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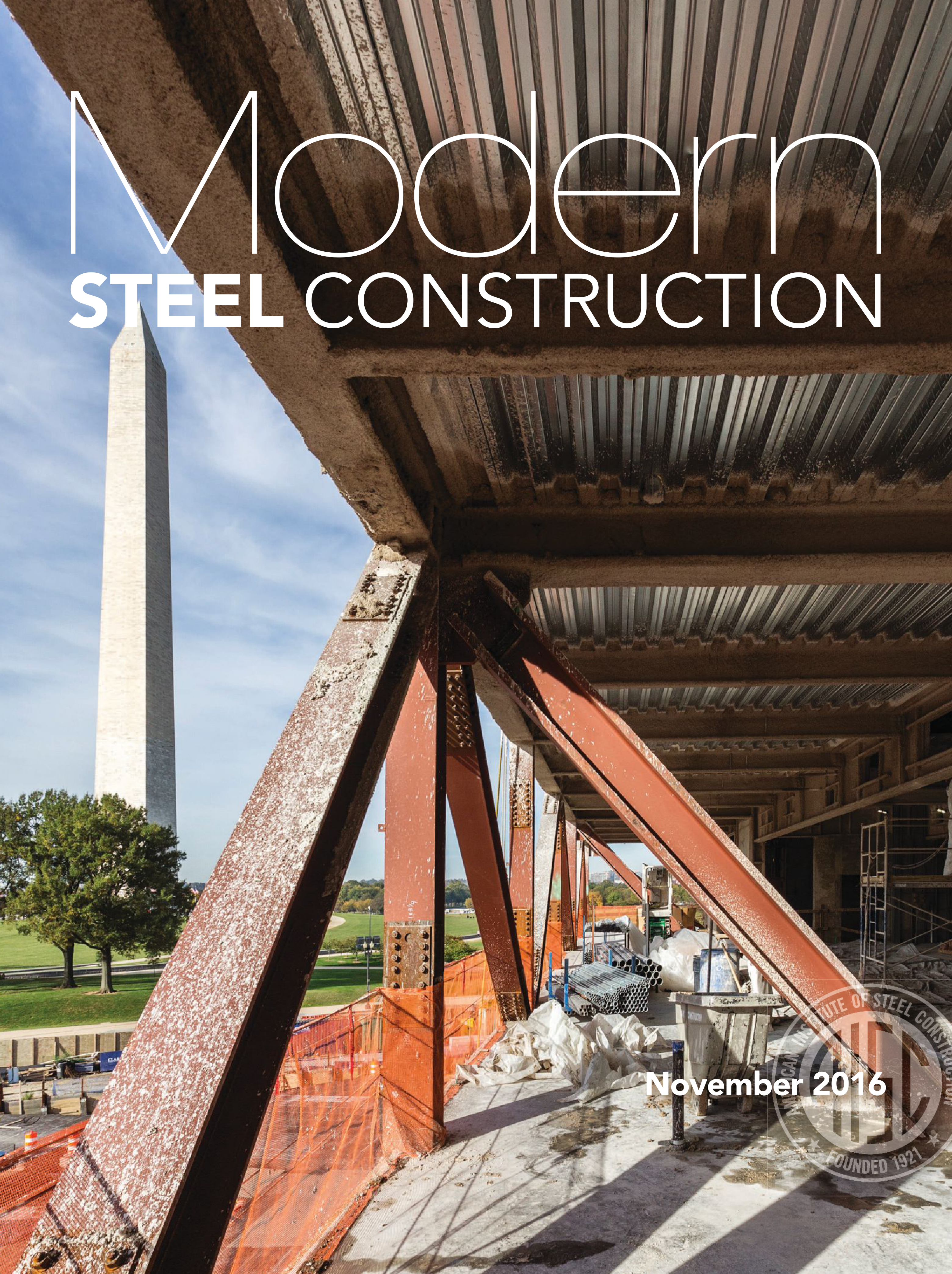

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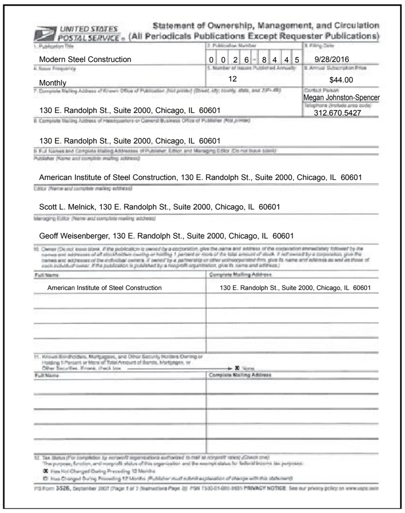

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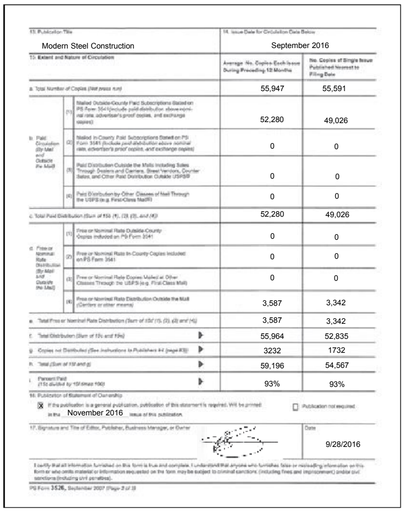

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ON THE COVER: Regarding a historic monument from a modern museum, page 30. (Photo: Richard Barnes)

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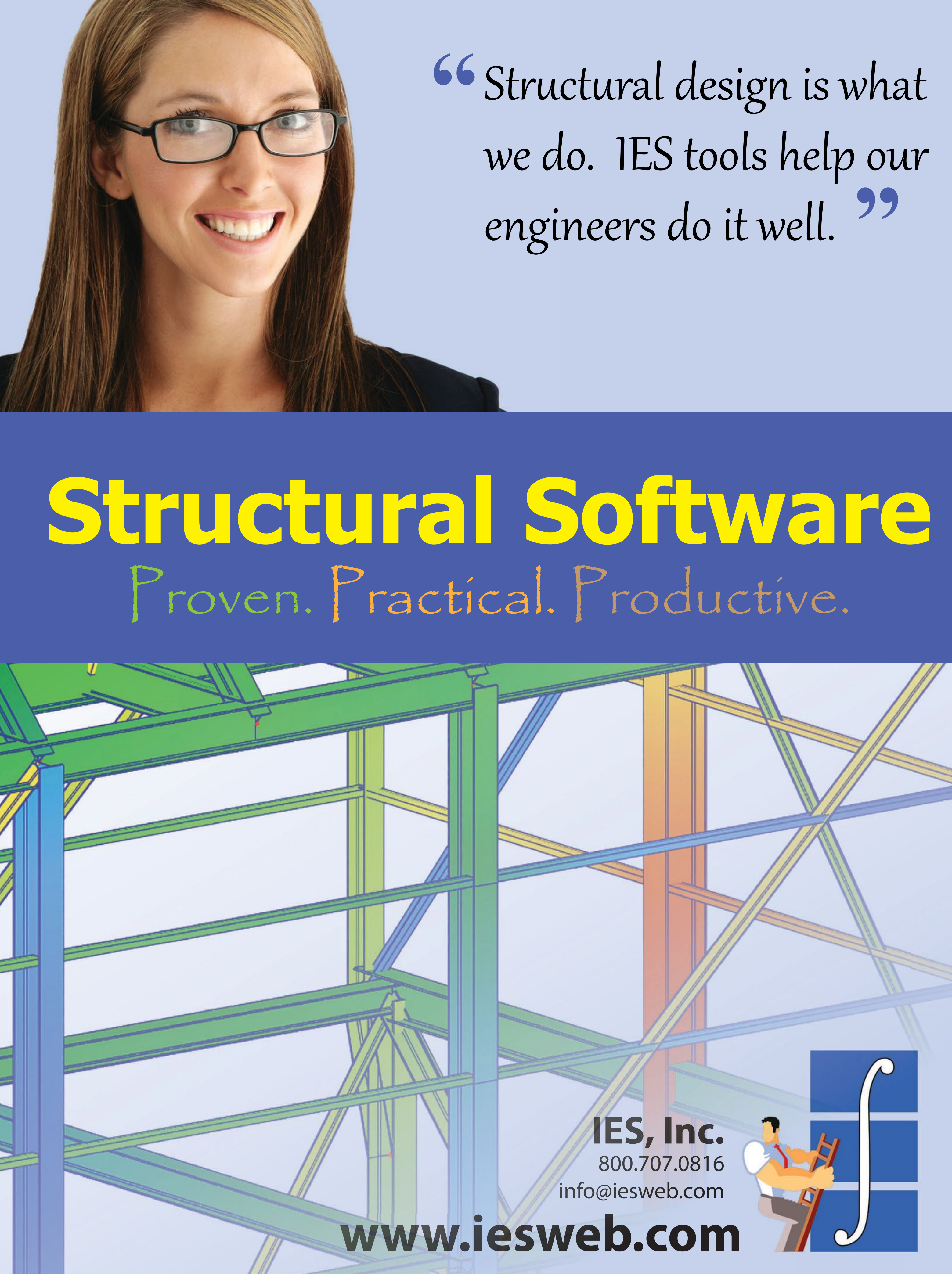
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editor's note

WAAAAAY BACK IN 1990, I ATTENDED A FULL-DAY SYMPOSIUM ON CONNECTION DESIGN RESPONSIBILITY. The speakers were on-topic, well-spoken and thought-provoking—until one speaker made a remarkably sexist off-hand remark. In the 1950s, no one would have thought twice. But this was a new era and the audience was markedly aware of the inappropriateness of the comment, and it made almost everyone uncomfortable.

In my naivety, I was certain the world had changed since those days. After all, just a few months ago my youngest son and I were listening to the radio when a story came on about a young female math prodigy who had almost given up on math because her teachers didn't think a girl could take the advanced classes she had signed up for. Jason ruefully shook his head at the report and responded succinctly: "That's stupid. Doesn't everyone know girls are better than boys at math?" His experiences, whether with girls on his hockey team or girls in his advanced math classes, were uniformly positive.

So when some AISC staff suggested a session at next year's conference (March 22–24 in San Antonio) on workplace diversity, I scoffed and wondered if such a session wasn't more relevant to a conference in the 1990s than in 2017. Still, there was enough interest that we added the program the conference—and even made it a session plus a networking luncheon; see page 12 of the advance program (included in this issue and online at **www.aisc.org/nascc**) for more details. And just last week, the need for such a session was hammered home when we received an email discussing a potential speaker and the writer of the email offered, as one of the speaker's qualifications, that she was "so dang cute."

Of course, we've made a lot of other changes in the conference for next year, starting with the schedule. In order to provide additional education opportunities, seminars will start at 8:30 a.m. on Wednesday rather than our traditional afternoon opening, giving attendees the opportunity to earn a total of 17 PDHs plus an additional 4 PDHs if they attend the optional Tuesday afternoon Short Course.

Other major changes include:

- With so many seminars running concurrently, it can be tough to select one that's right for you. So we've created tracks (or groupings) to help you find the seminars you're most interested in—but we don't require you to preregister for specific tracks, and we encourage you to explore a wide range of seminars.
- Sometimes, one session just doesn't go deep enough into a particular subject. So this year, we're offering two special tracks that provide a deeper dive into their respective topics. SSPC: The Society for Protective Coatings, has developed a four-hour program to provide bridge designers with valuable information on specifying coatings. A second special track is a series of related seminars on legal issues impacting the design and construction of steel buildings and bridges.

Registration opens December 12 and the cost for members that week is just $340—but the fee increases $10 every week, so be sure to register early!

See you in San Antonio!

SCOTT MELNICK
EDITOR

Modern STEEL CONSTRUCTION

Editorial Offices
130 E Randolph Street, Suite 2000
Chicago, IL 60601
312.670.2400

Editorial Contacts
EDITOR AND PUBLISHER
Scott L. Melnick
312.670.8314
melnick@modernsteel.com

SENIOR EDITOR
Geoff Weisenberger
312.670.8316
weisenberger@modernsteel.com

ASSISTANT EDITOR
Tasha Weiss
312.670.5439
weiss@modernsteel.com

DIRECTOR OF PUBLICATIONS
Keith A. Grubb, SE, PE
312.670.8318
grubb@modernsteel.com

PRODUCTION COORDINATOR
Megan Johnston-Spencer
312.670.5427
johnstonspencer@modernsteel.com

GRAPHIC DESIGN MANAGER
Kristin Hall
312.670.8313
hall@modernsteel.com

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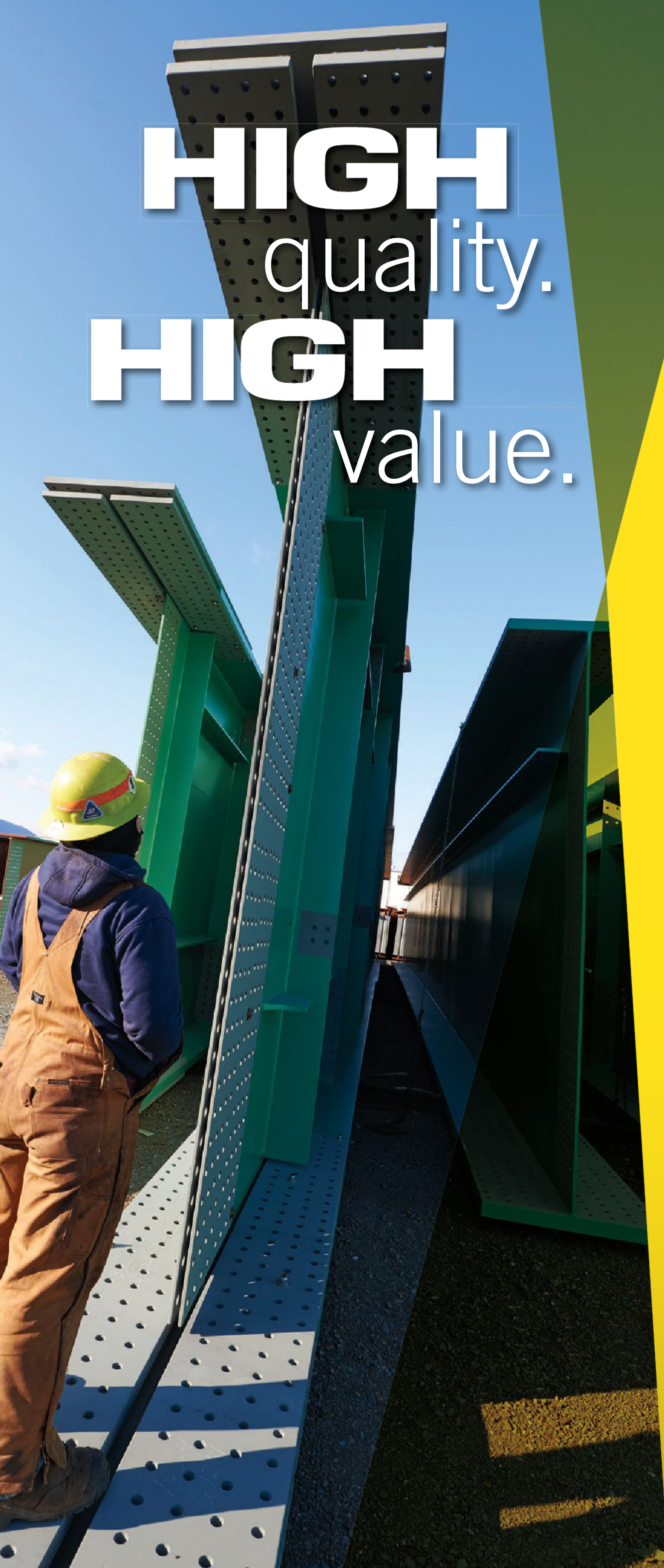
Advertising Contact
ACCOUNT MANAGER
Louis Gurthet
231.228.2274
gurthet@modernsteel.com

For advertising information, contact Louis Gurthet or visit **www.modernsteel.com**

Address Changes and Subscription Concerns
312.670.5444
subscriptions@aisc.org

Reprints
Megan Johnston-Spencer
312.670.5427
johnstonspencer@modernsteel.com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steel interchange

Composite Beams

A project on which we are installing shear studs specifies a composite steel system comprised of 2 in. concrete over 3 in. metal deck. Headed anchor studs, ¾ in. in diameter, are specified and noted to be a minimum of 1.5 in. above the deck and ½ in. below the top of the concrete. In an ideal situation, this can theoretically be achieved with 4⅞-in. studs that achieve 4½ in. of finished length. However, this only occurs where studs are installed through metal deck and ⅜-in. burn-through is theoretically achieved. At girders parallel to deck direction where the stud attaches directly to the girder flange, the theoretical burn-through is 3⁄16 in. and thus the finished length is 4 11⁄16 in. Both conditions run a high risk of being exposed when typical fabrication tolerances are considered (crown-up fabrication) even if there is no camber required. Section I3.2c of the AISC *Specification* has the following requirements: 2 in. minimum slab over deck, 1.5 in. minimum length above metal deck and ½ in. minimum of concrete cover to surface. Are there permitted deviations to this rule? Are two different stud lengths required in this situation?

The system you have described satisfies the requirements of the AISC *Specification* but, as you've noted, does not allow much room for tolerance. The specific provision in Section I3.2c(1)(2) states: "Steel headed stud anchors, after installation, shall extend not less than 1½ in. above the top of the steel deck and there shall be at least ½ in. of specified concrete cover above the top of the steel headed stud anchors." There are a couple of nuances within the wording here that are worth pointing out.

First and foremost, the 1½ in. minimum stud projection above the deck is structurally more important to the performance of the system than the ½ in. clear cover over the top. Purely from a strength perspective, the concrete cover over the top of the stud provides no recognized additional capacity. In the above referenced language, the phrase "specified concrete cover" was carefully chosen and deliberated over within the technical committee that maintains this section of the *Specification*. The intent is to ensure that designers specify a minimum of ½ in. concrete coverage to account for some of the field inaccuracies, but it was recognized that the in the final, as-built condition, the coverage could be less. The Commentary to this section of the *Specification* discusses ways an engineer can mitigate the potential for exposed studs in their slab system which are obviously more critical in a thin-slab system.

So, to answer your first question, it is acceptable to encroach into the ½ in. cover if necessary, but the 1½ in. minimum stud projection should be maintained.

As to whether or not two different stud lengths are required, I think that is a question that should be posed to the engineer of record. If the design specifies a uniform slab thickness of 2 in. everywhere, regardless of whether or not the final floor is level, then I would be inclined to say two different stud lengths are not necessary. However, if the design specifies a level floor finish then it is possible that if beam cambers do not come out, you could have exposed studs and that extra 3⁄16 in. of stud length could become very important.

Susan Burmeister, PE

Web Compactness for Singly Symmetric I-Sections

I am designing a singly symmetric I-shaped member in flexure. The plastic neutral axis for this section falls within the compression flange resulting in a negative value for $h_p/2$. How can I determine whether the web is compact, non-compact or slender? Note that if the web is not compact, then Section F4 of the *Specification* applies and since λ_p is equal to λ_r the denominators in Equations F4-9b and F4-16b become zero—again resulting in a result that is difficult to interpret.

Table B4.1b of the AISC *Specification* applies to compression elements of members subject to flexure. If $h_p/2$ is within the flange, then, under a plastic stress distribution, the web is in tension and therefore doesn't need to be classified. If $h_c/2$ is not within the flange, then, under elastic stress, some portion of the web will be subjected to a linearly varying compression load. In such a case, the magnitude of the compression stress will be relatively small when the section is elastic. As more and more of the section is strained beyond the elastic limit, the length of web in compression will decrease. Both of these trends tend to indicate that the stability of the web will not be a concern.

There are several possible approaches. First, the limits could be calculated based on Case 15, the doubly symmetric case, with the length of the web, h, assumed to be h_c. I believe this would be a conservative approach. The coefficient of λ_r is the same for the doubly symmetric and singly symmetric cases. Now consider the calculation of λ_p. If the equation for Case 16 is applied to a doubly symmetric I-shape h_c/h_p is 1.0. A reasonable value for the shape factor of a rolled wide flange is 1.12. This value produces a coefficient of 3.77—pretty close to the coefficient for Case 15, 3.76. So Case 16 produces about the same result as Case 15 assuming the same parameters.

There are two ratios that determine the value of λ_r for Case 16. The first is h_c/h_p. For a case like yours, with the larger flange in compression this ratio will always be greater than one. A negative value for hp does not make sense physically relative to checking the stability of the web. However, as h_p approaches zero, it can be seen that the value for h_c/h_p becomes very large. This again tends to indicate that buckling of the web becomes less and less of a concern. The other factor is related to the shape factor, Z_x/S_x, which is obviously in the same proportion as M_p/M_y. Up to a shape factor of about 2 the denominator will be less than one, tending to increase the coefficient. Beyond this shape factor, the coefficient will begin to decrease. Why should this be? The greater the shape factor, the more inelastic deformation will be required to fully yield the section. In other words, the demand becomes greater and greater. Also, at a shape factor of about 2, the shape is likely moving from a singly symmetric I-shape to something approaching a tee. It is interesting to note that there is no case addressing the web of a tee with the flange in compression. To me, this is another indication that at this extreme the stability of the web is not a concern.

This condition will be addressed in the Commentary to the 2016 *Specification*. The following statement has been added: "In extreme cases where the plastic neutral axis is located in the compression flange, $h_p = 0$ and the web is considered to be compact." This corresponds to the logic above.

If the web is compact, then Section F3, not Section F4, applies and a zero will not appear in the denominator.

I believe it is always appropriate (necessary!) to exercise engineering judgment. It is especially critical to do so when addressing conditions at the fringes of those considered in the *Specification*. It seems there are two different extremes that can cause the plastic neutral axis to be located in the compression flange. One would be where the compression flange is very clearly compact—i.e., it is very thick and relatively narrow. In such a case, it would seem the assumption that the web is compact is uncontroversial. At the other extreme, where the compression flange is very thin but very wide, I would be hesitant to treat the condition using Case 16. The distribution of stress typically assumed when calculating hc and hp might not be appropriate when the effective flange consists of a very thin but very wide element.

Larry S. Muir, PE

Not Qualified vs. Not Approved in ASTM F3125

The new ASTM F3125, which consolidates the previous ASTM A325, A490, F1852 and F2280 standards, indicates in Table A1.1 that F1136 coatings are not approved for use with twist-off bolts (Grades F1852 and F2280). It is my understanding that this indicates that these coatings are prohibited for use with twist-off tension-control bolts. Some vendors state that these bolt-coating combinations are not prohibited. What is the intent?

You are referring to an ASTM standard. Therefore ASTM would be the appropriate source for an interpretation. I will, however, provide my own opinion.

F3125 provides two different descriptions: not approved and not qualified. These terms are defined in the standard:

- "Not qualified" in Table A1.1 means that a particular coating has not been qualified and accepted by ASTM committee F16 for use on 150 ksi/1040 MPa bolts.
- "Not approved" in Table A1.1 means that a particular coating was not approved for a particular bolt style or grade in the individual standard prior to combination into F3125.

The reason for the different designations may not be immediately clear, since both would seem to discourage the use of the coating with the fasteners listed. However, the Annex also states:

> "Coatings listed in this Annex for 150 ksi/1040 MPa bolts have been qualified and approved where indicated for use with 150 ksi/1040 MPa strength bolts. For use on 150 ksi/1040 MPa bolts, other coatings must be qualified in accordance with IFI 144. Hydrogen embrittlement testing required by IFI 144 shall be performed in accordance with F1940 for internal hydrogen embrittlement and F2660 for environmental hydrogen embrittlement."

A footnote to the table in the Annex states:

> "Other metallic and nonmetallic coatings may be used on 120 ksi/830 MPa minimum tensile fasteners upon agreement between the purchaser and user. Performance requirements shall be specified by the purchaser and agreed to in writing. Coatings for 150 ksi/1040 MPa bolts must be qualified. See A1.1."

So the requirements for F1852 and F2280 are different. The standard does not prohibit any coating to be used with F1852 "upon agreement between the purchaser and user" with "performance requirements…specified by the purchaser and agreed to in writing." For F2280, coatings must be must be qualified.

The difference seems to involve hydrogen embrittlement. Galvanizing of A490 bolts (150 ksi) has been prohibited for some time. This is because the process can lead to hydrogen embrittlement, which can lead to failure of the bolt and is therefore a safety concern. The same concerns do not exist for 120 ksi bolts.

I take "not approved" as meaning that this combination has not been explicitly considered, but there is no reason to believe there is an inherent safety concern. Therefore, if you are going to do it you are on your own, relying on your own judgment and knowledge.

I take "not qualified" as meaning that there are known safety concerns with this combination, and it should not be used.

Larry S. Muir, PE

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Larry Muir is director of technical assistance at AISC. Susan Burmeister is a consultant 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.

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steel quiz

Steel Quiz made its first appearance in the November 1995 issue of *Modern Steel Construction*. This month's Steel Quiz takes a look at some of the best questions from 1998.

1 **True or False:** The permissible range of strain rate for use in tension testing of a steel coupon is specified in ASTM A370 Section 8.4.1. If the test is run at the maximum speed permitted therein, the resulting yield strength F_y will be greater than that which would be obtained had the test been run at the minimum speed permitted therein.

2 Why is pin bearing strength (AISC *Specification* Section J7) lower than bolt bearing strength (AISC *Specification* Section J3.10) for the identical standard hole size and material thickness (assume that edge distance is not a consideration)?

3 The increase in bolt tension due to deformation of the connected part is known as ________.

4 Torsional loading of a restrained shape of an open cross section produces three stresses. What are they?

5 Which limit states may govern the design of a steel beam?

6 **True or False:** All beams need to be checked for the limit state of lateral-torsional buckling.

7 **True or False:** A stick-out of two threads beyond the face of the nut is required for a properly installed bolt.

8 A column was designed assuming 50-ksi steel and the hypothetical wide-flange cross section chosen just met the width-to-thickness ratio limit, $0.56(E/F_y)^{1/2}$. Subsequently, it was discovered that the actual yield stress of the column supplied was 65 ksi, at which the section exceeds that width-to-thickness ratio limit. Must the designer reevaluate this column as a slender-element cross section?

9 Which of the following number of cycles is the threshold after which fatigue must be considered in design?
a. 5,000
b. 20,000
c. 100,000
d. 2,000,000

10 **True or False:** As the unbraced length of a compact-section beam is increased, the limit state that controls the design will change from yielding to inelastic lateral-torsional buckling to elastic lateral-torsional buckling.

TURN TO PAGE 14 FOR ANSWERS

steel quiz | ANSWERS

1 **True.** Unlike the tensile strength F_u, the yield strength F_y varies slightly with the range of strain rate that is permitted in ASTM A370.

2 In the latter case, the bolt head and nut provide confinement to the material that will undergo bearing deformations, which stiffens the material and increases the strength. However, a pin connected assembly usually does not benefit from the same level of confinement and the design strength is lower.

3 Prying action.

4 Shear stress due to pure (St. Venant) torsion, shear stress due to warping, and normal stress due to warping.

5 Beam limit states relate to strength, stability and serviceability, in flexure or shear. Those related to flexural strength and stability are: flexural yielding, lateral-torsional buckling, flange local buckling and web local buckling. Also related to strength and stability is the limit state of shear yielding and/or buckling. Serviceability limit states for beams are deflection and vibration. Kudos to you if you also recognize that beam connection limit states also matter! Extra kudos if you think about these as you design and select your beams!!

6 **False.** Lateral-torsional buckling is only applicable to beams bending about their major axis. Consequently, lateral bracing is not required for members loaded through their shear center and bending about their weak axis.

7 **False.** The RCSC *Specification* defines sufficient thread engagement as "having the end of the bolt extend beyond or at least flush with the outer face of the nut, a condition that develops the strength of the bolt."

8 No. Although the value decreases as yield strength increases, the critical stress for local buckling is a function of element slenderness, not the actual yield stress. Because this critical stress remains unchanged, local buckling still will not occur in the element at required strength.

9 **b.** Fatigue must be considered above 20,000 cycles of loading. This is stated in Appendix Section 3.1 of the AISC *Specification*.

10 **True.**

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**.

steelwise

WHAT'S NEW WITH PREQUALIFIED CONNECTIONS?

A look at the latest version of AISC 358.

BY MICHAEL D. ENGELHARDT, PhD, AND MARGARET A. MATTHEW, PE

WHEN IT COMES to the chronicles of steel construction, the concept of prequalified connections is recent history.

The first version of AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* was released in 2005, with subsequent major releases in 2010 and now in 2016. Supplements have been issued in between these versions as new prequalified connections were added to the standard.

So how did AISC 358 come to be in the first place? The impetus for its development goes back to damage observed to welded steel moment connections in the 1994 Northridge Earthquake. A major outcome of post-Northridge research was the view that any new steel moment connection detail intended for use in special moment frames (SMFs) or intermediate moment frames (IMFs) should have their seismic performance verified by testing at a realistic scale. AISC 341 *Seismic Provisions for Structural Steel Buildings* adopted performance criteria for SMFs and IMFs connections that required connections be capable of developing a story drift angle of ± 0.04 rad for SMFs and ± 0.02 rad for IMFs without significant loss in strength, and required that all moment connections demonstrate conformance with this performance requirement by conducting qualifying cyclic tests.

Prehistory

To facilitate and simplify conformance demonstration, the concept of "prequalified" moment connections was developed. Prequalified connections were first introduced in the SAC-FEMA program in report FEMA 350: *Recommended Seismic Design Criteria for New Steel-Moment Buildings*, which was released in 2000. Building on the recommendations in FEMA 350, the concept of prequalified moment connections was subsequently incorporated into AISC 341 in 2002. Paraphrasing the commentary to AISC 341, a prequalified connection is one that has undergone sufficient testing, analysis, evaluation and review so that a high level of confidence exists that the connection can fulfill the performance requirements for SMFs and IMFs. Recognizing that prequalifying moment connections require evaluation by a panel of knowledgeable individuals, AISC created the Connection Prequalification Review Panel (CPRP) in the early 2000s. The CPRP was assigned the responsibility to prequalify SMF and IMF connections. In addition to creating the CPRP, AISC also created AISC 358 as a new building standard for prequalified connections (the CPRP is responsible for development and updating of AISC 358).

Basic requirements that a connection must satisfy to become prequalified are specified in AISC 341, and the CPRP is guided by these requirements in prequalifying connections. In AISC 341, requirements for prequalification are specified in Section K1 "Prequalification of Beam-to-Column and Link-to-Column Connections." While there are numerous requirements, the most basic requirement for prequalification is the need for cyclic testing of the connection in accordance with AISC 341, Section K2 "Cyclic Tests for Qualifying of Beam-to-Column and Link-to-Column Connections"; it is not possible for a connection to become prequalified without large-scale testing. Further, Section K2 provides strict limits for extrapolating test results to larger member sizes, and the CPRP adheres to these size extrapolation limits in AISC 358.

The New 358

AISC 358-16 has 13 chapters and two appendices. Chapters 1 through 4 contain general requirements that pertain to all prequalified connections. Each subsequent chapter (5 through 13) contains requirements for specific prequalified connections. As new connections are prequalified, new chapters are added to the standard.

Michael D. Engelhardt (mde@mail.utexas.edu) is a professor at the University of Texas at Austin and chair of AISC's Connection Prequalification Review Panel (CPRP) and also serves on the Committee on Specifications, TC8 (Design for Fire) and TC9 (Seismic Provisions). **Margaret A. Matthew** (matthew@aisc.org) is a senior engineer with AISC, editor of *Engineering Journal* and secretary of the CPRP and the Bender/Roller Committee.

Table 1 provides a listing of prequalified connections in AISC 358 and the corresponding chapter for each connection. Several of the prequalified connections, identified with an (*) in the table, are proprietary connections. These are connections for which a patent is held, and designers must work with the patent-holder in the use of these connections in building construction projects. The rest of the connections are nonproprietary.

While the contents of each prequalified connection chapter varies somewhat, there are a number of common features of each connection:

- a general description of the connection and intended location of inelastic action
- systems for which connection is prequalified (SMFs and/or IMFs)
- prequalification limits (limits on beam and column sizes and types for which the connection is prequalified)
- additional prequalification limits (beam-column relationships, connection components, welding and bolting requirements, etc.)
- detailing and fabrication requirements
- design procedure

One of the most useful features of AISC 358 is that each prequalified connection has a detailed step-by-step design procedure, greatly facilitating the use of these connections.

Table 1 – Prequalified Connections in AISC 358-16

AISC 358-16 Chapter	Connection
5	Reduced Beam Section
6	Bolted Unstiffened And Stiffened Extended End-Plate
7	Bolted Flange Plate
8	Welded Unreinforced Flange – Welded Web
9	Kaiser Bolted Bracket*
10	ConXtech ConXL*
11	SidePlate*
12	Simpson Strong-Tie Strong Frame*
13	Double Tee

* proprietary connection

When AISC 358-10 was issued, there were five prequalified connections (Chapter 5 through 9). Since 2010, four additional prequalified connections have been added to the standard in Chapters 10 through 13. These recent additions to AISC 358 are briefly described below. The descriptions below highlight only a few key features of each connection. The reader is referred to AISC 358 for complete details, limitations and requirements.

ConXL Moment Connection

ConXtech ConXL is a proprietary moment connection that was first added in AISC 358-10 Supplement No. 1 in June 2011 (a schematic of the connection is shown in Figure 1). This system connects wide-flange beams to 16-in. concrete-filled square or built-up box columns using a completely field-bolted collar assembly. Reduced beam section (RBS) cutouts are provided in the beam flanges, as needed, to satisfy strong column-weak beam criteria. Portions of the collar assembly (collar corner assemblies) are shop-welded to the columns, and the remaining portions (collar flange assemblies) are shop-welded to the beam ends. In the field, the collar flange assemblies are bolted to the column corner assemblies. When the connection first appeared in AISC 358, both the collar corner assemblies and the collar flange assemblies were specified to be made of forged parts. Subsequently, in AISC 358-10 Supplement No. 2, the collar corner assemblies were specified to be made of cast or forged parts. Requirements for casting and forging are specified in Appendices A and B of AISC 358. Changes in AISC 358 for the ConXL connection will allow the flanges and webs of built-up box columns to be connected using partial joint penetration groove welds, previously specified as complete joint penetration groove welds. Requirements for welds are specified in Chapter 10 of AISC 358.

The ConXL moment connection is prequalified for use in SMFs and IMFs as well as in planar moment-resisting frames or in orthogonal intersecting moment-resisting frames. In typical applications, these connections are used to provide biaxial moment connections at nearly every beam-column joint in the structure, essentially providing a complete 3D moment frame system.

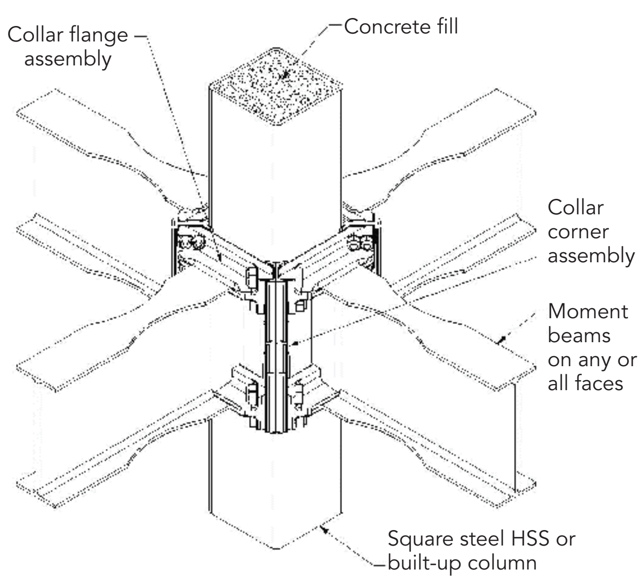
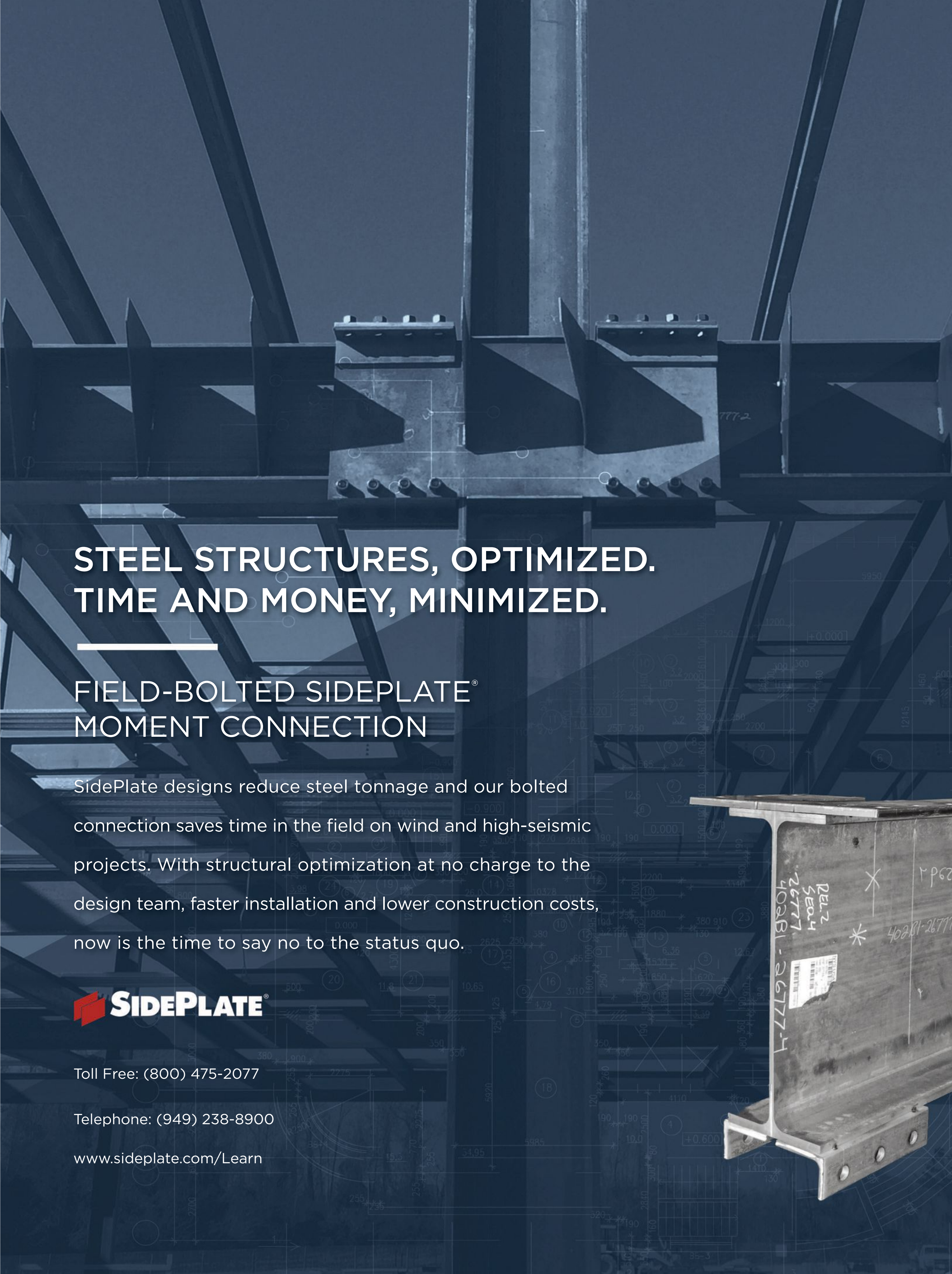


Figure 1. An assembled ConXtech ConXL Moment Connection. (Figure 10.1 from AISC 358-16)

The ConXL connection is prequalified for rolled wide-flange or built-up I-shape members up to W30 in depth, beam flange thickness up to 1 in. and beam flange width up to 12 in. Columns for this system are always 16-in. square HSS or 16-in. square built-up box sections. The columns are completely filled with structural concrete. Additional prequalification limits are listed in Chapter 10 of AISC 358.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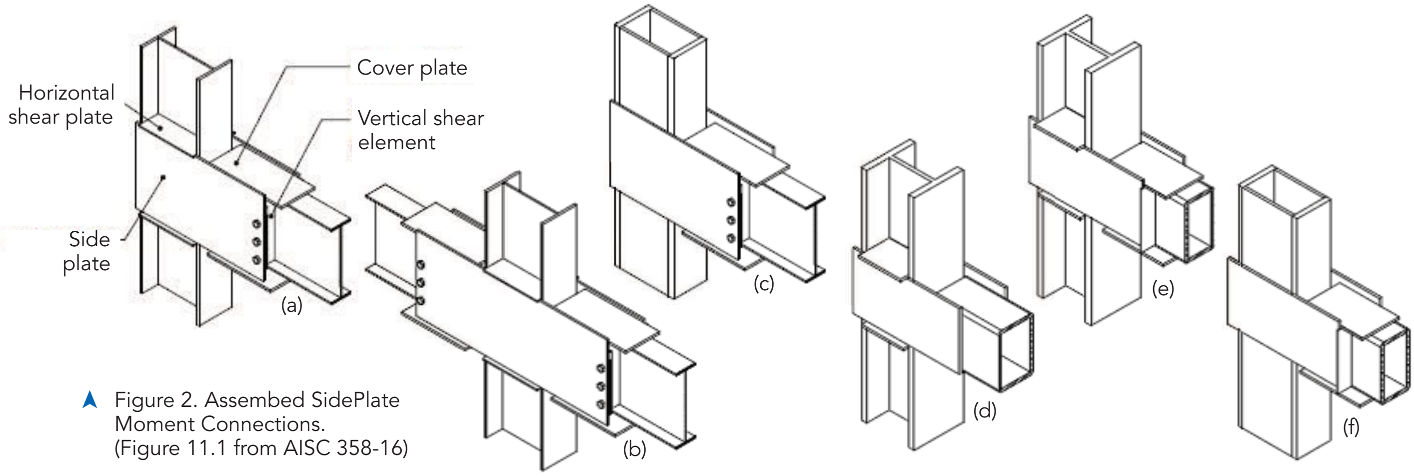

Figure 2. Assembed SidePlate Moment Connections. (Figure 11.1 from AISC 358-16)

SidePlate Moment Connection

The SidePlate moment connection is a proprietary connection that was first added in AISC 358-10 Supplement No. 2 in February 2014. (A schematic of the connection, when used with planar moment frames, is shown in Figure 2.) With this connection, the beam is not directly connected to the face of the column; there is, in fact, a gap between the end of the beam and the face of the column. Forces are transferred from the beam to the column through parallel side plates that sandwich the beam and column. The beam is welded to the side plates using horizontal beam flange cover plates and vertical shear elements, and the side plates are welded directly to the column flanges and welded to the column web through horizontal shear plates. Thus, forces at the beam ends are transferred to the side plates and then from the side plates into the column. A key feature of this connection is that all welds are fillet welds; the only exception is when HSS beams are connected to the side plates using flare bevel groove welds at the rounded edges of the HSS section. In addition to the planar moment frame SidePlate connections shown in Figure 2, the SidePlate connection can also be used in biaxial configurations where beams connect orthogonally to columns. SidePlate is prequalified for both SMFs and IMFs.

The connection is prequalified for rolled wide-flange or built-up I-shape members up to W40 in depth and beam flange thickness up to 2.5 in. SidePlate is also prequalified for HSS beams for depths up to HSS10 for SMFs and HSS12 for IMFs. Beam weight is limited to 302 lb/ft. Columns can be rolled wide-flange or built-up I-shaped sections up to W44 in depth, flanged cruciform sections or built-up box sections with a width not exceeding 24 in. Additional prequalification limits are listed in Chapter 11 of AISC 358.

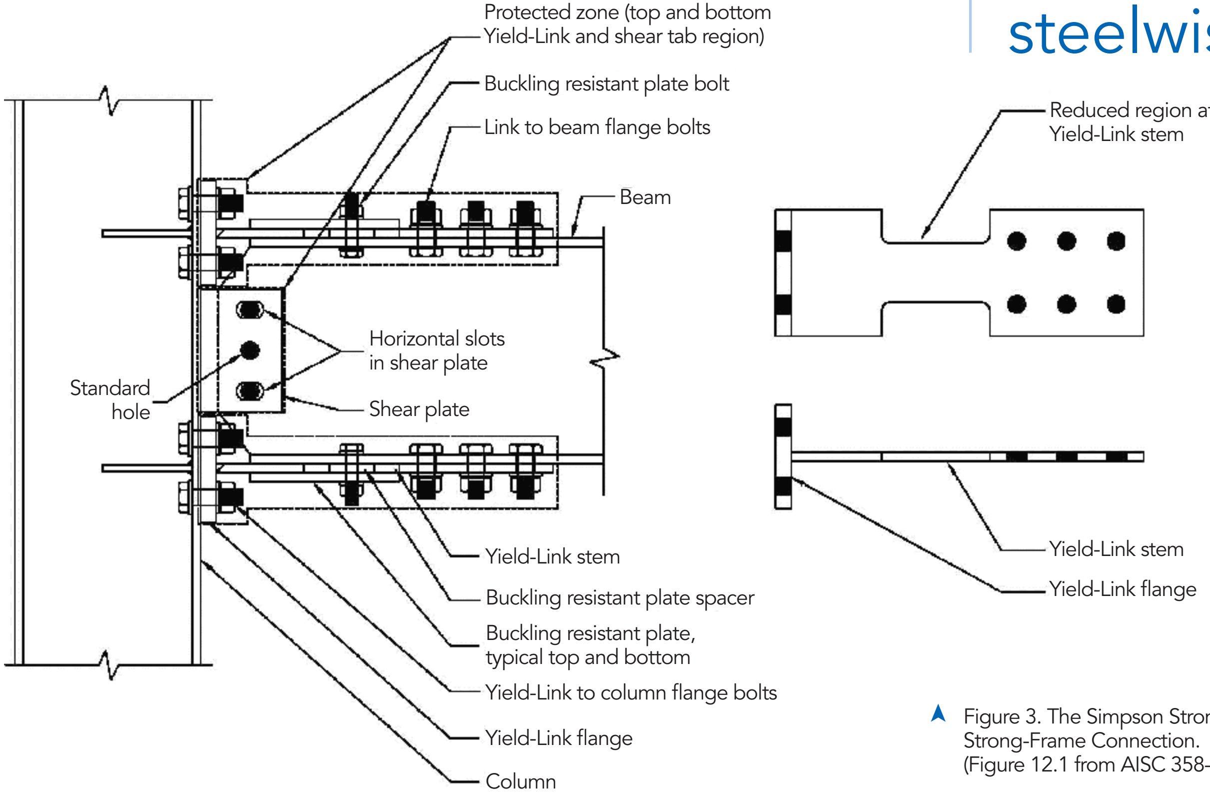

Figure 3. The Simpson Strong-Tie Strong-Frame Connection. (Figure 12.1 from AISC 358-16)

Strong-Frame Connection

The Simpson Strong-Tie Strong Frame connection is a newly prequalified proprietary connection in AISC 358 (a schematic of the connection is shown in Figure 3). The Strong-Frame connection is the first moment connection classified as partially restrained to be prequalified in AISC 358. The term "partially restrained moment connection" is defined in AISC 360 as "a connection capable of transferring moment with rotation between connected members that is not negligible." This means that the connection's rotation flexibility must be included in the structural analysis model used to predict frame drift and frame member and connection forces. Detailed guidance is provided in Chapter 12 of AISC 358 on how to include connection flexibility in the building frame model. In addition to being classified as partially restrained, the connection can also be considered a partial-strength connection, as the connection's flexural capacity is typically well below M_p of the connected beam.

A key feature of the Strong-Frame Connection is the "Yield-Link" used to connect the beam flanges to the column flange. The Yield-Link (shown in the right portion of Figure 3) is a modified T-stub that has a reduced section in the stem. The stem of this link is bolted to the beam flange, and the link's flange is bolted to the column flange. The connection is designed so that during a severe earthquake, yielding occurs within the reduced section of the Yield-Link and the beams remain essentially elastic, which differs from all other prequalified connections, where yielding is intended to occur primarily in the beams. When the stem of the Yield-Link goes into compression, buckling of the Yield-Link is prevented by a separate buckling restraint plate that is bolted over the reduced section of the Yield-Link. Also note that construction of the Strong-Frame connection in the field requires no welding; all components are field-bolted.

One of the challenges faced by the CPRP in prequalifying the Strong-Frame connection for SMFs applications is that it did not fit the definition of an SMF connection in AISC 341-10. According to AISC 341-10 Section E3.2, SMFs "are intended to provide significant inelastic deformation capacity through flexural yielding of the SMF beams and limited yielding of the column panel zone." The Strong-Frame connection did not meet this definition, as inelastic deformation capacity is provided through yielding of connection elements and not through yielding of the beam or column panel zone. Further, AISC 341-10 requires that beam-to-column connections, when subject to cyclic loading tests, demonstrate a measured flexural resistance of at least $0.8M_p$ of the connected beam at a story drift angle of 0.04 rad. Again, the Strong-Frame connection did not satisfy this requirement, as its flexural capacity is intentionally significantly less than M_p of the connected beam. Finally, AISC 341-10 provided no guidance on the use of partially restrained moment connections in SMF.

Because of the unique nature of the Strong-Frame connection, being both a partially restrained and partial-strength connection, the CPRP requested two supplemental studies to be completed as part of the prequalification process. These studies were intended to ensure that moment-resisting frames constructed with the Strong-Frame connection provide seismic-performance similar to more conventional SMF. To this end, comparisons were made between frames constructed with this connection and more conventional SMFs constructed with RBS connections. The first study examined connection performance equivalency following procedures specified in Report FEMA P795: *Quantification of Building Seismic Performance Factors – Component Equivalency Methodology*, which examined the equivalency of the Strong-Frame connection with the RBS connection. The second supplemental study requested by the CPRP examined overall system behavior—and again compared systems using Strong-Frame connections with more conventional SMF systems using RBS connections. A series of nonlinear time-history analyses using a suite of strong ground motions were conducted to compare the systems, and both supplemental studies indicated that frames constructed with Strong-Frame connections provided performance that is essentially the same as an SMF with RBS connections. These studies were a key part of the successful prequalification of the Strong-Frame connection.

The connection is prequalified for both SMFs and IMFs and is prequalified for rolled wide-flange of welded built up I-shaped sections; beam depth is limited to W16 or less. Because beams are expected to remain essentially elastic in this system, they are not required to satisfy the compactness limits or lateral bracing requirements for SMFs specified in AISC 341. Rather, the beams need only satisfy the compactness and bracing requirements in AISC 360. This reduction in beam lateral bracing requirements is useful when providing such bracing is difficult, as may be the case with a steel SMF that is integrated in a wood structure. The Strong-Frame connection is further prequalified for use with rolled or built-up column sections up to W18 in depth, and the beam must be connected to the column flange. Additional prequalification limits are listed in Chapter 12 of AISC 358.

It should be noted that following the prequalification of the Strong-Frame connection in AISC 358, changes were made to the SMF requirements in Section E3 of AISC 341-16. This section now permits SMFs where inelastic deformation capacity is provided by yielding within the connection "where equivalent performance of the moment frame system is demonstrated by substantiating analysis and testing."

Double-Tee Connection

The double-tee connection is a new nonproprietary connection in AISC 358 (a schematic of the connection is shown in Figure 4). This is an all-field-bolted connection wherein the beam flanges are connected to the column flange using T-stubs cut from rolled sections, and the beam web is connected to the column using a single plate shear connection. The double-tee connection was the subject of significant research and testing in the SAC-FEMA program. This work was subsequently extended with additional research and testing, leading ultimately to prequalification in AISC 358.

The connection is designed such that in the event of a strong earthquake, yielding occurs in the beam near the ends of the stems of the T-stubs. Like all other prequalified connections, with the exception of the Strong-Frame connection, the double tee is considered to be a fully restrained moment connection. This means that connection flexibility need not be included in the analysis model. As part of the design procedure for the double-tee connection in Chapter 13 of AISC 358, equations are provided to check that the connection has adequate rotational stiffness to ensure that it behaves as fully restrained.

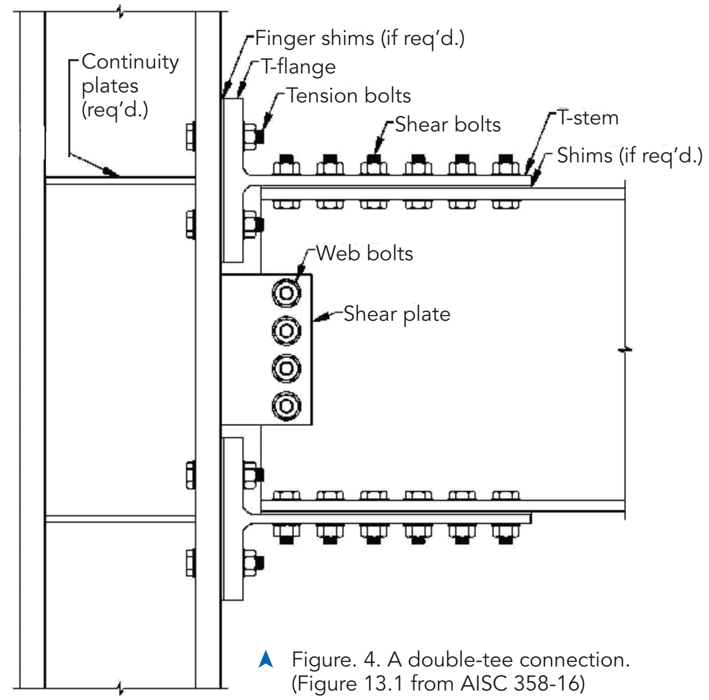

Figure. 4. A double-tee connection. (Figure 13.1 from AISC 358-16)

The double-tee moment connection is prequalified for use in SMFs and IMFs and for rolled wide-flange or welded built-up I-shaped members. Beam depth is limited to a maximum of W24, beam weight is limited to a maximum of 55 lb/ft and beam flange thickness is limited to a maximum of ⅝ in. Columns can be rolled wide-flange shapes, welded built-up I-shapes or flanged cruciform columns. Column depth is limited to W36 when a concrete structural floor slab is present and is otherwise limited to W14. Beams must be connected to the column flange. Additional prequalification limits are listed in Chapter 13 of AISC 358.

More Choices

With these four connections that have been prequalified since the release of AISC 358-10, there are now a total of nine prequalified connections in AISC 358, providing designers with many choices for beam-to-column moment connections in SMFs and IMFs. As we've provided only a few brief highlights here, we urge you to peruse the new AISC 358 for complete details and requirements for these and other prequalified connections. And remember that designers are not restricted to only using prequalified connections in SMFs and IMFs. AISC 341 also permits connections in SMFs and IMFs that have been qualified by testing but that have not necessarily been prequalified. Nonetheless, AISC 358 can be a valuable resource for designers by providing prequalified connections that are deemed to satisfy the connection performance requirements for SMFs and IMFs in AISC 341.

AISC 358-16 is expected to become available this month and will be posted at **www.aisc.org/epubs** in the "Specifications, Codes and Standards" section.

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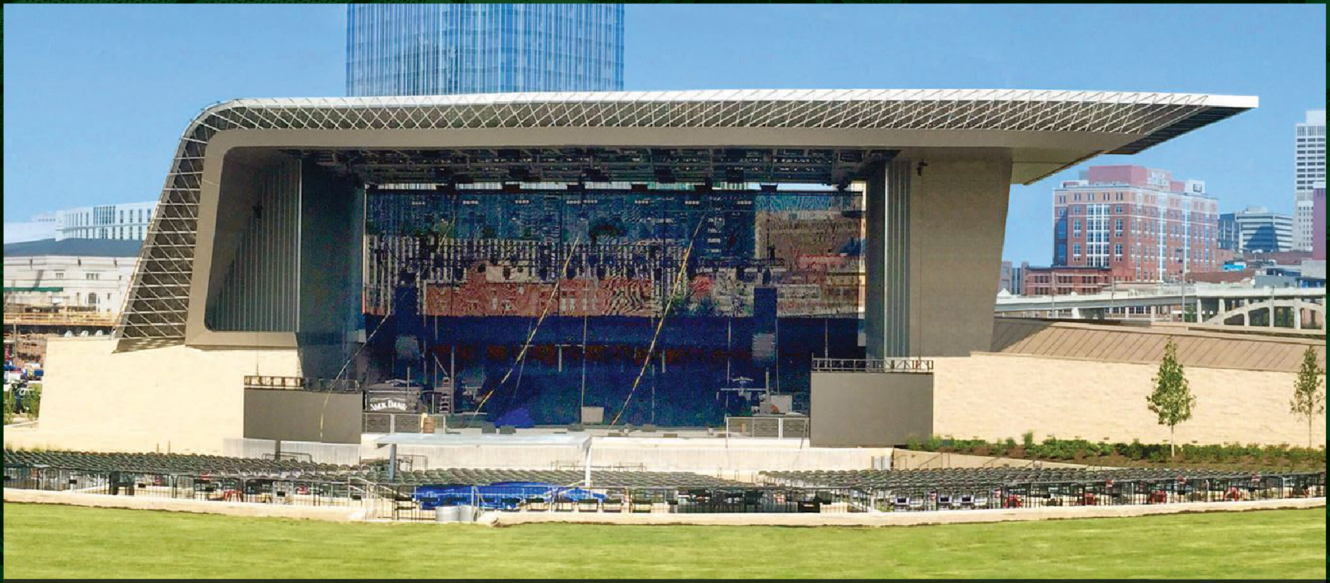
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Adopting a millennial mindset—or at least understanding it—is the key to selling to this new generation of buyers.

business issues

MILLENNIAL MINDSET

BY ANNE SCARLETT

CONSIDER THIS SCENARIO: Through your network, you uncover a hot lead. A local community college has completed its master plan for several new buildings and will now begin hiring a variety of firms (to spread the work!) for project execution.

Before picking up the phone to call the campus architect, you do your due diligence—general internet research, degrees of separation on LinkedIn, memberships within relevant organizations and so forth. As you put together the pieces, building up a "profile" for this prospective client, you suddenly discover that he is young—as in 30-years-young.

Whoa. Really? A "kid" in his early thirties is a decision-maker for this community college?

You think to yourself, "I must be 15 years his senior! Will I be able to successfully connect with someone of his generation? Well, we've certainly interfaced with client *contacts* in their late 20s and early 30s, particularly within some of our high-technology clients. But a millennial decision-maker? This is new."

It's real. More and more frequently, gen X-ers and baby boomers are starting to encounter millennials (born from the early 1980s through the early 2000s) in the role of prospective clients. If this hasn't happened to you yet, you should be prepared for it to in the next couple of years.

This means AEC business developers will no longer have the luxury of building relationships with peers who share our same generational sensibilities and preferences. As a result, we will have to modify aspects of our existing sales process and adjust our communication channels, processes, touch-points and messaging.

How to Appeal to Millennials

What is the highest and best approach to appeal to the millennial generation? Here's my set of initial tips, based upon my own direct interaction with millennials in the college classroom, casual conversations with folks that have already experienced the phenomena of "selling to a younger generation" and reliable sources citing observable millennial character traits.

Millennials recognize that the global economy demands constant change and innovation—and as such, they don't empathize with hierarchically companies.

Peer reviews and social media. Millennials value the opinions of others, even strangers. In most cases, they will not make a decision—large or small—without first looking at reviews. Often, these reviews are located online via social media outlets.

Tip: Make sure to have *multiple* references for each project in your portfolio. Seek the perspectives of not just the owner, but also the client contact, representatives from user groups and other stakeholders. Note that a simple phone list of references will not suffice. In fact, since millennials overwhelmingly prefer texting over phone conversations, their motivation to call a reference is likely low. Instead, you need to secure written, robust testimonials. (For the purposes of visibility, don't forget to obtain the authors' permission to include testimonials across *all* your marketing mediums.)

Respect. You may find the millennials' definition of respect to be different compared with prior generations. Millennials inherently desire praise, feedback and to be taken seriously. Yet on the flip side, they are less forthcoming with their displays of respect—until they believe you have earned it.

Tip: When selling to millennials, ask them (often) about their perspective, their experiences and their preferences. But wait. How is this approach any different from your typi-

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

cal consultative-based selling methodology of asking your prospect probing questions followed by acute listening? Here's the difference: In the millennials' case, aim to inquire about their opinion on topics not related to their AE project. In fact, you might even ask for their take on something related to *your* company's business. Essentially, millennials want to be treated as equals even if they are interacting with a seasoned professional. They don't always assume that with experience comes wisdom; they too want their voice heard and considered.

Just-in-time knowledge transfer. When it comes to nitty-gritty details, millennials want the information when they need it—not before. In order to truly absorb new information, they must believe that it is directly relevant at that moment in time. Millennials are less likely to embrace knowledge simply because it will be "helpful down the road." Today's university students, for example, have a habit of weeding through course content, discarding whatever they feel will not directly impact their success on graded assignments. Maybe this mindset stems from growing up with a 24/7 inundation of too much information ("TMI" in millennial-speak) thanks to technology.

Now, more than ever, building multiple levels of relationships is key.

Tip: Put yourself in their shoes. Do the filtering for them. Carefully choose the details that are sensible to share and relevant in the moment. While it may sometimes be tempting to toss everything at a prospect to see what sticks, this approach won't align with the millennials' mentality. And for cross-selling purposes, when you are trying to educate the client on your firm's services across departments, perhaps it is best to keep asking the probing questions first, and then carefully select what information to share.

Nimble, flat organizations. Millennials grew up with the ability to obtain information at lightning speed via technology. They recognize that the global economy moves fast, demanding constant change and innovation. As such, millennials don't empathize with hierarchically companies. When presenting your firm's design process, the millennial decision-maker will keep a keen eye out for perceived waste. Is the team top-heavy? Do responsibilities overlap? Are the design and delivery processes protracted? (Side note: What does *your* organizational chart look like? Is it simple, streamlined and somewhat flat? Or is it cumbersome, complicated and deep?)

Tip: When describing your firm's approach, you need to come across as nimble, resourceful and efficient. For their specific project, you may earn bonus points for offering clever and innovative ways to further eliminate redundancies and increase speed-to-completion. Challenge your own processes—regularly—to ensure they are the most appealing to this set of decision makers.

Unconventional work habits. Early in my business development career, I remember feeling elated whenever I exchanged home phone numbers with prospects and clients. To me, that was the mark of a solid business relationship, one that would survive hiccups and potentially last for the long haul.

As technology evolves and access increases—thanks to cell phones, text messaging, email and social media—it becomes more difficult to draw boundaries around availability. And while it's never a concern with my peer gen X-ers or the boomers, I feel somewhat compelled to "be there almost always" for the millennials. With their mastery in all things technology, it's no surprise that they are not interested in a fixed 9-to-5 work schedule. Many are comfortable with telecommuting and flex time. They don't mind blurring the lines between work life and personal life. This is a plus in the sense that it is easier to reach them. However, they may expect quid pro quo from you.

Tip: I've noticed that if my response time is ultra-speedy and thorough during conventional business hours, millennials are less likely to attempt to reach me outside of 7:00 a.m. – 6:00 p.m. If they do make contact during your sacred personal time, one option is to quickly respond with a promise to fully answer and/or handle it as a priority the next business day. This way they know they have been heard, and you have subtly drawn a boundary to protect your personal space.

Job hopping. Job hopping is something you should plan for regardless of your prospect or client's age. More so than with previous generations, we can expect millennials to switch jobs many times throughout their careers.

Tip: For at least the past 15 years, we've become accustomed to the notion that our client contact may move to another organization. We've aimed to stay in their good graces, with the hope that they would bring our firm on board once they settle into their new digs. Hopefully, we've also had the good sense to carefully build multiple relationships within the original organization so that our firm can be viewed as indispensable rather than as a commodity. Now, more than ever, building multiple levels of relationships is key. That said, it must be handled with sensitivity, since your millennial client contact may not (or may!) be open to it.

As we continue to discover more about millennials, over time we will identify effective approaches for selling to this exciting—albeit sometimes a bit perplexing—generation. ■


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Engineering an EXPERIENCE

BY REBECCA LABERENNE, PE,
BEN ROSENBERG, PE,
AND ERICH OSWALD, PE

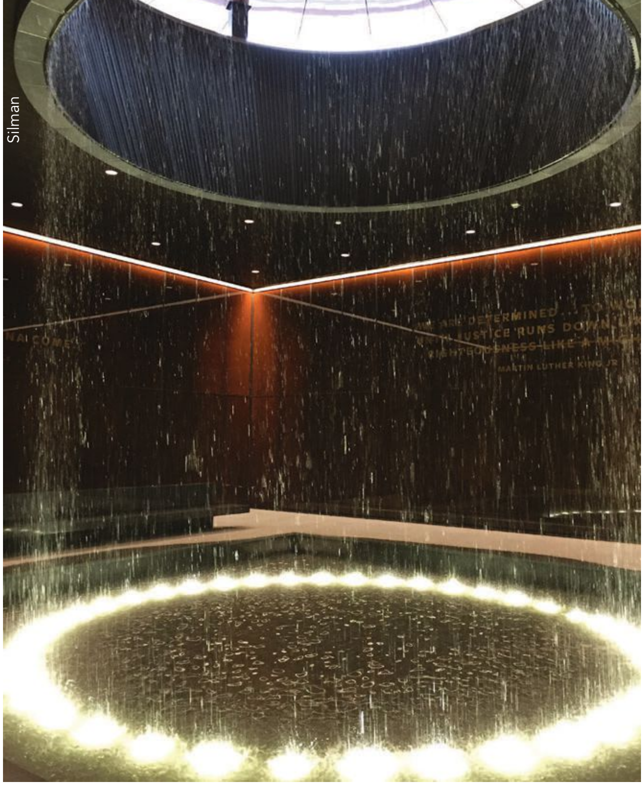
At the brand-new National Museum of African American History and Culture in Washington, the building itself is a striking and enduring part of the exhibit.

THIRTEEN YEARS AFTER IT WAS ESTABLISHED by an Act of Congress in 2003 and nine years after the design was selected through an international competition, the Smithsonian Institution's new National Museum of African American History and Culture (NMAAHC) was dedicated on September 24 by President Barack Obama and a host of dignitaries and benefactors.

Located on the National Mall in Washington, D.C., the museum chronicles and memorializes the African American experience in a new 376,000-sq.-ft building that itself becomes part of the story. The structure uses steel throughout (just over 4,000 tons) to achieve the sweeping programmatic and aesthetic goals and enhance the visitor experience.

Two-thirds of the museum's space is below grade, extending 65 ft underground on four different levels. The below-grade structure is primarily framed in cast-in-place concrete, including the mat slab and pile-supported foundations, concrete floor slabs and concrete columns. The museum's main permanent historical exhibition, located below grade with a green roof above it at grade level, was originally to be framed in cast-in-place concrete as well. However, due to the fast-track schedule and in conjunction with the owner, exhibit designer and contractor, the roof was redesigned in structural steel after construction of the facility had already begun.

Long-span steel beams and custom plate girders with only two lines of columns create the interior open space critical for gallery exhibitions for this level, and a central steel-framed oculus allows light

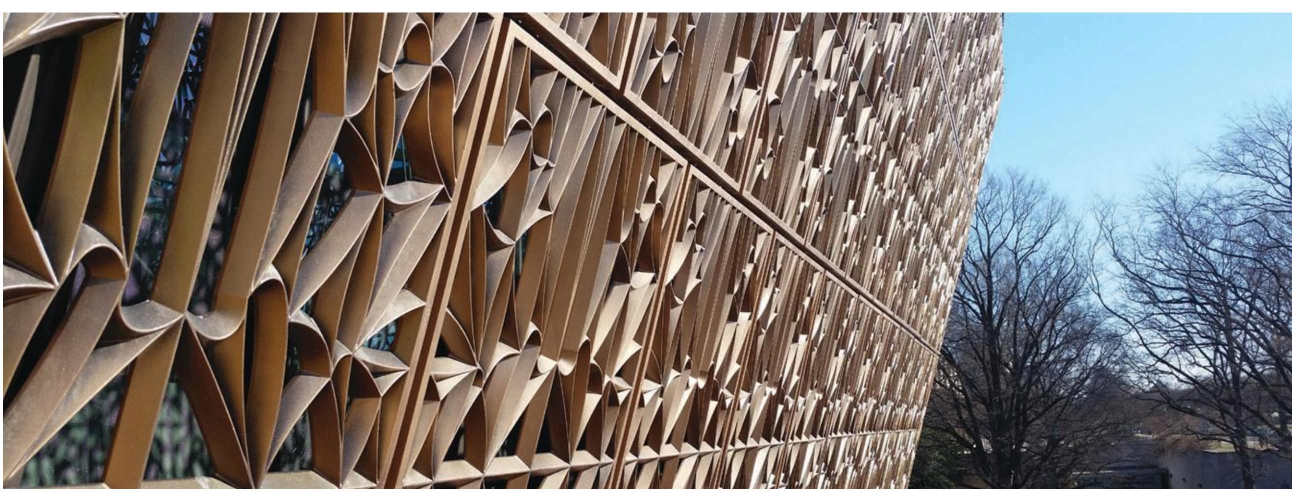


The oculus from below.

The museum's delicate cladding system resembles an African carved wood crown, or corona.

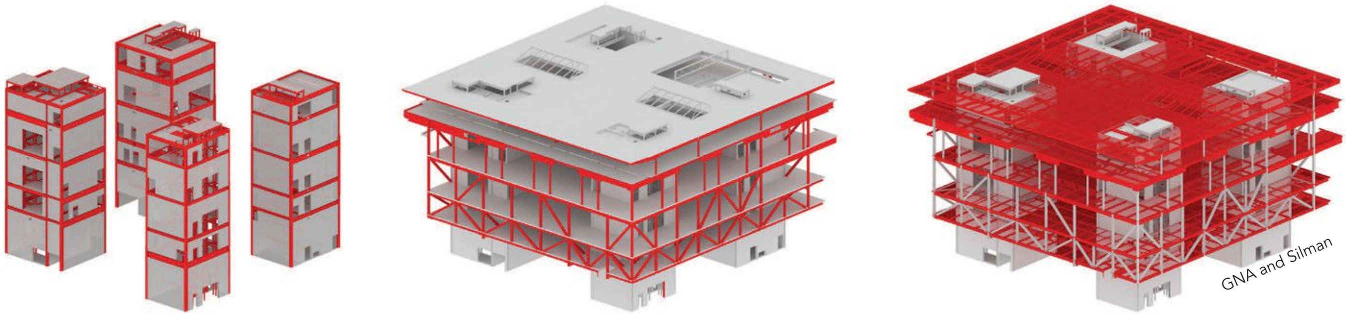
The steel-framed history gallery roof (at grade). The plywood box at right is temporary protection around the installed Pullman train car.

The Corona frames into the building structure only at level 5 and at the base of the museum, thus creating a continuous atrium surrounding all sides of the museum.



into the exhibition space from grade level. The oculus is framed with rolled W40×183 steel beams at the first floor, and the pop-up roof is built from tapered plate girders that radiate out. Steel also eliminated the need for complicated shoring within the gallery during concrete curing, which would have inhibited ongoing work below. It also allowed for the installation of large exhibit items—such as a Pullman train car and a Louisiana State Penitentiary guard tower—without delaying roof construction. This level was quickly redesigned as construction progressed to allow enough time for steel fabrication while maintaining the overall schedule.

Rebecca Laberenne (rlaberenne@100resilientcities.org) is associate director of 100 Resilient Cities, **Ben Rosenberg** (rosenberg@silman.com) is an associate with Silman and **Erich Oswald** (eo@nordenson.com) is an associate partner with Guy Nordenson and Associates.



GNA and Silman

Steel composite cores, perimeter trusses and floor framing.

Steel Composite Cores

The five-story above-grade structure of the museum, including its iconic facade system, is supported entirely from four structural cores in order to create a column-free ground-floor lobby. The four structural cores are composite systems made up of steel floor beams and corner steel columns. These beams and columns are made composite through shear studs with reinforced cast-in-place concrete infill walls and support both the entire gravity load of the building as well as serve as the primary lateral force-resisting system. This composite system provided greater lateral stiffness, allowed for greater flexibility in the coordination of large penetrations through the walls for building services and provided greater resistance to blast loading and progressive collapse than a traditional braced-frame core system. It also allowed for simpler connections between the cores and the long-span and cantilevered steel framing in the building than typical reinforced concrete walls.

The reinforced concrete infill walls vary in thickness from 10 in. to 16 in. depending on the shear force transfer required. They are perforated in numerous locations for architectural openings, mechanical ducts and other utilities. The concrete infill walls are connected to the steel columns and beams with headed shear studs, which allow shear and axial force transfer between the steel frame and walls, and in some locations reinforcing bars are coupled to the steel frame. The boundaries between the steel and concrete where the shear studs are located are heavily reinforced with closed stirrups to provide confinement around the shear studs. A blast-loading analysis was carried out on the core systems to determine out-of-plane pressures on the infill walls and to identify upgrade requirements to steel framing sizes.

Four separate structural models were built in ETABS (v9.7.2) to analyze and design the composite cores:

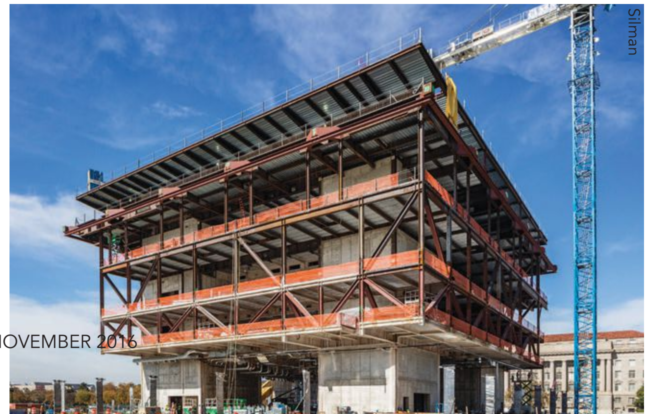
- Model 1 was a steel-only model with "dummy" diagonal braces used to design the steel core beams and columns for gravity loads and overturning forces due to lateral loads. This model was also used to determine connection design forces for the steel framing.
- Model 2 was a concrete-only model with large openings, explicitly used to determine the required thickness and reinforcement for each infill wall and the total number of shear studs required between the concrete and steel elements.
- Model 3 was a composite model with steel framing elements and concrete shell elements used to calculate the sharing of load between these components and to calculate drift.
- Model 4 was similar to Model 3 except that the stud connector elements were modeled explicitly as nonlinear elements.

Deep composite plate girders span between the structural cores at each level to support the main gallery levels. Additionally, each gallery floor plate projects outward from the cores and is supported by cantilevered steel beams on the north side and perimeter vertical trusses and cantilevered brace "outriggers" framing into the composite cores on the east and south sides.

The Corona

One of the most prominent design features of the museum is its delicate cladding system resembling an African carved wood

The project uses just over 4,000 tons of steel in all.



Silman

The level 2 platform with perimeter vertical steel trusses.

Richard Barnes

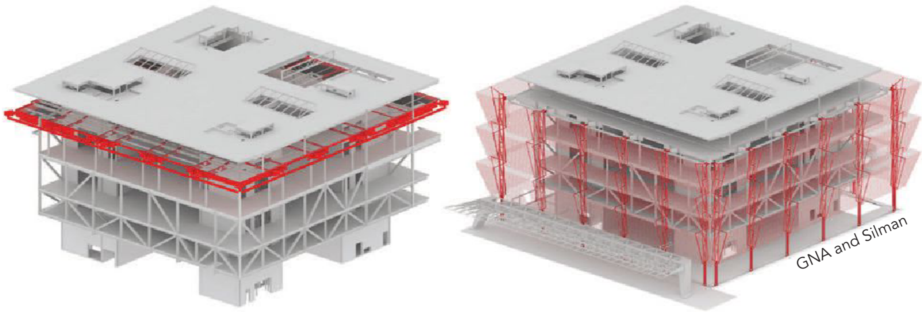
crown, or corona. While conventional building enclosures are typically connected at each floor level of a building, the Corona frames into the building structure only at level 5 and at the base of the museum, thus creating a continuous atrium surrounding all sides of the museum.

Based on several framing studies conducted early in the design process, it was determined that suspending the glazing and cladding support structure from the top of the building rather than from the base would be advantageous in limiting the member sizes of the cladding structural elements, as the self-weight would effectively pretension these elements against compression loads induced by wind and other transient loads. The decision to support the cladding from the top of the museum structure led to a number of complexities in the design of the building's superstructure. These included stringent drift requirements for the lateral load-resisting system, complex base connections requiring fixity and freedom of movement along different axes and consideration of unusual load paths for wind and other transient loads such as blast and seismic acting on the facade system.

The schematic design for the Corona support structure included a series of suspended horizontal trusses supported by vertical cables and braced with in-plane diagonal X-cables. Gravity loads were resisted in tension by the vertical cables, and lateral loads such as wind loading on each face were transferred laterally by the horizontal trusses to the orthogonal faces of the structure, which in turn functioned as inverted braced frames to transfer these lateral forces in-plane to level 5.

However, as a result of value engineering and construction sequencing considerations, this 3D structural system was replaced by hung vertical trusses comprised of architecturally exposed hollow structural steel sections (HSS) spanning approximately 92 ft from the base of the building to level 5 in the final built design. The glazing that provides the enclosure of the building envelope is located on the inner face of the vertical trusses. Perforated cast aluminum panels clad the outer faces, and service catwalks are located in the cavities between these surfaces.

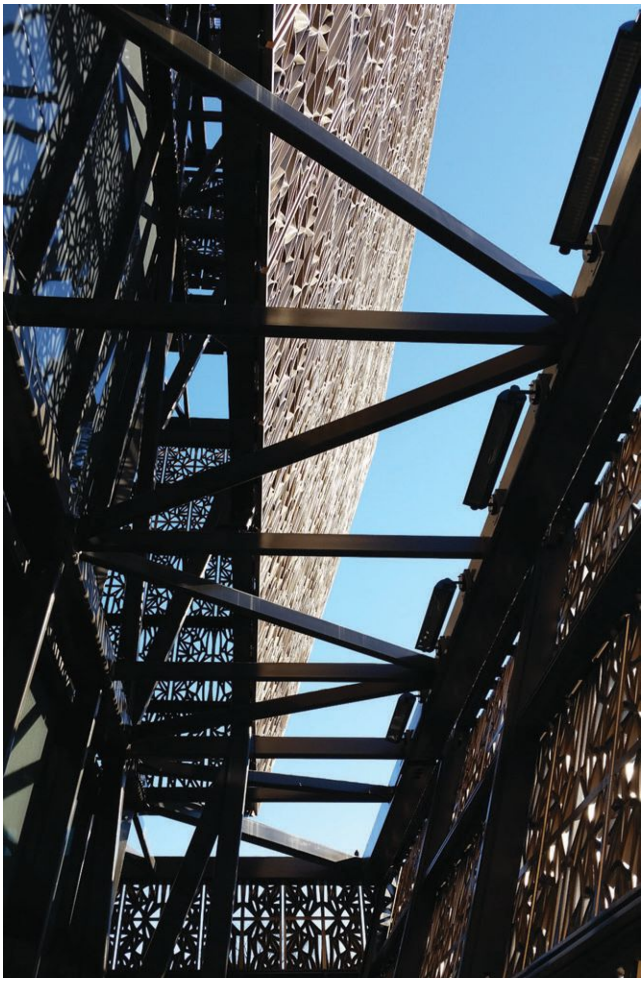
Wind-tunnel testing was conducted for the Corona system to determine wind loading requirements on both the glazing and the cladding, and special analysis was conducted to calculate the significant ice loading requirements on the exposed steel and cladding panels.



Level 5 cantilevers for the Corona support (red at left) and the Corona as it hangs from the overall superstructure (right).

The Porch Canopy

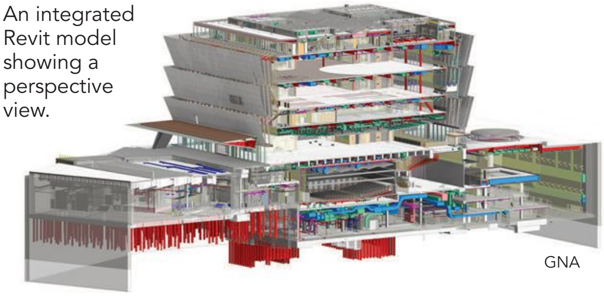
Approaching the museum lobby from the south entrance, visitors pass below a massive freestanding canopy structure referred to as the "Porch," which is independent from the rest of the above-grade structure and supports an unoccupied green roof. The structure is made up of an approximately 9-ft by 6-ft steel box girder that spans 172 ft between two built-up steel plate box columns supported from below by concrete columns in the below-grade loading dock. Tapered fabricated steel plate beams cantilever from the south edge of the box girder to create a triangular or wing-shaped cross sec-



GNA and Silman

The porch canopy structure, under construction.

An integrated Revit model showing a perspective view.



GNA

The five-story above-grade structure of the museum, including its iconic facade system, is supported entirely from four structural cores.

A Louisiana State Penitentiary guard tower is one of the museum's exhibits.

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tion to the canopy that supports the green roof surface. As a result of this cantilevered geometry, gravity, wind and blast loading conditions induce significant torsional forces on the primary box girder. The entire steel structure is clad in ultra-high-performance precast concrete panels.

Construction and Coordination

The project team performed trade coordination during construction due to the tight schedule; construction began in November 2011 and was completed in September 2015. For both the below- and above-grade core walls, this meant reviewing MEP penetrations on a floor-by-floor basis, often only a few days before construction or even the day that concrete was to be poured. The structural engineering team joined the daily phone calls to discuss open coordination and field issues so that they could be resolved immediately and construction could continue. These included steel coordination issues such as connections conflicting with architectural finishes or headed shear studs conflicting with core wall reinforcement. In addition, because the National Mall is federal land run by the National Park Service—and disrupting the surrounding roads is not feasible except in the most extraordinary circumstances—the contractor crafted a site plan to allow room for steel laydown and erection that was coordinated with ongoing construction.

Thanks to these coordination efforts as well as structural design changes that made for a more efficient structure, the latest Smithsonian addition to the National Mall is now open to share its portion of the ongoing American story.

Owner
Smithsonian Institution

General Contractor
Clark/Smooth/Russell, A Joint Venture

Architect
Freelon Adjaye Bond/SmithGroup

Structural Engineer
Robert Silman Associates Structural Engineers, Washington, D.C.
Guy Nordenson and Associates, New York

Steel Team

Fabricator
SteelFab, Inc., Charlotte

Erector
Bosworth Steel Erectors, Inc., Dallas

Detailer
SteelFab, Inc., Charlotte
Prodraft, Inc., Chesapeake, Va.

The new home of the Minnesota Vikings ushers in a new age of NFL stadiums with its steel-supported ETFE roof and giant operable wall panels.

TIGHT Coverage

BY
JOELLE NELSON, PE, AND THOMAS DUFFY, PE
PHOTOS COURTESY OF THORNTON TOMASETTI

Joelle Nelson (**jnelson@thorntontomasetti.com**) is a senior associate and **Thomas Duffy** (**tduffy@thorntontomasetti.com**) is an associate principal, both with Thornton Tomasetti's New York office.

U.S. BANK STADIUM gives Minnesota Vikings fans the best of both worlds.

With natural sunlight pouring in through the clear roof and the glass western wall, the stadium conveys a feeling of being outdoors without subjecting fans to the sometimes harsh winter weather conditions and extreme temperature swings for which Minnesota is well known—especially late in the NFL season.

This outdoor experience is exactly what stadium officials were looking for, though not quite how they imagined it. The trend

U.S. Bank Stadium, which opened in time for the 2016 NFL season, seats more than 66,000 and is expandable to 73,000.

Each panel has a triangular truss that wraps around a 36-in.-diameter steel pipe building column.

in state-of-the-art stadiums was to incorporate a retractable roof, and that's what the Minnesota Sports and Facilities Authority initially envisioned. However, after much consideration the design team, led by architect HKS and structural engineer Thornton Tomasetti, determined that with Minneapolis' ground snow loads in the range of 50 lb per sq. ft and drift loads over three times that, a retractable roof, which would rarely be opened, would come at a hefty price tag. Instead, they presented the idea of a fixed transparent ETFE (ethylene tetrafluoroethylene) roof, the largest application of its kind in North America, paired with the world's largest pivoting wall panels—and that is what the project team delivered.

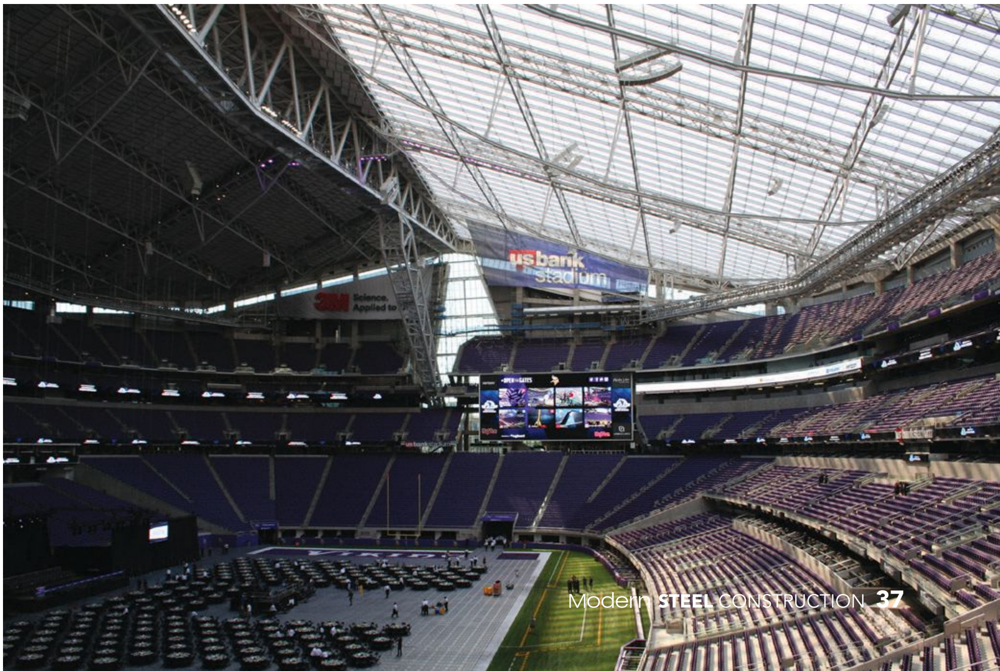
In Motion

Without the constraints of a retractable roof, the design team was free to shape the roof in a way that best accommodated the environmental and structural needs. And while the sharp peak and slope of the roof (which is supported by a 989-ft-long, 1,700-ton ridge truss located 214 ft above the field) make an architectural statement, they came as a result of snow load studies performed by RWDI in coordination with Thornton Tomasetti. The two companies worked together to optimize the roof slope for reduced snow loading, and the slope, orientation and low friction coefficient of the ETFE material allow snow to slide off the roof instead of accumulating and ponding—a problem at the old Metrodome. At U.S. Bank Stadium, the heavy, sliding snow is safely captured and contained by a sophisticated snow catchment system, where it is melted and drained away.

An operable wall, not subject to snow loading, was another suitable option for Minnesota and an appropriate complement to the roof. However, this type of wall comes with an entirely different set of design challenges, and a pivoting operable wall had never been achieved at this scale before. Located at the main entry point for the stadium along the western face, the operable wall consists of five panels that, when open, form an opening in the west wall roughly nine stories high and nearly as long as the football field itself. Each wall panel, ranging between 75 ft and 95 ft in

The pivoting wall panels are the world's largest.

The ETFE roof is the largest of its kind in North America.



Doors in doors: ground-level access doors in the movable wall panels.

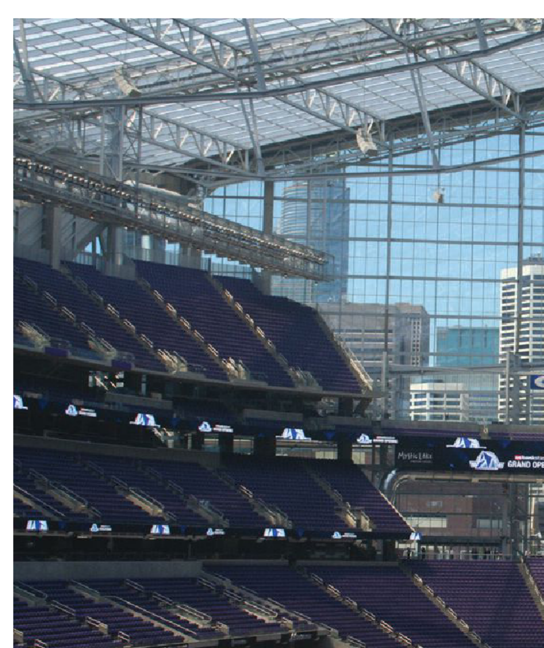
height and 57 tons to 68 tons in weight, pivots open 90°; 6 ft of the 55-ft width of each panel swings inward into the main concourse space behind the facade, and the remaining 49 ft cantilevers out into a large public plaza. Doors of similar size would typically slide like a hangar door, and a pivoting panel of this size was unprecedented and uniquely challenging to design.

Triple Option

Each panel has a 10-ft-deep vertical steel truss, triangular in plan, that wraps around a 36-in.-diameter steel pipe building column, which also supports the upper concourses and roof. The truss geometry incorporates Vierendeel panels to provide clearance for rotating through the W33 and W36 girders that extend out of the pivot columns to support the upper stadium floors. This tri-chord truss, comprised of standard box HSS, forms the spine of the panel, with a series of cantilevered wings (also made from box HSS) extending off the truss to support the glass curtain wall. Steel rods run diagonally, serving a triple purpose: providing gravity support for the cantilevered wings, stiffening the panel and plumbing the panel during erection. The eccentricity of the facade load to the panel structure would pull the panel open at the top tip, so the engineers carefully calculated the pretension in each rod to resist this effect and keep each panel plumb.

Each panel's mechanical components, designed by Hardesty & Hanover in close coordination with Thornton Tomasetti, consist of an upper bearing, a lower bearing, three pairs of hydraulic cylinders and three to four actuating lock pins. The entire weight of each panel

The new roof was designed to easily handle Minneapolis' high snow loads.



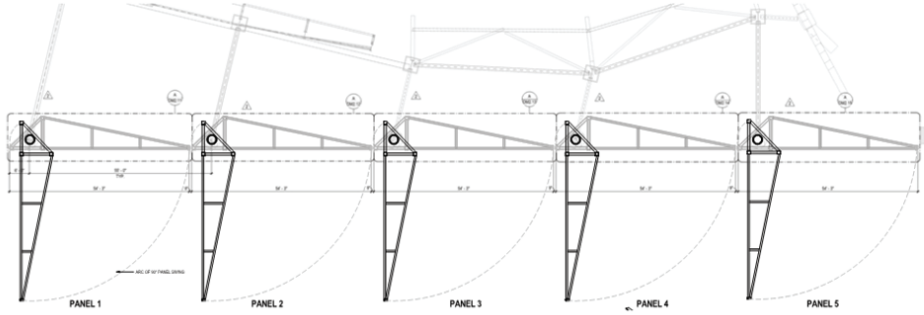
is supported by a spherical thrust bearing at the bottom, which, along with an HDPE (high-density polyethylene) sleeve bearing at the top, provides the lateral support for the panel, with allowance for differential movement between the panels and the stadium structure. Three pairs of hydraulic cylinders within the tri-chord truss serve as the driving force behind each panel, rotating the panels open and closed in five minutes and providing torsional resistance to the wind load.

Force to be Reckoned with

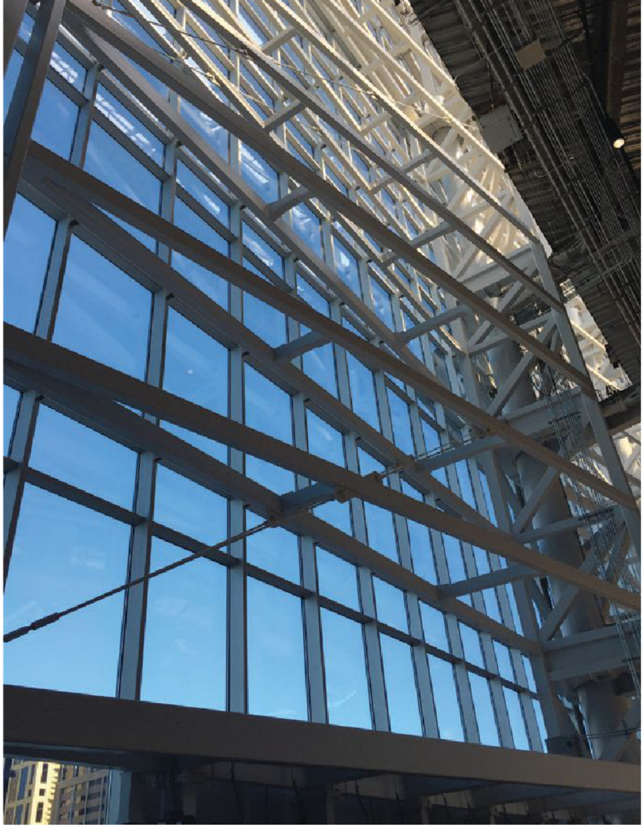
The eccentric gravity load from the panel's 49-ft cantilever imparts a significant horizontal force to the upper bearing at the top of the door and the lower bearing at the bottom of the door. The direction of this horizontal force changes as the door swings open, and the building structure had to be designed to resist this force in any direction with limited deflection.

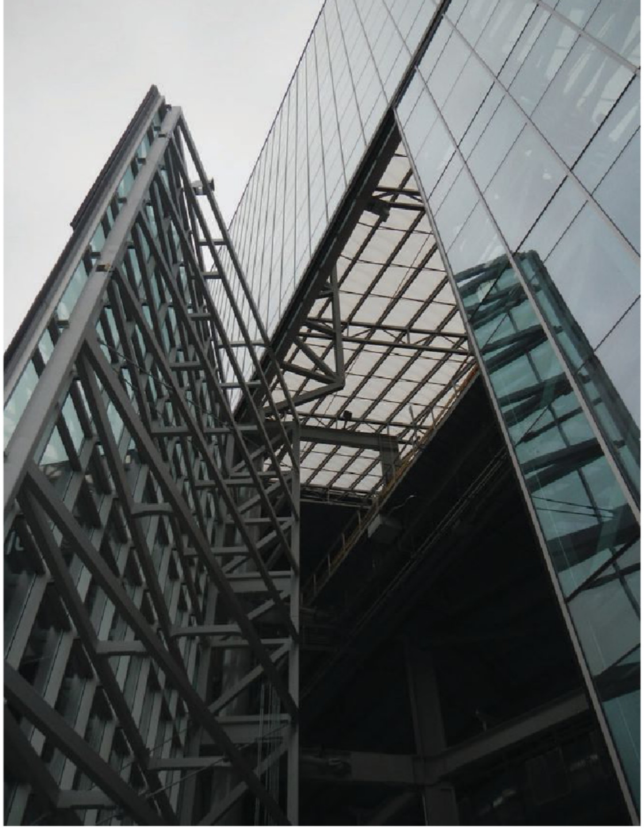
The panels are not to be operated when wind loads exceed 40 mph, but they are designed to be held open in a 67.5-mph wind, which equates to a one-year recurrence interval. For the full-design wind event, a 90-mph wind, the panels are closed and pinned together by a total of 19 actuating pins. (An anemometer has been placed in the stadium's parking lot to alert officials when the wind speeds get too high and the panels should be closed.) While NFL regulations require that the panels be either open or shut prior to kickoff and remain in that position throughout the entire game, they allow for them to be closed should wind speeds become suddenly elevated.

The final design of the wall panel structure was completed in SAP2000, with the hydraulic cylinders treated differently under various stages. For the stationary conditions, the hy-



A diagram of the panel walls, open and closed.





Since the wall panels serve as the main entry point for the stadium even when they are closed, they include a series of standard entry doors at the base. These door banks must be ADA-compliant, so only a minimal threshold was allowed.

draulic cylinders are locked-off such that they behave as rigid supports—but lateral load due to overturning is to be resisted entirely by the upper and lower bearings. Therefore, using non-linear construction sequencing, each pair of cylinders was initially modeled as a torsional support and replaced by a pair of lateral supports only after the panel self-weight was added to the model.

When the panels are operating and the wind is blowing, the hydraulic cylinders do not respond as rigid supports. Instead, the control system delivers equal pressure (and load) at each pair of cylinders. For each load case, the reaction in the cylinders had to be calculated by hand and applied in the model as loads.

Sealing the Perimeter

While all of the above may sound challenging, many on the design team would say the most difficult aspect of the operable wall design was the perimeter seals. Because Minnesota has such a wide temperature range, high winds and heavy snowfall, weather-tight seals were absolutely necessary. However, this large temperature swing also causes significant thermal movement. The vertical seals are formed as the tip of one panel presses against the butt of the adjacent panel. Given the multiple swinging parts, thermal movement, curtain wall depth and minimum clearances when any single panel operates, achieving weather-tight and architecturally pleasing seals that function optimally under Minnesota's extreme temperature conditions required extensive collaboration between design team members and mock-up testing.

On top of this, because the wall panels serve as the main entry point for the stadium even when the panels are closed, they include a series of standard entry doors at the base. These door banks must be ADA-compliant, so only a minimal threshold was allowed. Thus, the personnel doors themselves form a tight seal at the base when the panels are shut but also clear the ground so as not to drag while the panels swing open. Therefore, each wall panel has an operable door bank. Before the wall panel swings open, the entire door bank, consisting of five sets of standard double doors, raises up 4 in. while the remaining length of the panel has operable sills—small panels at the base that lift a few inches before the wall panel moves.

Using 17,250 tons of structural steel, the stadium is ready for whatever the weather can throw at it, and the 66,655 Vikings fans that fill it on football Sundays (seating capacity is expandable to 73,000 for the Super Bowl) will experience the game in a first-of-its kind venue that adds a whole new dimension to the game-day experience.

Owner
Minnesota Sports and Facilities Authority

Construction Manager
M.A. Mortenson Company, Minneapolis

Architect
HKS, Inc., Dallas

Structural Engineer
Thornton Tomasetti, Inc., New York

Mechanization Engineers
Thornton Tomasetti, Inc./Hardesty & Hanover, LLC, New York

Steel Team

Fabricators
LeJeune Steel, Minneapolis
Merrill Iron & Steel, Inc., Schofield, Wis.

Erectors
Danny's Construction Co., Shakopee, Minn.
Derr and Isbell Construction, LLC, Euless, Texas

Detailers
LTC, Inc., West Salem, Wis.
Aarbee Structures Pvt., Ltd., Bangalore, India

MIX and Match

A new mixed-use development is boosting one of New York's most prominent business districts while working to blend in with its existing neighbors.

BY AMIT MAVALANKAR, PE, JAMES J. BONANNO, PE, AND STEPHEN V. DESIMONE, PE

Amit Mavalankar (**amit.mavalankar@de-simone.com**) is a senior project manager, **James J. Bonanno** (**james.bonanno@de-simone.com**) is a principal and **Stephen V. DeSimone** (**Stephen.desimone@de-simone.com**) is president, all with DeSimone Consulting Engineers.

A rendering of the overall complex.

Phase 1 of the project uses 4,500 tons of steel.

The residential building is at left and the commercial building at right.

DOWNTOWN FLUSHING IN QUEENS is the fourth-largest central business district in New York—nothing to sneeze at in a city of 8.4 million.

As the epicenter of Flushing—a neighborhood of roughly 72,000 residents—the area is a stone's throw from Flushing Meadows Corona Park, home of the USTA Billie Jean King National Tennis Center, and the New York Mets' Citi Field.

Currently rising from its center is a 1.8-million-sq.-ft complex known as Flushing Commons. The mixed-use project is organized into four components comprising a residential building and office, retail, parking and community space in a total of five distinct buildings. Phase I is a development of approximately 670,000 sq. ft, with a 14-story residential tower and an 11-story office building, both steel-framed. Underneath the buildings is a concrete-framed four-story, 270,000-sq.-ft below-grade garage with 980 parking spaces.

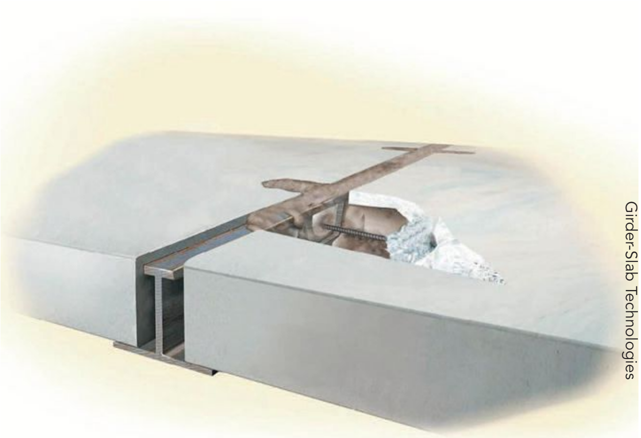
The massive project achieved its first milestone this past spring with the topping out of the residential tower and the office building. Using 4,500 tons of structural steel, Phase I is expected to be completed early next year, with the entire project scheduled to open by 2021.

➤ Comprising 1.8 million sq. ft, the entire project will feature a total of five distinct buildings once complete.

➤ Planks over the D-Beams for a typical residential floor, prior to grouting.

Residential

Designing a mixed-use residential tower is never an easy or straightforward task, as the column grids for parking and retail portions do not typically synch with the column spacing for the residential units. The columns for the residential units are usually located within the room layouts at locations where they may be most easily camouflaged by architectural elements, and trying to match them with the commercial column grids often imposes constraints on floor layout. The design team looked to the Girder-Slab system to address this issue. The system is an assembly of asymmetrical steel beams referred to as "D-Beams" and is fabricated from a standard rolled wide-flange section and a flat bar. It supports a hollow-core precast plank on its bottom flange.



Girder-Slab Technologies

A rendering of the Girder-Slab System

Construction of the residential building.

The gravity system for the residential building incorporates D-Beams for the typical residential levels to support 4-ft-wide, 8-in.-deep planks that span 28 ft, which matched the column spacing in the retail levels and parking garage. In the other direction, the beams span 18 ft to 20 ft between the W12 columns. The lateral force-resisting system was designed to react to wind loads and consists of concentrically braced frames around the stair and elevator core, using brace and column sizes up to W14×370. The system is predominantly drift-controlled, and fewer braced frames were used to control the overall as well as inter-story drift. The lateral shear forces from the steel braced frames are transferred to the concrete shear walls by strap beams.

The residential tower's facade appears as a window wall, blending the brick and glass to form a typical residential look that fits in with the surrounding neighborhood. The facade panels are supported from the edge of the floor system and are attached to perimeter W10 beams in the north-south direction and specially designed solid precast plank with embeds in the east-west direction.

Commercial and Office

The gravity system for the commercial building consists of conventional composite wide-flange steel beams carrying 3-in. metal deck with 3¼-in.-thick lightweight topping concrete. The beams in in turn are supported by W14 columns. The parking column grid of 30 ft by 28 ft is also used for the retail and office floors above. As with the residential tower, the lateral force-resisting system consists of concentrically braced frames around the stair and elevator cores to resist wind loads, and uses wide-flange braces and column elements. Larger wide-flange sections were used limit the inter-story drifts. The lateral system is drift-controlled, and fewer brace frames were used at limited locations to keep the design in line with the architect's requirements.

Phase 1 includes a 14-story residential tower and an 11-story office building, both steel-framed, on top of a four-story, 270,000-sq.-ft below-grade garage with 980 parking spaces.

Construction of the 11-story office tower.

The facade consists of brick panels at the retail floors and high-performance curtain wall and patches of brick veneer at the commercial floors. The prefabricated panels are hung from the perimeter slab edges, and $\frac{5}{16}$-in.-thick to ½-in.-thick steel plates cantilever from the top flanges of the perimeter beams to receive the embed plates for the various facade attachments.

A strap beam erected on-site.

Construction Coordination

The architect and engineers used 3D building information modeling (BIM) to coordinate the design with various other disciplines. The use of a Revit model in the preliminary phases of design facilitated early coordination between different trades and resolve clashes between structural and MEP components. The steel fabricator used Tekla to model the steel-framed buildings. This helped the general contractor coordinate the strap beams, concrete podium and facade embeds to support the panelized facade system with the structural interfaces. The contractor and design team used Autodesk's Constructware as a submittal tracking system, which helped smoothen the construction administration process.

This level of coordination, over a project with so many components and in a tight urban setting, is impressive but also expected for such a high-profile development at the heart of New York's largest and second-most populous borough.

Owners
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General Contractor
Tishman Construction, New York

Architect
Perkins Eastman Architects, New York

Structural Engineer
DeSimone Consulting Engineers, New York

Steel Fabricator, Erector and Detailer
Berlin Steel Construction Co., Kensington, Conn.

A Revit model of Phase 1.

On the Fast TRACK

BY BEN NEAL

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

All photos: The Ruhlin Company

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

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

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

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

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

◀ Removing an existing span.

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

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

Minimizing Delays

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

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

The Importance of Preplanning

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

Unloading steel in the lay-down yard.

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

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

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

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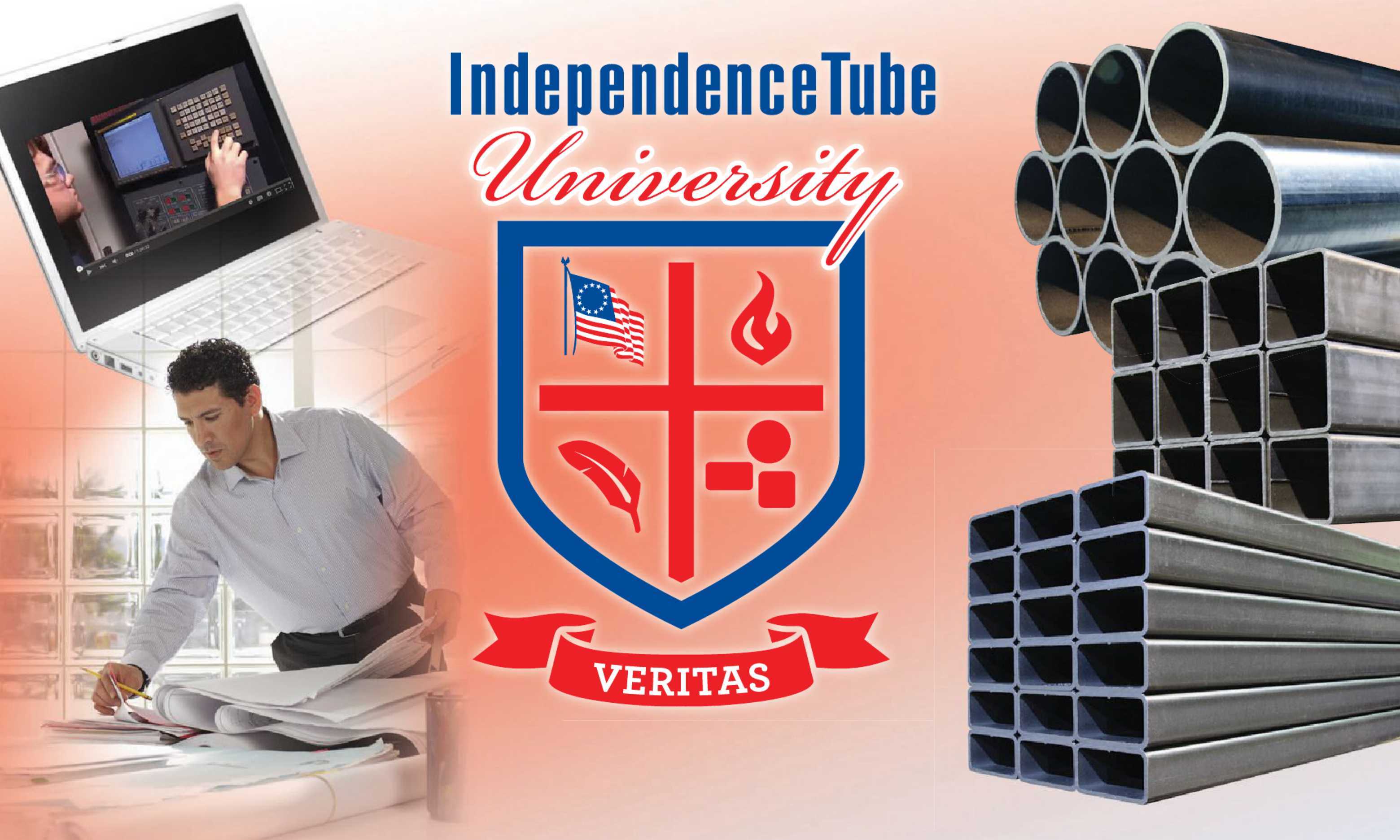

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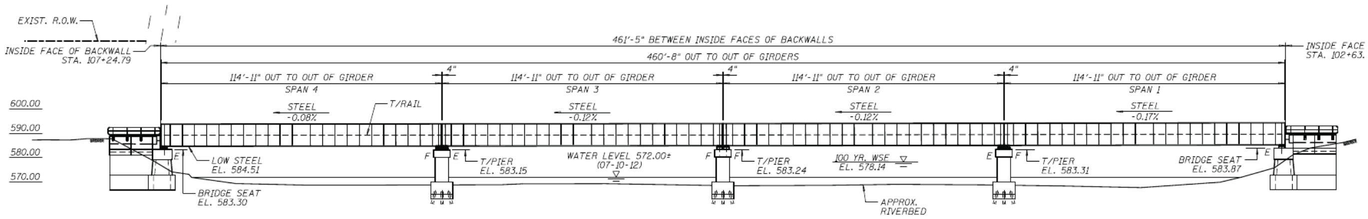


An elevation view of the new bridge.

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

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

Up and Running

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

Owner
Norfolk Southern, Norfolk, Va.

General Contractor
The Ruhlin Company, Akron, Ohio

Structural Engineer
Alfred Benesch and Company, Chicago

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

IN MEMORIAM

Bridge Engineering Expert Dennis Mertz Dies at 63

Dennis Mertz, PhD, professor of civil engineering and director of the Center for Innovative Bridge Engineering at the University of Delaware (UDel), passed away last month after a prolonged battle with cancer. He was 63.

A professor at UDel for 25 years, Mertz dedicated a large portion of his professional career to advancing the state of the art in bridge engineering. The research he conducted, combined with his efforts on numerous technical committees, helped shape the industry into what it is today. His contributions were recognized earlier this year at the International Bridge Conference, where he was presented with the John A. Roebling Medal for his lifetime achievements in bridge engineering. He previously earned a Steel Bridge Forum Award in 1997, an Innovation in Steel Bridge Award from AISI in 1998, a Special Achievement Award from AISC in 2000, the Richard R. Torrens Award from ASCE in 2003 and the Richard S. Fountain Bridge Task Force Award from AISI and AASHTO in 2005. Mertz touched the lives of hundreds of students and directly shaped the career of many bridge engineers practicing today. He also mentored many young professors throughout the U.S., providing them with sound advice and guidance as they moved forward in their own academic careers.

"Dennis was a well-regarded leader and noteworthy contributor amongst his colleagues," said Bill McEleney, NSBA's managing director. "In addition to his superlative technical skills, his quick wit, upbeat personality and genuine friendship drew many to him. He was a great friend to many of us, and he will truly be missed."

Mertz is survived by his wife, Madelyn, and one brother.

SUSTAINABILITY

Steel Industry Unites for Material Transparency

SMDI has compiled a comprehensive list of industry-wide environmental product declarations (EPDs) for steel building products. These EPDs summarize the results of a life-cycle assessment (LCA) for specific steel products in the construction industry to describe their potential environmental impacts.

Construction professionals interested in viewing and using EPDs and other transparency resources in their building projects can visit **www.buildusingsteel.org** for the list of steel product EPDs and updates on other sustainability resources. Currently, the site includes EPDs for 11 product categories, including Fabricated Hot-Rolled Structural Sections and Fabricated Steel Plate from AISC (which are available for free at **www.aisc.org/epd**).

"Similar to a nutrition label on food packaging, environmental product declarations present concise information to help building professionals make better-informed product decisions," said Mark Thimons, vice president of sustainability for SMDI. "In an effort to be as transparent as possible, these steel industry EPDs are more comprehensive than those for many other building materials, creating a truly all-encompassing view of each product's environmental impacts."

Building professionals can use this list of EPDs to help earn credits in green building rating systems such as the U.S. Green Building Council's LEED v4, which offers opportunities for steel in a revamped materials section including credits for LCAs, EPDs and transparency.

SMDI will update its list as the steel industry continues to develop EPDs to maintain an informative, current resource for building professionals interested in the sustainability of steel building products.

People and Firms

- **Independence Tube Corporation** has entered into an agreement to sell the company to **Nucor Steel**. Founded in 1972 in a 53,000-sq.-ft facility on the southwest side of Chicago, Independence Tube has grown to 1.7 million sq. ft under-roof with two manufacturing divisions in Illinois and two more in Alabama. Nucor plans to install a electric-resistance-weld mill in the south, which will be capable of producing large HSS and pipe piling sizes.

- **Jaime Garza, SE**, has joined **John A. Martin and Associates, Inc., Structural** Engineers as senior project manager. Garza brings over 13 years of expertise in seismic design and retrofit of existing buildings, and his past clients include the University of California, the Los Angeles Unified School District, the Los Angeles Police Department, Los Angeles International Airport and most recently the Waldorf Astoria Hotel and Beverly Hilton Garden in Beverly Hills. To see a nonbuilding steel structure that Garza designed, check out the January 2015 Structurally Sound item "Vital Fluid" in the Archives section of **www.modernsteel.com**.

AISC NEWS

Larry Kruth Named AISC Vice President of Engineering

Lawrence F. Kruth, PE, has been named the new vice president of engineering and research at AISC, where he will oversee all technical activities. He succeeds Charles J. Carter, SE, PE, PhD, who has been promoted to president of AISC, effective December 5.

Most of Kruth's career has been with Douglas Steel Fabricating in Lansing, Mich., most recently as a vice president. He has notable expertise in fabrication and erection, quality systems, safety and connection design. Before joining Douglas Steel in 1984, Kruth had stints with H & G Fabrication Corp. in Grand Ledge, Mich., Kaiser Engineers of Pennsylvania in Pittsburgh, Master Engineers in Pittsburgh and Franklin Associates in Somerset, Penn.

"Larry brings an amazing breadth of expertise and proficiency in fabrication and erection and the associated engineering," said Carter. "I've worked with him closely on a number of AISC technical committees and have always been impressed by his knowledge and his ability to work with a wide range of people." Kruth has served on the AISC Specification Committee and its task committees on connection design and quality control and assurance, the AISC Safety Committee and the AISC Research Committee. He also has assisted with AISC'S efforts to provide resources for construction management education, is a 25-year veteran of the National Student Steel Bridge Competition, and served four years on the AISC Board of Directors.

Outside of AISC, Kruth is a member of the Research Council on Structural Connections, Structural Engineers Association of Michigan, MIOSHA Part 26 – Steel Erection Advisory Committee and MIOSHA Part 10 – Lifting and Digging Equipment Advisory Committee. He also has served as adjunct faculty at Michigan State University for the Capstone Structural Engineering class. In 2011, he was named Engineer of the Year by the Structural Engineers Association of Michigan.

Investing Intelligently in Infrastructure

The quote “There seems to be one point on which everyone is in agreement: the need for increased infrastructure funding” from Scott Melnick’s September editor’s note (available at **www.modernsteel.com**) is correct, but it requires an important clarification. While sufficient funding is a main issue, it is not the only problem. The highest priority is fixing substandard bridges. The deterioration of our bridges is the most dangerous issue related to infrastructure. A failure of or even serious damage to a bridge may cost lives and will close a highway for a long period of time.

According to the U.S. Department of Transportation, at the end of 2015 there were 661,845 highway bridges in the country. More than 142,900 of these bridges are “substandard” (structurally deficient or functionally obsolete) bridges; this constitutes 27% of all bridges (based on the percentage of deck areas). In several states, the substandard percentage is over 50%! This is an alarming, unacceptable situation. A well-developed and maintained transportation system is vital to the safety of the traveling public as well as the economy. Yet the current yearly improvement is less than 0.4%. Assuming that the deterioration rate for bridges does not increase, at the current pace of improvement we will need 66 years to replace or retrofit all of the substandard bridges in the country!

It is important that we invest in our infrastructure that we need, but what may be even more important is how wisely we use the limited funding in order to fix or replace more substandard bridges for a shorter time. Unfortunately, many of our newest bridges are far from efficient structures. One example is the recent replacement of the east span for the San Francisco-Oakland Bay Bridge. This massive structure took 14 years to build at the cost of $6.5 billion. The cost was $33,330 per square meter, when for bridges with similar spans, the average unit cost is about $3,500 per square meter! This means that if Caltrans has used more efficient systems, they could have replaced nine times more bridge area.

Historically, engineers have played an important role in designing and building the country’s highly developed network of railroads, highways and bridges that permitted the U.S. to become the leader in the world economy. And now engineers should once again take the lead in selecting, designing and building highly efficient structures in order to improve the poor condition of our bridges. Such an approach will demand significant revisions of the established methods of planning, designing and managing the process. But there is no other option if we want to solve the deterioration of our bridge infrastructure in a reasonable timeframe.

The transportation authorities should introduce a system that will motivate the various departments of transportation and all their personnel to strive for more efficient and economical bridges; similar motivation should be introduced for the general contractors awarded the projects construction. A good approach would be to have all bridge projects larger than a specific amount—say, $30 million—to be awarded based on competition.

The use of inefficient new structures is an enormous problem, and the first step towards resolving it is to recognize that such a problem exists—and then deal with it. The current poor condition of our bridges is not acceptable for the high standards of American bridge engineering. American engineers should not accept being left out of the development process for new, more efficient structures. With the necessary efforts and persistence, our engineers would be able to revive the golden age of American bridge engineering that produced the Brooklyn Bridge, the George Washington Bridge, the Golden Gate Bridge, the original 1936 San Francisco-Oakland Bay Bridge and the Verrazano Narrows Bridge.

—Roumen V. Mladjov, SE, PE

An Approachable Approach to Bolts

I was struck by James LaBelle's September article on anchor rods, "Strength and Engagement" (available at **www.modernsteel.com**). I found it very interesting, not only because of the topic but also because it was important and written in a simple and engaging manner. Much of the technical information we read today is academic and complicated. It is highly unusual to read about the strength of bolt threads in steel construction, yet it is certainly topical and the article was certainly approachable. In my undergraduate years in the early 1950s, for some reason we studied a number of the different thread designs used around the world and how stress raisers were handled by the geometry chosen. So it is of interest to follow later studies using sophisticated methods like photoelasticity, which were not available at the time.

—Lambert Tall, PE, PhD
Tucson, Arizona

SCHOLARSHIPS

Annual AISC Scholarship Winners Announced

This year, AISC has administered a total of $177,000 in financial aid to 54 deserving undergraduate and masters-level students for the 2016-17 academic year.

AISC's David B. Ratterman Fast Start Scholarship program for freshman and sophomore students, now in its fifth year, awarded 14 scholarships to students at two-year and four-year colleges. These students, who are relatives of AISC member company employees, are full-time freshmen or sophomores during the current 2016-17 academic year.

The AISC Education Foundation, in conjunction with several other structural steel industry associations, awarded 45 scholarships totaling $137,000 to sophomore, junior, senior and masters-level students for the 2016-17 academic year. We would like to offer our sincere thanks to these organizations for their generous continued support of our student programs.

Congratulations to the following students for earning their well-deserved scholarships for the current school year:

David B. Ratterman Fast Start Scholarships

$1,000 Award Recipients

- Carli Beveridge, University of Connecticut
- Breeanna Cash, Kansas City Kansas Community College
- Makaylah Howard, Mount Wachusett Community College

$2,000 Award Recipients

- Kaitlyn DeVos, Lake Area Technical Institute
- Carter Riechmann, Indian Hills Community College
- Shelby Snider, Lansing Community College
- Sean Templeton, Missouri State Technical College
- Sarah Unger *(not pictured)*, Blue Ridge Community College
- Jaelyn Walters, Casper College

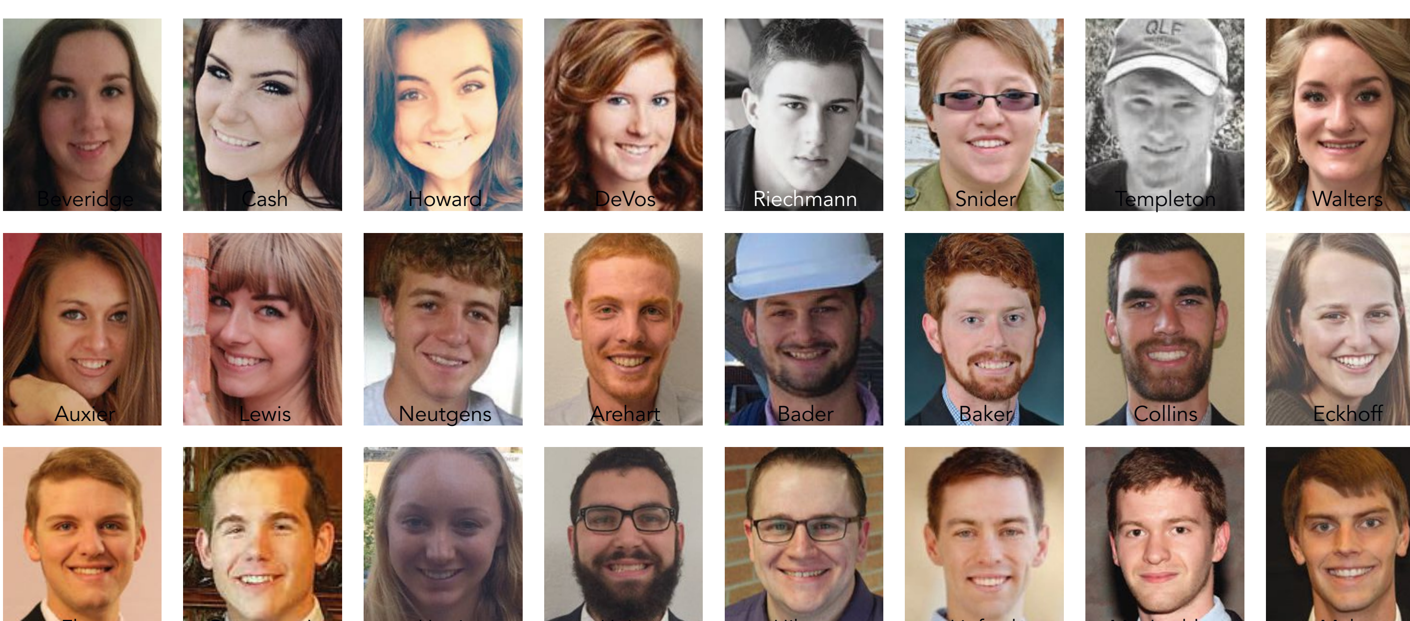
$5,000 Award Recipients

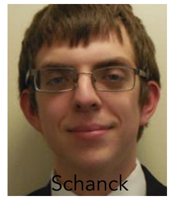
- Autum Auxier, Ball State University
- Serena Lewis, University of Denver
- Alberto Montemayor *(not pictured)*, Southern Methodist University
- Dylan Neutgens, Montana Tech
- Jailyn Smith *(not pictured)*, Mississippi State University

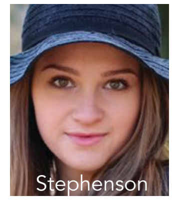
AISC Scholarships for Juniors, Seniors and Masters-Level Students

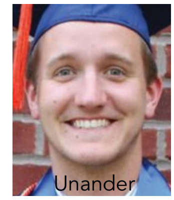
AISC Education Foundation

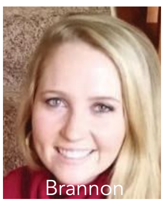
- Joseph Arehart, University of Colorado Boulder
- William Bader, University of Illinois at Urbana-Champaign
- Ethan Baker, University of Arkansas
- Leon Collins, The College of New Jersey
- Kathryn Eckhoff, Northwestern University
- Eric Fleet, Oklahoma State University
- George Grzywacz Jr., University of Michigan
- Anna Harris, Santa Clara University
- Nicholas Heim, Case Western Reserve University
- Derek Hibner, Michigan State University
- Sara Ibarra *(not pictured)*, University of Washington
- Jacob Linford, University of California, San Diego
- Duncan MacLachlan, The University of Kansas
- Jackson Mahrt, Arizona State University

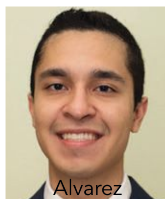


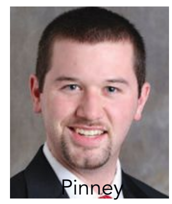


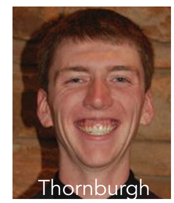


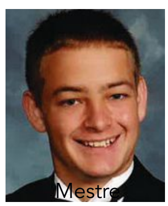


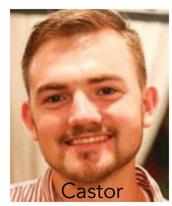


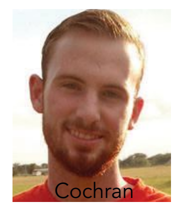


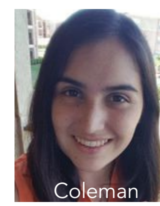












AISC Education Foundation *(cont.)*

- Caleb Mitchell, Kansas State University
- Jerren Paradee, University of Washington
- Lauren Santullo, The College of New Jersey
- Andrew Schanck, The University of Maine
- Melanie Stephenson, Colorado School of Mines
- Andrew Unander, Massachusetts Institute of Technology

AISC/Great Lakes Fabricators & Erectors Association

- Joseph Schmitt, Michigan Technological University

AISC/Ohio Structural Steel Association

- Kamron Skaggs, University of Cincinnati

AISC/Rocky Mountain Steel Construction Association

- Joseph Arehart, University of Colorado Boulder

AISC/Southern Association of Steel Fabricators

- Alison Brannon, University of Kentucky
- Jacob McCranie, Georgia Southern University

AISC/Associated Steel Erectors of Chicago

- Joseluis Alvarez, University of Illinois at Chicago
- William Bader, University of Illinois at Urbana-Champaign
- Kathryn Eckhoff, Northwestern University
- Bora Ozaltun, University of Illinois at Urbana-Champaign
- Joshua Pinney, Rose-Hulman Institute of Technology

AISC/Technical Committee on Structural Shapes

- Lauren Santullo, The College of New Jersey
- Andrew Schanck, The University of Maine
- Travis Thornburgh, University of Iowa

AISC/Indiana Fabricators Association

- Kristen Belyea, Rose-Hulman Institute of Technology
- Gerard Guell Bartrina, Indiana University-Purdue University Fort Wayne
- Madalyn (Maddy) Sowar, University of Notre Dame
- Megan Voss, Valparaiso University

AISC/W&W Steel/ Oklahoma State University

(Program includes sophomores, juniors and seniors)

- Matthew Mestre, Civil Engineering
- Kaylee Roper, Architectural Engineering
- Randall Castor, Construction Management
- Dillon Cochran, Civil Engineering
- Alexa Coleman, Architectural Engineering
- Lauren Breedlove *(not pictured)*, Civil Engineering
- Jose Reyna *(not pictured)*, Construction Management
- Kennedy Stephens, Architectural Engineering

The AISC Scholarship jury consisted of the following individuals:

- Benjamin Baer, Baer Associates Engineers, Ltd.
- David Bibbs, Cannon Design
- Christopher Brown, Skidmore Owings & Merrill, LLP
- Christina Harber, AISC
- Luke Johnson, American Structurepoint
- Colleen Malone, H.W. Lochner, Inc.

The David B. Ratterman Scholarship jury consisted of the following individuals:

- Brad Bourne, AISC Education Foundation Chair
- Lawrence Cox, AISC Board Member
- Babette Freund, AISC Board Member
- Lawrence Kruth, AISC Board Member
- Patrick Leonard, AISC Board Member
- Rex Lewis, AISC Past Chair
- David B. Ratterman, AISC General Counsel

NSSBC

2017 NSSBC Rules Posted

The rules for the 2017 ASCE/AISC National Student Steel Bridge Competition (NSSBC) are now posted at **www.aisc.org/steelbridge**. Oregon State University is set to host the national championship event May 26-27 in Corvallis, Ore.

This annual collegiate competition is an exciting visual display of engineering students' structural design and analysis skills at work. Throughout the academic year, student teams from across North America devote countless hours to designing, fabricating and constructing one-tenth-scale steel bridges. During competition, teams are challenged to assemble their bridge in the shortest time and under building constraints that reflect real-life structural specifications and construction regulations.

To reach the national event, each team must place among the top schools in one of 18 regional competitions held around the country in the spring. Bridge rankings are based on the categories of display, construction speed, stiffness, lightness, construction economy and structural efficiency.

See highlights from this year's NSSBC in the August article "Proven in Provo" (available at **www.modernsteel.com**).

ENGINEERING JOURNAL

Fourth-Quarter *Engineering Journal* now Available

The fourth-quarter 2016 issue of *Engineering Journal* is now available at **www.aisc.org/ej**. Articles in this issue include:

➤ **Stability of Rectangular Connection Elements**
Bo Dowswell

Connection elements are commonly designed using the flexural buckling and lateral-torsional buckling provisions in AISC *Specification* Sections E3 and F11, respectively, as well as the combined-load requirements of Section H1. Because these provisions were developed for main members, their accuracy for designing connection elements is questionable. The factors affecting the stability of connection elements are discussed, with an emphasis on the differences between main members and connection elements. The available experimental and theoretical results are compared to the AISC *Specification* equations. Where required, new equations are derived, and practical design solutions are recommended. Recommendations are also made for connection elements subjected to combined axial and flexural loads. Examples are provided to illustrate the proposed design procedures for double-coped beams.

➤ **Dynamic Shear Strength of Riveted Structural Connections**
Christopher P. Rabalais and C. Kennan Crane

Riveted lap-spliced specimens were tested to observe how the fasteners' shear strengths were affected by joint configuration, number of shear planes, and loading type. A 200,000-lbf-capacity dynamic loader was used to fail the specimens under a monotonic dynamic or monotonic quasi-static load. The test data were normalized by the number of shear planes loaded in each test and estimated ultimate tensile strength of the driven rivet. A statistical analysis was conducted to determine the significant factors affecting the fastener shear strength. Conclusions from the analyses indicated that the loading type has the most significant effect on shear capacity, resulting in a dynamic increase factor of 1.72 relative to the rivet's quasi-static shear capacity.

➤ **Updates to Expected Yield Stress and Tensile Strength Ratios for Determination of Expected Member Capacity in the 2016 AISC *Seismic Provisions***
Judy Liu

The expected yield stress and expected tensile strength ratios for hollow structural sections (HSS), pipe, and steel reinforcement for steel-concrete composite construction have been updated for the 2016 AISC *Seismic Provisions for Structural Steel Buildings*. For HSS, each grade of steel, including the new ASTM A1085 specification, has its own R_y and R_t values. Expected yield stress and tensile strength ratios have also been defined for different grades of steel reinforcement. The revisions were based on analysis of mill test data for HSS and pipe from a number of producers and a comprehensive mill survey conducted by the Concrete Reinforcing Steel Institute (CRSI).

➤ **Steel Structures Research Update Steel-Concrete Composite Beams at Ambient and Elevated Temperatures**
Judy Liu

Ongoing and recently completed research on steel-concrete composite beams and floor systems at ambient and elevated temperatures is presented. The research highlighted here includes investigations into shear connector slip, composite beams with high-strength steel, and tests of real-scale composite floor systems subjected to fire and structural loading.

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Ficep TIPO A31 CNC Drill & Thermal Cutting System, 10' x 20' x 5" Max. Plate, Ficep Minosse CNC, 2009 **#25937**
Controlled Automation ABL-100-B CNC Flat Bar Detail Line, 143 Ton Punch, 400 Ton Single Cut Shear, 40' Infeed, 1999 **#24216**
Controlled Automation 2AT-175 CNC Plate Punch, 175 Ton, 30" x 60" Travel, 1-1/2" Max. Plate, PC CNC, 1996 **#23503**
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**
Peddinghaus AFCPS 823/B CNC Anglemaster Angle Punch & Shear Line, 8" x 8" x 3/4", 130 Ton Punch, 400 Ton Shear, Marking Press, 1998 **#26594**

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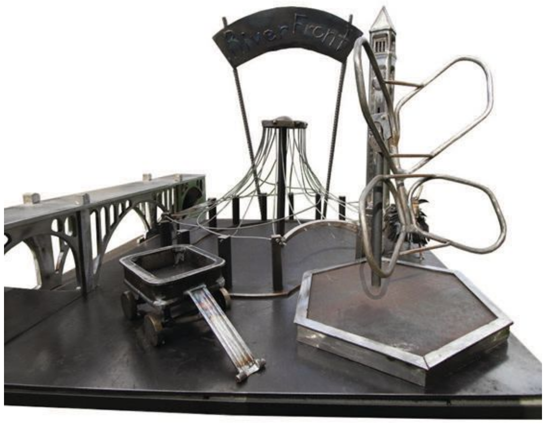
structurally sound

FINAL FIVE



AND THEN THERE WERE FIVE.

Nine creative steel objets d'art were submitted for this year's SteelDay Sculpture Competition. Votes were cast via Facebook (you can view all of the entries at **www.steelday.org/sculpturecompvoting**), and the five sculptures that received the most "Likes" are on their way to the 2017 NASCC: The Steel Conference, March 22-24 in San Antonio, where attendees will vote for their favorite. Clockwise from top-left, they are: *Riverfront Park* by Metals Fabrication, *The Galveston Bridge* by Hirschfeld Industries, *Man of Steel* by Universal Steel of North Carolina, *Steel... The Best Option* by Universal Steel (Buford, Ga.) and *Cube-X* by SteelFab TX, Inc.

Speaking of NASCC, this month's issue of *Modern Steel Construction* is bundled with the NASCC Advance Program, which provides details on sessions, exhibitors, keynote speakers, hotels, registration, tours and more. You can also visit **www.aisc.org/nascc**.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