Modern STEEL CONSTRUCTION

March 2016

Modern STEEL CONSTRUCTION

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ON THE COVER: The San Francisco 49ers' new home, host to Super Bowl 50, under construction, Page 20. (Photo: Hawkeye Photography)

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editor's note

EVERY YEAR, MY YOUNGEST SON AND I GET OUT A SIX-PACK OF ROOT BEER, ORDER PIZZA, CONSUME MUNCHIES AND DIP AND WATCH THE SUPER BOWL.

My son was already exposed to the important information he needed (I showed him an advance copy of this month's article on Levi's Stadium—the host stadium for Super Bowl 50—on page 20) but this time he also had to listen to one of my long-and-drawn-out stories about the steel in football stadiums around the country.

Just a few days before the big game, I happened to see an article from the Alliance for American Manufacturing touting the American credentials of the stadium. According to Tim Salak, project manager from SME Steel Contractors, "All of the major structural steel—rolled sections, wide-flange and channel—came out of Nucor in Blytheville, Ark." Salak pointed out that it wasn't a case of any Buy America clause but rather a standard practice for this American fabricator.

Of course, that got me thinking about the other Super Bowls I'd watched with Jason. Last year's game was at the University of Phoenix Stadium in Glendale, a beautiful steel stadium and winner of a 2007 IDEAS^2 Award (see the May 2007 issue for details on the fabulous roof fabricated by Schuff Steel Company). The 2014 and 2013 games were in older stadiums, but the 2012 game was in the amazing Lucas Oil Stadium in Indianapolis—featured, of course, in the March 2009 issue of *Modern Steel Construction* and fabricated by Hillsdale Fabricators of St. Louis.

I think the first game my son remembers, though, was the 2011 game in one of the NFL's most spectacular stadiums: Cowboys Stadium near Dallas. As the December 2008 *Modern Steel* article explained: "There's a joke that's popular among Dallas Cowboys fans: Why is there a hole in the roof of Texas Stadium? So God can watch his favorite team play." Of course, it also gives a great view of the giant trusses fabricated by W&W Steel out of Oklahoma City. (If you're ever in Dallas, the stadium offers wonderful tours—and if you're lucky, when you're walking across the field, you might be able to pick up a ball and toss it to a friend!)

Regrettably, Heinz Field (where we held an NASCC: The Steel Conference dinner back in 2011) has never hosted the big game. And Orlando, where the conference will be held this year (have you registered yet?), doesn't have a pro team.

While football stadiums aren't built every day, they are one of the most visible uses of steel. And to the surprise of those who aren't in the steel industry, the vast majority of steel used in construction is made and fabricated in the U.S.A.

So the next time you attend a live football (or baseball or basketball or hockey) game, take a few minutes to look away from the action on the field to the structure surrounding you. And if you like what you see, visit **www.modernsteel.com** and search for the stadium. It's likely we've written about it, and you're bound to read an interesting story.

Scott Melnick

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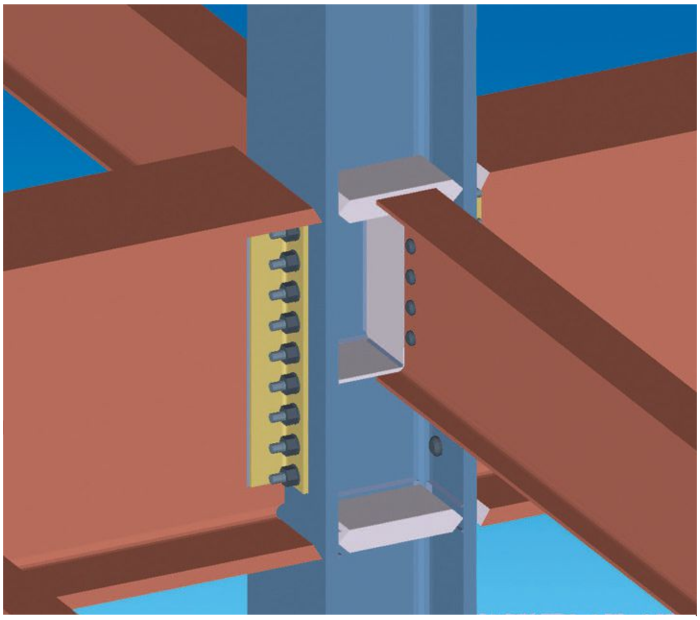

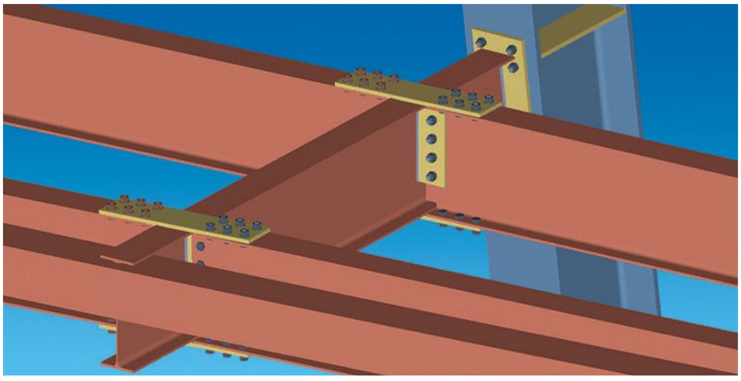

If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

Historical Requirements for Secondary Members

Until the 1978 AISC *Specification*, allowable stresses for bracing and secondary members with l/r ratios greater than 120 were provided with a higher allowable stress than main members by dividing the allowable compression stress by the factor [1.6–(l/200r)]. The justification was based on these members being relatively unimportant and also because of the greater effective end restraint likely to be present at their ends.

The stress increase was no longer allowed beginning with the 1989 AISC *Specification*. It seems that any secondary members in structures evaluated under the older specifications would be deemed unsafe when analyzed using the newer editions of the specification. Is this correct?

No. It is likely that the provision was removed in part because of the difficulty in defining a secondary or bracing member and also relative unimportance. That said, there is no Commentary describing the reason for the change, so all I can offer is my approach to the situation.

There seems to have been three independent provisions in the 1978 AISC *Specification*:

1. when $Kl/r < C_c$, when $Kl/r > C_c$
2. when $l/r > 120$
3. when the member is a secondary or bracing member

However, the third case is also a subset of the other two and is really a simplification that allows the designer to assume in certain cases that $K = 1.0$. This increases the allowable stress to account for "greater effectiveness of end restraint likely to be present at their ends." As stated in the Commentary in 1969: "The formula should be restricted to members that are more or less fixed against rotation and translation at braced points."

The earlier editions of the *Specification* do not prohibit the use of the first two approaches for secondary members.

With this in mind, the comparison should not be made to the current equations with $K = 1.0$, but rather to the current equations with $K = 0.65$. The existing condition would also have to be evaluated to ensure that the original intent was met—i.e., the member was "more or less fixed against rotation and translation at braced points." I have made the comparison (in the table below) between the increase allowed for the secondary members to the increase resulting from the change in K from 1.0 to 0.65. Note that this comparison is likely not exactly what you are interested in, since the increase for secondary members would be from the older column equations and the increase due to K values is based on the current AISC *Specification*. However, it does reflect the conservative nature of the antiquated provision for secondary members. Even if the restraint were more towards the "less" side of more or less, you could probably still justify the strength of the member.

As indicated in the earlier commentary, since the member is flexible it would not take much of a connection to justify a fixed condition.

l/r	$0.65l/r$	1/(1.6–(l/200r))	Table 4-22 ASD stress		Increase Due to K
			l/r	$0.65l/r$	
120	78	1.00	10.1	15.6	1.54
140	91	1.11	7.67	13.9	1.81
160	104	1.25	5.78	12.2	2.11
180	117	1.43	4.64	10.5	2.26
200	130	1.67	3.76	8.86	2.36

Larry S. Muir, P.E.

Short-Term Corrosion

We have a project with minor rusting that has started since the steel was erected about two months ago. The building will be exposed for another four months and then will be enclosed. We are particularly concerned about the effect on the bolts, since they have a smaller cross-sectional area than the members. Should we be concerned about the corrosion?

Albrecht and Hall (2003) compiled atmospheric corrosion data for carbon and weathering steels, with graphs of thickness loss versus exposure time for rural, industrial and marine environments. The graphs can be used to predict the surface material loss due to uniform corrosion. In the presence of oxygen and moisture and the absence of contaminants, Xanthakos (1996) noted that unprotected steel has a uniform corrosion rate of about 0.008 in. per year. If the element is continuously wet or exposed to chlorides found in deicing salts and marine environments, then pitting or local corrosion can occur at a rate of 0.012 in. per year. Using the worst case from both publications, the corrosion loss for six months of exposure is much lower than the bolt dimensional tolerances. Therefore, any light uniform corrosion incurred during normal erection of uncoated steel is typically acceptable.

Additionally, if the joints were designed and installed according to 2010 AISC *Specification* (available at **www.aisc.org/2010spec**) Section J3, any corrosion should be reduced at the areas of the bolt that are highly stressed. After the bolts are installed, the bolt shank and engaged threads will tend to be protected from moisture. For sealing against the penetration of moisture, the geometry of bolted joints must provide tight contact between faying surfaces. This is accomplished by limiting the fastener spacing and edge distance according to AISC *Specification* Section J3.5. This is also an important though often-forgotten reason that bolts are required to be installed to at least a snug-tightened condition as required in *Specification* Section J3.1. The snug-tightened condition is defined as the tightness required to bring the connected plies into firm contact.

Here are the details on the references we cited:

- Albrecht, P. and Hall, T.T. (2003), "Atmospheric Corrosion Resistance of Structural Steels," *Journal of Materials in Civil Engineering*, Vol. 15, No. 1, February, pp. 2-24.
- Xanthakos, P.P. (1996), *Bridge Strengthening and Rehabilitation*, Prentice Hall.

Bo Dowswell, P.E., Ph.D

Percent Composite Action

The Commentary to Section I8 of the *Specification* states: "The degree of composite action, as represented by the ratio $\Sigma Q_n/F_yA_s$ (the total shear connection strength divided by the yield strength of the steel cross section), influences the flexural strength." Must the percent composite action always be greater than 50%?

The AISC *Specification* does not specify a minimum percent composite. The section you have highlighted is part of general discussion in the Commentary. It is not a requirement, so you are allowed to use a lower percent of composite action based on your own engineering judgment.

The 50% recommendation has raised a number of questions, and the AISC committee that oversees the composite design provisions has spent a fair amount of time in the last few years discussing and evaluating minimum composite requirements. There will be some new language on this topic in the 2016 AISC *Specification* and Commentary. I'll try to give you the brief synopsis of what you'll see in the near future without overwhelming you with too much detail.

Historically, a minimum of 25% composite action has been recommended (but not required) for composite beams. However, certain research over the years has indicated that in some scenarios, low percentages of composite behavior could result in a non-ductile failure of the headed studs at the beam ends with the potential for a "zippering" effect of stud failures and a drastic reduction in member capacity. As a result of this research, the language that you highlighted was added to the Commentary, recommending a minimum of 50% composite action.

The current Commentary recommendation implies all beams should meet the 50% composite minimum, but this could be excessive in many situations. The 2016 AISC *Specification* will still not require a minimum percentage of composite action, but it will include a new requirement to "consider ductility."

The 2016 Commentary to the *Specification* will remove the current language that discusses 25% composite as a minimum and include new language that gives guidance on how to "consider ductility." Within this discussion it identifies three exceptions where ductility need not be evaluated:

- Beams spanning 30 ft or less in length
- Beams with 50% composite action
- Beams with an average of 16 kips/ft shear connector capacity (this equates to roughly 1 stud/ft but can also be used when looking at beams with skewed deck or similar conditions where 1 stud/ft cannot be installed)

For beams that do not meet these criteria, ductility will need to be more carefully considered. The Commentary indicates that data obtained from numerical analysis can be used as one method of considering ductility, and provides references to a few different analytical approaches, including the June 1995 *Journal of Structural Engineering* article "Composite Beams with Limited-Slip-Capacity Shear Connectors" by Oehlers and Sved.

Susan Burmeister, P.E.

Multiple Conditions in a Single WPS

Is it permissible for one prequalified welding procedure specification (WPS) to list multiple combinations of variables?

Yes. AWS D1.1 gives latitude to the fabricator relative to the form the WPS takes. Annex Q provides some guidance. The Commentary to AWS D1.1, C-Table 3.8 – Item 3 addresses and permits multiple combinations of variables in a single WPS.

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**.

Larry Muir is director of technical assistance at AISC. Bo Dowswell and Susan Burmeister are consultants to AISC.

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

This month's Steel Quiz takes a look at the design for torsion in W-shapes as addressed in AISC Design Guide 9: *Torsional Analysis of Structural Steel Members* and Chapter H of the AISC *Specification*.

Figures 1 and 2 illustrate normal stresses due to bending (σ_b) and warping (σ_w) and shear stresses due to bending (τ_b), torsion (τ_t) and warping (τ_w) in a wide-flange beam subject to shear, bending and torsional loading. The figures also indicate the location of the maximum stresses, which can be used to determine the locations on the cross section that need to be checked.

The calculated normal stresses due to bending and warping are:

σ_b = 18.7 ksi σ_w = 8.24 ksi

The calculated shear stresses due to bending, torsion and warping are:

τ_b flange = 0.420 ksi τ_b web = 1.61 ksi
τ_w flange = 0.375 ksi τ_w web = 0 ksi
τ_t flange = 6.76 ksi τ_t web = 4.11 ksi

Using ASD, verify that the peak normal stress, f_n, and peak shear stress, f_v, satisfy the requirements in Chapter H of the AISC *Specification*. Assume that the limit state of buckling does not control. Note that the stresses at the 10 highlighted locations can be calculated to determine the location of maximum stress.

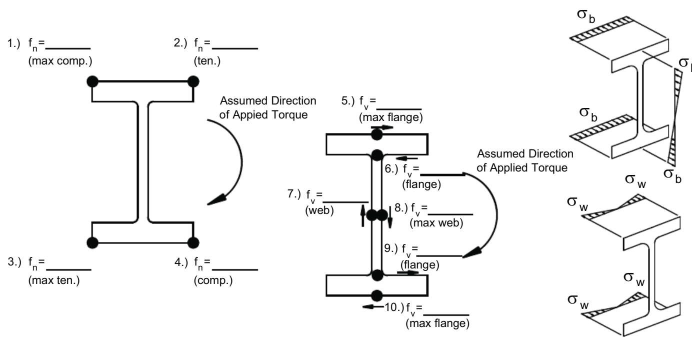
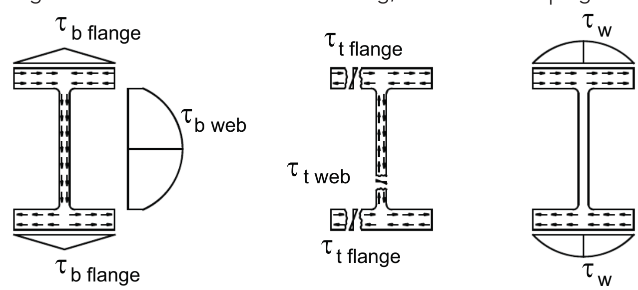


Figure 1. Normal stresses due to bending and warping.

Figure 2. Shear stresses due to bending, torsion and warping.



TURN TO PAGE 14 FOR ANSWERS

Combining the shear stresses and normal stresses at the locations shown in Figure 3, the maximum normal stress and maximum shear stress are:

$F_{n\,max}$ = 26.9 ksi
$F_{v\,max}$ = 7.56 ksi

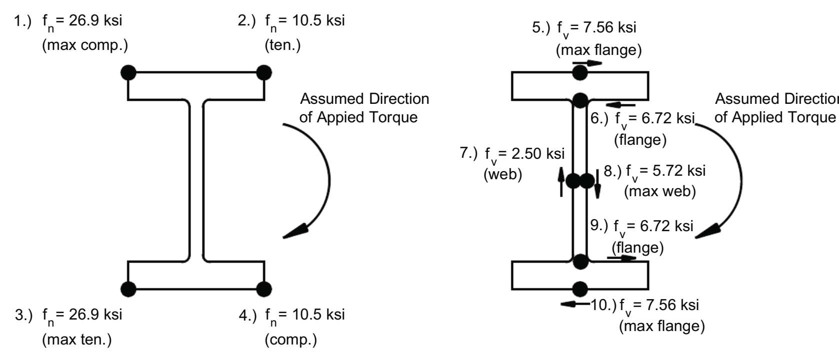
The allowable normal stress per equation (H3-7) is:

$$\frac{F_n}{\Omega} = \frac{F_y}{\Omega} = \frac{50\text{ ksi}}{1.67} = 29.9\text{ ksi} > 26.9\text{ ksi (OK)}$$

The allowable shear stress per equation (H3-8) is:

$$\frac{F_n}{\Omega} = \frac{0.6F_y}{\Omega} = \frac{30\text{ ksi}}{1.67} = 18\text{ ksi} > 7.56\text{ ksi (OK)}$$

Figure 3. Combined stresses where f_n = sum of σ_b and σ_w and f_v = sum of τ_b, τ_w and τ_t.



This information is taken from AISC Design Example H.6.
The complete design example can be viewed at **www.aisc.org/examples**.

Steel Solutions Center

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**.

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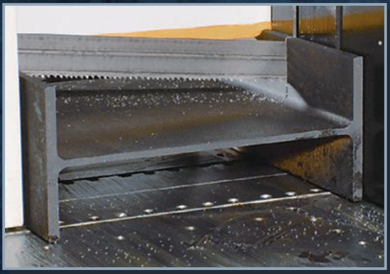

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Three crucial questions for uncovering unconscious bias in the workplace.

business issues

BATTLING BIAS

BY NATALIE HOLDER

DIVERSITY AND INCLUSION have definitely received increased attention over the past 20 years.

Studies have shown that diversity management tops the list of priorities that businesses will have in the coming years. And within the last 10 years there has been an explosion of senior-level diversity officer roles in corporations, higher education and law firms. With all of these resources being put toward increasing diversity, why have most organizations have yet to achieve the change they seek?

You might not have an answer because despite much societal advancement, there are still plenty of reminders that people are treated unfairly because of their faith, how they look or how they sound. Part of the problem is likely that we do not know *how* to achieve the diversity we seek.

> Inviting someone to sit at the table is not the same as inviting them to *engage* at the table.

Diversity or Inclusion?

In the workplace, part of the issue is not knowing the difference between diversity and inclusion. Think of the high school lunch table as a metaphor for experiencing the distinction between the two. Do you remember what your high school cafeteria looked like, sounded like and smelled like? You probably had a group of friends that you ate lunch with every day. Imagine that one day, you asked a different group if you could sit with them, and they enthusiastically made room for you. However, after a few minutes at this new table, you noticed that you were not a part of the conversation. People were making plans for the weekend without asking if you would like to join them. When you tried to tell a joke, everyone stared at you dismissively. People talked over you and cut you off mid-sentence. While you were invited to sit at the table, you were not invited to *engage* at the table. Many organizations do a great job of recruiting for the diversity they seek but fail to create inclusive environments.

Engagement is a measurement of a person's inclusion in an organization and drives the overall quality of the human capital brought to the table. Esteemed psychologist Abraham Maslow's hierarchy of human needs poses that everyone has needs that must be met before they can reach self-actualization. In the workplace, an employee's safety and psychological needs are most likely taken care of because their jobs provide the financial resources to clothe and feed themselves. However, the difficulty in most workplaces starts with the social needs.

When you have friends and positive relationships at work, it creates a sense of *belonging*. The next level of the hierarchy is *esteem* needs. Everyone has a need to have their work recognized by senior leadership. If employees never hear that they are doing a good job, they may doubt their work and themselves.

Lastly, if all your other needs are met, you may reach the level of *self-actualization* at work. Self-actualization is the point where you take initiative and solve the critical problems in your organization. When your social and esteem needs are met, you have the space, room and security to think about new and different ways to contribute to your company's business goals. But if one of these rungs on the ladder to engagement is missing, it can have a negative impact the organization. For instance, employee turnover is one consequence of not having engagement. If your organization had 7,500 employees—and 50% are women and non-white—but saw a 3.6% attrition rate with this population, it would cost the organization $220,000 if it costs $10,000 to replace one employee.

Unconscious Bias

So how and why does exclusion still take place when there are direct benefits to inclusion? Often, without even realizing it, people engage in micro-inequities that are driven by their unconscious biases. Micro-inequities are the subtle gestures, comments and interactions that make you feel included or excluded by others. It's feeling ignored when you're talking

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For more information on Natalie, visit **www.questdiversity.com**.

to someone and they glance at their watch when you make an important point. It's being left off of an email chain when you should have been included. Think of micro-inequities as the waves that threaten to erode your beautiful beach house that sits on wooden stilts. Over time, the waves deteriorate the wooden stilts, often in ways that are unseen by the eye.

While there are a number of ways to uncover exclusion and unconscious bias in an organization—and eventually eradicate it—the process can start with three questions:

- Is there a team member who would view my feedback as negative if I give them any feedback at all?
- Who on the team do I dislike working with?
- Which person on the team am I having an exceedingly difficult time getting to know?

> Often, without even realizing it, people engage in micro-inequities that are driven by their unconscious biases.

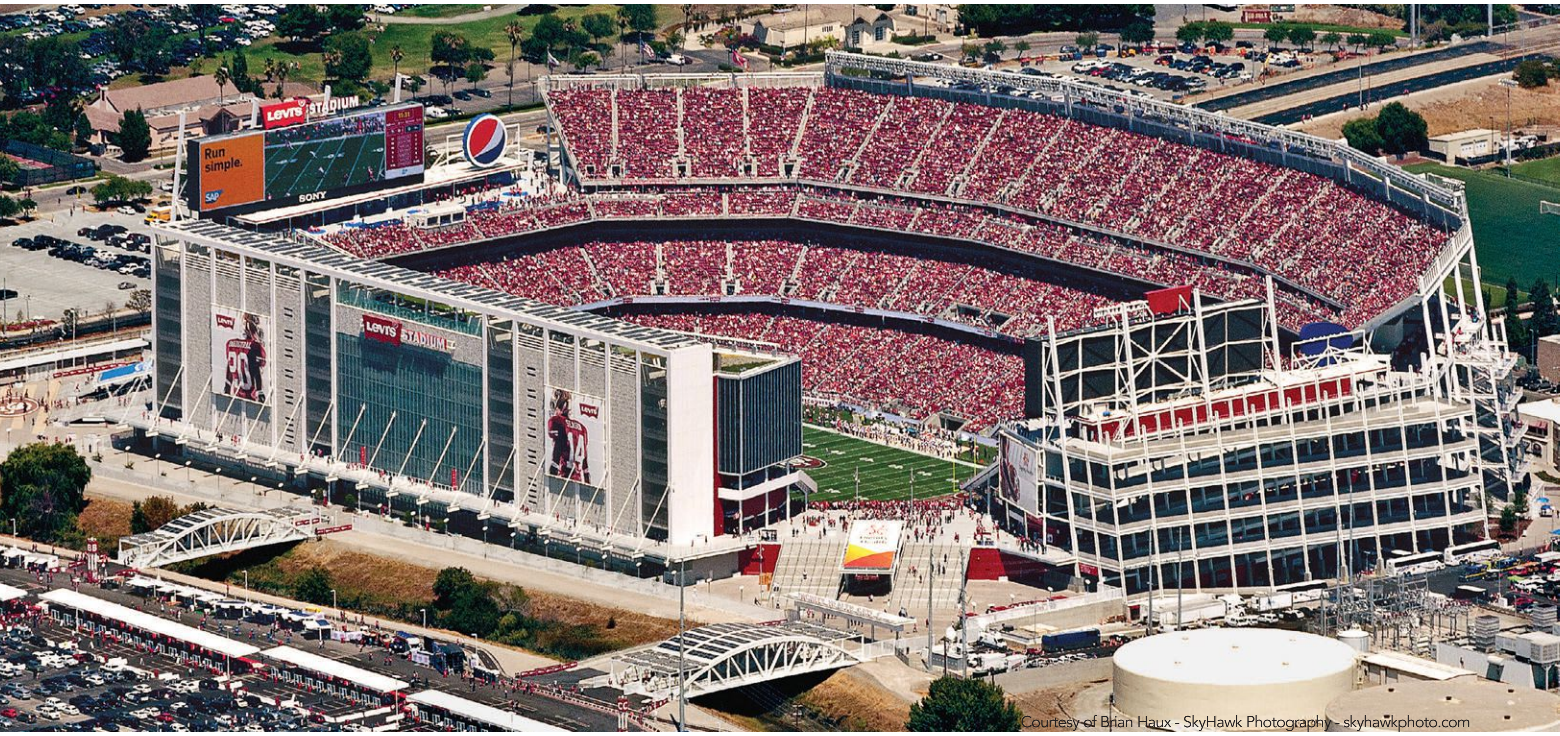
Most likely, the person(s) who surface in your responses are feeling excluded from your work groups.

In a training session for a large government agency, there was a senior leader who admitted that while he was committed to diversity as a cause, he was not putting his actions into practice with certain individuals on his team. He courageously admitted that he created a self-fulfilling prophecy where his favorite employees were excelling while the others, whom he did not connect with and had ignored, were struggling. Invitations to his afternoon coffee excursions to Starbucks were only extended to the people on his team that he liked and connected with.

Even those with the best intentions have difficulty tying their words to their actions. Creating an inclusive culture requires shaking our unconscious minds awake and questioning our actions. ■

It's Up… IT'S GOOD!

BY BRIAN DICKSON, S.E., P.E., AND WYATT HENDERSON, S.E., P.E.



Courtesy of Brian Haux - SkyHawk Photography - skyhawkphoto.com

Levi's Stadium, home of the San Francisco 49ers and Super Bowl 50, sets NFL records for seismic design and timely completion.

Brian Dickson (bdickson@mka.com) is a senior principal and **Wyatt Henderson** (whenderson@mka.com) is a senior associate, both with Magnusson Klemencic Associates.

AS OF LAST MONTH, Levi's Stadium has its first Super Bowl under its belt.

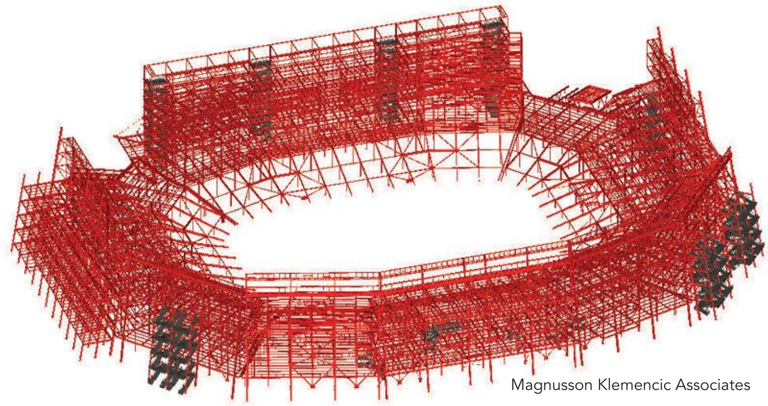
And like the two teams in that game (the Denver Broncos, who won, and the Carolina Panthers), the stadium's design and construction team had to overcome some pretty big obstacles: a project location that tops all NFL stadiums in terms of seismicity and a construction schedule that was accelerated by one year with project design already underway.

Hybrid Project Delivery

Located in Santa Clara, the new home of the San Francisco 49ers has a seating capacity of 68,500 but is expandable to 75,000 (and in fact has already exceeded the latter number by nearly 2,000 during a professional wrestling event last year). To best facilitate the delivery of this large and complex structure within the aggressive construction schedule, the 49ers and the stadium's owner, the Santa

➤ Levi's Stadium has a seating capacity of 68,500 but is expandable to 75,000 (and in fact has already exceeded the latter number by nearly 2,000).

➤ The stadium uses 17,000 tons of structural steel.



Magnusson Klemencic Associates

Clara Stadium Authority, opted for a new hybrid project delivery model dubbed "integrated bridging design-build" (IBDB). Levi's is the first NFL stadium to use IBDB delivery, a highly collaborative and enhanced version of traditional bridging design-build (BDB). This approach was chosen in order to limit risk and eliminate potential owner, architect and contractor disputes and cost overruns common with stadium projects.

In March 2006, the 49ers and the Stadium Authority selected Turner Construction for preconstruction services as well as architectural firm HNTB and structural engineer Magnusson Klemencic Associates (MKA) for the initial design phases. For five years, the design and preconstruction team endured a series of project starts and stops, including a site move from San Francisco's Candlestick Point to Santa Clara in Silicon Valley.

As the architectural and structural design efforts advanced, an ongoing process of value analysis and cost modelling allowed development of a reliable initial guaranteed maximum price (IGMP) for the project to be developed based on a preliminary design. With the IGMP established and preliminary design accepted, the Stadium Authority retained Turner/Devcon, Joint Venture, (TDJV) as the design/build contractor. Under the IBDB process, the design team worked first for the owner/developer through the IGMP phase, then became designers-of-record under direct contract with TDJV to complete the design of the project. This was critical in terms of holding the design and construction team to the established guaranteed price and satisfying the requirements of the preliminary design, while also meeting an aggressive procurement and construction schedule.

A financing opportunity presented itself in the fall of 2011 but included a one-year project acceleration, and the project team went into overdrive to finish in time for the 2014 NFL season. Once all IBDB team members—both as firms and individuals—committed to the extreme acceleration, the team collectively phased, streamlined and "stacked" the design schedule, shortening it by seven months.

MKA agreed to phased delivery of the structural packages, with structural steel on the critical path, requiring that construction documents be completed in the middle of the design development phase of the project. To facilitate local permitting, MKA also proposed a combined peer review/plan check process to expedite a rigorous structural design review and permitting process. By splitting the stadium's structure into eight plan-check packages, MKA continued to engineer packages while others

Magnusson Klemencic Associates

➤ In addition to football, the 1.9-million-sq.-ft facility is designed for soccer, concerts and other high-attendance events.


Turner Construction

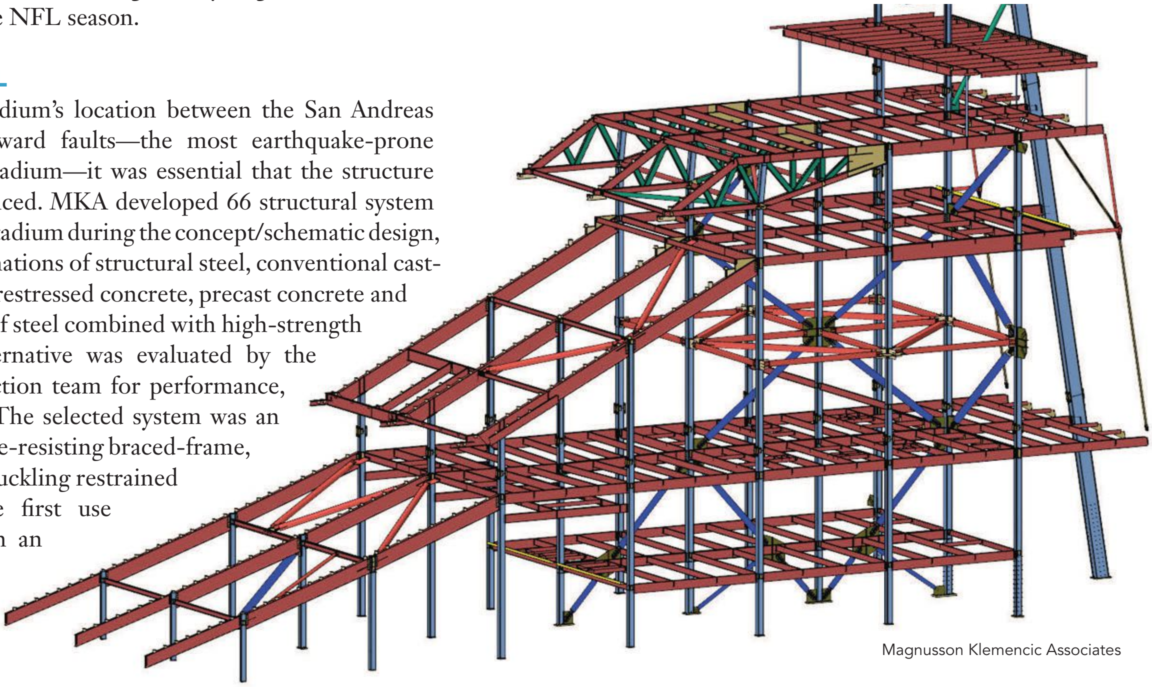
were in review, saving more than two months in terms of project delivery. In addition, the stadium was designed as four separate structures to facilitate simultaneous four-quadrant construction versus more traditional (and lengthier) "racetrack" sequencing. Ultimately, construction of the stadium took just 28 months, starting in April 2012 and finishing in early August 2014, in time for the kickoff of the NFL season.

A First for the NFL

Given Levi's Stadium's location between the San Andreas and Calaveras/Hayward faults—the most earthquake-prone of any U.S. NFL stadium—it was essential that the structure be seismically advanced. MKA developed 66 structural system alternatives for the stadium during the concept/schematic design, including all combinations of structural steel, conventional cast-in-place concrete, prestressed concrete, precast concrete and composite systems of steel combined with high-strength concrete. Each alternative was evaluated by the design and construction team for performance, schedule and cost. The selected system was an all-steel, lateral-force-resisting braced-frame, incorporating 529 buckling restrained braces (BRBs)—the first use of such a system in an NFL stadium. BRBs are high-performance

▲ Steel framing for the seating.

▼ A sample section of the stadium's framing system; BRBs are shown in blue.


Magnusson Klemencic Associates

The BRBs range in capacity from 100 kips to 1,800 kips and are arranged in U-shaped "brace cores" around concessions and other areas.

steel brace elements designed to specifically absorb forces generated during an earthquake—and the buckling restrained braced frames (BRBFs) used here are more efficient and provide higher capacity and greater architectural flexibility compared to conventional steel braced frames. The lateral system configuration developed by MKA required only half the number of braces compared to a traditional braced frame system while effectively addressing the area's intense seismic demands.

Erecting the BRBFs was no more difficult than conventional braced frames, with erection of the steel frame starting in August of 2012 and topping out less than five months later. The incorporation of BRBs produced a structural system six times lighter than a concrete shear wall design and one that required significantly less steel than a moment-resisting frame. The BRB system also improved seismic performance, reduced structural costs by 13% and reduced foundation costs by 20%.

Although Levi's Stadium sits on one of the smallest stadium sites in the NFL, the highly compact framework created by the steel/BRB structure delivered the wide-open spaces envisioned. MKA also optimized the space by cantilevering and "hanging" 40 ft of concourse off the suite tower structure, providing column-free plaza space below. In addition, the stadium's perimeter box columns are sloped, and the bay spacing increased to 64 ft with an inverted king-post rod truss to achieve an open and visually interesting exterior. (The box columns, 24 in. wide by 36 in. deep, are approximately 200 ft in length when spliced together.)

The BRBs range in capacity from 100 kips to 1,800 kips and are arranged in U-shaped "brace cores" around concessions, stairs and restrooms on each level of the five-story stadium and eight-story suite tower. The system creates generous interior passageways with an open main concourse measuring more than 60 ft wide by 45 ft high. The concourses are concrete slab on metal deck, and the tri-level seating bowl comprises precast concrete tread and riser units. In-plane steel bracing below the seating bowl transfers lateral forces to the braced frames and provided stability to the structure during erection. The superstructure is supported by 3,000 drilled concrete pilings up to 60 ft deep that anchor the stadium through soft soils into solid ground.

Box Score

Square Footage	1.9 million sq. ft
Project Cost	$930 million
Under Budget	$80 million
Structural Concepts Evaluated	66
Structural Steel	17,000 tons
BRBs	529
BRB Capacity	Up to 1,800 kips
Construction Time	28 months

Levi's Stadium incorporates 529 BRBs, the first use of such a system in an NFL stadium.

Integrated Bridge

The *bridging design-build* (BDB) project delivery method is a fairly common variation of design-build (DB). An A/E team is contracted by an owner to design a project to a "design intent" level of documentation, then DB teams compete for the project using that documentation. Upon contract award, the DB team completes the design (under the obligation to satisfy the design intent) and constructs the project. The A/E team is under contract with the contractor in a DB arrangement.

Under the *integrated bridging design-build* (IBDB) model, the A/E team is initially contracted by an owner to develop the "design intent" for the project. This A/E team is then assigned to the owner's selected contractor, who will then be responsible for designing and constructing the project under a DB contract. This arrangement effectively allows for better project design continuity because the original design team completes the design from which they originally established the intent. In this scenario, the initial A/E team completes the design of the project under contract with the contractor in a DB arrangement.

Express Delivery

In addition to football, the 1.9-million-sq.-ft facility is designed for soccer, concerts and other high-attendance events. The eight-level luxury suite and press tower on the stadium's west side is topped by a living green roof terrace and solar panels, contributing to the project's LEED Gold rating (another NFL first). The all-steel superstructure uses 17,000 tons of structural steel plus approximately 6,000 tons for stairs and miscellaneous use. The stadium's delivery—over one month ahead of even the accelerated schedule, another NFL record—was celebrated with an inaugural soccer game on August 2, 2014. At $930 million, the final construction cost was more than $80 million under budget, which has accelerated the payoff of public debt on the stadium—another big win.

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Going NATIONAL

BY VICTORIA CSERVENYAK

Over the past quarter-century, the National Student Steel Bridge Competition has evolved from a small academic competition in Michigan to a national showcase of skill and ingenuity that preps students for real-world bridge design.

IN 1993, DON SEPULVEDA was ready to drop out of school—until he joined his university's steel bridge building team, that is.

"I was a full-time student with a family and I was on the verge of burning out," he reflected. "Being on the bridge team [at California State University, Northridge], I saw a purpose and it kept me going. It led to meeting and networking with people, and that led to where I'm at today in my career."

It also led to meeting his wife, Karen, at a regional student steel bridge building competition.

Twenty years later and halfway across the country, Emily Bajwa was a senior in high school and didn't know what she wanted to study or where she wanted to attend college. Her cousin, who was on the steel bridge team at the University of Akron, invited her to participate in activities with the team. The experience inspired her to study engineering at Purdue University and join its bridge team, where she presided over hosting the national competition in 2010, one of her favorite and proudest memories at Purdue.

For the past 25 years, the National Student Steel Bridge Competition (NSSBC) has not only emboldened students like Don and Emily and countless others in their studies, but has also offered them the opportunity to use their shop skills to design, fabricate and erect one-tenth-scale bridges, as well as develop professional connections. But when the competition was first introduced, no one knew it would be more than a one-time event.

Starting Simply

NSSBC's origins date back to 1987 when Bob Shaw, former AISC university programs director, created a small educational competition for civil engineering students in Michigan. Students were given a problem to solve—to build a bridge across a river—an almost identical format to today's competition, which is modified yearly to include different construction constraints, member sizes and weight limits.

Shaw said he chose bridges instead of buildings because "bridges are always exciting. It's something close to the ground and that was manageable and something that created a real scenario possibility. I wanted to have a competition that actually taught the students something and gave them hands-on experience."

So, Shaw set up a pilot competition and three teams enlisted: Lawrence Technological University (LTU), Michigan Technological University and Wayne State University. Local Michigan fabricators sponsored and volunteered at the competition, which was held in LTU's parking lot in March of 1987.

1987
Three Michigan universities compete in the first student steel bridge competition

1992
Michigan State University hosts the first national competition with 13 schools from across the U.S.; MSU wins

1993
Rules Committee is formed

1996
40 schools compete—by invitation only—and the University of Alaska-Anchorage wins

2000
Official name becomes ASCE/AISC National Student Steel Bridge Competition

2015
224 compete regionally and 47 compete nationally at the University of Missouri-Kansas City

Trucks used for load testing were built by Bob Shaw.

Early competitions took place outdoors, with little regard for safety since there weren't official rules.

Wayne State University using ropes to move their bridge (1987).

Michigan Technological University load tests their bridge with their team members (1987).

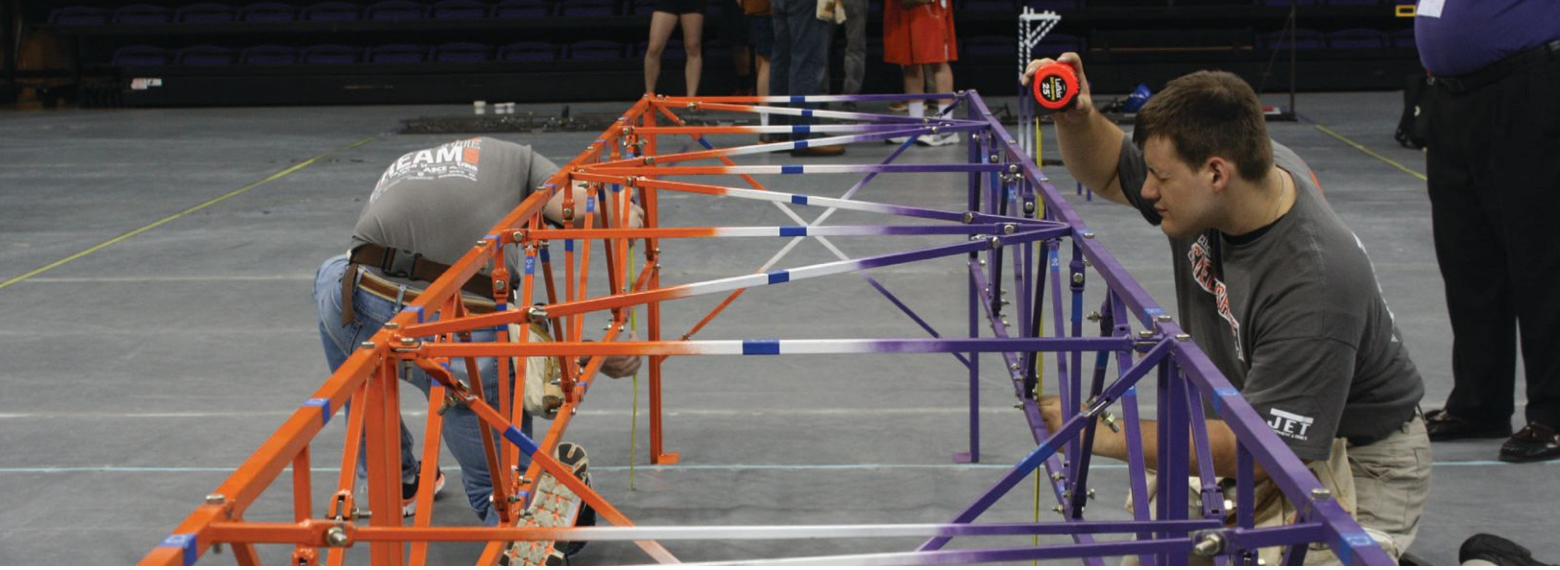
Lawrence Technological University (1987).

"The highlight of the competition was the erection scheme that was used by LTU that basically had students crawling down on a 6-in.-deep member to the very end and reaching across the 'river' to receive the piece from the other end, while being counterbalanced by another student sitting on the end of the beam," Shaw laughed. "It was worrisome and it was scary, but it was very, very entertaining."

Despite how amusing watching the students compete was, the safety issues, hours spent on erection, multiple builders and hundreds of pieces per bridge provoked Shaw to wonder how and if the competition could run at a larger scale.

But over the next few years, the students promptly learned more efficient construction techniques as other universities joined Michigan's competition and launched their own regional competitions throughout the country.

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

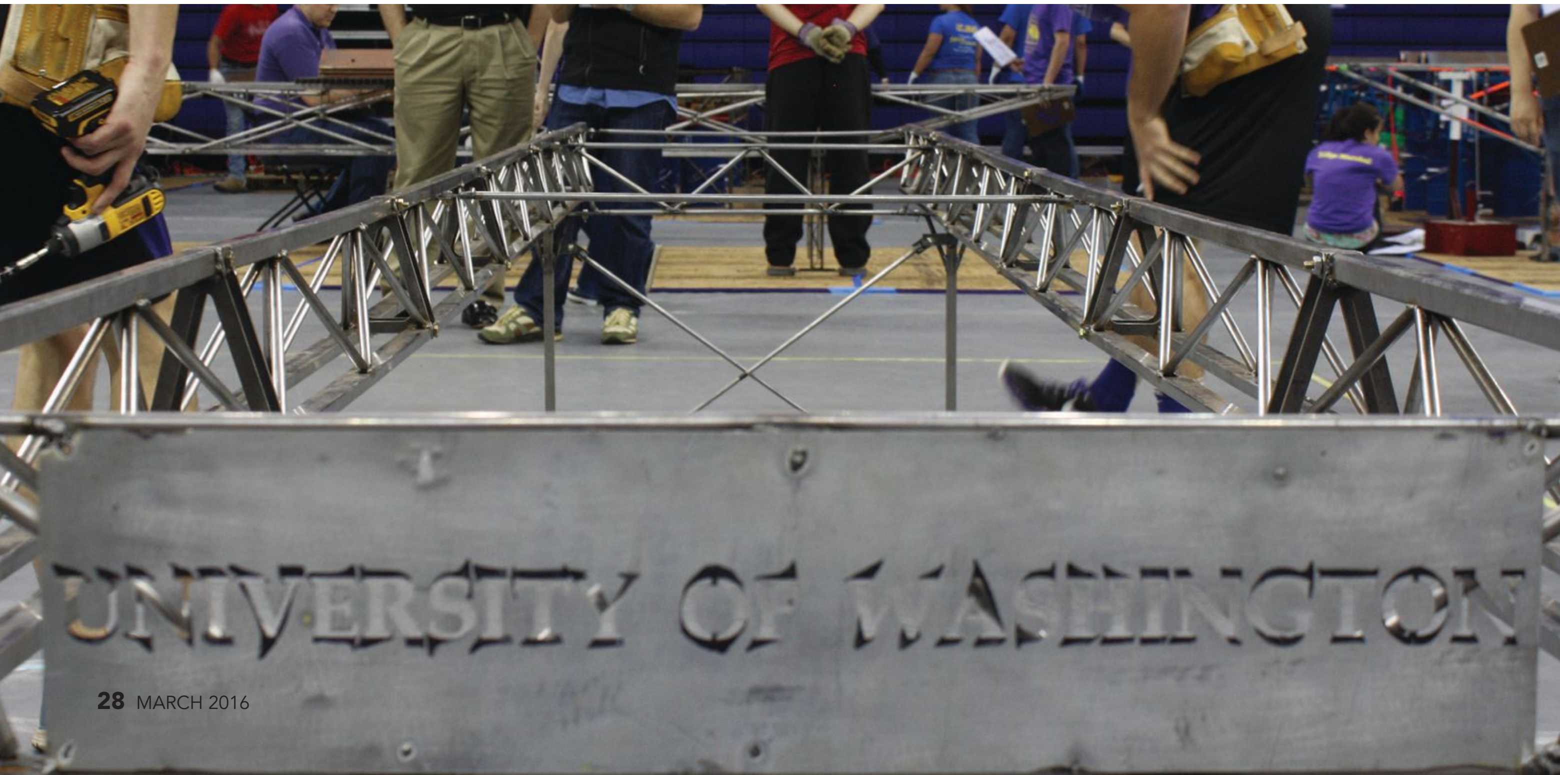


Clemson University uses colored beams to differentiate their bridge from their competition (2013).

The Texas A&M team, sporting cowboy hats (2000).

A bridge fails at the 2000 NSSBC at Texas A&M University.

All schools' names are required to be visible on their bridge.



Across the Country

In 1992, Michigan State University (MSU) challenged all interested bridge teams to compete in the first national competition, with 13 teams signing up. The bridges were cumbersome, with a "fast" bridge taking more than 30 minutes to erect (for perspective, the University of Florida's winning bridge last year took less than five minutes). Frank Hatfield, MSU's bridge team advisor since 1988 and rules committee chair since 1993, said, "In 1992 the winning bridge weighed nearly half a ton. Last year's winner: 85 pounds."

In 1996, due to the large number of teams, the competition took on a by-invitation-only format, with 40 of the top teams from the regional conferences advancing. In 2000, the competition was officially dubbed the ASCE/AISC National Student Steel Bridge Competition (NSSBC) and still maintains the same name and structure.

Each year, with more than 200 teams competing regionally and less than 50 qualifying for NSSBC, teams must be resourceful. Every team starts preparing in the beginning of the school year, spending up to 40 hours a week on their bridge before the regional competitions commence in March.

Bridge development emulates a professional project, with the students being part of a project team and having to develop solutions from start to finish. The students receive a request for proposal (the problem statement) then design, fabricate, load test, practice construction of, select builders for and compete to build the bridge in the competition. For students that are new to the bridge building team, it's also their first experience applying their skills outside of the classroom, balancing a budget and schedule, attaining funding and managing a project.

"Working as a team and making technical decisions—maintaining an environment where the best ideas come forward—is probably the most difficult part," said Gary Fry, Texas A&M University's student bridge team advisor.

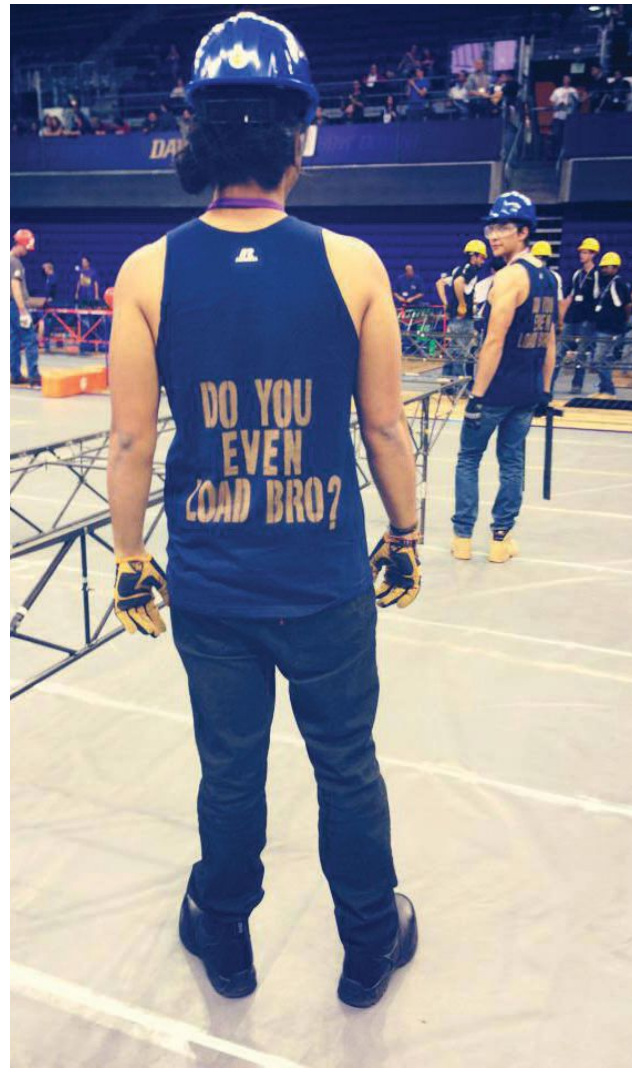
Do You Even Weld, Bro?

As bridge technology evolves, so does the various teams' creativity. While there's no award for team spirit, campy costumes and sprightly shirts have become the unofficial dress code at the national competition, and fervent family and fans, the accessory.

Despite the team members' visual expression, bridge designs are unadorned except for the required school name and the occasional use of colored beams. Teams are judged in six categories—aesthetics, lightness, stiffness, construction economy, speed of erection and structural efficiency—as well as overall performance, with penalties resulting in additional cost, time or weight. And if a bridge collapses, the team is automatically disqualified.

"The first time my bridge failed, we were loading it up and we were watching it get close to breaking and we actually had to stop loading," recalled Sepulveda. "That led a lot to my understanding of the mechanics of it all." Now, not only are there fewer failures, but the number of builders, time and weight have also been reduced as students incorporate contemporary construction methods and technology advancements into their projects and bestow them to the next generation.

Sepulveda explained that connections at the competition have also evolved since he was a student, noting that while bolts, nuts and plates were the norm then, today's competitors use



Don Sepulveda inspecting the University of California, Berkeley's bridge at the 2013 NSSBC.

Apparently they do load, because the University of California, Berkeley team won that year (2013).

Larry Kruth, rules committee member (left) and Frank Hatfield, rules committee chair (right) at the 2015 competition.

dovetails and slotted joints and often devise sophisticated connections that are intricately machined.

Even creativity in the application of tools has expanded. While several teams do their own welding in advance, power tools and welding are prohibited during the competition; only hand tools are accepted as long as they fit into a certain dimension. Students have created piers, employed gadgets to align a portion of the bridge while they're bolting and even used counterweights to not be charged for another builder (which affects their score).

Hatfield believes part of the competition's progress is a consequence of students' having a better grasp of the behavior of compression members. "This is evidenced by the increased use of alloy steels, built-up members and compression members with integral balancing," he said.

Crossing the Bridge into the Workforce

Employers respect potential hires' experience with the competition and students list their role with their school's bridge team on their résumé, which sometimes leads directly to interviews and employment.

Fry said that team membership prepares students for their first job after graduation because it demonstrates that they know how to communicate, stay focused on technical logic and decision-making and be a positive member of a design team.

Six years after meeting a Bechtel executive—the company where she dreamed of launching her career during her time on the bridge team—Bajwa now works there as a contracts professional. She said the competition not only gave her the opportunity to network with industry leaders, but also dexterity as a future project manager and team member.

Divining the Future

Today, students from all 50 states, Canada, China, Mexico and Puerto Rico have participated in regional competitions, with many making it to nationals. The future of the competition is already expanding its global reach, with Iran, Japan and Poland now hosting their own competitions and several other countries inquiring how they can participate.

Twenty-four years after first joining the bridge team, Sepulveda is still immersed in the competition—first as a regional conference head judge, then as a national competition judge and for the past decade as a rules committee member. He continues to volunteer not only because of the competition's personal impact, but also in the interest of seeing it continue to expand.

"The students are our future," he said. "By seeing their activities and the energy that they bring to this competition, it makes us confident that our future is in good hands."

This year's NSSBC will take place at Brigham Young University in Provo, Utah, May 27-28. For more information, visit www.aisc.org/nssbc.

conference preview

CONNECTION DESIGN, DETAILING AND FABRICATION FOR SEISMIC RESISTANCE

A few important things to know when looking at a potential job for the first time.

BY ROBERT WHYTE, S.E., P.E.

THE SEISMIC FORCE-RESISTING SYSTEM (SFRS) is that part of the overall structural system that has been considered in design to provide the required resistance to the code-prescribed seismic forces.

Originally known as the seismic load-resisting system (it was changed to SFRS in the 2nd Edition AISC *Seismic Design Manual*), it may consist of the following structural elements: the steel roof deck as designed as a diaphragm, the chord beams, collector beams, struts, trusses, horizontal bracing frames and the vertical frames (braced or moment). These elements are required to be indicated in both plan and elevation in the structural design drawings. Other non-steel elements such as concrete shear walls, for example, may be part of the SFRS. The steel members that connect to them should also be defined as SFRS and properly identified on the design drawings. If the SFRS is not clearly defined in the design drawings, this could result in an early RFI (request for information) on a job.

This is one of many items to keep in mind when initially exploring a structural steel project. And knowing what to look for early on can help get a project started off on the right foot and avoid potential questions or problems down the line. Below are several terms to consider from the outset.

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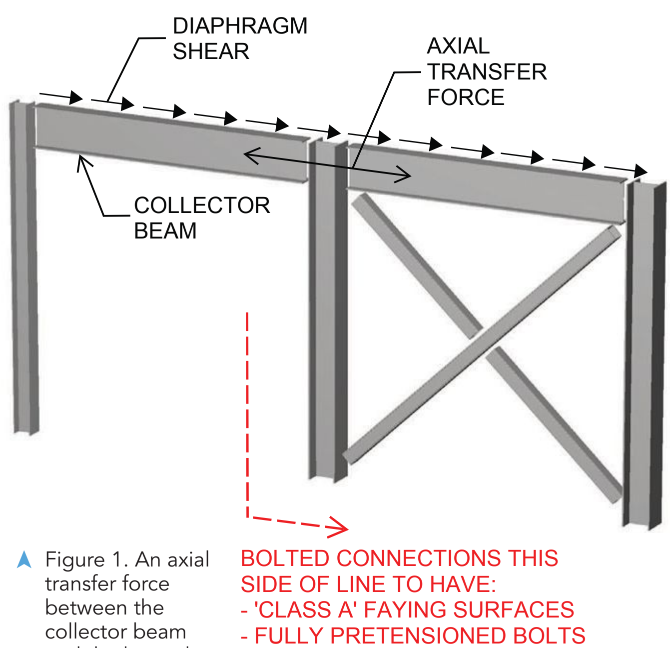


Figure 1. An axial transfer force between the collector beam and the braced bay in an OCBF is something to keep an eye out for.

Response modification factor. From a fabricator's and steel detailer's perspective, the first things to look at when estimating a job are the response modification factor, commonly referred to as the *R* factor, and the description of the lateral force-resisting system (LFRS). These are typically found in the design criteria section of the general notes. The *R* factor is an indicator of the ductility of the lateral system; the larger the *R* factor, the more ductile the system, which means larger deformations under seismic loading. If the *R* factor is greater than 3, all the requirements of *Seismic Provisions* immediately kick in.

Seismic design category. Another term that is good to be

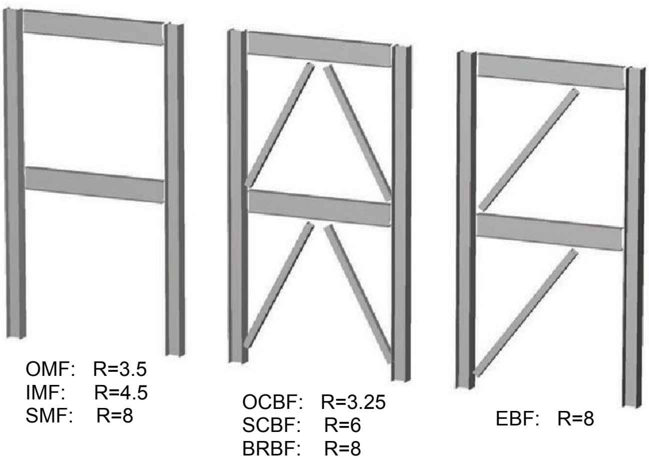


Figure 2. Systems such as intermediate moment frames, special moment frames, special concentrically braced frames and eccentric braced frames have the most stringent detailing and fabrication requirements.

familiar with is the seismic design category (SDC), which is a direct measure of the seismicity of the structure due to its location and soil type supporting the structure. An SDC of A is the lowest and an F is the most severe; the most stringent steel detailing requirements kick in at an SDC of D and higher.

Overstrength factor. Designated as Ω_o, overstrength factor is something that must be accounted for in the sizing of the structural members in the SFRS, and it is important to understand the reasoning behind it. Some elements of the SFRS are intended to dissipate earthquake energy by the ductility of certain "sacrificial" members in the form of yielding and buckling, while other components in the system are intended to remain elastic during the seismic event. The latter are those elements to which the overstrength factor has been applied. SFRS components that require application of the overstrength factor are defined in ASCE 7 *Minimum Design Loads for Buildings and Other Structures* and others are only found in the *Seismic Provisions*. Examples of these are: chord and collector beams and their connections, columns in an ordinary concentrically braced frame (OCBF), horizontal bracing and column splices. Connection design forces for structural members that have the overstrength factor should be clearly indicated in the design drawings.

Bolted connections. Due to the full load reversal nature of seismic loading, all bolted connections in the SFRS must be made with fully pretensioned bolts. The faying surfaces at these locations must be prepared with a Class A surface preparation, which may mean blocking paint. This requirement notwithstanding, bolts may still be designed as bearing. The surfaces to be blocked must clearly be identified in the shop drawings. Field bolted joints of the SFRS should be shown on the erection drawings and bolt pretensioning requirements noted. Exceptions to the surface preparation requirement are allowed for end-plate moment connections. For example, depending on the paint requirements of the structure, it may be less labor-intensive to fully paint an entire column with a coating system that achieves a minimum Class A or Class B surface rather than blocking every surface where a beam or a gusset plate connects to the column.

No mixed connections. Connections where bolts in combination with welds resist a common force are prohibited. Due to the potential of full load reversal and the likelihood of inelastic deformations in the connection plate elements, bolts may exceed their slip resistance under significant seismic loads. A problematic condition can occur at brace connections to columns with the gusset plate welded to the column and the beam web bolted to the column will transfer forces differently from all-welded or all-bolted connections. The welded joint of the gusset to the column will tend to resist the entire vertical force at the column face: the vertical component of the brace force plus the beam end reaction. This is due to the high stiffness of the welded connection versus the bolted connection. This can be a common mishap in the field when the bolts at the gusset plate-to-column interface happen to have been mis-drilled. The obvious fix would be to add welds and eschew the bolt strength at this interface. However, this would immediately require the addition of welds at the beam-to-column interface, which could get overlooked. The objective is to ultimately have proportional levels of connection stiffness at each interface.

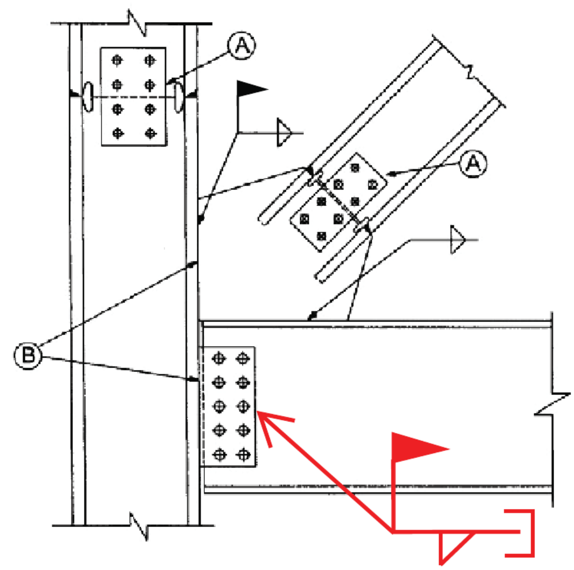


Figure 3. Both interfaces at the column flange must be all bolted or all welded, but not mixed.

Width-to-thickness requirements. Beam, column and brace sections in the SFRS that satisfy width-to-thickness requirements are grouped in the *Seismic Provisions* as moderately ductile or highly ductile. These are sections that ensure a required level of ductility that is dependent on the system chosen.

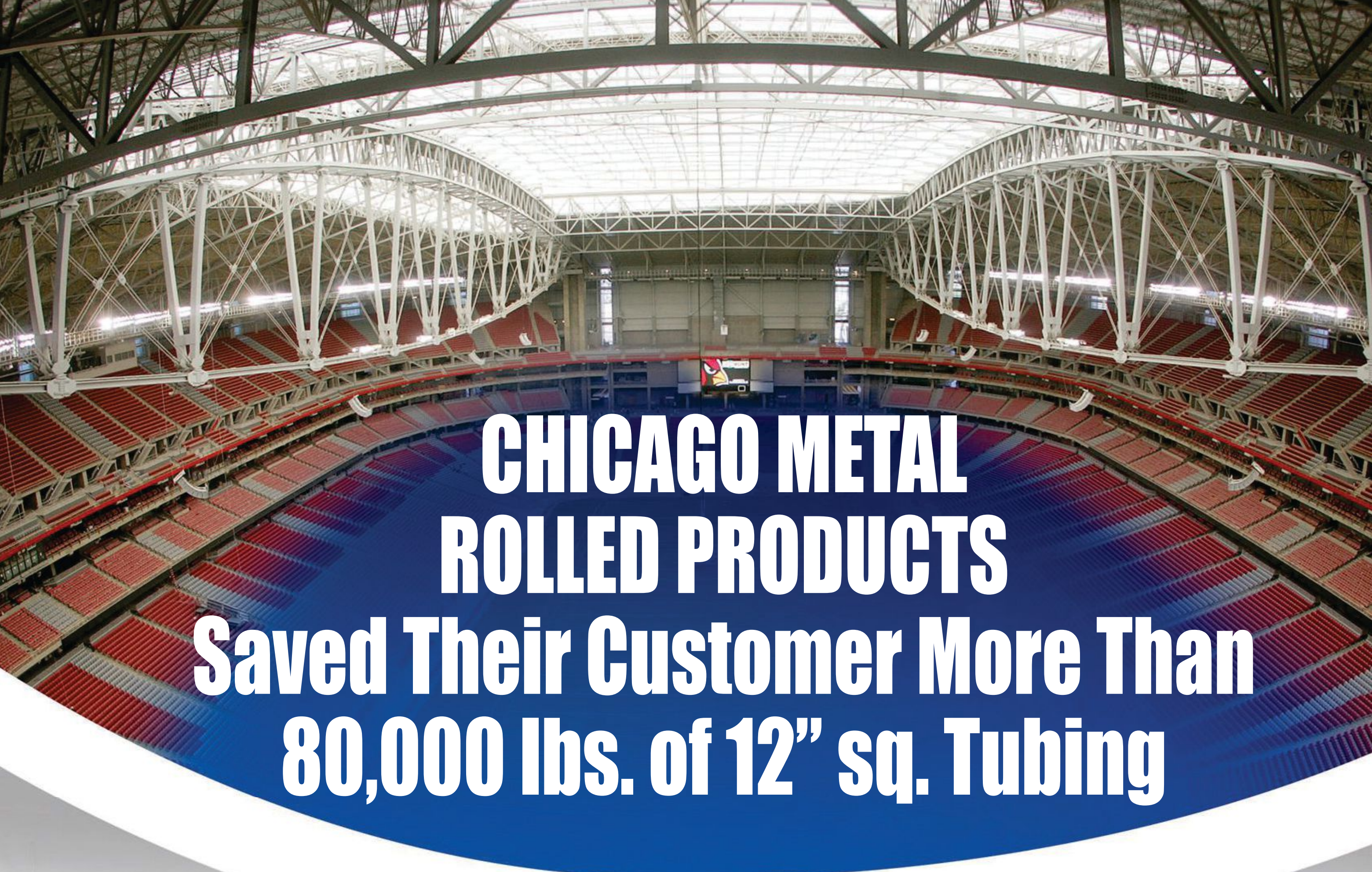
As the required level of system ductility is greater—e.g., in an system with an *R* factor of 6—the sections that satisfy these requirements become stockier. It is a good idea to be familiar with the member cross sections that are permitted under these requirements for a given lateral system.

Protected zones. Fabrication and erection work, and the subsequent work by other trades over the duration of the project, have the potential of causing discontinuities in locations of SFRS members where high inelastic strains are expected under a seismic event. Testing has demonstrated high sensitivity to these discontinuities, which can be anything from a small hole or a "nick" in the steel to a temporary handrail attachment that wasn't properly repaired. Thus, regions where high inelastic strains are anticipated need to be identified and protected. It is a requirement of the *Seismic Provisions* to locate and dimension the protected zones for a given system in the structural drawings. Subsequently, they should be identified in the shop drawings and on the physical "as-built" structural members (in the field). Protected zones include moment frame hinging zones, links in EBF systems and the ends and center of braces in an SCBF system.

Demand-critical welds. Consideration for the high inelastic strain demands mentioned and the consequence of failure require that certain welds in the SFRS be designated as demand-critical. Demand-critical welds must be made with filler metals that meet certain Charpy V-notch toughness requirements using two test temperatures and specified test protocols. It is understood that low temperatures have an adverse effect on the ductility of these welds, thus the "lowest anticipated service temperature" (LAST) should be determined for the given location and use of the steel structure.

Keeping these areas in mind at the outset of a project will go a long way in helping it progress smoothly. ■

This article is a preview of Session N11 "Connection Design, Detailing and Fabrication for Seismic Resistance" at NASCC: The Steel Conference, taking place April 13–15 in Orlando. Learn more about the conference at www.aisc.org/nascc.



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conference preview

EFFECTIVE PROJECT MANAGEMENT

BY LYN BUSBY

Achieving the best results for the whole starts with the successful management of each of the parts.

THERE ARE MANY TASKS and responsibilities associated with managing a steel project. And dividing the job into smaller, more manageable phases will help you better manage each portion and the project as a whole.

Making a Plan

The first phase is the start-up or planning phase. Generally the shortest and arguably the most important phase, it begins when the project gets turned over to the project manager. Establishing a thorough preplan (or not) will affect the rest of the job. The planning should include a review of the contract, contract drawings and specifications as well as the project schedule and your company's proposal. The results of these reviews need to be communicated to the rest of the team: the detailer, the shop, QA/QC and the purchasing department. These communications are very important and should happen even if the project manager has already issued a copy of these documents to all involved parties. Even though some of these documents, such as specifications, may appear to be generic, provisions that are specific to the project should always be identified and noted. The communication of these reviews is even more effective when special written instructions are issued.

During this start-up phase, the project manager should be introducing themselves to the customer in an effort to establish clear lines of communication. The customer needs to know who is responsible for the project, and the project manager needs to know to whom the commercial and technical issues or questions are sent. This is also a good opportunity to arrange an in-person kick-off meeting with the customer. Include the key members of your team in these discussions. If a project is going to succeed, it will depend on the entire team. Along these same lines, it is sometimes beneficial, depending on the size and/or complexity of the project, to also arrange an in-person kick-off meeting with the detailer. The benefits of these face-to-face meetings always result in better communications and help establish a better working relationship.

There is an art and a science to effective project management. The project manager needs to be fluent in both aspects.

In the early phase of the project, the project manager should start setting up all the project-specific documents such as the job schedule, the status report, RFI log, change-order log, cash-flow projection, schedule of values, invoice log, etc. These are typically standard forms used on every job and, if kept in a generic format, can be relatively quickly and easily adapted to project specific requirements.

Following Through

After the planning has been completed, the project moves into the next phase. This phase consists of following the plan that has been established. The detailer should be working and RFIs are starting to flow. A detailed RFI log should be kept that references the detailer's RFI number, the project manager's RFI number and at times even the customer's RFI number. Also included should be a brief description of the RFI, when it was submitted and how long it has been out. In addition to the log, the outstanding RFIs should be documented in a detailed status report, which should be shared with the customer on at least a weekly basis.

The project status report is an important tool used to track pertinent project specific items and in addition to addressing outstanding RFIs should address the detailer's status, including estimated number of drawings by area, the number of drawings that have been submitted for approval, how long they have been out and how many have been issued for fabrication. The report can also include material and fabrication status; this status is particularly helpful to both the customer and the project manager on projects with tight schedules. In addition, the status report can address action items and change-order requests

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and how long they have been outstanding. It is important to push to set up a weekly call with the customer in order to review this status report. If it is not reviewed and discussed on a regular basis, the report tends to get filed and forgotten about. This problem becomes more prevalent on projects with longer lead times, where your company is not necessarily the focus of attention at the moment.

Sometimes customer changes start appearing very soon after the start of a project; other times they start after detailing is well underway. Either way, a clear, detailed system for identifying these changes, reviewing the changes for cost and/or schedule impacts as well as submittal and tracking of these impacts should be developed. Do not let changes linger; nothing good will come of that. Address changes early and make sure the customer is kept aware of the issues. This system should include a clear and simple process for presenting changes as well. More than likely, the project manager will be sitting across the table from the customer at some point explaining how these cost and/or schedule impacts were developed. The more complicated the presentation, the longer this will take and the less likely the outcome will be in the project manager's favor.

Invoicing is another very important step in this phase. Try to get a small invoice out as quickly as possible after the start-up. There are a couple of reasons for this: A small invoice is easier to get paid than a large one, and the project manager will also learn what pitfalls await when the larger invoice is submitted. Very likely there are levels of documentation needed to process the invoice, multiple levels of customer approvals required or cut-off dates to get the invoice in. Learn this process early and you will increase the likelihood of the invoice getting paid on time.

There are many other tasks being performed by the project manager during this second phase including review of shop drawings, vendor coordination, subcontractor communications, materials, fabrication and delivery scheduling and answering shop and field questions. At times, these tasks may seem to become overwhelming, but the key to being effective is establishing a clear plan early, following this plan and addressing the issues that will inevitably come up, quickly and decisively.

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Closing Out

The last phase of the project is the close-out. There are submittals of documents needed such as warranties, as-builts, etc. The final invoice has to be submitted and more than likely, the retainage will need to be addressed. This phase can be short and sweet or may last for months. The project manager should stay in regular communication with the customer to make sure this process stays on track. After the scope is complete, the customer will be heavily involved in the next trades and you will no longer be the priority. But do not go silent.

There is an art and a science to effective project management. Some project managers are very good at one or the other, but to be effective requires the project manager to be fluent in both aspects. The science portion centers around the mechanics of every project. This consists of the checklist items such as RFIs, drawing submittals, vendor and subcontractor purchase orders, documenting and tracking changes, status reports, invoicing, scheduling and shipping. With a clear plan and a little experience, these tasks become repetitive and efficient.

The art portion is a little tougher to define but consists mostly of the ability to establish a productive relationship with the customer, vendors and subcontractors; presenting and selling extras; effectively communicating priorities; recognizing potential issues; and staying in front of those issues.

No project is too large or complex to manage if a clear plan is developed and then followed. However, an effective project manager needs to recognize there are a large number of tasks that have to be performed, and being able to effectively delegate some of these tasks, when possible, is sometimes necessary. The project manager should also realize that no matter how well a project is planned and set up, plans can change abruptly and intelligent, logical decisions must be made quickly in order to keep the project moving in the right direction. ■

This article is a preview of Session N16 "Effective Project Management" at NASCC: The Steel Conference, taking place April 13-15 in Orlando. Learn more about the conference at www.aisc.org/nascc.

conference preview

FUN IS IN THE DETAILS

BY TERRI MEYER BOAKE

When leaving structural steel exposed to view, clean connection details make a difference.

THE EXPOSED STEEL DETAILING that was the basis of the innovative high-tech movement of the 1970s and 1980s, as characterized by the work of Foster, Rogers and Piano, is now mainstream.

Expressed and exposed steel is now a part of many construction projects, ranging from big-box stores to airports. The connection concepts that were developed by these early pioneers have been refined and developed for widespread incorporation into contemporary projects. Current detailing software that feeds into improved fabrication technologies has encouraged this shift in design thinking when it comes to structural steel.

A Clean Connection

What sets architecturally exposed structural steel (AESS) well apart from almost any other type of application of structural steel is the focus on connection detailing and design. From the simplest uses of AESS that might incorporate standard structural shapes, HSS and slightly cleaner and more fastidious alignments, to more complex designs comprised of highly customized elements, the design of the connections is critically important to the aesthetics of the design.

AESS detailing uses a combination of bolted and welded connections in ways that can challenge the design team. A high level of communication is required between the architect, engineer and fabricator to ensure a successful project outcome. Established best practices in AESS connection detailing acknowledge three primary factors that impact detailing choices: use of the building, viewing distance and coatings. (This is covered in detail in my book *Architecturally Exposed Structural Steel: Specifications, Connections, Details*, published by Birkhäuser in 2015.)

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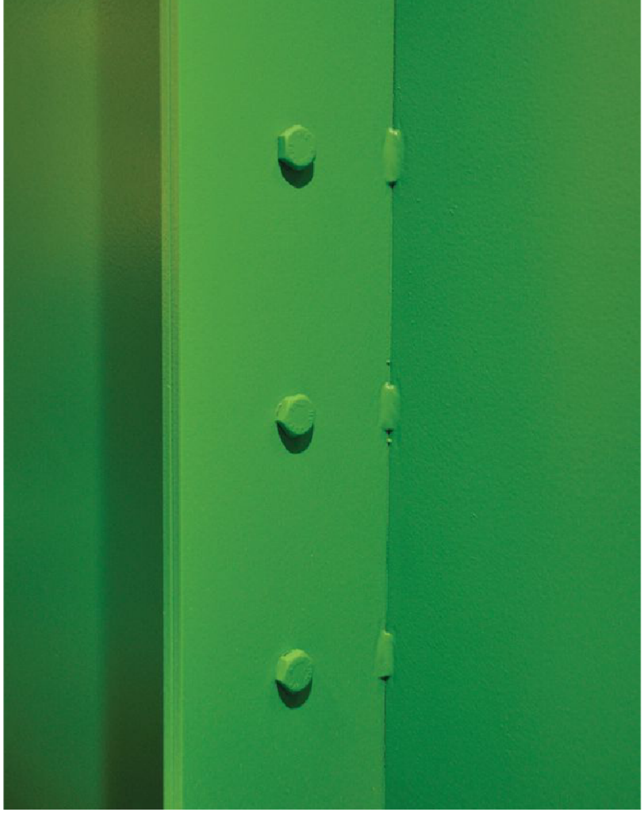
Yet AESS connection details are based on standard connection strategies. These routine practices are modified, enhanced or streamlined to leverage the fabrication to a level of artfulness that can indeed be "fun" when aesthetics are driving the project, even when answering to structural requirements. Because the resulting structures tend to have a higher level of articulation, designers must make an effort to maximize the amount of welding that is done in the shop if control and outcomes are to improve. Many innovative bolted connections have been generated to answer to erection issues for connections that for many reasons must take place on-site. These bolted connections can be seen as a design enhancement and not simply a way to avoid site welding.

Here are my top 10 favorite exposed connections from an aesthetic and structural standpoint.

The detailing of the large arched trusses that span the departures hall of this airport is playful. The members have been chosen to highlight their tensile versus compressive function. Pin connections are extensively used to attach the tension members while plates inserted into the round tubes that comprise the compression members create a visual contrast.



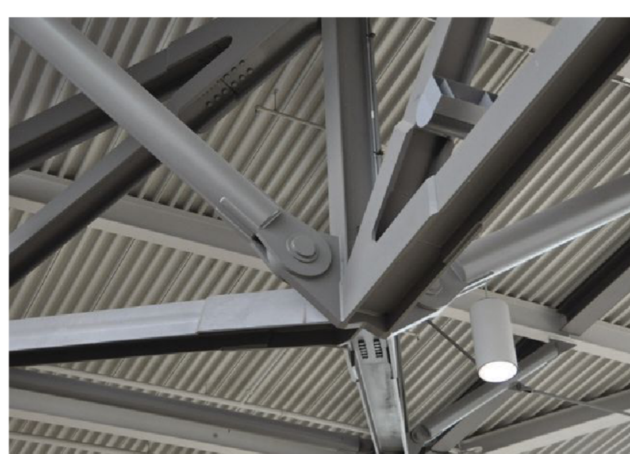
Instead of attempting to make a continuous weld (not structurally required) or use filler to mimic one, the intermittent welds have simply been aligned with the bolts. This looks better (in my opinion) than what might have been more routinely done.

The design of the tapered ends of these tubular struts has become the standard way of gracefully handling the pin attachment of tubes to plates. A higher level of AESS is needed as the parabolic plate inserts do require that the welds be ground in order to achieve visual smoothness. The stainless steel connectors used in place of standard structural bolts give the finishing touch to the detail.

Galvanized steel, being rougher in appearance, seems to demand details that are more rugged. This structure of square HSS tubes was shop welded into larger chaotic looking arrays and bolted on-site. The simple use of side plates on the bolted connections created a very trim appearance yet with a texture that suited the overall aesthetic of the structure.

Even something as simple as a bolted beam-to-column connection can be elevated to artistry if the bolts are carefully located with geometry and symmetry in mind.

This structure uses an artful combination of wide-flange shapes and round tubes. The tapered ends of the side plates used to shop weld the wide-flange makes for a trim appearance. The site-connected round struts use pin connections, with the round ends of these elements making a purposeful contrast with the rectilinear aesthetic of the primary trusses.



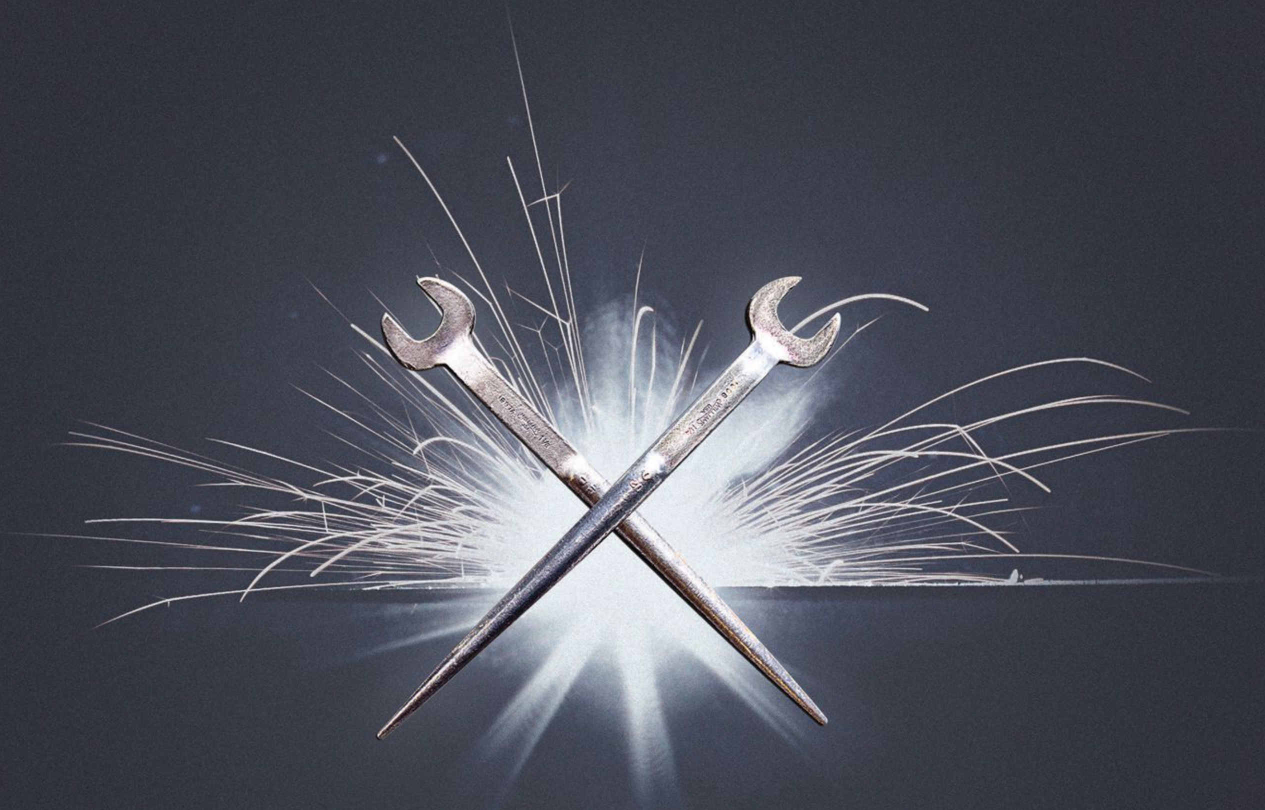
▲ This connection showcases a high level of thought in accommodating the transition from the small diameter of the cable to the attachment of about 20 plates to a mast. The thickness of the plate at the pin connection has been increased to resist pull-through forces. The thickened element tapers down to meet the primary plate, whose purpose is to transfer loads to the mast. It widens to provide adequate transfer via its fillet welds. There is just enough room between the many plates to do the welding (although I expect it was tight!).

▼ This colorful structure makes use of pin connections to address assembly on-site. Clevises have been used with the narrower members and simple plate extensions are used to connect the wide-flange members.

▲ Discreet site bolted connections can be beautiful. The connections that were shop welded to the ends of each of the (many) connecting round HSS tubes in this transit hub allowed for quicker erection and also added a level of visual interest to the structure that would have been quite plain and more expensive if invisible tube-to-tube welds had been requested.

▼ Joining multiple members at a point of focus is always a geometric challenge. The tapered end connections on these round HSS tubes not only respond to the geometry but also provide a simple bolted connection.

This article is a preview of Session N28 "Fun is in the Details" at NASCC: The Steel Conference, taking place April 13–15 in Orlando. Learn more about the conference at ***www.aisc.org/nascc****.* ■



Collaboration is one of the keys to constructability.

conference preview

IT'S NOT ALL ABOUT ME: A HOLISTIC APPROACH TO CONSTRUCTABILITY

BY PHIL JONES, P.ENG., P.E.

I HAVE GOOD NEWS for the structural steel industry: The construction world continues to change and structural steel stands to benefit.

To elaborate, the evolution of construction contracts has largely moved beyond the stipulated sum, making way for countless variations of project contracts focused on shedding risk while improving cost and schedule certainty. The "dumb contractor" role of old has been pushed to the outskirts of the industry as owners and developers place higher expectations on general contractors to shoulder and better manage their risk. The excuse of "it wasn't on the drawings" has been replaced with "I should have known to include that." This is certainly evident in the public-private partnership (P3) model as contractors now are responsible for designing, constructing, financing, operating and maintaining projects. In the last ten years, the P3 model has quickly become a contract of choice for complex Canadian infrastructure projects (e.g., it now represents twice the "traditional" revenue for my firm). The P3 model is now emerging in the United States as well and is likely to gain much traction for its ability to assign risk and, at least for the client, mitigate legal conflicts of responsibility.

Of course, the corollary of evolving contract models for general contractors is a trickle-down effect to the subcontractors. The result has not necessarily been through a corresponding change to the contracts but rather in the expectations. While the proactive contractor must possess a greater variety of in-house technical skills and services, the need for collaboration with subcontractors has dramatically increased. We can no longer work within our black box of knowledge and scope but instead must blur the lines to achieve an understanding of how our work affects other sub-trades around us, in an effort to achieve true constructability.

Defining Constructability

The word constructability implies a variety of definitions but for our purposes here, it is defined as "working to achieve efficient construction through optimization of labor, materials and cost." Simply put, constructability is something that already exists at the core of the structural steel industry. Some of this has been driven by the trend towards delegating connection design to the fabricator, but arguably more so due to the preexisting reliance on the fabricator to incorporate and coordinate the many other project typical details and the requirements of other sub-trades.

Incorporation of curtainwall connections, mechanical openings in slabs or beams and attachment points of secondary framing, for example, is something largely unique as a sub-trade requirement to structural steel when compared to cast-in-place concrete construction. This has pushed most of the structural steel fabricators to embrace 3D BIM software for modeling and collaboration well ahead of most consultants and contractors. For structural steel, the BIM revolution is old news as the rest of the construction industry now works to catch up. However, as the session and article title implies, this is not about you and it is not about me, but instead about *us* and how we all work together. With this experience comes great responsibility to operate as leaders and collaborators.

One secondary effect of 3D collaboration and clash detection is the noticeable increase in buildable architectural expressions and complexities. Even modest buildings seem to increasingly incorporate sloping columns, two-dimensional curvatures and intentional randomization of geometry. Modeling and coordination of these elements has become easier, but the need for constructability and holistic design approaches has grown significantly. In many cases, issues of poor constructability come from the delineation of scope—better known as the "by others" scapegoat—where the impact of one trade's work creates costs to others and is not always captured in a scope of work. An example of this would be a temporary steel support located over a concrete slab below, requiring reshoring by oth-

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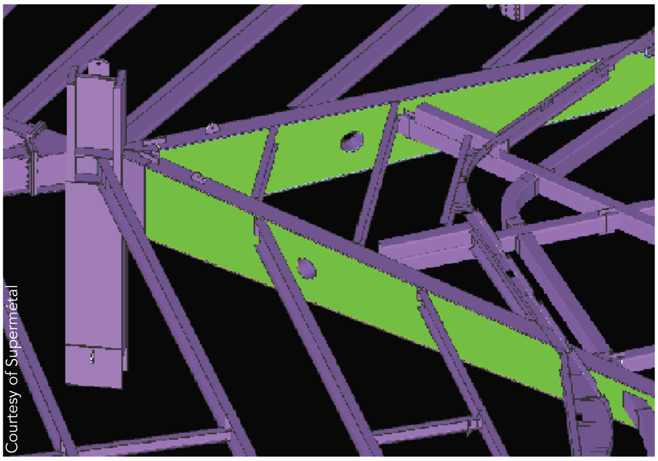
Courtesy of Supermétal

▲ Steel coordination of web penetrations for MEP systems and notches in the curved edge plates for curtainwall box-outs.

▼ Tower cranes for Brookfield Place take into consideration installation requirements of the various sub-trades.

Phil Jones

ers, instead of locating it on a column or wall. However, in most cases the difference between a solution of elegance and cost-effectiveness and a solution of significant expense and schedule implication is usually very subtle. Too often we are all reminded of this through 20/20 hindsight because these subtleties require extensive experience to conceptually plan a project at the macro level while taking into consideration the implications at the micro level.

Trades also must understand and accommodate the needs of each other. This includes a working knowledge of the other trades, including dimensional tolerances, construction methodologies and keeping an open mind to doing things differently to better the project. A good example of this was the Brookfield Place project in Calgary, Alberta, where the tower cranes were planned to suit the requirements of the structural steel framing and advancing self-climbing concrete core. Custom tie-backs were developed, taking into consideration the installation of the curtainwall mullions and worker headroom on the affected floors. Tie-back connection shoes and connection rods at the concrete core were detailed specifically to accommodate the core zone reinforcing and avoid interference of the connection rods with the elevator installation behind.

Implementing Constructability

The simplest first step to constructability is having the right people involved in the preplanning, and this includes the general contractor and sub-trades at the conceptual and schematic design stages. Of course, this requires trusting relationships between all parties to work in the best interest of the project rather than themselves. Newer contract approaches, such as integrated project delivery (IPD), work towards this concept by involving various parties in a collaborative contract focused on reward for project success over individual success (for more on IPD, see "The Business Case for Integrated Lean Project Delivery" in the February 2015 issue, available at **www.modernsteel.com**).

While the IPD method takes this concept to a new level contractually, the core concept of collaboration for the sake of the project still applies to other delivery methods, often well ahead of any contract being in place. In my experience, the results of early collaboration and constructability efforts can be the difference between project viability and never being constructed. Unfortunately, the invitation to collaborate at project conception does not always come with immediate payment. The reward though can be well worth the investment should the owner decide to sole-source the construction contract. At the very least, this provides a leg up on the competition in understanding the project scope and the client's needs when bidding.

As I stated at the beginning, the structural steel industry is well positioned to benefit from the changing landscape of construction. Your experience matters, and those who are willing to embrace deeper collaboration and freely exchange knowledge will succeed. Remember: We're all in this together. ■

This article is a preview of Session N14 "It's Not All About Me: A Holistic Approach to Constructability" at NASCC: The Steel Conference, taking place April 13-15 in Orlando. Learn more about the conference at ***www.aisc.org/nascc****.*

conference preview

THE SPLICE IS RIGHT

BY SYLVIE BOULANGER, P.ENG., PH.D.,
CAROL DRUCKER, S.E., P.E., P.ENG.,
LAWRENCE F. KRUTH, P.E.,
DUANE K. MILLER, P.E., AND TERRI MEYER BOAKE

Sixteen tips for fostering collaboration when it comes to splices.

THE UNSUNG HERO in connection design is the splice.

However, when the splice is right, the price is right. Getting it right means meeting engineering, fabrication, transport and erection requirements. It also means paying special attention when the field splice is all welded or fulfills an architecturally exposed structural steel (AESS) role. Following are sixteen tips to initiate discussion between the architect, engineer, fabricator and erector on splices.

1. Design for the required forces. The actual required weld or bolts at the splice should be determined based on the required forces. When designing splices with moment, the minimum axial load in the column may be used to offset the tension force from the moment. AISC Design Guide 1: *Base Plate and Anchor Rod Design Section 3.4* "Design Procedure for a Large Moment Base" is a good reference for connections with moment and axial load.

2. Avoid overusing complete joint penetration (CJP) welds, especially at field welded column tension splices. When welded connections are to be used, and if properly sized partial joint penetration (PJP) welds are suitable to transfer the applied forces, there are advantages in using PJP welds in lieu of CJP welds. In addition to lowering the cost of welding due to less weld volume and testing requirements, PJP welds are typically detailed with no gap, allowing for some direct bearing at the flanges. CJP welds are typically detailed with a root opening, requiring more robust erection aids to temporarily hold the column in place before the welding is complete. Note: Avoid CJP joints at tension splices.

3. For gravity splices, select members such as W14 sections for clean column lines. AISC's *Manual* gives various gravity column splice details that may be used for typical conditions. These details are more historic in nature and do not require further calculations. However, they are not necessarily applicable for non-typical gravity splices or tension splices but are still a good lower bound for these connections. W14 columns are preferred for clean column splices and to facilitate framing connections. The advantage of W14s is that the inside-flange-to-inside-flange dimension of all W14 columns (W14×43 and greater) is equal and thus provides full-contact bearing at the thinner column flange. If temporary wind conditions are to be checked, ASCE 7/ASCE 37: *Design Loads on Structures during Construction* can be used to determine the temporary wind forces.

4. Consider end-plate connections for compression members other than columns. When splicing compression members such as truss web members or truss chords, consider end-plate connections to take advantage of the compressive force in the member. When compression splices for members other than columns are finished to bear, AISC's *Specification* Section J1.4.2 allows the splices to be proportioned for either 1) a tension force = 50% of the required compression force or 2) the moment and shear resulting from a transverse load applied to at the splice location equal to 2% of the required compressive strength of the member. The minimum of these forces may be used and the forces act exclusively from other forces at the splice.





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5. Weld multi-member joints in the shop and introduce a simple splice away from the joint. Ideally, a splice is introduced at a location where a connection is already naturally occurring. However, if three or more members intersect at one location, it is often more feasible to weld the joint or node in the shop for a better control of the geometry and assembly. In such a case, stubs will project away from the joint and often practical end-plate bolted connections will be used for compression splices and overlapping plates for tension splices.

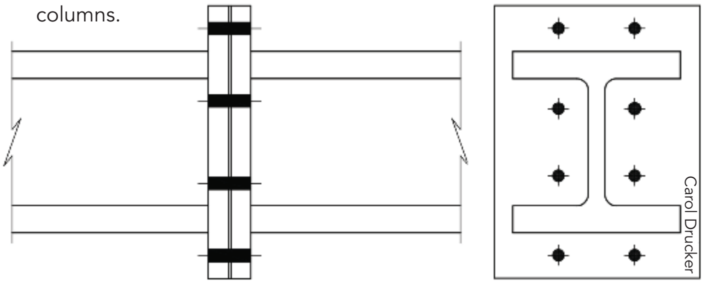
6. Trial fit splices with complex geometries in or near the shop. The consequences of a splice, or several coordinated splices, not fitting in the field generally justify a trial fit-up in the fabricator's shop or yard. This trial fit-up can be part of the specifications for the project, but often the fabricator will initiate the procedure to mitigate risk. Other factors affecting fit-up are loading conditions and camber requirements.

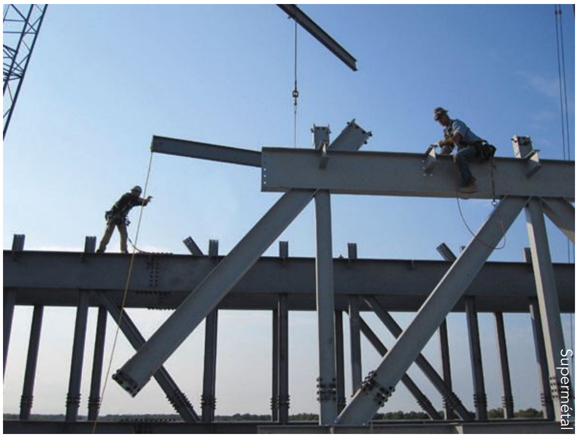
7. Select splice location to optimize the handling of the components. Overhead crane capacity in the shop, size and weight limitations during transport and crane access on-site can all impact on the ideal location of splices. A collaborative effort is needed to determine the optimum location, which may be a stick-built construction, a modular approach, segments of an otherwise continuous component, nodes separated from other components or frames or a combination thereof. In some situations, it is necessary to reduce the weight of the components by using high-strength steel and compact sections or by field welding the splice.

8. When evaluating a column splice for erection purposes, an essential consideration is safety. OSHA 1926 Subpart R – "Safety Standard for Steel Erection" has some specific requirements to consider for column splices. 1926.756 "Beams and columns Paragraph (d) Column Splices" states: "Each column splice shall be designed to resist a minimum eccentric gravity load of 300 lb (136 kg) located 18 in. (46 cm) from the extreme outer face of the column in each direction at the top of the column shaft." This is a part of the regulations to account for a 200-lb (91-kg) ironworker wearing a tool belt that can weigh 70 lb (32 kg) standing on a spud wrench in a hole at the top of the column, creating a moment at the splice joint.

9. The column splice should be conservatively located 5 ft (1.5m) above the beam for better access. In addition, Subpart R 1926.756 (e)(1) states: "The perimeter columns extend a minimum of 48 in. (122 cm) above the finished floor to permit installation of perimeter safety cables prior to erection of the next tier, except where constructability does not allow." 1926.756 (e)(2) states: "The perimeter columns have holes or other devices in or attached to perimeter columns at 42 to 45 in. (107-114 cm) above the finished floor and the midpoint between the finished floor and the top cable to permit installation of perimeter safety cables required by § 1926.760(a)(2), except where constructability does not allow." However, the 4-ft (1.2-m) column extension limit is sometimes not enough since it can cause an interference between the splice plates and the holes for the safety cable. For constructability reasons, it is recommended that the column splice be conservatively located at 5 ft (1.5 m) above the top of the beam.

Tip 4 – Consider end-plate connections for compression members other than columns.


Carol Drucker


Supermétal

Splice design requires strategy and collaboration as it has the potential to greatly impact fabrication, erection time and cost. Getting the splice right matters.

Tip 3 – Provide a clean column line at the splice.
Tip 12 – Take into account splice alignment and temporary support during field welding.


Supermétal

Tip 6 – Weld complex nodes in the shop and bolt on-site as much as possible. The joint shown in contact bearing was machine finished. All field joints were bolted. All shop welds were CJP.
Tip 7 – Select a splice location to optimize the handling of the components. High-strength steel was used to make the node lighter in this 2-story-tall transfer truss at the bottom of a 50-story building.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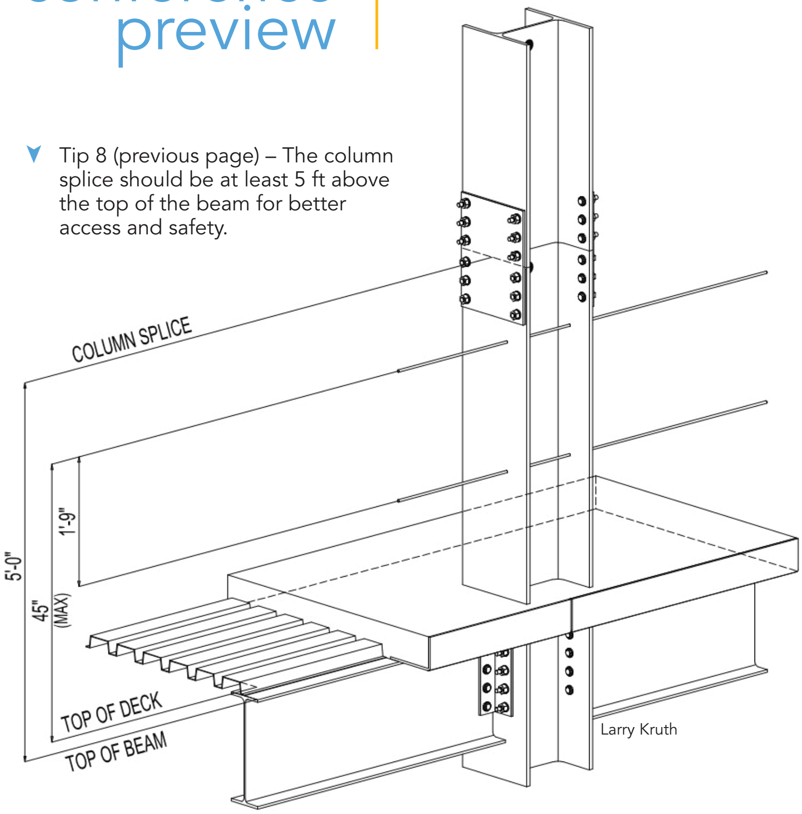


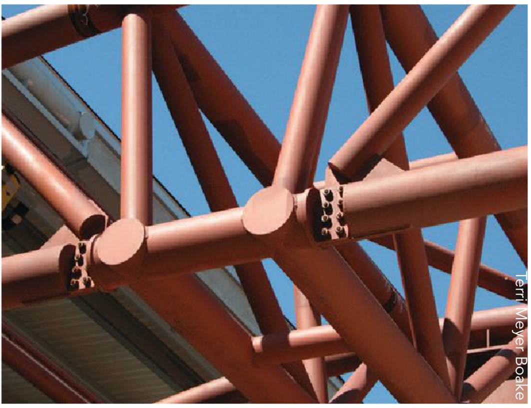
Larry Kruth

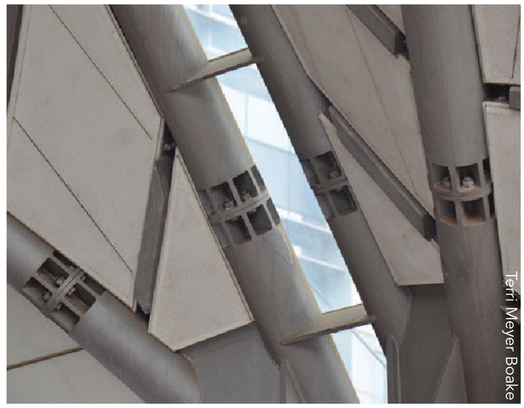
Tip 8 (previous page) – The column splice should be at least 5 ft above the top of the beam for better access and safety.

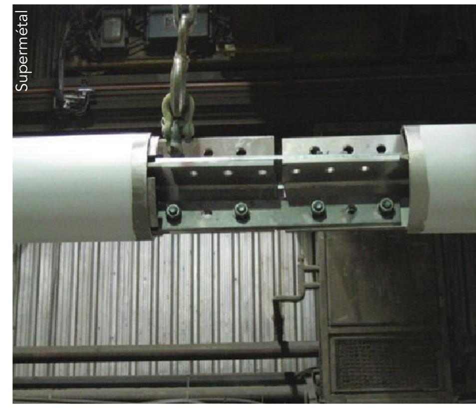
10. For welded splices, make it direct. The ideal welded splice is direct: shape-to-shape or tube-to-tube. No gussets, knife edges, flanges or lapped plates; a directly welded splice made with groove welds (complete or partial, depending on the loading conditions) makes the splice right. Direct-welded connections fulfill architect Louis Henry Sullivan's objective of "form ever follows function" in an economical and efficient manner. Field welding can be used to reduce the weight of the splice to satisfy specific loading requirements or aesthetic criteria.

11. Optimize the details such as backing, access holes and tabs. "The devil is in the details" and welded splices are no different. Such details include backing, weld access holes and weld tabs. Left-in-place steel backing must not introduce stress concentrations. Properly sized and prepared weld access holes can be used to reduce constraint in welded splices. Weld tabs enable quality welding but may impact performance of the splice in service. Certainly, left-in-place weld tabs detract from the appearance of splices.

Tips 14, 15 and 16 – When AESS is close to view, reduce the visual impact of the splices as much as possible.







12. The splice must be spliced; field-welded connections require special attention. Splices may be easily designed and detailed, but eventually splices must be spliced (i.e., fabricated and erected). For field-welded connections, the "splicing system" must consider splice alignment, temporary support, access for welding (including out-of-position welding) and when required, post-welding inspection. Collaboration between the architect, engineer, fabricator and erector is required to develop innovative "splicing systems" to make these seemingly simple splices in a cost effective and reliable manner.

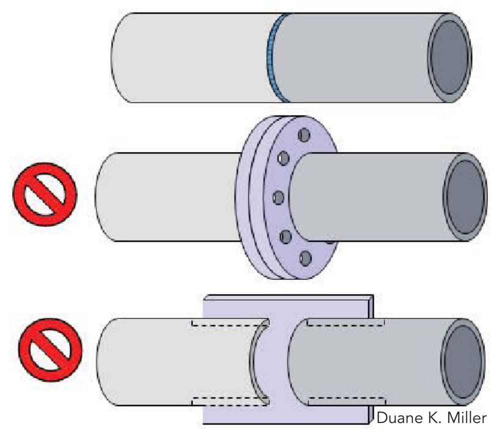
13. Leave welded splices in the as-welded condition. A properly made weld will have a visually pleasing appearance. Spatter and other small imperfections can be removed by localized grinding or chipping. However, complete removal of the weld reinforcement (crown) usually detracts from the aesthetics of the welded splice. Costly grinding operations usually create surfaces that are visually different than the as-received base metal condition, attracting more attention to the splice. Time and effort should be directed to making the initial weld correctly, along with a desirable appearance, as compared to a focus on removing all signs that the splice was made by welding.

14. Consider the distance to view when deciding on the splice detail on AESS members. As with all AESS, it is critical to consider the distance to view (greater or less than 20 ft—6 m) when deciding on the detailing and level of remediation required. When the installed steel has connections that are very close to view, it may be appropriate to fully remediate (grind) reinforcement from butt-welded connections. However, the same treatment for connections that are over 20 ft (6 m) away is typically unjustified and simply adds cost to the project.

15. Make the connections between large AESS members appear inconspicuous. The majority of connections used in AESS tend to enhance or make a design detail out of the act of connecting the steel. However, the reverse is normally the case when larger aggregated members require splicing, usually as a result of transportation and lifting limitations. The object here is to make the connections between the members appear unapparent.

16. For reduced visibility of large splices, consider hidden or discreet connections. If the intention is to simply make the splices appear less visible, given either the level of exposure of the AESS or budgetary restrictions, there are two other very effective cost-saving options:

- **Hidden connections:** To join HSS members, bolted connections can be hidden beneath specially designed cover plates that are shaped to match the form of the primary attaching members. Plates are attached to each end of the joining members.
- **Discreet connections:** Here, the bolted connection between the tubular members leaves the bolts visible, but the connection is designed in such a way as to reduce their visual impact and retain the trim visual lines of the joining members.



Tip 10 – When AESS requires a clean line, make it direct.

A splice is not an afterthought. A splice location and its design require careful consideration early in the process as well as collaboration between the architect, engineer, fabricator and erector. When the splice is right, all parties—including the client and the end user—benefit.

This article is a preview of Session N87 "The Splice is Right" at NASCC: The Steel Conference, taking place April 13-15 in Orlando. Learn more about the conference at www.aisc.org/nascc.

conference preview

NONBUILDING STRUCTURES AND NONSTRUCTURAL COMPONENTS

Knowing the differences between the two will enhance the design experience and outcome for both.

BY CHRIS KIMBALL, S.E., P.E.

STRUCTURAL ENGINEERING DESIGN typically falls under one of the following three categories: building structures, nonbuilding structures or nonstructural components.

While the majority of structural engineering encompasses building structures, most designers will also deal with nonbuilding structures and nonstructural components throughout their careers. And it's important to recognize the differences between the latter two categories.

ASCE 7: *Minimum Design Loads for Buildings and Other Structures* defines a *nonbuilding structure* as a "self-supporting structures that carry gravity loads that may be required to resist the effects of earthquake." *Nonstructural components* are defined as architectural, mechanical, plumbing or electrical components "that are permanently attached to structures." Oftentimes a component may actually be categorized as either a nonbuilding structure or a nonstructural component (see Figure 1).

Nonbuilding Structures

In addition to knowing which elements are nonbuilding structures and which are nonstructural components, it's also important to understand how both categories compare to building structures.

While nonbuilding structures are similar to buildings in terms of basic structural dynamics and ground motion hazards, they have different structural characteristics and performance objectives. In addition, the design of nonbuilding structures often requires additional considerations such as fluid dynamics and networked systems. Nonbuilding structures fall into one of two classifications: similar to buildings or not similar to buildings.

Nonbuilding structures meeting the first of these two classifications have structural systems that are designed and constructed similar to those for buildings in addition to having a similar dynamic response. Examples of nonbuilding structures that are similar to buildings include pipe racks, steel storage racks, power generation facilities and towers for tanks or vessels. The design of these structures follows the requirements of Chapter 15 and applicable parts of Chapters 11, 12 and 14 of ASCE 7.

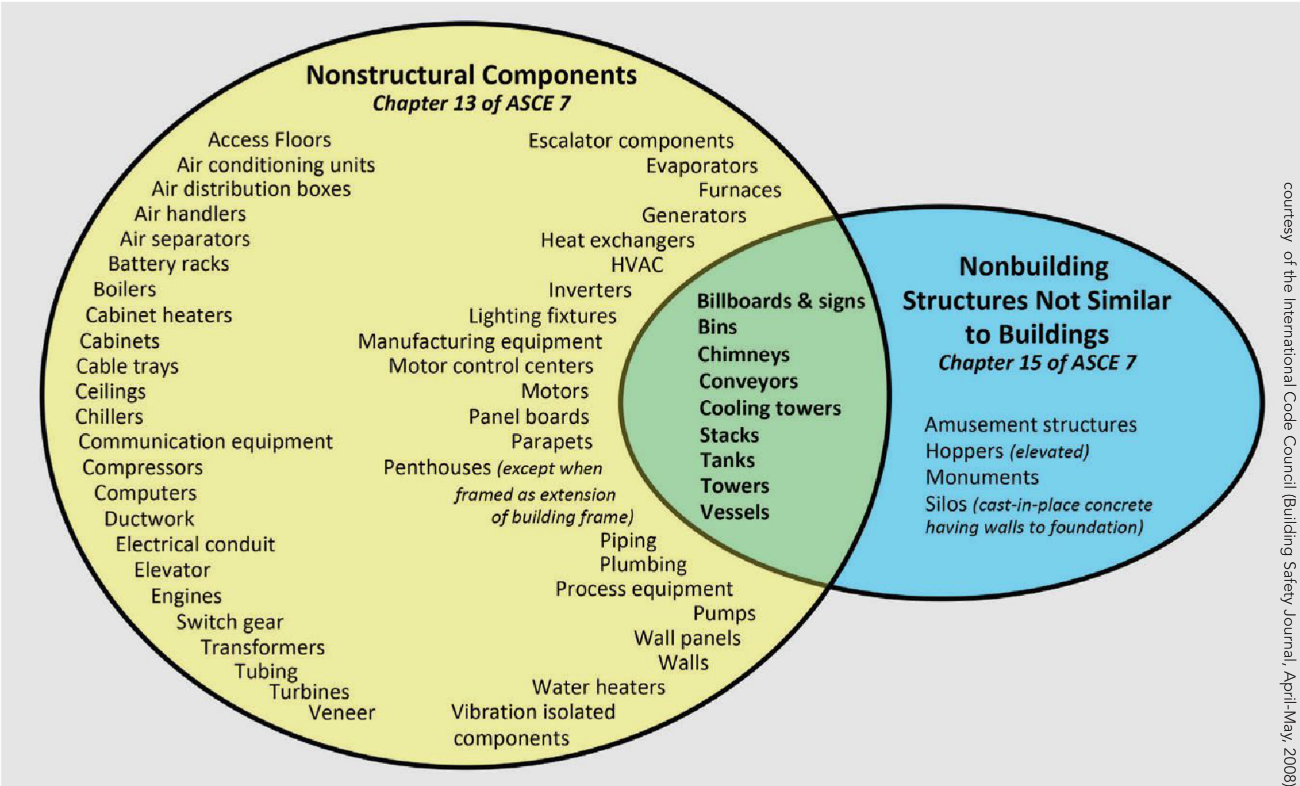
Examples of nonbuilding structures that are not similar to buildings include earth-retaining structures, tanks and vessels, telecommunication towers, stacks and chimneys. The dynamic response of these structures is different than that of buildings and often requires specialty design guides and standards. Chapter 15 of ASCE 7 provides direct references to many of these standards (e.g., API 620, API 650, AWWA D100, etc.), but regardless of what is obtained from these standards, the design loads shall not be less than what is provided in ASCE 7.

When it comes to design loads for nonbuilding structures, they must take into account the dead load of the structure in addition to the weight of the normal operating contents. While the drift limitations of Chapter 12 do not apply, a rational analysis must be performed showing that structural stability and attached components are not adversely affected. The approximate period provided in Chapter 12 of ASCE 7 cannot be used for nonbuilding structures, therefore requiring the period to be substantiated by means of analysis. In addition, the importance factor used in the design of nonbuilding structures should be the largest value obtained from applicable referenced standards, such as Table 11.5-1 of ASCE 7 or as elsewhere specified in Chapter 15 of ASCE 7.

Nonbuilding structures that are supported by other structures must meet the requirements of Section 15.3 of ASCE 7. In following this section, there are three possible outcomes:

1. If the nonbuilding structure weighs *less than 25%* of the combined seismic weight of the nonbuilding structure and the supporting structure, it should be considered a nonstructural component and designed in accordance with Chapter 13 of ASCE 7.

Chris Kimball (chrisk@wc-3.com) is the Utah Regional Manager for West Coast Code Consultants, Inc.



courtesy of the International Code Council (Building Safety Journal, April-May, 2008)

Figure 1. Intersection of nonstructural components and nonbuilding structures.

2. If the nonbuilding structure weighs *25% or more* of the combined seismic weight and has a period of *less than 0.06 seconds*:
   - The nonbuilding structure is considered a "rigid" element
   - Analysis shall use the *R* factor of the supporting structure
   - It shall meet the design requirements of Chapter 13 of ASCE 7 but use the response modification factor (R_p) obtained from Table 15.4-2 and an amplification factor (a_p) of 1.0
3. If the nonbuilding structure weighs *25% or more* of the combined seismic weight and has a period equal to or *greater than 0.06 seconds*:
   - The nonbuilding structure and the supporting structure must be modeled together
   - The nonbuilding structure shall use the lesser response modification factor (R_p) of the nonbuilding structure and the supporting structure
   - It shall meet the design requirements of Section 15.5 of ASCE 7

Nonstructural Components

Nonstructural components consist of architectural, mechanical, plumbing and electrical components that are permanently attached to structures. There is often debate as to who is responsible for the anchorage and restraint of nonstructural components. Is it the structural engineer, architect, mechanical engineer, etc.? Chapter 12 of FEMA 389 discusses how a unified approach can be used by the design team to ensure that a scope of work is provided for each element thus clarifying who in the design team is responsible for that item. FEMA 389 also includes sample checklists to aide in this endeavor.

Concerning forces and displacements, Chapter 13 of ASCE 7 provides clear guidance on how to calculate the seismic design force for each component. Separate response modification and component amplification factors are provided in Table 13.5-1 for architectural components and in Table 13.6-1 for mechanical, plumbing and electrical components. If the component has multiple connection points, the design must also take into account displacement that can occur within the structure or displacements that could occur between separate structures.

When not exempt by Chapter 13 of ASCE 7, the construction documents must show the seismic restraint requirements for nonstructural components and their supports and attachments. They shall also note the special inspection requirements for their installation per Section 13.2.7 of ASCE 7. In addition, Section 1705.12.3 of the *International Building Code (IBC)* requires that the design professional list on the construction documents the certification requirements for designated seismic systems. Section 202 of the *IBC* defines designated seismic systems as "those nonstructural components… for which the component importance factor is greater than 1". Chapter 13 of ASCE 7 then clarifies that the following conditions would result in an importance factor of greater than 1:

- The component is required to function for life-safety purposes after an earthquake
- The component conveys, supports or otherwise contains hazardous, highly toxic or explosive substances and can pose a threat to the public
- The component is in or attached to a Risk Category IV structure, and failure could impair the continued operation of the facility

In essence, designated seismic systems are those nonstructural components that are most likely to affect life-safety after a seismic event. Section 1705.11.4 of the *IBC* also requires that special inspection be provided for designated seismic systems.

When it comes to supports and anchorage, nonstructural components shall be "bolted, welded or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity." This anchorage shall be provided for not only the component but also its support. A good example would be of a rooftop unit. While appropriate anchorage may be provided for the unit to the rooftop curb, the curb itself must also be properly anchored to the supporting structure. Per Section 13.4.2 of ASCE 7, anchors in concrete are to comply with Appendix D of ACI 318, anchors in masonry are to meet TMS 402/ACI 530/ASCE 5, post-installed anchors are to be prequalified for seismic applications and power-actuated fasteners resisting sustained tension loads or friction clips are not allowed in high-seismic regions.

A quick note on seismic design requirements: The current requirements for nonbuilding structures are found in Chapter 15 of ASCE 7 while the requirements for nonstructural components are located in Chapter 13 of ASCE 7. For nonbuilding structures such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances, nuclear reactors and piers or wharves, the seismic design requirements are not provided in ASCE 7 but rather in industry standards specific to the element type. ■

This article is a preview of Session N43 "Nonbuilding Structures and Nonstructural Components" at NASCC: The Steel Conference, taking place April 13–15 in Orlando. Learn more about the conference at www.aisc.org/nascc.

conference preview

STEEL JOIST FLOOR SYSTEMS: BEST PRACTICES

Where to look for guidelines and practical advice on using open-web steel joists in floor construction.

BY MICHAEL A. WEST, P.E., AND DAVID SAMUELSON, P.E.

OPEN-WEB STEEL JOISTS have been a popular option for floor framing in steel buildings ever since their introduction to the market nearly a century ago.

And there are multiple resources available to help guide their use in building projects. Listed below are multiple publications that address steel joist design, organized by topic.

Design considerations. Although not specifically codified, design for serviceability is an essential element of successful use of joists in floor construction. Two main aspects of serviceability design are the consideration of deflections and vibrations. Other aspects include the accommodation of mechanical, electrical and plumbing systems and the consideration of future adaptation of the structural framing for changes in use or occupancy, including changes in loads and the addition of floor openings. The *International Building Code (IBC)* provides design loads (uniform and concentrated) for floor construction in Chapter 16 and provisions for the use of open web steel joists in Chapter 22. Provisions for building height and area, with respect to occupancy and class of construction, are found in Chapter 5.

Constant shear joists. Constant shear joists (KCS joists) have published tabulated values for moment resistance and shear. In these joists, the resistance to shear, as the name indicates, is constant from joist end to joist end. KCS joists also have constant moment resistance end-to-end. "Special joists" can be supplied for specifically specified uniform, nonuniform and concentrated loads. Lastly, composite joists are listed in the Steel Joist Institute's (SJI) *First Edition Standard Specifications for Composite Steel Joists Catalog*.

Vibration. Design for serviceability has been facilitated by the publication of two AISC design guides: Design Guide 3: *Serviceability Design Considerations for Steel Buildings* and Design Guide 11: *Floor Vibrations Due To Human Activity*. Likewise, SJI has published Technical Digest No. 5 *Vibration of Steel Joist—Concrete Slab Floors*.

Fireproofing. Many buildings are not required to be fireproofed based on their height, area and occupancy. But when required, floors framed with joists can be fireproofed. SJI's Technical Digest No. 10 *Design of Fire-Resistive Assemblies with Steel Joists* provides useful guidance to engineers on this topic.

Adaptability. Joist framing can be adapted when required by changes in use and occupancy. SJI's Technical Digest No. 12 *Evaluation and Modification of Open Web Steel Joists and Joist Girders* provides tools and techniques for doing this.

New Tools

SJI also provides free online tools to assist structural engineers with joist design (visit **steeljoist.org** and click the Design Tools link). These tools include Roof Bay Analysis, Virtual Joists, Virtual Joist Girders and Joist Girder Moment Connection Design.

The Floor Bay Analysis tool is expected to be available in May. This tool can optimize the layout and select optimal depths for non-composite K-Series, LH-Series and DLH-Series joists and joist girders as well as the CJ-Series composite joists. Common steel floor decks can be selected from a drop-down box and the tool determines the maximum deck span, available steel deck total uniform load carrying capacity and estimated deck deflection due to the weight of the wet concrete.

A summary for the joist design is provided, including available joist total uniform load carrying capacity and predicted live load deflection. Joist girders are summarized by available panel point concentrated loading and estimated live load deflection. The user has the option to input unit cost data for the concrete, slab reinforcement joists, joist girders, welded shear studs and bridging. They can save a "Run Summary," change one of the floor design parameters and see what effect this changed parameter has on the floor total cost per square foot.

Michael A. West (**mwest@csd-eng.com**) is a vice president with Computerized Structural Design and **David Samuelson** (**dave.samuelson@nucor.com**) is a structural research engineer with Nucor New Products and Market Development, Vulcraft/Verco Group.

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In designing a lateral load resisting frame, the engineer must include the joist girders and joists at the column lines in the frame model, and must also provide end-moments and member end-forces to the joist manufacturer for incorporation in the final design. Historically, the coordination process has been hampered because the engineer would not know, at the time of modeling, the design properties for the joist girders or any special joists designed for loads other than uniform gravity loads tabulated in the SJI Standard Load Tables.

For joists and joist girders with nonuniform loading, concentrated loads of varying magnitude and/or externally applied local moments, SJI has recently developed the Virtual Joist and Virtual Joist Girder Tables. These virtual section properties are equivalent-beam section properties based on top and bottom chord material sizes commonly available from joist manufacturers.

The tabulated Virtual Tables do not necessarily represent the final joist or joist girder design. They do, however, yield reasonably close approximations of the final joist and joist girder chord area, effective moment of inertia and weight for use in the structural models.

With the recent incorporation of the Virtual Joist Girder Tables within RAM Structural System, RISA and SCIA Engineer, an engineer can readily determine the resulting joist girder weight and required effective moments of inertia in a lateral load resisting frame. Once approximate joist and joist girder depths are selected using the Virtual Tables, the engineer can specifies the joist/joist girder design using conventional nomenclature.

Today's structural engineers have at their fingertips an increasing number of codes, specifications, design guides and design tools to make the design of joists and joist girders more efficient than ever before. The above tools and others will be discussed at our NASCC session. ■

This article is a preview of Session N60 "Steel Joist Floor Systems Best Practices" at NASCC: The Steel Conference, taking place April 13–15 in Orlando. Learn more about the conference at ***www.aisc.org/nascc****.*

Steel Joist Institute references are available at ***www.steeljoist.org****. AISC design guides are free downloads for AISC members at* ***www.aisc.org****.*

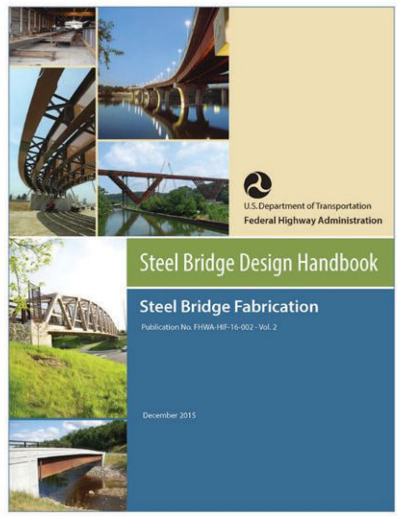
news

BRIDGES

New Edition of *Steel Bridge Design Handbook* Available

An updated version of the *Steel Bridge Design Handbook* is now available for free download. The *Handbook* includes 19 volumes (with topics ranging from design for constructability to corrosion protection) and six design examples that are current with the AASHTO-LRFD *Bridge Design Specifications*, Fifth Edition, U.S. Units.This new edition was developed by the National Steel Bridge Alliance (AISC's bridge division), the Federal Highway Administration and HDR Engineering.

The handbook can be downloaded for free at **www.steelbridges.org** and **www.fhwa.dot.gov**.



EVENTS

MIT to Host Architectural Iron and Steel Conference

The biennial "Architectural Iron and Steel in the 21st Century: Design and Preservation of Contemporary and Historic Architecture" conference will be held at the Massachusetts Institute of Technology (MIT) in Cambridge, Mass., April 2-3.

More than 35 architects, engineers and researchers will give presentations on the use, performance, maintenance and preservation/restoration of cast iron, wrought iron, structural steel, architecturally exposed structural steel (AESS), stainless steel and other steels used in new and existing buildings and monuments. The conference also includes an optional tour program on April 4, focusing on fabrication facilities and structures using these materials.

The conference is organized by Technology and Conservation, the MIT Department of Architecture's Building Technology program and the Boston Society of Architects' Historic Resources Committee. Visit **www.architects.org** for the full agenda and registration information.

MARKET NEWS

AISC's John Cross to Speak at Steel Markets Conference

AISC Vice President John Cross is slated to present at Platts' 12th annual Steel Markets North America Conference in Chicago, March 14–15. He'll be speaking at the session "Steel Demand Outlook in North America—Key Sectors" on March 14 at 2 p.m., and will discuss the current level of nonresidential and multistory residential building construction in the U.S., the intensity of steel usage in that construction, anticipated future trends in the building construction market and the impact those trends will have on steel demand.

The conference provides insight into North American steel market trends and covers such topics as economics, pricing, markets, export trends, logistics, supply chain and other issues impacting the U.S., Mexico and Canada. Visit **www.platts.com/events** for more information and to register.

(And see Cross' article "How Long Will the Good Times Last?" from our January issue, available at **www.modernsteel.com**, for a look at the construction market in 2016 and beyond.)

People and Firms

Lehigh University recently received a grant of $5 million to participate in a national initiative aimed at improving the resiliency and sustainability of the nation's civil infrastructure to better withstand the effects of earthquakes and other natural hazards. The five-year award from the **National Science Foundation (NSF)** is allowing Lehigh to perform research at its Advanced Technology for Large Structural Systems (ATLSS) Research Center.

According to **James Ricles**, Lehigh's Bruce G. Johnston Professor of Structural Engineering and principal investigator on the Natural Hazards Engineering Research Infrastructure (NHERI) program (which organizes the initiative), ATLSS Center was chosen for its ability to conduct real-time, large-scale and multidirectional structural experiments that mimic the demands on structures from natural disasters.

Other types of large-scale experiments have been performed at the center as well and will help NHERI researchers evaluate and confirm the resiliency of structural designs. These experiments include hybrid simulation, geographically distributed hybrid simulation, real-time hybrid earthquake simulation, dynamic testing and quasi-static testing.

Lehigh recently held a workshop at the facility, which comprised information sessions, tours and hands-on demonstrations tailored to attract applicants to develop research proposals to use the facility, and attracted registrants from 16 different universities.

The NHERI program also provided funding to six other universities for various types of research: Florida International University, Oregon State University, the University of California–Davis, the University of California–San Diego, the University of Florida and the University of Texas at Austin.

NASCC

Meet the Invisibles at NASCC

Who are the invisibles? That's one of the questions NASCC: The Steel Conference keynote speaker David Zweig will answer in his session at this year's show, which takes place in Orlando, April 13-15.

Zweig is the author of *Invisibles*, a new book that examines the skilled professionals in your organization who often go unseen or unrecognized—even though they're vital to your company's success. And he'll discuss why—and how—identifying and nurturing invisibles is so beneficial to your organization.

Driven by narratives of invisibles in action in a wide range of fields—from an interpreter at the United Nations to a cinematographer on a multimillion dollar film set to a structural engineer at the world's second-tallest skyscraper—and fortified by business and psychological research, Zweig explains the critical and often misunderstood, undervalued role invisibles play within organizations. Knowing how to spot, hire, retain and reward invisibles is essential for those concerned with optimizing their organizational culture.

You can register for NASCC, as well as view the exhibitor list and descriptions of the dozens of sessions at the show, at **www.aisc.org/nascc**.

UNIVERSITY RELATIONS

Registration Open for Student Steel Design Competition

Registration is now open for the 16th annual ACSA/AISC Steel Design Student Competition for the 2015–2016 academic year. The program challenges architecture students, working individually or in teams, to explore a variety of design issues related to the use of steel in design and construction. A total of $14,000 in cash prizes will be awarded to the winning students and their faculty sponsors.

Competitors may enter one of two categories. The "Tall Buildings" category challenges students to find alternative design approaches for tall buildings and create high-rise buildings that are inspired by the cultural, physical and environmental aspects of place. The "Open" category gives students flexibility to select a site and building program on their own. Both categories require steel to be used as the primary structural material, with special emphasis placed on innovation in steel design.

Individual students or teams participating in this year's competition are required to have a faculty sponsor, who must fill out the registration form by March 30, 2016. There is no fee to enter. For more information, please visit **www.aisc.org/studentdesign**.

CORRECTION

The February article "Tempering Tremors" erroneously credited multiple photos to Skidmore, Owings & Merrill, LLP. The photos—top-right on p. 39, top-right and bottom-middle on p. 40, middle on p. 42 and top-right on p. 43—were actually provided by Schuff Steel (an AISC Member and Certified fabricator). *Modern Steel* regrets the error.

NSSBC

BYU to Host 2016 NSSBC

This year's National Student Steel Bridge Competition is set to take place May 27–28 at Brigham Young University in Provo, Utah. Sponsored by AISC and the American Society of Civil Engineers, the national competition brings together approximately 50 teams annually from the 18 ASCE conference competitions. Structured to simulate a real-world project, teams build a 1:10 scale model bridge to meet a particular challenge, which is different each year. There are six categories: lightness, efficiency, stiffness, economy, construction speed and display. Winners are picked for each category, and the team who performs the best in all of the categories is the overall winner (last year, the University of Florida team took home top honors).

To learn more about the competition, including reading the official rules, visit **www.aisc.org/steelbridge**. And learn more about the history of the competition in "Going National" on page 26.

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Peddinghaus F1170B CNC Plate Punching Machine, 170 Ton, Ext Tables, Fagor CNC, 30" x 60" Trvl., Triple Gag Head, 2005 **#19659**
Peddinghaus FPB1500-3E CNC Plate Punch with Plasma, 177 Ton, Fagor 8025 CNC, 60" Max. Width, 1-1/4" Plate, 1999 **#25161**
Controlled Automation BT1-1433 CNC Oxy/Plasma Cutting System, 14' x 33', Oxy, (2) Hy-Def 200 Amp Plasma, 2002 **#20654**
Peddinghaus Ocean Avenger II 1000/1B CNC Beam Drill Line, 40" Max. Beam, 60' Table, Siemens CNC, 2006 **#25539**
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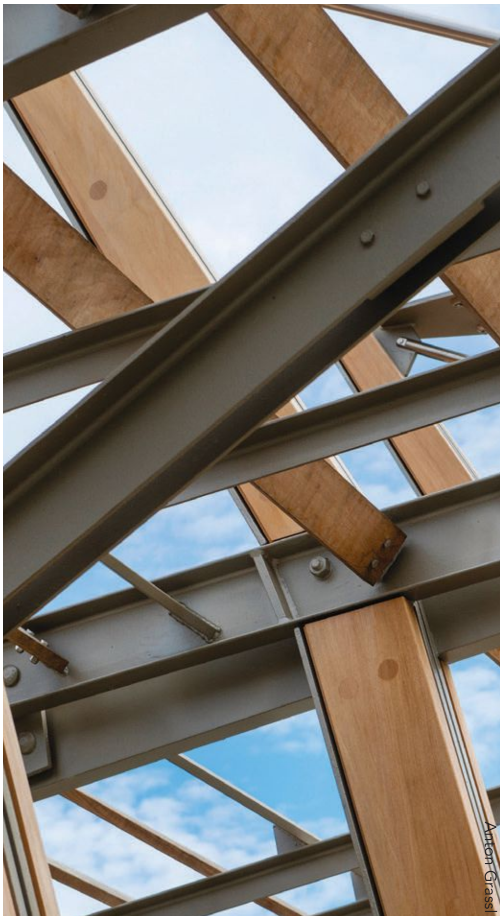


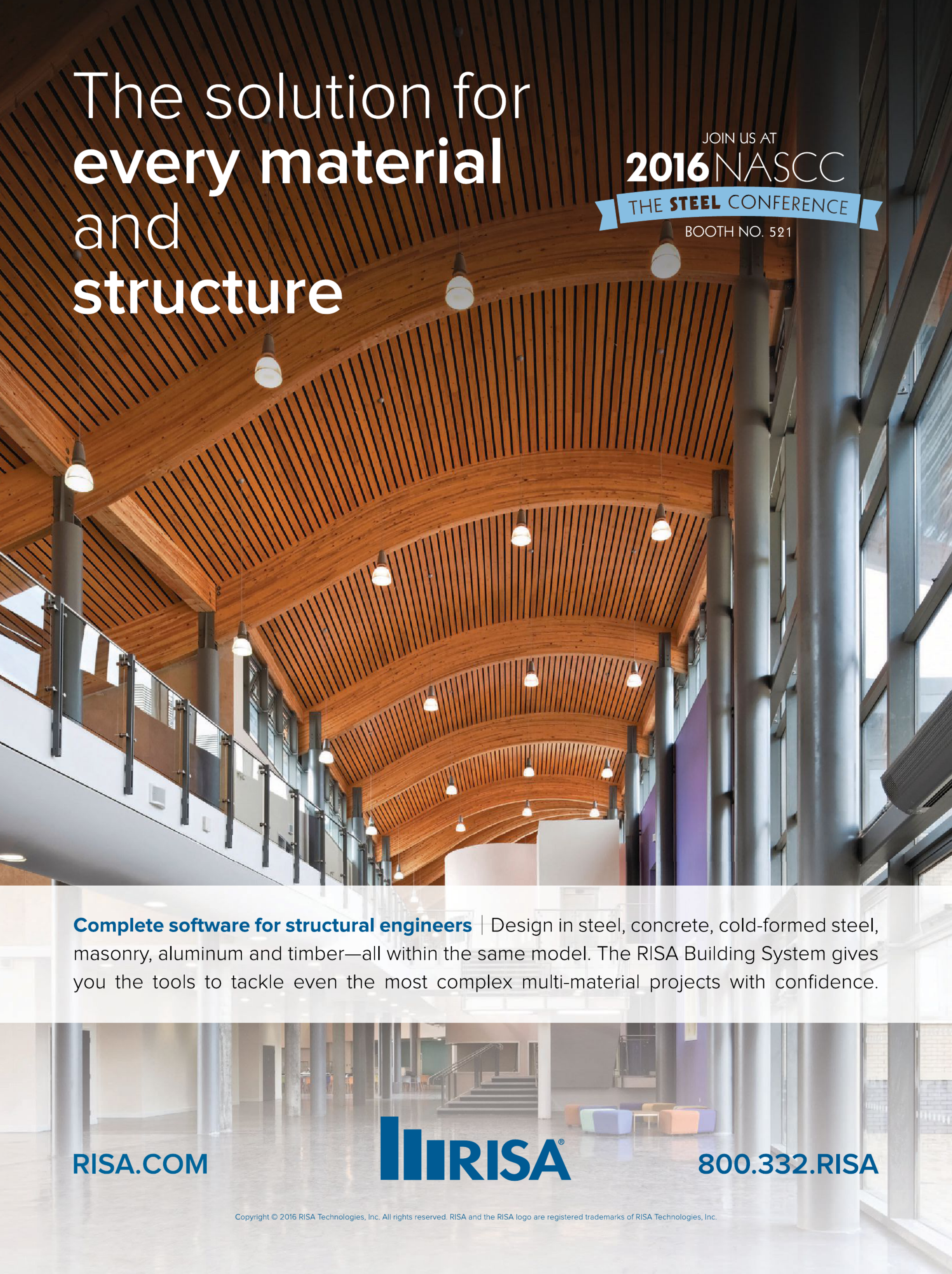
BRIGHTON COMMON PARK PAVILION is the center of attention in a small Boston green space.

Set in Brighton Common Park (also known as Brighton Square), the new pavilion and park transformed an underused public space into a new venue for events and performances.

The pavilion's form and detailing are derived from the rich rail yard history of the surrounding Brighton neighborhood, once the center of all livestock trade for Boston. Designed by architect Touloukian Touloukian, Inc., and structural engineer RSE Associates, Inc., the pavilion's wood and steel structure was designed to represent converging and diverging railroad tracks and the strong roof-line forms of historic railway stations.

The frame is composed of steel channels, angles and plate, and lateral stability is achieved through the use of ⅜-in. stainless steel cables and turnbuckles and 1¼-in. stainless steel rods that tie the whole primary frame together and allow the open roof framing to cantilever out over the stage area. As a complement to the framing system, an ipê wood skin was integrated into the steel members. The overall composition creates a balance of materials that work together to create an inviting atmosphere for visitors.

Additionally, solar studies drove the design of varying and alternating wood louvers and steel spacing in the roof framing to provide shade for the stage and seating area, as well as to facilitate direct sunlight for the climbing vines at the back of the structure. The result is a dynamic changing pattern of light and shadow.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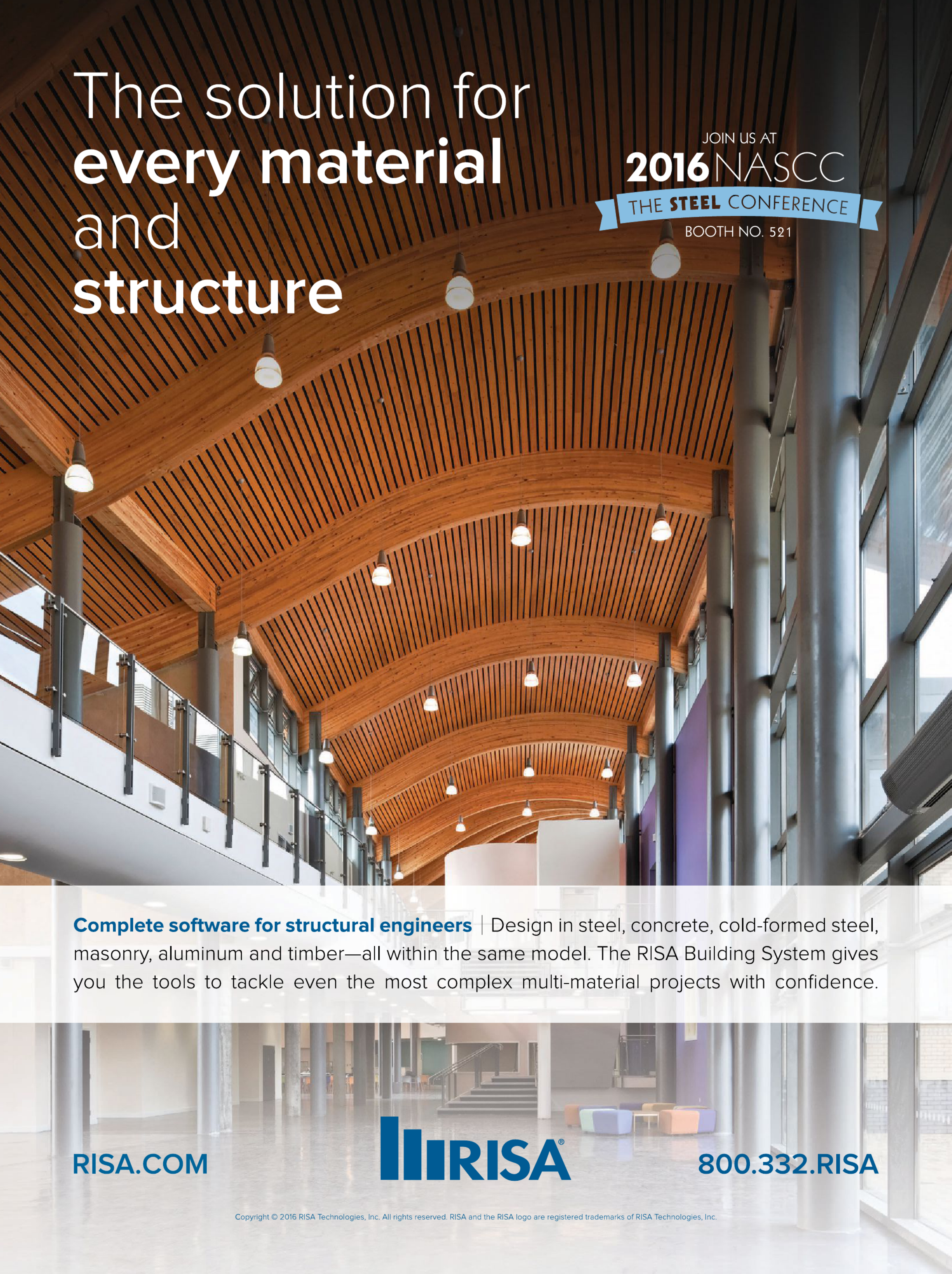
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