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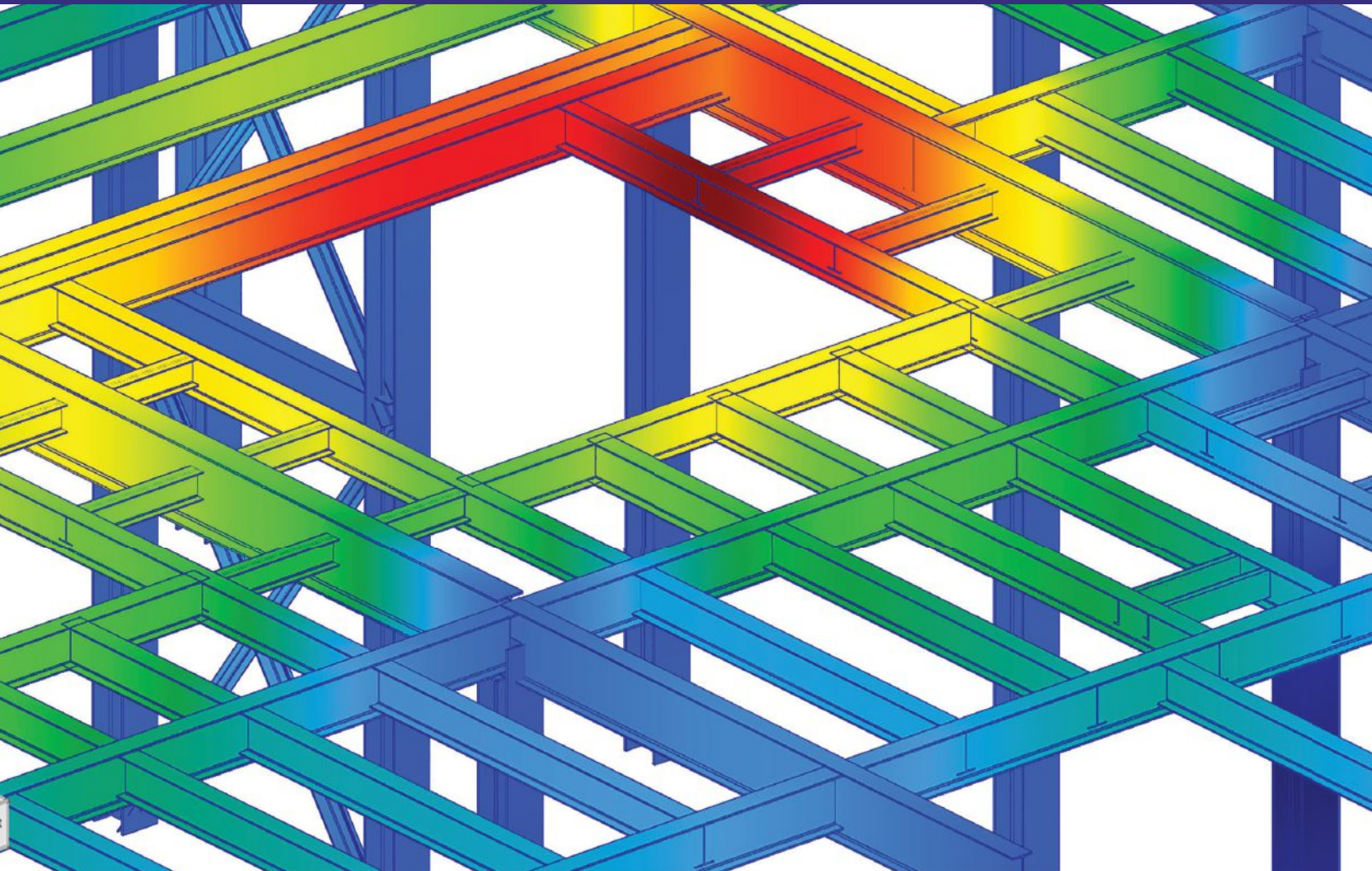
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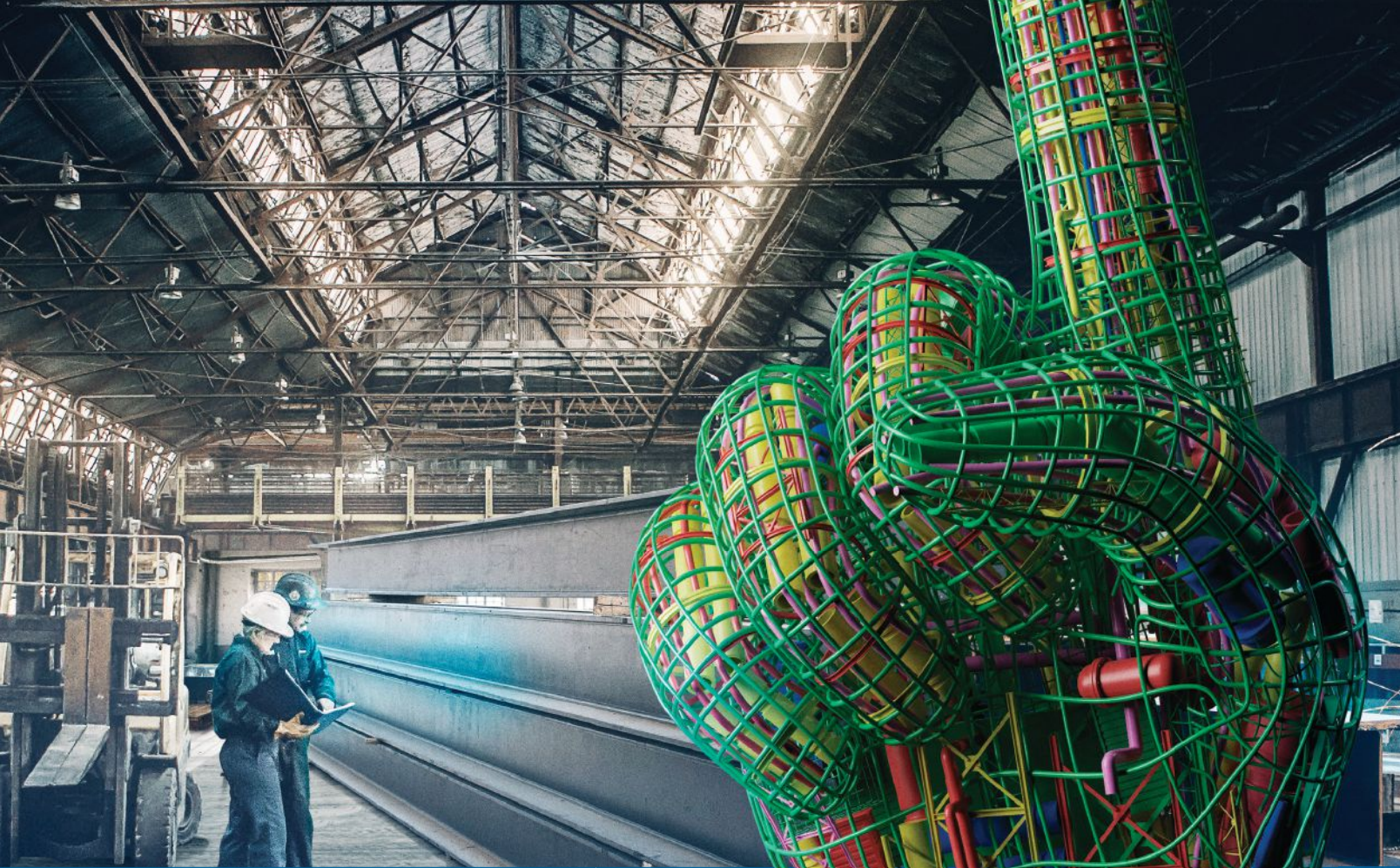
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editor's note



From conspiracy theorists to science deniers, the news is filled every day with people who seem to have their own versions of reality.

Almost daily, I'm irked by articles that parrot false narratives and ignore facts and detailed research. My personal news feed too often contains articles touting wood-framed construction as some sort of environmental savior while neglecting to consider the true environmental issues and negative impacts of wood harvesting and end-of-life issues.

In reality, unless a project-based, whole-building life-cycle assessment is performed, claims that one material is environmentally superior compared to another are worthless marketing hype. And when these studies are performed, wood often does come out looking good. For example, the book *Embodied Carbon in Buildings: Measurement, Management, and Mitigation* includes a chapter by Donald W. Davies, president of Magnusson Klemencic Associates, discussing the difference between various building materials. In summary, it states, "This paper does not show that a decisive GWP (global warming potential) winner can be chosen between the four different building frame options studied, based upon a material system choice alone." In addition, it advises that "designers should choose materials that are most materially efficient for the intended building use, and then optimize and economize the design to save on quantities while also finding ways to decrease the embodied carbon of that material choice."

AISC has produced a white paper (www.aisc.org/discover) that lays out the myths propagated by wood evangelists and provides a framework to evaluate environmental claims.

According to the white paper "Caveat Emperor: Sustainability, Wood, and the Environment," when evaluating wood you need to consider such factors as the tree species (there are substantial variations on biomass density from species-to-species), harvesting practices

(despite decades of effort, only 12% of harvesting practices are certified as being sustainably performed), the age of the trees harvested (seedlings and mature trees have different carbon uptake), the amount of waste (some studies have pegged the amount of a tree that ends up in a finished wood product as only about 36%), the amount of debris produced (which contributes to both CO₂ and NO₂), and the negative impact of burning wood waste (potentially more harmful than coal emissions).

The paper also advises taking a closer look at the environmental product declarations (EPDs) being touted by some wood proponents. As the Sierra Club so succinctly put it, "The primary purpose of current EPDs for wood appears to be to divert attention away from destructive forest management practices which cause disturbances to forests, streams, wetlands, and eliminates habitat for wildlife, all to sell more wood."

While not the subject of this particular white paper, the same critical eye should be used in examining wood's claims about fire resistance, cost, and speed of construction. Whether you're talking about heavy timber or cross-laminated timber (CLT), charring is simply a type of burning and all wood products need to be evaluated for the amount of potential fire load, the possibility of burn-through, and the impact of delamination.

I urge everyone to read AISC's white paper, share it with your colleagues and clients, and critically evaluate the claims made. As always, if you need more information, please contact AISC's Steel Solutions Center at solutions@aisc.org or 866.ASK.AISC.

A handwritten signature in black ink that reads "Scott Melnick". The signature is fluid and cursive, written in a professional style.

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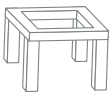
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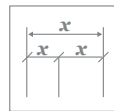
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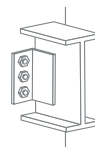
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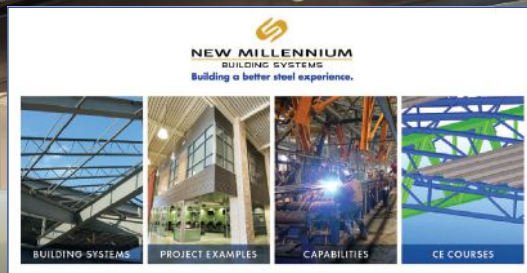
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steel interchange

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End-Loaded Connections

For end-loaded connections, footnote [b] from Table J3.2 in the AISC Specification for Structural Steel Buildings (ANSI/AISC 360) states that F_{nv} shall reduce to 83.3% of the tabulated values where connection lengths exceed 38 in. For a beam-to-girder connection where the top flange of the beam is coped, we are using a single-plate connection that exceeds 38 in. in length. Would this be considered an end-loaded connection? Would the 83.3% reduction addressed in Table J3.2 apply?

The 83.3% reduction would not apply to the case you have described as it would not be considered an end-loaded connection. The commentary to Section J3.6 provides guidance on end loaded connections that may be helpful. In addition, Figure C-J3.1 (provided below) looks at four different connection types and identifies those that are end-loaded and those that are not.

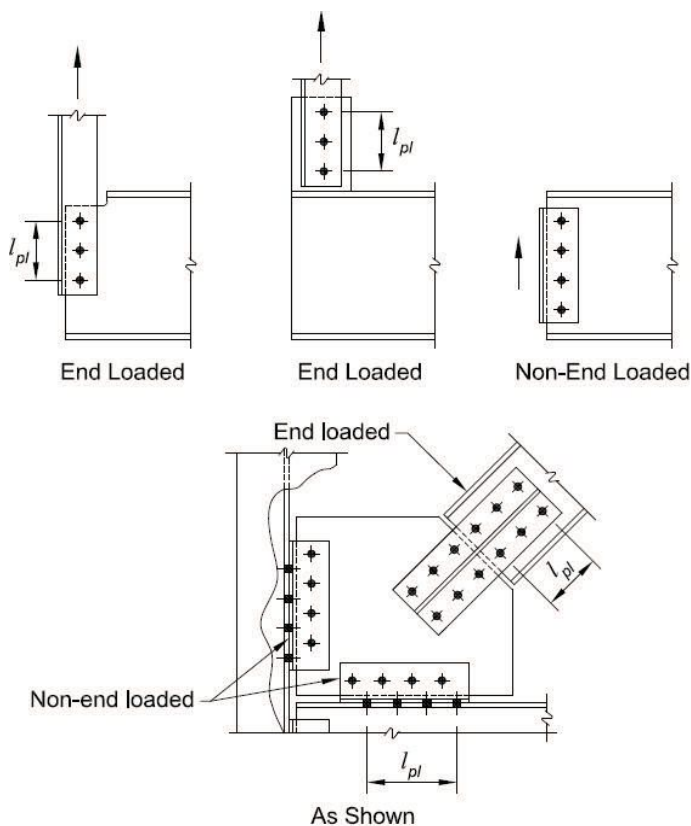


Fig. C-J3.1. End-loaded and non-end-loaded connection examples; l_{pl} = fastener pattern length.

One way to look at it might be to take a cut right before the first bolt and the last bolt in the group. If the force at the cut is equal to the full load transferred by the bolt group, then the connection is end-loaded. If the force at the cut is close to zero, as it would be for a typical beam end shear connection, then the connection is not end-loaded. There may be conditions between these extremes for which additional judgment must be exercised.

Taking as an example the Figure C-J3.1(a) and C-J3.1(c) "Why one is an End-Loaded and the other one a Non-End-loaded type?" I provide some thoughts for each detail (a) – (c).

Detail (b): If I apply my criteria described above and cut the angle (hanger) just above the first (topmost) bolt, the force at the cut is the full load transferred. If I cut the plate just below the last (lowest) bolt, the force at the cut is also the full load transferred. This indicates that the connection is end-loaded.

Detail (c): If I apply my criteria described above and cut the angle just above the first (topmost) bolt, the force at the cut is essentially zero. If I cut the angle just below the last (lowest) bolt, the force at the cut is also essentially zero. This indicates the connection is not end-loaded. The flow of shear force from the beam or girder web to the bolt group is essentially uniform throughout the connection length.

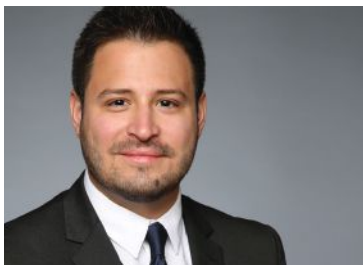
Detail (a): If I apply my criteria described above and cut the angle (hanger) just above the first (topmost) bolt, the force at the cut is the full load transferred. However, if I cut the beam web just below the last (lowest) bolt, the force at the cut will be only a small portion of the total load transferred by the connections. This likely puts the distribution somewhere between that of details (b) and (c). It is more conservative to consider the condition end-loaded and potentially unsafe to assume the more uniform distribution. Therefore, the recommendation in the Commentary is to treat it as end-loaded. Ultimately, you must use your own judgment.

Larry S. Muir, PE

Curved Member Distortion

I have a project where I will need to curve some wide-flange members (W5 and W6 section sizes) the hard way. We are being told that the members will distort. This is not an issue visually since the members will be cladded. However, what effect would the deformation have on the strength of the member?

Because all curved members distort to some degree during the bending operation, it is a good idea to establish a tolerable amount of distortion before bending. It is always best to consult with a bender-roller company prior to specifying curved members. Bender-roller companies specialize in curving steel members, and a list of bender-rollers is provided at www.aisc.org/benders. The typical types of distortion and their causes are discussed in Chapter 3 of AISC Design Guide 33: *Curved Member Design*. Bending of



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steel members cold-works the member, which causes the yield stress to increase. This is discussed in Chapter 5 of the Design Guide.

Excessive cross-sectional distortion caused by the bending operation can reduce the local buckling strength of the distorted element (flange or web). Section 5.5 of the Design Guide provides a method to calculate the strength reduction, if any, caused by this distortion. Also, I presented a session at the 2019 NASCC: The Steel Conference—"Distortion of Curved Members"—and went through a design example illustrating the use of Design Guide Equations 5-5 and 5-6 to check distortion. You can view the presentation www.aisc.org/2019nasconline.

Bo Dowswell, PE, PhD

WT Shapes Bent About the Weak Axis

How do I determine the available weak axis bending strength of a WT shape?

Section F9 of the AISC *Specification* addresses the design of WT shapes for flexure but only when loaded in the plane of symmetry. While the *Specification* does not address the strength determination about the weak axis, the Commentary provides guidance, stating: "Flexure about the y-axis of tees and double angles does not occur frequently and is not covered in this *Specification*. However, guidance is given here to address this condition. The yield limit state and the local buckling limit state of the flange can be checked by using Equations F6-1 through F6-3. Lateral-torsional buckling can conservatively be calculated by assuming the flange acts alone as a rectangular beam, using Equations F11-2 through F11-4. Alternately, an elastic critical moment is given as: $M_e = \pi/L_b \sqrt{EI_x GJ}$ may be used in Equations F10-2 or F10-3 to obtain the nominal flexural strength."

Jonathan Tavarez, PE

Single-Plate Connection Design Procedure: Mandatory or Recommended?

The conventional configuration procedure for single-plate connections covered in Part 10 of the 15th Edition AISC *Steel Construction Manual* contains a maximum plate or web thickness limit in Table 10-9. Is this limit mandatory? Can I neglect this limit in cases where my calculated available strength exceeds the required strength?

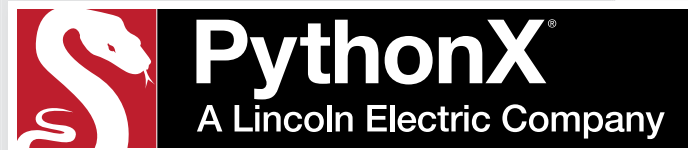
Before we get into a discussion on the thickness limit, let's first clarify that the AISC *Manual* does not provide requirements. It provides recommendations, guidance, and design aids. Therefore, the thickness limitation is not mandatory, nor is the use of the design procedure itself mandatory. Unlike the *Specification*, the *Manual* is not adopted into the building code. The August 2013 SteelWise article "Says Who?" provides more discussion on this. You can read it at www.modernsteel.com.

Now on to the question about the thickness limit. The conventional configuration procedure is based on a second-quarter 2011 AISC *Engineering Journal (EJ)* paper, "The Development of a New Design Procedure for Conventional Single-Plate Shear Connections." As discussed in that paper, the plate (or web) thickness limitation is intended to limit the strength and stiffness of the plate (or web) and allow the bolts to plow through the material to accommodate beam end rotation. For simple shear connections, the AISC *Specification* requires per Section B3.4a that a "simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure."

The authors of the paper believe that the thickness limitation is one way of satisfying this AISC *Specification* requirement. The AISC *Manual* is recommending the use of the conventional configuration procedure. This recommended procedure uses a thickness limitation as a way of satisfying a requirement (Section B3.4a) in the *Specification*. There is nothing in the AISC *Specification* that would prohibit an engineer from taking a different approach to designing a single plate connection as long as the engineer can show that the requirement in Section B3.4a, and all other applicable requirements have been met.

Carlo Lini, PE

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steel quiz

This month's Steel Quiz is all about architecturally exposed structural steel (AESS). The answers can be found in the 2016 *Code of Standard Practice*, available for download at www.aisc.org/specifications.

- 1 What is the recommended minimum viewing distance for AESS Category 2?
- 2 What is the maximum weld projection size that is acceptable for butt- and plug-welded joints for Categories AESS 1 and 2?
- 3 **True or False:** Members fabricated of unfinished, reused, galvanized, or weathering steel that are specified as AESS 3 cannot have erection marks, painted marks, or other marks on the surface in the completed structure.
- 4 **True or False:** For Categories AESS 2, 3, and 4, seams of hollow structural sections (HSS) shall be treated so they are not apparent.
- 5 Which of the following is correct regarding curved structural AESS members?
 - a. The curvature tolerance for curved AESS members is not reduced from that used for curved non-AESS members.
 - b. The as-fabricated variation from the theoretical curvature for curved AESS member shall be equal to or less than the standard camber and sweep tolerances permitted for straight members in the applicable ASTM standard.
- 6 Which of the following is not listed in Table 10.1 as a characteristic for AESS 2?
 - a. Welds uniform and smooth.
 - b. Fabrication marks not apparent.
 - c. One-half standard fabrication tolerances.
 - d. Joint gap tolerances minimized.
- 7 The matching of abutting cross sections is required for which AESS categories?

The questions and answers were contributed by Yishan He, a graduate student at Northwestern University. Thank you, Yishan!

TURN TO PAGE 14 FOR THE ANSWERS

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steel quiz ANSWERS

- Greater than 20 ft.** Section 10.1.1 of the AISC 2016 Code defines the five categories of AESS. AESS 2 shall be specified for feature elements viewed at a distance greater than 20 ft.
- 1/16 in. (2 mm).** Section 10.4.9 provides requirements for butt and plug welds: For Categories AESS 1 and 2, weld projection up to 1/16 in. (2 mm) is acceptable for butt- and plug-welded joints. For Categories AESS 3 and 4, welds shall be ground smooth/filled.
- False.** Section 10.4.2 states that members fabricated of unfinished, reused, galvanized, or weathering steel that are to be AESS may still have erection marks, painted marks, or other marks on the surface in the completed structure. Special requirements, if any, shall be specified as Category AESS C.
- False.** Section 10.4.12 provides requirements for HSS weld seams: For Categories AESS 1 and 2, HSS seams shall be acceptable as produced. For Category AESS 3, seams shall be oriented as specified in the contract documents. For Category AESS 4, seams shall be treated so they are not apparent.
- c.** Both A and B are correct, which can be verified by section 10.4.4 and its commentary.
- d.** The requirement for minimized joint gap tolerances is listed in Table 10.1 for only Categories AESS 3 and 4.
- Abutting cross sections are required for **AESS Categories 3 and 4**. This information is in both Table 10.1 (Id 3.4) and Section 10.4.3 (a).

More information, as well as a sample specification for AESS, is available at www.aisc.org/aess.



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Changes to Chapter K of the 2016
AISC *Specification* improve consistency,
usability, and clarity—and provide a better
resource for HSS connection design.

steelwise
**CONNECTION
CONSISTENCY**

BY CARLO LINI, PE

ENGINEERS OPENING THE 2016 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) may notice some significant changes to Chapter K.

But don't panic! This article will walk you through the changes. Chapter K addresses the design of hollow structural section (HSS) and box member connections and is organized in a more useful manner. Chapter J was also revised to reflect the Chapter K changes. Among the reorganizational changes are:

- a new Section, K1: General Provisions and Parameters for HSS Connections
- significant reduction in the volume of material in Section K2 (formerly K1): Concentrated Forces on HSS
- significant reduction in the volume of material in Section K3 (formerly K2): HSS-to-HSS Truss Connections
- significant reduction in the volume of material in Section K4 (formerly K3): HSS-to-HSS Moment Connections
- a new subsection J10.10

These changes are intended to generalize some of the limit states that were formerly only applied to HSS connections and to improve the understanding of these limit states by highlighting the parallels between HSS connections and connections to wide-flange members, which have dominated our industry.

A majority of the changes affect only rectangular HSS connections. Only minor changes have been made to the provisions for round HSS connections.

Why Change?

The publication of Design Guide 24: *Hollow Structural Section Connections* (available at www.aisc.org/dg) caused the AISC Committee on Specifications to rethink some things. One consideration was consistency in terms of the type of information that should be presented in the *Specification* versus what should be presented in other documents. Some may recall that the 2005 *Specification* did not include the now familiar illustrated tables related to HSS connection design. Instead, the 2005 *Specification* presented only a sometimes seemingly endless stream of text and equations, which made it difficult to decipher. Recognizing the challenge to practicing engineers, both the 2010 *Specification* and Design Guide 24 presented illustrations of various connection configurations in a tabular format to try to more clearly describe the conditions being addressed. This represented a significant departure from the approach to connection design in Chapter J of the *Specification* not only in terms of the use of illustrations in place of text, but also the concentration on connection configuration as opposed to limit states. In some respects, the information in Chapter K of the 2010 *Specification* resembled information that would more commonly be included in a guide rather than a specification.

Redundancy also had to be addressed. Design Guide 24 and the 2010 *Specification* were developed in parallel. The design guide reproduced the *Specification* tables in their entirety. There is no reason to produce two documents containing the same information.

The final considerations—probably the most important—were usability and clarity. Though the illustrations and tabular format made the design of connections precisely matching the configurations shown very simple, they also sometimes led to confusion



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To access both the AISC *Specification for Structural Steel Buildings* and the AISC *Steel Construction Manual*, visit www.aisc.org/specifications.

TABLE K1.1
Available Strengths of
Plate-to-Round HSS Connections

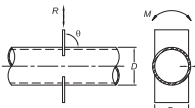
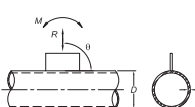
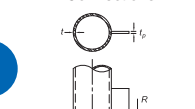
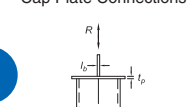
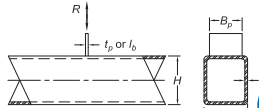
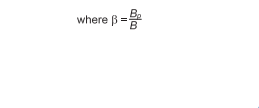
Connection Type	Connection Available Strength	Plate Bending	
		In-Plane	Out-Of-Plane
 <p>Transverse Plate T- and Cross-Connections</p>	Limit State: HSS Local Yielding Plate Axial Load $R_n \sin \theta = F_y t^2 \left(\frac{5.5}{1 - 0.81 \frac{B_p}{D}} \right) Q_t \quad \text{K1-1}$ $\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$	—	$M_n = 0.5 B_p R_n$
 <p>Longitudinal Plate T-, Y- and Cross-Connections</p>	Limit State: HSS Plasticification Plate Axial Load $R_n \sin \theta = 5.5 F_y t^2 \left(1 + 0.25 \frac{l_b}{D} \right) Q_t \quad \text{K1-2}$ $\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$	$M_n = 0.8 l_b R_n$	—
 <p>Longitudinal Plate T-Connections</p>	Limit States: Plate Limit States and HSS Punching Shear Plate Shear Load For R_n , see Chapter J. Additionally, the following relationship shall be met: $t_p \leq \frac{F_u}{F_{yp}} t \quad \text{K1-3}$	—	—
 <p>Cap Plate Connections</p>	Limit State: Local Yielding of HSS Axial Load $R_n = 2 F_y t (5 t_p + l_b) \leq F_y A \quad \text{K1-4}$ $\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$	—	—

TABLE K1.2
Available Strengths of
Plate-to-Rectangular HSS Connections

Connection Type	Connection Available Strength
 <p>Transverse Plate T- and Cross-Connections, Under Plate Axial Load</p>	Limit State: Local Yielding of Plate, For All β $R_n = \frac{10}{B/t} F_y t B_p \leq F_{yp} t_p B_p \quad \text{K1-7}$ $\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$
 <p>where $\beta = \frac{B_p}{B}$</p>	Limit State: HSS Shear Yielding (Punching), When $0.85B \leq B_p \leq B - 2t$ $R_n = 0.6 F_y t (2t_p + 2B_{sp}) \quad \text{K1-8}$ $\phi = 0.95 \text{ (LRFD)} \quad \Omega = 1.58 \text{ (ASD)}$
	Limit State: Local Yielding of HSS Sidewalls, When $\beta = 1.0$ $R_n = 2 F_y t (5k + l_b) \quad \text{K1-9}$ $\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$
	Limit State: Local Crippling of HSS Sidewalls, When $\beta = 1.0$ and Plate is in Compression, for T-Connections $R_n = 1.6 t^2 \left(1 + \frac{3l_b}{H - 3t} \right) \sqrt{E F_y} Q_t \quad \text{K1-10}$ $\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$
	Limit State: Local Crippling of HSS Sidewalls, When $\beta = 1.0$ and Plate is in Compression, for Cross-Connections $R_n = \left(\frac{48 t^3}{H - 3t} \right) \sqrt{E F_y} Q_t \quad \text{K1-11}$ $\phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$

and conflict about technical and contractual requirements for conditions which did not precisely match the included configurations. Many engineers assumed that all of the equations in Chapter K were empirical, though many can be readily derived and many are simply modified forms of limit states that have been presented in the *Specification* for decades.

It is hoped that the more general presentation, which is more consistent with Chapter J, will allow engineers to better understand and apply the limit states to rectangular HSS connections.

Where Did They Go?

This section reproduces some of the tables in 2010 *Specification* that were completely or partially removed with labels to sections in this article explaining how it can be addressed per the 2016 *Specification*.

1 Single-Plate to HSS Shear Connection – Punching Shear

Equation K1-3 in the 2010 *Specification* addressed punching shear at the HSS face. While this equation has been removed from the 2016 *Specification*, Equations 10-7a and 10-7b of the AISC *Steel Construction Manual* have been added to address punching shear at single-plate connections to HSS. Section J10.10 requires one to consider punching shear and plastification when transverse forces are applied to plate

elements. The *Manual* notes that yielding (plastification) of the HSS face “has not been a governing limit state in physical tests.”

It should be noted that while the 2010 check was previously described as applying to “Longitudinal Plate T-Connections,” it was intended to address single-plate shear connections. Also, while the previous check was based on the available strength of the single-plate shear connection, the new check is based on the estimated moment caused by the beam reaction. This should prove to be a welcomed change to the many who have struggled with this requirement in the past.

2 Local Yielding

For HSS sidewall local yielding the 2010 *Specification* equations, K1-9, K2-9, K3-7, and K3-10 can all be checked using Section J10.2 of the 2016 *Specification*.

2010 Spec. Eq. K1-9: $R_n = 2 F_y t (5k + l_b)$ 2010 Spec. Eq. K2-9: $P_n \sin \theta = 2 F_y t (5k + l_b)$ 2010 Spec. Eq. K3-7: $M_n = 0.5 F_y t (H_b + 5t)^2$ 2010 Spec. Eq. K3-10: $M_n = F_y t (B - t)(H_b + 5t)$	} } } }	2016 Spec. Section J10.2
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For equation K3-7, $(H_b + 5t)/2$ represents the moment arm and $2 \times (H_b + 5t)/2$ represents the length over which the force couple is applied for both HSS walls. For equation K3-10, $(B - t)$ represents the moment arm and $(H_b + 5t)$ represents the length over which the force couple is applied to the HSS chord wall.

While resembling a web local yielding check, the 2010 *Specification* equations K1-4, K1-14a, and K1-14b are used to address local effects, similar to those addressed by shear lag in Chapter D or the Whitmore section referred to in the User Note to Section J4.1. As noted in AISC Design Guide 24, “The design method for cap-plate connections to the ends of HSS members, where the axial force in a plate is transferred to the HSS via a cap plate (or via the flange of a tee-stub), recognizes that shear lag will be present in the HSS if some of the cross-section is not loaded.”

While these equations have been removed from the *Specification*, guidance on these checks can still be found in AISC Design Guide 24. The engineer should also note that such local effects must also be considered relative to the design of the weld as well.

3 Local Yielding of Transverse Plate T- and Cross-Connections

2010 *Specification* equation, K1-7 can be checked using equations K1-1 and J4-1 of the 2016 *Specification*.

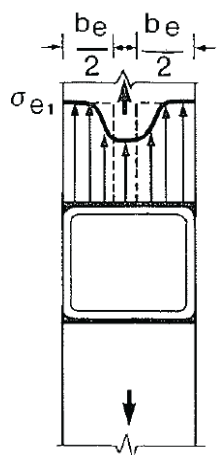
2010 Spec. Eq. K1-7: $R_n = \frac{10}{B/t} F_y t B_p \leq F_{yp} t_p B_p$

An effective width of the transverse plate needs to be determined due to the variable stiffness across the HSS wall. In some respects this is again similar to the local effects addressed in in Chapter D.

Effective Width

The stiffness of an HSS wall subjected to a transverse force is not constant. It is stiffer nearer to the sidewalls and less stiff near the center of the wall. The width of the wall of the main member, the widths, thicknesses and strengths of both the main member and the branch members determine the stiffness. This variation in stiffness will result in variations in stress for the main member, the branch member and the welds. The variation in stress is illustrated in Figure 1. The variations are accounted for through the use of an effective width, B_e .

2016 Spec. Eq. K1-1: $B_e = \left(\frac{10t}{B}\right) \left(\frac{F_y t}{F_{yt} t_b}\right) B_b \leq B_b$



In the 2010 *Specification*, this effective width was presented for each connection configuration sometimes in slightly different forms, though the model is essentially the same for all configurations.

It is worth spending some time with the equation.

Fig. 1. Effective width.

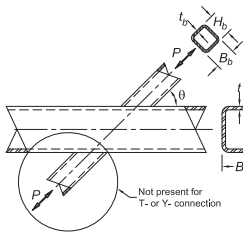
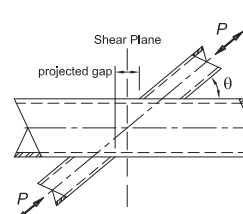
TABLE K1.2. (continued)
Available Strengths of
Plate-to-Rectangular HSS Connections

Connection Type	Connection Available Strength
Longitudinal Plate T-, Y- and Cross-Connections, Under Plate Axial Load 	Limit State: HSS Plastification $R_n \sin \theta = \frac{F_y t^2}{1 - \frac{t_p}{B}} \left(\frac{2l_b}{B} + 4 \sqrt{1 - \frac{t_p}{B}} Q_t \right) \quad (K1-12)$ $\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$
Longitudinal Through Plate T- and Y-Connections, Under Plate Axial Load 	Limit State: HSS Wall Plastification $R_n \sin \theta = \frac{2F_y t^2}{1 - \frac{t_p}{B}} \left(\frac{2l_b}{B} + 4 \sqrt{1 - \frac{t_p}{B}} Q_t \right) \quad (K1-13)$ $\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$
Longitudinal Plate T-Connections, Under Plate Shear Load 	Limit States: Plate Limit States and HSS Punching Shear For R_n , see Chapter J. Additionally, the following relationship shall be met: $t_p \leq \frac{F_u}{F_{yp}} t \quad (K1-3)$

TABLE K1.2. (continued)
Available Strengths of
Plate-to-Rectangular HSS Connections

Connection Type	Connection Available Strength
Cap Plate Connections, under Axial Load 	Limit State: Local Yielding of Sidewalls $R_n = 2F_y t (5t_p + l_b), \text{ when } (5t_p + l_b) < B \quad (K1-14a)$ $R_n = F_y A, \text{ when } (5t_p + l_b) \geq B \quad (K1-14b)$ $\phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)}$
	Limit State: Local Crippling of Sidewalls, When Plate is in Compression $R_n = 1.6t^2 \left[1 + \frac{8l_b}{B} \left(\frac{t}{t_p} \right)^{1.5} \right] \sqrt{EF_y \frac{t_p}{t}}, \text{ when } (5t_p + l_b) < B \quad (K1-15)$ $\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$

TABLE K2.2
Available Strengths of Rectangular HSS-to-HSS Truss Connections

Connection Type	Connection Available Axial Strength
T-, Y- and Cross-Connections 	<p>Limit State: Chord Wall Plastification, When $\beta \leq 0.85$</p> <p>7 $P_n \sin \theta = F_y t^2 \left[\frac{2\eta}{(1-\beta)} + \frac{4}{\sqrt{1-\beta}} \right] Q_t$ (K2-7)</p> <p>$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</p> <p>Limit State: Shear Yielding (Punching), When $0.85 < \beta \leq 1 - 1/\gamma$ or $B/t < 10$</p> <p>4 $P_n \sin \theta = 0.6 F_y t B (2\eta + 2\beta_{eop})$ (K2-8)</p> <p>$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</p> <p>Limit State: Local Yielding of Chord Sidewalls, When $\beta = 1.0$</p> <p>2 $P_n \sin \theta = 2 F_y t (5k + t_b)$ (K2-9)</p> <p>$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</p>
Case for checking limit state of shear of chord side walls 	<p>Limit State: Local Crippling of Chord Sidewalls, When $\beta = 1.0$ and Branch is in Compression, for T- or Y-Connections</p> <p>5 $P_n \sin \theta = 1.6 t^2 \left(1 + \frac{3t_b}{H-3t} \right) \sqrt{E F_y} Q_t$ (K2-10)</p> <p>$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)</p> <p>Limit State: Local Crippling of Chord Sidewalls, When $\beta = 1.0$ and Branches are in Compression, for Cross-Connections</p> <p>6 $P_n \sin \theta = \left(\frac{48t^3}{H-3t} \right) \sqrt{E F_y} Q_t$ (K2-11)</p> <p>$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</p> <p>Limit State: Local Yielding of Branch/Branches Due to Uneven Load Distribution, When $\beta > 0.85$</p> <p>8 $P_n = F_{yb} t_b (2H_b + 2b_{oc1} - 4t_b)$ (K2-12)</p> <p>$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</p> <p>where</p> <p>$b_{oc1} = \frac{10}{B/t} \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b$ (K2-13)</p>

The effective width equation can be rewritten in a form in which some of the relationships might be clearer:

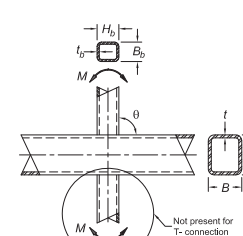
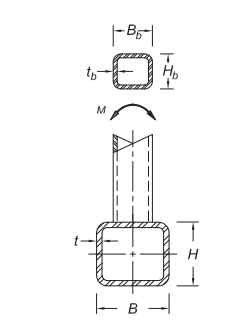
$$B_e = 10 \left(\frac{F_y}{F_{yb}} \right) \left(\frac{t^2}{t_b} \right) \left(\frac{1}{B} \right) B_b \quad (1)$$

It can then be rewritten in to produce a nominal strength:

$$R_n = F_{yb} B_e t_b = F_{yb} \left[10 \left(\frac{F_y}{F_{yb}} \right) \left(\frac{t^2}{t_b} \right) \left(\frac{1}{B} \right) B_b \right] t_b = 10 F_y t^2 \left(\frac{B_b}{B} \right) \quad (2)$$

The strength is therefore dependent on the yield strength of the main member, the thickness of the main member, and the ratio of the branch-width-to-main-member-width. 10 is an empirical coefficient. Note that the check is derived based on conditions where the width of the branch (or plate) and the width of the main member are similar.

Table K3.2
Available Strengths of Rectangular HSS-to-HSS Moment Connections

Connection Type	Connection Available Flexural Strength
Branch(es) under In-Plane Bending T- and Cross-Connections 	<p>Limit State: Chord Wall Plastification, When $\beta \leq 0.85$</p> <p>7 $M_n = F_y t^2 H_b \left[\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{(1-\beta)} \right] Q_t$ (K3-6)</p> <p>$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</p> <p>Limit State: Sidewall Local Yielding, When $\beta > 0.85$</p> <p>2 $M_n = 0.5 F_y t (H_b + 5t)^2$ (K3-7)</p> <p>$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</p> <p>Limit State: Local Yielding of Branch/Branches Due to Uneven Load Distribution, When $\beta > 0.85$</p> <p>8 $M_n = F_{yb} \left[Z_b - \left(1 - \frac{b_{oc1}}{B_b} \right) B_b H_b t_b \right]$ (K3-8)</p> <p>$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</p>
Branch(es) under Out-of-Plane Bending T- and Cross-Connections 	<p>Limit State: Chord Wall Plastification, When $\beta \leq 0.85$</p> <p>$M_n = F_y t^2 \left[\frac{0.5 H_b (1+\beta)}{(1-\beta)} + \sqrt{\frac{2 B B_b (1+\beta)}{(1-\beta)}} \right] Q_t$ (K3-9)</p> <p>$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</p> <p>Limit State: Sidewall Local Yielding, When $\beta > 0.85$</p> <p>2 $M_n = F_y t (B - t) (H_b + 5t)$ (K3-10)</p> <p>$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)</p> <p>Limit State: Local Yielding of Branch/Branches Due to Uneven Load Distribution, When $\beta > 0.85$</p> <p>8 $M_n = F_{yb} \left[Z_b - 0.5 \left(1 - \frac{b_{oc1}}{B_b} \right)^2 B_b^2 t_b \right]$ (K3-11)</p> <p>$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</p>

4 HSS Wall Punching Shear

2010 *Specification* equations, K1-8 and K2-8, addressing punching shear have been removed. Punching shear is addressed in Section J10.10 of the 2016 *Specification*. Section J10.10 refers to guidance provided in Part 9 of the 15th Edition *Manual*. A punching shear model (Equation 9-29) is provided in Part 9 of the *Manual* that could be used in place of K1-8 and K2-8.

$$\begin{aligned} & \text{2010 Spec. Eq. K1-8: } R_n = 0.6 F_y t (2t_p + 2B_{ep}) \\ & \text{2010 Spec. Eq. K2-8: } P_n \sin \theta = 0.6 F_y t B (2\eta + 2\beta_{eop}) \end{aligned}$$

↓
2016 Spec. Section J10.10

↓
15th Edition *Manual* Eq. 9-29: $R_n = 0.6 F_y t_p (2C_{eff} + 2L)$

5 HSS Sidewall Local Crippling

2010 *Specification* equations K1-10 and K2-10 addressing sidewall local crippling have been removed. Section J10.3 of the 2016 *Specification* is used to check this limit state. Note that Chapter K applies to HSS members and box sections of uniform wall thickness so the web to flange thickness ratios in Equation J10-4 drop out. By referring to Chapter J of the *Specification*, it should now more clearly be recognized that HSS members and box sections can be checked when a concentrated compressive force is applied at a distance from the member end that is less than $d/2$. This may not have been clear in the 2010 *Specification* since this condition was not specifically addressed in Chapter K.

$$2010 \text{ Spec. Eq. K1-10: } R_n = 1.6t^2 \left(1 + \frac{3l_b}{H - 3t} \right) \sqrt{EF_y} Q_f$$

$$2010 \text{ Spec. Eq. K2-10: } P_n \sin\theta = 1.6t^2 \left(1 + \frac{3l_b}{H - 3t} \right) \sqrt{EF_y} Q_f$$

$$2016 \text{ Spec. Eq. J10-4: } R_n = 0.8t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{EF_y} \frac{t_f}{t_w} Q_f$$

The web local crippling check in Section J10.3 is not directly applicable to check the local crippling strength of an HSS wall at a tee-stub cap plate connection (Equation K1-15) although it did serve as the basis for the check as indicated in the design guide. You can still refer to the guidance in AISC Design Guide 24 for guidance on performing this check. Note that the depth of the HSS member used in Section J10.5 is taken as the depth of the flat of the HSS section (typically $H - 3t$), as discussed in the commentary to Section J10.

$$2010 \text{ Spec. Eq. K1-15: } R_n = 1.6t^2 \left[1 + \frac{6l_b}{B} \left(\frac{t}{t_p} \right)^{1.5} \right] \sqrt{EF_y} \frac{t_p}{t} \rightarrow \text{Refer to Design Guide 24}$$

6 HSS Sidewall Compression Buckling

2010 *Specification* equations, K1-11 and K2-11, addressing HSS sidewall compression buckling, can be checked using Section J10.5 of the 2016 *Specification*.

$$\left. \begin{aligned} 2010 \text{ Spec. Eq. K1-11: } R_n &= \left(\frac{48t^3}{H - 3t} \right) \sqrt{EF_y} Q_f \\ 2010 \text{ Spec. Eq. K2-11: } P_n \sin\theta &= \left(\frac{48t^3}{H - 3t} \right) \sqrt{EF_y} Q_f \end{aligned} \right\} \rightarrow \text{2016 Spec. Section J10.5}$$

7 HSS Wall Plastification (Yield-Lines)

2010 *Specification* equations K1-12, K1-13, K2-7 and K3-6 can be checked using Section J10.10 of the 2016 *Specification*.

$$2010 \text{ Spec. Eq. K1-12: } R_n \sin\theta = \frac{F_y t^2}{1 - \frac{t_p}{B}} \left(\frac{2l_b}{B} + 4 \sqrt{1 - \frac{t_p}{B}} \right) Q_f$$

$$2010 \text{ Spec. Eq. K1-13: } R_n \sin\theta = \frac{2F_y t^2}{1 - \frac{t_p}{B}} \left(\frac{2l_b}{B} + 4 \sqrt{1 - \frac{t_p}{B}} \right) Q_f$$

$$2010 \text{ Spec. Eq. K2-7: } P_n \sin\theta = F_y t^2 \left[\frac{2\eta}{(1 - \beta)} + \frac{4}{\sqrt{1 - \beta}} \right] Q_f$$

$$2010 \text{ Spec. Eq. K3-6: } M_n = F_y t^2 H_b \left[\frac{1}{2\eta} + \frac{2}{\sqrt{1 - \beta}} + \frac{\eta}{(1 - \beta)} \right] Q_f$$

2016 Spec. Section J10.10

15th Edition *Manual* Eq.: 9-30 and 9-32

Take It to the Limit

As stated previously, engineers often struggled to address conditions which did not precisely match the configurations shown in the 2010 Chapter K tables. Many engineers assumed that other configurations were prohibited, sometimes citing the User Note language, "Connection parameters must be within the limits of applicability." Instead the User Note was intended to emphasize that the tables could only be considered self-contained when addressing conditions matching the configurations shown and within the limits of applicability listed. Other conditions were left entirely to engineering judgment.

The 2016 *Specification* states: "Connections not complying with the limits of applicability listed are not prohibited and must be designed by rational analysis. This is consistent with language in Section A1 which states: "Where conditions are not covered by this *Specification*, designs are permitted to be based on tests or analysis, subject to the approval of the authority having jurisdiction. Alternative methods of analysis and design are permitted, provided such alternative methods or criteria are acceptable to the authority having jurisdiction."

The limit states related to concentrated forces provided in Section J10 have long been applied to wide-flange member outside of the limits such as those were provided in Chapter K of the 2010 *Specification*. Hopefully, it is now clearer that such limits need not constrain the application of these limit states.

The plastification (yield line analysis) and punching shear checks are also based primarily on rational models to which engineers can apply their own judgment. While the plastification checks previously provided applied only to a limited range of conditions (i.e. HSS and box members with CJP groove welds in the connection region with the member centerlines lying in a common plane), the checks can and have been applied to a wide range of conditions and the more general presentation in the 15th Edition *Manual* should prove helpful to engineers.

This is not to say that the limits of applicability are not useful. Some of the design equations in Chapter K, including those related to K-Connections, are empirical in nature and therefore tied to the limits of testing. Maintaining the limits of applicability will also tend to limit deformations to reasonable levels. Finally, limits of applicability can facilitate the development of "canned" connections for which only a small number of limit states need to be considered and for which little in the way of engineering judgment is required. It is likely that the updated Design Guide 24 will provide these sorts of "canned" connections, which are both easily designed and comply with the *Specification*.

steelwise

Section J10.10 refers to guidance provided in Part 9 of the 15th Edition *Manual*. Equations 9-30 and 9-32 provided in Part 9 of the *Manual* can be used to check HSS wall plastification and will provide the same results as the equations that were removed from Chapter K. The equations provided in Part 9 of the *Manual* allow for the calculation of strength for branch members located on and off the chord centerlines providing designers with some additional flexibility. Note that the 2010 *Specification* equations only addressed HSS branches located on the chord centerline.

The equation for through plate connections is taken as twice the value calculated when connecting to only one HSS wall. Some engineers have expressed concern about this approach arguing that a fillet weld at the through plate to HSS wall location should not be considered “clamped” but instead be treated as “pinned.” Part 9 of the 15th Edition *Manual* does not address a pinned condition at the through plate to HSS wall interface. The February SteelWise article “Stated Limits” (available at www.modernsteel.com) provides guidance on performing yield-line analysis that can help address these types of situations.

8 Local Yielding of Branch Due to Uneven Load Distribution

2010 *Specification* equation K2-12, K3-8, K3-11 can be checked using equation K1-1 in the 2016 *Specification*. Equation K1-1 can

be used to determine the effective width (see earlier discussion on effective width) and the resulting section properties combined with Sections J4.1 and J4.5 to determine the available strength.

$$2010 \text{ Spec. Eq. K2-12: } P_n = F_y b t_b (2H_b + 2b_{\text{eff}} - 4t_b)$$

$$2010 \text{ Spec. Eq. K3-8: } M_n = F_y b [Z_b - (1 - \frac{b_{\text{eff}}}{B_b}) B_b H_b t_b]$$

$$2010 \text{ Spec. Eq. K3-11: } M_n = F_y b [Z_b - 0.5(1 - \frac{b_{\text{eff}}}{B_b}) B_b^2 t_b]$$



2016 Spec. Section J4.1 and J4.5 and Spec. Eq. K1-1

Summary

This article is intended to clarify that though significant re-formatting has occurred, the actual limit states that must be considered and the design procedures applicable to HSS connections have changed relatively little. All of the information that Chapter K intended to address in 2010 is still addressed in Chapter J, Chapter K, Parts 9 and 10 of the *Manual*, and Design Guide 24.

The changes made to Chapter K of the 2016 *Specification* may initially take some time getting used to. But hopefully engineers will see the benefits of these updates, which were made to improve usability and clarity. Additionally, AISC Design Guide 24 is currently in the process of being updated and will continue to serve as a great resource for engineers designing HSS connections. ■

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Thoughts (from a long-time detailer) on how the steel detailing profession has changed over the years and how it fits into today's fabrication process.

business issues

THE EVOLUTION OF DETAILING

BY MARK TURMAN

DETAILING IS ONE OF the most important and fascinating positions in the fabrication industry today.

Its evolution over the past several years has changed the way fabricators think about not only detailing but also how steel is fabricated. The “detailer,” if that is even a relevant term in today's industry, has become so much more than an employee putting lines on paper. They have become an integral part of every facet of the fabrication process.

When I was hired as a detailer many years ago, I worked in a drawing room with six to eight other people, sitting in front of a drafting table, putting lines on paper, and making details for fabrication. The only concern I had was to meet an approval or fabrication deadline. Detailing, like so many things in life, was much simpler then.

A detailer's responsibility was to generate drawings, not fight for information, coordinate design documents, attend meetings, review download files for the shop, etc. The detailer was given a schedule that was realistic and with information that was complete, so they could just put their head down and go to work. Making drawings was their only responsibility, and today it is still an important part of a detailer's job—but now there is so much more. With today's innovations in detailing technology, the job has evolved well beyond what I could have imagined when I started in the profession.

Today's “detailers” are not sitting in a room with other detailers in white shirts, face to a board. Rather they are sitting at home in shorts and T-shirts, at a computer, looking at three or four monitors while generating 3D models. The modeling software has allowed, (forced?) the detailer into a world that far exceeds the simple world we once knew.

Detailing software has put much more information in detailers' hands. This information is allowing detailers to share more than just drawings to help in fabrication. Sure, models “automatically” generate details, plans, and sections, but the transfer of information through associated files has changed how the detailer is used. Sounds like a detailer's life is getting easier by the minute, right? But the amount of information that can be generated by the software has grown the detailer's role to include many different facets of fabrication.

The information detailers can generate through models has made them the lifeline of fabrication. Not only are the fabricators requesting more, but detailers continually must manage incomplete information on contract documents, making their position even more crucial.

Construction documents in today's world sometime have a lot to be desired in the form of information needed to complete fabrication details. Owners and developers are constantly trying to cut costs and expedite projects. In many cases, this leaves the design team no choice but to release drawings sooner than they would normally like in order to maintain the schedule.

That puts an added burden on the fabricators and detailers. The detailers have unknowingly been given the responsibility to check contract drawings while also being handed the responsible of coordinating steel with other trades. Many projects today require information transfer through 3D model coordination meetings, putting the responsibility on the detailer to generate a model. They are also often asked to attend building information modeling (BIM) meetings and adjust their model if any conflicts arise with other con-



Mark Turman (m.turman@snjs.org)

is a detailing manager with Southern New Jersey Steel Company, Inc., an AISC member fabricator.

With today's innovations in detailing technology, the job has evolved well beyond what I could have imagined when I started in the profession.

business issues

struction elements. This process continues throughout most projects, even beyond when steel is already erected. This keeps the detailers involved throughout the building process, not just the steel detailing portion.

The days of calling a design team to get an answer are long behind us, and today's fabricators are required to generate written RFIs when there is a conflict between trades

that cannot be worked out in a coordination meeting with the design team. RFIs are needed when additional information that may not be provided on contract documents is required. This responsibility is put on the shoulders of the detailers, who are tasked with generating questions—often along with snapshots to forward to the general contractor. Most detailers are required to keep a RFI

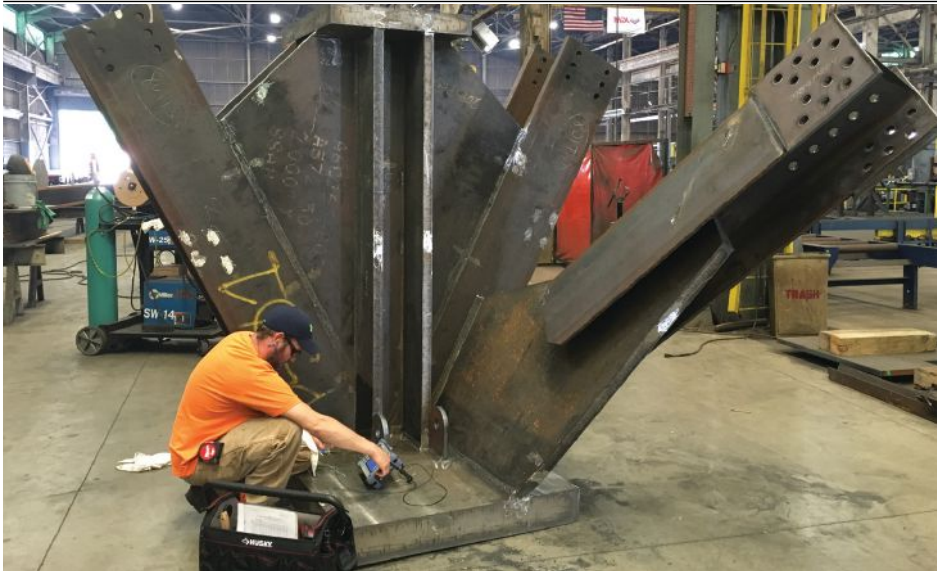
log current for all open and closed questions. Often, fabricators request a list of materials being held due to outstanding questions, requiring the detailer to coordinate these materials and questions so the fabricator can discuss what is needed and what is being affected with the general contractor.

Today's projects are a revolving door for changes, with many of these changes impacting the cost of the structural and miscellaneous steel package. It is typically the fabricator's project manager's responsibility to price and submit changes for additional cost and time to the general contractor. And project managers today often request the detailer to be a part of this process by documenting any changes to the contract documents via revisions or approval comments. Detailers are asked to provide information that may impact the schedule or cost for detailing as well as a list of all added or changed materials. The detailer's role plays a vital part of getting the change order process started. Though it is not the responsibility of the detailer to write change orders to the contract, it is their responsibility to provide the requested information to either the project manager or the estimator.

It's clear that detailers' responsibilities do not end with drawings, and many fabricators request CNC files for their automated shop equipment. The files are generated through detailing software and allow fabrication equipment to work more efficiently to drill and punch with fewer mistakes. As such, the detailer is more crucial to the shop than ever.

It is easy to see that today's structural detailers have evolved beyond drawing lines and adding numbers. They have become an important part of all aspects of the fabrication industry. It's a far cry from years gone by, and thankfully detailers have embraced the added responsibilities and pushed forward too.

Finally, keep in mind that the fabricator-detailer relationship is a partnership—with both entities working with, not against, each other, and the fabricator should push design teams for information and involve the detailer early in the planning stages. Remember: We are all working toward the same goal, and we need to work together to achieve success! ■



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This information was covered in the 2019 NASCC: The Steel Conference session "Detailing: It's Not Just that Anymore." You can view a recording of the session at www.aisc.org/2019nasconline.

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Exposed steel creates visual drama in a Miami mall expansion—and plays a pivotal role in the addition of a second level mid-construction.

Turnberry Associates



BY WILLIAM R. O'DONNELL, PE, CARLOS A. TURIZO, PE, AND JOSE TELLEZ

AS BRICK-AND-MORTAR retail space continues to lose ground to online shopping, the pressure is on for retail projects that do move forward to make their spaces as attractive, diverse, and useful as possible.

A newly expanded Miami mall has done just that. Today, Aventura Mall is the third-largest shopping mall in the U.S. The mall has more than 2.7 million sq. ft of retail, dining, and recreational area, and also serves as a transportation hub for both Miami-Dade Transit and Broward County Transit services. Recently, the mall also underwent a three-story, 315,000-sq.-ft expansion.

So how did the mall reach its current, massive size? After considering initial site proposals, owner Turnberry Associates and developer Simon Property decided to expand into vacant land adjacent to the mall and challenged the design team of structural engineer DeSimone and architects Carlos Zapata and JPRA to conceive a monumental structure with soaring cantilevers, an ultra-modern façade, multiple glass skylights, and a giant window wall to promote the flow of natural light.

As retail development is exceptionally schedule-driven, leases are typically negotiated far in advance and involve strict deadlines for delivering tenant space and penalties for dates missed. That considered, structural steel was selected as the optimal framing material to achieve fast erection, the long span, and large cantilevers to meet the architectural vision.

Carlos Zapata, the project's design architect, conceived the expansion as a V-shaped building joined by an open "valley" space. Two wings emanate from this point, creating a gradually widening, open concourse with steel-supported skylights and clerestory windows that provide natural light. The two diverging branches are connected via multiple steel pedestrian bridges in spans of 60 ft to 80 ft. In addition to retail and dining components, a luxe concierge level welcomes VIP shoppers to the mall.

A 51-ft-tall by 80-ft-wide glass wall, supported by steel framing designated as architecturally exposed structural steel (AESS), encloses the northeast end of the concourse. AESS Category 2—feature elements viewed at a distance greater than 20 ft (6 m)—was specified for these components.

The glass wall incorporates a convex curve that blends into the curved roof, and a 34-ft section of the glass wall is bracketed off a bridge spanning across the concourse just inside the glass. This enabled a transparent ground-level entrance into the mall below a glass canopy that cantilevers outboard from the glass wall.

The design team maintained an extremely aggressive schedule to deliver an initial steel mill package (4,000 tons of structural steel were used in all) and drawings followed shortly after to allow detailing and foundation construction to begin. As the schedule eventually overlapped erection with design, the project team made budget allowances from the design development phase through



DeSimone Consulting Engineers

opposite page: The mall expansion features two diverging branches split by a “valley” over which span multiple steel pedestrian bridges—all topped by a nearly full building-length and gradually widening steel-framed skylight.

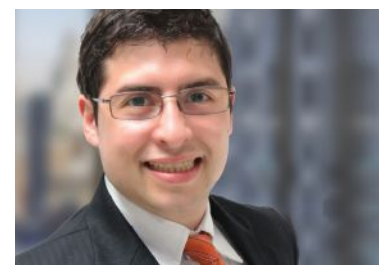
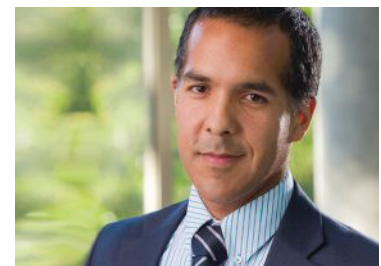
above: The valley is indicated by the skylight ridge in the center of the building.

construction planning. These allowances facilitated the development of miscellaneous steel details towards the end of the construction phase, which included perimeter steel supports for all of the façade elements.

Adding to the list of challenges, the owner required a majority of the existing mall to remain open during the construction phase. Temporary walls capable of withstanding tropical storm-force winds were installed to enable commencement of demolition and alterations necessary for the future connection. Columns that would have been in the middle of the connecting concourse had to be removed by demolishing and reframing the area. This phase of the project began with the construction of new access corridors and the erection of a new infill frame on the existing side of the mall to connect the two buildings. The steel framing consisted primarily of cantilever frames attached to the existing frames with moment connections and deep beams measuring W27 and larger. The beams joined the two main corridors and supported new escalators to create a seamless transition to the expansion.

Although foundations were already installed and structural steel fabricated, the owner requested an additional retail level on the east wing—while construction was ongoing. This new level would comprise a composite deck system with rigidly connected frames as the lateral system, and a design that was structurally identical to the levels below. However, due to the fact that the first level was already erected, columns on this existing level were reviewed to provide increased capacity for the gravity and lateral-load system moment frames. A portion of the erected columns required reinforcing to increase axial capacity. Roof framing that had already been fabricated was simply shifted one level higher, with the columns being reinforced with cover plates and the steel on the upper level being supported by full-penetration welding shafts to the lower sections.

The expansion includes special architectural features requiring detailed design and analysis, as well as a creative application of steel design principles. Some of these steel-framed features included an interior curved skylight over the main concourse; a monumental staircase that combined a concrete sculptural plinth with a cantilevered deck, extending 40 ft beyond its supports, to create an observation deck and porte cochère;



William R. O'Donnell is a partner and managing principal, **Carlos A. Turizo** is a managing principal, and **Jose Tellez** is a project engineer, all with DeSimone Consulting Engineers.

and a pedestrian bridge, covered with an architectural canopy, spanning 80 ft to the adjacent parking garage.

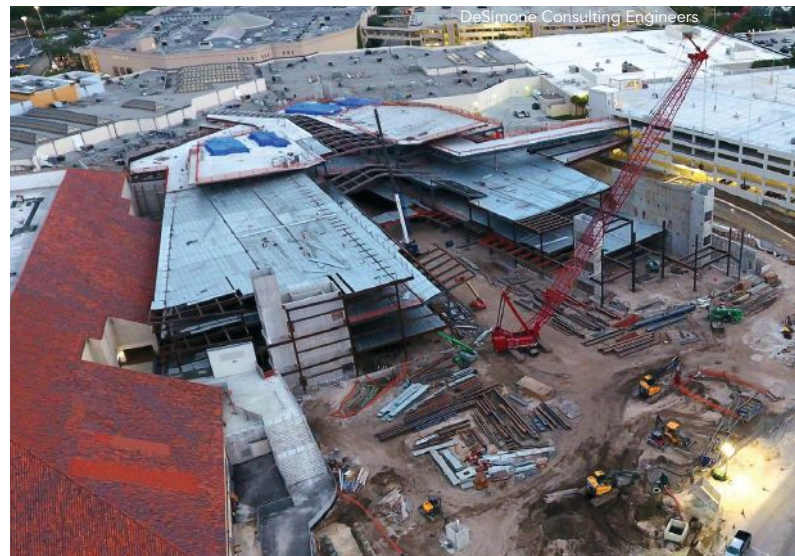
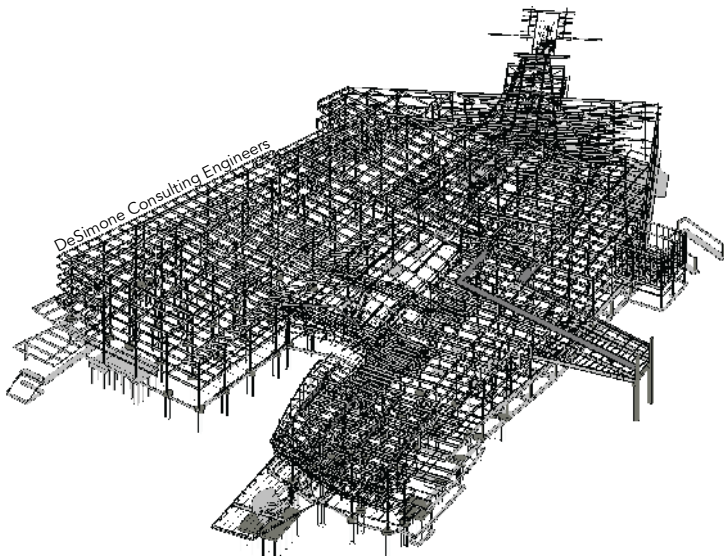
The steel-framed roof enclosure above the concierge area is complex in shape and features a double curvature interrupted in the middle by a depressed mechanical zone. The pronounced architectural curvature could not be achieved through commonly used hot-rolled shapes, so the solution was to assemble built-up beams using individual plates that were bent to achieve the desired curvature, then shop-welded to create the structural elements.

The typical deck system consists of wide-flange beams that support the concrete slab on steel deck. Headed shear studs welded through the metal deck to the beams create the composite beam action, resulting in better deflection control and more economical beams. The gravity load-resisting system uses standard composite beams connected to their supporting girders through shear connections, and the curved skylight structure is supported by a series of transfer elements and special connections that enable thermal expansion and contraction of the system spanning across the main



Harrison Boyce

A 51-ft-tall by 80-ft-wide glass wall, supported by steel framing designated as AESS, encloses the northeast end of the concourse. Built-up beams using individual plates form the roof elements.



The 315,000-sq.-ft expansion—shown as a structural model and under construction—connects the existing mall with a parking garage.

concourse. The lateral system used a combination of steel moment frames and concrete shear walls, and the lateral-resisting system elements were strategically located to accommodate large floor and roof openings, creating diaphragm discontinuities throughout the building.

A 3D modeling approach was crucial to the success of the project and allowed for a more detailed exchange of information between the architecture and engineering teams. The interface between the skylight and the deck levels consists of sloping frames that attach to the curved HSS members to form the skylight supporting frame. The nature of the architectural design was markedly intricate at this level, and the 3D models were instrumental in the development of a suitable framing system. This information also assisted in developing the supporting steel framing by tracking load paths and working out element connectivity based on the architectural proportions and spaces.

In the end, the expansion was erected in 40 weeks including the time required to provide the additional level mid-stream, and tenants moved in prior to the Thanksgiving shopping season, satisfying both ownership and retailers. ■

Owner

Turnberry Associates, Miami

General Contractor

Plaza Construction, Miami

Design Architect

Carlos Zapata Studio, New York

Architect of Record

JPra Architects, Farmington Hills, Mich.

Structural Engineer


DeSimone Consulting Engineers, New York

Connection Designer


Drucker Zajdel Structural Engineers, Inc., Chicago

Steel Team


Fabricator

Steel Fabricators, LLC,  Fort Lauderdale, Fla.


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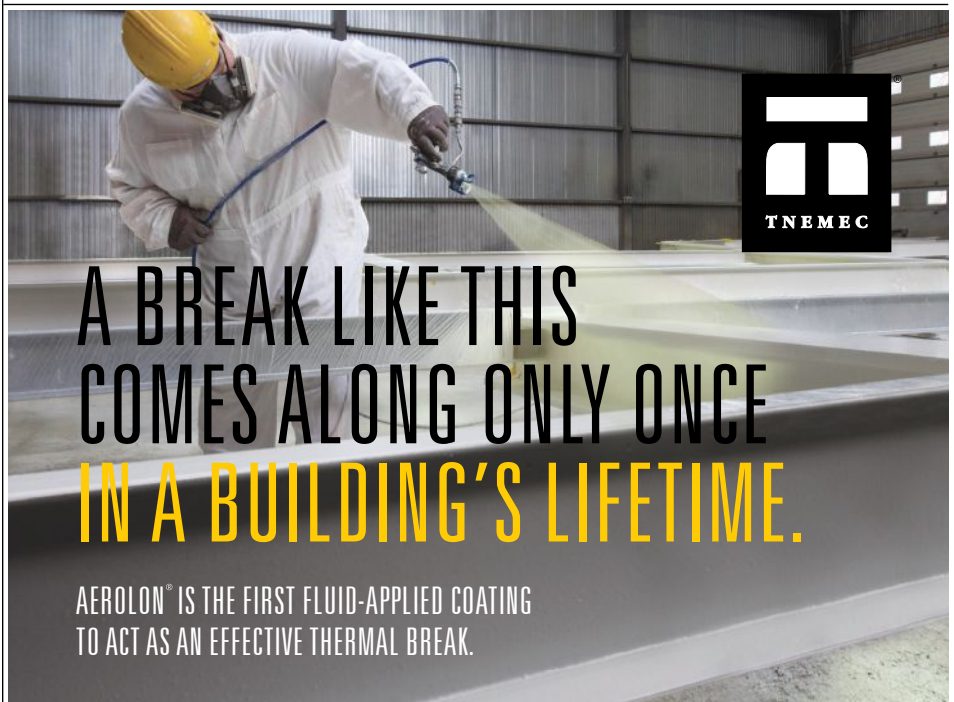
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Scientific Method

BY MARK KANONIK, PE, AND
CODY MESSIER, SE, PE



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Steel framing forms the nucleus of an expanding science complex at a small liberal arts college.

UNION COLLEGE HAS BEEN committed to the sciences for more than two centuries.

Founded in 1795, the school has a long history of rigorous science and engineering programs, particularly computer science in recent decades. In the late 1950s, the college reinforced its commitment to the sciences in the form of the 125,000-sq.-ft Science and Engineering Building, comprised of five interconnected “academic towers” housing multiple engineering disciplines—a remarkable undertaking for a liberal arts college with a total enrollment of only around 2,200 students (roughly the same as the school’s current enrollment). But as science and engineering curricula and equipment evolved over the decades, both faculty and students began to realize that the building was becoming

outdated to the point of hindering academic progress.

Beginning about 15 years ago, Union College embarked on an ambitious program, complete with world-class facilities, to reinvigorate its science and engineering departments so as to further the school’s reputation as an elite technical program within a liberal arts college. The first step in this transformation was the construction of the Peter Irving Wold Center, which was completed in 2010 (see “Science on Display” in the September 2011 issue, available at www.modernsteel.com). Continuing on the success of this facility, the Science and Engineering Building is now in the middle of a multi-year renovation and expansion, taking advantage of steel framing, to better serve the school’s growing population of technically minded students.

opposite page: The new horizontally curved, steel-framed Ainlay Hall rises four stories (including a full mechanical penthouse) atop a concrete podium basement.



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© Union College Communications

above: A cross section of the academic main street, with the monumental stair at right.

left: Steel framing for Ainlay Hall. The new building uses 575 tons of structural steel in all.

Tower by Tower

The existing building is a four-story steel-framed building, including a mechanical attic, atop a concrete podium basement. Towers 1, 2, and 3 of the existing building, totaling about 74,000 sq. ft, are being completely renovated, and Towers 4 and 5, comprising roughly 50,000 sq. ft, will be demolished in 2020 and turned into green space. As with any renovation project, the existing building presented the design team with many challenges, including an unusual column grid, unacceptable vibrational characteristics of the floor framing, and incomplete drawings.

For the most part, the existing building has no interior columns. Instead, the floors are framed with W33 beams that span from exterior wall to exterior wall on a column grid measuring 12 ft x 58 ft, 4 in. in plan. It was this unusual framing that posed the first significant challenge to the building's occupants and the design team charged with modernizing it. While the floor framing is suitable for classrooms and offices when analyzed in accordance with AISC Design Guide 11: *Floor Vibrations Due to Human Activity* (www.aisc.org/dg) the floors were not suitable for sensitive equipment used by today's faculty and students. It was even reported that some faculty elected to perform their research on weekends when few students were present.



Mark Kanonik (mkanonik@eypae.com) is a senior structural engineer and senior associate at EYP Architecture and Engineering and is the engineer of record for this project. **Cody Messier** is a project engineer with Bergmann and was previously with EYP.



above: An aerial view of the Science and Engineering complex, with the Wold Center (featured in the September 2011 issue of *Modern Steel*) at lower-right and the curved Ainlay Hall at center.

below: The steel-framed, multi-level North Connector joins Steinmetz and Butterfield Halls with Ainlay Hall.



The need to improve the vibrational characteristics of a 60-year-old building with long-span framing added to the challenge. Vibrational characteristics of floor beams are a function of the span length, the stiffness, and the weight supported by the beam. The floor-to-floor height of the building ranged from 12 ft to 14 ft, and the headroom from floor to structure above was only about 9 ft to 11 ft. In addition, the W33 beams were encased in concrete for fire protection. Combined with the low ceiling height, this meant that reinforcing the beams to increase their stiffness would be effectively impossible. Adding weight would overstress the floor framing and the foundations, so this was not a viable solution either. The only remaining method to improve the vibrational characteristics would be to reduce the span, so HSS4½×4½×¾ “vibration posts” were added inside the partition walls, thus reducing the spans of the W33 beams by about half. The posts generally do not align floor-to-floor due to building use constraints, and some transfer beams were necessary where the vibration posts did not align with the W33 beams. The posts are carried down to the basement and founded on spread footings. Through careful coordination with the architectural floorplans and judicious placement of the vibration posts, the vibrational velocity of the floor framing was improved to approximately 8,000 micro-in. per second based on a moderate walking pace, a dramatic improvement without any limitations to the use of the spaces. Some areas of the building that were programmed for offices or other “non-sensitive” areas, such as utility rooms, were not stiffened since the floor framing was adequate for these uses. Because the vibration posts are not required to carry gravity loads but rather were installed purely for serviceability reasons, it was not necessary to fireproof them.

Another challenge was the fact that the existing building had no explicit lateral force-resisting system (LFRS). The exterior walls and most of the interior partition walls are unreinforced masonry and, at the time of original construction, it was not uncommon for structural engineers to consider such walls as lateral load-resisting elements,



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even if they were not explicitly detailed as such. In order to repurpose the building, all masonry walls had been completely removed, leaving the building theoretically unstable. A series of eight braced frames were added throughout the building to recreate the LFRS. Since Schenectady, N.Y., where Union College is located, is in an area of relatively low seismicity, the braces were designed as $R = 3$ without any specific seismic detailing. The foundations were adequate for the new LFRS, but some of the columns and their base plates were reinforced to resist wind and seismic loads.

Reconnecting

Connected to the building on the east side is Bailey Hall, a 1920s-era building without a basement—which posed yet another challenge to the design team. Bailey Hall was built with no elevator and only one stair, so the Science and Engineering Building provided both the elevator and a second stair to Bailey Hall. As neither the stair nor the elevator is compliant with the current *New York State Building Code* (which is based on the 2015 edition of the *International Building Code*) significant reconfiguration of the Bailey Connector between the two buildings was necessary. It was ultimately decided that building a new connector would be easier and quicker (and therefore less costly) than reconfiguring the existing one.

Prior to the original construction of the Science and Engineering Building’s basement, a deep foundation system was installed under Bailey Hall. While some documentation of this system was available, no “as-built drawings” were able to be found. As there was no evidence of adverse settlements in either building, the design team decided to reuse the existing foundations and basement walls up to the underside of the first floor, with new steel framing employed above, which eliminated the need for excavation bracing adjacent to Bailey Hall. The architects had to carefully coordinate the floor plans with existing structural elements, but the decision ultimately saved construction time and money.

below: The signature beam, installed in the penthouse roof at the southwest corner of Ainlay Hall.

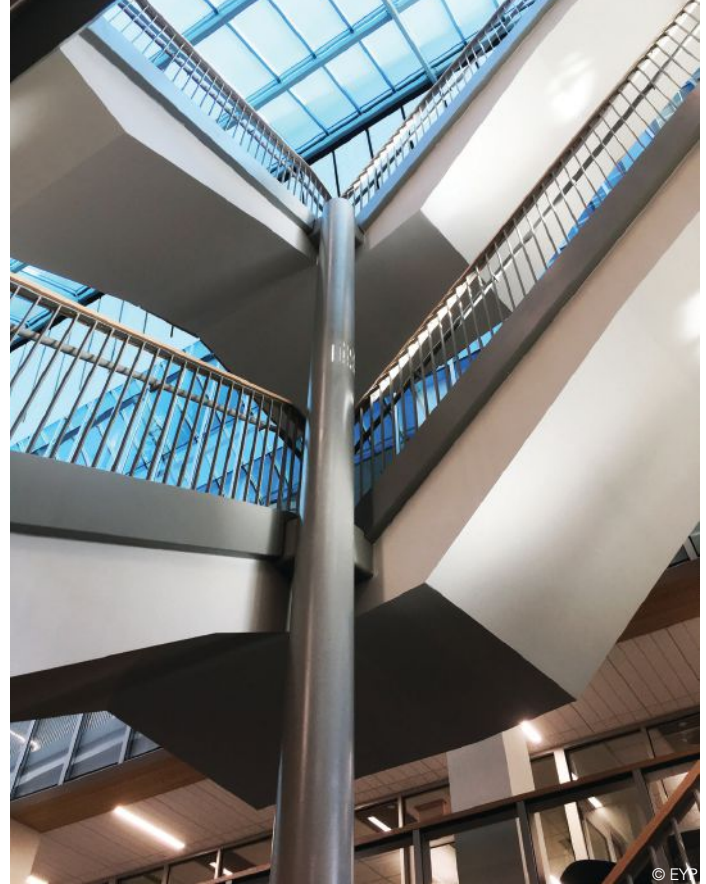


above: A tapered beam used to create the “academic main street” joining the existing building to the new atrium space. These beams are welded to and cantilever from existing columns. The existing beams are W33s, so W33s were also used for the academic main street to simplify moment connections at the cantilever. The beams are tapered from 33 in. to 6 in. so that the end of the cantilever looks thin when viewed from the atrium space.

below: A pair of vibration posts amidst a forest of metal studs. These HSS elements were installed inside partition walls to improve vibrational characteristics.



© Union College Communications



One of eight braced frames installed throughout the existing building. The left column is part of the original construction, and the right one is new.

The staircase post is a round HSS12.75x0.500. Visible portions of the post were specified as AESS Category 3.

In addition to establishing a new connection with Bailey Hall, the Science and Engineering Building was also expanded to the north via an “academic main street”—an open balcony fronting a five-story atrium space, itself a dramatic space that invites students and faculty to stop and socialize. The balcony is supported by W33 beams that cantilever from the existing columns to create a column-free space; W33 beams were chosen to match the existing floor framing to simplify the moment connections at the existing columns. The beams are tapered down to 6 in. at their ends to reduce the profile of the floor structure and to provide an interstitial space for utilities. The roof of the atrium is a significant structural element using architecturally exposed structural steel (AESS) fabricated to AESS Category 3, and is complemented by a monumental stair comprised of steel designated as AESS Category 2.

The building is also currently being expanded to the south, with an addition of about 7,900 sq. ft of new office space over three floors, and the framing for this portion, which houses offices without any stringent vibrational criteria, was also designed in accordance with AISC Design Guide 11. This south addition will be connected directly to the Science and Engineering Building without any isolation joints.

In all, approximately 195 tons of structural steel is being used to frame the building, including the Bailey Connector and the north and south expansions. The Bailey Connector, a small portion of the south expansion, and the academic main street were completed last year, and the remainder of the work is expected to be completed this summer, with demolition of Towers 4 and 5 to be completed by next year.

New Lab Space, Old Constraints

The overall project also includes Ainlay Hall, a new building located north of but isolated from the Science and Engineering

Building, which was completed last year. Named for Stephen Ainlay, the recently retired president of Union College, the horizontally curved, steel-framed building rises four stories (including a full mechanical penthouse) atop a concrete podium basement. The new building, which incorporates 575 tons of steel framing, houses laboratory spaces, faculty offices, student gathering spaces, and a mechanical penthouse, and is connected to the Science and Engineering Building via a five-story atrium. Structural steel was the obvious choice for the framing system, given the complex geometry and large open spaces necessary for the laboratories (though a concrete podium at Level 1 was chosen simply to match the framing of the Science and Engineering Building). The framing in the western half of Ainlay Hall is relatively orthogonal in configuration, and the framing in the eastern half is mostly radial and tangential in configuration. The columns on the north and south façades are closely spaced, eliminating the need for horizontally curved edge beams.

The architectural plan of Ainlay Hall presented two conflicting challenges. The new building was designed to vertically match the Science and Engineering Building but was also designed to current laboratory standards. The sheer volume of air circulated in today’s laboratories is significantly larger than when the Science and Engineering Building was originally designed and constructed, but the relatively low floor-to-floor height of the original building did not easily accommodate the larger ductwork required by today’s lab spaces. In addition, the majority of the new lab space required column-free spaces, which can exacerbate vibrational issues if not properly addressed.

As a result, routing the ductwork became a very difficult task. Shallow beams would simplify routing the ductwork, but deeper beams would improve the floor’s vibrational characteristics. A compromise was found with an atypical floor framing scheme: shallow filler beams

were placed in the short direction and deep girders were placed in the long direction. The ductwork was successfully routed to all spaces without beam penetrations; main supply ducts were placed on one side of the girders and main return ducts were placed on the other side of the girders, with only small ducts crossing under the girders. As a result, the floors have acceptable vibrational characteristics, the rooms have ample vertical space and air is circulated throughout the spaces with minimal effort. An unintentional but surprising benefit of this atypical floor framing scheme was that it resulted a slightly lower total tonnage of steel when compared with a traditional scheme (that is, filler beams in the long direction and girders in the short direction). The vast majority of Ainlay Hall was designed for a vibrational velocity of 2,000 micro-in. per second based on a medium walking pace in accordance with AISC Design Guide 11, although the vibrational criteria was relaxed in “nonsensitive” areas such as offices, gathering spaces and the mechanical penthouse.

The Final Piece

Completing the school’s science campus puzzle is the North Connector, a three-story addition between two other nearby buildings, Steinmetz and Butterfield Halls. Currently under construction, the steel-framed building provides an elevator and an egress stair, gathering spaces for students, and utility rooms and is joined with Ainlay Hall via a two-story steel-framed bridge with glass cladding. The bridge was constructed early to provide temporary

egress from Ainlay Hall, but the remainder of the North Connector will not be completed until later this year, after an existing stair and elevator tower are removed. The North Connector and its bridge, which use approximately 70 tons of steel, are isolated from the three adjacent buildings to which they adjoin, and all lateral loads are resisted by rigid frames designed as $R = 3$.

With the upcoming completion of this multi-phase project, together with the Wold Center that was completed in 2011, Union College’s students and faculty will have gained nearly 150,000 sq. ft of state-of-the art science and engineering facilities, enabling this small school to continue providing world-class technical education and research opportunities on a large scale. ■

Owner

Union College, Schenectady, N.Y.

Construction Manager


Turner Construction Co., Albany, N.Y.

Architect and Structural Engineer

EYP Architecture and Engineering, Albany

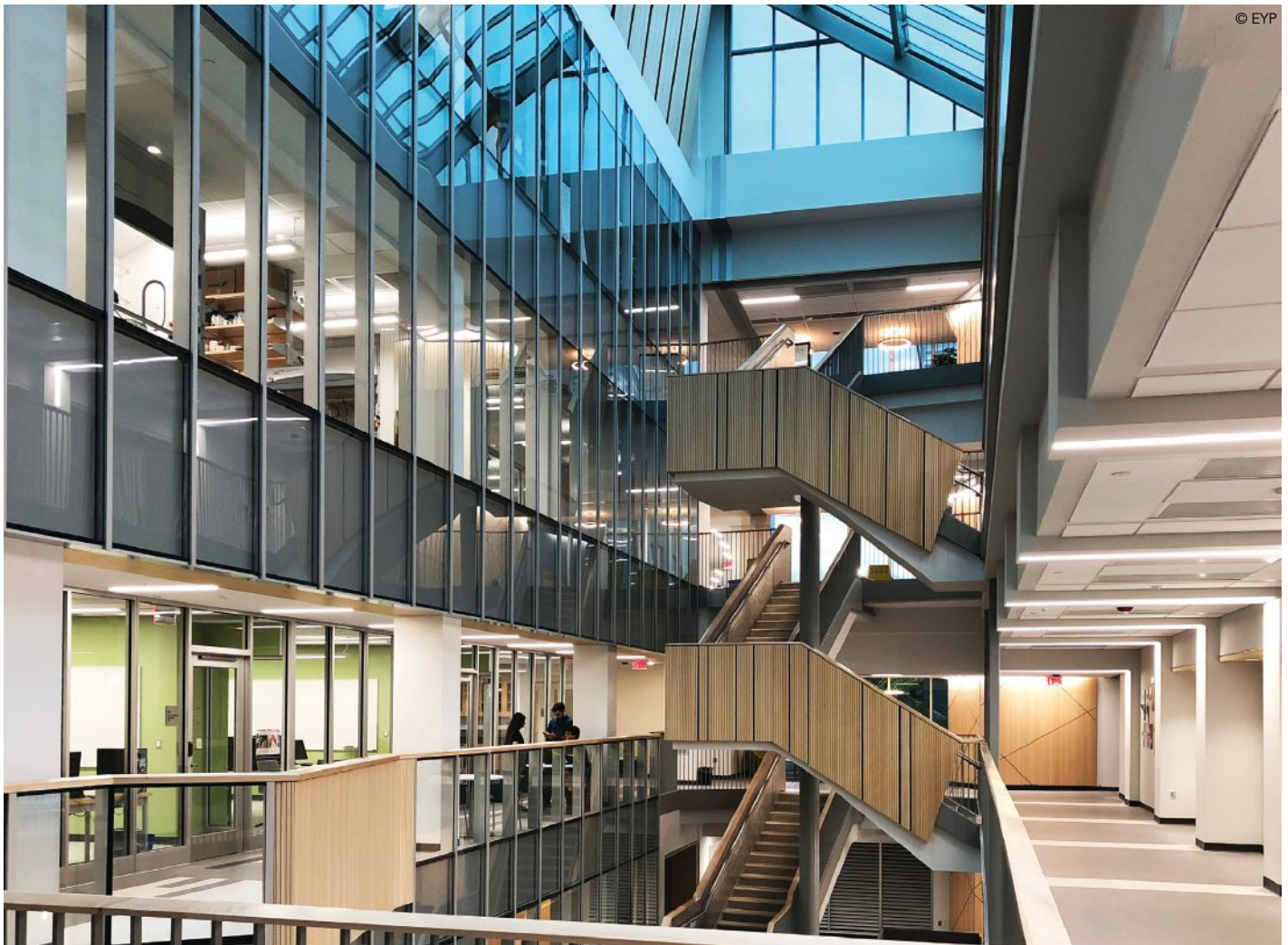
Steel Team

Fabricator

Stone Bridge Iron and Steel, Inc., Gansevoort, N.Y. 

Detailer

JCM and Associates, Ltd., Frankford, Ontario, Canada 



The academic main street, with the new Ainlay Hall on the left and the monumental stair from level 0 to level 3 at the far end. The atrium area is framed with tapered W33 beams seen on the previous spread.

A new take on a tried-and-true truss highlights
a fresh approach to factory aesthetics in suburban Chicago.

Modern Manufacturing

BY THORSTEN HELBIG, FLORIAN MEIER, AND THOMAS MUELLER



Thorsten Helbig (th@knippershelbig.com) is a partner and **Florian Meier** (fm@knippershelbig.com) and **Thomas Mueller** (tm@knippershelbig.com) are associates, all with Knippers Helbig, Inc.

MANUFACTURING HAS NEVER LOOKED quite so cool.

German machine tool and laser manufacturer Trumpf has turned high-tech machines and innovative production processes into exhibition-like showpieces. Part workshop and part exhibition hall, the company's new steel-framed Smart Factory facility in Hoffman Estates, Ill., a suburb of Chicago, consists of two attached buildings topped by a gently sloping continuous mono-pitch roof that reaches 40 ft at its highest point. The northern building houses offices, training rooms, and a café and connects at the corner to the southern block, which contains the factory/showroom area that presents a series of display windows facing the highway.

The building's exterior, with its cladding of corrugated weathering sheet steel, contrasts beautifully with its elegant, floor-to-ceiling glazing, emphasizing both the showroom's industrial context and its representative function. The main structural components are wide-flange columns, up to 38 ft tall and spaced at 5 ft, 4 in. along the exterior, that provide out-of-plane support for the aluminum and glass façade.

The showroom portion serves as an Industry 4.0 (a name for the current trend of automation and data exchange in manufacturing and commonly referred to as the "fourth industrial revolution") demonstration factory that presents the entire sheet metal process chain as an intelligently interlinked, holistic progression, linking the region's historic industrial know-how with contemporary computer-controlled capabilities.

A striking roof structure, supported by 11 Vierendeel trusses, spans over the 26,100-sq.-ft showroom area, creating a flexible, column-free space. The trusses, spaced at 16 ft apart, are each 145 ft long and up to 12 ft deep and weigh approximately 18 tons. Taking advantage of the ample space between the top and bottom chords is an open "skywalk" that runs through the roof trusses 21 ft above the showroom floor, offering visitors an impressive overview of the machinery below as well as an up-close look at the company's expertise, as the trusses' 50-ksi steel plates were laser-cut using Trumpf manufacturing equipment (they were also part of a separate contract from the main structural steel framing system, which was fabricated by Arlington Structural Steel).

Vierendeel, Reinterpreted

The trusses take a new approach to Belgian engineer Arthur Vierendeel's namesake design. Originally intended for bridges, Vierendeel developed his famed "girder without diagonals" as an economic alternative to the riveted steel truss girder, which at that time (the late 1890s)



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above: The exterior of Trumpf's Smart Factory meshes weathering steel columns and corrugated sheet steel with full-height glazing.

left: Exposed steel framing, particularly for the roofing, is evident in the building's lobby.

below: The facility consists of two attached buildings topped by a gently sloping continuous mono-pitch roof.



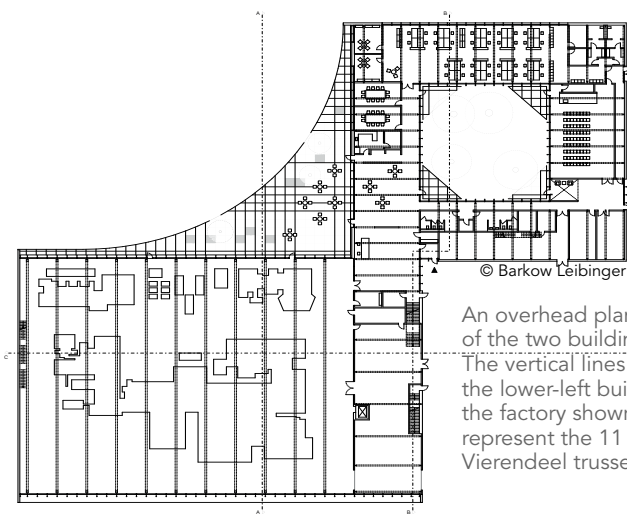
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could not be accurately analyzed and therefore required very high safety factors to account for secondary stresses in the truss.

For the Smart Factory trusses, the internal forces under uniform vertical loads were calculated based on the principles initially developed by Vierendeel, and the internal force diagrams further informed optimization of the truss geometry. Because of the constantly increasing shear force and the resulting higher frame moments towards the ends of the span, the spacing of the verticals

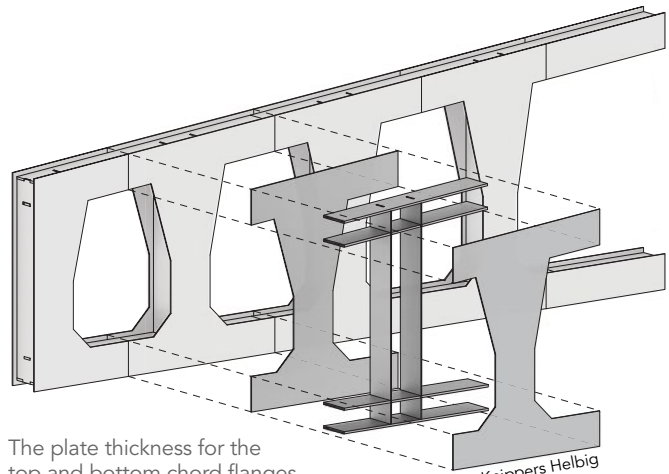
decreases and the width of the verticals increases toward the span ends. Because the vertical posts need to resist bending moments at the upper and lower ends, these elements have a deeper section at the top and bottom to resist these larger moments. In addition, the verticals taper to their minimal section size towards the center of the member to save material and provide more transparency.

The chords of the trusses are further optimized by responding to the flow of internal forces. Spacing between the upper and lower



© Barkow Leibinger

An overhead plan view of the two buildings. The vertical lines in the lower-left building, the factory showroom, represent the 11 Vierendeel trusses.



© Knippers Helbig

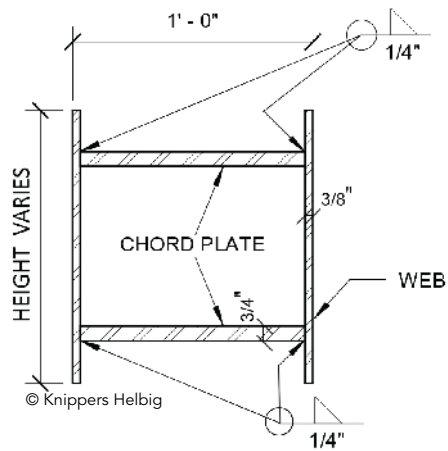
The plate thickness for the top and bottom chord flanges of the trusses is $\frac{3}{4}$ in. while the thickness of the vertical side walls is $\frac{3}{8}$ in. throughout.



chords is highest at mid-span and decreases toward the end, where the bending moment decreases. Additionally, the chord cross sections have greater depth towards mid-span, where their axial forces are highest. This modified version of the original Vierendeel truss creates an optimized redistribution of the stresses and therefore allowed the use of a uniform steel plate thickness for fabrication, which reduced the overall steel tonnage significantly. The girders have a self-weight of approximately 15 lb/sq. ft, an economically efficient value for a roof structure carrying additional pedestrian loads.

The fabrication and erection team were consulted early in the design stages and employed a 3D model-based exchange

and coordination process, which facilitated a smooth transition to the shop drawing phase. The truss chords and verticals are custom-welded rectangular box sections made of 50-ksi steel. The plate thickness for the top and bottom chord flanges is $\frac{3}{4}$ in. while the thickness of the vertical side walls is $\frac{3}{8}$ in. throughout. These side walls project beyond the flanges of the chords and vertical posts, which allowed for simple fillet welds and eliminated the need for weld preparation and plate beveling. Secondary beams made from standard wide-flange sections run perpendicular to the trusses and pick up the roof from the steel deck. The trusses were prefabricated in three sections apiece—



A cross-section view of a truss.

The main structural components are wide-flange columns, spaced at 5 ft, 4 in. along the exterior, that provide out-of-plane support for the aluminum and glass façade. The glass system is attached to the inside flanges of the weathering steel exterior columns as opposed to the outside flanges.



© Simon Menges Photographer

each roughly 50 ft long—connected by integrated bolted end-plate joints in the chords, and lifted on-site via two cranes. Cutouts in the chords allowed ironworkers to position large hydraulic wrenches and bolts *inside* the chords to tighten the four 2-in. by 7½-in. bolts at each chord connection. Since the bolt heads and nuts were so close together, workers had to use 2¼-in. shims and a very specific bolting order to get all the bolts properly



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The showroom serves as a demonstration factory presenting the entire sheet metal process chain.

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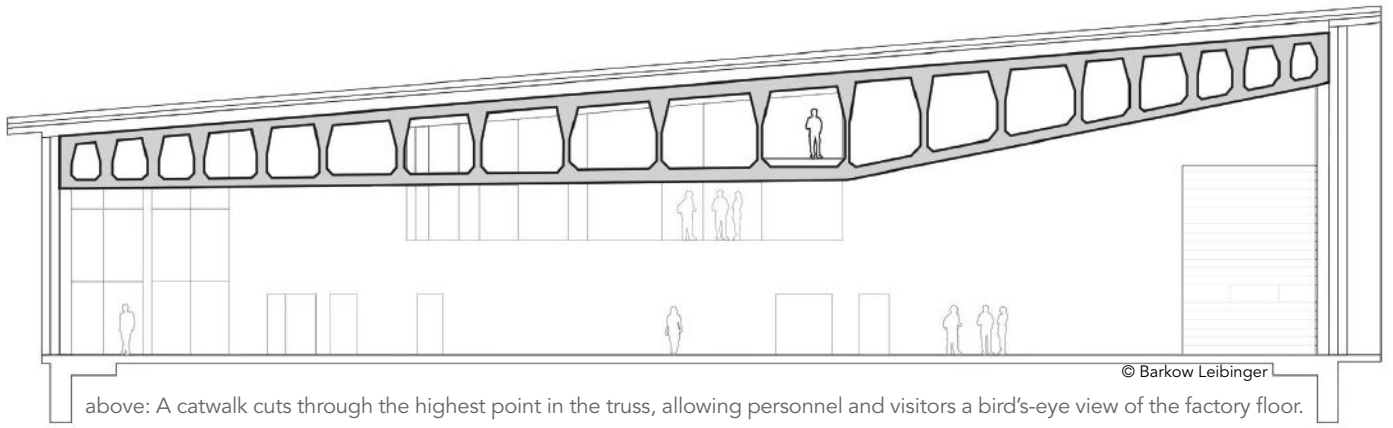
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above: A catwalk cuts through the highest point in the truss, allowing personnel and visitors a bird's-eye view of the factory floor. © Barkow Leibinger



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above: The truss erection sequence was as follows: The first two (of three) sections were bolted together on the ground and then picked up with two cranes. After the north end of the truss assembly was attached to the column, the north crane moved to the south end and picked up the south (third) section of the truss and lifted it in place.



tensioned. Once the main girders were in place, the secondary beams and roof deck were installed on the upper chords.

Both the building's façade and structure, particularly the trusses, demonstrate the wide spectrum of possibilities with steel. The steel elements also play a lead role in illustrating how factories can be elegant as well as functional, creating and supporting a modern manufacturing environment. ■

Owner

Trumpf, Inc., Farmington, Conn.

General Contractor

McShane Construction Company, LLC, Rosemont, Ill.

Architects

Barkow Leibinger, Berlin/New York
Heitman Architects, Itasca, Ill.

Structural Engineers

Knipppers Helbig, Inc., New York
IMEG, Naperville, Ill.

Steel Fabricator

Arlington Structural Steel Company, Inc., Arlington Heights, Ill. 

below: The 11 trusses, spaced at 16 ft apart, are each 145 ft long and up to 12 ft deep and weigh approximately 18 tons.



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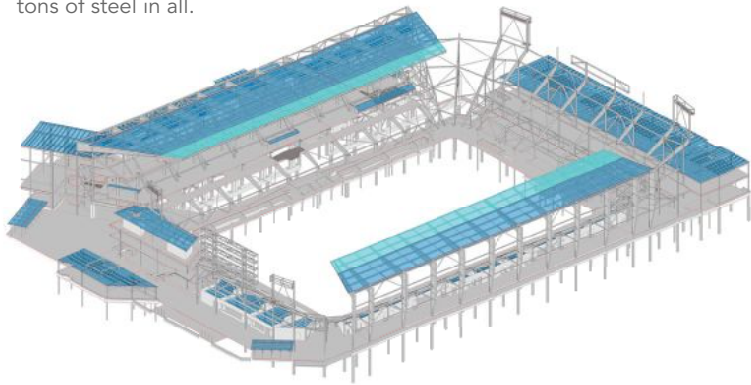


United in Steel

BY DIMITRIOS FRANTZIS, PE, AND CONSTANTINOS FRANTZIS

above: The new 20,000-seat Audi Field provides a soccer-specific venue for the DC United team, which previously played its home games at RFK Memorial Stadium.

below: An overall 3D structural model of the stadium, which incorporates 5,000 tons of steel in all.



DC UNITED NOW HAS a steel-framed home of its own: Audi Field.

Prior to the stadium's opening this past summer, Washington, D.C.'s Major League Soccer (MLS) team was using the old, virtually obsolete RFK Memorial Stadium, originally built for football, as both its home field and offices. But the team wanted what every soccer team wants: a soccer-specific stadium that brings fans close to the action and entices them back with attractive amenities (not to mention good soccer).

The new 20,000-seat stadium is located in the Buzard Point area of the capital, near the confluence of the Washington Channel and the Anacostia River, less than two miles south of the U.S. Capitol and a few blocks away from Nationals Park, home of the Washington Nationals baseball team. While the United organization was thrilled to secure a site for its new home, the location was not



all images in this article courtesy of A+F Engineers

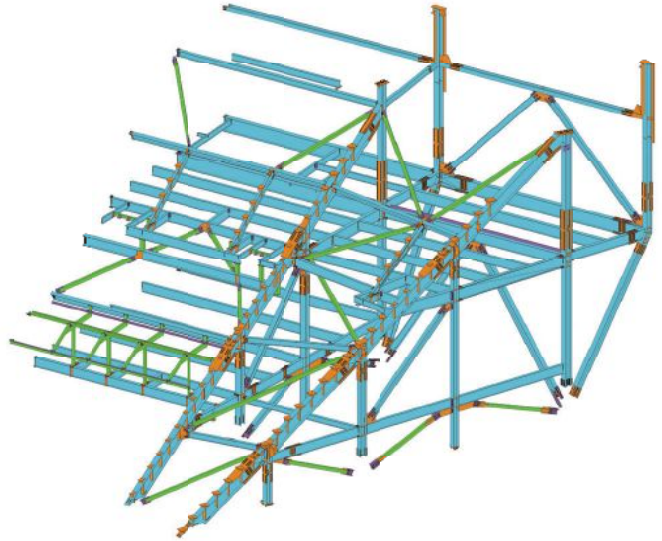
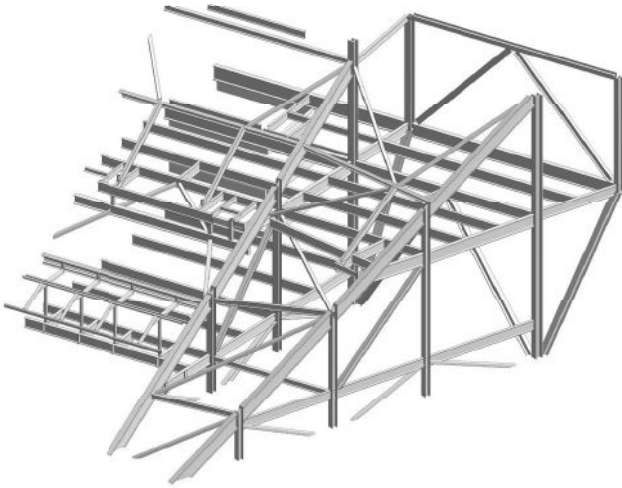
without its challenges. Not only is the footprint one of the smallest for any MLS stadium, but a large easement also runs through the area and had two significant effects on the project. First, it dictated that the main stadium structure, including most of the retail space, suites, and club area, had to all be on one side of the stadium (the east side) in order to keep the field on an area of the site that was clear of the easement. Secondly, it required that this (largest) area of the structure be raised above the easement, with no lateral bracing to the ground level in the east-west direction. These two constraints led to the stadium's exposed steel aesthetic. Because a steel framing system would be lighter and provide more support with less material, it would allow the team to make the most use of the limited available space. The stadium's framing system uses 5,000 tons of structural steel in all.

Steel not only composed the structural framing system for the project but was also a key architectural element. As such, the design team used Autodesk BIM 360 extensively and modeled specific connections (such as expansion joint connections) in Revit during the design phase to minimize changes later in the project. During the construction administration phase, all steel submittals were reviewed in 3D as well as 2D drawings. The structural model was detailed enough that the steel fabricator, Schuff Steel, was

D.C.'s soccer stadium makes the most of a small parcel of land thanks to an efficient built-up box girder framing scheme.



Dimitrios Frantzis (dfrantzis@af-engineers.com) is principal and lead engineer and **Constantinos Frantzis** (cdfrantzis@af-engineers.com) is BIM manager, both with A+F Engineers in Washington, D.C.



Using A+F's 3D model (left), steel fabricator Schuff was able to quickly develop shop drawings and incorporate connection details and elements into a separate 3D model (right) submitted with the shop drawing package. The ability to compare shop drawing packages directly in 3D allowed A+F to stay ahead of the aggressive construction schedule thanks to quick fabrication shop turnaround times.

able to use it as a template to create its fabrication and connection models, and the 3D shop drawings were overlaid on top of the structural model to expedite review. Furthermore, clash detection could be implemented with architectural and MEP models to see if certain connections had to be modified to better fit the building as a whole. To expedite construction, the vast

majority of the structure was designed with bolted connections and Class B faying surfaces. In addition, the use of plate girders for the canopies was a big time-saver, as the welding could be performed in-house by Schuff.

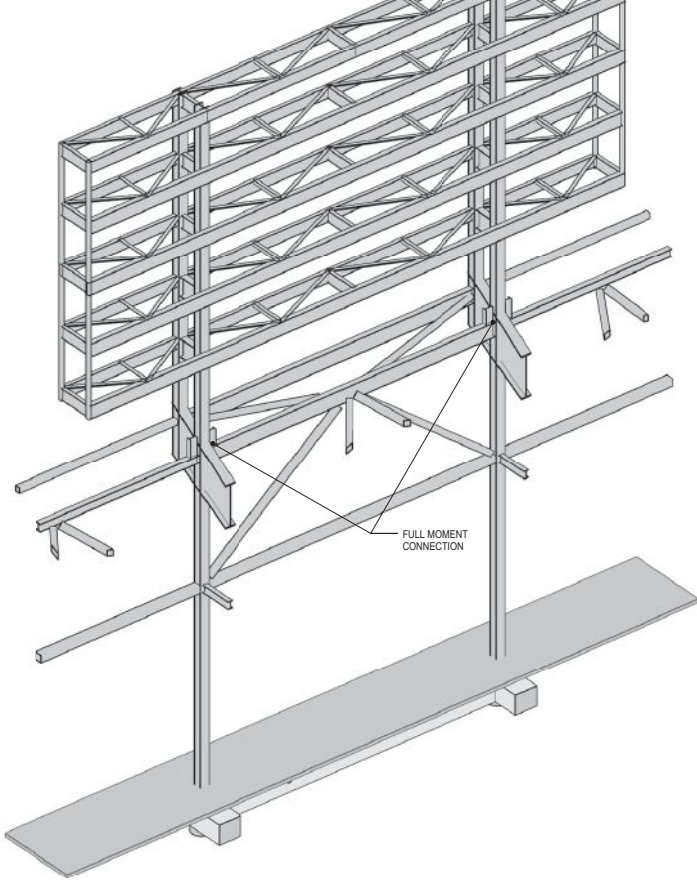
The most significant design challenge for the stadium was maintaining deflections and vibrations within acceptable limits.

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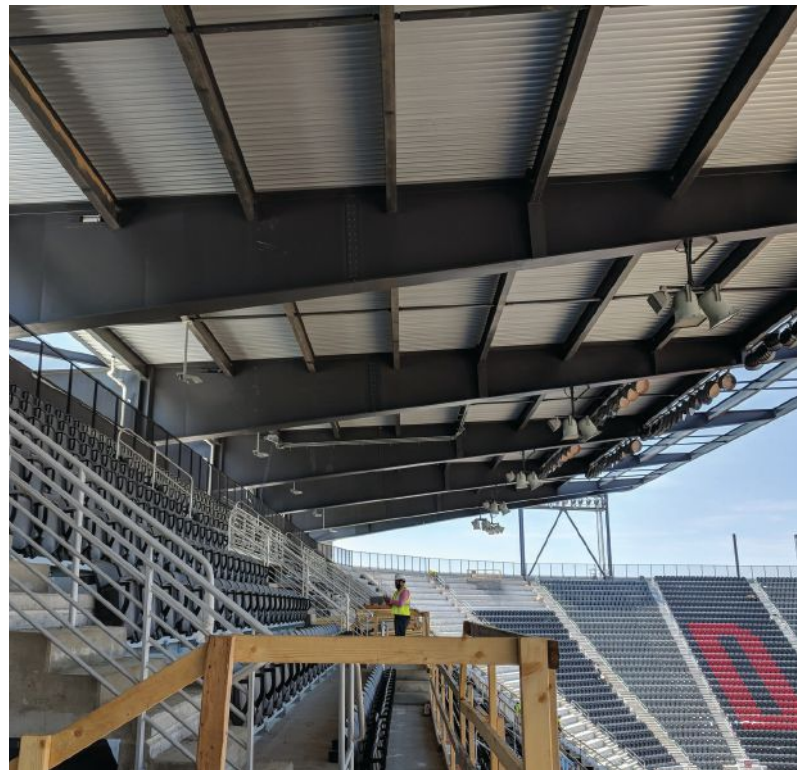
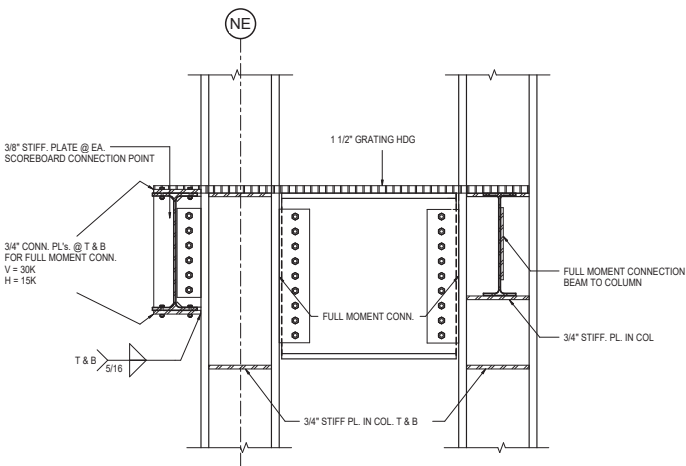
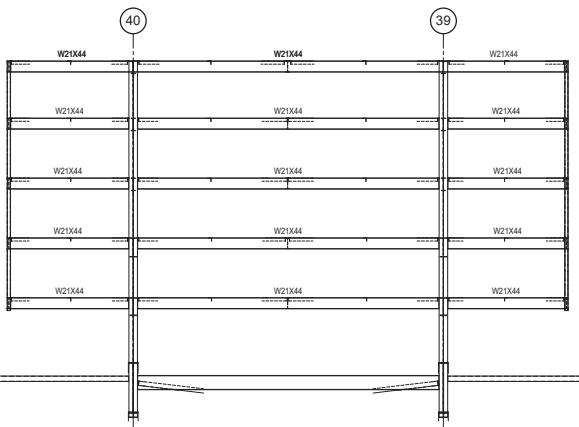
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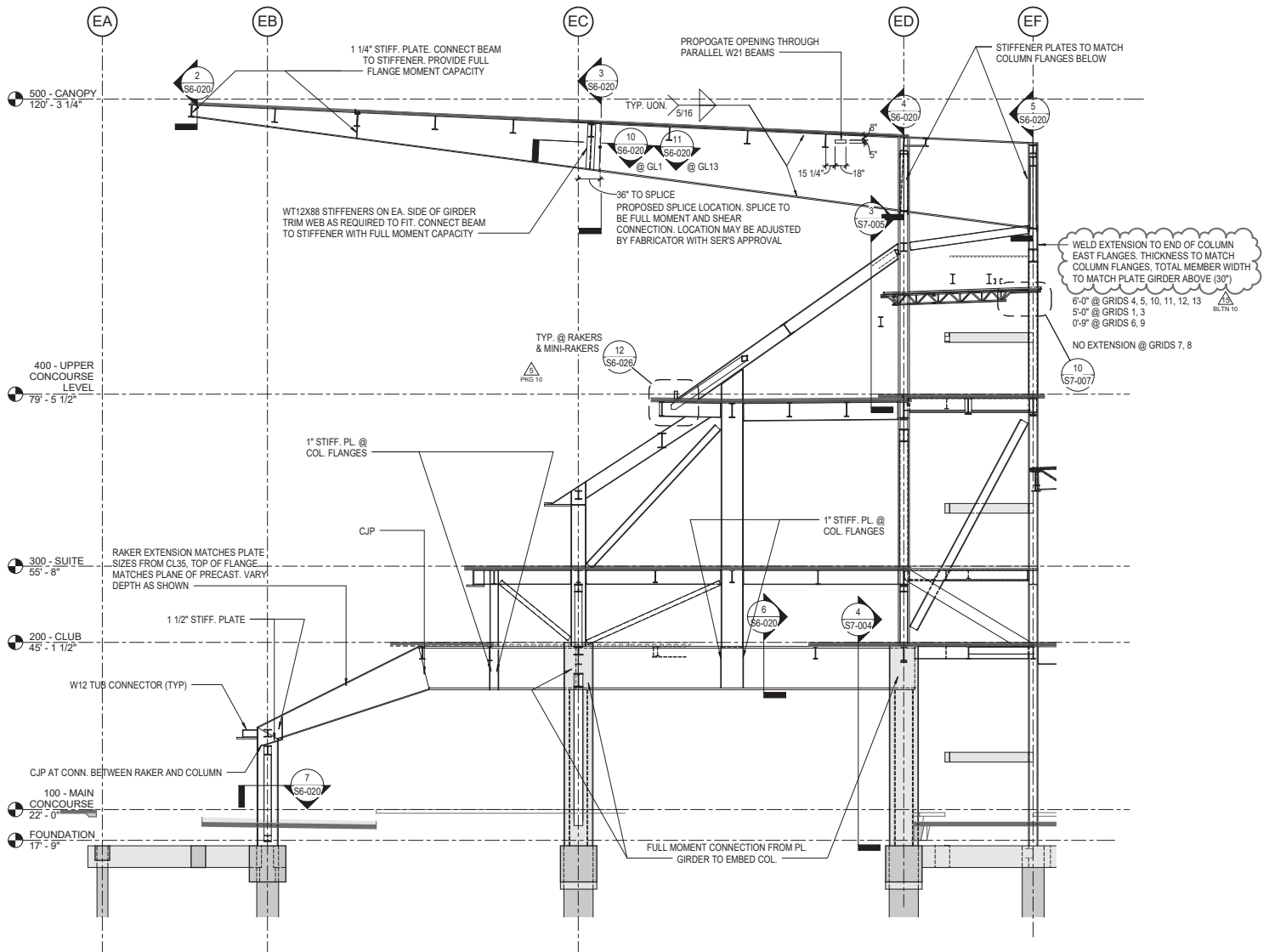
left: The 87-ft-wide by 39-ft-tall scoreboard, from top to bottom: southwest isometric view, north elevation view, and a typical section.

below: The scoreboard, located at the end of a raker cantilever, was fit up flat at grade and then erected with a single lift into its final position.



above: Tapered plate girders support the canopy on the east side of the stadium. At 116 ft long, they taper from almost 12 ft to 21 in. along the length.

Because ASCE 7 design wind loads are not specifically formulated for open structures like Audi Field, the stadium was wind-tunnel tested by consultant RWDI in order to obtain refined wind loading conditions. The wind tunnel analysis provided not only wind pressures to different components but also dynamic characteristics used to refine the 116-ft-long canopy plate girders



above: Canopy framing and plate girders for the east seating bowl. Column locations above the raised concourse were governed by inflexible seating configurations, so built-up plate transfer girders were used above the easements.

left: Complicated nodes such as this one, at the north end of the stadium, were designed to be shop-fabricated using automated welding processes as much as possible. This allowed for complicated moment connections without the need for any field welding, speeding up erection significantly.

and supporting framing. The final designs had to meet standard overall strength and deflection criteria, with the addition of frequency limits that needed to be maintained to satisfy the underlying assumptions of the wind tunnel results.

Additionally, several design features of the grandstands led to complicated analysis for vibrations under rhythmic loading. These include long stadia cantilevers on the south seating bowl, the combination of the cantilevered scoreboard in the north and its proximity to the most active portion of the fan base (think lots of people jumping), and the fact that large cantilever canopies were supported from the same framing as the seating in the east and west.

From a framing perspective, designing the east grandstand over the easement was the most challenging aspect. Column locations above the raised concourse were governed by seating and club configurations that had minimal flexibility, leading to the design decision to employ built-up plate transfer girders above the easements. As the team could not place diagonal braces through the easement, these girders also serve as the horizontal members of the moment frames. The design called for the two longer of the three columns supporting these girders at the moment frames to be encased in concrete in order to balance the shear transfer between them and the much shorter third column, which helped increase the stiffness of the overall frame.

Due to the exterior exposure of the vast majority of the steel and the need to maintain Class B faying surfaces at all slip-critical connections, the exterior steel is covered by a multi-layer polyurethane high-performance coating system (a shop-applied primer and field-applied finish coats).

In a city of stone monuments, Audi Field stands out as an exposed-steel shrine to soccer, making the most of its limited space via an efficient framing system and giving the team and its supporters the open, attractive venue they deserve. ■

Owner

DC United, Washington, D.C.

General Contractor

Turner Construction Co., Washington


Architect

Populous, Kansas City

Structural Engineer

A+F Engineers, Washington

Steel Fabricator and Erector

Schuff Steel Midwest,  Ottawa, Kan.

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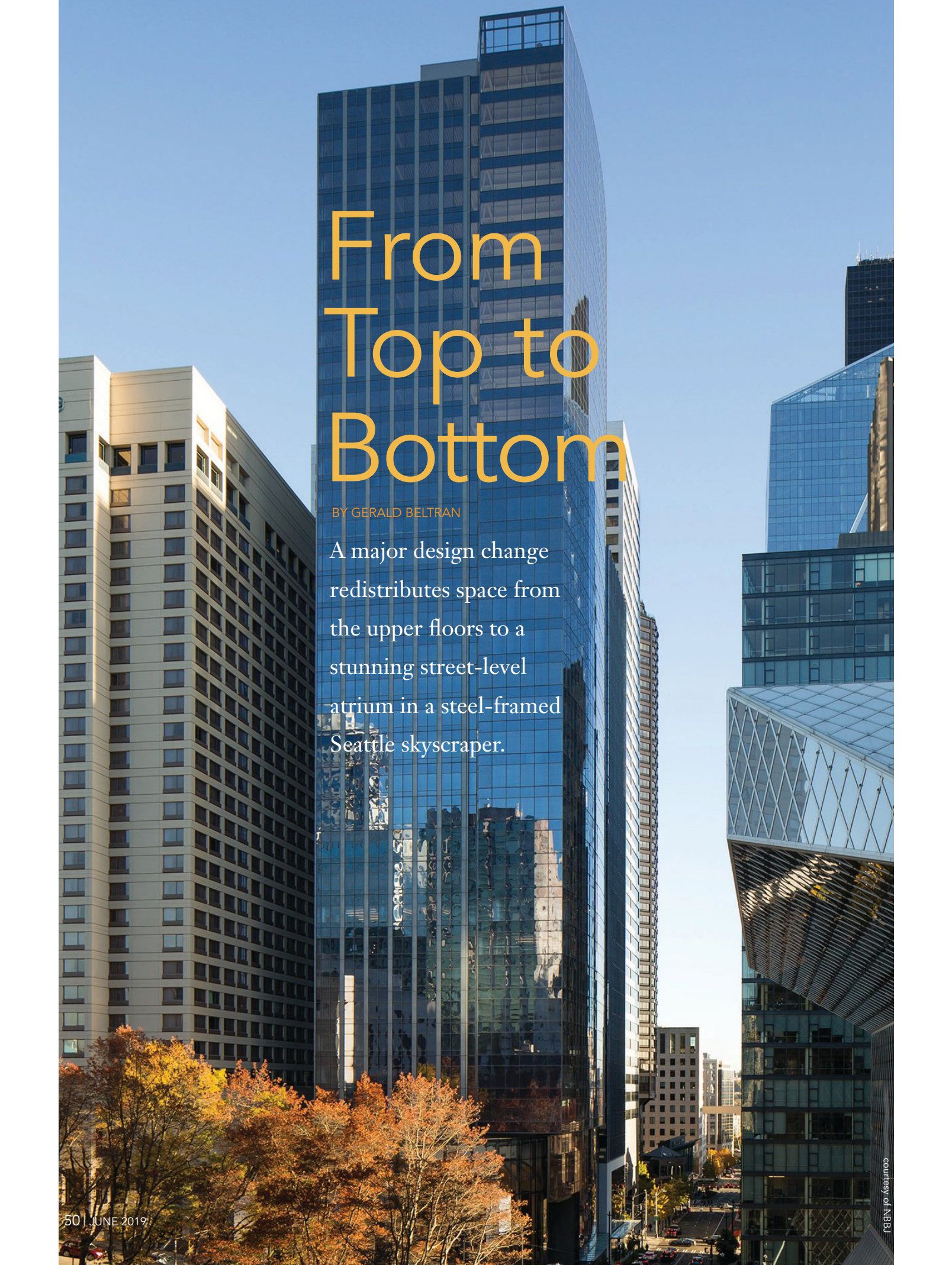
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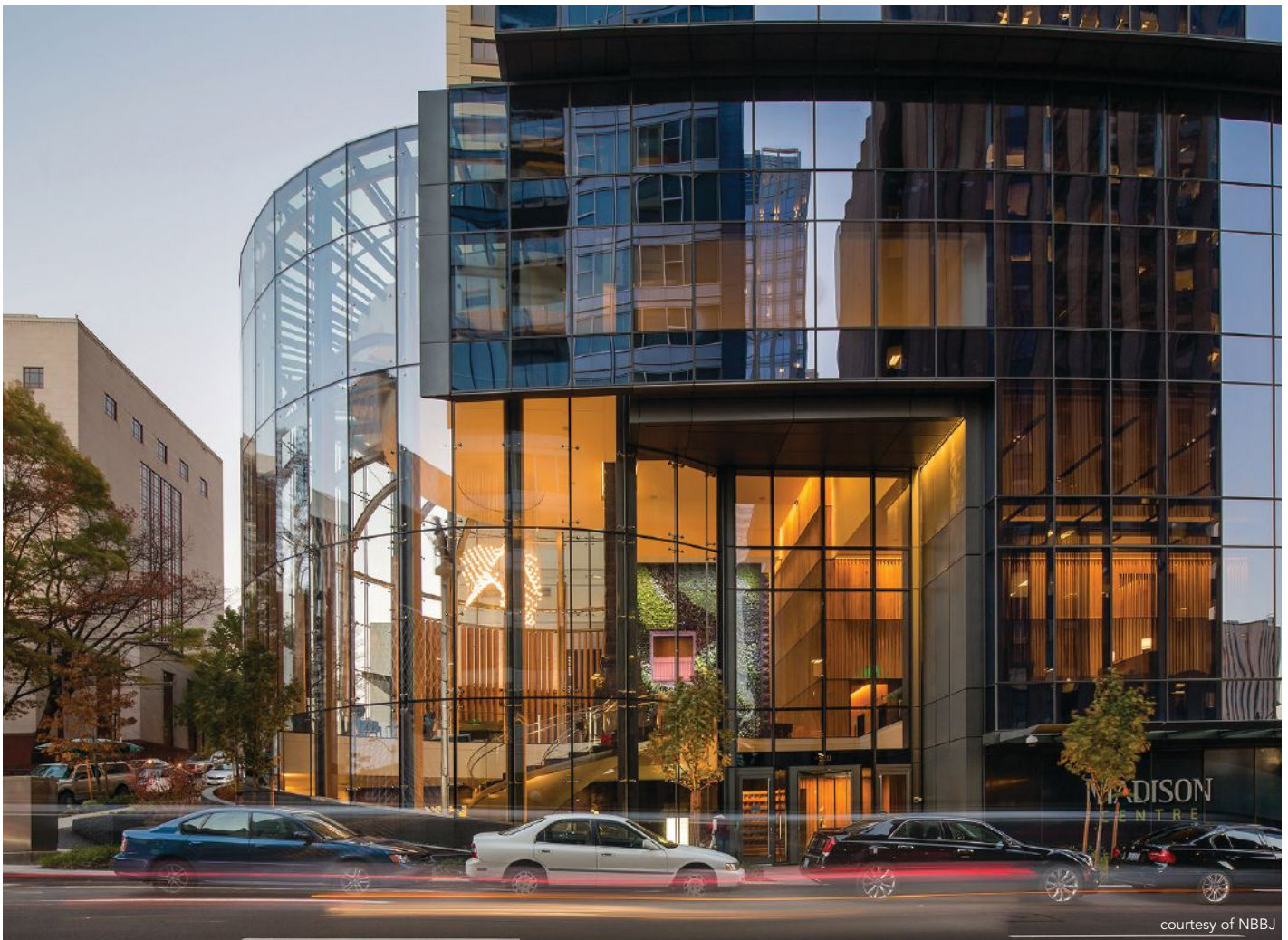
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From Top to Bottom

BY GERALD BELTRAN

A major design change redistributes space from the upper floors to a stunning street-level atrium in a steel-framed Seattle skyscraper.



THE MADISON CENTRE has been a long time coming, but the results made the wait worthwhile.

Like many urban high-rises planned for the 2000s, this project was a victim of the Great Recession and was put on hold for several years before being revived in 2012. Now completed, the 530-ft-tall, 36-story office tower (atop seven subterranean parking levels), at the east corner of 5th Ave. and Madison St. in downtown Seattle, includes retail space, a lobby, a commons area, a loading dock, offices, and a 4,500-sq.-ft rooftop deck, as well as 580 below-grade parking spaces—in just over 1 million sq. ft.

The framing scheme for the building is a cast-in-place concrete core surrounded by structural steel framing from the ground up, and the tower uses approximately 3,900 tons of structural steel in all. Structural steel columns in line with the tower columns are cast inside the drilled piers, and these tower columns support the post-tensioned slabs in the east portion of the building. The office floors consist of composite steel framing, with conventional and diagonal reinforced coupling beams spanning all door and lobby openings. The perimeter tower columns on the 5th Ave. side of the building slope outward from Level 3 to Level 6 to facilitate a required 6-ft, 10-in. offset. To counter the forces created by this offset, the tower columns along an adjacent alley also slope outward (in the opposite direction) from Level 3 to Level 6. The beams and connections opposing the columns are designed to resist the tension and compression forces induced by the slope.

above: The feature element of the building, the Great Room, was born of a mid-construction phase redesign that involved removing two stories from the top of the building and reallocating the available space to the ground level.

opposite page: The 530-ft-tall, 36-story building comprises over 1 million sq. ft of space, including seven below-grade parking levels.



Gerald Beltran (geraldb@sellen.com) is an estimator and preconstruction manager with Sellen Construction in Seattle.



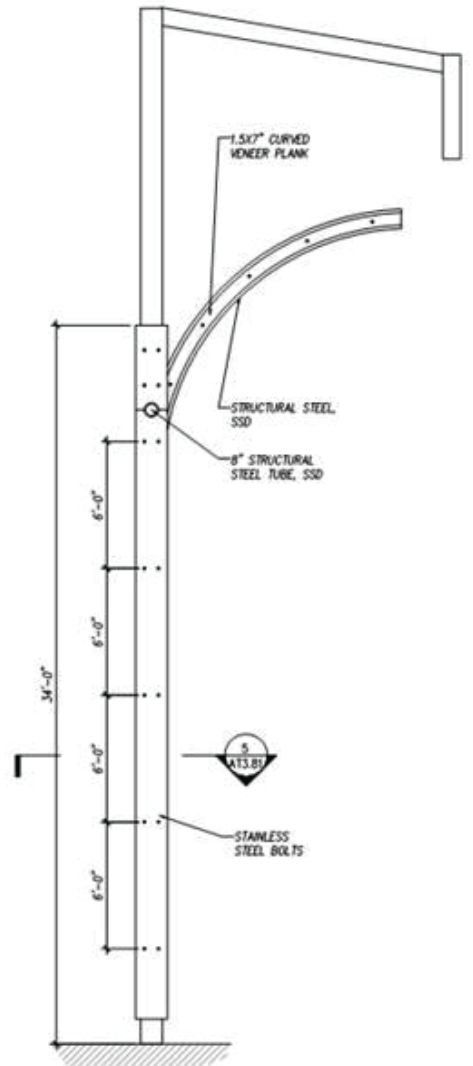
courtesy of Sellen Construction

The steel columns were specified as architecturally exposed structural steel (AESS) Category 1 (Basic Elements).

From Top to Bottom

One of the building's crowning achievements, from both a design and structural standpoint, is, interestingly, at the bottom. This is the "Great Room," a striking 45-ft-tall steel-framed entryway rotunda featuring a circular staircase and large glass plates suspended by cables. And this area, whose design was significantly altered well into the project's design phase, indirectly involves the tower's crown; the redesign included taking two stories off the top of the building and reallocating the available space to the ground level. Structural engineer DCI reworked the structural core of the tower to maximize the available subterranean park-

An elevation drawing of a rotunda column.



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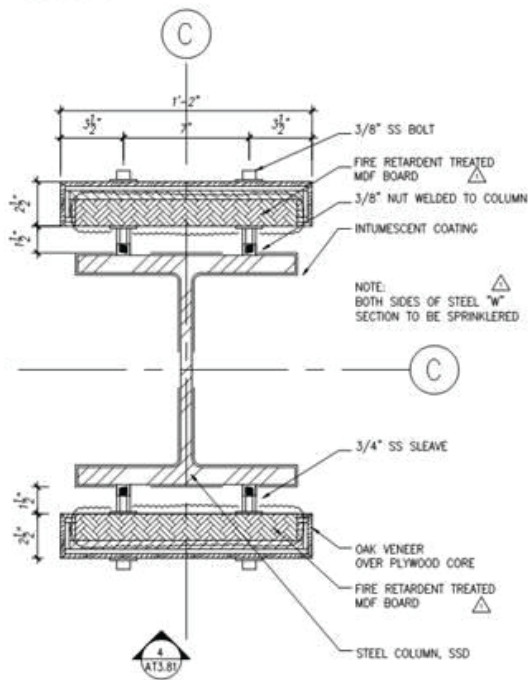


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above: A column plan for the rotunda.

below: The completed atrium, ringed by a series of 34-ft-tall wide-flange columns with wood cladding.



courtesy of NBBJ

ing area and improved the building’s public plaza, moving it to an area that receives more sunlight, thus creating the rotunda. The attractive space provides a lasting first impression for passersby—especially at night when it becomes a street-level lantern of sorts.

The rotunda is ringed by a series of 34-ft-tall steel columns attached by 8-in. round hollow structural sections (HSS). Branching off from the main columns near the top are inward-curving steel elements that join with a steel ring encircling the ceiling of the space. Wood paneling clads the flanges of the columns and is also inset within the flanges of the curved elements at the top.

The steel columns were specified as architecturally exposed structural steel (AESS) Category 1 (Basic Elements). In addition, the steel also received intumescent fireproofing, then a skim coat to relieve the “orange peel” finish created by the fireproofing. A layer of high-performance paint was then applied on top of the skim coat.

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The Great Room is a 45-ft-tall steel-framed entryway rotunda featuring a circular staircase and large glass plates suspended by cables.

The building uses 3,900 tons of structural steel in all.

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courtesy of DCI Engineers



courtesy of DCI Engineers

The perimeter tower columns slope outward from Level 3 to Level 6 on two sides of the building.

Following a years-long delay and a late, major design change, the Madison Centre has become one of Seattle's 15 tallest buildings. But it's the steel-framed atrium at the ground floor that steals the show, draws in visitors and tenants alike, and makes both the first and last impressions positive. ■

Owners

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Bellevue, Wash.

General Contractor

Sellen Construction, Seattle

Architect

NBBJ, Seattle

Structural Engineer

DCI Engineers, Seattle

Steel Team

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Go West

BY GEOFF WEISENBERGER

Labor, innovation, and automation were all a big part of the conversation at the 2019 NASCC: The Steel Conference in St. Louis.



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

The vast majority of the technical sessions were outstanding. And I *loved* the Women Who Weld session.

—Lindsey Hakala,
Midwest Steel

THIS PAST APRIL, and for the second time in the last six years, NASCC: The Steel Conference landed in St. Louis, a city best known for two structural steel marvels: the Eads Bridge, which opened in 1874, and the Gateway Arch, completed 91 years later. (And both times, the conference set an attendance record—more on that later.)

Both projects, one could argue, represent the power of contrarian thinking—i.e., doing something different from how it's been done in the past. The Eads Bridge was one of the first major structures in the world to employ structural steel as a framing material, and the Gateway Arch—well, there's really nothing quite like it, is there? There certainly wasn't before it was built. And why even designate a "Gateway to the West" in the 1960s, when Los Angeles was already on a trajectory to become the second-largest city in America? Because even then, the West still evoked a feeling of expansion into an unknown place, a place of adventure, a place where things might not be quite the same.

The concept of contrarian thinking was the focus of the conference's Wednesday keynote session, "The Power of Contrarian Thinking," presented by Ozan Varol, a professor at Lewis and Clark Law School (as well as a bestselling author and a bona fide rocket scientist). Quite simply, Varol's message was to avoid doing something a certain way just because that's the way it's always been done—while also respecting first principles. One of his examples was Dick Fosbury, whose "Fosbury Flop" turned high jumping on its head. Fosbury's goal was always to propel his body over the bar. He just invented a new way of doing it. His innovative "backwards over the bar" approach was considered laughable in appearance when he developed it in the mid-1960s—until it won him a gold medal at the 1968 Summer Olympic Games. And half a century later, it's still the standard technique used in competitive high jumping.

And this is what conferences are about: seeing what's new, interesting, and contrarian in your industry, not just covering the same ground—or at least not covering it in exactly the same way. And not everything can be as immediately earth-shattering as the Fosbury Flop. Changes, often small, are what typically move the industry forward, and they all happen for various reasons. As Varol noted in his keynote, "Breakthroughs are evolutionary, not revolutionary."

Varol's Wednesday session was just one of three keynotes this year. On Thursday, Jon Magnusson of Magnusson Klemencic Associates spoke on "The Joy of Steel... So Many Possibilities" and the importance of people working together to create incredible structures. And Friday's keynote, the T.R. Higgins Lecture, was given by this year's Higgins Award winner, Ronald D. Ziemian from Bucknell University, who discussed how most stability

problems can be understood by focusing on the big picture rather than on the details of the seemingly complex mathematics.

The keynotes were just the tip of the iceberg. The conference included more than 140 technical sessions (more than 20 of which were streamed live) and attendees had the opportunity to earn up to 17 PDHs—plus an additional 12 if they attended the optional pre-conference short course. If you weren't able to make the show this year but are considering attending in the future, keep in mind that one low registration fee gains you access to all of the technical sessions, the keynote sessions, the T.R. Higgins Lecture, and the exhibition hall. In addition, we try to make choosing sessions as easy as possible by organizing them into tracks (there are nearly 20). And you can view recordings of all of the sessions at www.aisc.org/2019nascconline.

.....

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training budget.**

—David Ruby, Ruby + Associates

.....

But back to breakthroughs, one that was apparent at The Steel Conference, and has been slowly been gaining momentum in the structural steel world, is the role of automation in welding and fitting operations. For a long time, the answer to why structural steel facilities didn't implement automation for welding/fitting was because the work isn't as repeatable as it is in, say, the automotive industry. But a couple of factors have helped turn the tide.

First, robots are able to be programmed faster and more efficiently, so what was once seen as a tedious process and not worth it for one-off jobs can now happen quickly. Simply put, robots have become smarter, and they've done this faster than in the past.

"Welding programming can now be automated," said Adrian Morrall with equipment manufacturer exhibitor Voortman USA. "Plus, robots are able to use sensors and probes to make measurements and adapt to the variety of pieces."

Second, labor has become more scarce and expensive.

"Companies aren't able to find fitters," said Andreas Hofer with Zeman, whose robotic welding/fitting solutions were on display at the Peddinghaus booth. "The time is right for automated fitting and welding."



above: The Wednesday keynote session, presented by law professor Ozan Varol, encouraged the audience to consider the power of contrarian thinking.

below: AISC award winners and presenters at the Wednesday keynote. From left: David Zalesne, David Ratterman, Jon Magnusson, Ron Ziemian, Michel Bruneau, John Cross, Heather Gilmer, Francesco Russo, John Judd, Matthew Yarnold, Charlie Carter, Matthew Hebdon, and Doug Rutledge. Visit www.aisc.org/awards for more on AISC's award programs and winners.



above: Booth experiences ranged from one-on-one demos to group presentations.

below: Virtual reality technology at several booths let attendees travel to job sites, steel mills, and other locations.



NASCC Live is a wonderful way to get a bunch of PDHs on interesting topics for a fantastic price!

—Paul Chabot, PE, EFI Global

In addition, it's a matter of cost, noted David Cody with exhibitor Beamcut Systems/Machitech Automation. "It's a matter of paying \$150 to \$200 per hour per ton for manual work versus \$6 to \$7 a ton for a robot."

While automation addresses one area of the labor shortage—shop welding and fitting—one forward-thinking initiative that was present at the show is using technology in a much different way to recruit workers, particularly younger ones, into the steel world: making manufacturing and construction jobs as a whole more attractive via virtual/augmented reality. At the Industry Lift booth, attendees could immerse themselves in various environments. The most exciting—and to some, the most harrowing—example involved donning a pair of VR goggles and being transported from the exhibit hall to a sidewalk in a bustling central business district full of gleaming high-rises. From there, you could ride an elevator to the top of the building, where you had the option to walk to the end of a wooden plank hundreds of feet above the street. When it came time to exit the simulation, one of the booth employees offered a choice: "You can take the easy way down or the quick way down." The quick way? Jumping off the plank. (Note: I took the quick way—and it was exhilarating—but I had to talk myself into it for a few seconds, while shaking and sweating, fully realizing that I was standing in a carpeted booth in an exhibition hall.) It's this type of exhilaration that just might be a key to making construction jobs more interesting to the younger generation. And in addition to Industry Lift, several other exhibitors also provided VR experiences at the show, including Gerda, which allowed attendees to take a virtual mill tour.



above: Participants in the Women Who Weld demonstration.

below: The conference provided plenty of time between technical sessions for attendees to connect with old friends and colleagues as well as establish new ones.



above: This was the second year in a row that attendance at The Steel Conference reached the 5,100 mark.

below: Audiences were engaged at the more than 140 technical sessions.





Several equipment manufacturers confirmed the proliferation of automated welding and fitting workflows.

You know how sometimes you are sitting in a conference, and all you can think is, “I have so many other things I could be doing that would be so much more worthwhile and valuable?” I never think that at this conference. I have pages of takeaways every year.

—Deb Sweigart, Shickel Corporation

The Conference Dinner offered opportunities to take selfies with the Budweiser Clydesdales.



An on-site bookstore let attendees peruse and purchase AISC’s various publications, from the whole series of Design Guides to the *Manual*.



Another recruitment-related initiative present at The Steel Conference—Women Who Weld—focuses on teaching women how to weld and find employment in the welding industry. The organization notes that while 400,000 welding jobs are expected to be open by 2024, the average welder age in the U.S. is 55, with many soon retiring—and only 4% of all domestic welders are women. As part of its outreach, the group provided a well-received welding demonstration in the exhibition hall. And it’s had tremendous success in general, boasting 100% program completion and program-to-employment rates.

And of course, coming up with or discovering something new and different doesn’t always have to happen at the convention center. Sometimes, a conversation with friends and colleagues over dinner and a beer is the best idea incubator of all, as more than 2,000 attendees and exhibitors experienced at the annual Conference Dinner. And what better place to get a beer than straight from the source? This year’s venue was one of the largest breweries in the world, the Anheuser-Busch St. Louis Brewery, where attendees were able to take a peek at some of the facility’s operations as well as get selfies with the famed Budweiser Clydesdales.

Speaking of big numbers, the show topped 5,100 attendees for the second year in a row, and more than 1,000 people logged in remotely to live-stream sessions. This represents a new level of success for The Steel Conference, not to mention a steady pace of growth, with attendance milestones of 4,500+ (Nashville, 2015), 3,700+ (Nashville, 2008), and 3,200+ (San Antonio, 2006) being achieved over the last several years. It’s a good sign not only for AISC but also for the steel industry and construction as a whole. Will the trend continue? We certainly hope so, but it’s up to you. Next year’s Steel Conference takes place in Atlanta, April 22–24, a stone’s throw from one of the city’s own structural steel marvels, this one quite modern: Mercedes-Benz Stadium (see “Rise Up” in the March 2017 issue, available at www.modernsteel.com). We hope to see you there! ■

I enjoyed the opportunity to attend so many in-person sessions of such high quality.

—Edward C. Westerman, SE, PE,
Clark Nexsen, Inc.

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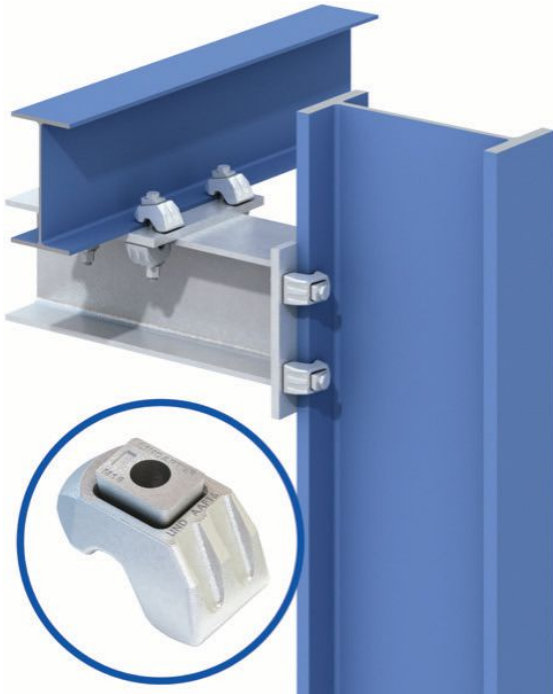
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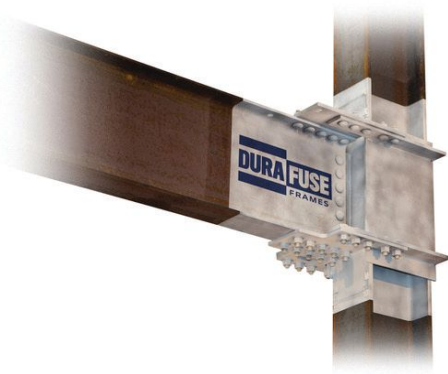
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RESOURCES

New Technical Resources Available on AISC Website

AISC has posted several new electronic resources that will benefit everyone using our standards and technical publications. Here's a quick look at these new free resources:

Linked Specification and Seismic Provisions. The electronic version of the 2016 *Specification* (ANSI/AISC 360-16) and 2016 *Seismic Provisions* (ANSI/AISC 341-16) are now enhanced with linked content and pre-set bookmarks. In these documents you will find a direct link at every section reference, saving you from having to scroll through the document or having to set up your own custom bookmarks. Navigating through these documents has never been so quick and easy.

New 15th Edition Manual Companion. We've released a new version of the "Design Examples" resource and given it a new name: "v15.1 *Companion to the AISC Steel Construction Manual*." In addition to the new name, we've split the document into two separate volumes: Vol. 1 *Design Examples* and Vol. 2 *Design Tables*. We want to highlight that this resource includes many useful design tables that supplement the *AISC Manual*.

If you're not familiar with this resource, we provide more than 1,600 pages of examples and tables that illustrate using the provisions of the 2016 *Specification* and the 15th Edition *Manual* for designing members, connections and structural systems.

Seismic Design Manual Interactive Reference List. Our website now has a complete listing of all the references found in the 3rd Edition *Seismic Design Manual*, including the 2016 *Seismic Provisions*. What makes this list interactive is a link is provided to where each reference can be obtained. Many of the references are available from the AISC website, while others are linked to the outside organization where the publication can be accessed or purchased. This resource will make it much easier to track down the background for the *Seismic Design Manual* and *Seismic Provisions*.

Visit the following URLs to access the above resources:

www.aisc.org/manualresources,
www.aisc.org/specifications, and
www.aisc.org/seismic.

UNIVERSITY NEWS

Robert Connor Becomes Named Faculty Member at Purdue University

Robert Connor, PhD, a professor at Purdue University's Lyles School of Engineering and a renowned expert on the fatigue and fracture of steel bridges, was recently ratified as Purdue's Jack and Kay Hockema Professor in Civil Engineering, one of the school's named faculty positions.

Connor, whose research on steel bridge fracture, fatigue, failure analysis, and design has led to many important technical standards, also serves as the director of Purdue's Center on Aging Infrastructure and the Steel Bridge Research, Inspection, Training, and Education Engineering Center (S-BRITE). He is also the associate director of the Bowen Laboratory for Large-Scale Civil Engineering Research.

Connor received the George S. Richardson Medal in 2016 and an AISC Special Achievement Award in 2012, and was the first recipient of the Robert J. Dexter

Memorial Lecture Award in 2005. He was also AISC's 2018 T.R. Higgins Lectureship Award winner.

Connor currently serves on AISC's Committee on Research and the NSBA Technical Committee.



Connor giving the AISC T.R. Higgins Lecture at the 2018 NASCC: The Steel Conference in Baltimore.

People and Companies

- **Patriot Erectors, LLC**, an AISC member and certified fabricator and erector based in Dripping Springs, Texas, has opened an additional fabrication facility in Rio Vista, Texas, adding 18,000 sq. ft to its existing 160,000 sq. ft of fabrication space. The new facility will produce a wide range of steel products, including embeds, stairs, and other heavy steel assemblies.

- **McLaren Engineering Group** announced that **Jeremy Billig, PE**, has been named the company's president. Billig, most recently the firm's vice president and New York City regional director, will lead McLaren along with **Malcolm McLaren, PE**, the company's CEO and founder. Billig's appointment follows a board-driven growth plan that includes the firm's recent expansions in New Jersey and Philadelphia. During his 15-year tenure at McLaren, Billig has directed a wide range of engineering projects while starting and growing the New York City office to over 40 staff members.



Billig and McLaren

- **DeSimone Consulting Engineers** and Solagna, Italy-based **Maffei Engineering** have announced a new joint venture to design sports and entertainment facilities as well as bridge and transportation projects globally. The strategic partnership will leverage the respective strengths of both firms and will advance new opportunities in Europe, Asia, and other high-growth markets.

CONSTRUCTION MARKET

Worker Shortage Possible as Construction Employment Soars

Associated General Contractors of America (AGC) warned that the combination of record-low numbers of unemployed job seekers and high weekly hour averages could point to a potential shortage of skilled workers. The group analyzed government employment data from March 2019.

“The data suggests that contractors are having a hard time finding qualified workers even though the industry pays better than the private sector as a whole,” said Ken Simonson, AGC’s chief economist.

AGC officials noted the lack of qualified construction workers could have an impact on future business and infrastructure projects, and have called on government officials

to double funding for training programs and facilitate immigration for skilled workers before a shortage stalls infrastructure projects.

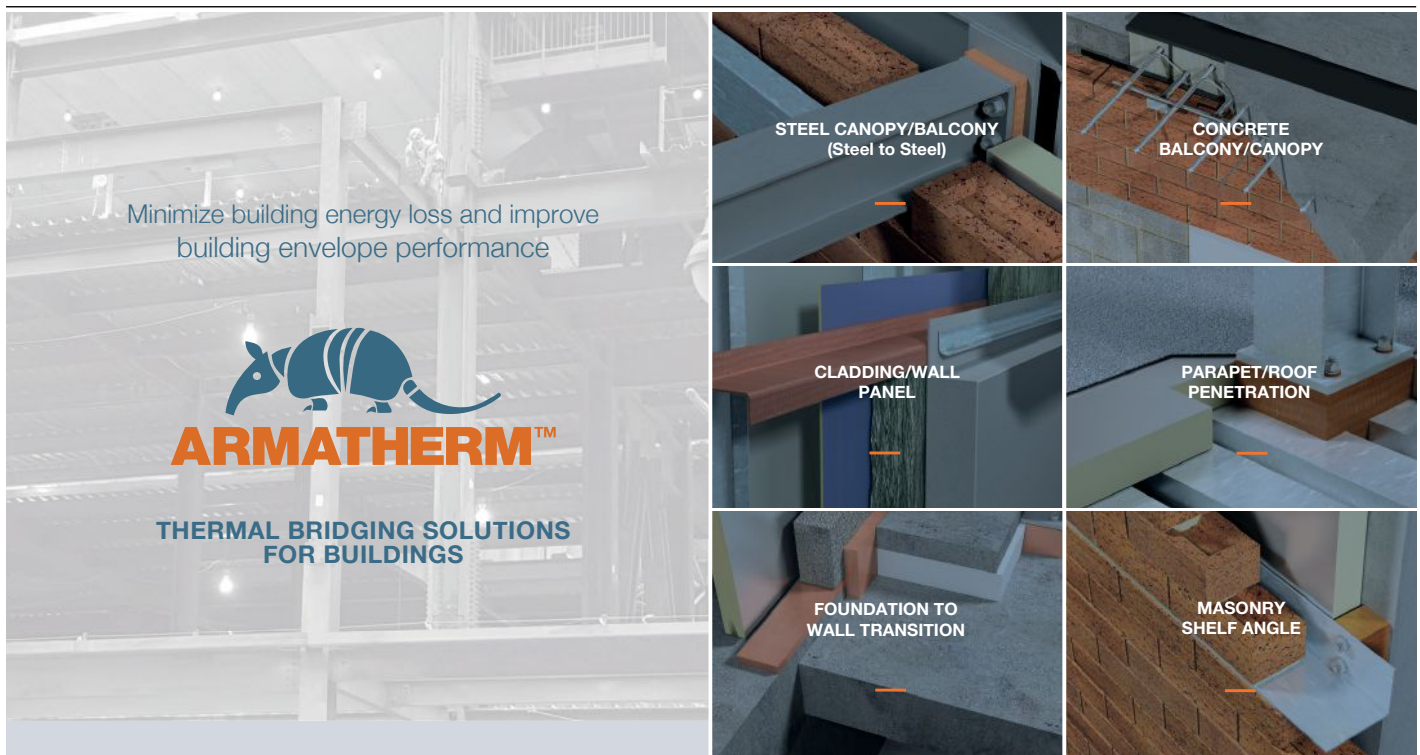
“Our member firms continue to worry about finding enough workers to fulfill the demand for construction,” said Stephen E. Sandherr, AGC’s CEO. “The only way to ensure that the construction industry continues to grow is to develop more skilled domestic workers that contractors need and to allow construction firms to seek qualified workers from outside the United States.”

According to the group’s analysis, total construction employment reached 7,447,000 in March 2019, rebounding from

losses in February that Simonson attributed to extreme weather conditions.

Construction industry employees worked an average of 39.9 hours a week last month, which is the highest March rate in the 14-year history of the series. Average hourly earnings have increased by 3.3% since the same time last year, to \$30.45.

The number of unemployed jobseekers whose last job was in construction also set a record. The 490,000 jobseekers last month constitute a record low since the series began in 2000; that number is also a steep decline from March 2016’s 696,000 unemployed job seekers whose last job was in construction.



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HIGGINS AWARD

Nominations Sought for 2020 Higgins Lectureship Award

Nominations are being accepted through July 15, 2019, for the prestigious T.R. Higgins Lectureship Award, which includes a \$15,000 cash prize. Presented annually by AISC, the award recognizes a lecturer-author whose technical paper(s) are considered an outstanding contribution to engineering literature on fabricated structural steel. The winner will be recognized at the 2020 NASCC: The Steel Conference, April 22–24 in Atlanta, and will also present their lecture, upon request, at various professional association events throughout the year.

Nominations should be emailed to AISC's Rachel Jordan at jordan@aisc.org. Or, if you'd prefer to mail your nomination, contact Rachel for mailing information. Nominations must include the following information:

- Name and affiliation of the individual nominated (past winners are not eligible to be nominated again)
- Title of the paper(s) for which the individual is nominated, including publication citation
- If the paper has multiple authors, identify the principal author

- Reasons for nomination
- A copy of the paper(s), as well as any published discussion

The author must be a permanent resident of the U.S. and available to fulfill the commitments of the award. The paper(s) must have been published in a professional journal between January 1, 2014 and January 1, 2019. In addition, the winner is required to attend and present at the 2020 Steel Conference and also give a minimum of six presentations of their lecture on selected occasions during the year.

The award will be given to a nominated individual based on their reputation as a lecturer and the jury's evaluation of the paper(s) named in the nomination. Papers will be judged for originality, clarity of presentation, contribution to engineering knowledge, future significance and value to the fabricated structural steel industry.

The current T.R. Higgins Lecturer is Ronald D. Ziemian, PhD, professor at Bucknell University, who received the award for his paper "Formulation and Validation of Minimum Brace Stiffness for Systems of Compression Members" as well as for his outstanding reputation

as an engineer and lecturer. If your organization is interested in hosting a T.R. Higgins lecture, please contact Christina Harber, AISC's director of education, at harber@aisc.org.

The award is named for Theodore R. Higgins, former AISC director of engineering and research, who was widely acclaimed for his many contributions to the advancement of engineering technology related to fabricated structural steel. The award honors Higgins for his innovative engineering, timely technical papers and distinguished lectures. For more information about the award, visit www.aisc.org/higgins.



Ron Ziemian, current T.R. Higgins lecturer.

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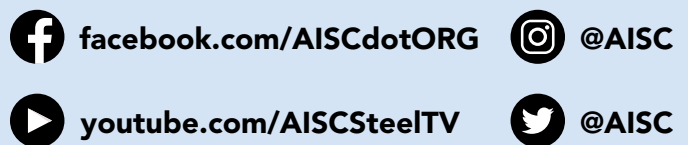
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structurally sound



FINAL(S) COUNTDOWN

BY THE TIME you read this, the 2019 AISC Student Steel Bridge Competition (SSBC) will be completed and the winner crowned.

While we can't show the results in this issue (that's not how printer deadlines work, plus we didn't have a crystal ball handy at press time—or at least not one that works) we can tell you that around 40 student steel bridge teams, comprised of the top few finishers from 17 regional competitions (such as the Pacific Northwest competition at Saint Martin's University in Lacey, Wash., pictured above), gave it their all at

the 2019 SSBC National Finals at Southern Illinois University in Carbondale, Ill., on May 31 and June 1. And for the first time ever, a wild card system was implemented, where a handful of teams who performed well but just missed the cut in their region were entered into a drawing and invited to compete in Carbondale.

Visit www.aisc.org/ssbc for more information on the competition as well as this year's results—which we'll cover in the August issue, including some great photography of the action.

SUCCESS STORY: Endres Manufacturing

Endres overcomes production challenges with PythonX

CHALLENGES

1. ELIMINATE BOTTLENECK CREATED BY FABRICATORS

Before the PythonX[®], a band saw and punch were used and a lot of time was spent interpreting drawings, laying out pieces and then performing copes and cuts manually.

2. CHANGING WORKFORCE

As more experienced employees were nearing retirement, Endres had to find a way to process steel with less experience on the shop floor.

SOLUTION

RESEARCH

Endres began to research several different pieces of equipment but they were drawn to the PythonX. One of the main reasons was the lifespan. Many PythonX systems have been in service for more than 10 years. They were also drawn to the number of PythonX systems that have been sold around the world, over 300.

THE PRODUCT

PythonX, a versatile and complete solution that requires only one operator and no programming. All processing operations are automated resulting in increased productivity, unmatched cut quality, predictable and consistent throughput.



RESULTS

QUICK ROI

When Endres purchased the PythonX, they estimated it would be about 3 years before they saw a return on investment but that was not the case. When the PythonX was fully up to speed, it had only taken 11 months for the PythonX to pay for itself.

DECREASED PRODUCTION HOURS

PythonX drastically reduced production time that led to delivering a better quality product faster. It opened opportunities to bid on different kinds of projects that led to a period of growth and now they are strongly considering purchasing a second PythonX.

Since opening its doors in 1926, Endres Manufacturing has been an innovative family business in its fourth-generation based in Waunakee, WI, U.S.A.



SAM BALLWEG

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