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ON THE COVER: A weathering steel bridge forges a new path over an old railyard in Minneapolis, page 26. (Photo: Mark Maves)

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Because You Start with a Model

editor's note

As I sit in my basement office, hopping from one virtual meeting to another, my mind constantly drifts to the future. I know that eventually we'll be able to resume our pre-COVID lives, but as with many people, I wonder what will be different.

At first, when I saw how well people could do their jobs remotely, I thought about all of the advantages of no longer having to commute, of being able to take my laptop out on the deck to enjoy the sunshine, and of saving money by visiting my kitchen instead of a nearby restaurant. But as the days turned into weeks, and the weeks turned into months, some of the charm wore off. I couldn't spontaneously meet with two or three colleagues to discuss something; we needed to schedule a meeting. Some of the casual ideas that occurred organically during wildly disparate discussions disappeared. And I imagined that on-boarding would become a nightmare and our corporate culture would slowly vanish.

Schools moved online, but it wasn't the same—and not just because many teachers were woefully unprepared and not trained or organized to handle the switch. The peer-to-peer interaction was lost and the interpersonal student-teacher relationship disappeared.

We successfully held online conferences, but we no longer met new people or took part in casual conversations that are almost always as valuable as (if not more so than) any technical presentations.

So in six months, when a vaccine is hopefully readily available, will the world be different than six months ago? Certainly.

We'll see fewer retail establishments, and the ones that prosper will be those who either offer immediacy (I have a headache and even if Amazon can deliver in two hours, I want my aspirin now!), a unique browsing experience (sure, Zappos is convenient and I can return shoes that don't fit, but it's still not the same as seeing my options laid out before me in the excitement of a retail environment), or those who offer superior service (yes, Home Depot is almost always less expensive than Crafty Beaver, but I often want a salesperson who can give me advice and not just show me my options).

Will cities change? Yes, but there's no substitution for the excitement of a crowded downtown and the population density that allows for fabulous dining options, notable museums, and live theater.

I know we'll take away from this time valuable lessons that teach us different ways of doing things. I'm certain that many people will never go back to five days a week in an office. I'm certain that we'll permanently see an increase in alfresco dining (even in places like Chicago, thanks to portable heaters). And the stigma of online degrees will dissipate. But to paraphrase the famous Barbra Streisand tune, people need people. And we should all feel lucky about that.

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If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

All mentioned AISC codes and standards, unless noted otherwise, refer to the current version and are available at ***aisc.org/specifications****. Design guides can be found at* ***aisc.org/dg****, and* Modern Steel Construction *articles can be found at* ***www.modernsteel.com****.*

Steel Dimensioning Tool

On the Structural Steel Dimensioning Tool page of the AISC website (aisc.org/dimensioningtool) there appear to be issues with the workable gage measurements not changing to reflect wider flange selections within a specific family.

For example, within the W10 family, the base workable gage dimension is 2.5 in. for a 4-in.-wide flange. It then increases to 2.75 in. when the flange width reaches 5.75 in. It then changes one last time within the W10 family, to 5.5 in., when the flange width reaches 8 in. From a flange width of 8 in. right to the maximum of 10.375 in., the workable gage remains unchanged at 5.5 in., which cannot be correct.

In another example, within the W12 family, the workable gage does not change for a flange width of 8 in. right to the maximum of 13.375 in., which again, cannot be correct.

Please investigate this issue, as there appears to be legitimacy with the functionality of the workable gage measurement.

The Structural Steel Dimensioning Tool lists the usual gage measurements used in steel fabrication. The typical double-angle shear connection for a beam to a column flange is made with either a L4×3½×5⁄16 or L4×3½×3⁄8 with the 4-in. leg being outstanding. To maintain edge distances on the outstanding legs of the angles, the gage in the column flange is typically limited to 5½ in. The last time that the wide flange usual gage was tabulated in the AISC *Steel Construction Manual* was 7th Edition. There is no requirement to use the usual gages as tabulated in the Steel Dimensioning Tool if the proper edge distances are maintained. For this reason, the usual gage is no longer listed in the *Manual*.

Jonathan Tavarez, PE

Fourth Edition *Seismic Design Manual*

Are there any plans to produce the new seismic design manual in hardcover like the AISC *Manual* or older AISC *Seismic Design Manuals*?

There is no plan to produce the 4th Edition *Seismic Design Manual* in a hardcover. In order to price the *Seismic Design Manual* for $100, which is one-half the price of the *Steel Construction Manual*, we decided to use a high-end softcover for the *Seismic Design Manual*. To use the *Seismic Design Manual* and the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) it is necessary to have the *Manual* and the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360). The softcover was used to make it more economical for purchase.

Jonathan Tavarez, PE

Conference Proceedings

Are the proceedings from the 8th International Conference on Composite Construction in Steel and Concrete available on your website? I could not locate them.

This proceeding was recently added to our website and is now available. You can find and download this proceeding for free at **aisc.org/publications/conference-proceedings**.

Jonathan Tavarez, PE

Helpful AISC Publications

I am working on several projects involving steel construction, and I do not know which AISC publication I should purchase to help me in my work. Would the 15th Edition *Manual* suit my needs, or is there another publication I should buy?

The 15th Edition *Manual* would be an excellent choice to use for any steel construction work. The *Manual* contains the AISC *Specification*, which is referenced in the *International Building Code (IBC)*. In addition, the *Manual* includes the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303) and the *RCSC Specification for Structural Joints Using High-Strength Bolts*. The *Manual* also contains many design tables to aid in the selections of members, connections, bolts, welds, and many design solutions related to steel construction. You may want to look at AISC's list of Design Guides, which provide guidance on specialized steel-related topics and are authored and reviewed by recognized industry experts and AISC staff.

Jonathan Tavarez, PE

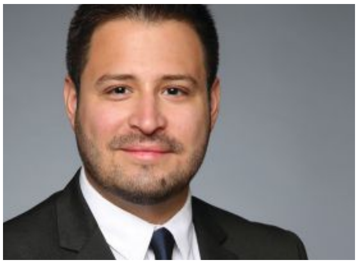
Eye Bolts

I need to provide an eye bolt style connection to a structural steel element to suspended items from a roof. The maximum vertical load that would be pulling on the eyebolt is 400 lb. But I don't see any AISC information on eye bolts—just clevises, turnbuckles, etc. Is there info on eye bolts?

AISC does not provide strength information on eye bolts. I recommend reaching out to manufacturers as they would be able to provide more insight. Check out the bolting section of *Modern Steel*'s online Product Directory under the Resources section at **www.modernsteel.com**.

You can also search for AISC member fabrication shops at **aisc.org/aisc-membership**.

Jonathan Tavarez, PE



Jonathan Tavarez (**tavarez@aisc.org**) is a staff engineer with AISC's Steel Solutions Center.

Ordinary Moment Frame Weld Requirements

I am having trouble interpreting the requirements in the AISC *Seismic Provisions* for an ordinary moment frame (OMF). I have a one-story frame in an SDC B. We would like to use R = 3.5 instead of R = 3. The plan for the connections is to weld flange plates to the columns with fillet welds and bolt to the beam flanges. It appears to me that an OMF requires full-penetration groove welds to the column and not fillet welds. Is this correct?

You will find the requirements for OMF systems in Section E1 of the AISC *Seismic Provisions* and requirements for the connections, specifically, in E1.6. I am not aware of a requirement for only complete-joint-penetration (CJP) welds to be used in OMFs, or a prohibition of the use of fillet welds.

I believe you might be confusing the requirement in Section E1.6b(c)(2), which requires that the beam flange to column flange be CJP welded. CJP welds are only required if this option is used, and it would require the beam flange to be directly welded to the column flange. A flange plate option is not addressed here, and this requirement for CJP in this option does not extend to flange plates designed using other options discussed in Section E1.6b. Another option would be to use a prequalified connection. The bolted flange plate prequalified connection would require that the flange plates be CJP welded to the column flange, as can be seen in Chapter 7 of *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358)—although again, there are likely more economical options than using one of the prequalified connections in an OMF system.

I will note that I believe it would be more economical to design the structure as R = 3 as opposed to R = 3.5 if this is permitted. You may want to discuss this with a fabricator likely to bid to project to get a better feel for what the difference in cost might be.

Jonathan Tavarez, PE

PDF Copy of *Seismic Design Manual*

Is there any way to purchase a pdf copy of the AISC *Seismic Design Manual*? With the majority of our office working remotely, we are struggling to share our resources among all our employees.

Unfortunately, we do not sell a PDF copy of the *Seismic Design Manual*. However, you can download a PDF of the *Seismic Provisions*. Also, AISC has provided electronic licensing to IHS Global, MADCAD, and TECHSTREET to publish PDF versions of our manuals (visit **aisc.org/customerservice** for more information).

Jonathan Tavarez, PE

STEEL SOLUTIONS CENTER

Steel Interchange is a forum to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Contact Steel Interchange with questions or responses via AISC's Steel Solutions Center: 866.ASK.AISC | **solutions@aisc.org**

The complete collection of Steel Interchange questions and answers is available online at **www.modernsteel.com**.

steel quiz

This month's Steel Quiz is all about bridges.

The answers can be found in the AASHTO *LRFD Bridge Design Specifications* (9th Edition), AASHTO/NSBA Collaboration G12.1: *Guidelines to Design for Constructability* (available at **aisc.org/gdocs**), NSBA's *Skewed and Curved Steel I-Girder Bridge Fit* document (available at **aisc.org/nsba/skewed-curved**), and even in this very issue of *Modern Steel*.

1 What is the minimum thickness requirement for structural steel according to the AASHTO *LRFD Specifications*?

a. ¼ in.
b. 5⁄16 in.
c. ⅜ in.
d. ½ in.

2 **True or False:** AASHTO *LRFD Specifications* Fatigue Category Details D, E, and E' for load-induced fatigue should always be avoided for new steel bridge designs.

3 **True or False:** According to Skewed and Curved Steel I-Girder Bridge Fit, total dead load fit (TDLF) should be specified for horizontally curved steel I-girder bridges when the maximum span length to radius ratio is 0.05.

4 When designing the bracing members for a steel I-girder bridge with a girder spacing of 12 ft and a girder depth of 4 ft, what type of cross-frame or diaphragm should be selected if there are large forces present in the members? (Hint: The answer can be found in this issue of *Modern Steel*.)

a. X-type cross-frames with top and bottom struts
b. K-type cross-frames with top and bottom struts
c. Solid diaphragm
d. Any of the above

5 **True or False:** According to AASHTO *LRFD Specifications*, bolted connections subjected to stress reversal, heavy impact loads, severe vibration, or located where stress and strain due to joint slippage would be detrimental to the serviceability of the bridge require slip-critical designed and designated connections.

6 **True or False:** Shear stud connectors on continuous composite steel I-girder bridges need only be designed for AASHTO *LRFD Specifications* Fatigue and Strength Limit State requirements.

TURN TO PAGE 14 FOR THE ANSWERS

steel quiz ANSWERS

Everyone is welcome to submit questions and answers for the Steel Quiz. If you are interested in submitting one question or an entire quiz, contact AISC's Steel Solutions Center at 866.ASK.AISC or **solutions@aisc.org**.

1 **b.** AASHTO *LRFD Specifications Section* 6.7.3 states that 0.3125 in. ($\frac{5}{16}$ in.) is the minimum thickness required for structural steel. This requirement generally governs for cross-frame and diaphragm members. AASHTO/NSBA Collaboration G12.1 recommends a minimum steel girder web thickness of ½ in. and a minimum steel girder flange thickness of ¾ in. Note that gusset plates used in trusses and orthotropic steel decks require other minimum thicknesses per Section 6.7.3.

2 **False.** While Fatigue Category Details D, E, and E' should be avoided when possible, they can be included in new steel bridge designs when necessary. However, when they are used, the designer must appropriately consider the factored fatigue stress range and the factored resistance for these details in accordance with AASHTO *LRFD Specifications*, Article 6.6. As long as the appropriate stress range and resistance are considered, designing a new bridge for Category Detail B is no different than designing for Category Detail E. In some cases, a Category Detail E' is unavoidable for cross-frame member welded end connections.

3 **False.** According to Table 3 of the *Skewed and Curved* document, steel dead load fit (SDLF) is recommended for this scenario. TDLF is not recommended for curved I-girder bridges with or without skew and a maximum *L/R* greater than 0.03.

4 **c.** See this month's SteelWise article "Keeping Cross-frames in Check." K-Type cross-frames should be selected in general when the girder spacing (*S*) to girder depth (*D*) is 1.5 or greater. However, when lateral member forces are large, a solid diaphragm composed of a channel, a bent plate, or a welded I-girder is recommended.

5 **True.** Per AASHTO *LRFD Specifications*, Section, 6.13.2.1.1, slip-critical connections shall be proportioned to prevent slip under Service II load combination and also provide bearing, shear, and tensile resistance at the applicable strength limit state combinations per Table 3.4.1-1.

6 **True.** Shear Connectors on continuous steel I-girder bridges are designed according to AASHTO *LRFD Specifications*, Section 6.10.10, including the fatigue and strength limit state requirements. If steel I-girders are designed such that they are noncomposite for negative flexure in the final condition, 6.10.10.3 (special requirements for points of permanent load contra flexure) also needs to be checked.

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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.

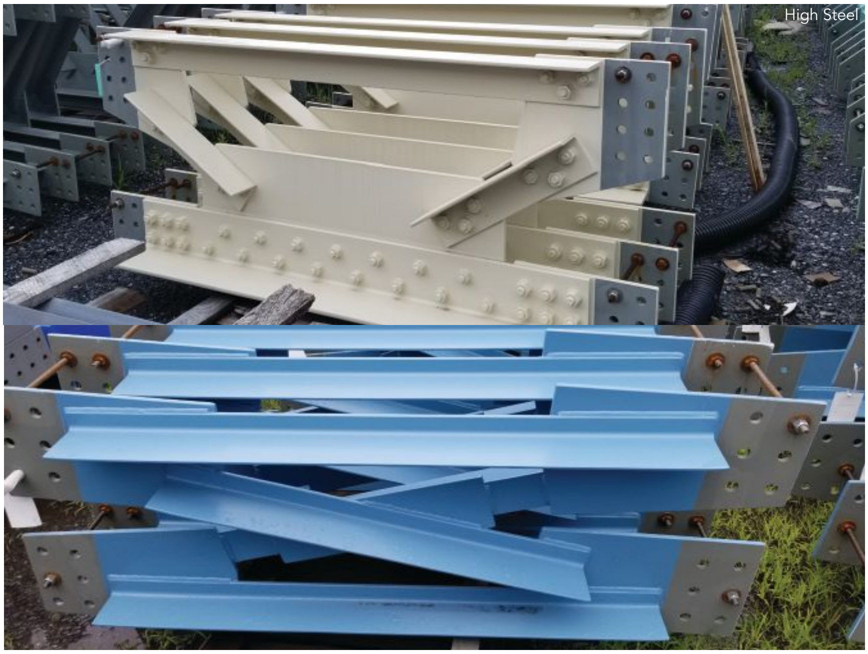
CROSS-FRAMES ARE A BIG DEAL—and they're getting bigger.

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.

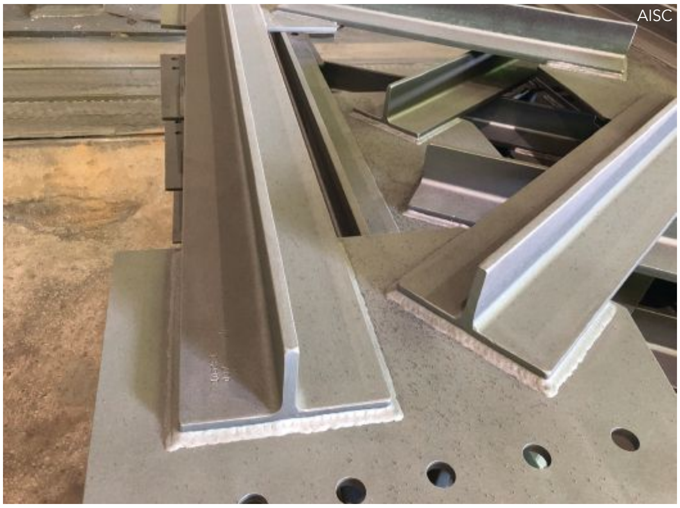
Along with the sizes of the members and connections getting larger, connections that were historically welded are now being bolted in place instead. Furthermore, X-type and K-type cross-frames are being used in situations where a solid bent plate or built-up diaphragm would make more sense from a geometry, 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 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 0.65*AE* 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 0.65*AE*."

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.

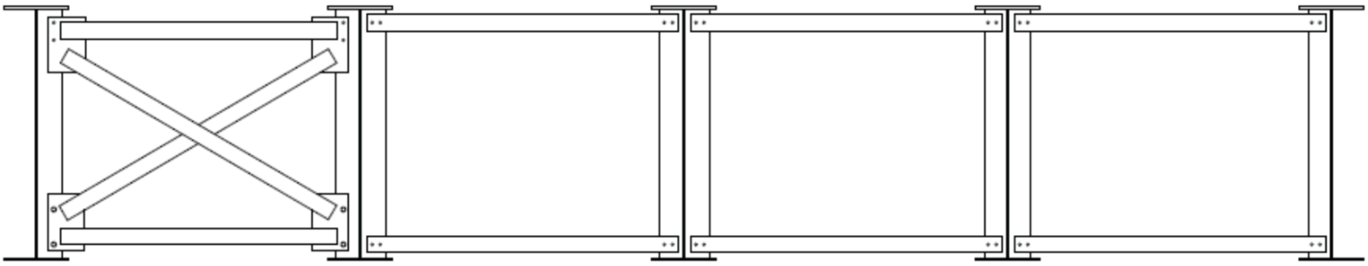
Designing Downsizing

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.

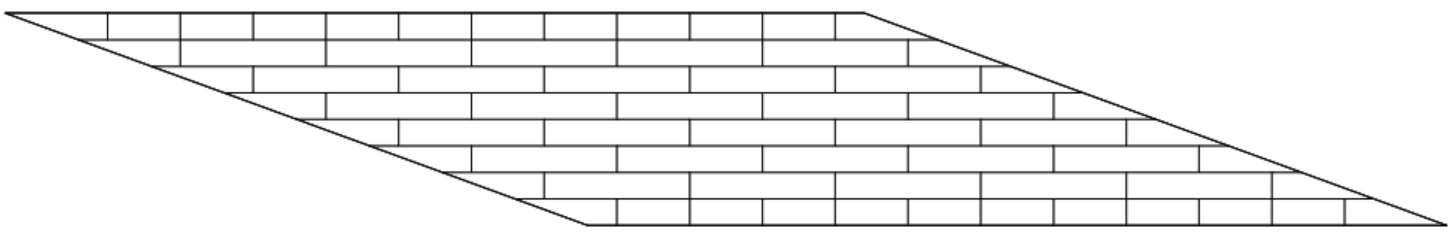
Design tip 2. A similar procedure to the above tip can be employed for bolting the end connections of cross-frame members to gusset plates. As mentioned previously, member end connections that were historically welded are now being bolted in-place because of computed fatigue stress ranges. However, the 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.

A K-type cross-frame with bolted end connections—not recommended unless absolutely necessary.



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*).



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

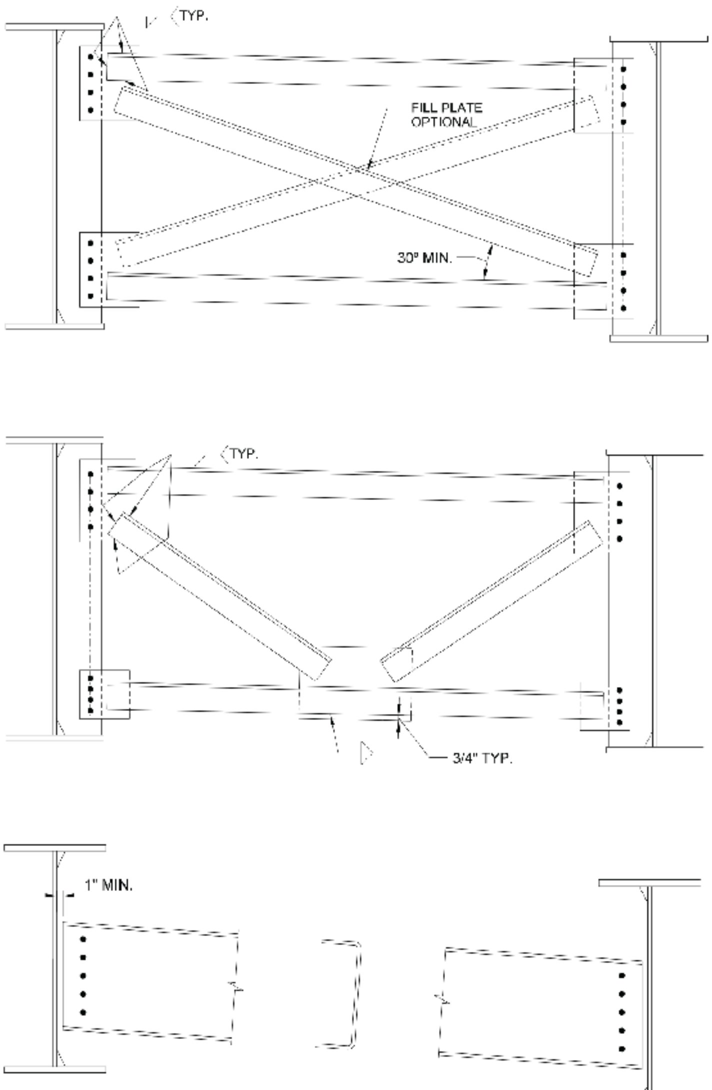
Now that we've provided some advice on reducing cross-frame sizes, let's discuss how to determine the best cross-frame type or diaphragm shape to use.

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.



X-type cross-frame and K-type cross-frame bent-plate diaphragms (figures are from AASHTO/NSBA G12.1-2016).

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! ■

field notes

INDUSTRY ACE

INTERVIEW BY
GEOFF WEISENBERGER

Project executive Angela Cotie is strengthening Houston's construction industry one student at a time.

Field Notes is *Modern Steel Construction*'s **podcast series**, where we interview people from all corners of the structural steel industry with interesting stories to tell. Listen in at **modernsteel.com/podcasts**.

ANGELA COTIE is the chair of ACE Mentoring Houston (we'll get to what, exactly, that is in a bit) where she takes a leading role in introducing the next generation to the wide breadth of careers in the construction industry. When she first started in college, she had a pretty specific idea of what her role in the industry would be, but then she discovered that it could be even more than she expected—and when she graduated, her career started off with a grand slam. Want to find more? Read on!

Did you always know you wanted to be in the buildings world?

I always wanted to be an architect. When I was little, I would always look at things and want to change them around, and my family would laugh at me because I'd see an old building and I would turn it into something else. Then I discovered Penn State architectural engineering and thought it sounded interesting and then I got to school and thought, "Wait a second, this is math and science." So I applied to the architecture school. As part of the AE program, I was taking a construction class while I was waiting to hear back from the architecture school, and it wasn't what I initially thought of when I thought of construction. It was math and science—but also money, logistics, puzzles, and all of what goes into actually getting the job done. I actually got accepted into the architecture school, I passed it up and stayed in architectural engineering, focusing on construction. And I'm glad I did. It's been a great challenge for me over the years.

On that note, did you have a favorite building when you were younger that inspired you and made you want to get into architecture in the first place?

I grew up outside of Washington D.C., and I liked the grandeur of the buildings there, the fact that the architecture felt so permanent. As an architect, you may only work on one big impressionable building that you'll be remembered for, but as a contractor I can influence multiple buildings. I can get involved early in a project, and I can actually assist and help the architect. If you also have a sensitivity towards architecture, you can help with executing the building even more successfully.

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

Tell me a bit about one of your first big projects.

Minute Maid Park, where the Houston Astros play, was my first job (I work for Gilbane Building Company, but this project was with a previous employer). I got spoiled because I got to see how a building could change a community. I never realized the power of a building until I saw what that ballpark did for Houston. Now, 20 or so years later, that entire area of Houston is completely different, and had that ballpark not been there, had they not chosen that spot to develop it, Houston wouldn't be what it is now. Since then, I've been able to work on a lot of projects that have been game-changers for their respective communities.

Speaking of communities, tell me about ACE Mentoring.

It's a mentoring program for high school students who are interested in architecture, construction, and engineering. It's free for the students and the mentors. There are affiliates across the country and each city has a different model, but in Houston we have eight locations where students can meet with mentors. They team up and work on a project, usually from November until April, and each week when they meet, for about two hours, they learn about architecture, the different types of engineering, and construction. We give them an RFP at the beginning of the school year, which they work on throughout the year, then they present it at the end of the year. Unfortunately, this year, thanks to COVID, we've had to go virtual. But despite the fact that we don't get to have face-to-face mentoring, I'm excited because virtually, we have the opportunity to reach more students. Last year, we had between 200 and 300 kids signed up and by the end, 225 kids completed the program and about 80% of those kids were coming from disadvantaged circumstances. We know Houston doesn't have a widespread public transportation system, so I'm excited that we've been able to bring more exposure to kids who are far from their mentors.

When it comes to engineering, a lot of kids interested in that career only think of petroleum engineers—because we're in Houston, right? But we're able to explain to them that "engineer" can mean so many different types of jobs: acoustical engineer, lighting engineer, structural engineer. They're able to see that not every architecture job is designing skyscrapers. They see that not every contractor is always on-site. Or they even see careers where they can use their hands. And that's one thing that we're really trying to introduce kids to, the idea that being part of a trade can be more lucrative than any of the other careers I just mentioned. Starting in a trade might be one of the best ways for them to reach one of those careers. The best electrical engineers I know were electricians first.

We also teach the students about confidence. At the end of the year, we have a final competition, and they have to stand up and present as a team. So not only do they have to learn about teamwork but they also each have to stand up and present their portion of the project. One of my favorite parts of the whole program is seeing them get dressed up for their "parts," and some are wearing business clothes and others are wearing hard hats, or they'll have made little business cards for their team, and they come in and introduce themselves as the project manager, and the architect, and so on. Usually, the kids on the team don't know each other when they first start because there's probably 60 different schools that participate in the Houston program, so the students are coming from everywhere. They'll get up there and present their part and talk about their design, and when we coach them, we emphasize that it is indeed their design. We tell them no one knows what it is; you have to pitch it. You have to sell it because it's yours, and the amount of pride and confidence that these kids have is amazing.

Do you see a good follow-through with the program?

This will be our 12th year in Houston, and we are starting to see students that have graduated from high school go through college and eventually come back as mentors. So we have a dozen or so students now that went from being mentees to mentors. One of my favorite stories is from I was doing a presentation at an architecture firm, and this young woman, an architect, comes up to me. She has a brochure in her hand, and she's pointing to the picture on the back, saying that the photo was of her old mentor, and she was wondering how to get ahold of her. That young woman is now a mentor and is on our associates' board, helping with our student recruitment and several other things. So I'm happy to see the cycle of keeping former students engaged. ■

This article is excerpted from my conversation with Angela. To hear the podcast in its entirety, visit ***modernsteel.com/podcasts****.*

business issues

LEVERAGING LAYOUT

BY IAN WARNER

Layout plays an important role in efficient, successful steel projects.

Ian Warner is product manager for Trimble's Field Technology Group. For more information, visit **buildings.trimble.com/field-technologies**.

THERE'S BEEN A LOT WRITTEN about how structures are conceptualized and designed, and there's no shortage of information about how every aspect of a structure is physically erected.

But for those who aren't closely involved in the full construction process, the integral set of steps between those two bookends is largely a mystery.

While often underappreciated, the project phase in which ideas generated by the architects, engineers, and detailers are first translated into the real world, also known as construction layout, is critical to success. Layout is required for every single item within a building, from the concrete foundations to the light fixtures, with steel anchor rods, embeds, and framing members leading the way for the project. Just as a building constructed on a weak or damaged foundation is bound to crumble, the installation of an anchor rod, column, or beam based on inaccurate layout points is bound to cause cascading issues for the rest of the project's duration.

The core purpose of layout is to transfer reference points and alignment from detailed drawings or models to the site to facilitate construction. Without the reference points and subsequent field points supplied by surveyors and layout professionals, no actual building work can commence, much less succeed. At the same time, errors made during the layout process can have a snowball effect on the rest of the build, leading to costly RFIs, rework, delays, and even potential safety hazards. This further underscores the important role layout plays in an efficient, successful building project.

Layout for Steel Erection

As one of the first trades to arrive on-site after the initial surveying and site prep work has been completed, steel layout often involves structural layout tasks along with more detailed point marking within the structure as construction progresses.

For decades, this work was a completely manual process that required at least two people, with large or complex projects usually requiring larger layout teams. These teams would carry around a host of tools, including string lines, 300 ft and 100 ft, and regular tape measures, levels, plumb bobs, piano wire, and more. Highly complex calculations were done manually as well, creating massive potential for human error when computing angles, using trigonometry, and even just adding dimension strings, let alone fractional feet—all in the field and on the back of scrap steel or wood.

The first electronic total stations introduced in the early 1970s were a step in the right direction, as were CAD advances related to the generation of layout points and verification of point placement accuracy, but there was still room for improvement. Most of the people using early total stations and CAD programs were surveyors and engineers, leaving steel installation crews feeling like these were magic processes that require advanced degrees to master. This perception still permeates the steel industry today, with many erectors and fabricators continuing to outsource complex layout to surveyors or general contractors.

Advances in Layout Hardware

In recent years, advances in technology have dramatically improved the accuracy, speed, and efficiency of construction layout. Robotic total stations go beyond the capabilities of traditional mechanical total stations by reducing the opportunity for human error and freeing up layout professionals for higher priority work. Controlled remotely using a tablet or controller, robotic total stations make it possible for one person to handle even the most complex layout tasks without needing to be a trained surveyor.

Newer robotic total stations can be operated from almost 2,000 ft away and equipped with self-stabilizing magnetic motors. While most total stations can measure to a prism, reflective glass usually mounted to a survey rod, some are equipped with visible laser pointers that can measure with the laser and show exactly where the building components are to be placed with true elevations. There are even models that measure with a red laser beam and then switch over to a green laser beam to indicate the point is at the exact position, all within a few seconds. Basically, the layout process can be as easy as: red light, green light, install!

The most advanced modern robotic total stations not only handle the necessary measurements and calculations on the fly, but also provide powerful visualization tools, such as photographic documentation and augmented reality-style overlays, and coordinate directly with powerful layout software solutions for true field-to-office connectivity. Similar in function and benefit to robotic total stations are laser-based rapid positioning tools and GNSS receivers that allow steel contractors to choose the features, tolerances, and price point required for a given project.

Layout Software

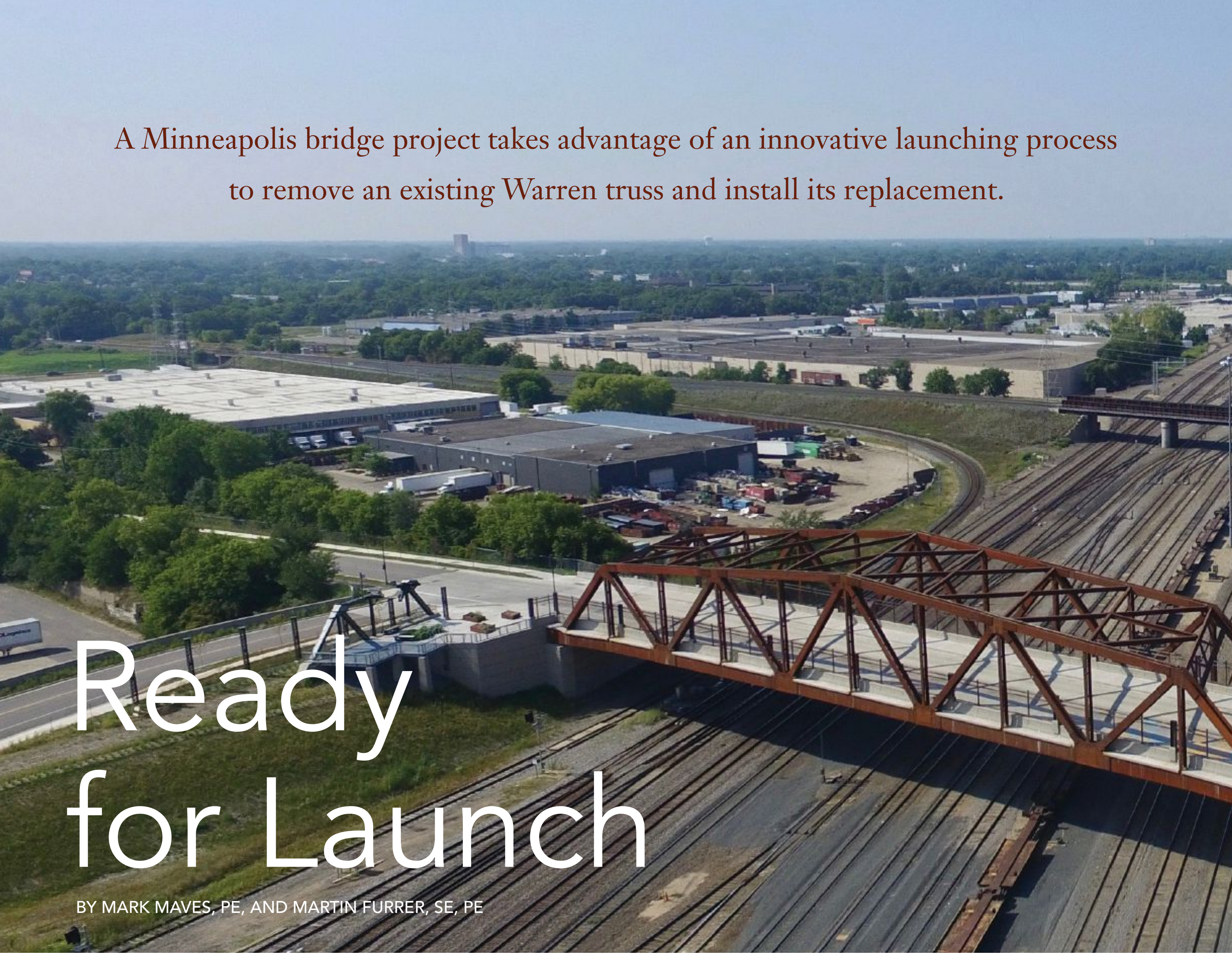
Moving beyond the paper drawings that layout professionals relied on in the past, cutting edge software seamlessly integrates the layout process into the overall construction workflow via building information modeling (BIM). These BIM platforms not only accommodate 2D plan views but also 3D views that provide a better understanding of the entire project. Some layout controllers in the field can now handle large 3D models, created by designers and detailers, that contain constructible data on components down to the nut, bolt, and rebar level of detail. These structural 3D models can be combined with architectural models, MEP models, or even PDF drawings to ensure notes, sections, and other details are not overlooked. The layout professional or foreman can combine models and PDFs and then use smart layering tools and section boxes to focus on certain items or areas, or easily access almost all of the project information.

Many steel fabricators and erectors now have the option to request a layout using IFC models, which are similar to CAD drawings except that the 3D objects contain data such as the steel shape, type, piece marks, weight, center of gravity (for lifting), elevations, and more. This allows the layout professional or foreman to help with installation while precisely pointing to the exact location with a laser or prism. With new improvements such as component-based set-up, fabricators and field crews can layout complex assemblies in the fabrication shop or in the field for precise pre-fabrication, even with the assembly laying down versus being drawn vertically, such as with stair railings or canopies.

As the project progresses, updated plans or models can be sent from the office to the field controller, and as-built data can be fed back from the field to the office to update the model in near real-time, so all project stakeholders stay informed. The data gathered by connected equipment on-site can easily be viewed in context by all stakeholders for clash detection, fabrication, and more. Fabrication shop or field crews can now update the models with installation status, field reports, and as-built deviations, and send these updates to the office with the click of a button.

As many of the anchor rods, embeds, sleeves, and other items are literally set in concrete by the time the steel erection crews arrive, adopting a robotic total station layout can help steel erectors and fabricators achieve quality installations. The ability to quickly check the concrete crews prior to the placement of these critical components can save thousands of dollars and weeks of rework, and enhance relationships with owners, designers, contractors, and other trades on the project—and it's worth considering for your next project. ■

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.

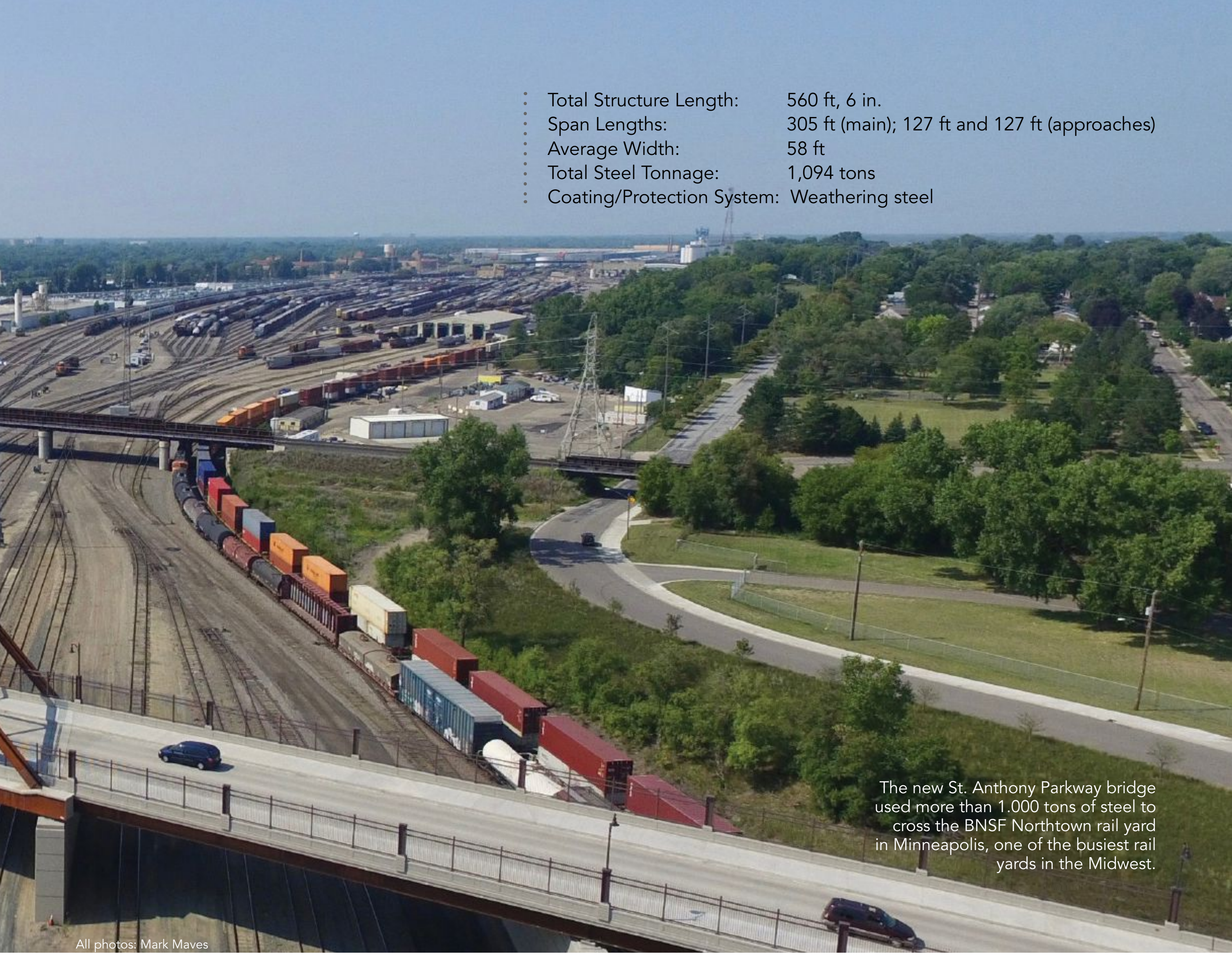
THE ST. ANTHONY PARKWAY BRIDGE is at a nexus of sorts.

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 severely impeded BNSF operations and flexibility for future expansion. Due to these



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

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.

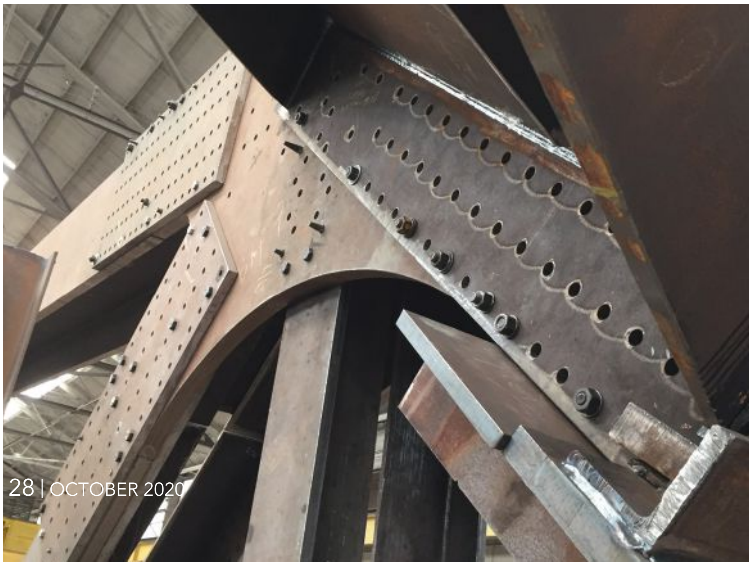
When it came to the new bridge's design, community feedback during public outreach meetings pushed for an above-deck structure visually similar to the existing steel Warren trusses and that the aesthetics should maintain an urban industrial feel. As a result, the bridge's main span consists of a redundant 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.

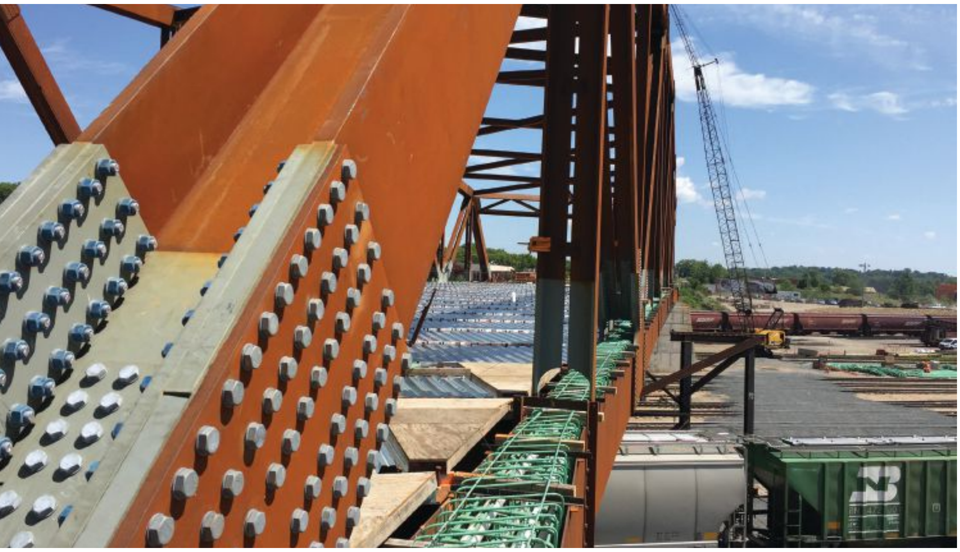
Selected contractor Lunda Construction Company's erection approach closely followed the launching approach envisioned by the design team with one notable change: The three easternmost truss spans were removed conventionally in the yard, using 10-day closure windows for the yard tracks. The launching system was sized for the roughly 800-ton new truss structure but was first used to remove the two existing westernmost steel trusses, The new bridge trusses were installed and the old ones removed via a set of launching beams.

including the concrete deck system, over the BNSF's main line tracks to the western approach embankment for demolition.

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.

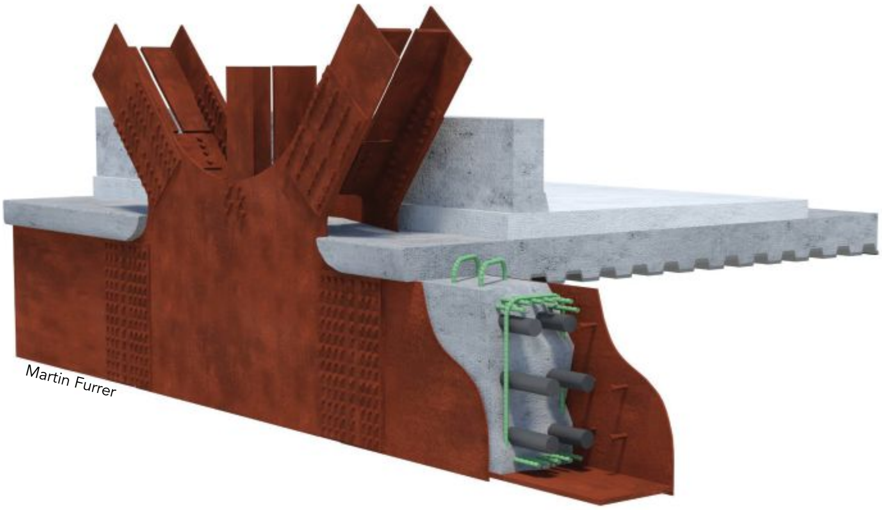
The longitudinal jacking setup consisted of a series of post-tensioning bars coupled together and supported on wood blocking spanning between the top flanges of the twin launching beams. Two jacks located at the end of the launching beams pushed against the transversely spanning jacking beam that in turn is connected to the leading post-tensioning

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.



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 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. ■

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.

Owner
City of Minneapolis - Public Works

General Contractor
Lunda Construction Company

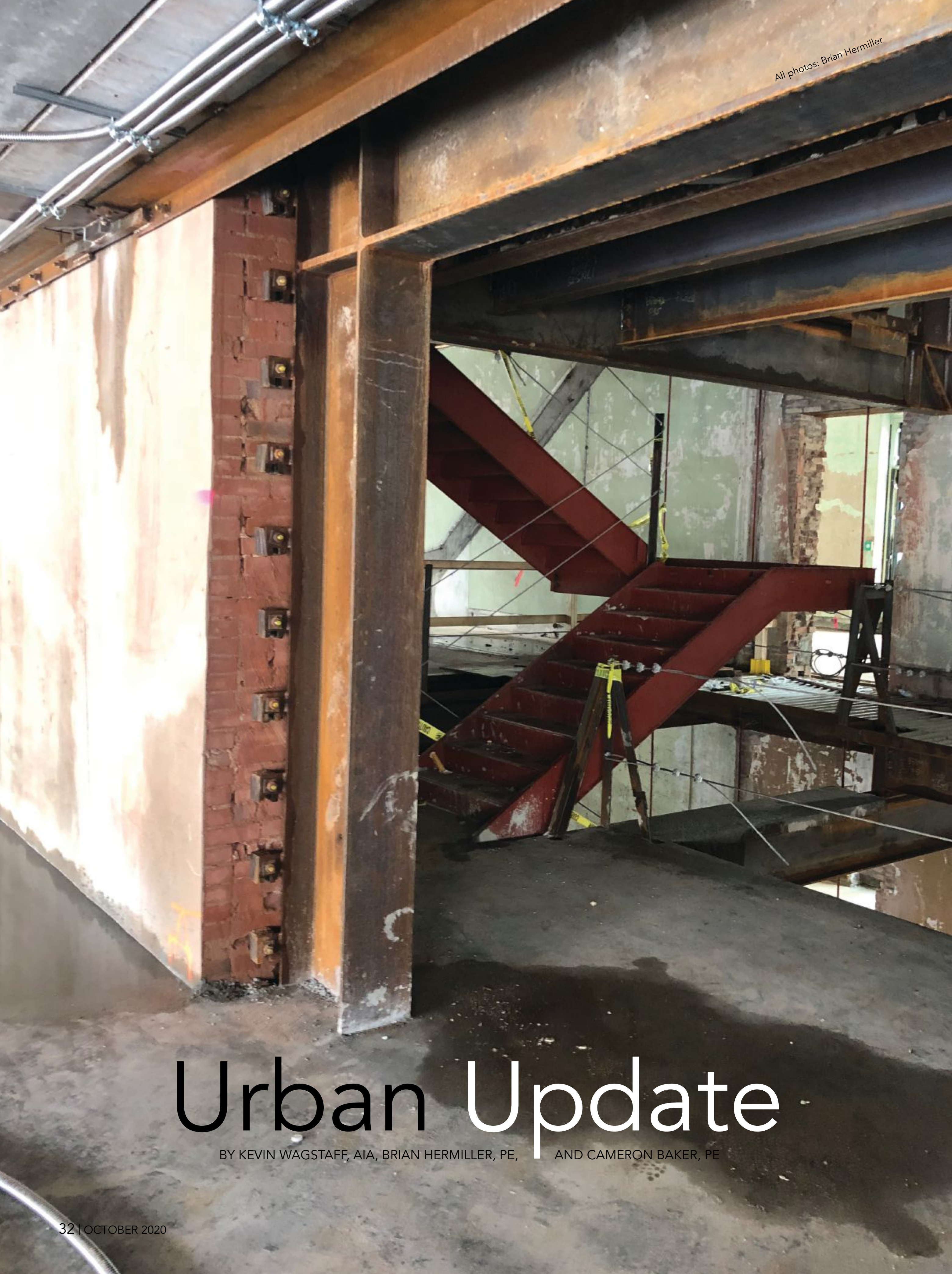
Structural Engineers
Short Elliott Hendrickson, Inc. (SEH), St. Paul, Minn. (EOR)
Parsons Corporation, Minneapolis and Chicago (EOR, truss span)

Steel Team

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

Erector
Danny's Construction Company, Inc., Shakopee, Minn.

Detailer
Tenca Steel Detailing, Inc., Quebec, Canada



All photos: Brian Hermiller

Urban Update

BY KEVIN WAGSTAFF, AIA, BRIAN HERMILLER, PE, AND CAMERON BAKER, PE

A downtown Pittsburgh structural expansion blends steel history with steel present.

PITTSBURGH'S STEEL STORY continues to this day.

Like many American cities that experienced rapid expansion during the industrialization of the late 1800s and early 1900s, Steel City has many "transitional" structures from that era integrating structural steel framing with various cladding and fire-resistive systems, terracotta floor and wall systems, mass masonry, and wood.

These structures are generally considered to have maintained robustness for floor loading, but the way systems were originally integrated does not lend to easy structural modification for adaptive reuse. In addition, the refined craft and preservation skills needed to work with these historic materials make them challenging to maintain and modify.

The McNally and Bonn Buildings in downtown Pittsburgh are two such transitional structures. The buildings, both eight stories with full basements, were built in the 1890s and encompass a combined total of 65,600 sq. ft of space. Both are long, slender structures with footprints approximately 25 ft by 140 ft. The McNally Building has perimeter masonry bearing walls and structural clay tile flat arch floor and roof construction supported by steel beams that span the width of the building. The Bonn Building has perimeter masonry bearing walls and wood floor deck supported by timber joists that span the width of the building. In fact, the two buildings share a masonry bearing wall.

And now they share much more. The $35 million Eighth and Penn project has combined these two historic structures with two new steel-framed buildings to create a total of 173,100 sq. ft of mixed-use space featuring 136 apartment units and street-level retail and restaurant space. The resulting structure is an elegant ensemble with an exterior that appears as a collection of old and new buildings and an interior that functions as a single building.

The design presented a number of unusual and complex structural challenges, including:

- Integrating a new vertical circulation system to serve three different floor-to-floor heights in a single core
- Integrating old and new at the interface between the existing structures and the new construction
- Maintaining the structural integrity of the mass shear walls of the historic buildings while removing large sections to make connections to the vertical circulation core in the Bonn Building

New Vertical Circulation

For optimal circulation and to maximize natural lighting in the apartment units, the new stair and elevator core were centrally located within the existing Bonn Building. Because the floors in the historic buildings and the adjacent addition do not align, the core was made accessible from four sides on each floor, with a front and rear opening elevator, ramps, and steps resolving the varying floor elevations at each story. And because the difference in floor elevations is inconsistent throughout, unique steel framing solutions were devised for each level of the new core. Each of the access points to the core required different steps and ramp designs at most levels. In order to accommodate these varying elevations throughout while maintaining clearances above and below, the team designed and fabricated mitered steel "Z-beams" that span the width of the Bonn Building. Posts and hangers were also used to support beams that could not span across the width of the building without causing interferences, even with a mitered beam configuration.

Connecting Historic to New Construction

The new portion of the project consists of two wings. The 11-story main wing, directly abutting the Bonn Building, is similar in height to the historic buildings but has lower floor-to-floor heights, allowing for three additional stories in the same building height. The steel framing cantilevers from the new column grid to avoid imparting gravity loads on the existing wall. The second wing, the rear wing, is seven stories

Kevin Wagstaff (**kwagstaff@pwwgarch.com**) is a principal-in-charge with PWWG, **Brian Hermiller** (**bhermiller@tsepgh.com**) is a principal and vice president with Taylor Structural Engineers, and **Cameron Baker** (**cbaker@sippelsteelfab.com**) is a structural engineer with Sippel Steel Fab.

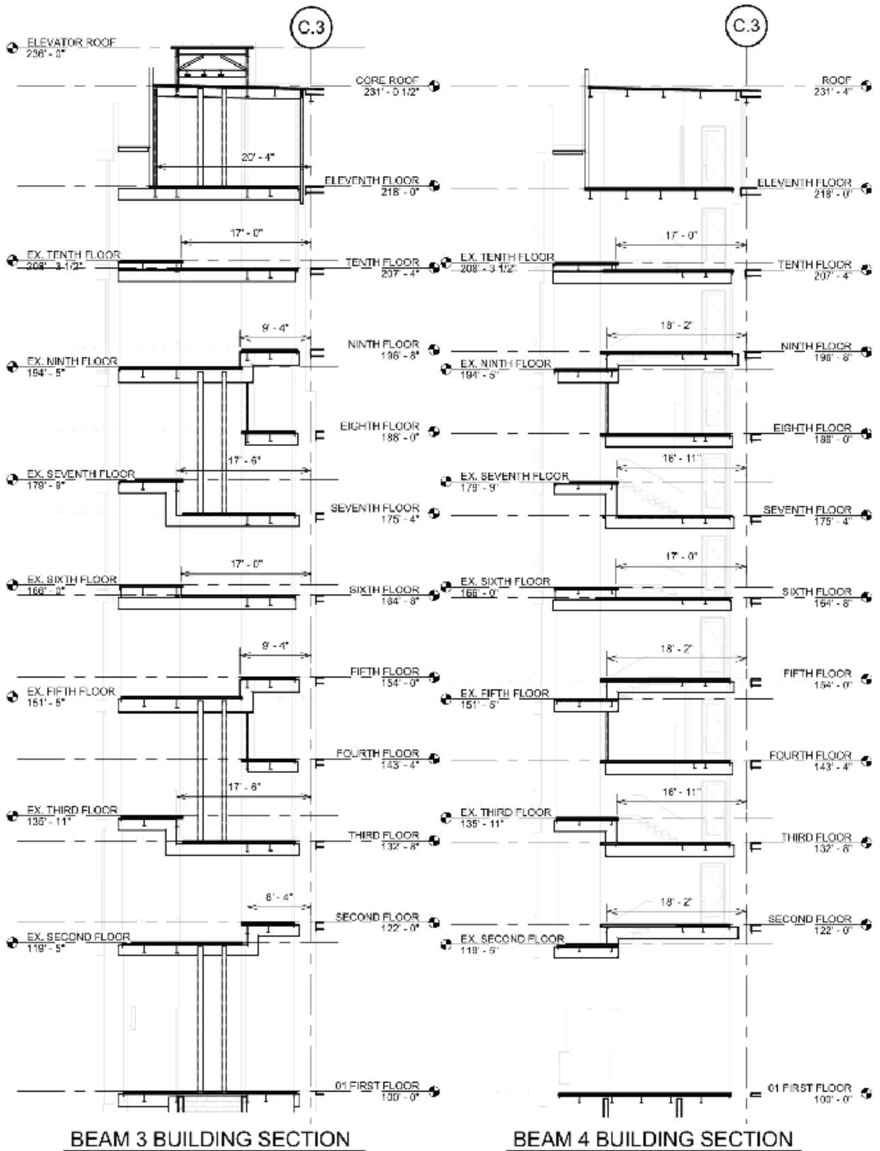
In order to accommodate varying elevations while maintaining clearances above and below, the team designed and fabricated mitered steel "Z-beams" that span the width of one of the existing buildings.

with a usable green roof and a connecting element to the main wing that bridges over a public walkway. The new portion uses a grout-topped hollow-core concrete plank floor system supported by a 700-ton structural steel frame and is stabilized by shared existing shear walls and moment-resisting frames.

The uneven surface of the Bonn Building's exterior masonry wall also created a challenge, especially since the height and scale of the elevation did not lend itself to conventional survey techniques. To overcome the challenge of connecting to this uneven surface, the general contractor retained local firm Cadnetics to create a 3D point cloud of the existing wall to map the existing conditions. The steel detailer (Sippel Steel, also the project's steel fabricator) then used this data to coordinate the final dimensioning of new steel framing with the existing conditions. The 3D point cloud allowed Sippel to adjust the length of the cantilevers throughout to the surveyed location, and a field adjusted pour stop allowed the erector to close the gap after erection and plumbing of the frame were complete but before the hollow-core plank was grouted.

Reestablishing Lateral Integrity

The two historic buildings relied on perimeter brick masonry shear walls for lateral stability in their long direction. Early in the project, it was determined that these existing shear walls would be sufficient to brace both the new and existing construction in one direction once the addition was completed. However, as the design of the stair and elevator core evolved, it was necessary to remove a large section of the masonry between the Bonn Building and the main wing of the addition, essentially breaking one long wall into two shorter shear walls. A Vierendeel-style steel coupling frame was installed to reestablish the connection of the two resulting shear walls with comparable stability to the original wall.



Where the Z-beam framing scheme could not clear span the building, posts and hangers connecting to adjacent levels were used to support the framing.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13. **Tee Stem Up** — MEAN DIA. — 22" x 142 1/2# Tee
14. **Angle Heel In** — INSIDE DIA. — 8" x 8" x 1" Angle
15. **Angle Heel Out** — OUTSIDE DIA. — 8" x 8" x 1" Angle
16. **Angle Heel Up** — MEAN DIA. — 8" x 8"x1" Angle
17. **Square Tube** — INSIDE DIA. — 24" x 1/2" Tube
18. **Rectangular Tube The Easy Way (Y-Y Axis)** — INSIDE DIA. — 20" x 12" x 5/8" Tube
19. **Rectangular Tube The Hard Way (X-X Axis)** — INSIDE DIA. — 20" x 12" x 5/8" Tube
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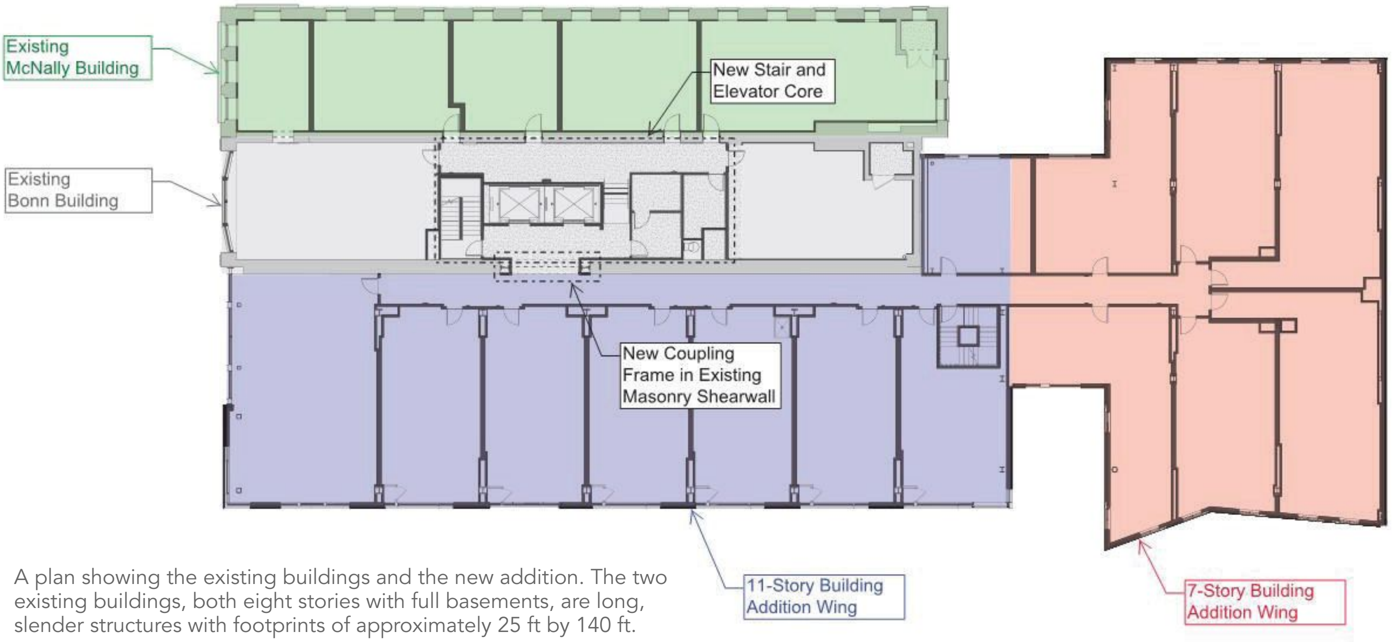


A plan showing the existing buildings and the new addition. The two existing buildings, both eight stories with full basements, are long, slender structures with footprints of approximately 25 ft by 140 ft.

To stabilize the original shear wall, the designers and construction team had to carefully coordinate demolition sequencing for the existing wall segment, installation of the coupling framing, and erection of the new construction. To maintain overall building stability, wall demolition was not performed until the new construction was completely erected and the permanent lateral system was established. Sections of the coupling frame were lowered into position by crane between the new and existing structures. Crane capacity and reach and shipping size limitations were constraints that drove fabrication decisions for the coupling frame, and splice

1. The two existing buildings prior to construction of the new portion.

2. Erection of the 11-story addition.

locations were chosen to eliminate field welding, which would have been nearly impossible given the location of the frame between two completed structures.

In the short direction of the existing buildings, there was inadequate lateral bracing to meet lateral loads mandated by today's building codes. Improving or supplementing the existing shear walls was determined to be cost-prohibitive and impractical without significant disturbance to the buildings' historic architecture. Therefore, a new lateral load resisting system was designed for the new building to provide stability for the entire completed ensemble. Steel moment-resisting frames were selected to accommodate expansive glass facades and the open first-floor plan. Drift limitations higher than those mandated by code were necessary to assure deformation compatibility with the existing masonry walls and to minimize second-order effects that result from the mass of both existing buildings leaning on the new structure.

3. The new steel is topped out.

4. Substantial completion of the new addition.

The Eighth and Penn project is an example of a creative structural adaptation that brings modern amenities, accessibility, and resiliency to a historic building while maintaining the urban fabric of a vibrant downtown area. It's an excellent illustration of the flexibility of structural steel as a framing option to solve complex geometric challenges when joining old to new. ■

Owner
Trek Development Group

General Contractor
Mistick Construction, Pittsburgh

Architect
Perfido Weiskopf Wagstaff + Goettel (PWWG), Pittsburgh

Structural Engineer
Taylor Structural Engineers, Pittsburgh

Steel Fabricator and Detailer
Sippel Steel Fab, Ambridge, Pa. AISC CERTIFIED FABRICATOR

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. 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,

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 ($/ft^2)	Capitalized Costs ($/ft^2)	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

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.

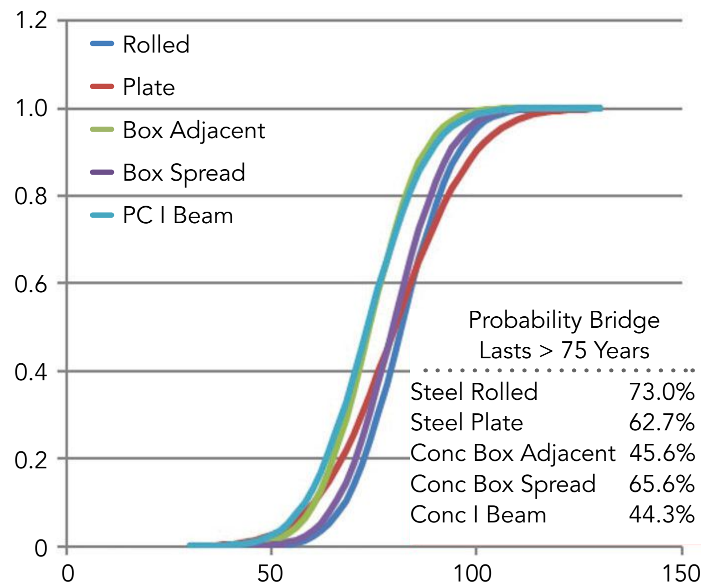


Fig. 1. Cumulative density function for bridge life (all bridges).

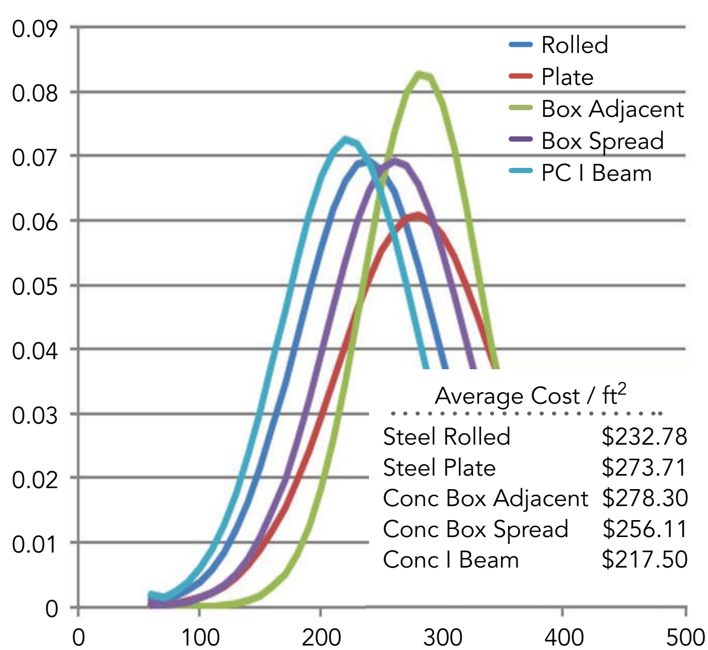


Fig. 2. Probability density function for capitalized costs (all bridges).

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**.

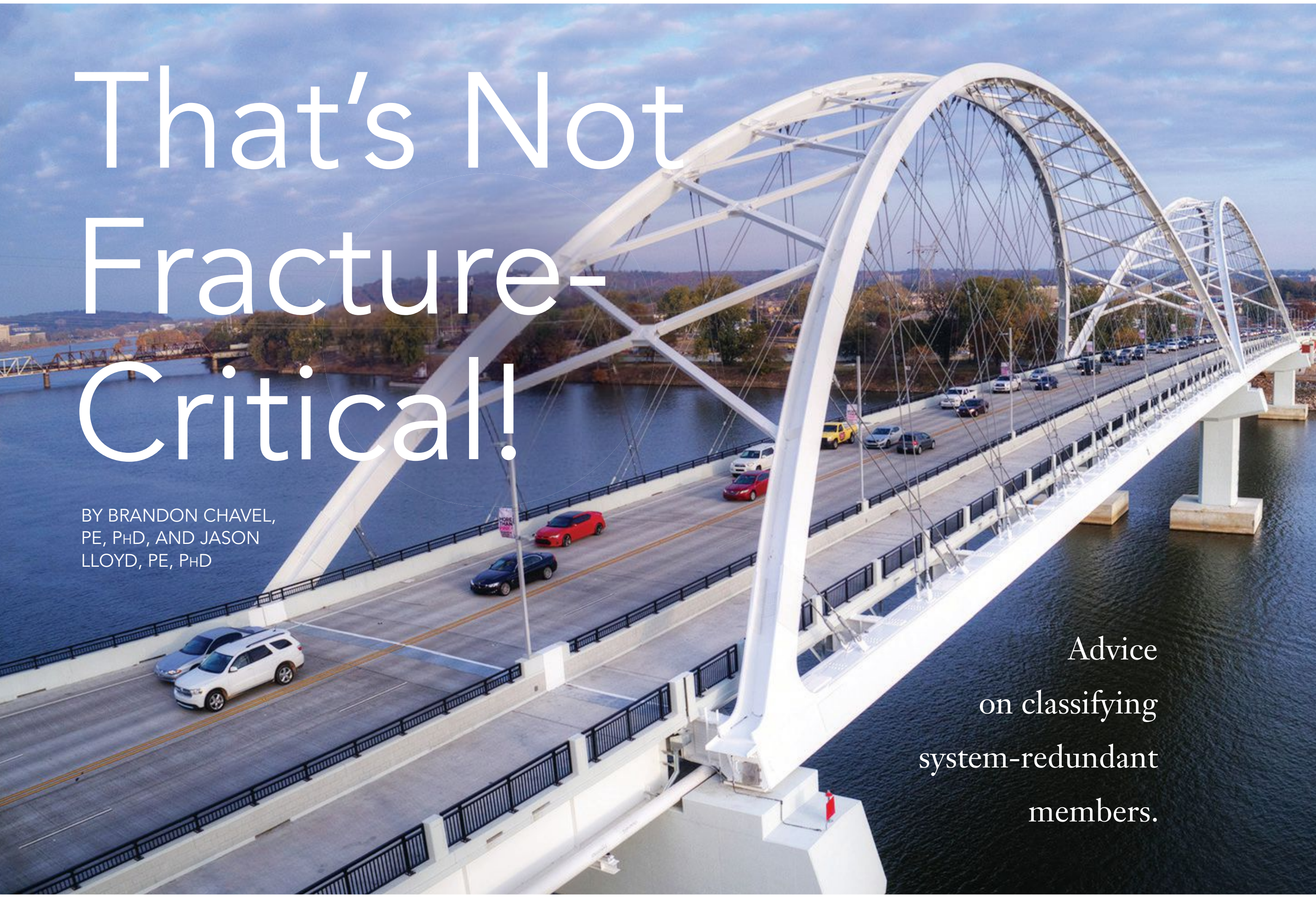

TEG Engineering

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.

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. ■



That's Not Fracture-Critical!

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

Advice on classifying system-redundant members.

Brandon Chavel (chavel@aisc.org) is director of market development, and **Jason Lloyd** (lloyd@aisc.org) is the West Region bridge steel specialist, both with AISC's National Steel Bridge Alliance.

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-left: The Broadway Bridge over the Arkansas River in Little Rock, Ark., a 2020 NSBA Prize Bridge Award winner (to read about it, see the July 2020 issue at **www.modernsteel.com**).

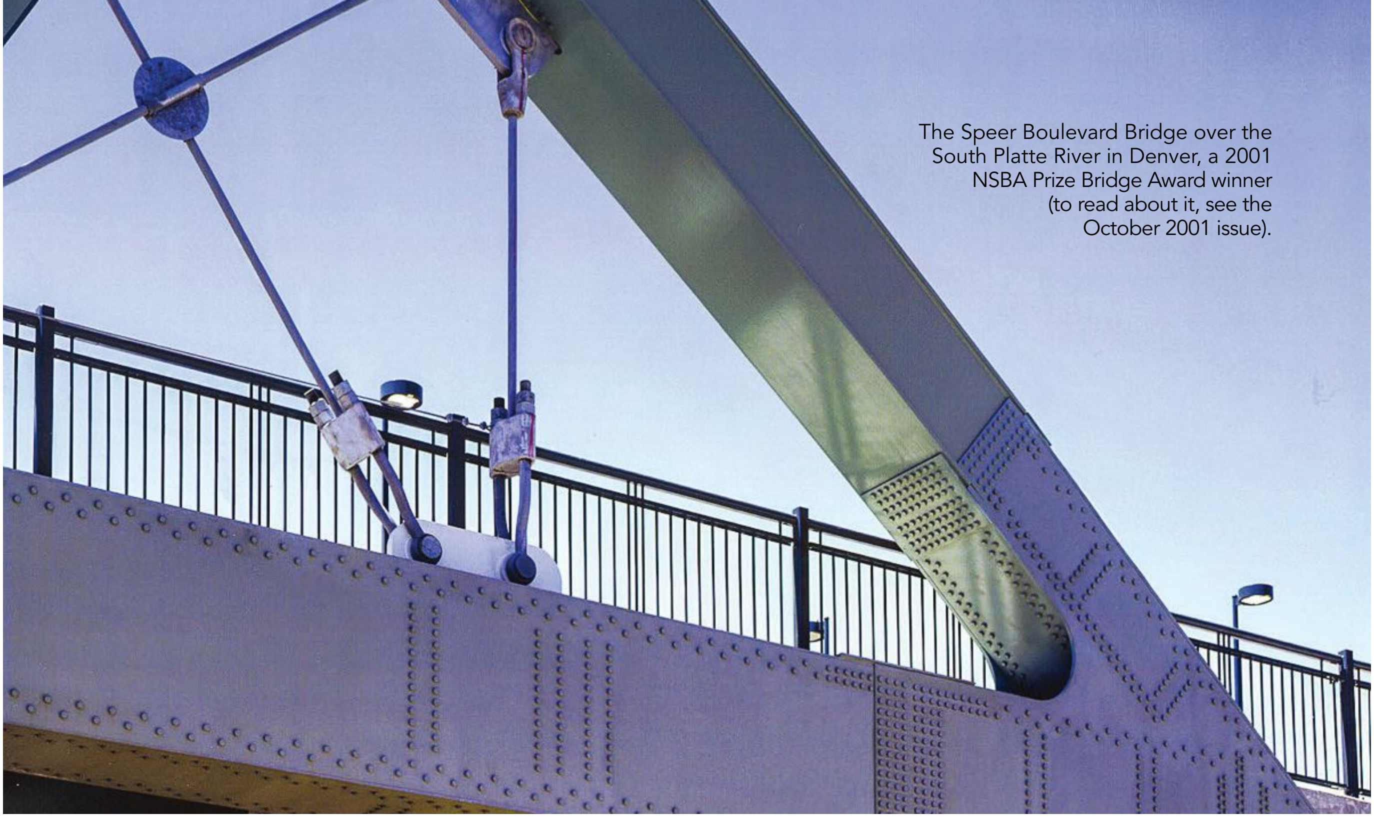
below: The 130th Street and Torrence Avenue Railroad Truss Bridge in Chicago, a 2014 NSBA Prize Bridge Award winner (to read about it, see the July 2014 issue).

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 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.



The Speer Boulevard Bridge over the South Platte River in Denver, a 2001 NSBA Prize Bridge Award winner (to read about it, see the October 2001 issue).

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.

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.

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

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.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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.

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-

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

Cem Korkmaz (**ckorkmaz@purdue.edu**) is a research engineer, and **Robert Connor** (**rconnor@purdue.edu**) is the Jack and Kay Hockema Professor and the Director of the S-BRITE Center (as well as an AISC T.R. Higgins Award winner), both with the Lyles School of Civil Engineering at Purdue University in West Lafayette, Ind.

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.

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

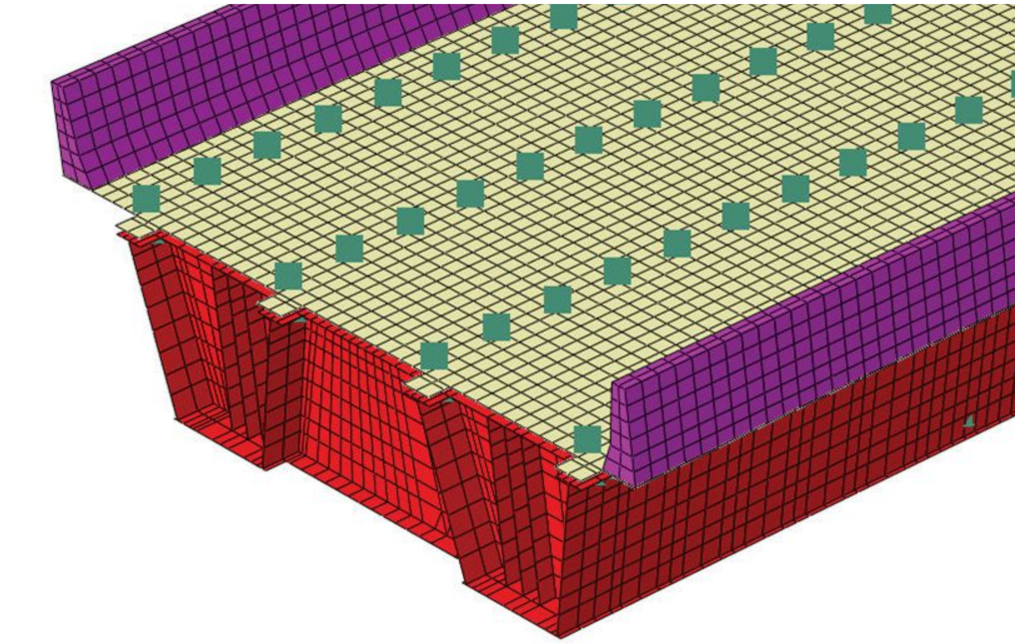
1. The shear stud provisions
2. The provisions for intermediate diaphragms
3. The bottom flange buckling resistance provisions
4. The maximum positive moment flexural resistance

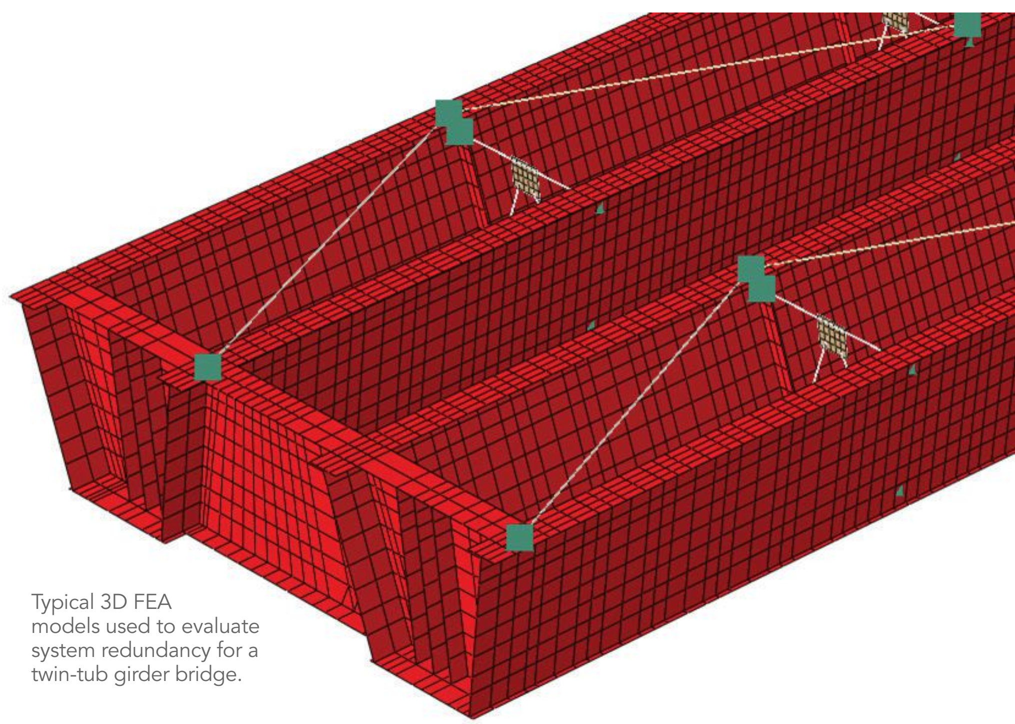
The study revealed that the following failure modes need not be explicitly considered under the Redundancy load factors in the faulted state if the above are satisfied:

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.





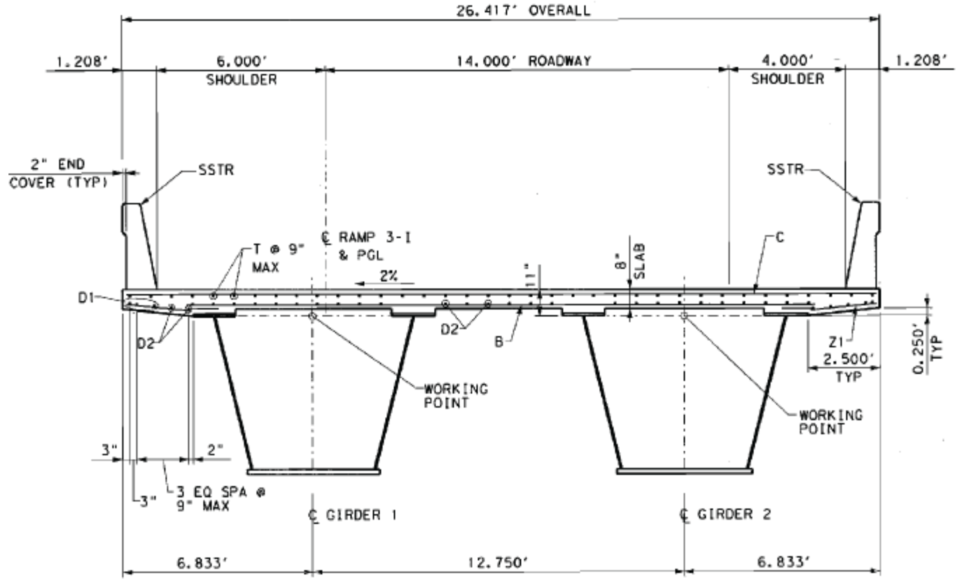
Typical 3D FEA models used to evaluate system redundancy for a twin-tub girder bridge.

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 $2/3$ of the maximum longitudinal spacing used for three studs, or 14 in. ($2/3 \times 22$ 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;



A sample twin-tub girder bridge cross section.

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.

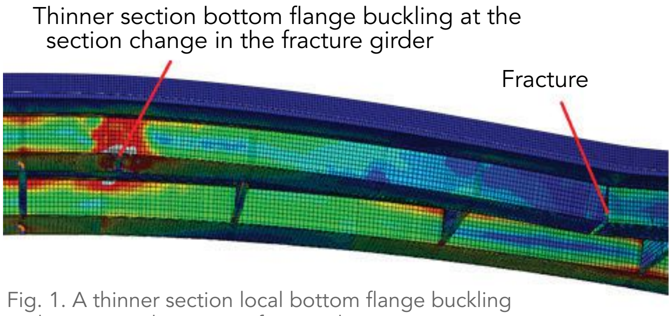


Fig. 1. A thinner section local bottom flange buckling at the section change in a fractured span.

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

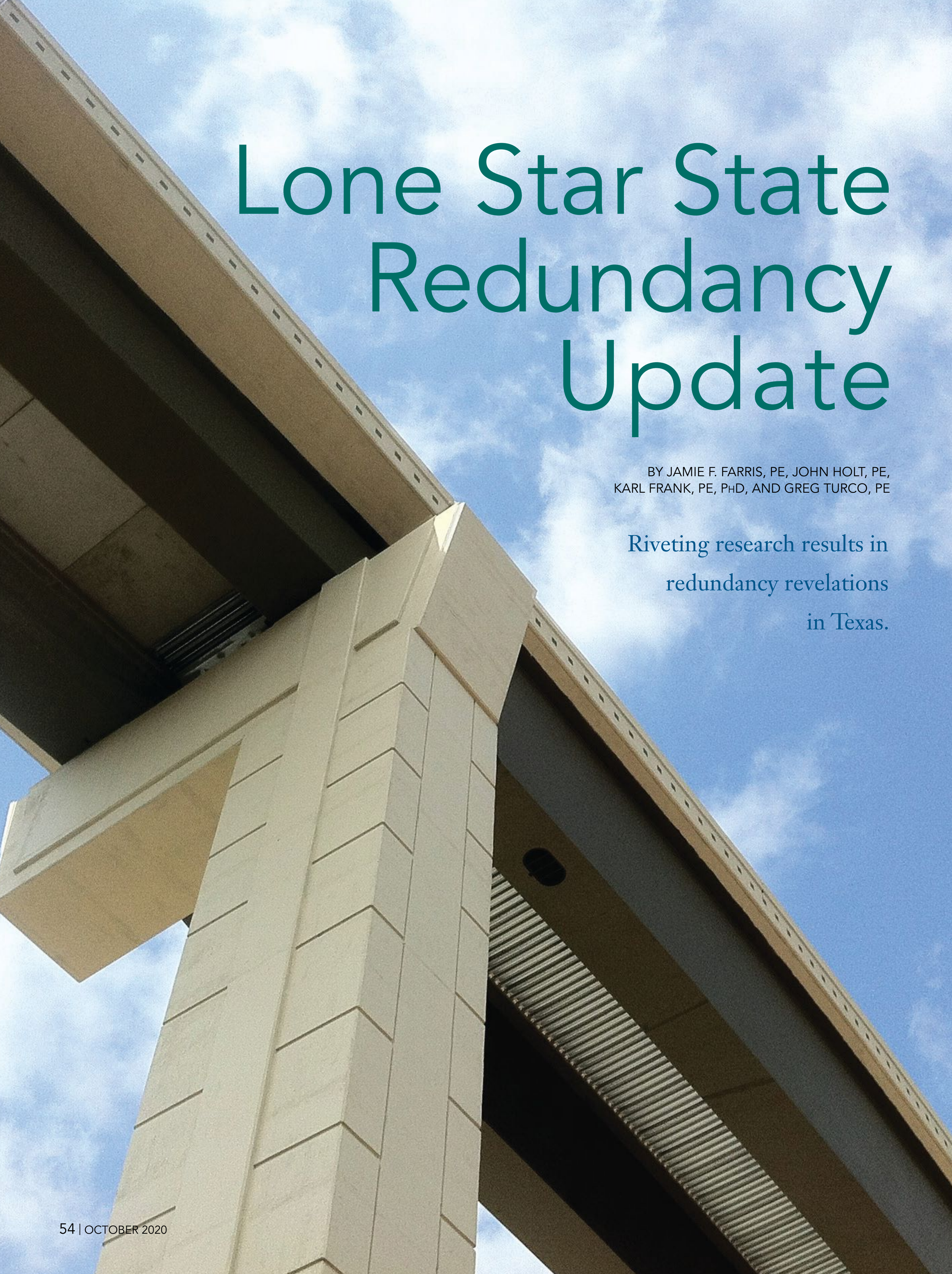
After fracture occurs, a significant amount of the load is redistributed from the fractured girder to the intact girder. In the fractured span, the positive moment flexural resistance of the intact girder should be checked. The most critical location for this check is at the maximum positive moment closest 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.

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.



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.

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.

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-

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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 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

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

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

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

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.

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 42), 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 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.

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

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

below: 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

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 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.

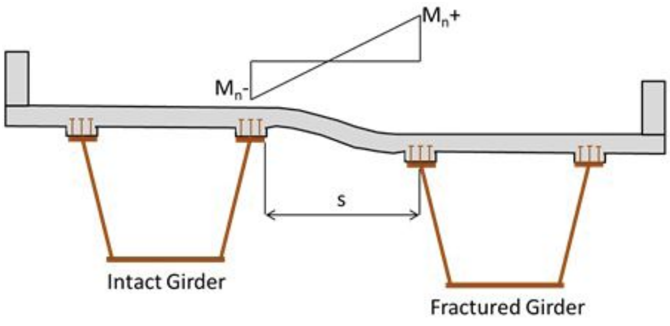

Fig 1. Deflected shape and moment diagram before any failure of shear studs.

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. ■

HSS

A Totally Tubular Resource

While HSS (hollow structural sections) are increasingly specified for columns and bracing, availability is still a concern with some designers. To help alleviate this issue, the Steel Tube Institute (STI) has developed a new tool to indicate which sizes are most often produced in the U.S. The free capability tool can be accessed at **steeltubeinstitute.org/hollow-structural-sections/capability-tool/**.

"This tool was designed to help specifiers and those within the steel supply chain locate producers of specific sizes of HSS," explained Joseph Anderson, STI's executive director. "To use, simply designate the size (or range), material grade (ASTM A500 B/C, A1085, or A1065) then click on the search button. A chart will display indicating the sizes that are produced by STI member companies. Click on any square to view which STI producers currently manufacture or can manufacture the size requested."

STI not only represents manufacturers of products like HSS, steel conduit, standard pipe, and mechanical tubing—as well as provides code and standard support—but also serves as a resource for end users of HSS and conduit.

"Over the last eight years, we have focused much of our efforts on developing specific resources—like webinars, e-newsletters, and design manuals—that engineers, fabricators, and service centers would find valuable," said Kim Olson, a consulting engineer at STI.

In addition to its Capability Tool, STI offers an extensive resource library on its website (**steeltubeinstitute.org/resources-overview/resources/**). Topics range from composite column connections to CJP HSS welds. In all, the library offers nearly 150 free articles, papers, and brochures.

"Another good resource is our e-newsletter," Olson added. The newsletter is sent out eight times per year and contains information about technical articles and recent developments in HSS design and availability. You can subscribe to the newsletter at **steeltubeinstitute.org/email-sign-up/**.

"As STI continues to increase our efforts, we feel it is essential to hear from you: the design community and supply chain," Anderson said. STI is specifically seeking input on the following questions:

- What can we create that will help you as it pertains to HSS?
- What resources do you need that you do not have?
- What topics should we expand on?

Everyone who submits input by October 31 will be entered in a drawing to win one of two $50 Visa gift cards. Please send any input to **hssinfo@steeltubeinstitute.org**.

PUBLIC REVIEWS

Seismic Provisions, Code, Structural Stainless Steel Spec All Available for Public Review

Public review periods for the 2021 or 2022 editions of three AISC publications are all underway or will be soon: the AISC *Seismic Provisions for Structural Steel Buildings* (AISC 341), the AISC *Code of Standard Practice* (AISC 303), and the AISC *Specification for Structural Stainless Steel Buildings* (AISC 370).

The *Seismic Provisions* draft is available for public review until October 16. A draft of the *Code* is available for public review from October 1 until October 31. Finally, a draft of the *Structural Stainless Steel Specification* will be available from October 14 to November 11.

Drafts of all three publications and their respective review forms are available at **aisc.org/publicreview**. The *Structural Stainless Steel Specification* will be completed and available in 2021, and the *Seismic Provisions* and *Code* are expected to be completed and available in 2022. Review copies are also available (for a $35 charge) by calling 312.670.5411.

Please submit comments using the forms provided online. You can also submit *Seismic Provisions* and *Structural Stainless Steel Specification* comments to Cynthia J. Duncan, AISC's director of engineering (**duncan@aisc.org**), by October 16 and November 11, respectively, for consideration. *Code* comments can be submitted to Jonathan Tavarez, secretary of the Committee on Code of Standard Practice (**tavarez@aisc.org**), by October 31 for consideration.

People and Companies

- **ASCE**'s newest publication, ***Composite Special Moment Frames: Wide Flange Beam to Concrete-Filled Steel Column Connections***, provides a state-of-the-art overview for designing connections for composite special moment frames (C-SMFs), focusing on beam-to-column moment connections for both continuous beam and continuous column connections, and C-SMFs with rectangular or circular concrete-filled tube (CFT) columns. Prepared by the Composite Construction Committee of the Metals Technical Administrative Committee, the book provides an experimental database of 165 tests conducted on beam-to-column connections for C-SMFs, design equations, and examples for estimating the panel zone shear strength of double split-tee. You can purchase it via the ASCE Bookstore at **ascelibrary.org**.

- **DeSimone** has welcomed **Joseph Castellano, PE**, as a new director in the firm's risk management services practice. As a licensed Professional Engineer and a construction professional, Mr. Castellano has held leadership positions at major firms in the construction, real estate, and insurance industries, focusing on construction advisory, forensic insurance and contract claims, and litigation advisory. He has served clients, both public and private, across multiple market sectors, including healthcare, hospitality, transportation, retail, higher education, industrial, and manufacturing.

ENGINEERING JOURNAL

Fourth Quarter 2020 *EJ* Now Available

The fourth quarter 2020 issue of AISC's *Engineering Journal* is now available. (You can access this issue as well as past issues at **aisc.org/ej**.) Below is a summary of this issue, which includes articles on self-centering beam moment frames, special cantilever column systems, and continuity plates.

Experimental Investigation of a Self-Centering Beam Moment Frame
Matthew R. Eatherton and Abhilasha Maurya

The self-centering beam (SCB) is a shop-fabricated unit that can be implemented in moment-resisting frames using conventional field construction methods to minimize permanent residual drifts after earthquakes and concentrate seismic damage in replaceable elements. An experimental program was conducted on five SCB specimens that were approximately two-thirds scale relative to a prototype building. The results showed that the beam end moments are not equal, as much as 60% different at peak moment, so total flexural strength, calculated as the sum of the moments at both ends, is a better way to characterize SCB flexural strength. Using this approach, the proposed equation to predict flexural strength exhibited an average error of 5% compared to the tests. The SCB was shown to have exceptional deformation capacity as the specimens were subjected to as much as 6% story drift, and the detailing was shown effective at concentrating inelasticity in the replaceable energy dissipating elements. The proposed design procedure is shown to be capable of controlling the story drift associated with undesirable limit states, limiting story drifts at zero force (eliminate residual drifts), and producing no observable inelasticity outside the energy-dissipating element at design-level drifts.

Technical Note: Unbraced Length Requirements for Steel Special Cantilever Column Systems
Robert J. Walter and Chia-Ming Uang

AISC *Seismic Provisions* Section E6.4b for steel special cantilever column systems (SCCS) requires clarification based on inquiries to the AISC Steel Solutions Center. In the 2016 edition, it is unclear if bracing is required for all special cantilever columns or for columns with unbraced lengths that exceed the maximum beam brace spacing of L_b per Section D1.2a for moderately ductile members. Instead of using Equation D1-2, which is applicable to I-shaped beams only, equations for SCCS columns have been derived for both I-shaped members and rectangular HSS or box-shaped members. The proposed revision provides specific situations when bracing is required.

Steel Structures Research Update: Continuity Plate Design for Special and Intermediate Moment Frames
Judy Liu

Recent advances in continuity plate design for special and intermediate moment frames are highlighted. The featured research includes a comprehensive experimental and computational study by Dr. Chia-Ming Uang and Dr. Mathew Reynolds. Chia-Ming Uang is a professor of structural engineering at the University of California, San Diego (UCSD). Dr. Reynolds completed this research as his doctoral work at UCSD under Dr. Uang's guidance and is now working as a bridge engineer for Kiewit in Burnaby, British Columbia, Canada.

FLASH STEEL CONFERENCE

AISC to Host First Flash Steel Conference this Month

Registration is now open for The Flash Steel Conference!

Taking place October 20-22, this new fall conference will deliver 20 timely (and efficient!) technical sessions to you online—in a flash! Each 30-minute webinar will be taught by top speakers from the structural steel industry, such as Larry Muir and Amit Varma (the keynote speakers), as well as Carol Drucker, Jon Magnusson, Duane Miller, and many more.

Participants can earn up to 10 PDHs, and there are registration options for both individuals and groups (AISC will assist with group registrations for those working from home). For more information on the sessions and speakers, as well as to register, visit **aisc.org/flash**.

CONSTRUCTION COLLABORATION

Design Assist or Delegated Design? Industry Experts Release Recommended Guidance

Over the past few years, AISC and the American Institute of Architects (AIA) and have worked together to study design collaboration techniques commonly used in the construction industry. As a result of this joint effort, AIA and AISC have published a paper titled "Design Collaboration on Construction Projects. Delegated Design, Design Assist, and Informal Involvement—What Does it all Mean?" (available at **aisc.org/design-collaboration**).

Along the way, they discovered that the terms "design assist" and "delegated design," while commonly used in the industry, often mean different things to different people. These differences can result in contrasting expectations amongst project participants. Thus, the two organizations set their goals for the paper: describe the roles and responsibilities of project participants in these design collaboration scenarios and offer definitions and guidelines that design professionals and the construction industry can adopt for their use.

This jointly authored paper will be released in two parts. Part 1 (now available) generally focuses on three collaborative techniques: informal involvement, design assist, and delegated design. Part 2, which will be published at a later date, will address design assist as it relates to fabricated structural steel.

safety matters

Welcome to Safety Matters, which highlights various safety-related items. This month's topics are gantry crane safety and new OSHA rules of agency safety practice and procedure.

Gantry Cranes

Overhead and/or gantry cranes are one of the largest safety hazards in the metal fabrication industry. Due to the size and weight of the objects often being lifted and transported by these cranes, routine inspections are necessary to ensure continued safe operation. An inspection of the crane (new or altered) is required prior to initial use. Once placed into service, overhead cranes require two different types of inspection. Frequent inspections are done daily to monthly, while periodic inspections are completed at monthly to annual intervals. The purpose of the two inspection types is to examine critical components of the crane and to determine the extent of wear, deterioration, or malfunction.

When it comes to periodic inspections, here is a list of items to be inspected:

- Deformed, cracked, or corroded members
- Loose bolts or rivets
- Cracked or worn sheaves and drums
- Worn, cracked or distorted parts, such as pins, bearings, shafts, gears, rollers, locking, and clamping devices
- Excessive wear on brake-system parts, linings, pawls, and ratchets
- Inaccuracies in load, wind, and other indicators
- Electric, gasoline, diesel, or other types of motors for improper performance
- Excessive wear of chain drive sprockets and excessive chain stretch
- Deteriorated electrical components, such as pushbuttons, limit switches, or contactors

In addition to the initial inspection, OSHA also requires that all new and altered crane-functions are tested for:

- Hoisting and lowering
- Trolley travel
- Bridge travel
- Limit switches, locking, and safety devices

Frequent Crane Inspections

Items To Be Inspected	Frequency
Functional operating mechanisms for maladjustment	Daily
Deterioration or leakage in lines, tanks, valves, drain pumps, and other parts of air or hydraulic systems	Daily
Hooks with deformation or cracks (visual)	Daily
Hooks with deformation or cracks (written record with signature of inspector and date)	Monthly
Hoist chains and end connections for excessive wear, twist, or distortion interfering with proper function, or stretch beyond manufacturer's recommendations (visual)	Daily
Hoist chains and end connections for excessive wear, twist, or distortion interfering with proper function, or stretch beyond manufacturer's recommendations (written record with signature of inspector and date)	Monthly
Running rope and end connections for wear, broken strands, etc. (written record with signature of inspector, rope identity, and date)	Monthly
Functional operating mechanisms for excessive wear	Daily to monthly
Rope reeving according to manufacturers' recommendations	As recommended

Practice and Procedure Rules

OSHA has revised its "Rules of Agency Practice and Procedure Concerning Occupational Safety and Health Administration Access to Employee Medical Records." Responsibility for the overall administration and implementation of the procedures has been transferred from the Assistant Secretary to the OSHA Medical Records Officer (MRO). OSHA reports that the procedures set forth in § 1913.10 are internal agency procedures and do not affect employer compliance with OSHA requirements. Employers and employees will benefit from the following revisions:

- Employers will know sooner if such a review is authorized at their work site
- The procedures to protect the security and privacy of employee medical records will be strengthened
- The elimination of the requirement to remove direct personal identifiers before taking medical information off-site will enhance employee privacy

Date of Interest

Fire Prevention Week, October 4-10, National Fire Protection Association, **www.nfpa.org**

We are always on the lookout for ideas for safety-related articles and webinars that are of interest to AISC member companies. If you have safety-related questions or suggestions, we would love to hear them. Contact us at **schlafly@aisc.org**. And visit AISC's Safety page at **aisc.org/safety** for various safety resources. In addition, AISC has established its own resource page with information on employment, contract, and safety issues regarding COVID-19. It's at **aisc.org/covid19**.

"Presumption is the opposite of prevention."
—Bhavik Sarkhedi

marketplace & employment

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Quality Management Company, LLC (QMC) is seeking qualified independent contract auditors to conduct site audits for the American Institute of Steel Construction (AISC) Certified Fabricators and Certified Erector Programs.

This contract requires travel throughout North America and limited International travel. This is not a regionally based contract and a minimum travel of 75% should be expected.

Contract auditors must have knowledge of quality management systems, audit principles and techniques. Knowledge of the structural steel construction industry quality management systems is preferred but not required as is certifications for CWI, CQA or NDT. Prior or current auditing experience or auditing certifications are preferred but not required. Interested contractors should submit a statement of interest and resume to **contractor@qmconline.org**.

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advertiser index

MOVABLE FEAST

THE VAST DESERTED AREA lining the Quebec City marina calls out for new public uses.

But how to fill the void?

Université Laval students Hatem Bouassida and Antoine Hurez have an idea: Food Machine. This visionary steel design would bring new life to surrounding streets in all seasons, providing a lively and stimulating addition to the urban promenade between the city's marina, Lower Town, and Upper Town. Its location would be easily accessible to cruise passengers, tourists, and residents, making it an ideal starting point for exploring the historic city. Much like a machine, the building's parts can move and pull apart, following the trajectory of the tracks in the former marshalling yard and evoking memories of the port area's industrial past.

Food Machine is one of the winners of this year's Student Design Steel Competition awards. Administered by the Association of Collegiate Schools of Architecture (ACSA) and sponsored by AISC, the competition encourages North American architecture students to explore the myriad functional and aesthetic uses for steel in design and construction. This year's competition included two categories: Category I (for which this project is a winner) was designated as Urban Food Hub, and Category II was an open competition.

You can learn more about Food Machine, as well as all of this year's winners, in next month's issue (you can also view the winners at **www.acsa-arch.org**).