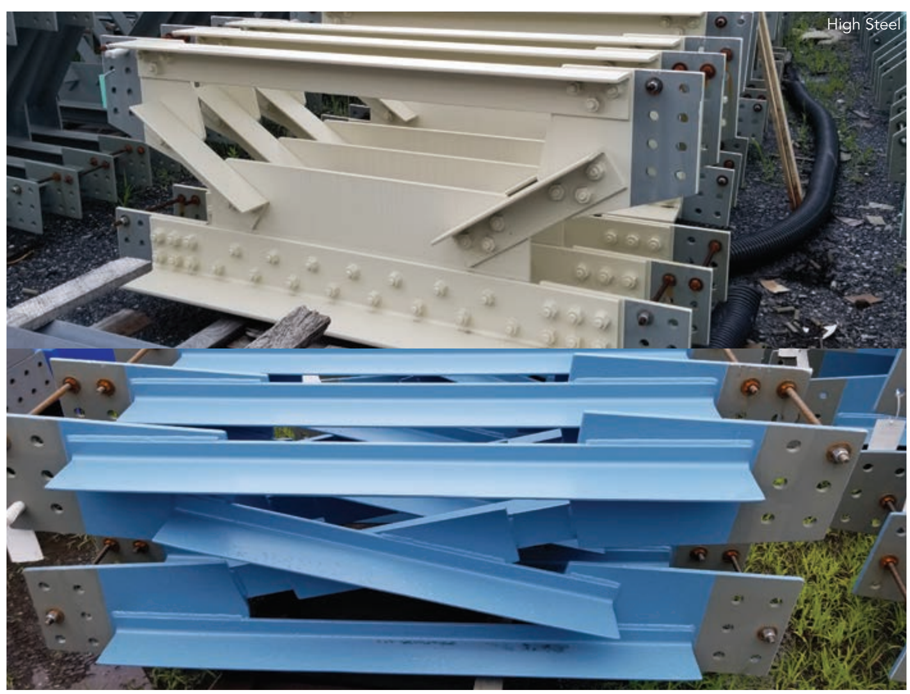
CROSS-FRAMES ARE A big deal—and they're getting bigger.

Cross-frames are important bridge components, as they provide stability to the primary longitudinal girder members and improve the lateral or torsional stiffness and strength of the bridge system during construction and service. They also help distribute gravity loads through the bridge system. In horizontally curved bridges, cross-frames transfer forces between adjacent girders in order to provide equilibrium, resulting in forces that need to be considered by the designer. And in straight bridges, they have been historically designed to transmit wind loads within the structure. Now, however, it seems designers are building overly complex 3D models and obtaining design forces from them.

Over the last few years, the steel bridge industry has seen a general increase in the size of cross-frames used in steel I-girder bridges across the country, in terms of both the individual member sizes and the connections themselves.

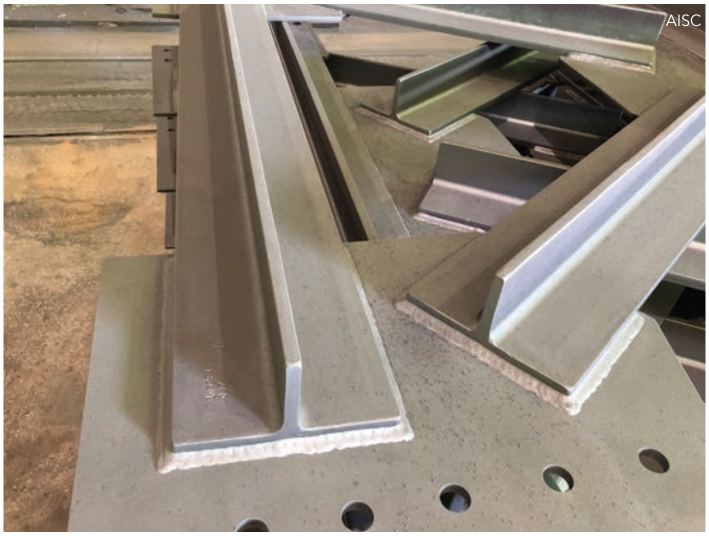
Along with the sizes of the members and connections getting larger, connections that were historically welded are now being bolted in place instead. Furthermore, X-type and K-type cross-frames are being used in situations where a solid bent plate or built-up diaphragm would make more sense from a geometry, fabrication, and installation perspective.

So why are cross-frames getting larger, and why might this create inefficiency—and what can we do to address this issue and ensure that they are sized efficiently?



High Steel

Cross-frames are becoming stouter—in some cases more than double the size of what they need to be to effectively and efficiently perform their job.



A welded member end connection (left) and an X-type cross-frame with welded member end connections (right).

Bigger but not Necessarily Better

One of the main reasons bridge designers have claimed larger cross-frames and their connections are warranted is because of modern fatigue requirements. Fatigue and fracture criteria have been evolving considerably in the AASHTO *LRFD Bridge Design Specifications* and have changed a great deal over the last ten to fifteen years. In 2009, the single fatigue load combination was replaced with Fatigue I (infinite fatigue) and Fatigue II (finite fatigue) load combinations. These different methods were effectively the same as the prior check but stated and arranged differently. Prior to Fatigue I and Fatigue II load combinations, there were no limits for average daily truck traffic (ADTT) in a single lane for infinite fatigue life. Infinite fatigue life has a significantly higher load factor (more than double) compared with finite fatigue life per the AASHTO *LRFD Specifications*.

In the 7th Edition, the 2016 interims increased the previous load factors from 1.50 to 1.75 for infinite fatigue life and from 0.75 to 0.80 for finite fatigue life. This increased the demand for finite fatigue life by 7%, and the demand for infinite fatigue life increased by 17% compared with the prior (2009) AASHTO *LRFD Specifications*. The 2016 interims also changed the fatigue detail category from E to E' for longitudinal fillet welded angle or tee sections connected to a gusset or connection plate (Table 6.6.1.2.3-1), effectively reducing the threshold stress range from 4.5 ksi to 2.6 ksi for cross-frame members welded to stiffeners or gusset plates. This detail category applied to all cross-frame members welded to a gusset plate or connection stiffener, a type that was not originally part of the 5th Edition but was introduced as Detail Category E in the 2010 interims.

This detail category change and reduction in the allowable threshold stress range resulted in a reduced fatigue resistance for typical cross-frames by 41% for finite fatigue limits and 73% for infinite fatigue life levels. These changes in the *LRFD Bridge Design Specifications* came from *The SHRP2 Project R19B – Bridges for Service Life Beyond 100 Years: Service Limit State Design* (Modjeski and Masters, 2015), which studied various aspects of the load and resistance models for calibration of the fatigue and service limit states.

However, with all the requirements stated above, it should be noted that the general consensus amongst the bridge industry is that no cross-frame has failed due to fatigue while in service or caused a failure of a steel bridge girder-system. This anecdotal evidence applies to cross-frames designed today, as well as all the cross-frames designed well-before the Detail Category E' designation was introduced.

Analysis Strategies

So what analysis strategies can designers use to help reduce the size of cross-frames per the AASHTO *LRFD Specifications*?

While the fatigue live load factors have increased, and the nominal fatigue resistance of the welded end connection has decreased, there have been other changes in the AASHTO *LRFD Specifications* that can help to reduce the fatigue design stress range. When a designer uses a refined analysis, these AASHTO recommendations should be considered.

Strategy 1. The AASHTO *LRFD Specifications* 2020/9th Edition Commentary Article C6.6.1.2.1 recommends that the fatigue truck be positioned to determine the maximum range of stress or torque, as applicable, **with the truck confined to one critical transverse position per each longitudinal position throughout the length of the bridge in the analysis.** This is because there is an extremely low probability of the truck being located in two critical relative transverse positions over millions of cycles. This provision allows the designer to use the fatigue live load stress range for the cross-frame members based on the fatigue truck in only one lane at a time, and not in two different transverse positions. The fatigue stress range for cross-frame members *should not* be based on stresses resulting from the fatigue truck in transverse positions 1 and 4, for example (i.e., two critical relative transverse positions). In a refined analysis, the designer need only take the envelope of the maximum fatigue stress ranges caused by the fatigue truck confined in lane 1, then lane 2, then lane 3, then lane 4, and so on. The fatigue live load stress range is, in theory, less under this method of load application than taking the maximum stress range from all of the individual configured lanes loaded differently

A K-type cross-frame with bolted end connections—not recommended unless absolutely necessary.

transversely and longitudinally and used together for the fatigue stress range (this recommendation was added in the 2014/7th Edition). Designers need to be aware of what their refined analysis software is doing. When using a refined analysis, consideration should be given to the different fatigue live load analyses required for girders and cross-frames. A slight adjustment to the analysis steps will contribute to reducing the cross-frame member and connection sizes.

Strategy 2. Designers can also reduce the force demand on cross-frame members in a refined analysis by reducing the member stiffness to $0.65AE$ to account for the connection stiffness and second-order stiffness softening (where A is the area of the cross-frame member and E is the modulus of elasticity for steel). Lowering the stiffness in the cross-frames results in a reduction of the strength design forces and the fatigue load stress ranges in the cross-frame members. The 2014 edition interims introduced the commentary article C4.6.3.3.4, which states: "In addition, the axial rigidity of single-angle members and flange-connected tee-section cross-frame members is reduced due to end connection eccentricities (Wang et al., 2012). In lieu of a more accurate analysis, $(AE)_{eq}$ of equal leg single angles, unequal leg single angles connected to the long leg, and flange-connected tee-section members may be taken as $0.65AE$."

Strategy 3. Designers should carefully consider the use of the Fatigue I and Fatigue II load combinations. In cases where there is low volume truck traffic and the details being considered are not on fracture-critical members, the Fatigue II load combination and its lower load factor may be permissible. In accordance with AASHTO *LRFD Specifications* Article 6.6.1.2.3, when the 75-year single-lane ADTT is less than or equal to the applicable value specified in Table 6.6.1.2.3-2 for the Detail Category under consideration Fatigue II, load combination may be used in combination with the nominal fatigue resistance for finite life.

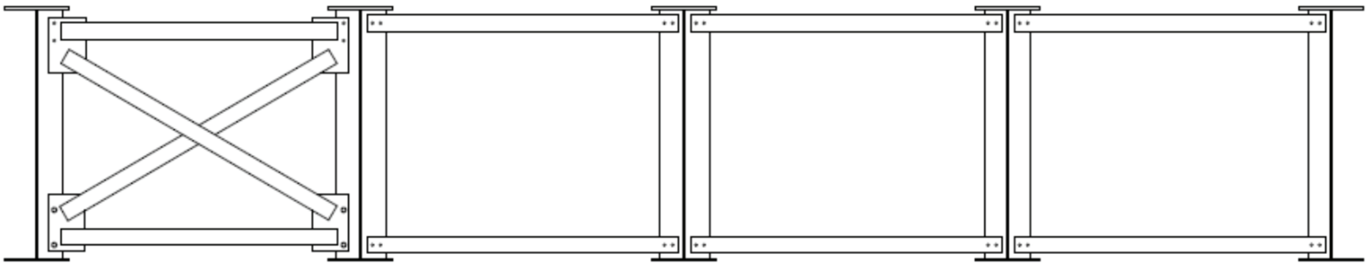
Strategy 4. When designers use a 2D grid, plate and eccentric beam (PEB), or 3D models using one member to represent the truss-type cross-frame, they should follow the *NCHRP Report 725 Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges* recommendations for shear-deformable (Timoshenko) beam element representation of cross-frames and for developing their stiffness and member area. *Bridge Design Specifications* article C4.6.3.3.4 and the AASHTO/NSBA *Steel Bridge Collaboration Guidelines for Steel Girder Bridge Analysis* G13.1 article 3.11.3 discuss this in greater detail. In general, the shear-deformable (Timoshenko) beam approach is considered to be a closer approximation for cross-frame modeling than the classical (Euler-Bernoulli) beam elements due to its more accurate prediction of the physical cross-frame behavior.

Designing Downsizing

Here are some design tips that can be used to help reduce the size of cross-frames.

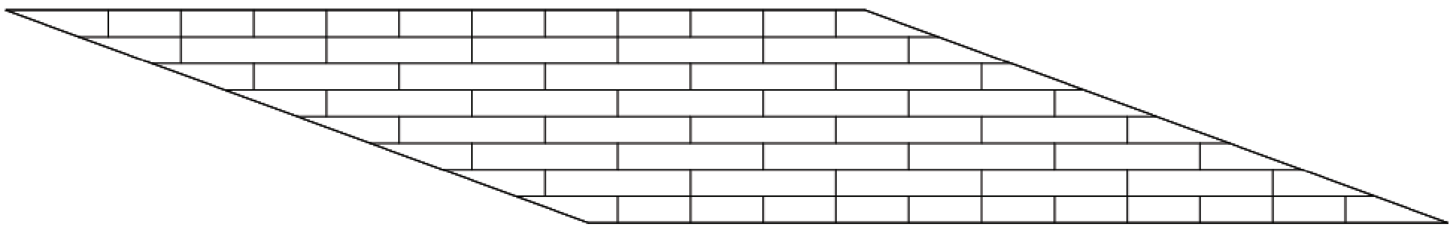
Design tip 1. A simple tip that can be used for reducing the sizes for the cross-frames is to group them and have multiple designs throughout the bridge. In some cases, bridge designers take the worst-case loading results from dead load, wind load, live load, fatigue, etc., combine these load effects, and design one cross-frame for the entire bridge. Most of the cross-frames do not experience the severity of this loading scenario, and having multiple cross-frame designs can result in a more efficient design throughout the bridge. For example, if the designer groups cross-frames by addressing different levels of loads and fatigue stress ranges, they could have an "x" number of "heavy" cross-frames, "y" number of "medium" cross-frames, and "z" number of "light" cross-frames. Most likely, the majority of cross-frames would be in the "medium" and "light" category, with a few on the "heavy" end of the spectrum. The vast majority of the cross-frames on bridges should not be designed for a few areas of high load effects.

Design tip 2. A similar procedure to the above tip can be employed for bolting the end connections of cross-frame members to gusset plates. As mentioned previously, member end connections that were



above: Lean-on cross-frame bracing shown in AASHTO/NSBA G12.1-2016: *Guidelines to Design for Constructability.*

below: An example framing plan of a staggered cross-frame layout (adapted from NSBA's *Skewed and Curved Steel I-Girder Bridge Fit*).



historically welded are now being bolted in-place because of computed fatigue stress ranges. However, the fabrication of welded end connections is often more cost-effective as compared to bolted end connections. Therefore, end bolted-connections should only be specified where needed, as all cross-frame members are probably not subjected to the maximum fatigue stress range. As with the first tip, cross-frame end connections can be grouped into those that need bolted end connections due to computed fatigue stress ranges and those that can be welded. If this procedure is adopted, the majority of cross-frames will most likely have welded end-connections. Note that welded and bolted member end connections should not be mixed in a single cross-frame. The welding, hole drilling, and bolting are typically done at different times and in locations within a fabrication facility, resulting in extra handling time and costs.

Design tip 3. Intelligent detailing practices and application of lean-on bracing or staggered cross-frame layout methods can be used appropriately to reduce stiffness of transverse load paths, especially in heavily skewed bridges. Applying lean-on bracing allows several girders to be braced across the width of the bridge by a single cross-frame, the adjacent girder bays "lean on" the cross-frame brace with top and bottom struts controlling the twisting action of girders (Helwig, et. al, 2012). Lean-on bracing will generally result in reduced cross-frame member strength forces and fatigue stress ranges. Lean-on bracing was the topic of a 2018 NSBA webinar, which you can view at **aisc.org/bridgebracing**.

Skewed bridges with considerable transverse stiffness can often result in large cross-frame forces, including increased live load and fatigue stress ranges. When bridge supports are skewed, designers should consider the recommendations of AASHTO *LRFD Specifications* Article 6.7.4.2 by placing cross-frames or diaphragms at supports along the skew and spacing them away from the supports.

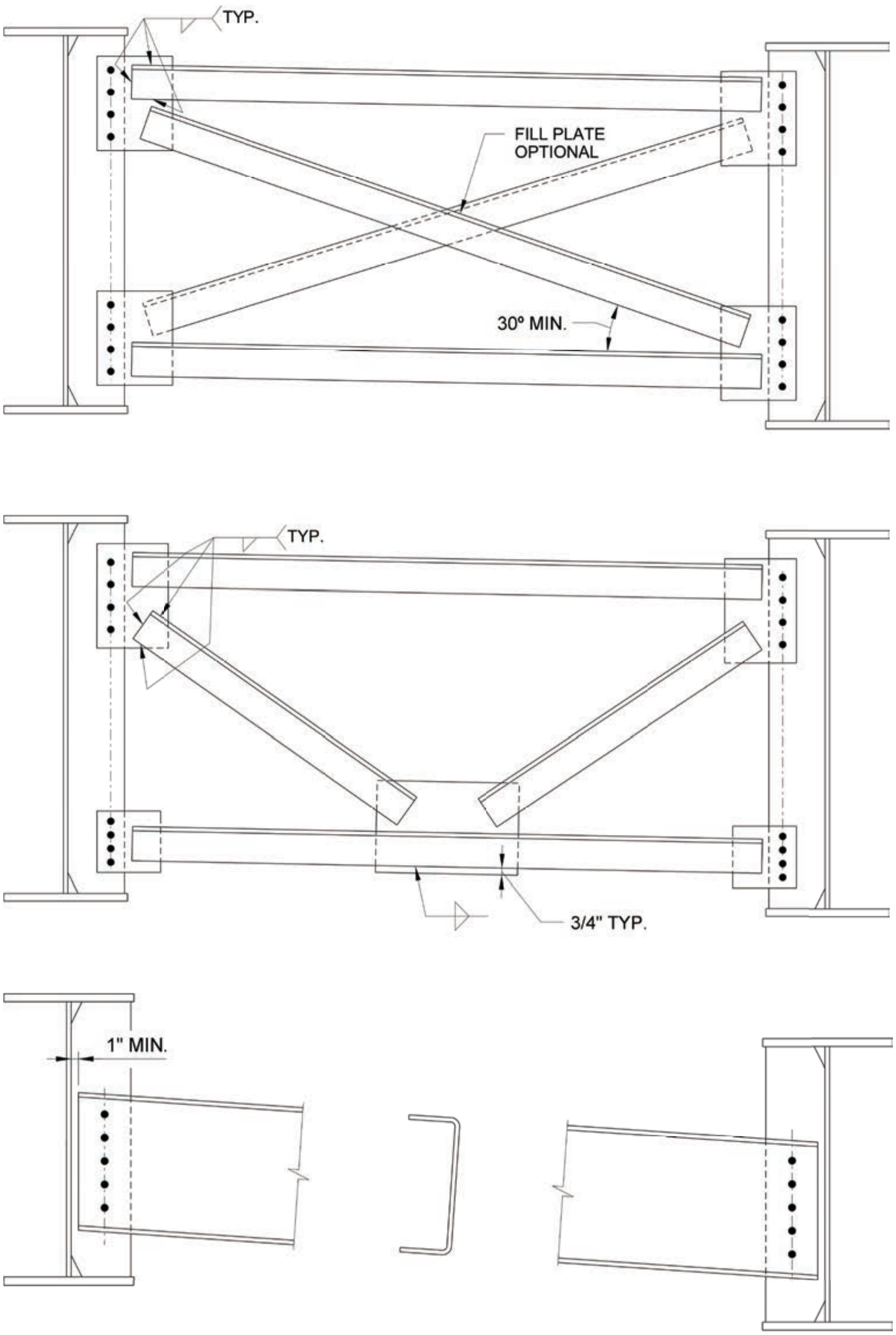
Design tip 4. In a refined analysis, boundary conditions can have a significant impact on the cross-frame forces, especially at locations near and at the bridge supporting substructure elements. Models can incorporate transverse and longitudinal stiffness associated with the pier and/or bearing instead of a hard point that is infinitely stiff and fixed. Allowing for appropriate levels of movement associated with a bridge's expected behavior will alleviate high force concentrations and provide a more realistic representation of the structure's response to force effects.

Design tip 5. When designers use 3D finite element models, it might be advantageous to use nodal or member end offsets to where the cross-frame work points are located. As compared to locating the end connection directly to the web/flange junction, offsetting the cross-frame ends will often result in reduced design forces in the cross-frame members. This offset will reduce the in-plane bending stiffness of the cross-frame, reducing its contribution to the transverse stiffness of the system.

Design tip 6. If the bridge is straight with no skewed supports, or has a skew index (see Appendix B of AASHTO/NSBA G13.1: *Guidelines for Steel Girder Bridge Analysis*) that permits a less rigorous analysis, a line girder analysis program such as LRFD Simon (available for free at **aisc.org/nsba/design-resources**) can be used to analyze and design the girders. Cross-frame members can typically be designed for wind load per *Bridge Design Specifications* and stability forces/stiffness requirements per the FHWA *Steel Bridge Design Handbook*, *Volume 13: Bracing System Design* (also available for free at **aisc.org/nsba/handbook**).

Shaping Up the Diaphragm

Now that we've provided some advice on reducing cross-frame sizes, let's discuss how to determine the best cross-frame type or diaphragm shape to use.



X-type cross-frame and K-type cross-frame bent-plate diaphragms (figures are from AASHTO/NSBA G12.1-2016).

An X-type cross-frame consists of top and bottom struts, and diagonals that intersect themselves near the center of the cross-frame bay. A K-type cross-frame consists of top and bottom struts, and diagonals that intersect the bottom strut. Generally, for intermediate cross-frames locations, the following guidelines are often employed by designers:

- X-type cross-frames are typically used in cases where the ratio of girder spacing (*S*) to girder depth (*D*) is 1.0 or less (i.e., $S/D \leq 1.0$)
- K-type cross-frames are typically used in cases where the ratio of girder spacing (*S*) to girder depth (*D*) is 1.5 or greater (i.e., $S/D \geq 1.5$)

In cases where the ratio of girder spacing (*S*) to girder depth (*D*) is between 1.0 and 1.5, either an X-type or K-type cross-frames may be used. However, the designer should consider the following two items:

- Achieve a generally efficient angle between the cross-frame diagonal and the horizontal (chord/strut) members as close to 45° as possible. Keeping this angle close to 45° degrees helps to limit either the depth or length of the gusset plate used to attach the cross-frame member to the girder connection plate.
- Minimize the shop handling of cross-frames by using K- cross-frames which do not need to be removed from their fabrication jig and inverted to weld the second diagonal. K-type cross-frames will have all welds on the same side of the cross-frame.

A solid plate diaphragm may consist of a channel, a bent plate, or a welded I-girder. These are generally used when it is necessary to address high diaphragm force effects and large diagonal and horizontal (chord/strut) members would otherwise be required for an X- or K-type diaphragm. Solid plate diaphragms are also typically used where the girders are tightly spaced or have a small depth, and the angles of the diagonals of an X- or K-type diaphragm are not efficient, thus making large gusset plate connections necessary.

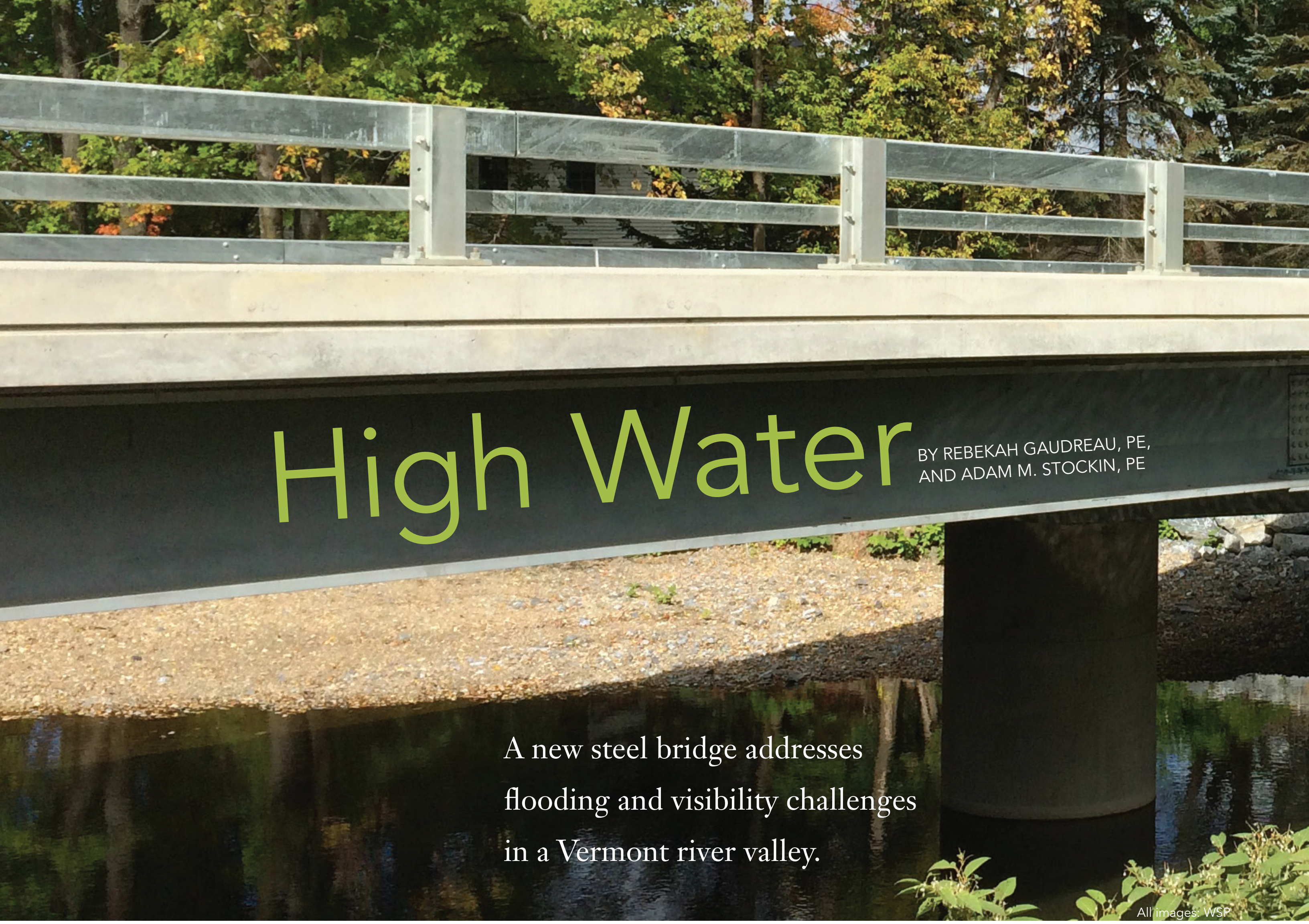
Additional design advice is forthcoming. The current study "National Cooperative Highway Research Program (NCHRP) 12-113" may offer some improvements in the design of cross-frames members and their end connections for fatigue. The NCHRP 12-113 researchers, led by Todd Helwig and Michael Engelhardt at the University of Texas at Austin, are investigating possible modifications to the AASHTO *LRFD Specifications* for the design and analysis of cross-frames as related to the proper loading conditions to establish fatigue design stress ranges, strength and stiffness requirements for stability bracing, and the influence of cross-frame member end connections on cross-frame stiffness in refined analyses. The research is expected to conclude by the end of 2020.

Common Sense Design

When detailing cross-frames, bridge designers can employ permitted AASHTO strategies and general guidance to produce solutions that will be efficient and proportioned in an intelligent way to preserve material, fabrication, erection, and the maintenance costs for the integrity and life of the structure. Stiffness attracts load, and increasing the sizes of our bridge members has an associated cost with making these cross-frame members unnecessarily bulky. The fabrication costs of cross-frames can be as much as five times more than the fabrication costs of the steel I-girders they frame into.

If you get significantly large cross-frame forces in bridge models or analyses, double-check your results and verify they make sense. Consider your framing plan and possibly reconfigure the cross-frame arrangement, as the layout for steel I-girder bridges can have the greatest influence on the loads in your bracing elements. Furthermore, when using a refined analysis, consider the methods allowed by the AASHTO *LRFD Specifications* and detailed in this article to help reduce the size of the cross-frame members and connections to improve the overall efficiency of your structure.

Please reach out to your local NSBA Bridge Steel Specialist (**aisc.org/nsba**) or an AISC member/certified fabricator or erector. All are here to help you with your designs and to provide beneficial feedback that improves the design and constructability of our steel bridges. And remember: When in doubt, don't just make your cross-frames stout! ■



High Water

BY REBEKAH GAUDREAU, PE, AND ADAM M. STOCKIN, PE

A new steel bridge addresses flooding and visibility challenges in a Vermont river valley.

All images: WSP

Rebekah Gaudreau (**rebekah.gaudreau@wsp.com**) is lead structural engineer and technical principal, and **Adam M. Stockin** (**adam.stockin@wsp.com**) is assistant vice president and supervising structural engineer, both with WSP.

THE RIVER ROAD BRIDGE over the New Haven River in New Haven, Vt., had a good run.

Built in 1935, the 170-ft-long three-span crossing was designed with straight steel girders on a curved alignment. Its design service life was eventually surpassed, and the deck and substructure components required replacement.

In 2017, its 164-ft-long two-span curved plate girder replacement structure took over the duty of carrying River Road over the waterway. The new structure has no joints or bearings, helping to decrease maintenance costs while increasing service life, with the hopes that it will outlast its predecessor's impressive life.

This accelerated bridge construction (ABC) project was completed within a 72-day closure window at a total construction cost of $3.5 million. And though the finished structure appears simple, the design and construction had its share of complexities.

The crossing is in a valley prone to flooding, and debris frequently became trapped under the former bridge. It was critical that the final design solution limited obstructions and maximized the vertical clearance over the river. However, raising the bridge's profile was not a practical solution, as it is located on a high point within the flood plain, and any increase would require earth and roadway work for a significant distance along the approaches. An intersection approximately 50 ft from the bridge added to the sight distance challenges of the curved alignment and further complicated a designed increase in the vertical profile. The site also has very poor subsurface conditions, with a 40-ft-thick layer of very soft clay with blow counts at weight of rod. This clay layer was located over additional stiffer clay layers and glacial till, and refusal was not achieved within the 120-ft depth of the subsurface investigation.

Bridge Design

The chosen bridge design consists of four curved steel plate girders, topped with an 8½-in. high-performance concrete composite deck, for a total width of 32 ft, 6 in. The

alignment was shifted to allow for a 6-ft, 6-in. shoulder on the inside of the curve to improve site distances and allow room for snowmobile passage across the bridge. The width of the pier cap matches that of the superstructure for clean lines and a pleasing aesthetic appearance.

A two-span configuration was chosen to eliminate a pier location, and the integral abutments were placed radially and supported on HP14×102 ASTM A572-50 piles orientated on the weak axis. The central pier consists of a single 6-ft-diameter circular column supported by an 8-ft-diameter, 115-ft-long mono-shaft, which significantly reduced hydraulic impacts when compared to a wall pier or multicolumn pier configuration. The substantial length of the mono-shaft was required due to the soft clay layer at the site. The traditional hammerhead pier cap was moved upward into the superstructure to further reduce hydraulic impacts and lower the potential for debris collection, an innovative technique that required the plate girders to be cast through the cap.

The girders were fabricated with 2-in-diameter holes in the webs located 3 in. below the top flange. This design decision accommodated the introduction of ten #11 bars for reinforcement continuity to meet the large moment demands at the top of the cap. A second row of ten #11 bars was cast into the deck, and additional holes for #6 bars were included vertically for the side reinforcement in the cap. A matrix of 56 ⅞-in.-diameter by 6-in.-long studs

above: The new 64-ft-long two-span curved plate girder River Road Bridge in New Haven, Vt., replaced a Great Depression-era crossing.

below: The existing bridge frequently experienced debris jams from high flows.

above: The full 103-ton steel superstructure.

left: The precast cap with girder stubs being prepared for transport.

was affixed to each side of the webs to ensure the system acted integrally in this high negative moment region of the plate girders. The shear studs were encircled by two #6 hoops on each side, 3 in. from the web face.

The steel plate girders—33¼ in. deep with a 16-in. by 2-in. bottom flange and a 16-in. by 1¾-in. top flange—were horizontally curved with radii ranging from 714 ft to 742 ft and cambered for dead load deflections and vertical profile. Horizontal curvature was achieved by flame-cutting the flanges simultaneously on both edges in accordance with Vermont's *Standard Specifications for Construction*. Diaphragms consisted of W24×84 rolled sections, and the steel framing was vertically offset to allow for a 4% super-elevation. A total of 103 tons of structural steel was required to complete the superstructure framing.

Given the frequent high water levels at this crossing, weathering steel was not an option, and traditional paint systems were also ruled out due to continual maintenance needs. The design team chose to metalize the AASHTO M270 Grade 50 structural steel members with an 85%–15% zinc-aluminum blend, which

Placing the precast cap unit on the mono-shaft.

was shop applied using electric arc thermal spray equipment to a thickness of 8-12 mils, followed by a sealer coat. All surfaces in contact with concrete were treated with a zinc primer, as aluminum reacts negatively with concrete. Faying surfaces were metalized but not sealed, and steel fabricator Casco Bay Steel provided documentation that a slip-critical coefficient required for a category B connection would be achieved.

ABC Challenges

Due to the tight closure window for bridge removal and construction, there was not adequate time to cast the cap around the structural steel in the field, thus requiring the cap to be precast together with the steel girder segments off-site. In order to meet shipping requirements, the girder segment length was set to 12 ft, which resulted in the need for bolted field splices to be designed at 6 ft from both sides of the centerline of the pier.

Because of the significant geometry control required for placing the superstructure on a central pier, plus the need to land the superstructure on precast abutment seat locations, prescriptive contract requirements were included to ensure that there would be no alignment challenges in the field. A special provision was developed to introduce a third-party engineer, hired by the contractor, to perform quality control and coordination between the general contractor, precast contractor, and Casco Bay. The structural steel framing was fabricated at Casco Bay's Portland, Maine, shop and shipped to the concrete contractor in Clarendon, Vt., where the structural steel

above: Erecting the steel superstructure.

below: In order to meet shipping requirements, the girder segment length was set to 12 ft, which resulted in the need for bolted field splices to be designed at 6 ft from both sides of the centerline of the pier.

system was erected with full geometry control and all field splices and diaphragms were fully connected. Reinforcement was then tied, and the cap was cast around the girders. Once the cap had cured, the structural framing was disassembled and shipped to the project site.

Simple yet Elegant

The construction team completed the bridge in the 72-day closure window, meeting the Town of New Haven's desire for the local school bus schedule to remain unhindered by the project. At final inspection, it was clear that the complex design of this bridge yielded a simple and elegant structure, with the only visible joint being the ½-in. grout pad between the pier cap and the column. In addition, the single circular column is the only obstruction in the river, significantly reducing the likelihood of debris being trapped.

When it made the decision to replace the original River Road Bridge, the Vermont Agency of Transportation (VTrans) sought a successor that would increase hydraulic capacity and safety at the nearby intersection, eliminate maintenance issues associated with joints and bearings, provide extended service life, and inflict minimal delay to residents. The new bridge accomplished all of these goals, serving as an excellent example of how engineering, innovation, and prescriptive contract requirements can be blended to meet the client's and the traveling public's needs under very challenging site conditions. ■

above and below: The completed integral structure, with an open stream crossing and metalized continuous plate girders, from below and above.

Owner
Vermont Agency of Transportation (VTrans)

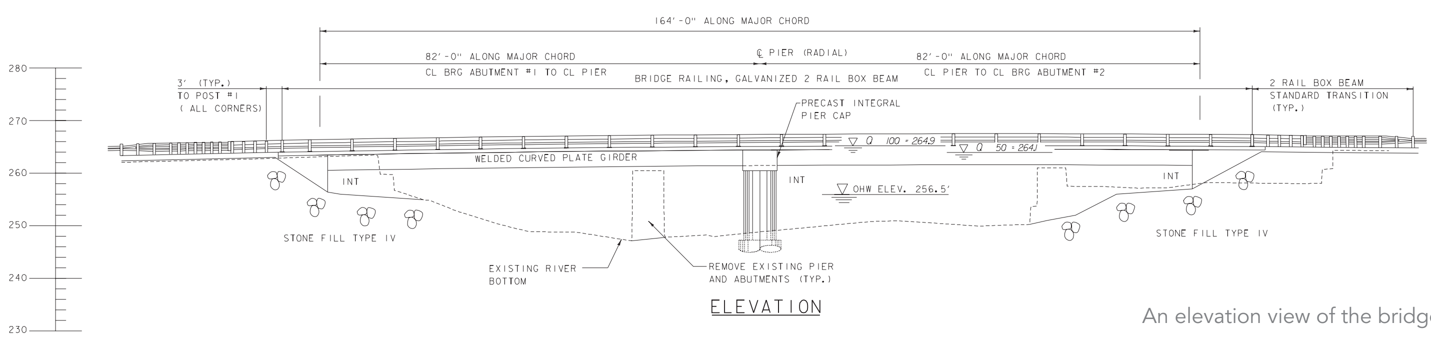
General Contractor
CCS Constructors, Inc.

Structural Engineer
WSP USA, Inc.

Steel Team

Fabricator
Casco Bay Steel AISC CERTIFIED FABRICATOR, Portland, Maine

Detailer
Tensor Engineering AISC DETAILER, Indian Harbour Beach, Fla.



An elevation view of the bridge.

New resources from the American Galvanizers Association can help you determine the maximum steel element size for maximum efficiency in your next progressive-dip galvanized job.

Maxing Out

BY ALANA FOSSA

Steel elements that are too large to be galvanized in a single dip can be progressive-dipped.

ONE OF THE MOST COMMON QUESTIONS about after-fabrication batch hot-dip galvanizing is, "What is the largest piece that can be hot-dip galvanized?"

The quick answer is, "There's not a one-size-fits-all answer." Hot-dip galvanizing is an immersion process, so size limitations are often governed by the galvanizing kettle dimensions.

Although the average kettle length in North America is around 40 ft, there are many kettles between 50 ft to 60 ft long. Whatever the kettle size, the practice of progressive-dip galvanizing—colloquially though erroneously referred to as "double-dipping"—allows large steel elements to be dipped, even when they exceed the dimensions of the chosen kettle. And new free tools from the American Galvanizers Association (AGA) can help specifiers determine the maximum article size appropriate for successful progressive dip galvanizing depending on kettle and facility constraints.

So why is determining maximum article size even important? Isn't it possible to simply break down assemblies to their smallest elements and dip them in several batches? Absolutely. But keep in mind that galvanizing is a very involved process, and galvanizers don't charge by the assembly but rather by the number of dips. So from a time and financial standpoint, maximizing sizes and minimizing the number of dips is your best option.

Maximum Article Size

Steel assemblies intended to be galvanized are typically designed in sections or individual members to fit within the kettle and are then bolted or welded together following the dipping process. If an item or is too long or deep for a single immersion in a kettle, this is where progressive dipping can come into play. In this process, articles are partially galvanized at an angle, flipped, rehung, and then galvanized on the remaining surface to fully coat the assembly with a small overlap area.

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The maximum progressive dip length is most easily approximated by modeling the steel article as a solid, 3D box partially immersed in the galvanizing bath, where the width of the article is less than the kettle width. (You can find kettle dimensions for all of AGA's member hot-dip galvanizers at **galvanizeit.org/galvanizers**.) In addition to the kettle and article dimensions, the dross line height and freeboard height are important variables in the calculation (dross forms by reactions between molten zinc and loose particles of iron in the galvanizing kettle, and typically drops to the bottom of the kettle because it is heavier than the molten zinc). The kettle is not filled to the top with molten zinc, but rather to a few inches below the top. If this distance from the top of the zinc to the top of the kettle, called the freeboard height, is unknown, it can generally be estimated at 4 in. The galvanizing dross line height varies over time depending on the plant's kettle maintenance schedule, but steel assemblies are often lowered to a point just above this line to minimize the potential for dross inclusions (aka "dross pimples") on the surface of the article. For steel elements where inclusions are not acceptable—such as handrails, architecturally exposed structural steel (AESS), and steel to be painted or powder coated after galvanizing—a dross height of 8 in. can be estimated, a measurement that errs on the side of caution. To maximize the size of an article to be galvanized, for which dross inclusions are acceptable, you can drop the dross line value to zero.

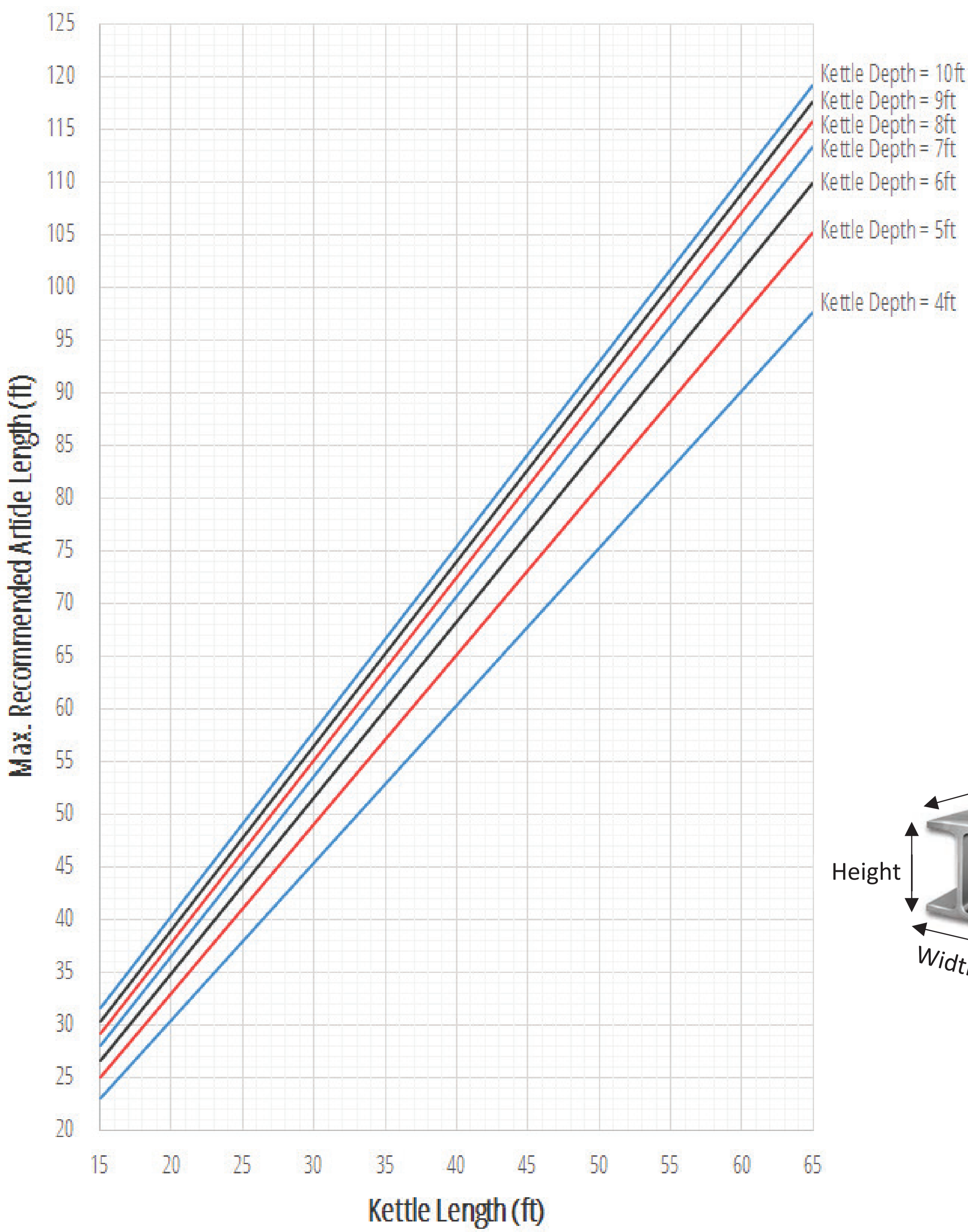
A lengthy steel member taking its first dip in the zinc bath.

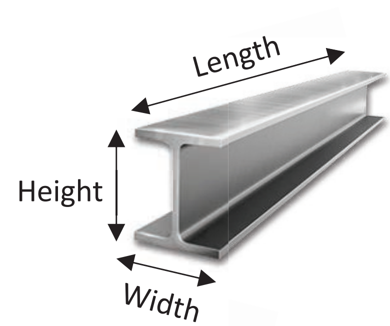
A large steel element going in for its second zinc dip.

Progressive Dip Charts and Calculator

Once you determine the necessary tank measurements, there are two new resources to assist you with calculating the maximum article size: the AGA Progressive Dip Charts and AGA Progressive Dip Calculator.

The AGA Progressive Dip Charts (**galvanizeit.org/pdcharts**) estimate the maximum article length if the article height and galvanizing kettle dimensions are known to the nearest foot. The four charts provide the maximum length for articles of 1 ft, 2 ft, 3 ft, or 4 ft in height when considering a range of typical galvanizing kettle lengths and depths.





A sample AGA Progressive Dip Chart. All of the charts are available at **galvanizeit.org/pdcharts**.

For a step-by-step look at the hot-dip galvanizing process, see "Galvanizing Illustrated" in the August 2014 issue, available at **www.modernsteel.com**.

The AGA Progressive Dip Calculator (**galvanizeit.org/pdcalculator**) offers a customizable format to determine whether an article of specific dimensions can be hot-dip galvanized within a known galvanizing kettle size. This calculator informs you whether the steel item can be progressive-dipped successfully in one particular galvanizing kettle. It also provides vital information that can be discussed between the designer and galvanizer to make educated decisions about lifting arrangements that may influence the final decision to galvanize. The outputs from the calculator include a visual 2D model, allowable angles for successful galvanizing, available options for article orientation during galvanizing, and guidance on whether rotating the article width and height affects galvanizing success.

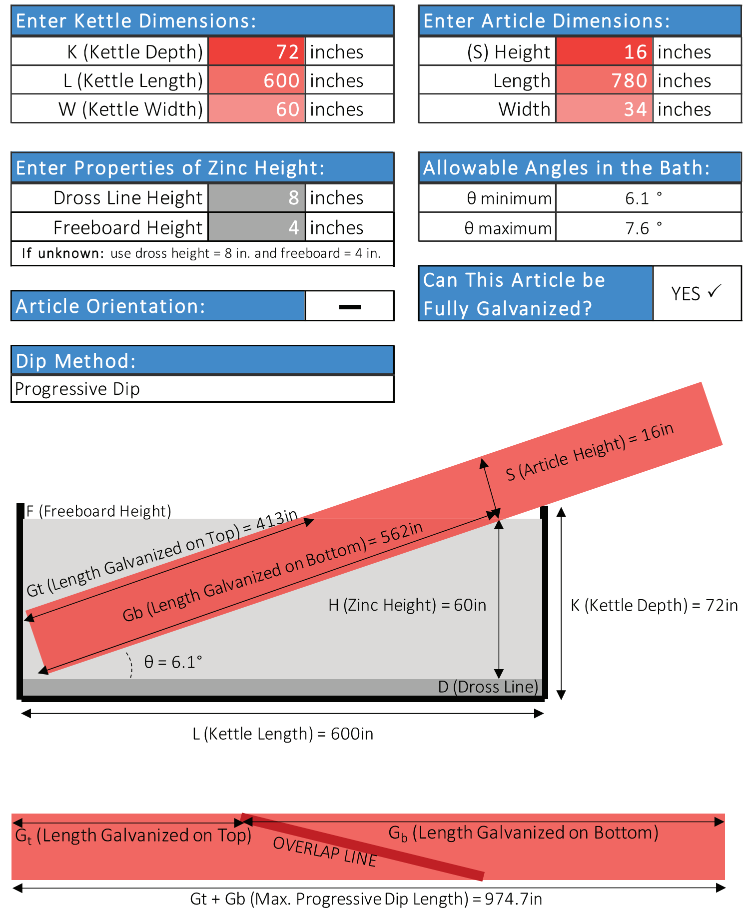
Other Considerations

While maximum article size is indeed a crucial consideration for progressive dipping, it's not the only one. Progressive dipping jobs are also influenced by dimensional constraints, aesthetic requirements, and zinc temperature.

Lifting orientation and handling. The galvanizing plant must be capable of handling the articles to be dipped, which depends on the facility's crane load capacity, crane height, and overall plant layout. Discuss lifting orientation and lift points directly with the galvanizer to avoid the article clashing with nearby walls or equipment at the plant for both passes, and confirm the article weight is within the safe working load limit of the available lifting equipment.

Aesthetics. Progressively dipped pieces have an overlap area that often appears darker, develops a thicker coating, and may not weather consistently with the rest of the product. Although the overlap line does not affect the overall corrosion protection, it can be buffed or ground down even with the surrounding coating to improve the look. Grinding is also beneficial and necessary in situations where the increased coating thickness of the overlap area impacts a connection point with other pieces.

Process temperature concerns. Uneven heating and cooling are inevitable during progressive dipping, as one end of the article will be in the molten zinc bath (~850 °F) while the other end is exposed to cooler air—and exposure to such wide temperature variations may lead to distortion of fabricated assemblies. The risk of distortion can be reduced by designing for the increased stresses from thermal expansion at the zinc bath temperature. Analyzing the conditions of the first dip is critical, as the temperature gradient will be greatest for this step and less severe for the second dip due to heat retained in the steel from the first immersion. Additionally, ensuring that vent and drain holes are adequately sized and placed will facilitate rapid immersion and withdrawal of the object from the bath. Bracing (permanent or temporary) may also be incorporated to provide stability during the thermal expansion and contraction. Additional details for minimizing the potential for distortion are available in ASTM A384: *Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing of Steel Assemblies* (available at **astm.org/atandards/a384.htm**).



above: A sample steel item and kettle scenario as run through the AGA Progressive Dip Calculator. Visit **galvanizeit.org/pdcalculator** to give the calculator a try.

below: Progressive dipping typically produces an overlap surface condition that is thicker and darker in appearance than the surrounding area but can be smoothed out via grinding if required.

Minimize Dips, Maximize Efficiency

Steel elements and assemblies that are too long or deep for a single dip present some additional challenges in the galvanizing process. But with the right tools and communication with your galvanizer, you can ensure that the process will run smoothly and result in a quick turnaround and long-lasting corrosion protection for your hot-dip galvanized steel projects. ■

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