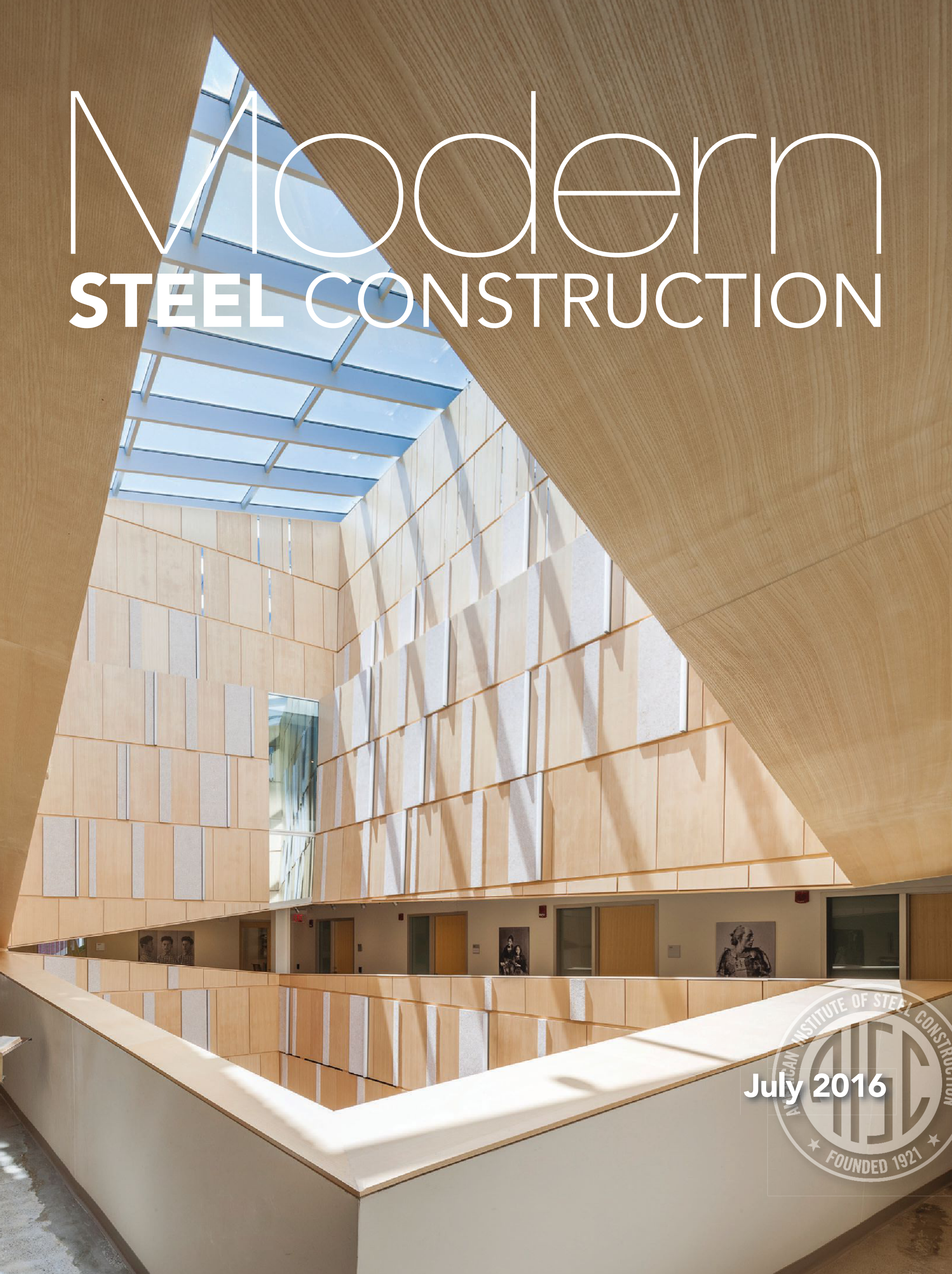
Modern STEEL CONSTRUCTION



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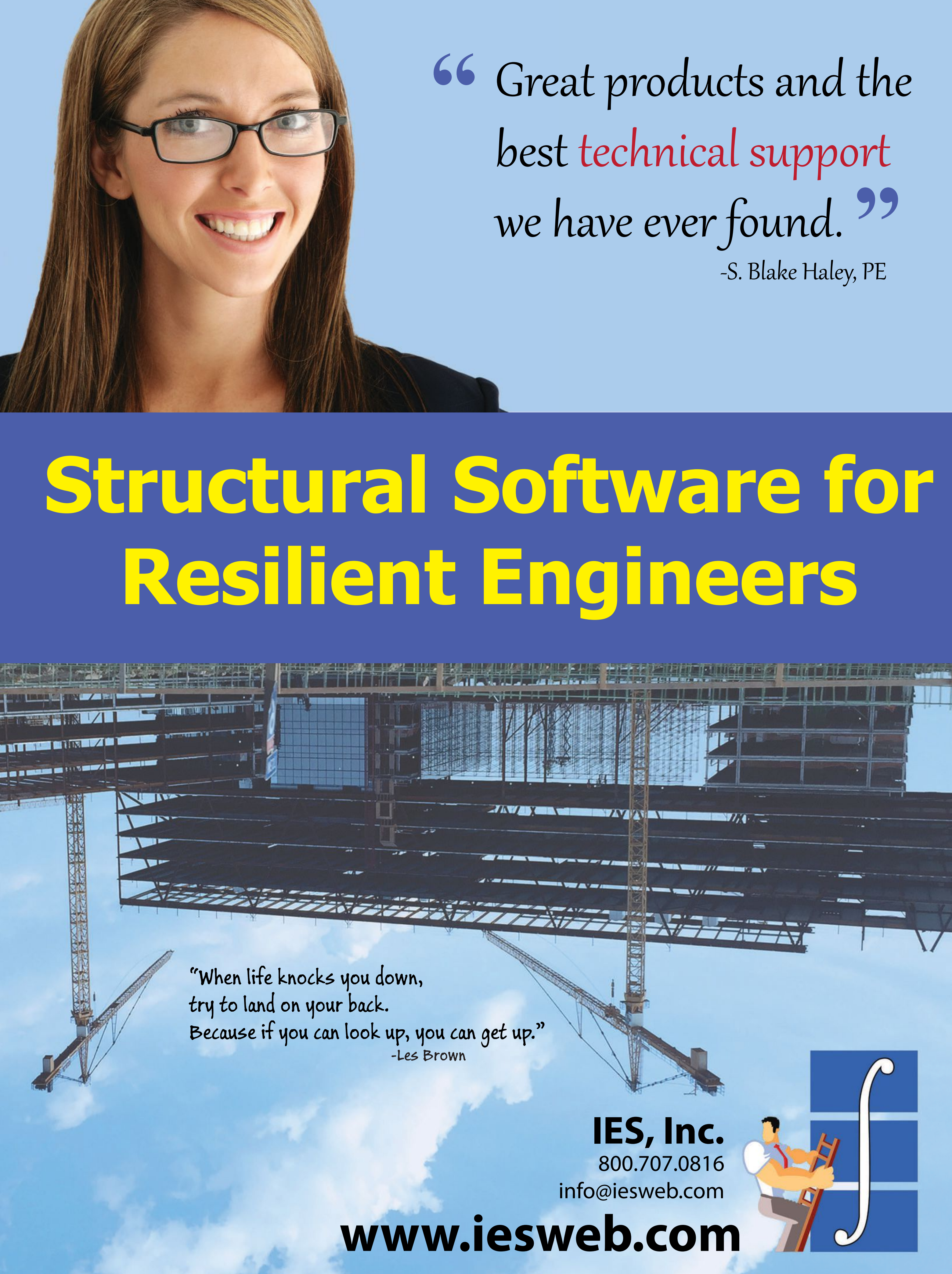
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Modern STEEL CONSTRUCTION | July 2016

columns

in every issue

features

ON THE COVER: An angular addition tops Tozzer Library at Harvard University, p. 26. (Photo: John Horner)

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editor's note

MY DAUGHTER, JULIA, IS INTENT ON GRADUATING FROM COLLEGE IN FOUR YEARS. But since her multiple majors make that difficult, she's resorting to augmenting her normal coursework by taking online courses through a couple of community colleges both over the summer and during the school year.

I'm pretty happy about the ability to take courses online, especially from a financial standpoint (there's a few thousand dollars' difference for each course!). I'm also happy she's taking a mix of online and in-person courses, though.

From a personal perspective, I find I get more out of an in-person course, primarily because I pay more attention to a person with whom I'm in the same room. I have a similar problem with conference calls (though some of this is alleviated when we do video calls). In addition, I often learn more from the audience give-and-take than simply from the presentation; and I definitely value the opportunity to speak with a presenter after a course (or during breaks).

That said, online presentations also have their advantages. They often cost much less, you don't have any travel costs to attend, and you can usually stop and rewind to repeat something you either missed or didn't understand.

AISC has moved to primarily providing continuing education through online courses, with the pinnacle being our wonderful Night School program (with our most recent offering focusing on steel bridge design). Night School tries to combine the best of both worlds by initially offering the session live (through streaming) and later as a recorded program. To learn more, visit **www.aisc.org/nightschool**.

For live seminars, of course, your best bet is NASCC: The Steel Conference, held each year in the spring (for 2017, the conference will be March 22–24 in San Antonio). But in the spirit of our commitment to disseminating information as widely as possible, we also stream some of those sessions live and offer all of the sessions online after the conference. You can currently view more than 100 of the 2016 sessions at no charge by visiting **www.aisc.org/2016nascconline**. But act soon; beginning in late September, the videos will still be free to AISC members but non-members will have to pay for access.

Happy watching!

Scott Melnick

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steel interchange

Shear Fatigue in Bolted Connections

I am designing a structure that could be subjected to nearly one million loading cycles per year. The steel members have been designed considering fatigue. We have specified slip-critical connections with ASTM A490 bolts. However, I cannot find a section in the *Specification* from which to determine a reduced bolt stress based on the number of cycles in shear. Is merely specifying slip-critical connections sufficient to ensure the bolts do not fatigue?

Yes. The Commentary to Section 3.4 states: "Bolts installed in joints meeting all the requirements for slip-critical connections survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts." Because slip-critical joints transfer shear loads via friction between the faying surfaces, in theory the bolts are not subjected to shear stress. The practice of neglecting secondary bolt shear stresses in slip critical joints, which can occur due to joint deformations, has been verified experimentally. Therefore, the joint fatigue performance of the connected material in simple lap joints can be evaluated using Case 2.1 in 2010 AISC *Specification* Appendix 3 Table A-3.1.

Some connections are more complicated than the lap joint in Case 2.1. The potential for fatigue cracking is higher at copes, transitions, weld terminations and other areas of stress concentration. In these locations, a suitable case from Table A-3.1 should be selected to evaluate the fatigue performance.

Bo Dowswell, P.E., Ph.D.

Fireproofing and Long-Slotted Holes

Is it acceptable to use a connection with long-slotted holes when fireproofing is required? It seems that the degree of movement allowed by the long-slotted holes could damage the fireproofing.

Yes, it is permitted. The use of long-slotted holes is permitted by the AISC *Specification*. Generally long-slotted holes are provided to accommodate tolerances during erection, not to accommodate movement in service. Using slots to accommodate movement is not addressed in the AISC *Specification*, and the engineer must rely on their own judgment when evaluating this condition.

AISC has provided recommendations to avoid the use of bolts moving in long-slotted holes to accommodate expansion. There are concerns that the bolts could bind, preventing the intended movement, or that a sawing effect could occur with repeated motion. The May 2011 SteelWise article "Expansion Joint Considerations for Buildings" (available in the Archives section at **www.modernsteel.com**) provides further information.

Given the possibility that the movement in the slot could damage the bolt, it seems your concerns with the bolt damaging the fire coating could be valid. However, it may not be the only problem with the proposed detail.

Carlo Lini, P.E.

Stiffness Considerations for Fully Restrained Connections

I have several questions related to stiffness requirements for fully restrained moment connections:

1. **Wouldn't the presence of many connection types, even those traditionally used as moment connections, cause a reduction in stiffness below that of the beam (e.g., bolted or welded flange plate moment connections and extended end-plate moment connections)?**
2. **Is it necessary to explicitly account for this decreased stiffness in practice?**
3. **Can a flush end plate moment connection be used as a fully restrained connection?**
4. **Section J6 in the AISC *Specification* states: "Groove-welded splices in plate girders and beams shall develop the nominal strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice." The provision for groove-welded splices would seem to ensure sufficient strength and stiffness. Why is the same requirement not applied to other configurations of splices?**

I have addressed each of your questions below.

1. Yes, it is likely that the stiffness would be lower local to the connection.
2. Generally, no. As long as the connection can be considered fully restrained, then there is no need to account for the reduced stiffness.

   The Commentary to Section B3.6 states: "In many situations, it is not necessary to include the connection elements as part of the analysis of the structural system. For example, simple and FR connections may often be idealized as pinned or fixed, respectively, for the purposes of structural analysis. Once the analysis has been completed, the deformations or forces computed at the joints may be used to proportion the connection elements… For simple and FR connections, the connection proportions are established after the final analysis of the structural design is completed, thereby greatly simplifying the design cycle."

The *Specification* provides general requirements for fully restrained connections in two different sections. Section B3.6b states: "A fully restrained (FR) moment connection transfers moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states." Section J1.3 states, "End connections of restrained beams, girders and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section B3.6b." Both of these criteria mention stiffness. In practice, stiffness is often not explicitly checked but rather judged by inspection.

The definition of a fully restrained connection (or Type 1 connection, as it was once called) has varied some with time, as has the guidance related to this topic. The Commentary to the 2010 *Specification* provides a definition based on stiffness: "If $K_S L/EI \geq 20$, it is acceptable to consider the connection to be fully restrained (in other words, able to maintain the angles between members)." K_S is the secant stiffness of the connection at service loads. Relative to strength, it states: "The strength of a connection can be determined on the basis of an ultimate limit-state model of the connection, or from a physical test. If the moment-rotation response does not exhibit a peak load then the strength can be taken as the moment at a rotation of 0.02 rad (Hsieh and Deierlein, 1991; Leon et al., 1996)."

AISC Design Guide 16: *Flush and Extended Multiple-Row Moment End-Plate Connections* (a free download for AISC members at **www.aisc.org/dg**) refers to criteria related to both strength and stiffness: "For beams, guidelines have been suggested by Salmon and Johnson (1980), and Bjorhovde, et al. (1987, 1990), to correlate M-θ connection behavior and AISC construction type. Traditionally, Type 1 or FR connections are required to carry an end moment greater than or equal to 90% of the full fixity end moment of the beam and not rotate more than 10% of the simple span rotation (Salmon and Johnson 1980)." It also suggests a change might have been in the works when the design guide was being written: "More recently, Bjorhovde, et al. (1987, 1990) has suggested rotation criteria as a function of the connected beam span." Such criteria are now given in the *Specification*, as indicated above.

There is generally some correlation, though not direct, between strength and stiffness. Often, when designing for a specified strength, you get the stiffness for free, so to speak. This is not always the case, so some care must be exercised. Moment connections should look like they have significant rotational stiffness. If there is doubt, then more rigorous analysis should be conducted.

3. Yes, though this is the one connection type for which AISC provides a formal adjustment to account for the inherent flexibility of the connection. Relative to flush end-plates AISC Design Guide 16 states: "For FR rigid frame construction, the required factored moment, M_u, must be increased 25% to limit the connection rotation at ultimate moment to 10% of the simple span beam rotation. Therefore, the factor $\gamma_r = 1.25$ is used in the procedure for the flush connection plate design."

   For the conditions addressed in Design Guide 16, the authors use a 25% increase in the demand to "limit the connection rotation at ultimate moment to 10% of the simple span beam rotation," which was taken to be the criterion to consider full fixity in the model. (See Answer 2 for references.) The 25% increase seems to come from the early work of Thomas Murray at the University of Oklahoma; in that work it was applied as a factor of 1/0.8.

   The following reports seem to be pertinent:

   - "Analytical and Experimental Investigation of Stiffened Flush End-Plate Connections with Four Bolts at the Tension Flange" Report No. FSEL/MBMA 84-02, September 1984 by Hendrick, Kukreti, and Murray seems to suggest that the strength of the end plate be reduced by a factor of 0.75 to ensure Type I (fixed) behavior.
   - "Unification of Flush End-Plate Design Procedures" Report No. FSEL/MBMA 85-01, March 1985 by Hendrick, Kukreti, and Murray uses the 1/0.8 factor, which is equal to the 1.25 used in the Design Guide. Section 4.2 provides some further discussion related to the stiffness of these connections.

   Both of the reports and others related to end plate moment connections can be downloaded at **tinyurl.com/OUreports**.

   Thomas Murray continued working on end-plate moment connections at Virginia Tech, so you can find further information there as well.

4. The J6 provision reflects a practical, not a theoretical, consideration. It is intended to address concerns that during a retrofit or evaluation, a groove-welded butt splice might be overlooked if it were covered by fireproofing or even paint. Other splices would involve additional plates and/or bolts, which would be more easily seen during a survey of the structure. Requiring the groove-welded splices, to develop the full strength of the member ensures that even if they are missed, they will perform sufficiently. The requirement is not related to stiffness considerations.

Larry S. Muir, P.E.

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

Steel Interchange is a forum to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

If you have a question or problem that your fellow readers might help you solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact Steel Interchange via AISC's Steel Solutions Center:

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steel quiz

Steel Quiz made its first appearance in *Modern Steel Construction* in the November 1995 issue. This month's edition takes a look back at that very first quiz and kicks off a series of quizzes that will revisit some of the best questions over the past 20 years.

Note that some answers have been modified to bring them up to date (the changes are shown in red).

1 True or False: Physically, the U.S. customary shape series and metric shape series are identical.

2 Cross-sectional dimensions and standard mill practice for rolled shapes may be found in which of the following documents?

a. ASTM A36
b. ASTM A6/A6M
c. AISC's *Steel Construction Manual*
d. a and c
e. b and c

3 True or False: An installed bolt with the point of the bolt flush with the face of the nut is acceptable.

4 Which of the following methods for bolt installation is not recognized by RCSC and AISC?
a. standard torque
b. turn-of-nut
c. snug-tight
d. calibrated wrench

5 In which of the following positions can weld metal be deposited at the fastest rate?
a. horizontal
b. vertical
c. flat
d. overhead

6 Which is more costly: a ¼-in. fillet weld that's 10 in. long or a ½-in. fillet weld that's 5 in. long?

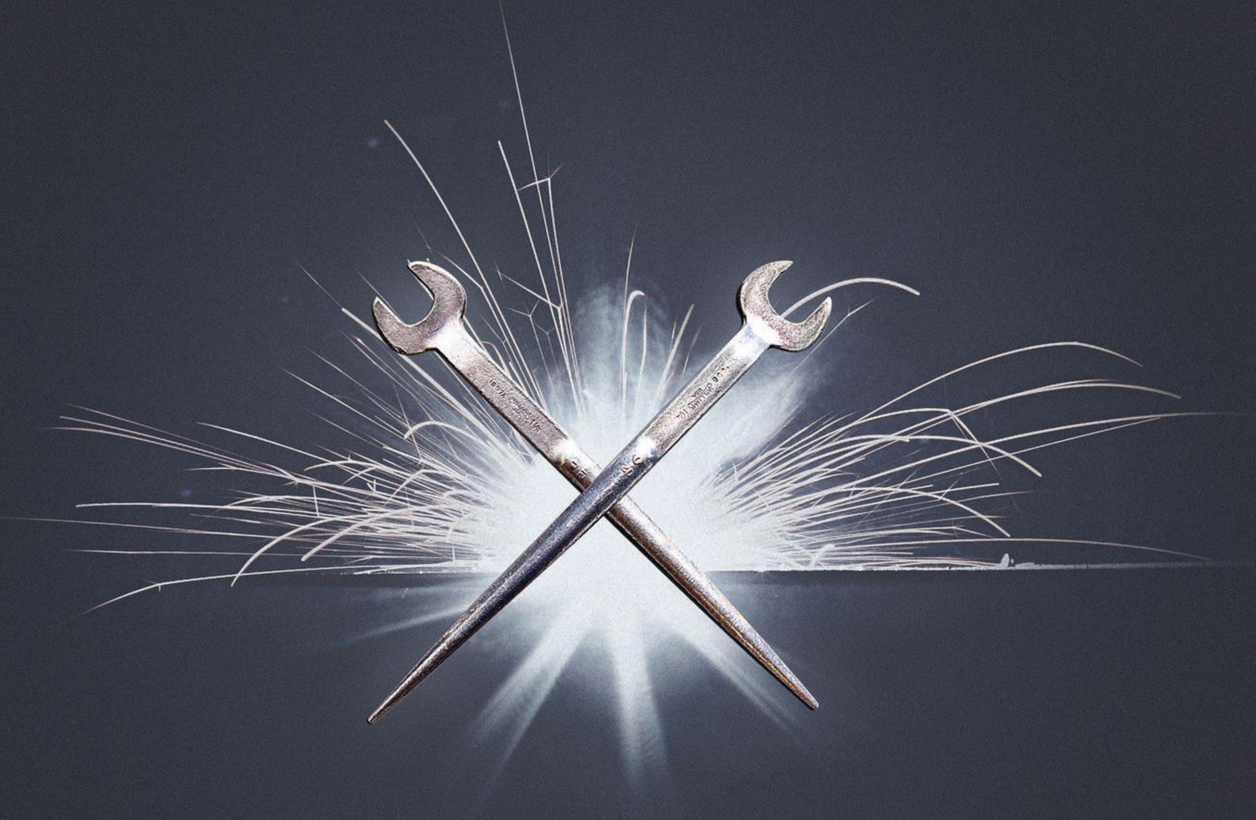
7 True or False: If an extended end-plate moment connection is specified as slip-critical, the slip resistance of the bolts at the tension flange must be reduced for the tension present.

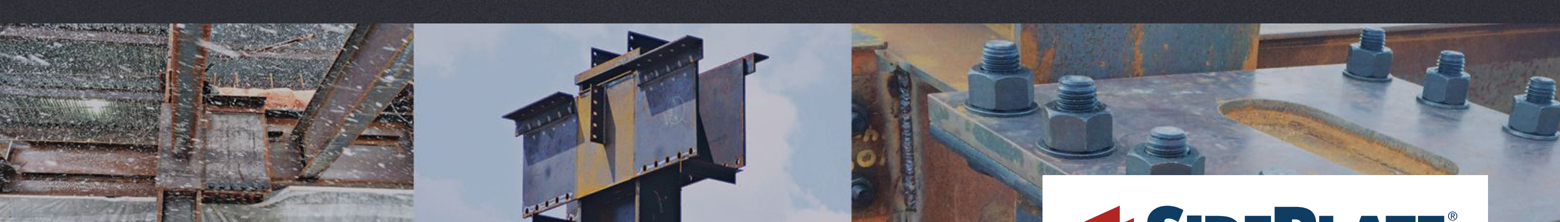
8 Which of the following factors is used to adjust for inelastic column behavior?
a. *K*
b. τ_b
c. *m*
d. *n*
e. *c* and *d*

9 In a partially composite beam, which of the following controls the flexural design?
a. compression in the concrete
b. tension in the steel
c. compression in the steel
d. shear strength of the shear stud connectors

10 Which of the following trusses does not use diagonal members?
a. Pratt
b. Fink
c. Warren
d. Vierendeel

TURN TO PAGE 14 FOR ANSWERS





steel quiz

ANSWERS

1 **True.** Because our current shape series is one of the most efficient in the world, and because the inch-series dimensions are nominal (e.g., a W14 is not exactly 14 in. deep), the metric series is simply a soft conversion of it.

2 **e.** ASTM A6/A6M is the standard that specifies cross-sectional dimensions and standard mill practice for rolled shapes. AISC's *Steel Construction Manual* summarizes this information in Part 1.

3 **True.** The RCSC *Specification* defines sufficient thread engagement as "having the end of the bolt extending beyond or at least flush with the outer face of the nut; a condition that develops the strength of the bolt."

4 **a.** Both RCSC and AISC discourage use of a standard uncalibrated torque value. Such an uncalibrated value may be too high and break well lubricated bolts or, more importantly, may be too low and result in under-tensioned bolts if the thread lubrication is poor or the threads are dirty or corroded. Therefore, if torque is to be used, it must be calibrated according to RCSC *Specification* Section 8.2.2.

5 **c.** Welding in the flat position allows the fastest deposition rate and is therefore the most economical welding position.

6 At first glance, these welds might seem to be of equal cost because they are of equivalent strength. However, because the volume of weld metal is proportional to the square of the weld size, the ½-in. weld uses twice as much weld metal as the ¼-in. weld. Additionally, a ½-in. weld will require multiple weld passes. In the end, the same strength will cost more than twice as much with the ½-in. weld.

7 **False.** Because the tensile and compressive flange forces are equal, any loss of slip resistance adjacent to the tension flange of the beam is compensated for by an increase in slip resistance adjacent to the compression flange. Note that end-plate connections can be designed as a bearing type connection. When this is done, it is common practice to assume that the compression bolts resist all of the shear force.

8 **b.** Commentary Section C2 in the 2010 AISC *Specification* states: "The τ_b factor is similar to the inelastic stiffness reduction factor implied in the column curve to account for loss of stiffness under high compression loads ($\alpha P_r > 0.5 P_y$), and the 0.8 factor accounts for additional softening under combined axial compression and bending."

9 **d.** The flexural strength of partially composite beams is controlled by the shear strength of the shear stud connectors.

10 **d.** A Vierendeel truss uses chord and vertical members without diagonals. Therefore, unlike other ideal trusses, Vierendeel truss members must also transmit member forces due to bending.

Steel SolutionsCenter

Everyone is welcome to submit questions and answers for 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 at **solutions@aisc.org**.

Correction

The answers provided for questions 2 and 4 in June's Steel Quiz incorrectly referred to Table 3-6 for the flexural strength values. The correct values can be obtained from Table 3-10 or Table 6-1. A corrected version of the quiz is posted in the Archives section at **www.modernsteel.com**. Special thanks to David Atkins with AECOM for being the first to bring this to our attention.

National Museum of African American History and Culture (NMAAHC)

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steelwise

A QUICK LOOK AT PRYING

There's no need to fear prying checks, and a new paper can help guide you through the process.

BY CARLO LINI, P.E.

THE PRYING CHECK PROCEDURE can be intimidating for first-time users.

There are many variables and equations in the procedure, which is presented in Part 9 of the 14th Edition of the AISC *Steel Construction Manual*, and the controlling limit state may not always be obvious.

For those that have struggled with this procedure, a paper has been posted on AISC's website that presents a different way to view the prying checks in the *Manual*. You can view the complete paper at **www.aisc.org/pryingcheck**. But for a summary of what it discusses, read on.

Increasing Strength

Prying may mistakenly be viewed as a flaw in a connection, a limit state that weakens the connection when the opposite can be true. As stated on page 9-11 of the *Manual*: "Alternatively, it is usually possible to determine a lesser required thickness by designing the connecting element and bolted joint for the actual effects of prying action with q greater than zero." One should view prying as a way to increase the strength of a connection. It is analogous to the post-buckling strength gained in plate girders from tension-field action.

Often, different models can be used in design with each producing an acceptable result. Simple models are often more conservative than more complex models. A simple, statically determinate model is shown in Figure 1.

In this model, the angle to the right of the bolt line is neglected. The moment is resisted through bending in the angle near the junction between the two legs. It is assumed that the capacity of the system has been reached when a single hinge is formed.

However, it must be recognized that a second hinge can form at the bolt line. A model that considers only the strength from this hinge is shown in Figure 2.

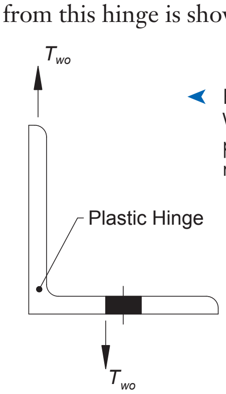


Figure 1. Without prying model.

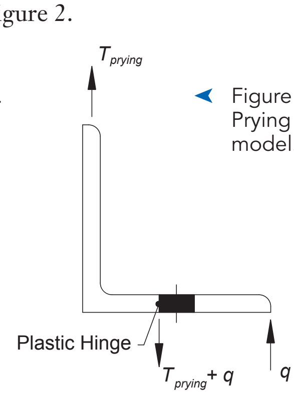
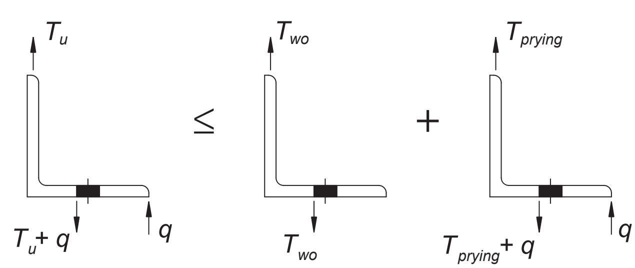


Figure 2. Prying model.

Essentially, additional restraint is added to the system when prying is considered, and adding restraint cannot weaken the system. (This is, in fact, a corollary to the lower bound theorem.)

To determine the available strength of an angle for prying, these two models can be superimposed as shown in Figure 3. Note that while this approach is shown for a single angle, it can easily be adapted to WT and wide-flange sections.



$$T_u \leq T_{wo} + T_{prying}$$

Figure 3. Breakdown of prying check.

Where:
T_{wo} = Connection Strength without Considering Prying
T_{prying} = Additional Connection Strength due to Prying
q = Prying Force

The load that can be carried based on the first model is the lesser of the moment that causes the first hinge or the strength of the bolt, B. If the model is limited by the bolt strength, then no additional strength can be gained from considering prying.

Carlo Lini (**lini@aisc.org**) is an advisor in AISC's Steel Solutions Center.

$$T_{wo} = \frac{\phi M_{wo}}{b'} = \frac{\phi F_u Z}{b'} = \frac{\phi F_u\left(\frac{p\,t^2}{4}\right)}{b'} \leq B$$

If the first model is not sufficient to transfer the load, then prying can be considered. The additional strength that can be added to T_{wo} can be calculated as follows:

The load that can be carried based on the formation of the second hinge is calculated.

$$T_{prying\_flexure} = \frac{\phi M_{prying}}{b'} = \frac{\phi F_u\left(\frac{(p-(d'))t^2}{4}\right)}{b'}$$

The strength of bolt must also be considered:

$$T_{prying\_bolt} = \frac{B - T_{wo}}{\left(1 + \frac{b'}{a'}\right)}$$

The available strength gained by considering prying is the lesser of the bolt strength and the angle strength:

$$T_{prying} = min \begin{cases} T_{prying\_flexure} \\ T_{prying\_bolt} \end{cases}$$

The total available strength of the connection is the sum of these:

$$T_{total} = T_{wo} + T_{prying}$$

where:
d' = width of the hole along the length of the fitting, in.
p = tributary length, in.
B = the available strength per bolt, kips.

Note that if T_u is less than T_{wo}, prying does not need to be considered. The connection is sufficient considering only one hinge.

The checks are summarized in Figure 4.

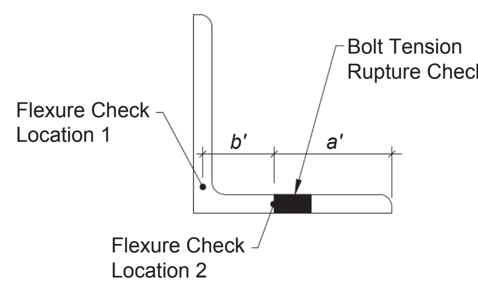


Figure 4. Limit states being checked.

The paper posted online goes into greater detail about this approach and provides a few examples. View the complete paper at **www.aisc.org/pryingcheck**. ■

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Good writing opens doors.

business issues

IMPROVE YOUR WRITING, IMPROVE YOUR BUSINESS

BY ANNE SCARLETT

YOU MAY HAVE EXPERIENCED one of the following three scenarios before.

Your colleague sends you a mediocre email correspondence about a co-selling opportunity. It contains minor grammatical and spelling errors, and most of the piece is written in the passive voice.

You open a sub-consultant's fee proposal to incorporate into your full-service package, and wince at its haphazard organization.

You review a highly anticipated written proposal from an IT firm to integrate your company's core technology systems—only to discover the proposed ideas are virtually impossible to decipher amidst the lengthy marketing babble.

When receiving less-than-adequate written materials, the reader has two choices: (1) Take the extra effort to tease out the message; or (2) Disengage and move on.

Good writing matters. Really. It helps you differentiate from another equally qualified firm. It captures your readers' attention to ensure they connect with your service offerings. It elevates your overall value.

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> The quality of your writing impacts your clients' perceptions of you and your firm's expertise.

How do you want prospects and clients to view you personally—as a consummate business development professional? An AEC thought leader? A technical dynamo? However you want to be seen, here are five actions to ensure that your writing enhances—rather than diminishes—your image.

1. Climb into the brain of your audience. Regardless of the type of business writing, it's essential to empathize with your readers. Morph into them—become them—by applying specific filters. This will help you punch holes in your content and message or to rework your delivery in terms of tone, language, sequence and length.

Action: Formally identify (write down!) logical audience filters. Examples: *Who am I? What must I do to achieve success in my role? What goal am I trying to accomplish? What challenges do I currently face? On what medium am I reading this piece—a computer, hard copy or mobile device? Will this piece inspire me, teach me or persuade me? Is it a worthwhile read? What else could I be*

Anne Scarlett is president of Scarlett Consulting, a Chicago-based company specializing in AEC-specific strategic marketing plans, marketing audits and coaching. She is also on the adjunct faculty of Columbia College of Chicago and DePaul University. She can be contacted via her website, **www.annescarlett.com**.

doing with my time right now? Then review your work with an adjusted perspective.

2. Cut, then cut some more. "I didn't have time to write a short letter, so I wrote a long one instead." At some point, we've all heard that famous Mark Twain quote. Composing a concise piece with a clear message requires meticulous proofreading, precision and a great deal of thought. Conversely, if we are not willing to give proper attention to our written work, then the result is an unstructured, sloppy piece. Our message becomes submerged in word muck.

Action: Challenge yourself to whittle each piece down by one-third. Set it aside. When reviewing it again, is your message fully intact? Can it breathe? Could you reduce it even further? Think about how you feel when you dramatically purge email accounts, computer folders or a file drawer. Condensing may be difficult, but it's also liberating.

Make time to read high-quality writing, which can improve your vocabulary as well as your writing mechanics. You'll be surprised by how much!

3. Hone your writing mechanics. I'm talking about the stuff we learn in grade school: grammar, spelling and punctuation. Do not rely solely upon your word processing software (e.g., Microsoft Word) to identify technical issues. We all know many errors slip right through the system. My personal go-to sources are Purdue University's Online Writing Lab (**https://owl.english.purdue.edu**) and **www.grammarbook.com**. Or dust off that old standby *Elements of Style* (Strunk and White) from your college days.

Action: Locate these resources for easy access. You should also make time to read high-quality writing. (Maybe it's time to allow yourself the luxury of pleasure reading!) This can improve your vocabulary as well as your writing mechanics. You'll be surprised by how much!

4. Make sure ghost writers write in your voice. In a past role as business development director, I authored some sensitive internal correspondence on behalf of our firm's president. He was pleased with my work. The problem? Weeks later, some internal folks asked me if I had written the pieces. Unfortunately, they recognized that the writing style was not fully aligned with our president's style. This caused him to lose some credibility during an already tumultuous time. (Note: I should have known better! Live and learn.)

Action: Don't slack off on this matter. Carefully review any work prepared by a ghost writer and ask yourself: Is this close to how I would personally do it? Would I be able to elaborate further in the same voice if I were questioned?

5. Recognize that all writing is not created equal. Your firm's social media presence serves to tantalize readers with quick, simple teasers. Your fee proposals aim to clearly explain your customized approach, along with your firm's methodologies, processes and deliverables. Your white papers and published articles educate readers on completely new—and often complex—content.

Each content type requires a unique approach and style. Successful writing in one form does not immediately translate into successful writing in another form. So even if you have earmarked terrific writers within your firm, make sure to individually assess each person's specific written communication abilities. Here's a parallel in the AEC industry: A brilliant core-and-shell architect may not be able to create an interior environment with the same ease, proficiency and finesse as a trained interior designer.

Action: Match the right kind of writer with the given task. Further, when preparing a high-profile report, white paper, opinion piece, etc., don't hesitate to hire a writing consultant to help you shape and express your message in a manner that best suits your target audience. It's a prudent investment.

If your readers cannot easily grasp your message within your written communications, then your amazing ideas and insights will not get the attention they truly deserve. Further, you will not put forth your very best professional self. And that—on every level, ranging from initial business development through project delivery—would be a real shame. ■


REBUILDING AMERICA
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STEEL

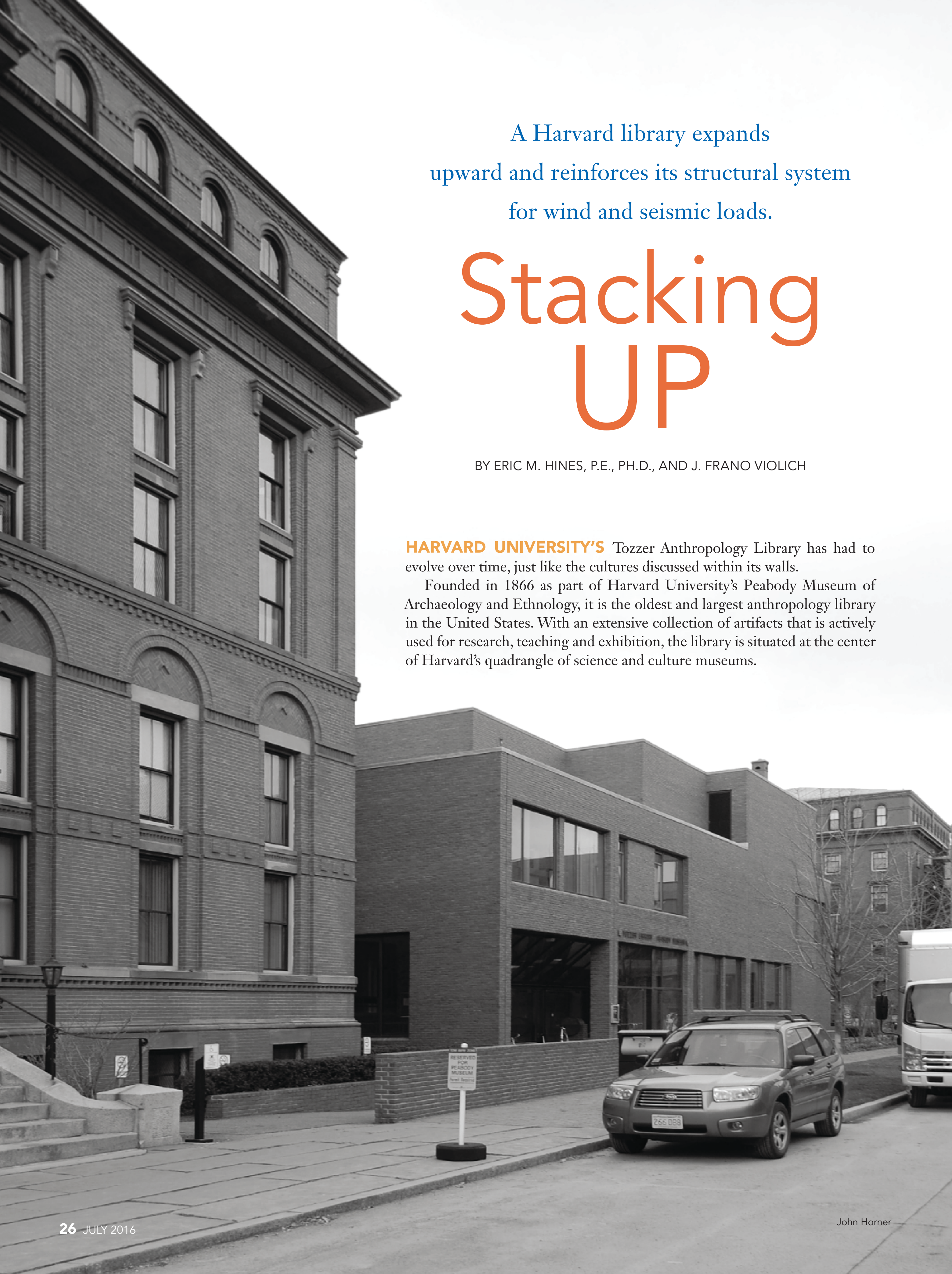
A Harvard library expands upward and reinforces its structural system for wind and seismic loads.

Stacking UP

BY ERIC M. HINES, P.E., PH.D., AND J. FRANO VIOLICH

HARVARD UNIVERSITY'S Tozzer Anthropology Library has had to evolve over time, just like the cultures discussed within its walls.

Founded in 1866 as part of Harvard University's Peabody Museum of Archaeology and Ethnology, it is the oldest and largest anthropology library in the United States. With an extensive collection of artifacts that is actively used for research, teaching and exhibition, the library is situated at the center of Harvard's quadrangle of science and culture museums.



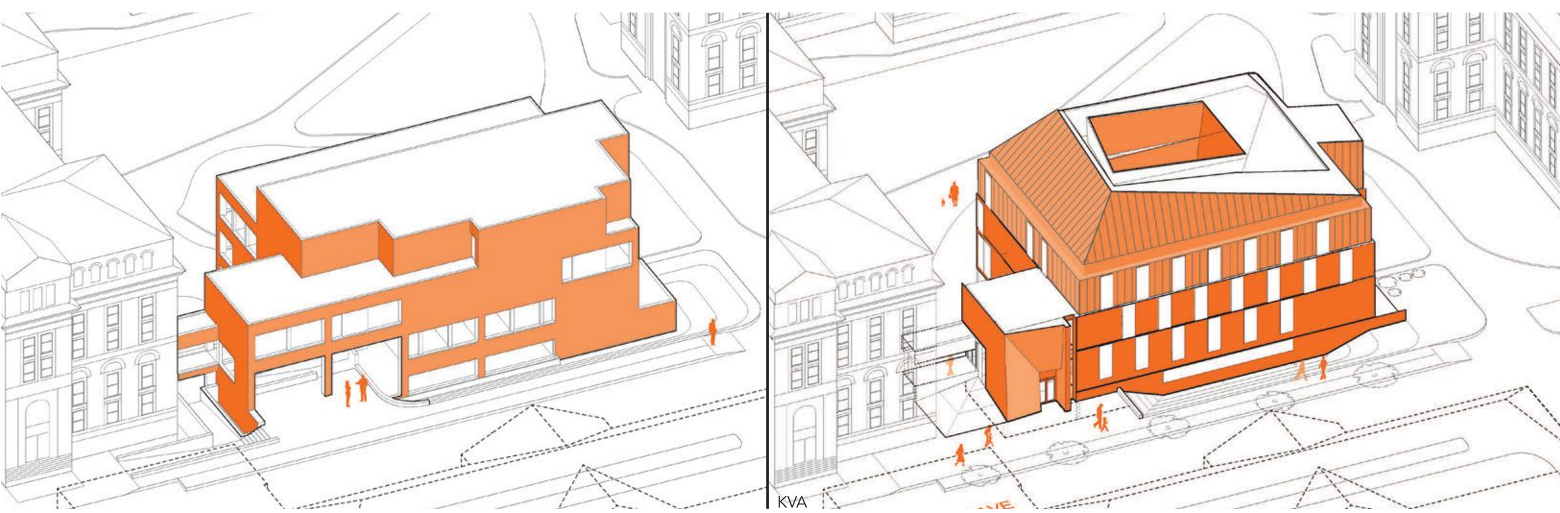
John Horner

The university decided to expand Tozzer Library into a comprehensive departmental facility by adding faculty offices, graduate student spaces, seminar rooms and public spaces to the existing resources. The university's vision for an updated facility challenged the design team to create more space for departmental research activities while maintaining the character of the existing building. The new design needed to reinvigorate an austere contemporary structure, where the library has existed since 1973, with a warm materiality and an openness emblematic of the department's mission.

In order to accommodate library stack loading, the 1973 building was designed for a live load of 150 psf. The renovation and expansion project planned to move most of the stacks to the at-grade lower level. Planned live loads of 50 psf for new office space made it possible to add two additional floors to the existing two-story structure without overloading the building columns and foundations. The new third floor replaced a previous mechanical penthouse, and the new fourth floor was topped by a hipped roof cut at an angle to create a light well throughout the top three stories of the building. The new structure is 72 ft tall at the highest point of the angled roof and extends 1.5 times higher than the original penthouse roof. The new roof required the existing building's steel moment frame structure to be braced in both directions to satisfy wind load serviceability requirements.

Existing System

The original 1973 structure was framed with wide-flange members on a grid of four bays at 22 ft, 8 in. each in the long direction and three bays of 26 ft, 16 ft and 26 ft in the short direction. Two existing open bays at the center of the original penthouse floor/low roof formed the basis for the light well, which was designed to extend up through the new fourth floor and roof. Existing floor construction was 4½ in. lightweight concrete on 1½ in. 20-ga. steel deck. In the long direction, W24×84 interior girders and S20×75 exterior girders framed into the weak axes of W12 columns to form a moment frame. In the short direction, W16×26 beams at 4 ft, 6 in. spanned between the girders, bearing on the girder top flanges and elevating the bottom of the floor and roof slabs 16 in. above the girders' top to allow openings within the framing system for ductwork. On the column lines, W16×31 beams framed into the column strong axes to form a moment frame lateral system in the short direction. Analysis showed that the beams plus strong-axis columns created short-direction moment frames with similar stiffness characteristics—story drift of approximately h/400—to the long direction moment frames consisting of girders plus weak-axis columns.



▲ Tozzer Library before and after renovation.

◀ The existing facility was built in 1973.

▼ The new structure is 72 ft tall at the highest point of the angled roof and extends 1.5 times higher than the original penthouse roof.

Eric Hines (**ehines@lemessurier.com**) is a principal with LeMessurier and a professor of practice at Tufts University in Boston. **J. Frano Violich** (**fviolich@kvarch.net**) is a principal with Kennedy and Violich Architecture, Ltd.

LeMessurier

John Horner

To stiffen the building against wind loads in both directions, the team developed two special bracing systems that responded to both the new architecture and the existing framing.

Planned live loads of 50 psf for new office space made it possible to add two additional floors to the existing two-story structure without overloading the building columns and foundations.

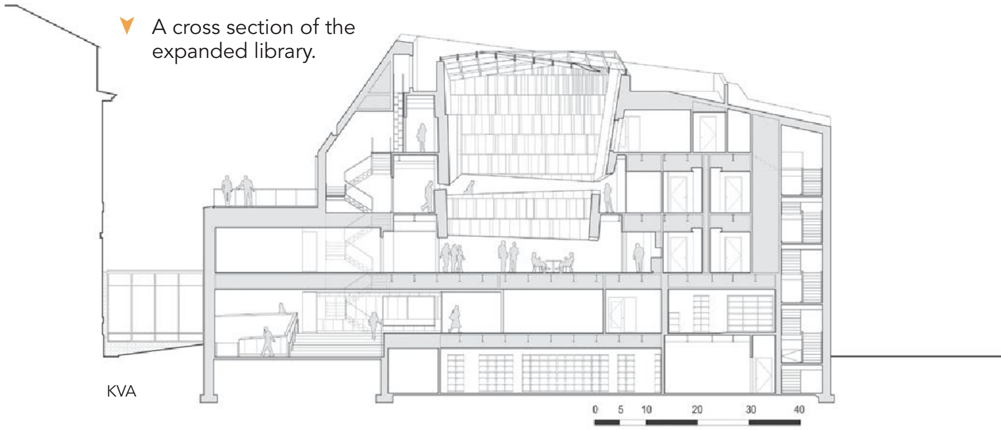
Lateral Upgrade

Considering the increase in building height from 48 ft to 72 ft, plus the increase in design wind pressures at higher elevations, the wind loads in both directions increased by a factor of approximately 67%. In the short direction, the original lateral force resisting system (LFRS) had been designed assuming an additional line of framing that was not constructed, resulting in an effective increase of 100% to the short direction wind loads. To stiffen the building against wind loads in both directions, the design team developed two special bracing systems that responded to both the new architecture and the existing framing. In the short direction, eccentric braces were used to stiffen two lines of framing while keeping the public area surrounding the light well clear. In the long direction, a single bay of concentric bracing was designed for the one bay in the building where space for new bracing was available.

While stiffening the building with bracing solved the problem of wind loads, it also caused the seismic loads to increase to a level on par with the wind loads. Recognizing Harvard's long-term interest in the building, the design team discussed with the owner the concept of developing the seismic design according to an approach that went beyond prescriptive requirements but without incurring substantial cost. Formally, the building seismic system was designed assuming an R factor of 3, which meant that wind loads would continue to govern in the short direction but seismic loads would govern in the long direction.

In going beyond prescriptive code requirements, the design team elected to consider the existing moment frame structure as a reserve system, which was analyzed independently according to an array of damage scenarios. A typical damage scenario consisted of removing the bracing from a given story, calculating the resulting building period and ensuring that the damaged structure would satisfy the code, assuming $R = 3$ for the reserve system. This approach was considered to be conservative in light of research that has shown that R factors higher than 3 are satisfactory in the presence of a reserve system that can resist collapse. This "belt and suspenders" approach to collapse resistance takes advantage of the fact that once bracing capacity is lost in a system, the building period increases and results in a reduced seismic force level. Such a system can be thought of as a primary braced frame with a moment frame reserve system. But in the case of Tozzer Library, where the original LFRS was a moment frame that required stiffening rather than strengthening, this moderately ductile dual system was conceived as a moment frame, stiffened for wind loads with lateral bracing.

In the short direction, existing beam-column connections of the new eccentrically braced frames were strengthened to carry the beam plastic moment capacity and resulting shear. By staggering the braced bays on each framing line, it was possible to distribute overturning forces to an extent that no column reinforcement was required. The existing moment frames were able to deliver horizontal pass-through forces between staggered bays without additional reinforcement. In the long direction, it was preferable to develop the entire bracing scheme in a single bay because the floor slab, acting as the building diaphragm, was elevated above the girders by the height of the W16 beams. Within the braced bay, special inter-story shear transfer details were developed to create a continuous load path between stories.



A cross section of the expanded library.

KVA

Hipped Roof and Light Well

Central to the new library's architecture is the angled, hipped roof, which was designed to fold in on itself and create a dynamic relationship between the building's exterior and interior. This new roof relates more intimately than its predecessor to the surrounding courtyard of 19th century museum structures, and it also gives the library a distinctive form whose feeling changes depending on an observer's point of view. On the exterior of the building, the roofing extends to the bottom of the third floor, balancing the massing of the first two brick masonry stories and moderating the new building's scale. From its function as cladding on the third floor, the copper standing seam roof transitions seamlessly into the four side slopes that governed the addition's geometry. Slight twists of the light well create a dynamic interplay of the roof ridges and their descent into the building. An algorithmic model was developed for the "twisted hip-roof" condition to allow continuous standing seam panel ribs to cross the roof's four hipped conditions without interruption, and interior views into the light well are framed with sensitivity to this feeling of movement. Existing columns near the light well were transferred back into the office spaces, and new bracing was coordinated so as not to interfere with the ambulatory space around the light well at each level, giving the roof and light well a sense of fluidity and wholeness.

The brick masonry facade is detailed down to the individual brick.

The design team elected to consider the existing moment frame structure as a reserve system, which was analyzed independently according to an array of damage scenarios.

Two existing open bays at the center of the original penthouse floor/low roof form the basis for the light well, which extends up through the new fourth floor and roof.

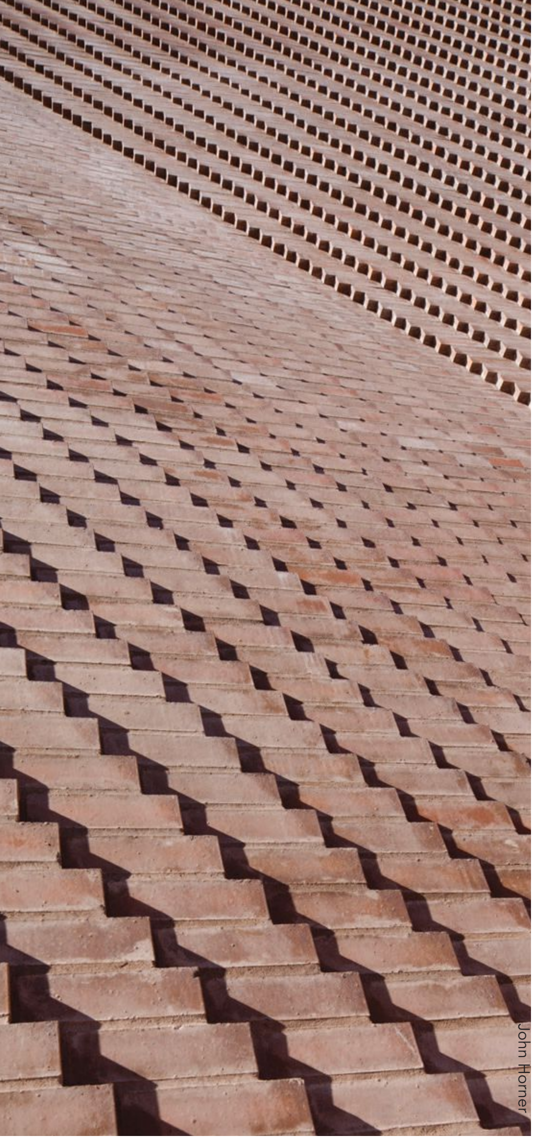
Windows

The architectural experience of the building as a whole required integrating interior and exterior while expanding the building's square footage and maintaining a sense of human scale within the constraints of the intimate site. In order to respect and preserve this concept, the design team worked to locate horizontal and vertical joints in the brick masonry facade with discretion. Windows were designed to span from floor to floor, staggered in elevation and designed to project a few inches proud of the brick in order to minimize the visual presence of joints. These architectural moves required subtle detailing of relieving angles that varied on a floor-by-floor and wall-by-wall basis due to the stacked framing configuration.

The brick masonry facade is detailed down to the individual brick dimension, and special attention was given to the development of a detailed field survey during construction prior to the development of structural steel shop drawings. The results of this survey, which was designed to detect framing differences to within ⅛ in., revealed that certain slab edges were off by several inches. Having planned to respond to the survey results prior to steel fabrication, the design team was able to redesign the facade to incorporate the exact as-built dimensions and maintain a deliberate placement of every brick. The vertical realignment of elements within the new building envelope also met two critical aspects of high-performance building enclosures—air/water tightness and thermal performance—while meeting NFPA 285 requirements, which limit the extent of combustible materials on non-bearing wall assemblies.

Integrated Design Process

Critical to the transformation of Tozzer Library and the consolidation of Harvard's Department of Anthropology was the ability to iterate through multiple design ideas at the correct moment in the design process. The roof and light well were established early in schematic design, with transfers and lateral system reinforcement budgeted at a sufficient level of detail to anticipate eventual construction costs. Establishing these parameters early on allowed for intense focus on facade support and the integration of windows and masonry patterns during the development of construction documents. Structural details for the facade and window support were developed over multiple iterations that were as extensive as those for the entire lateral system and its complex details. A willingness to integrate structural and architectural thinking about the facade was critical not only for the building facade as a whole, but also for



Looking up at the brickwork of the building's front entrance.

the front entrance specifically, which required masonry facade elements to function in a structural capacity. In certain cases, the conditions of the building demanded this integration to be so precise that full-scale mock-ups were required to make sure that unique elements could be assembled with the requisite precision. The result is a building that conveys the importance of firsthand experience with physical cultural objects, not only in scholarly research but also for the greater public. ■

Owner
Harvard University, Cambridge, Mass.

General Contractor
Consigli Construction Co., Inc., Milford, Mass.

Architect
Kennedy and Violich Architecture, Ltd., Roxbury, Mass.

Structural Engineer
LeMessurier, Boston

Creating HOME

BY ERIC HENDRICKSON, P.E., AND MATT METTEMEYER, P.E.

Mike Fager-Fisheye

The Oaknoll Spring Street expansion provides a growing Iowa City senior community with an alternative to traditional nursing homes and assisted-living facilities.

Eric Hendrickson (ehendrickson@shive-hattery.com) and **Matt Mettemeyer** (mmettemeyer@shive-hattery.com) are both structural engineers with Shive-Hattery in Iowa City.

AS ONE OF IOWA'S largest life-care communities, Oaknoll Retirement Residence has served retired citizens in Iowa City, Iowa, since 1966. Providing independent and assisted living, as well as health-care options, the Oaknoll community has experienced tremendous growth in recent years.

As a testament to its evolution, Oaknoll has completed two large expansion projects in the last decade, including the recent Oaknoll Spring Street expansion. Providing independent housing for 100 active seniors, the $46 million, six-floor, 69-unit addition is Oaknoll's largest project to date, adding 236,000 sq. ft of usable space. A glass skywalk provides residents with safe and easy year-round access the amenities in the existing main building—including a movie theater, game rooms, a library, a coffee shop and an internet café—while also enjoying a sports pub, restaurant, art studio and exhibition area in the new facility.

Mike Fager-Fisheye

◄ ▲ ► Oaknoll Retirement Residence in Iowa City recently completed a $46 million, six-floor, 69-unit, 236,000-sq.-ft addition that includes a sports pub, restaurant, art studio and exhibition area. A glass skywalk (above) provides residents with safe and easy year-round access the amenities (such as the rooftop deck at right) in the existing main building and the new one.

Mike Fager-Fisheye

The new building uses 515 tons of structural steel, which helped solve several jobsite challenges including the accommodation of 100-ft (and longer) spans, winter construction timelines and a skywalk with floor-to-ceiling walls of glass. It also provided more control over the building's design by using hollow structural sections (HSS) for support beams to reduce beam depths as compared to traditional W-shapes.

Connectivity

Oaknoll residents expressed a strong preference for feeling connected to the outdoors while inside. This was accomplished through extensive use of large windows and public areas, including a $1 million, glass-enclosed pedestrian skywalk between the existing Oaknoll facility and the new addition. Situated in a residential neighborhood with houses on all sides, the goal was to complete the skywalk with minimal disruption to pedestrian and vehicular traffic—especially before and after school. Steel was delivered to the site in manageable packages and was picked directly from the trucks via a tower crane, minimizing street closures.

The steel framing for the 112-ft-long skywalk facilitated a 7-ft horizontal bend and a 2-ft elevation change between the existing facility and the new addition. The 60-ton skywalk is hung from an upper girder, which supports the 75-ton floor of the structure plus the 100-lb-per-sq.-ft live load on the walkway. Steel also provided flexibility in connection types by using 1-in.-diameter stainless steel hanger rods hanging the floor structure from the roof primary steel girders. The designers wanted clean lines and an all-glass enclosure for the skywalk, and concealing connections was a key part of accommodating this vision. Overall stability of the skywalk walk was achieved by using beam-to-beam moment connections (see Figure 2).

Oaknoll Spring Street features large, open communal gathering spaces for residents and guests, made possible by steel framing that includes long-span joists. Perimeter columns are concealed in the exterior walls, further opening up the interior. Structural steel beams and long-span joists lend themselves to creating 62-ft by 62-ft column-free meeting rooms on the fifth level that minimized interior columns. In addition, perimeter columns were concealed within the exterior walls, which also helped open the interior spaces.

The tight construction timeline required that construction continue regardless of the weather. But building through all four seasons in eastern Iowa means dealing with large temperature swings (potentially -20 °F to over 100 °F) on building materials that were not yet enclosed. Connections were detailed to allow the steel beams to expand and contract with the seasons while still providing integrity to the not-yet-complete structure. (One example of the thermally unrestrained connection is the steel beam bearing on a masonry wall detail in Figure 1.)

Placing Plank

Although structural steel was the star of the project, the Oaknoll Spring Street building used precast concrete plank for the upper four floors above the two levels of cast-in-place parking area. The

The steel framing for the 112-ft-long skywalk facilitated a 7-ft horizontal bend and a 2-ft elevation change between the existing facility and the new addition.

Large, open communal gathering spaces are made possible by steel framing that includes long-span joists.

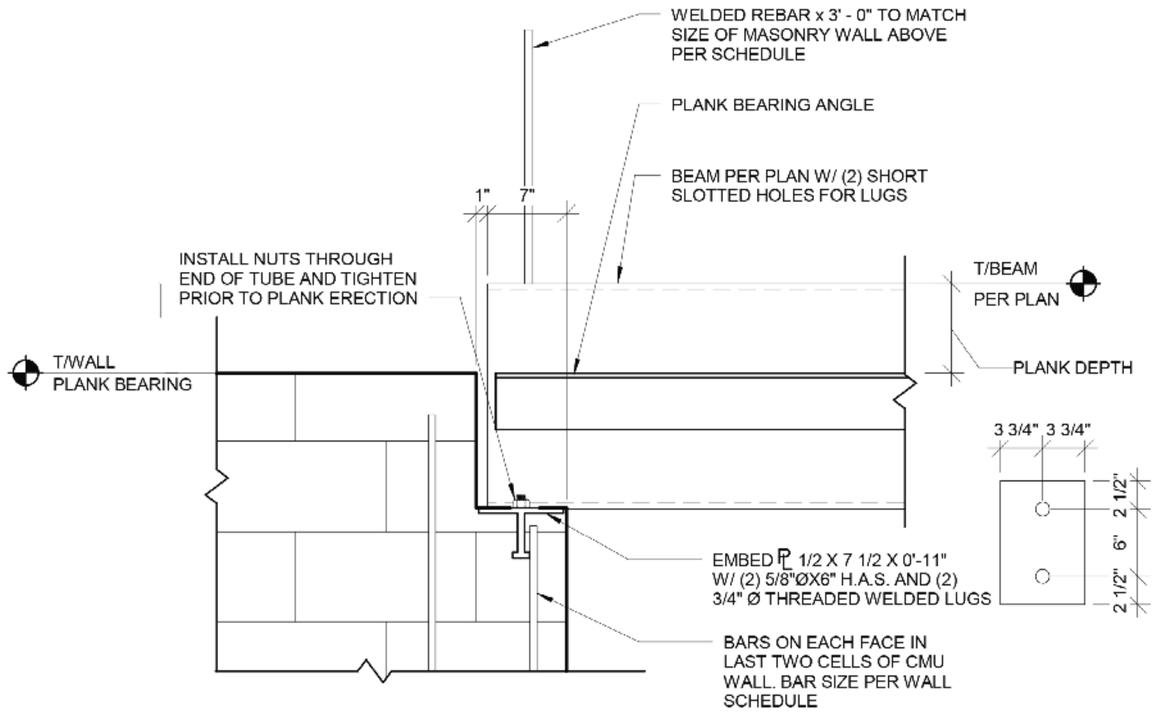
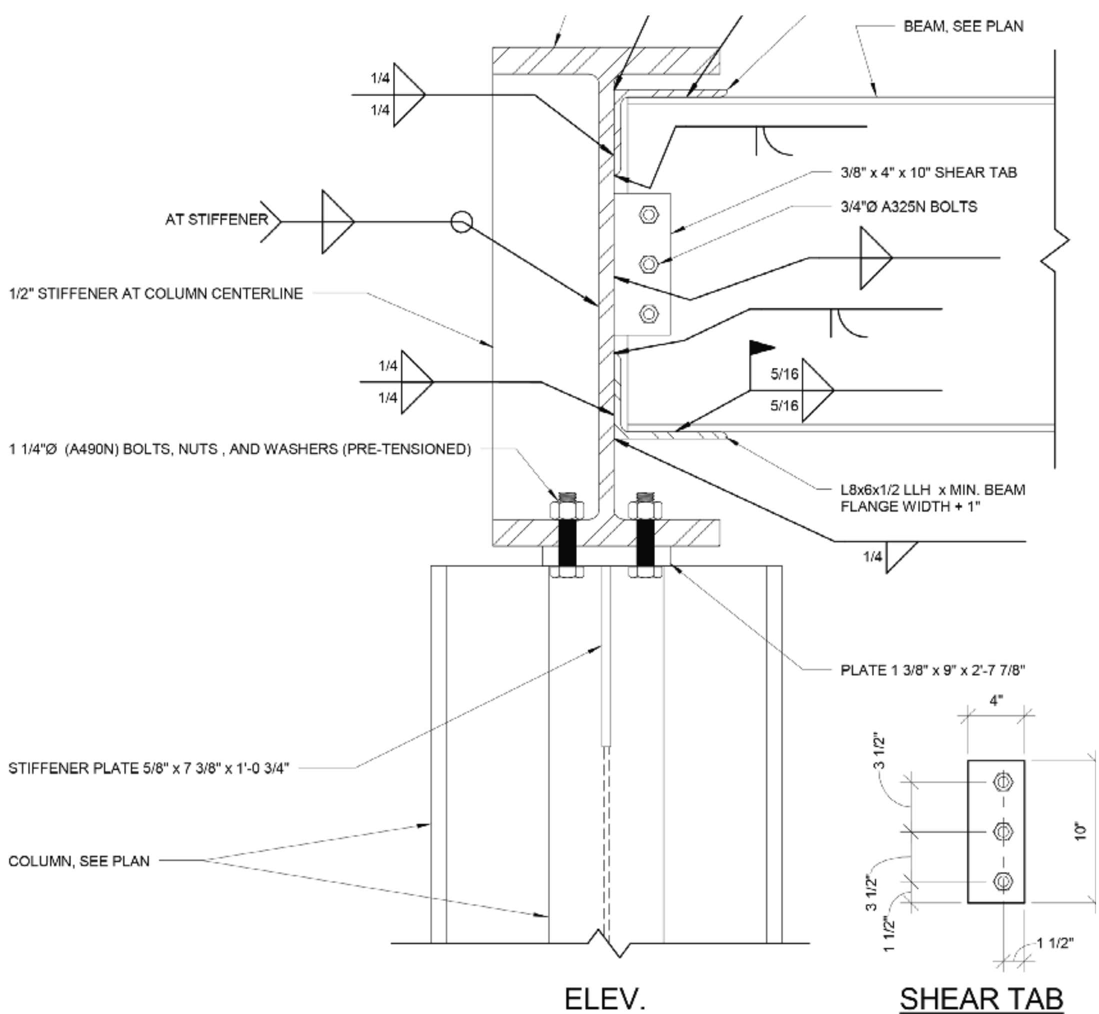


Figure 1. A detail of a beam bearing on a CMU.

Figure 2. A beam-to-beam moment connection detail.



team learned valuable lessons about erection stability when combining the two materials.

For example, beams and girders are designed to support precast plank installed on both sides—a balanced condition. But if erection isn't complete—e.g., if erection must be stopped at a certain point before it's finished—the beam or girder needs to be braced for potential rotational issues when the plank load is only on one side.

However, the erection sequence drawings required that some of the girders be loaded on one side at various times—even though they're designed to be braced with precast on both sides. To accommodate this in the erection sequence, temporary solutions vertical support columns were used for interim stability and stiffness until all structural elements were assembled. The team worked closely with the contractor to minimize rotational issues associated with one-sided girder loading.

In-Depth Solution

The 13-ft floor-to-floor height and the 9-ft floor-to-ceiling height limitations presented a challenge in finding a structural system shallow enough to create two levels of parking and four levels of occupied space, all while keeping the building under 70 ft tall—including any elevator overruns—as the overall building height

Shive-Hattery

The 13-ft floor-to-floor height and the 9-ft floor-to-ceiling height limitations required a structural system that would be shallow enough to create two levels of parking and four levels of occupied space, all while keeping the building under 70 ft tall. The 60-ton skywalk (right) is hung from an upper girder, which supports the 75-ton floor of the structure plus the 100-lb-per-sq.-ft live load on the walkway.

was limited due to City of Iowa City zoning and the U.S. Federal Aviation Administration (FAA) regulations on building height.

The system needed to allow the beam to be placed *within* the depth of the floor structure rather than *below* it. Using precast planks, the plank would typically be placed on top of the W-section, but that couldn't be done in this situation. The team also wanted to steer clear of placing a precast plank on the bottom flange or putting an angle in the web of a W-section, which would render the system un-erectable. Instead, 100 16-in.-deep and larger HSS were used, with bearing angles on each side.

The new building uses 515 tons of structural steel.

Mike Fager/Fisheye

Shive-Hattery

As Iowa City's senior population continues to grow, the Oaknoll Spring Street addition offers a unique alternative to traditional nursing homes and assisted-living facilities, and this addition provided a structural template for potential future expansion to the facility should the need arise. ■

Owner
Oaknoll Retirement Residence, Iowa City, Iowa

General Contractor
McComas-Lacina Construction, Iowa City

Architect and Structural Engineer
Shive-Hattery, Iowa City

Arizona State's new law school in downtown Phoenix continues the legacy of the Supreme Court justice who cut her legal teeth there.

LEGAL Lessons

BY JEFF THOMPSON, P.E., AND ERIKA YARONI

Jeff Thompson is an associate structural engineer and oversaw the project from design through construction, and **Erika Yaroni** is a structural engineer. Both are with BuroHappold Engineering's New York office.

THE FIRST WOMAN nominated to the Supreme Court of the United States now has her name on a law school in the city and state that launched her to national prominence.

The new Sandra Day O'Connor College of Law will expand Arizona State University's downtown Phoenix campus with a modern facility that connects students and faculty to the existing Phoenix legal community. The location couldn't be more fitting, as O'Connor served as Assistant Attorney General of Arizona, The new Sandra Day O'Connor College of Law will expand Arizona State University's downtown Phoenix campus with a modern facility that connects students and faculty to the Phoenix legal community.

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The facility's signature V-column, which supports the five stories above it.

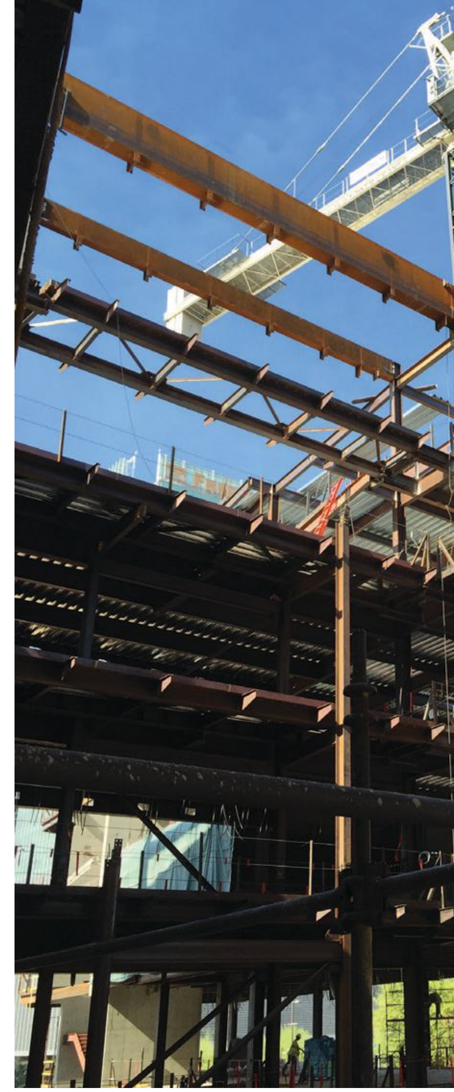
the Arizona State Senate, on the Maricopa County Superior Court and also on the Arizona State Court of Appeals.

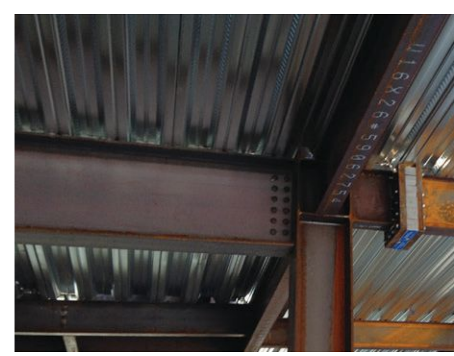
The 280,000-sq.-ft steel-framed building will include classrooms, offices, research clinics, a law library, a public cafe, a bookstore, the ASU Alumni Law Group and below-grade parking; these program spaces are configured in six stories above grade and two below. Prominent architectural features include stacked double height spaces located in the heart of the building, a 250-seat auditorium with a retractable seating system and facade system on level 1, the law library on level 3 and a shaded exterior roof courtyard on level 5. An indoor/outdoor continuous circulation track at each level connects the east and west portions of the building with pedestrian bridges, and a canyon-inspired outdoor pedestrian promenade traverses through the site at level 1, providing sight lines into interior spaces and a public face to the law school.

Building Blocks

The design was completed within one year to meet an aggressive fast-track schedule, with construction beginning in the summer of 2014 and being substantially completed last month. A steel structure was the ideal solution to meet programmatic requirements for long-span, column-free interior spaces that maximize the clear floor-to-floor height. The steel superstructure uses composite slab construction with typical bay sizes ranging from 24 ft by 35 ft to 24 ft by 50 ft. A 3-in.-deep metal

The 280,000-sq.-ft steel-framed building will include classrooms, offices, research clinics, a law library, a public cafe, a bookstore, the ASU Alumni Law Group and below-grade parking; these program spaces are configured in six stories above grade and two below grade.





deck with 4½-in.-deep normal-weight topping slab is supported by composite wide-flange beams and girders spanning to steel columns. Lateral stability is provided by six 12-in.-thick concrete cores located at egress corridors and elevator shafts. Embed plates are used to connect steel floor framing into the cores and drag diaphragm loads into shear walls.

Approximately 1,750 tons of structural steel in the form of approximately 3,000 members is used above grade. The structural steel design features suspended outdoor pedestrian bridges linking the building across the "canyon," a sloped exterior V-column supporting five stories above and a roof diaphragm truss to support a fabric canopy shading system over the courtyard.

Bridging Education

Five stories of suspended, open-air pedestrian bridges span approximately 60 ft to 75 ft connecting the east and west portions of the building. The bridges are part of the main horizontal circulation corridor and stitch together important interior programmatic spaces. Each bridge will be clad in an architectural mesh serving the dual purpose of providing fall protection and allowing the opportunity for programmable content to be displayed via LEDs embedded in the mesh. Structurally, the bridges presented multiple technical challenges, including providing adequate redundancy for five stories of stacked bridges supported from the roof level, linking buildings together with different stiffness properties, minimizing thermal bridging between interior and exterior steel and meeting strict vibration criteria to ensure pedestrian comfort.

The bridge deck is formed by a composite metal deck supported by steel stringers and beams with diagonal angle bracing to provide lateral stability. Moment-connected W12 secondary beams with tapered outriggers span between con-

Erecting the pedestrian bridge.

The angle of each leg of the V-column was determined by equalizing the horizontal thrust forces resulting from dead loads transferred from the columns above.

Thermal break isolation (at center of photo) between steel elements.

tinuous W14 bridge stringers. Outriggers are field welded to continuous 3-in.-diameter high-strength rods spaced at approximately 7 ft on center and hung from built-up steel box beams at the roof level. To provide redundancy, the stringers were engineered using heavy W14 column sections to support bridge gravity and live loads without the use of hangers. Fabreeka bearings at each end allow stringers to translate longitudinally and rotate to accommodate thermal expansion/contraction and differential building movements. The bridge was designed to achieve a vertical natural frequency of 5 Hz through the combined stiffness of roof supports, bridge stringers and high-strength hanger rods. The bridge deck was shop fabricated and fully assembled by Able Steel Fabricators before being shipped to the site, where it was erected and supported by a temporary tower until hanger rods were field welded into position.

Balancing Act

An exterior double-height V-column supporting five stories of the west building forms a unique architectural feature at the northwest corner of the site. The column base connects to the plaza at a single point, minimizing the amount of structure touching the ground and opening the building to the canyon space beyond. A tiered outdoor seating area wraps the column base providing a shaded public space, which will be furnished with tables and benches. The V-column included unique technical challenges such as optimizing the geometry based on the applied loading and minimizing thermal bridging between interior and exterior steel in the level 3 soffit.

Each leg of the column is formed by sloped W14×426 members welded to the underside of a W30 beam acting as a tension strut. The top of the column aligns with vertical columns above level 3, and the base is supported by a concrete column within the plaza podium structure. The angle of each leg of the V-column was determined by equalizing the horizontal thrust forces resulting from dead loads transferred from the columns above. Live loads are assumed to be unbalanced with the worst-case design condition of one column unloaded and one column fully loaded. Unbalanced horizontal thrust forces are resolved at level 3 by a steel diaphragm truss formed from wide-flange floor beams and HSS diagonals, which connect the V-column to a concrete core to provide stability. Shock Isokorb thermal breaks are located at all steel beams and braces spanning from interior to exterior space as well as the structural slab at the level 3 soffit, and insulation and a topping slab above the level 3 structural slab form the climate barrier between the interior and exterior space of the soffit.

Floating Shades

A PTFE fabric shading canopy spans the north-south direction approximately 80 ft across the level 5 exterior courtyard of the east building. The canopy provides shade for added thermal comfort in addition to creating a unique 3D sculpture. Canopy catenary cables are attached to triangulated steel trusses located above the roof level on both sides of the courtyard. Large horizontal truss reactions are taken into the base building through two 14-ft-deep floor diaphragm trusses that span 90 ft to 170 ft between concrete shear walls under the roof structural slab. Canopy truss reactions were coordinated to coincide with roof diaphragm truss panel points, and truss chords were formed using a combination of wide-flange roof beams and W14 sections, with HSS forming the truss diagonals. To provide added redundancy, the composite roof slab was also designed to resist the canopy reaction loads.

The project is currently scheduled to be completed for the fall 2016 semester at ASU. The final design will provide a new landmark for ASU's downtown campus, further enhancing the existing urban fabric for both students and public alike, and paying homage to the legal legacy of its namesake.

Owner
Arizona State University, Tempe

General Contractor
DPR Construction, Phoenix

Architects
Ennead Architects, New York (core and shell)
Jones Studio Architects, Tempe (interior)

Structural Engineer
BuroHappold Engineering, New York

Steel Team

Fabricator
Able Steel Fabricators, Inc., Mesa, Ariz.

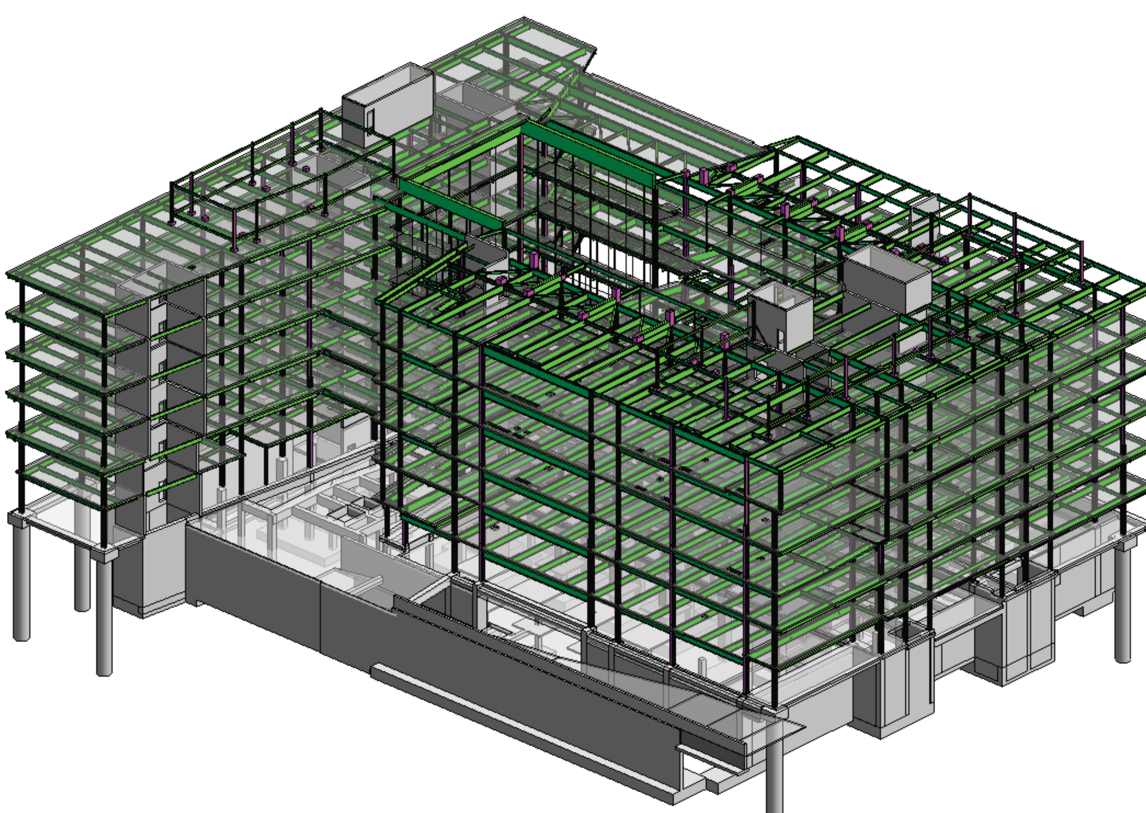
Detailer
LTC, Inc., West Salem, Wis.

▲ The "canyon."

◄ The southern pedestrian bridges.

► The V-column prior to cladding.

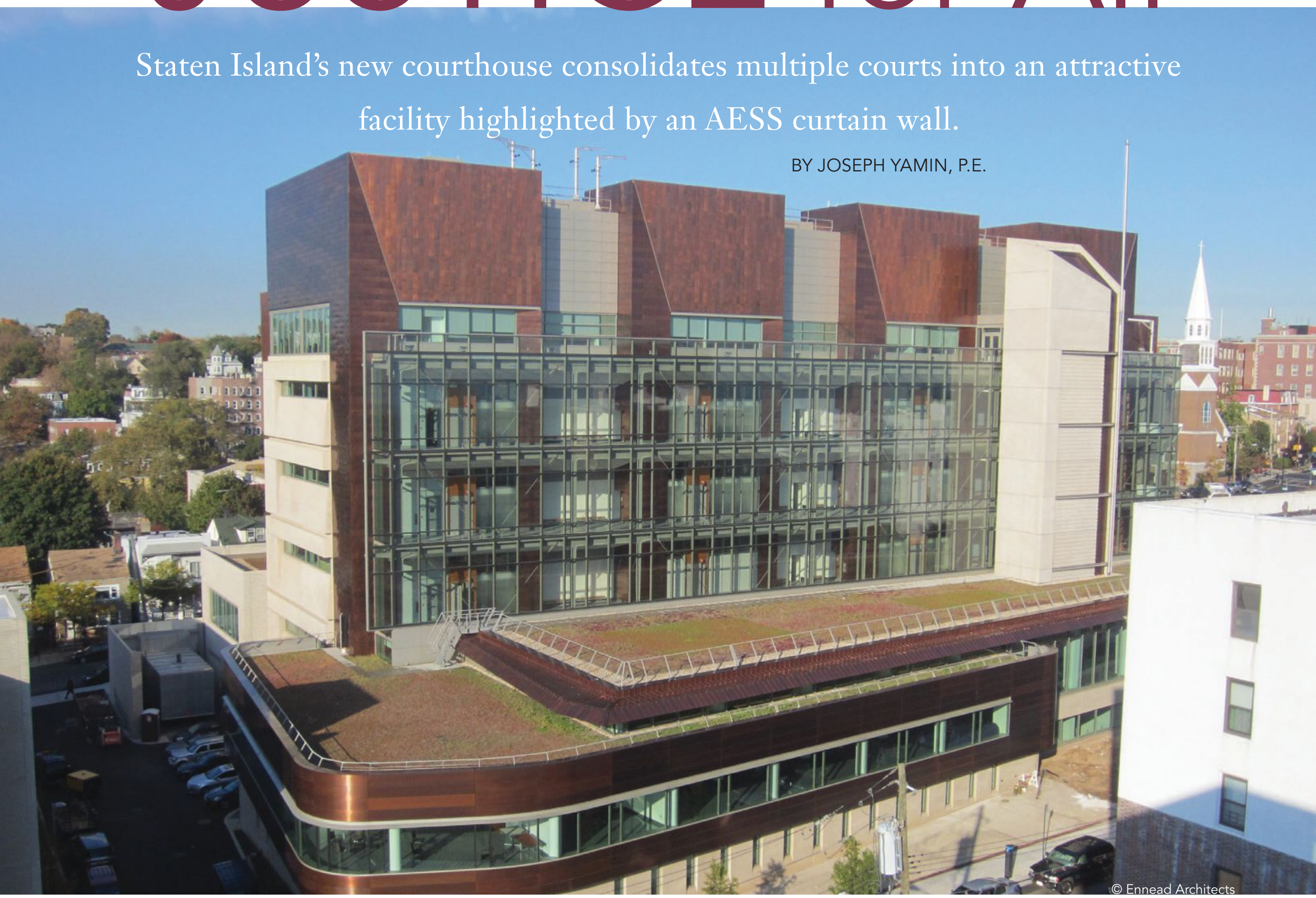
▼ A Revit 3D model of the building.



JUSTICE for All

Staten Island's new courthouse consolidates multiple courts into an attractive facility highlighted by an AESS curtain wall.

BY JOSEPH YAMIN, P.E.



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STATEN ISLAND is a new center of justice for New York.

The recently completed 183,000-sq.-ft Staten Island Courthouse is a new governmental services building located in the St. George neighborhood. This six-story steel-framed building houses the New York Supreme Court, the Richmond County Criminal Court and related agencies. With its opening this past year, it was the first new freestanding courthouse constructed on Staten Island in over 80 years and will fulfill the modern programmatic requirements of the courts. The facility, which is managed by the Dormitory Authority of the State of New York (DASNY), includes a new courthouse, memorial green outdoor public space and a 660-space parking garage, and was built at an approximate cost of $230 million.

Located a short walk from the Staten Island Ferry Terminal, the courthouse occupies a highly visible hilltop providing impressive views for court staff and visitors. The program space is organized in such a way that the structure forms four "towers of justice" containing the courtrooms, which can be seen when approaching the island from the ferry. Structural engineer LERA worked closely with Ennead Architects to devise structural framing schemes that respected these courtrooms as well as the circulation corridors between them. This included locating the majority of the braced frames outside of the building core and placing them between the courtrooms and corridors. Also, the gravity columns were located at a 38-ft, 10-in. spacing to create a column-free courtroom space.

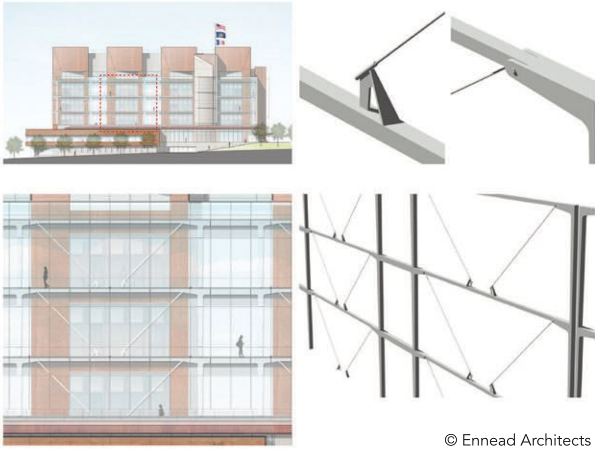
The north, south and west façades of the building are clad with architectural precast concrete and glass-fiber reinforced concrete (GFRC) panels. In addition, copper sheet cladding was incorporated into the façade and public waiting and circulation areas. The east wall of the courthouse contains an architecturally exposed structural steel (AESS) frame—the majority of which is exposed to view within 20 ft—that supports a three-story glass wall.

A ship's bulkhead served as inspiration for the detailing of the exposed frame, invoking a likeness to the familiar Staten Island ferry. This structure consists of tapered built-up columns with radial transitions to parallel outriggers. These outriggers support an HSS12×8 spandrel beam via an exposed pin detail. This spandrel beam was designed to be as shallow as possible to provide a mini-

© Ennead Architects

▲ The six-story building was built at an approximate cost of $230 million.

▼ Renderings of the east curtain wall.



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Unearthing Local History

During early site excavations for the project, several human skeletal remains were discovered unexpectedly. Subsequent archaeological investigations concluded that the site was formerly the home of the New York Maritime Hospital and Quarantine Station, which served as a quarantine for immigrants coming to the United States, from 1799 to 1858. In order to preserve this historic site and serve as a remembrance of those who were buried there, a "Memorial Green" public space was incorporated. The Memorial Green is surrounded on all sides with architectural concrete retaining walls designed to match the precast panels on the courthouse. The walls are located to respect the boundaries of the unexamined cemetery.

◄ ▼ The recently completed 183,000-sq.-ft Staten Island Courthouse houses the New York Supreme Court, the Richmond County Criminal Court and related agencies.

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Joseph Yamin (joseph.yamin@lera.com) is an associate with Leslie E. Robertson Associates (LERA) in New York.

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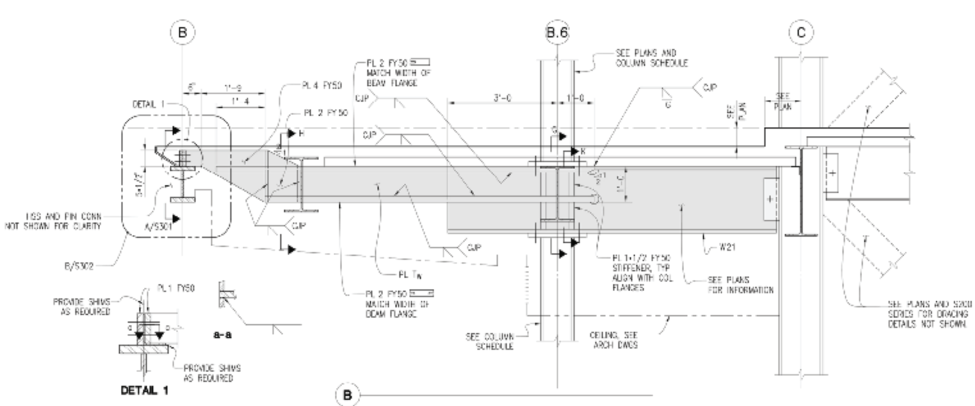
The structure consists of tapered built-up columns with radial transitions to parallel outriggers. These outriggers support an HSS12×8 spandrel beam via an exposed pin detail.

LERA

Close-up of an AESS pin.

A detail of the gooseneck cantilever assembly.



"Gooseneck" Cantilever

mal sight line through the curtain wall and it spans the building's typical bay spacing of 38 ft, 10 in. while supporting the edge of the corridor between courtrooms. For stiffness and vibration serviceability requirements, intermittent supports were introduced to reduce the overall span of the spandrel beam; these supports are in the form of 1½-in.-diameter exposed steel rods in a "trapeze" pattern (a nickname the designers gave the pattern since it reminded them of two acrobats on a trapeze in mid-swing) connected to the outriggers with custom AESS brackets.

Owing to the AESS nature of the east curtain wall frame and the fact that it supports an egress corridor, the frame was required to be analyzed for and capable of withstanding fire conditions. The result of these studies led to the design of a redundant supporting system whereby the entire frame can be supported by secondary fireproofed cantilevers from the main building columns in the event of a fire. In order to maintain the minimal sight line requirement, these cantilevers, known as "goosenecks," were designed as built-up sections cantilevering 10 ft and tapering down to a depth of just 5½ in. The goosenecks connect to the top flange of the exposed steel outrigger, which has two benefits: It hides the supporting connection in the concrete slab and provides the necessary fireproofing for that connection. In all, the east curtain wall is an iconic feature of the courthouse, serving as a symbol of transparency and connectivity to the surrounding community.

Owner
Dormitory Authority of the State of New York (DASNY)

Architect
Ennead Architects, New York

Structural Engineer
Leslie E. Robertson Associates (LERA), New York

Construction Manager
Jacobs Engineering, New York

General Contractor
Delric Construction, North Haledon, N.J.

Steel Fabricator and Detailer
Owen Steel Company, Inc., Columbia, S.C.

Nebraska Medicine looked to its past to find the right structural solution for its new parking garage.

Second Time AROUND

BY RYAN CURTIS, P.E.

All photos: Ryan Curtis

NEBRASKA MEDICINE has been at the forefront of patient care, research and education for several years.

The Omaha-based facility recently became a household name, as it is home to one of the largest biocontainment units in the Unites States and assisted with multiple Ebola patients during the West Africa outbreak in 2014. As one of the leading academic medical centers in the Midwest, with over 1,000 physicians and a rapidly expanding campus, parking capacity on-site has not been able to keep pace with increased demand.

Relief came in the form of a new eight-story, 730-spot parking garage. A steel superstructure coupled with a post-tensioned parking deck was selected, a framing scheme that is somewhat of an anomaly among parking garages in the region. So why did Nebraska Medicine buck the trend? Because they had been very pleased with the performance, cost and appearance of a similar, adjacent parking garage that was built more than three decades ago.

Ryan Curtis (rbcurtis@leoadaly.com) is an associate and structural project engineer with Leo A Daly Company.

If it Ain't Broke

The existing "Lot 5" garage—built in 1983 by the same design and contractor team of Leo A Daly and Kiewit Building Group—is six stories tall and houses 677 cars. Its structural system was somewhat unique for its era in that the design incorporated a one-way sloped ramp climbing upwards and a high-speed spiral exit ramp. The lateral force resisting system used cast-in-place reinforced concrete shear walls, and with the overall plan dimensions of 335 ft long by 112 ft wide, the steel beams spanned the transverse direction of the garage with two long-span bays of 56 ft. The deck construction is composed of steel frame bents at 21 ft center-to-center, with no framing members spanning between columns, and a 6-in. post-tensioned deck.

Lot 5 has proven itself over the decades, recognized by the owner as the most durable and lowest-maintenance garage on the medical center's campus. Given this success, coupled with the fact that the new "Lot 6" garage would be built right next door, the owner decided to use Lot 5 as the template for the Lot 6 garage.

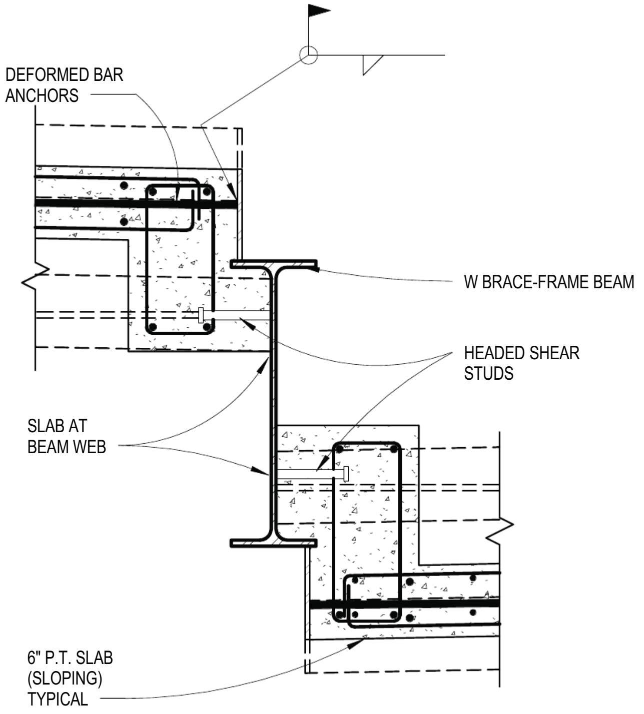
Big Sister

Coined the "big sister" of Lot 5, Lot 6 shares many features with its little sister. The garage uses wide-flange steel columns and beams as the superstructure, and a two-part coating system was used on all steel framing members. The first coating consists of a heavy zinc primer (2.5 to 3.5 mils) and the second is an epoxy coat (8 to 10 mils). Final touch-up coatings were applied in the field in the final weeks leading up to substantial completion. The deck system was also constructed of a 6-in. cast-in-place post-tensioned slab (described in AISC Design Guide 18: *Steel Framed Open Deck Parking Structures* as historically the most durable deck for Omaha's climate zone, Region B).

The garage is rectangular in plan, 325 ft long by 116 ft wide, and uses a double-helix ramp system with one-way traffic and crossover locations where sloping decks intersect. This was deemed the most efficient method of moving cars in and out of the structure during peak times for staff.

While cast-in-place concrete shear walls were used in the Lot 5 garage, the double-helix geometry of Lot 6 resulted in the need for an alternative lateral load resisting system. Due to a much tighter construction schedule—and current code-driven open-air requirements for garages—concentric brace frames with wide-flange braces were selected as the lateral force resisting system for Lot 6. Two steel braced frames are located in the transverse direction of the rectangular garage, both occurring where the parking deck ramps are "flat." However, one braced frame was located with its strong axis along the longitudinal direction situated one bay south of where the double-helix ramps intersect at a vehicle crossover location, which presented a unique challenge with regards to lateral load collection. In order to transmit diaphragm forces from the two sloping ramp slabs into the collecting members of the brace frame, a 24-in.-deep wide-flange cross beam with concrete encasements was employed. By using headed studs and attaching plates to the beam webs, the detail was compatible with the sloping ramps as they ramped up and down at varying locations. In addition, the steel braced frame provided visual openness to people walking or driving through the parking garage and eliminated blind spots at vehicle turns.

Another unique feature of Lot 6 was the post-tensioned slab design. In Lot 5, post-tensioning was used in only the longitudinal direction of the garage, with conventional reinforcing being used in the short direction. Because post-tensioning slab design methods have evolved since the construction of Lot 5, the design team decided to use post-tensioning in both directions for Lot 6, as this was deemed a better measure in sustaining slab compression forces to aid in crack control and overall slab durability. With post-tensioning in both directions and composite beams now parallel to the short-span post-tensioning, several design and detailing enhancements were employed to ensure the compatibility of both systems. In reviewing *Effects of Slab Post-tensioning*

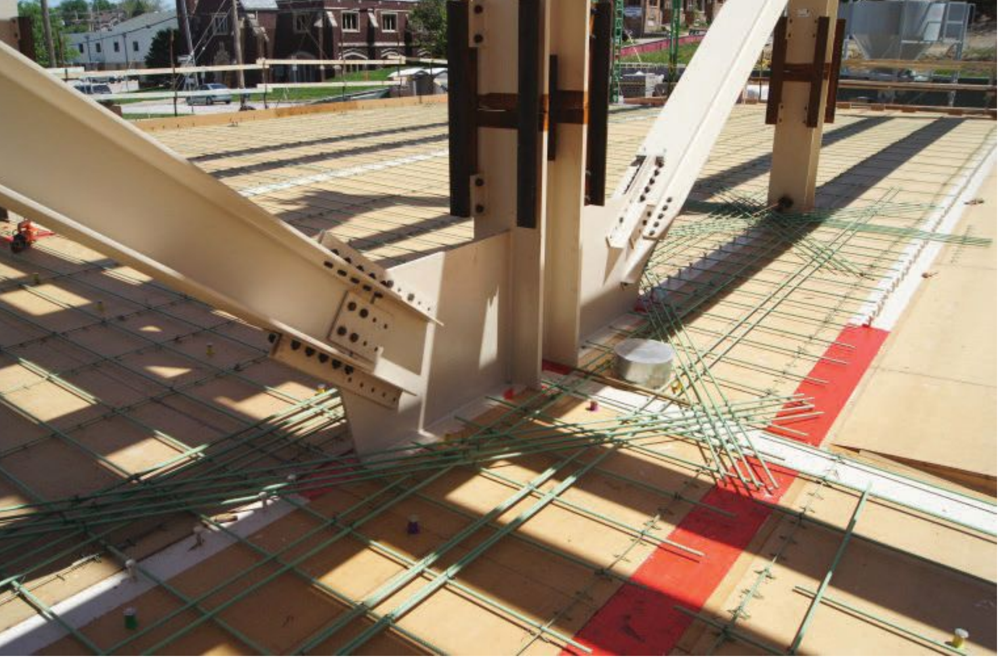


▲▼ Braced framing (above) in the new Lot 6, which is eight stories tall and can accommodate 730 cars. It was built adjacent to the existing Lot 5.

▲ A detail of the sloping slabs at the braced frame.

◀ Lots 5 (left) and 6 (right).

▼ Erecting the braced frame.



The garage is rectangular in plan, 325 ft long by 116 ft wide.

on Supporting Steel Beams (Sharma and Harries, 2007) and Design Guide 18, the team concluded that any effects of the post-tensioning on the composite action of the steel beam and slab assembly were negligible. As stated in Sharma and Harries, "The flexure-induced strains associated with post-tensioning are approximately 10% of those associated with the transfer of full dead load to the beams (as the forms are released). Additionally, bottom flange strain associate with the axial post-tensioning force and that associates with flexure-induced forces virtually cancel each other."

With two-way post-tensioning and composite beams, the detailing of the slab and beam interaction was critical to minimize the potential for cracking caused by slab restraints with a volume-changing post-tensioned deck. For example, to isolate slab shortening from the restraint provided by the structural steel framing, a bond-breaker was applied to the top flange of composite beams for the first 5 ft at each end. In addition, no shear connectors (headed studs) were allowed to be installed within the 5-ft zone. When shortening of the slabs occurs, this detail allows the slabs to "slip" without restraint that may have caused cause cracking. At brace frames where a high concentration of diaphragm shear collection occurs, additional bars were placed in both longitudinal and transverse directions of the braced frame strong axis. This distribution of reinforcing steel strengthened the diaphragm and also provided more shrinkage crack control. Also, the detailing of the post-tensioned slabs around the steel columns included isolation joint material at the column surfaces to minimize any detrimental cracking caused by restraint from the rigid steel column element.

In the steel scenario used in Lot 6, deck formwork began as soon as the slab below (supporting the forms) reached appropriate strength (usually three to five days).

On the Clock

With approximately one year to design and build the parking garage, the steel framing was essential to the speed of construction. In a cast-in-place scenario, slabs must be placed and columns must be cast

Lot 6 uses a double-helix ramp system with one-way traffic and crossover locations where sloping decks intersect.

Using field-installed steel connections to join the precast skin with the wide-flange columns facilitated adequate flexibility and tolerances on-site.

The braced framing design provides visual openness and eliminates blind spots at vehicle turns.

prior to shoring up the decks for the next level. In the steel scenario used in Lot 6, deck formwork began as soon as the slab below (supporting the forms) reached appropriate strength (usually three to five days). The short cycle for each round of shoring, pouring and re-shoring allowed the contractor to keep the project on time. With approximately 800 tons of structural steel used in the project, the steel erection took four months.

For the precast skin along the perimeter of the entire garage, the panels were designed to span laterally to column supports every 21 ft. At each steel column, precast column wraps were used vertically from the foundation to the roof. Each connection contained field-welded anchors and various slotted supports held off the columns, allowing movement of the panels to occur under various loading scenarios. Using field-installed steel connections to join the precast skin with the wide-flange columns facilitated adequate flexibility and tolerances on-site as well as allowed proper alignment of the precast panels.

Completed in December 2015, Lot 6 improved upon the success of its parking predecessor, and may just provide the template for future parking projects on the Nebraska Medicine campus. ■

Owner
Nebraska Medicine, Omaha

Architect and Structural Engineer
Leo A Daly Company, Omaha

Design-Build Contractor
Kiewit Building Group, Omaha

Consultant
Walker Parking Consultants, Chicago

Steel Team

Fabricator
Paxton and Vierling Steel Co., Carter Lake, Iowa AISC CERTIFIED FABRICATOR

Detailer
Industrial Detailing, Inc., St. Louis AISC DETAILER

Erector
Davis Erection, Omaha AISC CERTIFIED ERECTOR

Steel framing provides long, clear spans in the new garage.

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

TWO HALVES are Better than None

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

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

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

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

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

Research

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

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

Design

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

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

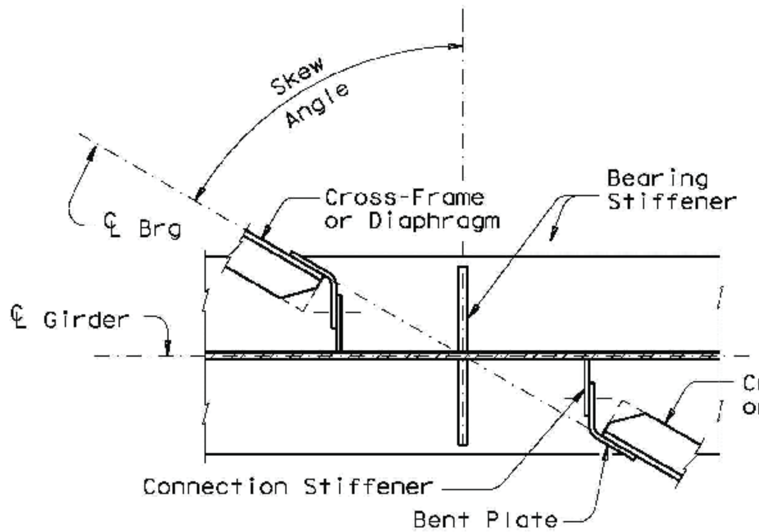
➤ A painted stiffener on a weathering steel girder with a weathering steel connection plate welded to the split pipe.

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

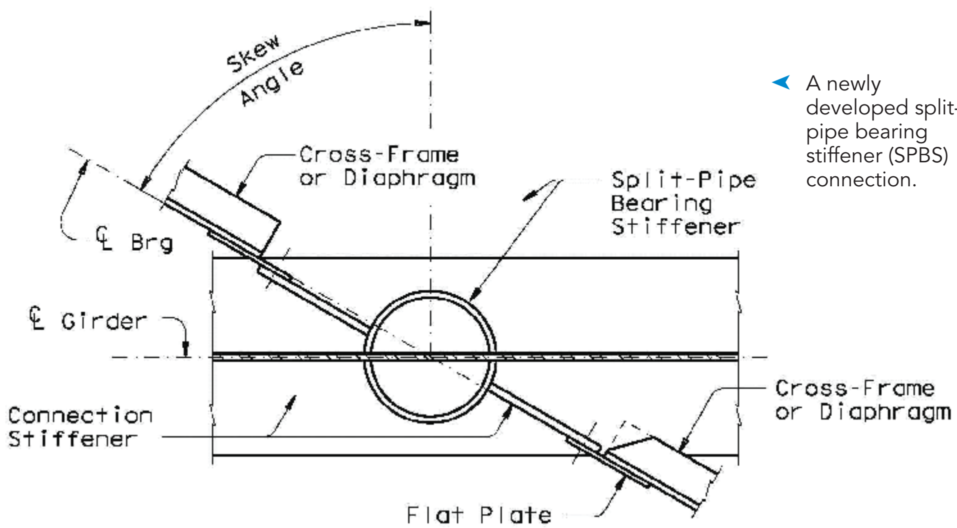
In their design recommendations, the researchers recommended the cross frame connection plate be welded to the HSS only and not welded to the girder flanges. This avoids a fillet weld partially along the length of the flanges, a weld that could not be classified as Category C' like conventional cross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

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

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



A traditional bent plate connection.



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

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

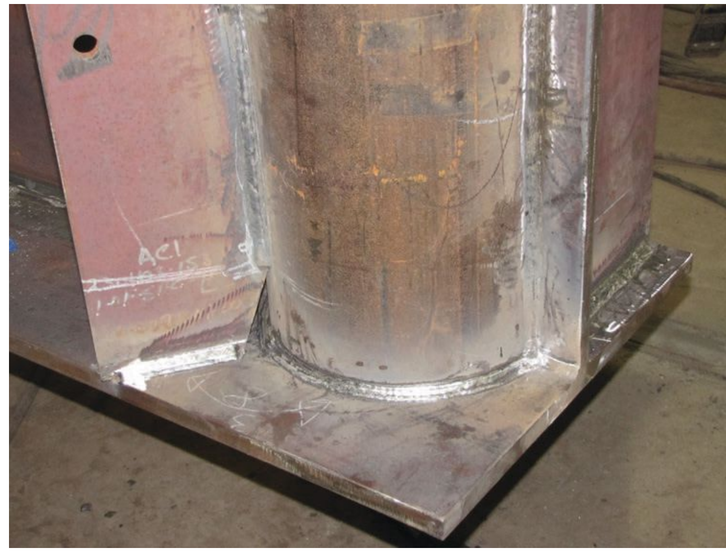
The most oft-recommended material specification for round and rectangular HSS is ASTM A500. Thick-wall extra-strong or double-extra-strong ASTM A53 pipes are generally not available in large diameters or must be bought at a premium; the use of this material is therefore discouraged by fabricators. With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

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

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

Fabrication

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



Cross frame connection details at highly skewed support bearings.

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

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

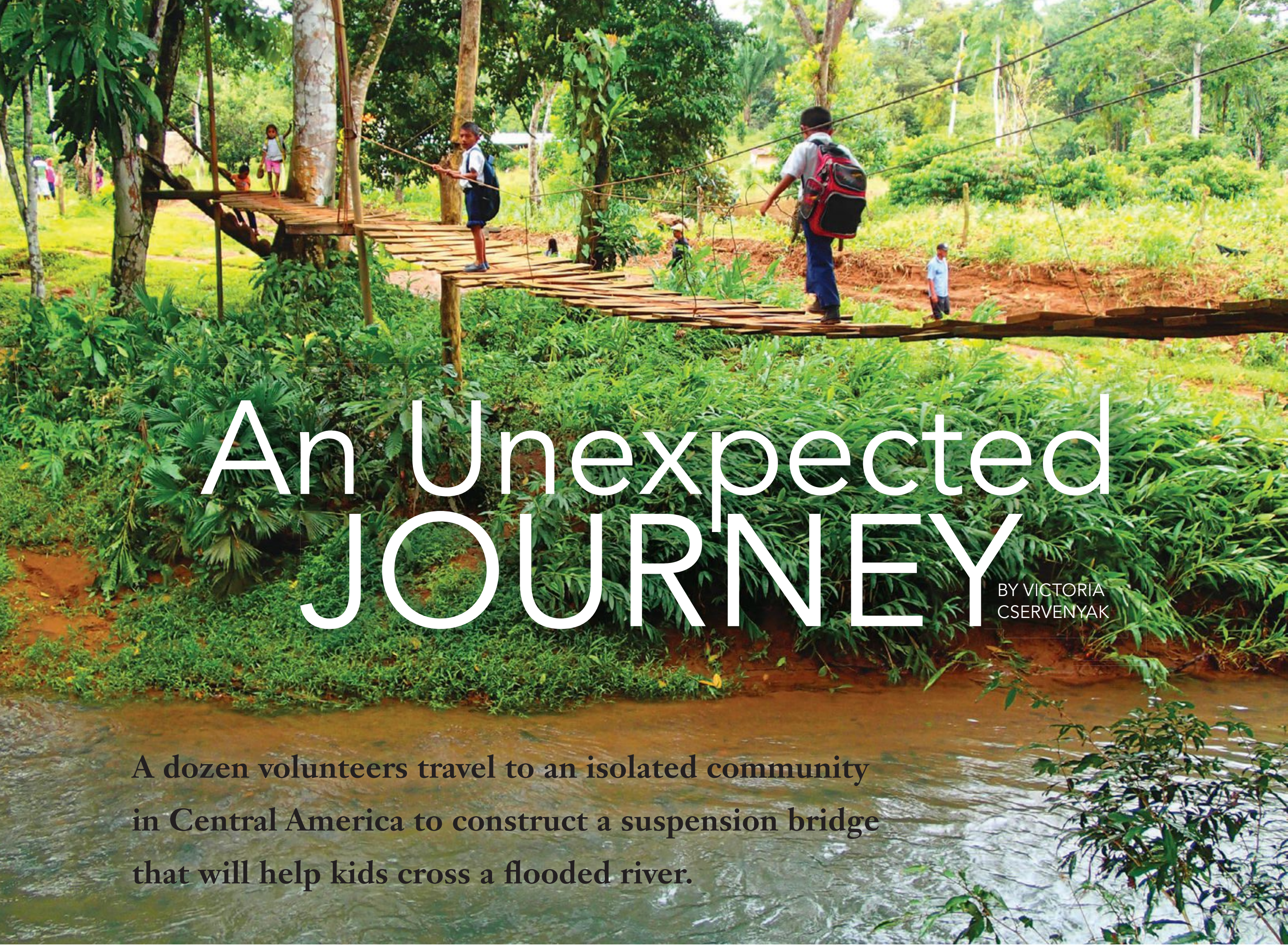
ers. Due to the larger bearing area of the pipe stiffener relative to that of the normal plate stiffener, less area needs to be in contact in the case of the pipe stiffener. A maximum gap of $\frac{1}{16}$ in., the limit for interior connection plates, can be used instead. This is based on findings of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

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

Implementation

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

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



An Unexpected JOURNEY

BY VICTORIA CSERVENYAK

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

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

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

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

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

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

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

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

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

Unanticipated Progress

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

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

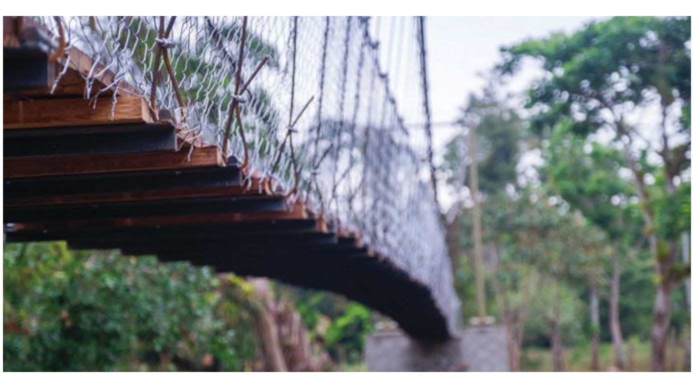
▲ According to the United Nations' Sustainable Development Goals, inadequate infrastructures thwart inhabitants from leaving their community to access agricultural, educational, economic and health-care resources, perpetuating poverty.

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

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

Jonathan Hirschfeld

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



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

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

Holly Bartelt/Brenda Troyo

▲ The team connecting angles to the main tower structure.

▼ The team pulling the main cable through the anchors.

Brenda Troyo

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

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

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

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

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

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

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

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

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

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

Overwhelming First Steps

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

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

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

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

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

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

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

Isolated Community, Connected Commonality

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

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

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

▲ The team on the completed bridge.

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

Keeping Fillet Welding in CHECK

BY CARLO LINI, P.E.

A couple of common questions (and answers) on checking fillet weld designs.

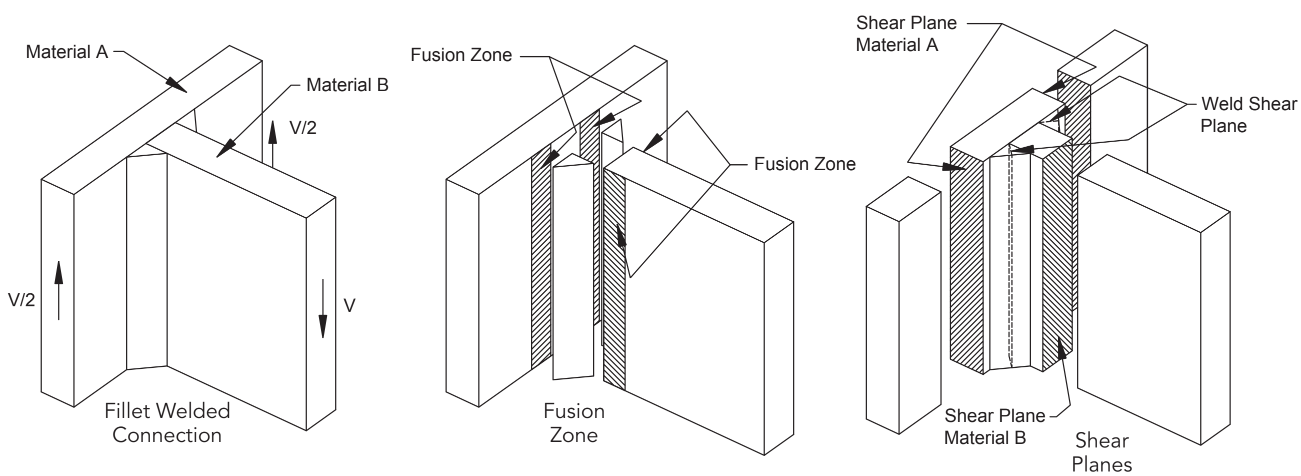


Figure 1. Shear planes and fusion zones for longitudinal shear.

WHEN IT COMES to welding, the AISC Steel Solutions Center receives quite a few questions on these two fillet welding topics: (1) the need to check the fusion zone for fillet welds; and (2) how and when to check the shear plane for fillet welds. Here are some insights on both.

Fusion Zone

Let's start with the fusion zone question. Figure 1 illustrates both the fusion zone and the shear planes for a near-side far-side fillet weld. The weld shear planes are at the bisector of the dihedral angle—what we commonly call the effective throat of the fillet weld.

The strength of fillet welds is covered in Section J2.4 of the AISC *Specification*. For the base metal, the strength is calculated based on the cross-sectional area of the base metal, A_{BM}, which is based on the shear planes identified in Figure 1. For the weld metal, the strength is based on the effective area of the weld, A_{we}, which is shown in Figure 1. Note that Figure C-J2.10 in the Commentary on the AISC *Specification* also identifies the shear planes for fillet welds loaded in longitudinal shear (Figure 2).

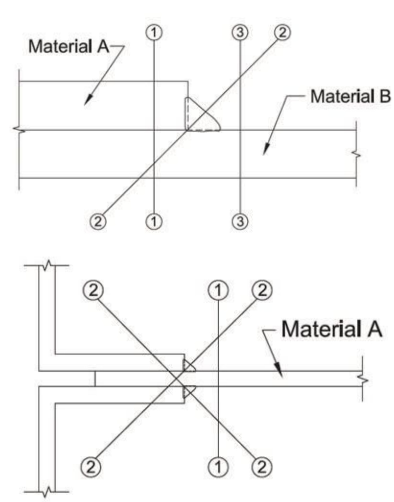


Figure 2. Shear planes for fillet welds loaded in longitudinal shear.

Carlo Lini (lini@aisc.org) is an advisor in AISC's Steel Solutions Center.

Notice that there are no requirements for checks on the fusion zone in Section J2.4. This check is not required because the required filler metal strength per Table J2.5 of the AISC *Specification* must be equal to or less than matching. Note that Table J2.5 does permit the use of filler metal with a strength level one greater than matching.

As stated in the footnote to Table J2.5, Section 3.3 of AWS D1.1/D1.1M defines levels of matching. However, one can loosely summarize the terms as follows:

TABLE J2.5 (excerpt) Available Strength of Welded Joints, ksi (MPa)					
Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Stress (F_{nBM} or F_{nw}) ksi (MPa)	Effective Area (A_{BM} or A_{we}) in.^{2} (mm^2)	Required Filler Metal Strength Level [a][b]
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T–JOINTS					
Shear	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	ϕ = 0.75 Ω = 2.00	$0.60F_{EXX}$ [d]	See J2.2a	
Tension or compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				

- Minimum Weld Strength < Minimum Material Tensile Strength—Undermatching Filler Metal
- Minimum Weld Strength = Minimum Material Tensile Strength—Matching Filler Metal
- Minimum Weld Strength > Minimum Material Tensile Strength—Overmatching Filler Metal

A good discussion on matching, undermatching and overmatching filler metal strengths is provided in AISC Design Guide 21: *Welded Connections—A Primer for Engineers* (a free download for AISC members at **www.aisc.org/dg**). As summarized in the guide, "Standard design procedures do not consider the base metal strength, since the assumption is that the weld metal throat will theoretically control. This is a conservative assumption, provided that matching or undermatching filler metal is used."

Base Metal Shear Plane Checks

Regarding the second topic—when and how to check a shear plane in the base metal adjacent to the fillet welds—this stems from what seems to be an overuse of Equation 9-2 and 9-3 in Part 9 of the 14th Edition AISC *Steel Construction Manual*. The intent of providing these equations is as follows: "In many cases, the load path from a weld to the connecting element can be evaluated directly. However, in some cases, the available strength of the connecting element is not directly calculable. For example, while the strength of the beam-web welds for a double-angle connection can be directly calculated, the strength of the beam web at this weld cannot. In cases such as these, it is often convenient to calculate the minimum base metal thickness that will match the available shear rupture strength of the weld(s)."

Figure 3 illustrates a common condition where Equations 9-2 and 9-3 are used to check the base metal strength.

Unfortunately, it is not uncommon to see a similar check of the base metal even when the load path from a weld to the connecting element can be evaluated directly, such as when connecting a single-plate connection to a column flange. In such cases, it is completely sufficient to check the base metal directly and not necessary to use the comparative calculation approach we adopt when the direct check is not possible. The comparative approach, in a lot of cases, will be overly conservative when checking the base metal strength relative to the weld strength, which is likely based on a fillet weld size that has been rounded up to the nearest 1/16th of an inch or is based on a minimum fillet weld size.

One should also consider if a shear rupture check of the base metal is necessary. Whether one decides to check shear rupture at this location or similar locations is a matter of engineering judgment, although I do not believe this check is typically necessary It would be similar to checking the column flange of a bolted connection for shear rupture (Figure 4), which is typically not done. If this happens to be a controlling limit state, you should consider if the check is necessary before reinforcing is required or the member size is increased.

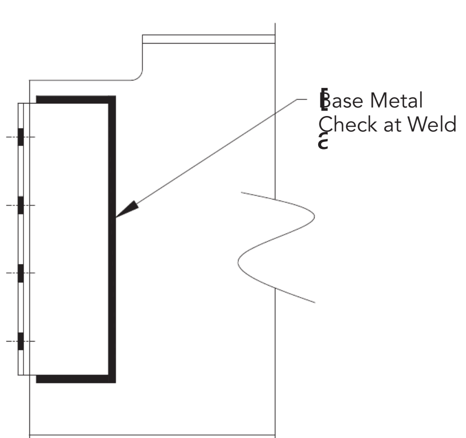


Figure 3. Base metal check along C-shaped weld.

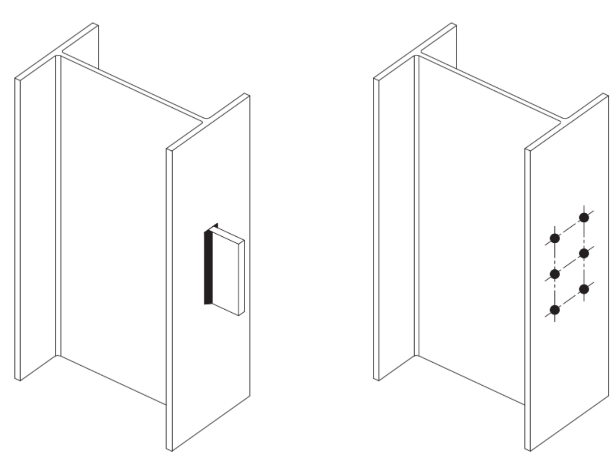


Figure 4. Comparison of a welded and bolted connection to a column flange for shear rupture checks.

news

RESILIENCY

The New York Times Got it Wrong

The *New York Times* got it wrong. In the article "Skyscraper at Trade Center Rises From the Inside Out" (May 25), reporter David W. Dunlap discusses why the contractor for the new 3 World Trade Center building pushed for a concrete core surrounded by steel framing rather than the more typical (for New York) steel core.

According to the *NYT*, the reason goes back to the design of the original World Trade Center. Unfortunately, fact checking at the *New York Times* seems to be out of style. They wrongly characterize the original WTC as having a steel core. But the WTC didn't have a steel core and instead relied on a series of small perimeter columns.

"The World Trade Center towers used an unusual and then-innovative shaft wall core system that was built with layered gypsum board—it was a drywall core, not steel-framed as the *New York Times* incorrectly stated," explains Charles J. Carter, an AISC vice president and chief structural engineer.

"The reality is that steel-framed buildings are the most robust buildings on the market today," adds Carter. "The Federal Emergency Management Agency published their report FEMA 439A, which studied the Alfred P. Murrah Federal Building in Oklahoma City. It states clearly that all concrete-framed buildings need special design requirements to make them blast resistant, and that 85% of the damage seen there resulted because the Murrah Building's concrete framing didn't have it. In contrast, FEMA 439B reported that an equivalent steel-framed building would not have collapsed at all, even without any special treatment in design. Even the Oklahoma City Federal Campus building that was constructed to replace the destroyed Murrah Building shows that steel is more robust and as a result uses a steel plate wall system. Analysis demonstrates it takes a foot of concrete to equal the blast resistance of an inch of steel--and it still would not perform to the same level as the steel wall because the surface concrete always becomes flying debris in a blast, which could potentially injure any nearby people." (For more information, visit **http://tinyurl.com/FEMA439B**.)

The steel-framed WTC 3, under construction.

The decision of whether to select a concrete or steel core or framing system for a modern high-rise is not a safety issue. Rather, the determining factors remain the architectural and structural design requirements, construction cost and scheduling, and the preferences of the specific contractor and designer. Steel framing is the preferred construction material for offices in New York City and throughout the nation. In fact, nationwide three steel-framed office buildings are built for every one concrete office building—and the ratio is even higher in New York City. History shows steel offers unmatched cost, schedule and environmental benefits.

letters to the editor

Engineering is Art

The June issue of *MSC* is really something! The cover photo of the Seattle South Park Bridge is something else, though my favorite is the Hastings Bridge. The picture on page 24 is nearly magical. Who says that engineering isn't art?

Great job by the jury!

—Dr. Reidar Bjorhovde
The Bjorhovde Group, Tucson, Ariz.

People and Firms

- The Board of Directors of the **American Iron and Steel Institute (AISI)** has elected **John Ferriola**, chairman, president and CEO of **Nucor Corporation** (an AISC member), to serve as chairman of the Institute until May 2017. "John is a dedicated and outspoken advocate for our industry, and has engaged his employees and the supply chain to speak out on behalf of steel with their elected officials, especially on the issue of unfair trade," said **Thomas J. Gibson**, president and CEO of AISI. "John's leadership is recognized worldwide, as he also will be chairing the World Steel Association next year. We are grateful to have him lead AISI for the next twelve months."

- **Stites and Harbison, PLLC**, attorney and partner **David Ratterman**, general counsel for **AISC**, has been inducted into the **University of Kentucky (UK) College of Engineering** Hall of Distinguished Alumni. He is a member of in the firm's Construction Service Group, and his practice focuses on general construction law, with particular emphasis on the fabricated structural steel industry. Since its inception in 1992, the Hall of Distinction has honored those alumni who have demonstrated distinguished engineering professional accomplishments, outstanding character and commitment to community service.

STEELDAY

Steel Sculpture Competition Now Accepting Entries

Are you ready to make your structural steel vision come to life? Enter AISC's sixth annual Steel Sculpture Competition! If you're an AISC full or associate member, join this year's competition and create your own innovative steel sculpture for a chance to have your company featured in *Modern Steel* as well as receive a catered lunch!

Here are the rules:

- The sculpture must be steel (and only steel), but shapes, sizes and steel type can be your personal preference.
- The sculpture must be made entirely by your staff.
- The finished sculpture must fit in a 2-ft by 2-ft by 2-ft box (for shipping purposes).
- All entries must include a photo of your sculpture with a title and the name of the company submitting the project. Tips for an award-winning photo: Shoot your sculpture by a window to use natural light (direct sunlight will wash out your sculpture), or face a light towards your sculpture; use a solid background that highly contrasts your sculpture (all white usually works best); and take the photo with a DSLR for optimum resolution.
- You choose the theme! But keep in mind these characteristics of steel: adaptable, economical, quick and sustainable.
- Entrants must be AISC full or associate member companies.

Submit your sculpture entry by September 9, 2016 to AISC's Jenny McDonald at **mcdonald@aisc.org**.

From September 19-23, all entries will be posted on at AISC Facebook page (**www.facebook.com/aiscdotorg**) where visitors can vote for their favorites. The five entries that receive the most "likes" will be put on display at the 2017 NASCC: The Steel Conference, March 22-25 in San Antonio, where the ultimate winner will be chosen by attendees. The winner will also be featured in *Modern Steel* and receive a catered lunch for their company (up to a $500 value).

For more about the competition, go to **www.steelday.org/sculpturecomp**. The competition is part of SteelDay, the structural steel industry's largest educational and networking event, held nationwide. It's scheduled for September 30 this year; mark your calendars! If you're interested in hosting or attending an event, visit **www.steelday.org**.

SUSTAINABILITY

Environmental Product Declarations for Fabricated Steel Now Available

AISC and NSBA have released Environmental Product Declarations (EPDs) for Fabricated Hot-Rolled Steel Sections and Fabricated Steel Plate. These EPDs satisfy the reporting requirements of Version 4 of the LEED rating system, as well as other green codes and rating systems, and are available for free at **www.aisc.org/epd**. These are the first EPDs available that include full documentation of structural steel products from both the mill production and fabrication stages before delivery to the project site.

The EPDs document the environmental impacts associated with domestically produced and fabricated steel products used in the construction of structural steel framing systems for buildings and bridges from the production stage (cradle) through the fabrication facility (gate). Determination of the life cycle environmental impacts was based on industry average production data from AISC member steel mills and survey data from nearly 300 AISC member fabricators.

Structural steel has long been considered the premier green construction material. The structural steel industry remains the world leader in the use of recycled material and end-of-life recycling, with the recycled content of the structural steel beams and columns produced at U.S. mills averaging 90%. Currently 98% of structural steel is recovered at the end of the life of a building or bridge for reuse or recycling into new steel products.

The structural steel industry continues to improve its environmental performance through a history of continuing reductions in greenhouse gas emissions. The results of steel industry efforts are evident in recent findings on greenhouse gasses, which show that on a per ton basis the iron and steel industry reduced carbon emissions by 37% and energy intensity by 32% between 1990 and 2014.

The EPDs are available on an informational basis to architects and engineers involved in the design of buildings and bridges. In addition, they can be used by AISC full members to fulfill the documentation requirements for attaining credits under the LEED and other green building rating systems. To learn more, visit **www.aisc.org/epd**.

AWARDS

2017 IDEAS² Awards Call for Entries is Open

Architectural and engineering firms, structural steel companies, general contractors and owners are encouraged to enter their steel-framed building projects in the 2017 Innovative Design in Engineering and Architecture with Structural Steel (IDEAS2) Awards competition. Conducted annually by AISC, the awards recognize excellence and innovation in engineering and architecture on structural steel projects across the U.S. Entries are now being accepted for the competition at **www.aisc.org/ideas2**. Entrants should note this year's earlier entry deadline of August 26, 2016.

In addition, project entries will receive special recognition on SteelDay (September 30). New to the competition this year is an opportunity for the public to join in the judging of the projects entered in awards program by selecting their favorites at www.aisc.org/ideas2, starting the week of SteelDay. The project that receives the most votes will receive a People's Choice Award in the competition. All winners will be announced at the 2017 NASCC: The Steel Conference in San Antonio, March 22-25.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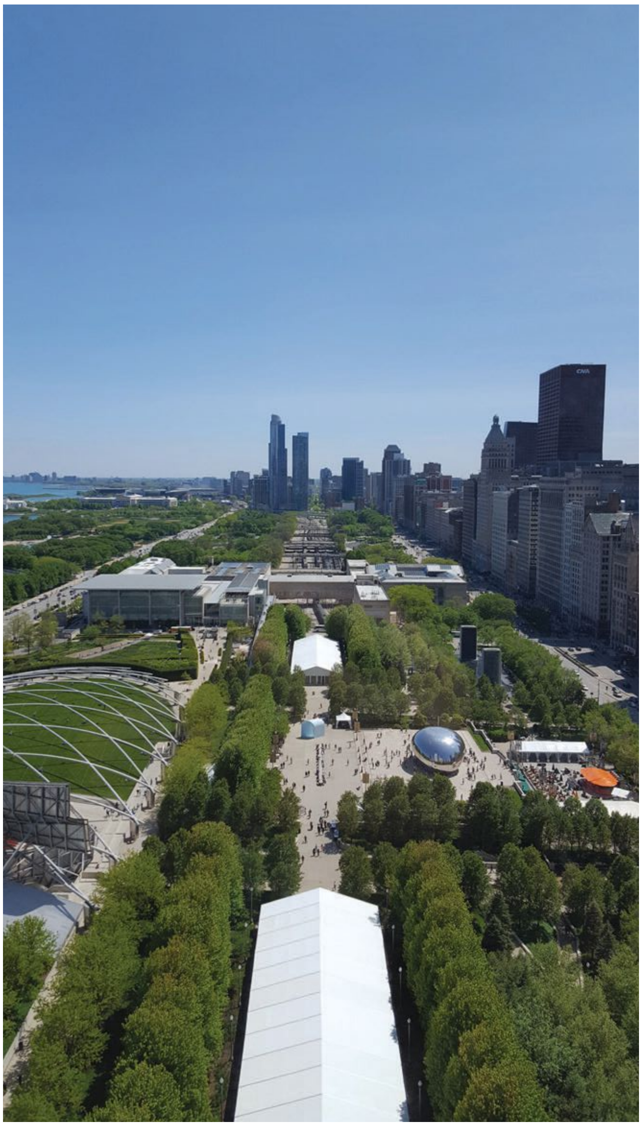
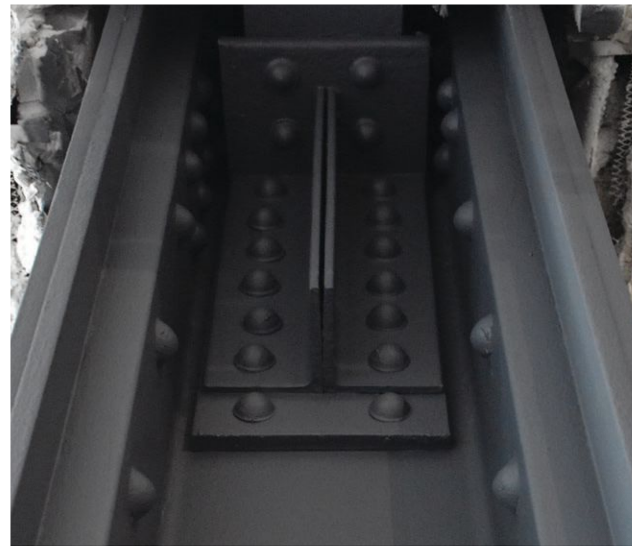
AISC'S NEW VIEW FROM PRU





AISC HAS MOVED! (But not far.)

On June 27, AISC relocated its national headquarters from its longtime home at 1 E. Wacker Drive. "We're going to miss our location in the first U.S. building to use 50-ksi steel, but we're also looking forward to a new office that will provide additional needed space," said Roger E. Ferch, AISC's president.

AISC's new home will be on the 20th floor of 130 E. Randolph St., a 41-story steel-framed building overlooking Chicago's Millennium Park and Grant Park landmarks. Completed in 1955, the Prudential Building (also called One Prudential Plaza) was constructed using the air rights above the Illinois Central Railroad tracks. It was one of the first post-war skyscrapers constructed in Chicago and featured a rooftop restaurant and observation deck that were, at the time, the highest public spaces in the city.

Starting in 2013, the mid-century modern building received an extensive exterior and interior makeover. The aluminum and limestone facade was polished and cleaned, windows and interior finishes were replaced and mechanical systems were upgraded.

But one feature hasn't changed: the structural steel frame, comprised of riveted built-up shapes. The steel is in such good condition—and so visually striking—that many new tenants have chosen to expose elements of the frame. In fact, AISC's space prominently exposes two riveted columns, now protected by a decidedly 21st century intumescent coating.