Modern Steel Construction

February 2024


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Modern Steel Construction

February 2024

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ON THE COVER: A steel flyover has helped alleviate a bottleneck at a busy train interchange in Chicago, p. 32. (Photo: Dave Burke Photography)

MODERN STEEL CONSTRUCTION (Volume 64, Number 2) ISSN (print) 0026-8445: ISSN (online) 1945-0737. Published monthly by the American Institute of Steel Construction (AISC), 130 E Randolph Street, Suite 2000, Chicago, IL 60601. Single issues $8.00; 1 year, $60. Periodicals postage paid at Chicago, IL and at additional mailing offices. Postmaster: Please send address changes to MODERN STEEL CONSTRUCTION, 130 E Randolph Street, Suite 2000, Chicago, IL 60601.

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editor's note

By the time you read this, the Super Bowl will be just a couple of weeks away.

The magical time of year known as college football bowl season will have ended (from heroic battles between directional state universities to the usual big-name suspects vying for a high final ranking), and we'll all be knee-deep in our new year's resolutions—or have already abandoned them or rolled them over to next year.

It's a time when many of us are looking forward to (or bracing for) whatever the still-new year has in store for us. We all incorporate benchmarks into our calendars, and one of mine is trips to look forward to, whether for work, vacation, a family visit, or a combination of those. I have a great one planned for late January, and I'll brag—er, talk—about that one next month. My next scheduled trip after that one is to San Antonio in March for NASCC: The Steel Conference, and if you're thinking of putting it on your own list of 2024 travel plans—and you definitely should be—head over to **aisc.org/nascc** for registration details, session and exhibitor information, and more.

Speaking of trips (and Steel Conference exhibitors), my wife and I traveled to Ireland in September, where we spent most of our time in Dublin and Galway—which are on opposite coasts. Luckily, unlike the U.S., a coast-to-coast train trip in Ireland takes less than two hours. Among other things, we had a pint in what is allegedly the country's oldest pub (which dates back to the 1100s), another pint at the country's most famous brewery (you know the one, and yes, it tastes better there), and several other pints in some of the most charming pubs you'll find on earth. We ate oysters as big as my palm, we visited the Cliffs of Moher, we marveled at the variety of soda bread recipes, we watched Ireland beat South Africa in a group play match of the Rugby World Cup at a crowded and raucous pub, and we generally had a great time wandering around and discovering things like a statue of Phil Lynott, the late lead singer and bass player of famed Irish rock band Thin Lizzy.

While on this trip, we also paid a visit to the world headquarters of Combilift in the lovely town of Monaghan and attended the company's 25th anniversary celebration, which included a tour of its manufacturing facility. Among other things, the tour provided an introduction to the new Combi Connect technology, designed to remotely monitor the performance of a facility's various Combilift machines, as well as a new autonomous lift that's geared toward moving materials at steel service centers. The company's small-town origins and rise to becoming one of the world's most successful forklift (and other lifting/moving machinery) manufacturers are truly inspirational, as was a quote I heard from one of its employees on the tour: "Customers are the best engineers."

Combilift is just one of the more than 300 exhibitors that will be showcasing their products, machinery, and services—as well as their own inspirational stories—at The Steel Conference. And every single one of them is focused on improving its own link in the steel supply chain, whether that's safety, ease of use, speed, efficiency, or sustainability. If you want to see the tools that help make your steel projects run as smoothly as possible, there is simply no better time or place to be than March 20–22 in San Antonio.

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steel interchange

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

Shim and Filler Plate Thickness

What is the minimum thickness of shim or filler plates that can be used in a structural steel connection?

There are no minimum thickness requirements for shim or filler plates in the AISC *Specification for Structural Steel Buildings* (AISC 360-22, a free download at **aisc.org/standards**). It's uncommon to see plates less than ⅛ in. thick used in practice.

From a practical standpoint, shims are generally avoided unless structures are detailed anticipating the use of shims, which is sometimes done when moment end plate connections are used or in the case of bolted flange plate moment connections. Where shims are used, they are usually made as thick as practical to fill the gap while still being able to be inserted in a reasonable manner. Since shims greater than ¼ in. reduce the strength of the bolted connections, the use of such shims must be approved by the engineer of record. For the effect of fillers and shims on available joint strength, see *Specification* Sections J3.9 and J5.2.

Larry Muir, PE

Missing Steel-Headed Stud Anchors

The design drawings indicate an area with composite metal decking, but no studs are shown. The general contractor believes that steel headed stud anchors (studs) are required, but when asked, the engineer of record has indicated that no studs are required. Is this possible?

Yes, it's possible depending on how the system was designed. Concrete on metal deck supported by steel beams can be designed as composite or non-composite construction, the decision of which is left up to the engineer of record.

If the concrete on metal deck is designed as non-composite, then no studs or other load transfer mechanism between the steel beams and concrete slab is required.

If the concrete on metal deck is designed as composite, then steel headed studs are required. This is stated in the 2022 AISC *Specification*. Section I3.2c addresses composite beams with formed steel deck and states, "The concrete slab shall be connected to the steel beam with steel headed stud anchors welded either through the deck or directly to the steel cross section." The headed studs are required for the load transfer between the steel beam and concrete slab.

It's possible that the metal deck could be the cause of this confusion. The metal deck itself can be classified as either composite or non-composite—irrespective of the structural system being non-composite or composite with the steel beams. The difference is that composite deck has indentations in the ribs that help create a mechanical interlock between the concrete and metal deck. An engineer could choose a composite deck profile but design the steel beams as non-composite with the concrete slab.

Yasmin Chaudhry, PE

End-Plate Moment Connection Equation Discrepancy

I noticed that the yield line formula for unstiffened columns in Table 6-5 of AISC's *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (ANSI/AISC 358-22)* is similar to the four-bolt flushed unstiffened moment connection provided in Design Guide 39: *End-Plate Moment Connections*, Table 5-3 and Design Guide 16: *Flush and Extended Multiple Row Moment End-Plate Connections*, Table 3-3. The equations match when I set $c = p_b$, except for the $c^2/2$ term. Is there a mistake?

The yield line patterns for an unstiffened column flange and a four-bolt flush end plate are identical if $p_{fi} > s$. The equations for the yield line mechanism parameters (Y_c for the column flange and Y_p for the end plate) are, in fact, identical. However, they are in slightly different forms.

When comparing the form of the equation found in AISC 358-22 Table 6-5 (p. 10) to the form shown in Design Guides 39 (p. 10) and 16 (not shown), notice that h_1 and h_2 are also switched in the two terms preceding $c^2/2$. The three terms in the brackets from AISC 358 Table 6-5 can be rearranged as shown below:

$$\left[h_2\left(s+\frac{3c}{4}\right)+h_1\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]$$

$$=\left[h_2\left(s+\frac{c}{4}\right)+h_2\left(\frac{c}{2}\right)+h_1\left(s+\frac{3c}{4}\right)-h_1\left(\frac{c}{2}\right)+\frac{c^2}{2}\right]$$

$$=\left[h_2\left(s+\frac{c}{4}\right)+\left(h_1-c\right)\left(\frac{c}{2}\right)+h_1\left(s+\frac{3c}{4}\right)-h_1\left(\frac{c}{2}\right)+\frac{c^2}{2}\right]$$

$$=\left[h_2\left(s+\frac{c}{4}\right)+h_1\left(\frac{c}{2}\right)-\frac{c^2}{2}+h_1\left(s+\frac{3c}{4}\right)-h_1\left(\frac{c}{2}\right)+\frac{c^2}{2}\right]$$

$$=\left[h_1\left(s+\frac{3c}{4}\right)+h_2\left(s+\frac{c}{4}\right)\right]$$

All mentioned AISC publications, unless noted otherwise, refer to the current version and are available at **aisc.org/publications**. *Modern Steel* articles can be found at **www.modernsteel.com**.

The result is the same mathematical form as used in the design guide equations, without the extra $c^2/2$.

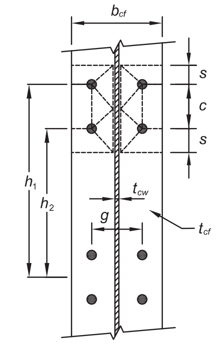
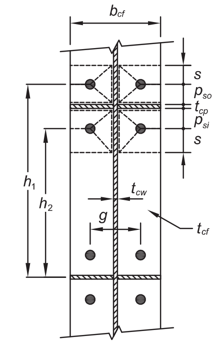
AISC Design Guide 39, released in 2023, has replaced Design Guide 16 and is the best resource for end-plate design. It is available as a free download for AISC members at **aisc.org/dg**.

Michael Desch, PhD

right: Table 6-5 (partial) from AISC 358-22.

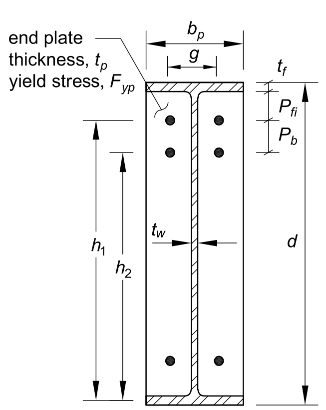
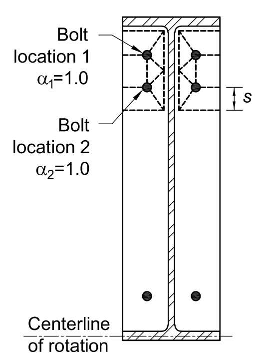
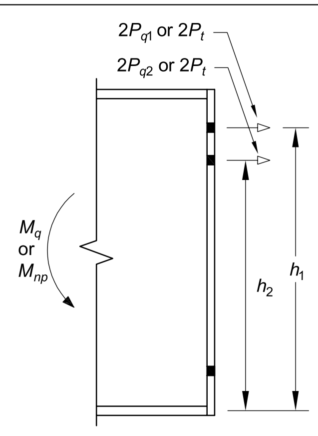
below: Table 5-3 (partial) from AISC Design Guide 39: *End-Plate Moment Connections*.

TABLE 6.5
Summary of Four-Bolt Extended Column Flange Yield Line Mechanism Parameter

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	

Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_2\left(\frac{1}{s}\right)+h_1\left(\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_2\left(s+\frac{3c}{4}\right)+h_1\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$ $s=\frac{1}{2}\sqrt{b_{cf}g}$

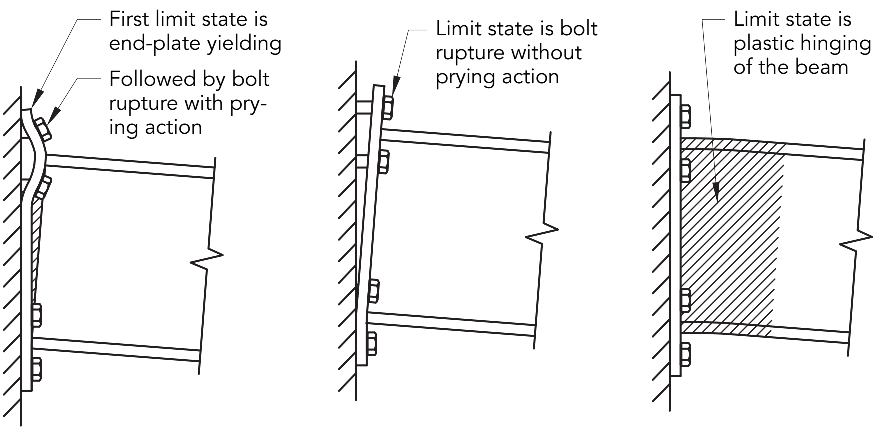
Table 5-3. Summary of Design Equations for Four-Bolt Flush Unstiffened End Plate

Geometry	Yield-Line Mechanism	Bolt Force Model
		

End-Plate Yield	$\phi M_n = \phi_b M_{pl} = \phi_b F_{yp} t_p^2 Y_p$ $Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}}\right)+h_2\left(\frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(p_{fi}+\frac{3p_b}{4}\right)+h_2\left(s+\frac{p_b}{4}\right)\right]+\frac{g}{2}$	$\phi_b = 0.90$ $s = \frac{\sqrt{b_p g}}{2}$ Note: Use $p_{fi} = s$ if $p_{fi} > s$

Michael Desch (desch@aisc.org) is a staff engineer and **Yasmin Chaudhry** (chaudhry@aisc.org) is a senior engineer, both in AISC's Steel Solutions Center. **Larry Muir** is a consultant to AISC.

What do you get when you add 4 to 16? Well, 39, of course. (Good thing that wasn't a quiz question!) This month's quiz tests your knowledge of the end-all, be-all for end-plate moment connections: AISC Design Guide 39: *End-Plate Moment Connections*. Download your copy of it today at **aisc.org/dg**, and check out the SteelWise article in the December issue for even more information. This new guide supersedes two previous end-plate design guides: Design Guide 4 and Design Guide 16.



(a) Thin end-plate behavior (b) Thick end-plate behavior (c) Plastic hinging behavior

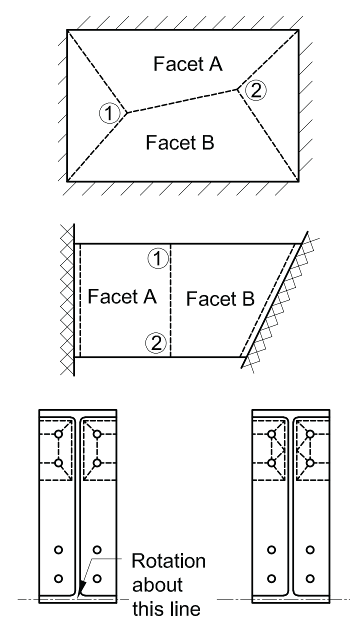
1 Which design approach(es) [left] is/are most common for gravity, wind, and low-seismic end-plate moment connections?
a. Thin end-plate behavior
b. Thick end-plate behavior
c. Plastic hinging behavior
d. **(a.)** and **(b.)**
e. All of the above

2 **True or False:** An end-plate moment connection must be designed as slip-critical.

3 **True or False:** End-plate moment connections designed according to Design Guide 39 are intended for fully restrained (FR) construction, not partially restrained (PR).

4 **True or False:** Design Guide 39 procedures assume that a concrete slab (if present) contributes to the end-plate moment connection behavior.

5 Which of the following is an *admissible* yield line mechanism.



6 **True or False:** The shear force at an end-plate connection is generally assumed to be resisted by the *tension* side bolts in the Design Guide 39 procedures.

TURN TO PAGE 14 FOR ANSWERS

Answers reference the recently published Design Guide 39. This new guide supersedes two previous AISC Design Guides—4 and 16. Download your copy today at **aisc.org/dg**.

1 **d. (a.)** and **(b.)** Design Guide 39 describes three distinct types of behavior for end-plate moment connections in Chapter 3: thin end-plate, thick end-plate, and plastic hinging behavior. Thin end-plate behavior and thick end-plate behavior are most common for gravity, wind, and low-seismic ductility design. In thin end-plate behavior, end-plate flexure is the controlling limit state, and the bolts are assumed to be subject to prying action. In thick end-plate behavior, the controlling limit state is bolt rupture without prying action. End-plate moment connections designed for plastic hinging behavior are used for high-seismic-ductility design where the design approach focuses on achieving inelastic rotation capacity which comes from plastic hinging of the beam.

2 **False.** According to Section 3.5, one of the many advantages of end-plate moment connections is that they do not need to be designed as slip critical for static (temperature, wind, and snow) or seismic loading. This allows relaxed surface preparation as compared to other bolted moment connections such as bolted flange plate connections.

3 **True.** End-plate moment connections designed according to Design Guide 39 are intended for fully restrained construction, per Section 3.2. Connections between steel members can be categorized as simple, partially restrained (PR), or fully restrained (FR). Simple connections, as defined in AISC *Specification for Structural Steel Buildings* Section B3.4a, transmit negligible moment. Moment connections are categorized as either PR or FR, as defined in the *Specification* Section B3.4b. FR connections are defined as transmitting moment with negligible rotation between the connected members.

4 **False.** Per Section 4.2.4, the design procedures in Design Guide 39 assume that if a concrete slab is present, it *does not* contribute significantly to the moment connection behavior. The composite slab should therefore be detailed with a block out around the end plate and column flange to permit insertion of compressible material. See Section 4.2 for more information on detailing considerations.

5 **d.** The end-plate and column flange bending strengths are determined using yield line analysis in the recommended design procedures. Yield lines are the continuous formation of plastic hinges along a straight or curved line. Yield lines are assumed to divide a plate into rigid facets. For a yield line pattern to be valid, two criteria must be satisfied: (1) All facets in the yield line pattern must be planar, and (2) displacements along the boundary of two facets must be compatible. Figures (a.), (b.), and (c.) are not valid because the patterns shown cannot meet both criteria. See Section 3.3 for further explanation.

6 **False.** The shear force at an end-plate connection is generally assumed to be resisted by the *compression* side bolts in the design procedures presented in Design Guide 39. This is a convenient assumption that allows the tension and shear forces to be separated into different groups of bolts. However, if the connection is subjected to axial tension, it may be necessary to design the bolts for the combined effects of shear and tension. See Section 3.5 for further explanation.

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

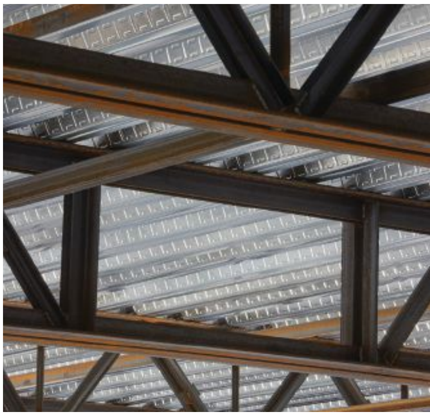


Table Talk

BY ERIC BOLIN AND THOMAS M. MURRAY, PE, PHD

Grade 50 connection material is highlighted in the reformatted and updated Part 10 tables in the *16th Edition Steel Construction Manual.*

THE AUTHORITATIVE PUBLICATION on steel design has revamped one of its most important elements.

The January issue of *Modern Steel Construction* featured an overview of all the major updates in the new 16th Edition *Steel Construction Manual.* This month's SteelWise section dives into one of the *Manual*'s most significant changes: the reformatted simple shear connection tables found in Part 10.

One major change is that all the connection materials in the *Manual* have been updated from ASTM A36/A36M to ASTM A572A572M Grade 50. The use of 36 ksi connection material is becoming less prevalent, and the *Manual* has been updated with the connection materials commonly used today.

In addition to the change in the connection material strength, the Part 10 connection tables that utilize bolts have been reformatted and expanded. The new tables will allow for an easier determination of the effective bolt shear transfer strength and the shear strength of the supported beam web when coped. The four tables in Part 10 that have these updates are:

- Table 10-1—All-Bolted Double-Angle Connections
- Table 10-4—Shear End-Plate Connections
- Table 10-10—Single-Plate Connections
- Table 10-12—Single-Angle Connections

All four have received similar treatment in the *Manual.* The changes in Table 10-1 are detailed in this article, and similar changes apply to Tables 10-4, 10-10, and 10-12.

A complete design example illustrating the use of the new Table 10-1 is provided at the end of this article. AISC has published design examples for all four of the new tables in Part 10 of the *Manual*, in addition to many other connection types, in the V16.0 *Companion to the AISC Steel Construction Manual, Vol. 1: Design Examples*, which is available for free download at **aisc.org/manualresources**.

New *Manual* Table 10-1

Table 10-1 is a design aid used to determine the available shear strength of all-bolted double-angle connections. In the 16th edition, Table 10-1 has been expanded into three sub-tables: Table 10-1a provides the available strength of the angles, Table 10-1b tabulates values to aid in the calculation of the available shear transfer strength at bolt holes, and Table 10-1c provides the coped beam web available shear strength. The available shear strength of the connection is the minimum value determined from the sub-tables.

The connection strength is determined differently whether the supported beam is coped. For an uncoped beam or a beam coped at the bottom flange only, the connection strength is the minimum strength determined from Tables 10-1a and 10-1b. When the beam is coped at the top, or top and bottom flanges, the connection strength is the minimum strength determined from Tables 10-1a, 10-1b, and 10-1c.

It is noted that flexural strength of the coped beam is not addressed in Table 10-1 and must be checked separately. *Manual* Part 9 includes a procedure for checking the flexural strength of a coped beam web.

Table 10-1a: Available Angle Strength

Table 10-1a (see Figure 1) provides the available angle strength of an all-bolted double-angle connection. This table is similar to Table 10-1 from the 15th edition. However, the limit states of bolt shear, bolt bearing, and bolt tearout were moved to the new Table 10-1b. Table 10-1a tabulates the available angle strength based on the limit states of shear yielding, shear rupture, and block shear rupture at the outstanding angle leg (OSL) and supported beam web leg. Table 10-1a is split into two parts, one for standard and short-slotted holes and one for oversized holes for use with slip-critical connections.

There is a recommendation in Manual Part 10 that the minimum connection length be one-half of the *T*-dimension of the supported beam to provide for beam end stability during erection. Table 10-1a includes a column of "Beam Sizes" which lists the nominal beam depths where a given connection length lands between the full *T*-dimension and the one-half *T*-dimension. This information will guide initial selection of the connection length for a given beam depth.

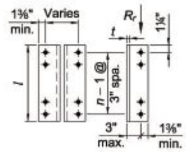
Table 10-1b: Available Shear Transfer Strength at Bolt Holes

Table 10-1b (see Figure 2) is a new addition to the *Manual.* This table aids in the determination of the available shear transfer strength at bolt holes.

The shear transfer strength at bolt holes is a culmination of multiple limit states that need to be checked where bolts transfer shear through a connection. This method is specified in a user note in *Specification for Structural Steel Buildings* Section J3.7, which states, "The effective strength of an individual fastener may be taken as the lesser of the fastener shear strength per Section J3.7 or the bearing or tearout strength at the bolt hole per Section J3.11. The strength of the bolt group is taken as the sum of the effective strengths of the individual fasteners."

An article in the May 2017 issue of *Modern Steel Construction* titled "A Tale of Tearouts" discusses the interaction of bolt shear, bolt bearing, and bolt tearout limit states in bolted connections.

Five limit states must be checked at each bolt hole: (1) bolt shear strength,



STD, SSLT Holes

Table 10-1a
All-Bolted Double-Angle Connections
Available Angle Strength

Number of Bolt Rows, n	Beam Sizes	Length, l, in.	Bolt Diameter, in.	Available Angle Strength, kips — Angle Thickness, t, in. — ¼ — R_n/Ω ASD	¼ — ϕR_n LRFD	5/16 — R_n/Ω ASD	5/16 — ϕR_n LRFD	⅜ — R_n/Ω ASD	⅜ — ϕR_n LRFD	½ — R_n/Ω ASD	½ — ϕR_n LRFD
12	W44	35½	¾	244	366	305	457	366	548	488	731
			⅞	229	344	286	430	344	516	458	687
			1	207	311	259	388	311	466	414	622
11	W44, 40	32½	¾	223	335	279	418	335	502	446	669
			⅞	210	314	262	393	314	472	419	629
			1	190	284	237	355	284	426	379	569
10	W44, 40, 36	29½	¾	202	303	253	379	303	455	405	607
			⅞	190	285	238	356	285	428	380	570
			1	172	258	215	322	258	387	344	516
9	W44, 40, 36, 33	26½	¾	182	272	227	340	272	409	363	545
			⅞	171	256	213	320	256	384	341	512
			1	154	231	193	289	231	347	308	463
8	W44, 40, 36, 33, 30	23½	¾	161	241	201	302	241	362	322	483
			⅞	151	227	189	283	227	340	302	453
			1	137	205	171	256	205	307	273	410
7	W44, 40, 36, 33, 30, 27, 24	20½	¾	140	210	175	263	210	315	280	420
			⅞	132	197	165	247	197	296	263	395
			1	119	178	149	223	178	267	238	356
6	W40, 36, 33, 30, 27, 24, 21	17½	¾	119	179	149	224	179	269	239	358
			⅞	112	168	140	210	168	252	224	336
			1	101	152	126	190	152	228	202	303
5	W30, 27, 24, 21, 18	14½	¾	98.7	148	123	185	148	222	197	296
			⅞	92.6	139	116	174	139	208	185	278
			1	83.5	125	104	157	125	188	167	250
4	W24, 21, 18, 16	11½	¾	78.0	117	97.5	146	117	176	156	234
			⅞	73.1	110	91.4	137	110	165	146	219
			1	65.8	98.7	82.3	123	98.7	148	132	197
3	W18, 16, 14, 12, 10[a]	8½	¾	57.3	85.9	71.6	107	85.9	129	115	172
			⅞	53.6	80.4	67.0	101	80.4	121	107	161
			1	48.1	72.2	60.2	90.3	72.2	108	96.3	144
2	W12, 10, 8	5½	¾	36.6	54.8	45.7	68.6	54.8	82.3	73.1	110
			⅞	34.1	51.2	42.7	64.0	51.2	76.8	68.3	102
			1	30.5	45.7	38.1	57.1	45.7	68.6	60.9	91.4

STD = standard hole
SSLT = short-slotted hole with length transverse to the direction of force
[a]Limited to W10×12, 15, 17, 19, 22, 26, 30.

Angles
F_y = 50 ksi
F_u = 65 ksi

Fig. 1. *Manual* Table 10-1a (the selection for the design example is shown in the red box).

Table 10-1b
All-Bolted Double-Angle Connections
Available Shear Transfer Strength at Bolt Holes

Available Bolt Shear Strength, kips[a]

Designation	Hole Type	Thread Condition	¾ — r_n/Ω ASD	¾ — ϕr_n LRFD	⅞ — r_n/Ω ASD	⅞ — ϕr_n LRFD	1 — r_n/Ω ASD	1 — ϕr_n LRFD
Group 120	STD/SSLT	N	11.9	17.9	16.2	24.3	21.2	31.8
		X	15.0	22.5	20.4	30.7	26.7	40.0
Group 144	STD/SSLT	N	14.4	21.6	19.5	29.3	25.5	38.3
		X	17.9	26.9	24.3	36.5	31.8	47.7
Group 150	STD/SSLT	N	15.0	22.5	20.4	30.7	26.7	40.0
		X	18.6	27.8	25.2	37.9	33.0	49.5

Available Slip Resistance Strength, kips[a], [b]

Designation	Faying Surface	Hole Type	¾ — r_n/Ω ASD	¾ — ϕr_n LRFD	⅞ — r_n/Ω ASD	⅞ — ϕr_n LRFD	1 — r_n/Ω ASD	1 — ϕr_n LRFD
Group 120	Class A[c]	STD/SSLT	6.33	9.49	8.81	13.2	11.5	17.3
		OVS	5.39	8.07	7.51	11.2	9.82	14.7
Group 144	Class A[c]	STD/SSLT	7.91	11.9	11.1	16.6	14.5	21.7
		OVS	6.74	10.1	9.44	14.1	12.3	18.4
Group 150	Class A[c]	STD/SSLT	7.91	11.9	11.1	16.6	14.5	21.7
		OVS	6.74	10.1	9.44	14.1	12.3	18.4

Available Bearing and Tearout Strength at Edge Bolt per Inch Thickness, kip/in.[d]

Bolt Edge Distance, l_{ev}, in.	¾ STD/SSLT r_n/Ω ASD	¾ STD/SSLT ϕr_n LRFD	¾ OVS r_n/Ω ASD	¾ OVS ϕr_n LRFD	⅞ STD/SSLT r_n/Ω ASD	⅞ STD/SSLT ϕr_n LRFD	⅞ OVS r_n/Ω ASD	⅞ OVS ϕr_n LRFD	1 STD/SSLT r_n/Ω ASD	1 STD/SSLT ϕr_n LRFD	1 OVS r_n/Ω ASD	1 OVS ϕr_n LRFD
1¼	32.9	49.4	30.5	45.7	30.5	45.7	28.0	42.0	26.8	40.2	–	–
1⅜	37.8	56.7	35.3	53.0	35.3	53.0	32.9	49.4	31.7	47.5	29.3	43.9
1½	42.7	64.0	40.2	60.3	40.2	60.3	37.8	56.7	36.6	54.8	34.1	51.2
1¾	52.4	78.6	50.0	75.0	50.0	75.0	47.5	71.3	46.3	69.5	43.9	65.8
2	58.5	87.8	58.5	87.8	59.7	89.6	57.3	85.9	56.1	84.1	53.6	80.4
2½					68.3	102	68.3	102	75.6	113	73.1	110
3									78.0	117	78.0	117

Available Bearing and Tearout Strength at Non-Edge Bolt per Inch Thickness, kip/in.

Bolt Hole Spacing, s, in.	¾ STD/SSLT r_n/Ω ASD	¾ STD/SSLT ϕr_n LRFD	¾ OVS r_n/Ω ASD	¾ OVS ϕr_n LRFD	⅞ STD/SSLT r_n/Ω ASD	⅞ STD/SSLT ϕr_n LRFD	⅞ OVS r_n/Ω ASD	⅞ OVS ϕr_n LRFD	1 STD/SSLT r_n/Ω ASD	1 STD/SSLT ϕr_n LRFD	1 OVS r_n/Ω ASD	1 OVS ϕr_n LRFD
3	58.5	87.8	58.5	87.8	68.3	102	68.3	102	73.1	110	68.3	102

STD = standard hole
OVS = oversized hole
SSLT = short-slotted holes with length transverse to the direction of force
N = Threads included
X = Threads excluded
[a]Strength for bolt loaded in single shear, multiply tabluated available strength by 2 for bolt loaded in double shear.
[b]Available slip resistance strength assumes no more than one filler has been provided.
[c]For Class B faying surfaces, multiply tabulated available strength by 1.67.
[d]Values above heavy line are governed by limit state of tearout. Values below heavy line are governed by limit state of bolt bearing. The available bolt bearing strength may be used where values are omitted below the heavy line.
– Does not meet the minimum edge distance requirement of AISC *Specification* Section J3.5.
Group 120 includes ASTM A3125/A3125M Grades A325 and F1852 bolts.
Group 144 includes ASTM F3148 bolts.
Group 150 includes ASTM A3125/A3125M Grades A490 and F2280 bolts.

Angles, Beam, and Support
F_y = 50 ksi
F_u = 65 ksi

Fig. 2. *Manual* Table 10-1b (the selection for the design example is shown in the red boxes).

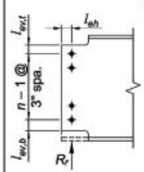
(2) bearing strength of the connection material, (3) tearout strength of the connection material, (4) bearing strength of the support or supported member, and (5) tearout strength of the support or supported member. The available shear transfer strength is the minimum of these strengths. Because the bolt tearout clear distance can vary between edge and non-edge bolts, the shear transfer strengths at each row of bolts need to be evaluated separately.

Table 10-1b includes all the necessary values to determine the shear transfer strength at bolt holes for the connection. The available bolt shear strength is tabulated at the top of the table. The table's middle and bottom portions provide the minimum of the bearing or tearout strength for either "edge bolts" or "non-edge bolts."

The "edge bolt" values are to be used where a bolt hole is near the edge of material in the direction of force. The "non-edge bolt" values are used for tearout between bolt holes, such as the center bolts in a typical shear connection. The available bearing and tearout strengths are tabulated in kips per inch of thickness, which means the tabulated value must be multiplied by the thickness of the material being checked.

Table 10-1b also includes the slip-resistance values for use in slip-critical connections. The available slip-resistance strength of the connection is determined by multiplying the slip-resistance strength per bolt by the number of connection bolts. AISC *Specification* Section J3.9 states that "slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections," therefore a slip-critical connection must still be checked for the available shear transfer strength limit states.

Table 10-1c
All-Bolted Double-Angle Connections
Coped Beam Web Available Shear Strength per Inch Thickness, kip/in.

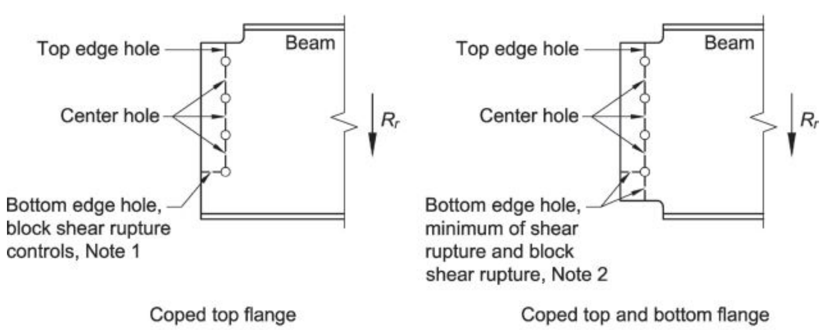


		Bolt Diameter, in.											
		3/4				7/8				1			
Hole Type		STD		OVS		STD		OVS		STD		OVS	
Top Edge Hole, kip/in.													
$l_{ev,t}$, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
1¼		15.8	23.8	14.6	21.9	14.6	21.9	13.4	20.1	12.8	19.2	–	–
1⅜		18.3	27.4	17.1	25.6	17.1	25.6	15.8	23.8	15.2	22.9	14.0	21.0
1½		20.7	31.1	19.5	29.3	19.5	29.3	18.3	27.4	17.7	26.5	16.5	24.7
Center Hole, kip/in.													
Bolt Hole Spacing, s, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3		41.4	62.2	39.0	58.5	39.0	58.5	36.6	54.8	35.3	53.0	32.9	49.4
Bottom Edge Hole, kip/in.[a]													
l_{eh}, in.[b]	$l_{ev,b}$, in.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
1¼	1¼	15.8	23.8	–	–	–	–	–	–	–	–	–	–
	≥1½	18.3	27.4	–	–	–	–	–	–	–	–	–	–
1½	1¼	15.8	23.8	14.6	21.9	14.6	21.9	13.4	20.1	12.8	19.2	–	–
	1½	20.7	31.1	19.5	29.3	19.5	29.3	18.3	27.4	17.7	26.5	–	–
	1¾	25.6	38.4	24.4	36.6	24.4	36.6	22.3	33.5	21.3	32.0	–	–
	≥2	26.4	39.6	24.4	36.6							–	–
1¾	1¼	15.8	23.8	14.6	21.9	14.6	21.9	13.4	20.1	12.8	19.2	–	–
	1½	20.7	31.1	19.5	29.3	19.5	29.3	18.3	27.4	17.7	26.5	16.5	24.7
	1¾	25.6	38.4	24.4	36.6	24.4	36.6	23.2	34.7	22.5	33.8	21.3	32.0
	2	30.5	45.7	29.3	43.9	29.3	43.9	28.0	42.0	27.4	41.1	26.2	39.3
	≥2¼	34.5	51.8	32.5	48.8	32.5	48.8	30.5	45.7	29.5	44.2	27.4	41.1
2	1¼	15.8	23.8	14.6	21.9	14.6	21.9	13.4	20.1	12.8	19.2	–	–
	1½	20.7	31.1	19.5	29.3	19.5	29.3	18.3	27.4	17.7	26.5	16.5	24.7
	1¾	25.6	38.4	24.4	36.6	24.4	36.6	23.2	34.7	22.5	33.8	21.3	32.0
	2	30.5	45.7	29.3	43.9	29.3	43.9	28.0	42.0	27.4	41.1	26.2	39.3
	2½	40.2	60.3	39.0	58.5	39.0	58.5	37.8	56.7	37.2	55.8	35.5	53.3
	≥2¾	42.7	64.0	40.6	60.9	40.6	60.9	38.6	57.9	37.6	56.4		
2¼	1¼	15.8	23.8	14.6	21.9	14.6	21.9	13.4	20.1	12.8	19.2	–	–
	1½	20.7	31.1	19.5	29.3	19.5	29.3	18.3	27.4	17.7	26.5	16.5	24.7
	2	30.5	45.7	29.3	43.9	29.3	43.9	28.0	42.0	27.4	41.1	26.2	39.3
	2½	40.2	60.3	39.0	58.5	39.0	58.5	37.8	56.7	37.2	55.8	36.0	53.9
	3	50.0	75.0	48.8	73.1	48.8	73.1	46.7	70.1	45.7	68.6	43.7	65.5
	≥3¼	50.8	76.2	48.8	73.1								

Beam
$F_y = 50$ ksi $F_u = 65$ ksi

STD = standard hole
OVS = oversized hole
[a]Values above the heavy line are governed by the limit state of shear rupture. Values below the heavy line are governed by the limit state of block shear rupture. When the beam is coped at the top flange only, the available block shear rupture strength is used. The available block shear rupture strength may be used where values are omitted below the heavy line.
[b]Tabulated values include a ¼ in. reduction in end distance, l_{eh}, to account for possible underrun in beam length.
Note: The limit states of coped beam flexural yielding and local buckling must be independently checked.
– Does not meet the minimum edge distance requirement of AISC *Specification* Section J3.5.

Fig. 3. Table 10-1c (the selection for the design example is shown in the red boxes).



Note 1: Use Table 10-1c (Bottom Edge Hole) value under heavy line
Note 2: Use Table 10-1c (Bottom Edge Hole) value given for l_{eh} and $l_{ev,b}$

Fig. 4. Limit states checked in Table 10-1c.

Table 10-1c: Coped Beam Web Available Shear Strength

Table 10-1c (Figure 3) is also a new addition to the *Manual* and is useful for determining the shear rupture and block shear rupture strengths of the supported beam web. Table 10-1c is only needed when the supported beam is coped at the top flange, or coped at the bottom and top flanges, because the limit states of shear rupture and block shear rupture of the beam web are not applicable when the beam is uncoped or coped only at the bottom flange.

As shown in Figure 4, shear rupture and block shear rupture share a similar shear failure path through the bolt group down to the bottom bolt. From the bottom bolt, block shear rupture has a tension rupture component perpendicular to the direction of force, whereas for shear rupture, the failure plane continues from the bottom bolt to the edge of the material parallel to the direction of the force. Table 10-1c is used to determine whether the block shear rupture or shear rupture component will control for a given set of bolt edge distances.

To determine the available strength of the beam web, find the appropriate values selected for the top edge hole and center hole in the top and middle portions of Table 10-1c. For the bottom edge hole, values are selected differently for a beam web only coped at the top flange and for a beam web coped at the top and bottom flanges.

In the bottom portion of Table 10-1c, a heavy line marks where block shear rupture controls over shear rupture. Where the beam web is only coped at the top flange, the bottom edge hole strength is taken as value under the heavy line for the given bolt diameter and hole type. Where the beam web is coped at the top and bottom flanges, the bottom edge hole is taken from the table using the l_{eh} and $l_{ev,b}$ dimensions for the connection geometry.

Three values from the Table 10-1c are added to determine the available beam web shear strength: top edge hole strength, bottom edge hole strength, and the center hole strength multiplied by the number of spaces between bolts. The values in Table 10-1c are reported in kips per inch of thickness, which means the tabulated values must be multiplied by the beam web thickness.

Design Example

Using *Manual* Table 10-1, determine the available shear strength of the all-bolted double-angle shear connection for the beam-to-girder connection shown in Figure 5. The beam and girder are ASTM A992/A992M material. The connection is designed for LRFD.

The available angle strength is obtained from Table 10-1a (see Figure 1):

ϕR_n = 117 kips

The available strength of the bolt group at the supported beam web is determined with two parts: Figure 6(a) shows the direction of force for each bolt at the supported beam web. Available strengths are taken from Table 10-1b (see Figure 2).

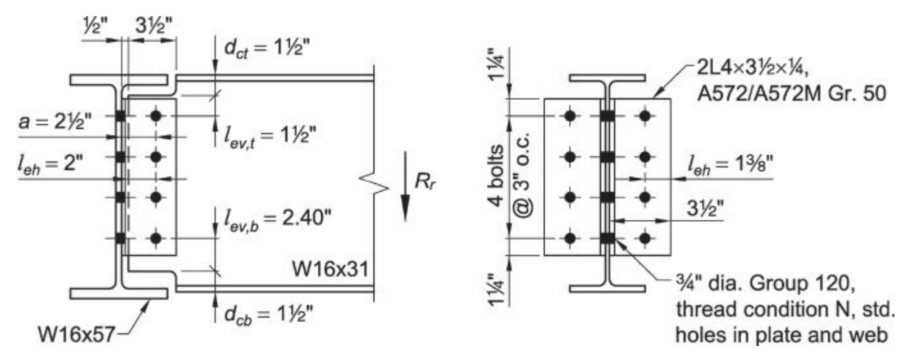


Fig. 5. Connection for Design Example.

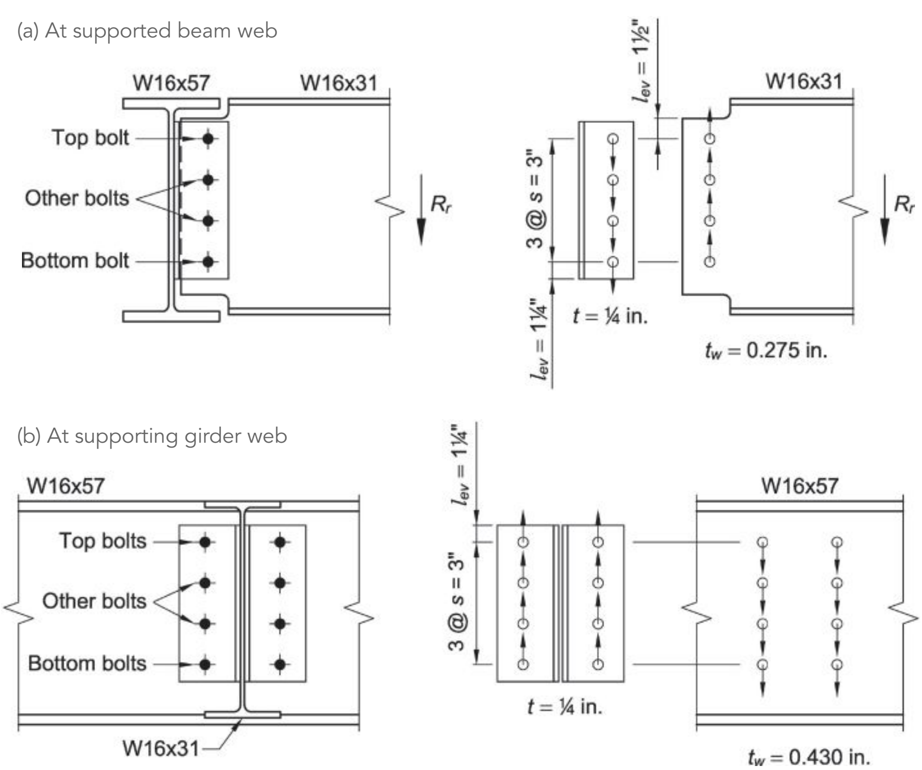


Fig. 6. Tearout pattern for bolt group.

For the top bolt:

Bolt shear:

ϕr_n = (17.9 kips)(2 shear planes)
= 35.8 kips

Bearing and tearout of angles (s = 3 in.):

ϕr_n = (87.8 kips/in.)(¼ in.)(2 angles)
= 43.9 kips

Bearing and tearout of beam web (l_{ev} = 1½ in.):

ϕr_n = (64.0 kips/in.)(0.275 in.)
= 17.6 kips **controls**

For the bottom bolt:

Bolt shear:

ϕr_n = (17.9 kips)(2 shear planes)
= 35.8 kips

Bearing and tearout of angles (l_{ev} = 1¼ in.):

ϕr_n = (49.4 kips/in.)(¼ in.)(2 angles)
= 24.7 kips

Bearing and tearout of beam web (s = 3 in.):

ϕr_n = (87.8 kips/in.)(0.275 in.)
= 24.1 kips **controls**

For the center rows of bolts:

Bolt shear:

ϕr_n = (17.9 kips)(2 shear planes)
= 35.8 kips

Bearing and tearout of angles (s = 3 in.):

ϕr_n = (87.8 kips/in.)(¼ in.)(2 angles)
= 43.9 kips

Bearing and tearout of beam web (s = 3 in.):

ϕr_n = (87.8 kips/in.)(0.275 in.)
= 24.1 kips **controls**

The available shear transfer strength at the supported beam web is:

$$\begin{aligned}\phi R_n &= \phi r_{n,top} + \phi r_{n,bot} + (n - 2)\phi r_{n,other} \\ &= 17.6 \text{ kips} + 24.1 \text{ kips} + (4 - 2)(24.1 \text{ kips}) \\ &= 89.9 \text{ kips}\end{aligned}$$

Figure 6(b) shows the direction of force at each bolt hole in the supporting girder web. Calculations for this bolt group are not shown because the procedure is similar to that shown above. The available shear transfer strength at the girder web is:

ϕR_n = 132 kips

Because the beam is coped at the top and bottom flanges, Table 10-1c is used to determine the available shear strength of the beam web (see Figure 3). If the supported beam is not coped, the use of Table 10-1c is not necessary.

From the top edge hole portion of Table 10-1c, with ¾-in.-diameter bolts, $l_{ev,t}$ = 1½ in., and STD holes:

$\phi r_{n,top}$ = 31.1 kip/in.

From the center hole portion of Table 10-1c, with ¾-in.-diameter bolts, s = 3 in., and STD holes:

$$\phi r_{n,center} = 62.2 \text{ kip/in.}$$

From the bottom hole portion of Table 10-1c, with ¾-in.-diameter bolts, l_{eh} = 1¾ in. (accounting for a possible ¼ in. beam underrun), $l_{ev,b}$ = 2 in. (conservatively used because the actual edge distance is 2.40 in. < 2½ in.), and STD holes:

$$\phi r_{n,bot} = 38.4 \text{ kip/in.}$$

The available shear strength of the coped beam web is:

$$\begin{aligned}\phi R_n &= t_w[\phi r_{n,top} + \phi r_{n,bot} + (n-1)\phi r_{n,other}] \\ &= (0.275 \text{ in.})[31.1 \text{ kips/in.} + 38.4 \text{ kip/in.} + (4-1)(62.2 \text{ kips})] \\ &= 70.4 \text{ kips}\end{aligned}$$

The available flexural strength of the cope must also be checked using the coped at both flange procedure from *Manual* Part 9. Calculations are not shown for this procedure, but the available flexural strength of the beam web is:

$$\phi R_n = 129 \text{ kips}$$

The available connection strength is then:

$$\phi R_n = \min \begin{cases} 117 \text{ kips} & \text{(angle strength from Table 10-1a),} \\ 89.9 \text{ kips} & \text{(shear transfer strength at bolt holes in supported beam web, from Table 10-1b),} \\ 132 \text{ kips} & \text{(shear transfer strength at bolt holes in supporting girder web, from Table 10-1b),} \\ 70.4 \text{ kips} & \text{(coped beam web strength from Table 10-1c),} \\ 129 \text{ kips} & \text{(cope flexural strength, see } \textit{Manual} \text{ Part 9)\}} \end{cases}$$

$$= 70.4 \text{ kips}$$

The beam web available shear transfer strength controls the connection design. ■

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Connecting the Dots

INTERVIEW BY GEOFF WEISENBERGER

John Schuepbach links people from all corners of the North American structural steel industry.

JOHN SCHUEPBACH'S role in the steel industry is best described in romantic terms.

He is, in essence, a steel matchmaker—with a Rolodex that spans coast to coast and from Canada to Latin America.

Schuepbach has carved out a unique niche in the industry. Since 2010, he has been matching equipment buyers with sellers, setting up mergers and acquisitions, and helping fabricators recruit employees as the managing director for his company, Phoenix Solutions Group International.

Schuepbach grew awareness about his services with email blasts that featured fabricators who were interested in buying and selling equipment. Those blasts have morphed into a newsletter with industry information pertinent to fabricator owners and executives who want his thoughts on how to improve their business and equipment trends in fabrication shops. He aims to provide a business improvement and trends perspective.

Schuepbach is on the planning committee for NASCC: The Steel Conference and has attended the conference for more than two decades. This year, it returns to his hometown of San Antonio, Texas, and he is shepherding several sessions by speakers he recruited.

Schuepbach spoke with *Modern Steel Construction* about his journey in the steel industry, which started before he even had a driver's license.

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

What was your path to your role in the steel industry?

I grew up in the family steel fab business in San Antonio, which my dad bought in 1989 when I was in high school. I started working there in 1989 picking up scrap and driving the forklift. I was 15 years old. It was called Trans-Tex Fabricating Co. He sold it in 2009. We acquired Bexar Metals in 1992 when I was still in high school. Then, we acquired a friendly crosstown competitor, Jackson Steel, in 2004. We grew that, I took over, and I sold it in 2009 to a company in Fort Worth, Texas.

Ever since then, I've been doing consulting, mergers, and acquisitions. The business has changed. When I sold Jackson Steel, I didn't have anybody or know anybody who could advise me on specifics of how to do a deal in our industry. I was looking for somebody who could support me through the sale process. That was the genesis of realizing that other people could use some guidance. I've been blessed to be able to make this work and I really love what I do with people.

A big part of what you do is buy and sell deals with equipment, right?

I broker deals with equipment. One of the first deals I did was with Able Steel Fabricators in Mesa, Ariz. They called me and said they had some equipment they needed to sell because they were putting new stuff in. I put it on an email blast, and a guy in Guatemala I met at The Steel Conference, Erick Luna, called me and said he wanted to look at it.

I met him in Phoenix. We went to Able and did a deal for that equipment. Then I took him around Phoenix for the next couple of days and toured other shops. I told them, 'I'm John Schuepbach, I wanted to meet you, and I'm here in town doing an equipment deal.' We met everyone in Phoenix.

Then when I was going somewhere to do that type of work, I would call other fabricators, tell them I'd be in town and ask to introduce myself. That's how I got to know people and developed a network. That way, when people have a question or need help, they call me.

Is it an as-needed basis where people say they're looking for some type of beam cutter, for example?

Yes, and people upgrading on a certain piece of equipment. I say I'm blessed because people keep calling me back. Able was one of my first clients, and as they've upgraded and changed equipment, they keep calling me back to ask if I can sell a piece because they're bringing in a new piece. I have several clients where I sold their equipment as they upgraded. I'm happy to work with them.

What else do you do besides broker equipment deals?

I also do recruiting. I do mergers and acquisitions, buying and selling with fabricators and erectors. I did one deal with a heavy crane rigging company. But most of these mergers and acquisitions are in the steel construction space.

You're the chair of the NASCC Safety Committee too. What do you do there?

I've also been on The Steel Conference planning committee for four or five years, and when I started, AISC senior vice president Scott Melnick told me that they talk about safety, but they don't have a good safety program at the conference. It always kind of fell flat.

I'm not a safety professional. But what I do have is a network. I reached out to the network, and it led me to phenomenal safety people I brought together to develop a program. I'm a facilitator, taking their ideas and putting them together in a program.

The goal with safety is to develop a library of videos that we can distribute industry-wide because many small fabricators and erectors can't attend the conference. They have to stay and work. But we still have to get the message out. We had 11 fabulous sessions in 2023, and 13 are scheduled for 2024. We're developing them into a TED talk-style presentation and video that we can distribute and use for the future.

I'm lucky to be able to tweak and facilitate it. I don't want to claim the title of leader. But I'm shepherding it. I get to tap the network I built and ask about important issues we're struggling with, how they can be addressed, and what topics and speakers do we need?

With your network, are you involved in any way with workforce development for the industry?

That's something I recognized but don't want to claim ownership of either. But I'm working with fabricator owners and executives every day. It's constant. I hear how they're looking at equipment because they can't find welders. I've reached out to the network to ask specifically about workforce development. What programs do you have? How are you doing it?

I talked to Mark Fultz at Able, my oldest client, and his colleague Kenny Hicks created their workforce development program. I was blown away and thought we needed him to speak at The Steel Conference in 2022 in Denver. That was really the catalyst.

At Kenny's session, AISC board of directors vice chair and STS Steel president Glenn Tabolt heard him speak about the program. He learned about the program Kenny developed and wanted to take this and develop it with AISC into a more robust program, just like we did with safety, and share that with the members.

Like safety, I was in the right place at the right time and connected the dots. It's because of the network and being able to reach out to people.

Now, AISC director of workforce development Jennie Traut-Todaro has that job. I see other companies post their workforce development stuff and I keep sending it to her. I asked her to tell me if I'm spamming her, because I want her to see all this.

I try to help her pull it together with all these different perspectives—see what works and what doesn't. Let's get this going, because just about every client I talked to at The Steel Conference in 2023 spoke about the need for workforce development and their initiatives. It will be a monster program if AISC can pull this together and distribute what we've aggregated, lessons learned, and what will help you develop a program. We need safety, and we need workforce development across the industry.

Are you on the road all the time?

The short answer is yes. It's hard to do what I do for clients on the phone or even using video conferencing. I try to limit the nights away. If I can get somewhere and get back by just spending one night, that's ideal. If I leave early, stay the night, and then get two full days with somebody, that typically covers what I need to do for that client. But I don't have as many direct flights out of San Antonio, so when I go farther away, it's harder to do one night.

But I get to do many cool things like helping people look at production and their shop for inefficiencies. It's not that I'm a genius. But I've been through 200 fab shops in Canada, the U.S., Mexico, and Guatemala with an eye for operations, production, productivity, and best practices. I don't come up with any new ideas. I see what somebody is trying to do, and I've probably seen another way that might be better and that they could implement.

Sometimes, I'll call another fabricator and tell them what one person is having trouble with, ask if they'd mind talking with that person, and put them in touch with each other. They're not in the same market. All I'm doing is connecting the dots. If someone comes to me with a problem, I probably know someone with a solution. I tell them about it and see if the other fabricator will talk to them. They get a solution and a relationship with another fabricator.

It really is a unique setup that developed organically.

It has all been derived organically by someone needing something else. I have a fairly deep background in accounting and finance. Many fabricators know what they're trying to do and the equipment they're looking to buy for that.

I can work backward to the return on investment for that piece of equipment based on what I see and the variables they tell me. It's sometimes way off from what they were expecting, and if they can't meet the ROI, they need to look for a different solution that's less expensive. It's really cool when the operations and finance come together, and you see the light bulbs go off. ■

This article was excerpted from my interview with John. To hear more from him, find the February 2024 Field Notes podcast at ***modernsteel.com/podcasts****.*

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

The Rise of Robotics

BY ADAM MacDONALD

Robotic machinery is an opportunity for fabricators, not a danger, and picking the right technology is crucial to successful implementation.

AGT

STEEL FABRICATORS NATIONWIDE have encountered hiring challenges with no easy fix.

A labor shortage has impacted the entire trade industry, and a perception of strenuous working conditions among new steel fabrication industry entrants compounds it. High-mix, low-volume manufacturing—especially prevalent in structural steel fabrication—requires well-trained workers. Leaders at fabrication shops must navigate these hurdles while staying on course to meet their objective: continuous-flow manufacturing without activities that hinder efficiency and eat into profits.

They have found help in robotics.

Fabricators are increasingly implementing collaborative robots (cobots) and integrated robotic systems, some of which are tailored for welding and fitting applications. The price range is wide—some cost around $100,000, while others clear $5 million. The technology in robotics is not entirely new, but its integration into an industry that traditionally relied on skilled labor represents a significant shift. And navigating it can be challenging for business owners.

The same key questions to ask when purchasing any fabrication equipment also apply to robotics. Is the software user-friendly, and can the existing data be leveraged? Can the system be operated by entry-level personnel, or does it require an expert? What maintenance and support will it need, and what does that cost? What level of throughput should be expected? How much labor can it replace, if any? Lastly, assessing the potential for obsolescence is essential in a rapidly evolving technological landscape.

The harder and lengthier part comes after these questions are answered: getting buy-in from the entire organization, which must accept and prepare for the transition.

Cost
- ❑ Is the cost of the system within our budget?
- ❑ Can we afford not to implement this system given the current challenges?

Data Integration
- ❑ Can we leverage existing model data for the new system?
- ❑ How easy is it to transfer data from model to the shop floor?

Ease of Operation
- ❑ Can the system be operated by an entry-level person with minimal training?
- ❑ Does it require the expertise of a specialist to run effectively?

Throughput
- ❑ What level of throughput can we realistically expect from this system?
- ❑ How will it impact our production efficiency and capacity?

Longevity and Upgradability
- ❑ Is the system likely to become outdated quickly, or is it designed for long-term use?
- ❑ Are there upgrade options available to keep the system current?

Maintenance and Support
- ❑ Do your due diligence and reach out to other owners and ask about maintenance.
- ❑ Ask for an example of how the support is given.

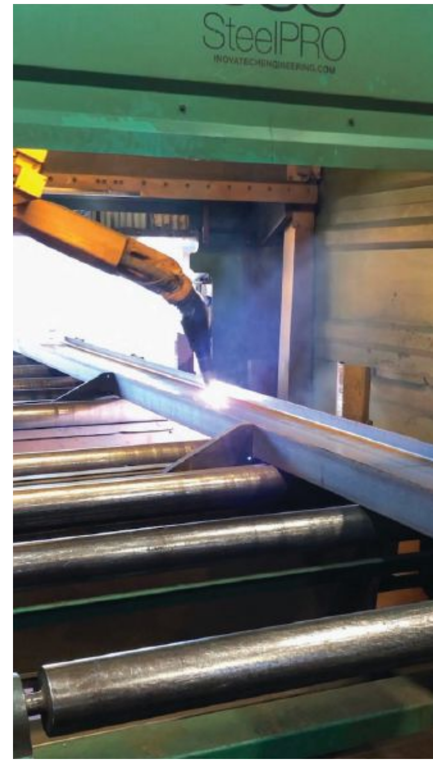
Transition Preparation
- ❑ Communicate and explain the purchase to all relevant stakeholders in the organization.
- ❑ Ensure that employees are aware of the upcoming transition and its significance.

Cultural Integration
- ❑ Foster a culture of acceptance and enthusiasm for the new system among employees.
- ❑ Encourage buy-in and participation from all levels of the organization.

Connection Cost Evaluation
- ❑ Investigate and evaluate the costs associated with your current connection design.
- ❑ Identify areas with cost-saving opportunities or where adjustments are necessary.

Design for Robotic Limitations
- ❑ Work with your engineering and design teams to develop or adapt connections that align with the limitations of the robotic system.
- ❑ Ensure that designs optimize the system's capabilities and efficiency.

Production Planning
- ❑ Develop production plans that leverage the capabilities of the investment, aiming for increased efficiency and throughput.
- ❑ Begin integrating the new system into your production planning processes.



Everyone—from the engineering department to detailing and from production control to the workshop—plays a crucial role in cultivating acceptance and, ideally, excitement.

Start with engineering, where you must reevaluate your connection designs and adapt them to align with the chosen robotic system. These modifications may deviate from the traditional methods designed around drill lines. Due to the lack of skilled welders, some shops have resorted to shop-bolted connections without scrutinizing the costs or because they had no other choice. The proper response is to design connections that accommodate robotic limitations. Yes, there are limitations. Robots may struggle with tight spaces, but future advancements in vision systems and arrays of torch bodies are expected to mitigate these constraints.

Production planning is also an important consideration. The production planning tool market is sparse, meaning proper planning can be daunting. But it's crucial. Proper planning can lead to substantial increases in throughput while reducing reliance on skilled labor and valueless activities. When combined with the adjustments made in the connection design, the dividends will be immediate. And they will lead to the intangibles: 85% arc on time,

increased quality, increased capacity on other machines, a safer and cleaner environment, the ability to gain control over production planning, and so much more.

Good luck with your endeavor and once you start your robotics journey, there is no looking back! ■

Visit aisc.org/2024advance for a full list of fabrication equipment exhibitors at 2024 NASCC: The Steel Conference.

Adam MacDonald (adam.macdonald@agt-group.com) is the East Coast territory sales manager for AGT Robotics.

Second Lives

BY BRIAN MCSWEENEY, SE, PE, AND JAMESE SPEARS, PE

Adaptive reuse is a powerful tool for reducing embodied carbon in construction.

THE IDEA OF SUSTAINABLE DESIGN can seem intimidating to a structural engineer when thinking beyond recycled materials and local sourcing. Programs like LEED have amplified those two initiatives and created a blueprint for executing them. Those are, though, just the beginning of today's sustainability push and designing a building that aims to reduce carbon dioxide emissions from construction and, eventually, demolition (also called embodied carbon, or EC).

But successfully achieving a low embodied carbon output is not overly burdensome. And one of the most powerful opportunities for embodied carbon reduction is through adaptive reuse of buildings.

Adaptive reuse aligns with the Structural Engineering Institute's (SEI) SE 2050 Commitment, which outlines ways embodied carbon reduction can be approached when designing a building. Its goal is to encourage engineering and architecture firms to design projects that will help achieve net zero embodied carbon structural systems by 2050.

Designers usually prefer new build construction over renovation projects, especially when the existing building is an antiquated or historic structure rife with design hurdles. New framing required even in a renovation, though, can deter EC reduction goals. Despite their challenges, reuse projects offer sustainability rewards that new construction cannot.

How, then, can adaptive reuse project engineers ensure success to the project and design team while maintaining the highest impact on embodied carbon reduction? There are three important considerations:

- **New Function vs. Old Function:** Building selection is crucial and comes before an engineer is even chosen. The selected building should require minimal change to use. The IEBC allows up to a 5% increase in gravity demand-to-capacity ratio without further structural evaluation and strengthening. Significant structural modifications may be necessary when the loads increase beyond the limit. Changes to live or dead loads (such as new finishes or increased fire-rating) can trigger this code threshold and increase design efforts, construction costs, and even affect the project's viability. Lateral demand capacity is permitted to increase by 10%, but even adding new rooftop units can exceed this limit. Know the limits of the IEBC (Section 806) and stay within the thresholds.
- **Good Bones:** This one seems obvious, but many times, the structural condition is overlooked in the project planning stages, and costs associated with structural repair can be substantial. The project should include a condition assessment phase before purchasing the facility to understand these risks fully.
- **The Toolbox:** Tap into the many available tools for the highest level of data collection and creative collaboration, both crucial to success.

Adaptive reuse projects can be challenging, especially when there are missing pieces of information, such as a lack of existing building drawings. There are three key pieces in the Toolbox:

- **The Team:** Consider anyone from the existing building owner, occupant, and maintenance staff to the authority having jurisdiction (AHJ) as important resources to the team. With engineer guidance, new building owners can help the project by making savvy building selections. A call with the AHJ to discuss the team's interpretation of IEBC and specifics of the building can be invaluable because it offers early design feedback that should help avoid tough permit comments later. Anyone with knowledge of the existing building can give insight into its previous use, undocumented renovations, or current concerns.
- **Site Access:** The building itself may hold important clues. Include a feasibility study in or alongside the conditions assessment and plan for an early site visit. This study should identify concerns with the adaptive reuse program regarding the structural capacity of the proposed building. If you're told existing drawings are unavailable, ask for them anyway while on site. It's surprising how often they are unearthed in a back room because no one previously knew what to look for or didn't have the time. Keep an eye out for old building pictures on site and photograph them for future reference. They might be the key to identifying the building's prior use. The IEBC allows some increase in loads based on any previous use. Be sure to state in the assessment proposal that selective demo, whether now or later, may be required by a contractor.
- **Historic Data:** Consider local construction practices of the era to help fill in gaps. Search the web for information on the building and similar antiquated structural systems. Engineers often share resources online, and university libraries have extensive collections of early design codes and material references. Also, consider taking a coupon of existing material when testing is necessary to complete designs. Always list assumptions you have made and have the contractor verify during the construction or selective demo phase.

The Kinley Hotel, an adaptive reuse project in Cincinnati.

Starboard and Port Photography

Virtue Feed and Grain, an adaptive reuse project in Alexandria, Va., that turned a feed and grain warehouse into a restaurant.

A column in the Jewler's Exchange Building.

There are many examples of projects that followed these guidelines. One recent reuse project was not designed with EC reduction goals in mind, but it showcases the power renovations have in sustainability.

The Jeweler's Exchange Building in downtown Cincinnati, Ohio, was constructed in 1915. It consists of nine-story structural steel moment frames over a one-story basement, plus an elevator penthouse. Terra-cotta pan joist framing spans between the structural steel members, and the steel is encased in concrete. The existing building was originally designed as an office and retail space, but the project sought to adapt it into a hotel.

An original connection in the Jewler's Exchange Building

Investigative field visits and research on the governing local building code of the era indicated that the building was well-suited for adaptation to a hotel. Floor capacities were required by code to meet 100 pounds per sq. ft. live load. The regular bay spacing readily accommodated the intended floor plans. Creative support solutions permitted the cutting of new elevator and mechanical shafts at the core of the building and the infill of abandoned shaft openings. Creative coordination between the structural and mechanical engineering teams kept loads within the IEBC's gravity and lateral thresholds laid out in Section 503.3-503.4 for the addition of new rooftop equipment.

The original structural steel frame facilitated relatively simple strengthening, repairs, and modifications to the existing structure despite the challenges of re-supporting the terra-cotta pan joist framing at new openings. Coupon testing of the existing steel confirmed some early assumptions during design, and the results allowed the construction crews to weld new elements to the existing building frame.

The Jewler's Exchange Building floor.

Brian McSweeney

The Kinley Hotel's lobby.

below: A room at the Kinley Hotel.

Starboard and Port Photography

Starboard and Port Photography

New structural steel was utilized for strengthening and repair work where necessary. At floor infill locations, structural steel supported concrete on metal deck. The lightweight nature of the steel retrofit generally stayed within the existing framing capacities.

After completion, we selected the building for retroactive study of sustainable design decisions. One key question in the study: if the ownership and architectural team had asked us early on to offer input on the sustainability impacts of a reuse or teardown, could we quickly respond from the structural perspective?

That answer was a clear yes. SEI provides an estimation tool on the SE2050 website called ECOM (Embodied Carbon Order of Magnitude). We compiled some conservative design numbers for a modern-day steel-framed equivalent building and arrived at rough reinforced concrete and steel quantities for the structural frame and foundations. The next step was to calculate the quantities of the structural materials used in the adaptive reuse of the building. We then entered material types and quantities into ECOM for each scenario.

We expected the adaptive reuse numbers to show a noticeable reduction in embodied carbon numbers for the structure. If the entire team used ECOM to conduct a whole-building estimate rather than a structure-only estimate, the reduction would be different. But we still anticipate an overall drop in embodied carbon compared to a new build.

We did not, though, expect a nearly 90% reduction in embodied carbon (emissions) achieved by reusing the existing structure. This building was well-suited to its new use in form and function and was generally in good structural condition. Those were two crucial pieces in the magnitude of the reduction.

All told, the Kinley Hotel project highlights the substantial impact that savvy building selection and a collaborative design team can make if an existing building is suitable for adaptive reuse. It also shows the ease with which engineers can make an early comparison to demonstrate the potential embodied carbon reductions for a particular project while the owner and developer choose a site. ■

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Tight urban spaces and strict displacement tolerances were overcome in designing and erecting a flyover at a critical CTA interchange.

Flying By

BY VINOD PATEL, SE, PE AND HEMAL PATEL, SE, PE

Dave Burke Photography

THE CHICAGO TRANSIT AUTHORITY (CTA) left no area uncovered in its plan to update its main North-South artery.

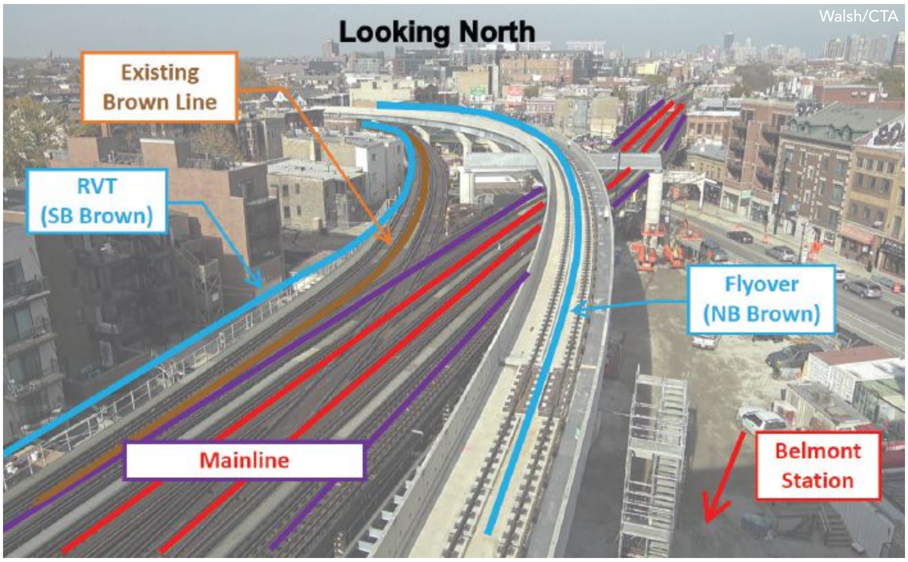
The Red and Purple Modernization (RPM) Project is a $2.1 billion project, the city's largest capital project to date. It involves completely reconstructing two miles of elevated track on Chicago's north side, most of which was originally built more than a century ago. The Red Line, going north-south through the city, is the CTA's busiest route and carries approximately 20% of all daily CTA riders. The Purple Line shares the same route but runs express during rush hour periods.

The RPM is being delivered as a design-build project and is slated for completion in 2025. One recently completed piece of it, the Red-Purple Bypass (RPB), has helped reduce congestion near the bustling Belmont station at a junction where a third route, the Brown Line, joins the Red and Purple lines. The three lines share the same set of tracks heading into the Belmont station before the Brown Line diverges to head west.

The old junction had four mainline tracks 20 ft above street level and was supported by a 120-year-old riveted steel structure. The northbound Brown Line traveled on the easternmost track and diverged by crossing three mainline tracks at the same level. It caused a major bottleneck, sparked frequent delays, and limited the overall capacity of the Red and Purple lines.

The RPB involved a total reconstruction of the old junction's track structure starting just north of the Belmont station and spanning three city blocks. Its signature component, though, is a new single flyover track that carries the northbound Brown Line over the four mainline tracks and eliminates the bottleneck. Designing the flyover demanded stringent displacement tolerances, and it had to be erected in a tight space that included alleys with multi-story buildings on both sides. In addition to the flyover, a temporary elevated track structure called the RVT was built to carry southbound Brown Line trains through the junction for construction staging.

The new permanent flyover and RVT provide two extra tracks, which allows for the demolition and reconstruction of the four-track mainline structure in two stages. The RPB will increase the Red and Purple lines' speed and capacity, including the ability to add more trains during rush hour periods and accommodate 7,000 additional daily commuters.



above: A northbound look at the flyover and mainline tracks.

below: An overhead view of the flyover, mainline tracks and the RVT.

Flyover Structure

The flyover bridge is a closed deck structure consisting of four parallel steel plate girders that are composite with a 10-in.-thick cast-in-place concrete deck varying in width from 14 ft, 6 in. to 16 ft. The top of the rail is approximately 46 ft above ground at the flyover's highest point, where it is supported by a straddle bent.

The total flyover length is approximately 1,800 ft, with spans varying from 45 ft to 130 ft and a radius varying from 750 ft to as tight as 425 ft to allow the bridge to snake through the Lakeview neighborhood's dense urban environment. The structure consists of two-span continuous units each less than 200 ft long, except for a signature 420 ft long, four-span continuous unit that's centered on the straddle bent. All structural steel was galvanized to ensure 100-year service life.

The flyover's straddle beam over the mainline tracks.

EXP

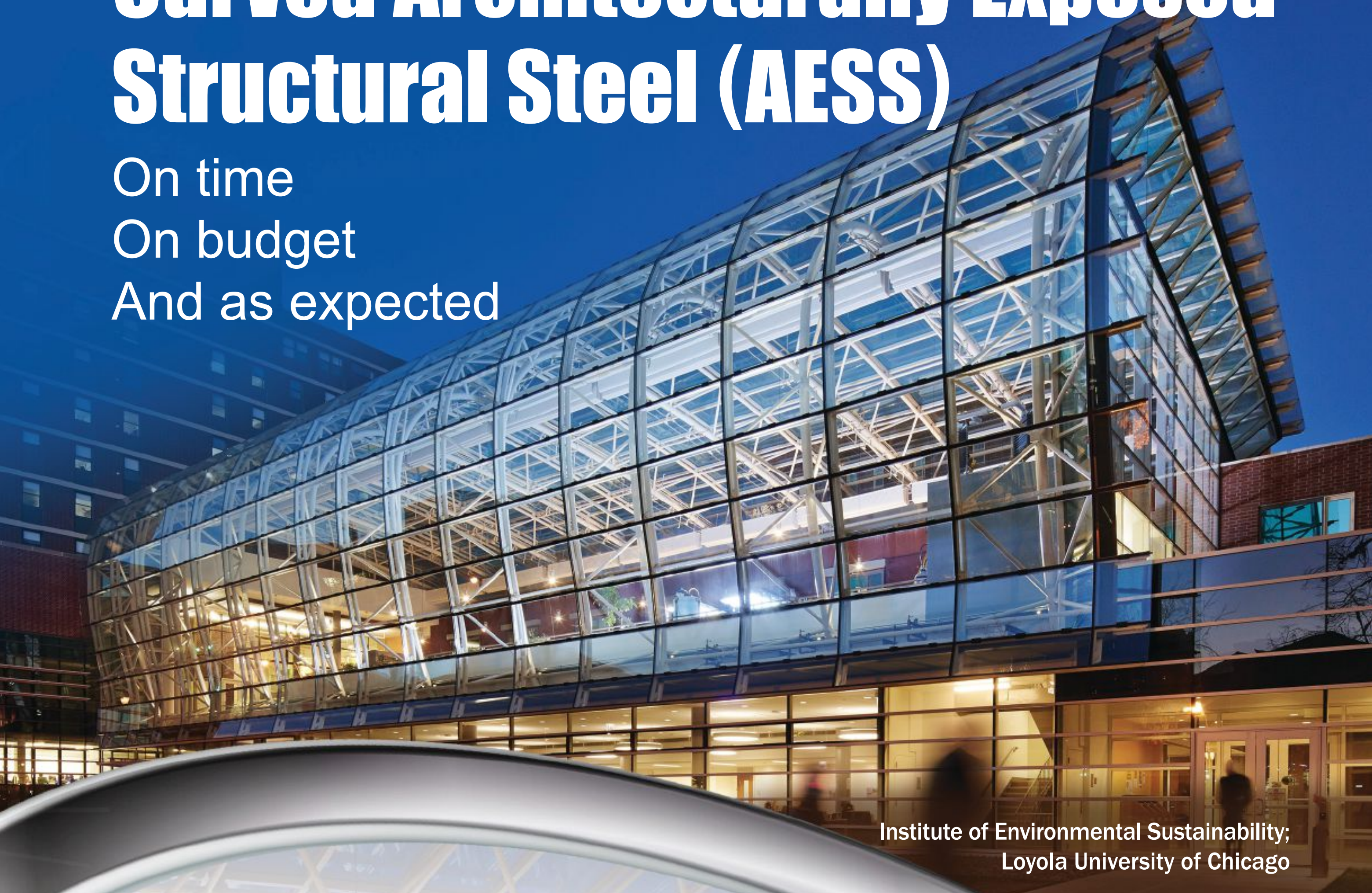
Cross frames spaced at approximately 12 ft along the entire length create a unified system where all four girders work as a single unit to counteract the torsional effects of the tight curvature. Lateral bracing between the interior girders was provided to limit the displacement of the girders during erection, which was accomplished without any temporary supports or shoring.

A combination of AREMA and AASHTO codes allowed for a comprehensive and effective design approach of a curved rail bridge with continuous structural units. AASHTO was used for guidance on curved girder analysis, dead load fit, and cross-frame designs. Meanwhile, AREMA provided allowable stress limits for girder plate sizing and governed the design of intermediate web stiffeners, bolted girder splices, and slip-critical bracing connections.

CTA trains are sensitive to minuscule magnitudes of structural movement and vibration, so an integral part of the design was ensuring adequate structural stiffness and frequency. That was especially important for the longer spans because it governed the girder sizing over strength requirements. The structure's stiffness also plays a vital role in limiting the stress in the rails, which was crucial at structural expansion joints, where differential displacements and rotations tend to cause localized tension and bending in the rails.

The design team performed a rail-structure interaction (RSI) analysis to quantify the thermal movement of the structure relative to the rails and resulting force transfer between the two. Nonlinear springs were used to represent the rail fastener clips, which clamp the rails to the flyover deck. The analysis determined rail stress at expansion joints, longitudinal shear in the piers, and any force transfer at the flyover ends into the existing CTA structure.

Originally, the project's technical requirements preferred continuous welded rail (CWR) for improved ride quality. However, CWR on the tightly curved alignment resulted in large transverse thermal forces and movements that could cause issues at the ends of the flyover, where it transitioned back into the existing



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CTA tracks. The design team and CTA ultimately decided that jointed rail with 80-ft rail segments would provide the best balance of serviceability and ride quality.

The rails were hung at their final elevations above the bridge deck, and plinth concrete was poured to the bottom of the fastener base plates. As a controlled secondary pour on top of the 10-in. deck, the plinths ensured that strict $\frac{1}{16}$-in. tolerances were met for the track elevation. They also raised the rails off the concrete deck to protect them from rainwater. The 10-in. structural deck followed the vertical profile of the low rail, while the plinth thickness varied from 4 to 6 in. to accommodate the track's changing radius and superelevation.

The closed concrete deck contributed to noise reduction compared to the original open deck configuration that had timber ties. Additionally, 4-ft-tall precast concrete walls running along the deck edges helped to contain the noise. Their outside surface incorporated form liner relief, enhancing the overall aesthetic appeal.

Flyover piers are single-column hammerhead type with a specialized architectural form liner relief. Each pier is supported by a single concrete caisson belled in hardpan about 90 ft below grade, minimizing the foundation footprint within the limited right-of-way. However, given the relative flexibility of the free cantilever system, plus the weak nature of fill and silty clay along Chicago's lakefront, extra care was needed to size the caissons to limit deflections at the track level.

Straddle Bent

One of the flyover's significant features is the "straddle bent" that supports it at the highest point where it crosses the mainline tracks. The straddle bent is supported on two columns and minimizes the foundation footprint needed in a dense area.

The cross beam of the straddle bent is a simply supported steel box girder that is 4 ft wide, 6 ft deep, and 80 ft long from center to center of bearings. The bearings are HLMR urethane discs, which allow rotation about each axis. Each includes a high-strength steel shear pin that transfers horizontal forces into the concrete columns and caissons.

The beam is a fracture-critical member and was one of the most scrutinized design elements of the project. It includes two bottom tension flanges for redundancy, though only one is needed for strength. The tension flanges are bolted to the web plates with L6×6 angles. The L6×6 angles

above: The flyover girders and straddle beam being erected.

right: The RVT structure looking south.

are not included in the section's flexural capacity for additional redundancy in the tensile region. Meanwhile, the top compression flange, which is not a fracture risk, is fillet welded to the web plates.

The longitudinal girders frame directly into the straddle beam's webs with bolted shear connections. A continuity tension plate is placed over the top of the box, connecting the girder top flanges on either side. Diaphragm plates with access openings are spaced 6 ft on center within the box. The interior features lighting to accommodate future maintenance and inspections.

The design process considered if the straddle beam plates, particularly the welded top flange-web assembly, could be fabricated as one continuous piece. However, at 80 ft long and 6 ft deep, the fabricator's galvanizing tub could not accommodate the entire beam, even with double dipping. Instead, a bolted splice was added near midspan and the two segments were connected on site.

The straddle beam's box components were shipped to the site and assembled on the ground adjacent to the tracks. Among them were T-shaped plates nicknamed "ears" that were bolted to the webs of the straddle box. These ears facilitated an easy connection of the straddle box to the longitudinal girders, resembling a conventional bridge field splice.

The CTA allowed a 48-hour weekend mainline closure to install the straddle beam and adjoining longitudinal girders. During it, the straddle beam and structural steel in the adjacent spans were erected. The 117-ton straddle beam was the most critical pick of the project and required a crane with a 158-ft boom length, 50-ft radius and 237-ton counterweight.

RVT Structure

The RVT structure, built to temporarily carry southbound Brown Line trains into the Belmont station, allowed the contractor to stage the demolition and reconstruction of the mainline. The RVT needed to be constructed and made operational while the existing mainline was active. Given the limited right-of-way, the main challenge was fitting the structure into a 16-ft wide alley flanked by residential and commercial buildings. It also had to clear utilities on the west edge of the alley and the mainline structure and its maintenance platforms on the east edge.

Structural steel was an ideal choice for RVT because it facilitated construction in such a tight space and provided flexibility for changing field conditions. The RVT structural system is comprised of longitudinal rolled stringers that frame into steel double-column bents supported on spread footings. The base of each column was considered pinned in both directions. Transverse sway was limited by the frame action of the double-column bents. Longitudinal sway was limited by truss bracing provided in every third span.

The design features skewed bents to avoid existing obstacles along the east and west edges of the alley. Simple bolted connections between stringers, cross bents, and bracing allowed for quick erection. In addition, steel afforded a relatively lightweight structure and made spread footing foundations feasible within the alley's limited footprint, affording the contractor savings in cost and schedule.

Various other considerations were made to address the geometric challenges. The mainline's maintenance platforms were relocated (including systems and communications equipment) while the overhanging track timber ties were trimmed. Aerial utility lines were

Dave Burke Photography

The south flyover connection at the Belmont station.

Walsh/CTA

relocated to an existing underground duct bank in the middle of the alley, and the supporting utility poles were either trimmed or removed altogether. All adjacent buildings were potholed to identify the bottom of their basement walls. The spread footings were extended to the bottom of the basement walls to avoid applying track surcharge loads to the walls.

The RVT steel bents and columns were positioned in the narrow gaps between the adjacent buildings along the west edge. The corners of several steel bents were also coped to make way for the adjacent proposed mainline beams along the east edge. The sway bracing on the west edge of the structure was placed at a higher elevation to maintain adequate clearance to the adjacent garages. Sway bracing on the east edge was placed at a lower elevation to provide clearance to the proposed mainline bents.

South and West Connections at Flyover Ends

The design-build team faced a challenge when tasked with connecting the south end of the flyover to the existing mainline structure at Belmont, one of CTA's busiest hubs. Any construction in its proposed final location would have required months of mainline track closures and triggered extensive system-wide impacts. CTA could only allow for a two-week closure window of the easternmost mainline track for making this connection.

In response, the contractor erected the new structure on falsework to the side of the existing alignment and then rolled it into place during the closure. The proposed structure was designed to match the type and span layout of the existing Belmont structure. It consists of longitudinal rolled stringers bolted to transverse welded-plate cross girders. The stringers are continuous through the cross girder with top and bottom splice plates. The floor system was also made composite with the cast-in-place concrete deck and welded shear studs.

Once the two-week closure window began, the original structure supporting the easternmost mainline track was demolished. Shortly after, the new structure was moved on Self-Propelled Modular Transporters (SPMT) into the final alignment in a mere six hours. Next, the new cross girders were connected to the existing cross girders with bolted field splices. The benefit of easy bolted connections to the existing structure saved time for installing and adjusting the track furniture and supporting equipment on top of the deck during the remainder of the closure window.

At the other end of the flyover, the west connection ties the structure into the existing Brown Line tracks and is threaded through a tight alley flanked by the existing Brown Line and residential buildings. The flyover transitions from closed deck to open deck to match the existing Brown Line track system.

The structural system consists of longitudinal rolled stringers framing into double-column steel bents. Each column is supported by a reinforced concrete footing on steel micropiles. Micropiles were chosen as the foundation type due to limited access to the alley, tight horizontal and vertical clearances, as well as the need to minimize vibrations on the adjacent buildings. Due to access requirements to adjacent properties, cross-bracing of columns was not feasible. Instead, the structure uses rigid moment frames in both directions to limit sway.

Work is ongoing around Belmont. Crews have completed two new mainline tracks and are demolishing and reconstructing the other two. All work is slated for completion in 2025. The flyover, though, has already made an impact. ■

Owner
Chicago Transit Authority

Structural Engineer
EXP (Flyover and RVT), Stantec (Mainline)

Architect
EXP

General Contractor
Walsh-Fluor Design Build Partners

Steel Team

Fabricators/Detailers
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An updated tool now makes it easier than ever for engineers to estimate the service life of zinc coatings and predict their performance.

Crafting a Coat

BY JOHN KRZYWICKI

ANCIENT SOCIETIES were the first to discover that coating metals with zinc protects them from corrosion.

Only in recent decades, though, could engineers predict just how long that protection would last, thanks to the introduction of a new assessment method. That method, the Zinc Coating Life Predictor (ZCLP), was recently refined and made available on the American Galvanizers Association's website (**zclp.galvanizeit.org**).

Originally designed as a sophisticated computational model by Gregory Zhang nearly 30 years ago, the ZCLP allows engineers, architects, and other specifiers to estimate the anticipated service life of zinc coatings based on various atmospheric exposure conditions. Factors such as humidity, temperature, sulfur dioxide concentration, and chloride deposition rate play a crucial role in determining the longevity of a zinc-coated structure. By integrating these variables, the ZCLP provides a reliable estimate of when the first coating maintenance might be due.

The ZCLP is web-based and accessible from any device with an active internet connection. It requires no downloads or installations, and thanks to its new responsive design, it works on all devices. The tool requires the user to input six yearly average atmospheric parameters: annual precipitation, sulfur dioxide deposition rate, relative humidity, airborne salinity, air temperature, and sheltering condition (open air, rain-sheltered or indoor). Thus, using the tool begins with data collection. The "Resources" tab at the top of the website contains links to suggested websites or local agencies to obtain this data.

Upon receiving data, the ZCLP processes them using a non-linear corrosion model developed by analyzing years of actual zinc corrosion rates collected over most of the 20th century. The tool does not incorporate data from accelerated corrosion testing methods, such as salt spray, that are widely used to compare corrosion performance between coatings but are known to be poor predictors of zinc's corrosion protection capabilities due to the absence of real-world wet-dry cycles.

Prediction models were created using neural network technology and statistical methods to capture the complex nature of atmospheric corrosion and the interactive variables used for the model. Neural networks excel when looking for patterns in training data sets. They can learn these patterns, and develop the ability to make forecasts and predictions.

How Zinc Protects Steel

Zinc, when applied to steel, acts as a barrier. More than a physical shield, zinc also undergoes a process of sacrificial corrosion. It willingly deteriorates so steel remains unharmed. The difference in electrochemical properties between zinc and steel makes zinc self-sacrificial. Zinc corrodes before steel when both are exposed to moisture, leaving steel protected.

Hot-dip galvanizing, where steel components are submerged in molten zinc, is the most common process of applying zinc to steel. It provides a thick, durable layer that should last many decades without maintenance.

Types of Zinc Coatings

There are several types of zinc coatings, each tailored for specific conditions. Batch hot-dip galvanizing (ASTM A123) is the process in which fabricated steel articles are immersed in a kettle filled with molten zinc. Continuous sheet galvanizing (ASTM A653), meanwhile, involves coiled steel and passes it through a molten zinc bath, providing a consistent coat ideal for products like steel sheets or wires.

Other zinc coatings include thermal spray zinc (TSZ) applied to steel like paint via a spray gun (SSPC-CS 23.00/AWS C223M/NACE No. 12), where zinc wire or powder is melted and sprayed onto the steel surface. TSZ is ideal for large pieces of steel that cannot fit in the kettle to be hot-dip galvanized or for coatings requiring a field application.

There's also electro-galvanizing (ASTM B633), a process in which a layer of zinc is applied using an electric current, and mechanical galvanizing (ASTM B695), where small parts are tumbled in a drum with zinc powder. Both methods provide a thinner, more uniform layer, ideal for specific applications such as small fasteners.

Each type has its unique lifespan, and the Zinc Coating Life Predictor can provide predictions for all.

For more on the hot-dip galvanizing process, see "Galvanizing Illustrated" in the August 2014 issue, available in the Archives section at **www.modernsteel.com**.

The ZCLP computes a corrosion rate in the chosen environment and allows the user to calculate a recommended zinc coating thickness based on the desired time to first maintenance. It can also estimate the time to first maintenance based on a given zinc coating thickness.

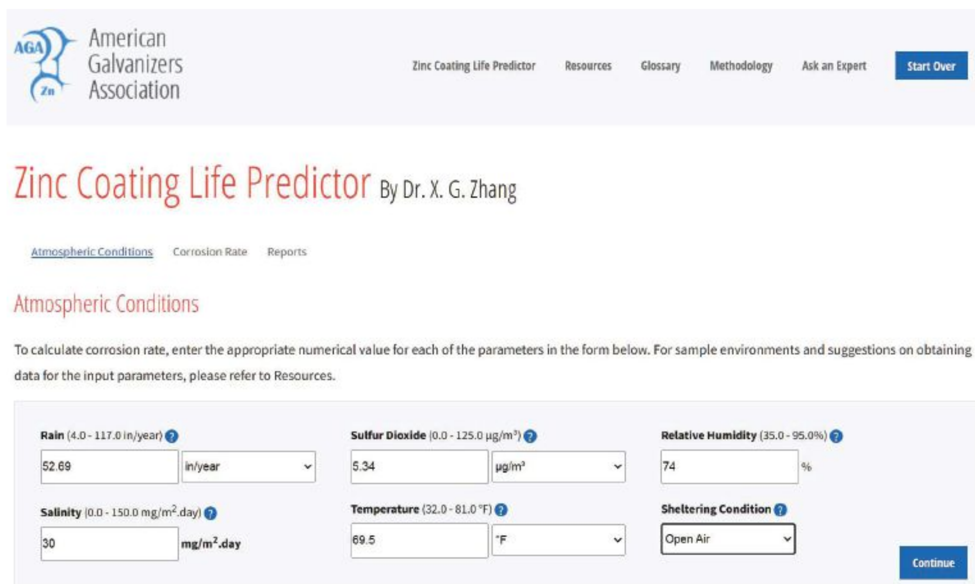
Concrete embedment, immersion, offshore, aquatic facility, chemical exposure, galvanic corrosion, treated wood, and soil embedment applications are not represented by the ZCLP. Additionally, yearly average wind direction can significantly affect corrosion rates for structures within one mile of coastlines. Evaluation of coastal applications can be limited to structures at least one mile inland or sheltered from coastal winds.

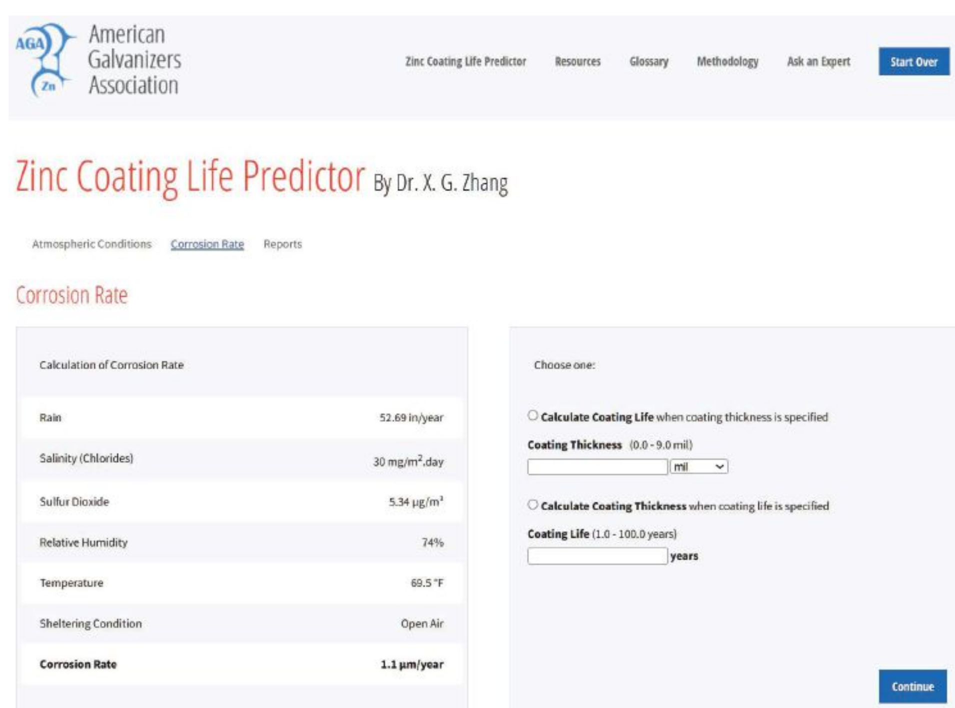
The practicality of the ZCLP becomes evident when applied to real-world scenarios. Consider an engineer tasked with designing a bridge in a coastal region. The salt-laden air, high humidity, and variable temperatures create a challenging environment for steel structures. By employing the ZCLP, the engineer can make informed decisions regarding the performance of the zinc coating, anticipate maintenance schedules, and ensure structures remain resilient through their intended service life.

In regions dense with industries, the atmosphere is often laden with pollutants, chemical emissions, and abrasive particulates that can rapidly accelerate the corrosion rate of steel. A large-scale manufacturing facility in Detroit or near an industrial area in Houston, for example, would be subject to faster corrosion rates. The ZCLP can help engineers and architects working in those areas make location-specific approximations regarding the longevity of their zinc-coated structures. They could then assess whether zinc-coated structures can retain their structural integrity, reducing maintenance costs and ensuring a lower life-cycle cost.

above: Galvanized steel ready for shipping to a jobsite.

below and opposite page: A look at input values for the Houston area in the Zinc Coating Life Predictor (below) and the resulting rates from those values (below and opposite page) assuming a 3.9 mil zinc coating thickness.





Steel being hot-dipped in a zinc coating.

Report: Coating Life

Calculation of Corrosion Rate	
Rain	52.69 in/year
Salinity (Chlorides)	30 mg/m^2.day
Sulfur Dioxide	5.34 µg/m^3
Relative Humidity	74%
Temperature	69.5 °F
Sheltering Condition	Open Air
Corrosion Rate	**1.1 µm/year**

Corrosion Rate	**0.0433 mil/y**
Coating Life	**98.7 years**

To find out the range of available galvanized coatings that will meet the requirements of your calculations please consult your local galvanizers or industrial associations.

The coasts of Oregon have milder temperatures but still pose a threat due to their saline air, and the ZCLP can be instrumental for projects like the development of ports, marinas, or highways. By offering insights into how zinc coatings will fare over time, it aids in making informed decisions on maintenance cycles, ultimately ensuring longevity and safety.

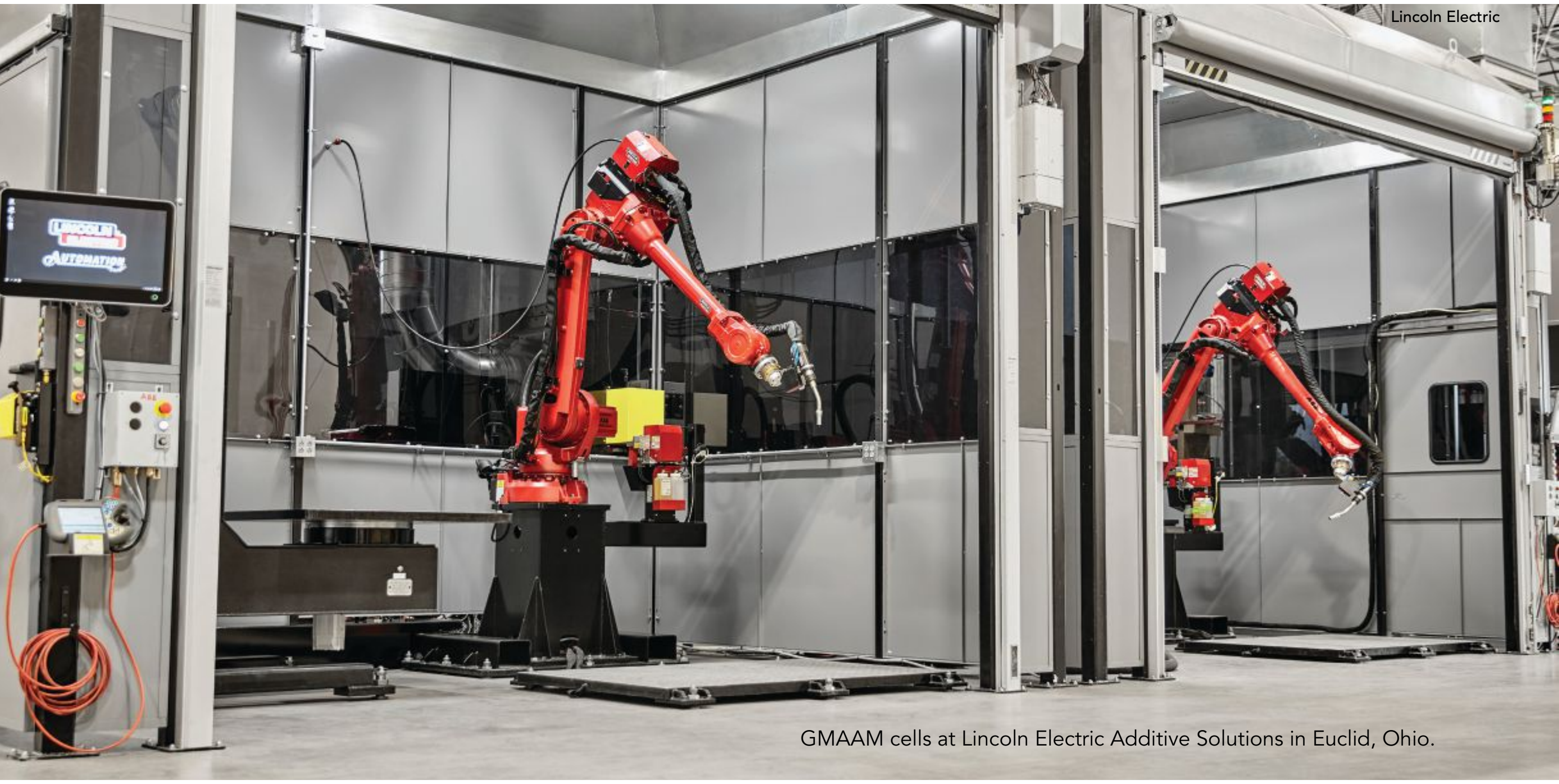
Elsewhere, suburbs usually have less human activity and fewer pollutants than urban centers, but factors like automobile emissions, diesel exhaust, and the use of fertilizers can impact steel structures more than in a rural environment. The ZCLP provides clarity on that in-between zone for anyone planning infrastructure projects like bridges or community centers in suburbs outside major cities.

Beyond immediate construction applications, the ZCLP has broader implications for sustainable urban planning and development. As cities grapple with the twin challenges of climate change and increasing urbanization, the longevity and durability of infrastructure become paramount. The ZCLP offers a pathway for cities to maximize their infrastructure investments, ensure long-term sustainability, and reduce life cycle costs. It empowers architects, engineers, and decision-makers to usher in an era where steel structures stand resilient against the test of time and environment.

John Krzywicki (**jkrzywicki@galvanizeit.org**) is the marketing director at the American Galvanizers Association.

Add to the Toolbox

BY CARLOS DE OLIVEIRA, PEng, CURT DECKER, SE, PE, PhD, AND MARK DOUGLASS, PhD



Lincoln Electric

GMAAM cells at Lincoln Electric Additive Solutions in Euclid, Ohio.

Versatility and new advancements have made additive manufacturing a real option in connection design.

AN INTRIGUING METHOD for creating architecturally exposed, geometrically complex, and heavily loaded structural steel connections has arrived. Recent developments and capabilities in gas metal arc additive manufacturing (GMAAM) have helped AM—also known as 3D printing—join casting and fabrication as a viable option for connection design.

Steel designers and practitioners are only beginning to understand AM's possibilities, but one way to shortcut commercially viable uses for AM is to consider how similar manufacturing methodologies are currently being used in steel construction. Steel castings are the steel product most similar to AM products. Before comparing the two, though, it's important to understand GMAAM.

GMAAM is categorized as a directed energy deposition (DED) additive manufacturing process. It uses a welding power source and welding wire filler material to deposit molten metal, layer by layer, onto a substrate. The process is typically shielded locally by an inert or active gas composition to prevent atmospheric contamination of the melt pool. The result is a three-dimensional part representing the desired geometry as specified in CAD.

GMAAM's fundamentals are based on the wealth of knowledge and technology surrounding traditional gas metal arc welding and robotics. However, tailored welding process procedures, feedback controls, and specialized software for automated path planning are required to enable the additive manufacturing application.

For most commercial applications, GMAAM is performed by manipulating the welding torch using an industrial articulated robotic arm. The robotic system can produce sizeable parts and is generally limited only by the robot's effective reach. Parts may easily exceed the build volume if the aspect ratio of the component is high (tall with a narrow footprint versus short with a wide footprint). Parts that exceed the build volume can be manufactured by joining multiple printed parts. They're still comprised of the same weld metal whether constructed in the printed or joined areas.

GMAAM and other DED processes are unique because they synchronize the robot and part using a rotating and tilting positioner, creating highly controlled orientation—which in turn allows for alternative build strategies and increased part complexity while minimizing support structures.

Freeform Geometry

Use of steel casting for structural steel connections took hold in the industry because it can be used to produce high quality steel in freeform geometry. With castings, part shaping can be more directly informed by the ideal load path through the junction, which results in connections that are lighter, yet stiffer and stronger, than their conventionally fabricated counterparts.

In structural steel casting design, freeform geometry enables the relocation of welds away from member intersections, which simplifies fabrication by improving weld access and replacing complex welded joints with simple joints. Relocation of welded joints away from member intersections also reduces the stresses within the welds themselves.

Together, the replacement of complex, highly stressed welds with simple, lower stressed welds vastly improves fatigue performance, often providing greater than an order of magnitude increase in the service life of the overall connection. It also reduces the likelihood of brittle fracture because high levels of constraint in welds and throughout the connection can be avoided. Furthermore, cast steel material is generally isotropic and free of significant residual stresses, and grades that are weldable and exhibit elevated notch toughness can be readily produced.

All told, cast steel connections are ideal for use at geometrically complex, heavily loaded, or fatigue critical structural intersections.

AM offers even greater geometric flexibility than casting because part shaping isn't limited by molding or feeding considerations. While there are production-related considerations for designing AM parts, such as minimum features sizes and thickness, they are far less onerous than those required for casting design.

The numerous welding consumables available (such as carbon steel, high strength steel, and stainless steel) allow AM to produce parts from a wide range of materials. GMAAM has also demonstrated improved internal quality over castings, such as smaller and fewer voids, porosity, and inclusions. Early fatigue testing results are encouraging, though there remains additional research to better characterize fatigue performance. AM also offers the unique ability to deposit material directly onto structural steel elements or members, which can provide fabrication efficiency that castings cannot attain.

Heavily Loaded Truss Connections

For an AM versus casting and fabrication example, consider a heavily loaded truss built with round hollow structural steel (HSS) members.

With conventionally fabricated HSS-to-HSS connections, local connection limit states like chord plastification or shear yielding/punching may drive member selection, requiring the upsizing of steel members and leading to material inefficiency on the structure's macro level. Furthermore, welded joints at the tube-to-tube junctions are often TYK joints, which often have poor fit up, are difficult to weld, and exhibit relatively poor fatigue performance.

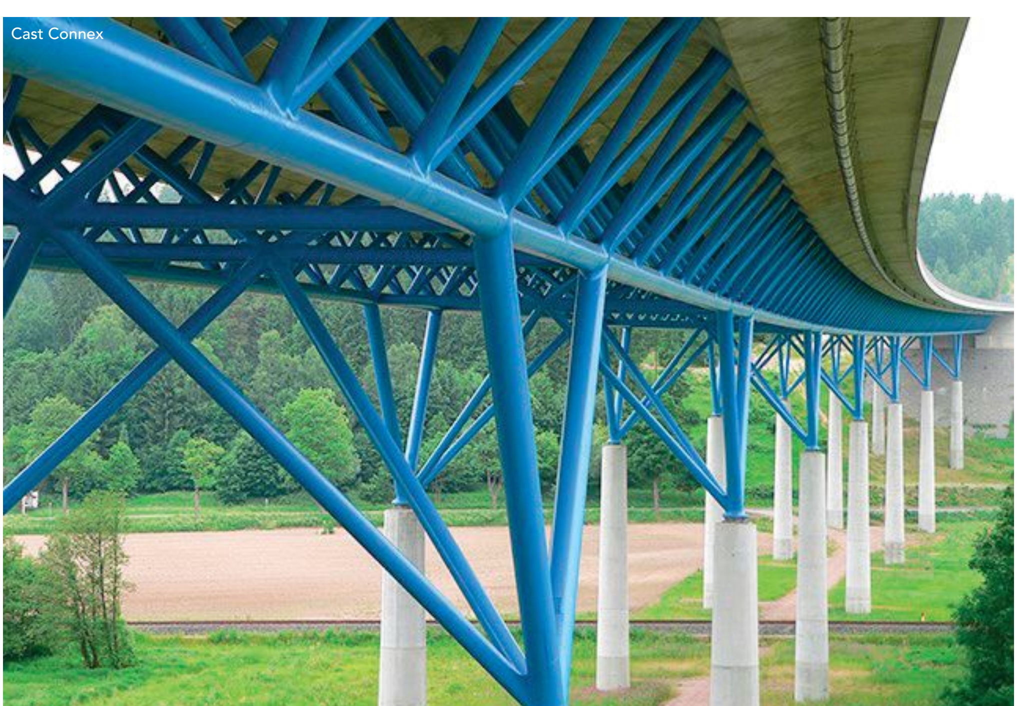
Member eccentricities are frequently introduced into the framing geometry to avoid overlapping weld toes at the connections, which can create additional bending moments and shear forces within the members of the truss. Additionally, connection flexibility at the HSS intersections may result in larger deformations and a longer fundamental period for the overall structure than predicted, unless structural analysis accounts for the potential semi-rigidity of the HSS connections.

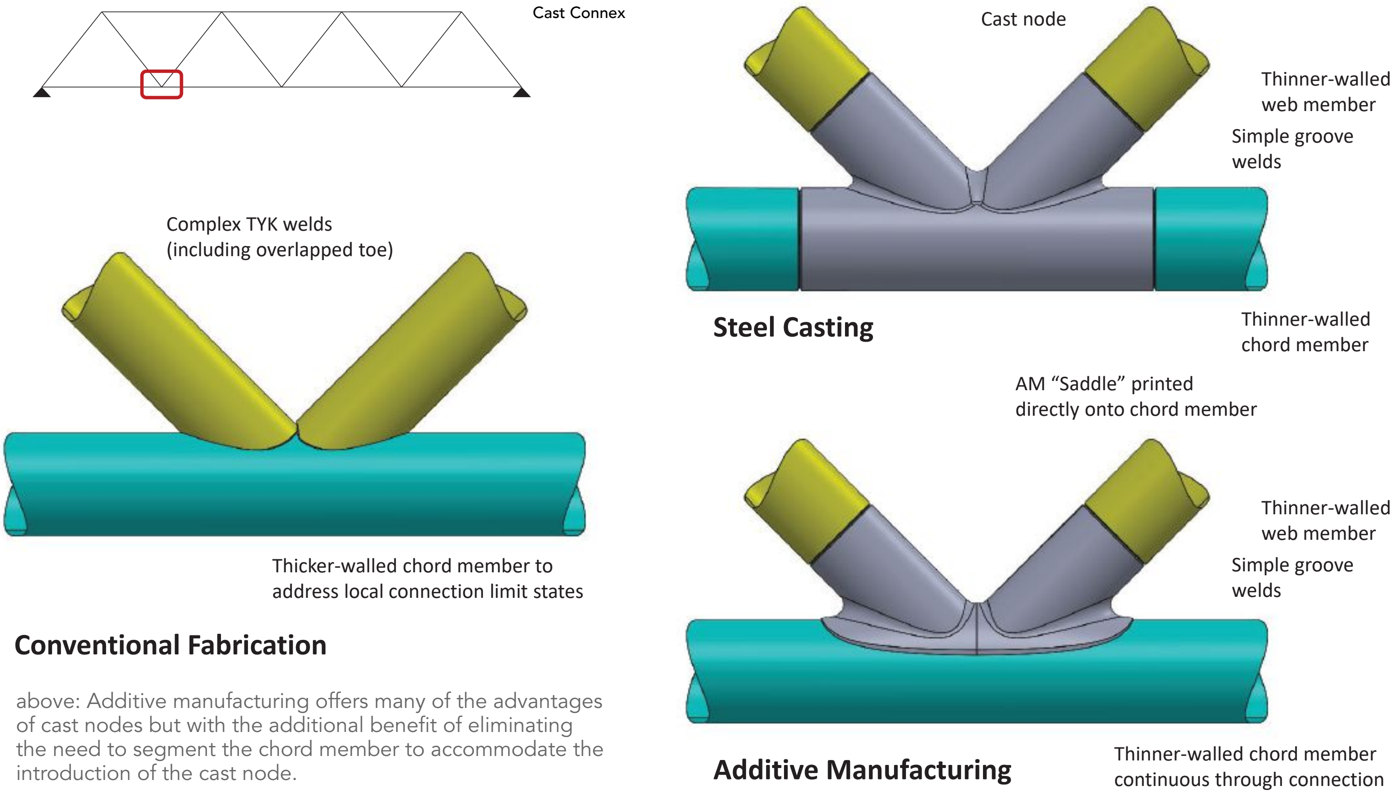
Using cast steel nodes at the HSS member intersections addresses those challenges. With cast nodes, local connection limit states are completely addressed within the node, which enables the use of lighter truss members and results in a lighter weight and more optimized structure. Local thickening at the connection region within the cast node also stiffens the connections, which improves the ability to estimate deformations and the fundamental period of the overall structure.

Cast nodal connections also simplify fabrication and improve the fatigue performance of the connection. Nodes eliminate the complex TYK joints present in a conventionally fabricated connection and replace them with straightforward groove joints, improving the fit up and ease of welding. The welds are also relocated away from the member intersections to regions of lower stress.

However, the use of cast nodes requires the interruption of the chord members of the truss, which introduces additional welded joints in the chord member that, depending upon the framing geometry and member sizes, may not have been necessary in the conventionally fabricated truss.

This highway bridge in Germany employs cast steel nodes at all of the HSS member intersections.


Cast Connex



above: Additive manufacturing offers many of the advantages of cast nodes but with the additional benefit of eliminating the need to segment the chord member to accommodate the introduction of the cast node.

below: An AM saddle component that was printed directly onto a round HSS member.

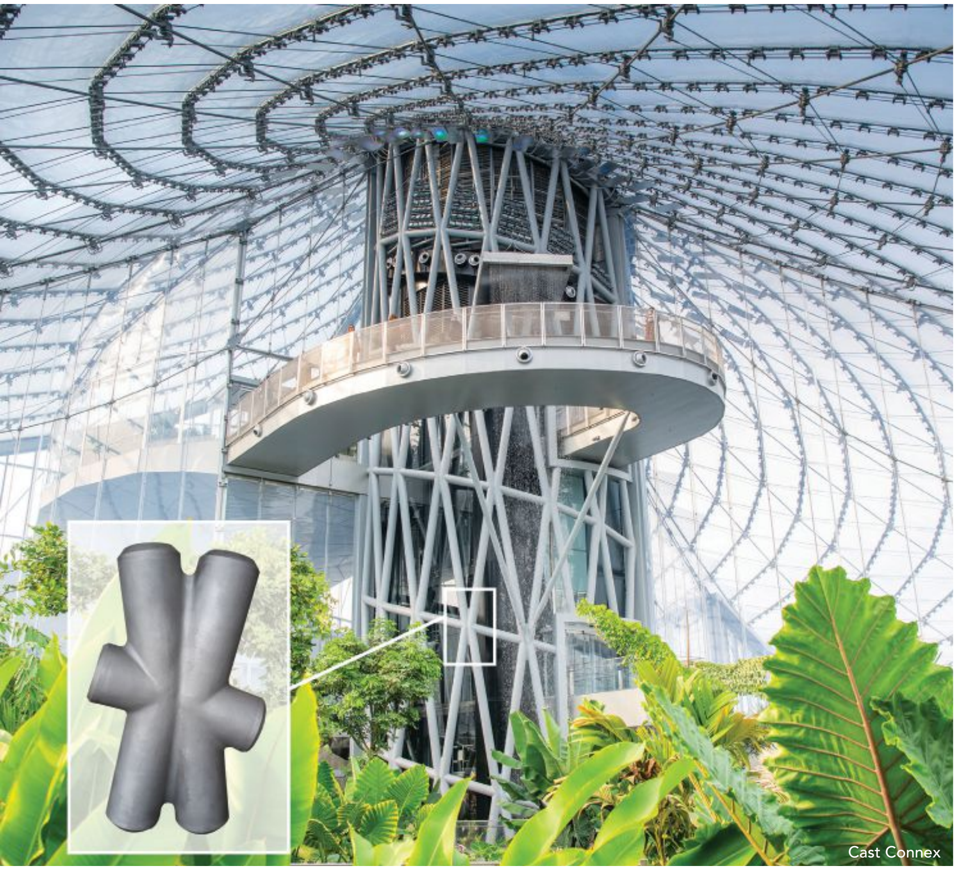
Incorporating AM nodes results in similar advantages to cast nodes. For example, AM allows for locally reinforcing the connection region of the chord member. It provides an additional advantage by printing a saddle component directly onto the chord member. With this approach, local connection limit states can be addressed without increasing member wall thickness, and connection regions are more rigid than conventionally fabricated ones. Similar to cast nodes, the complex TYK welds are eliminated, and the simple weld joints that remain are relocated to a region of lower stress.

AM provides cast nodes' benefits while also permitting the truss chord member to remain continuous through the connection zone. It has potential to print features internal to the node for performance enhancement, which are challenging to cast.

Architecturally Exposed Connections

Casting's appeal in architecturally exposed structural steel (AESS) connections is obvious. Its geometric capabilities enable production of flowing, freeform connections and components. Those can be part of and enhance the architectural expression of the building when exposed in the finished construction.

AM's heightened geometric flexibility



above: Two examples of architecturally exposed steel castings: nodes are used in the diagrid tower at The Leaf at Assiniboine Park in Winnipeg, Manitoba (left) and V-column bases are used at Charlotte-Douglas International Airport (right).

below: Stainless steel (316L) 8-in. gate valve body with as-printed and machined surfaces.

boosts the potential for architectural expression in connections and components. As with steel castings in AESS, the AM component's surface finish should be considered during design. As-cast surfaces of castings exhibit an orange-peel like surface texture and discontinuities such as metal removal marks and porosity depending on the non-destructive examination requirements specified for the parts. Supplemental processing is typical for steel castings intended for connections visible from close range in the completed structure.

AM parts produced via GMAAM exhibit an as-welded surface with visible ridges between adjacent weld beads. Like casting, supplemental processing such as grinding, machining, or post-fabrication filling and sanding can reduce or eliminate the as-printed surfaces of AM parts. The extensive interest surrounding AM, though, means some owners or designers may prefer to maintain the as-printed surfaces on AM parts to express or even showcase AM's usage.

One-Off Connections

Another significant advantage that AM has over casting manufacturing is that it can better address uniqueness across connection details.

Parametric design is gaining in popularity among architects and structural engineers. While connection repetition can be incorporated as a design constraint in parametric design, enforcing it limits the structure's achievable global geometry and may compromise efficiency. That's why connection repetition is a lower priority or a non-factor for designers, which in turn leads to designs with little to no repetition in connection geometry.

Using steel casting to manufacture multiple connections in

Lincoln Electric

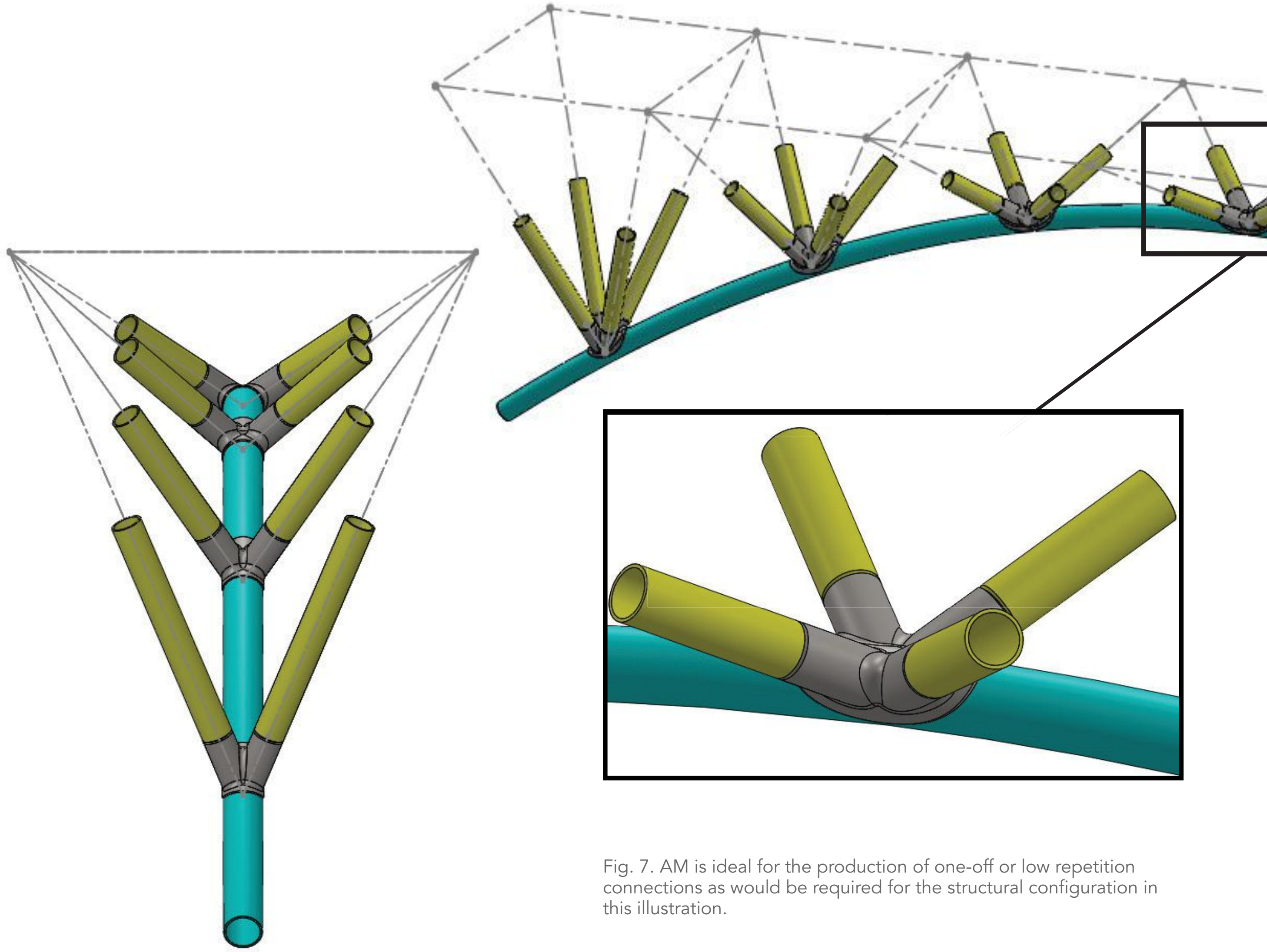


Fig. 7. AM is ideal for the production of one-off or low repetition connections as would be required for the structural configuration in this illustration.

projects lacking repetition can lead to prohibitive costs and timelines. Casting manufacturing requires molds, which require patterns to produce. While pattern change-out pieces and machining can address some geometric variation across similar components, their impact for addressing non-repetitive connections using castings is limited.

Meanwhile, AM is ideal for producing one-off connections. AM software, such as Lincoln Electric's SculptPrint™ OS, can readily slice and path plan incoming models and automatically program the AM system within hours. As long as a system is available with the appropriate wire feedstock, printing can begin straightaway after programming. There is no need for mold design and pattern manufacturing.

Cost, Design, and Specification Considerations

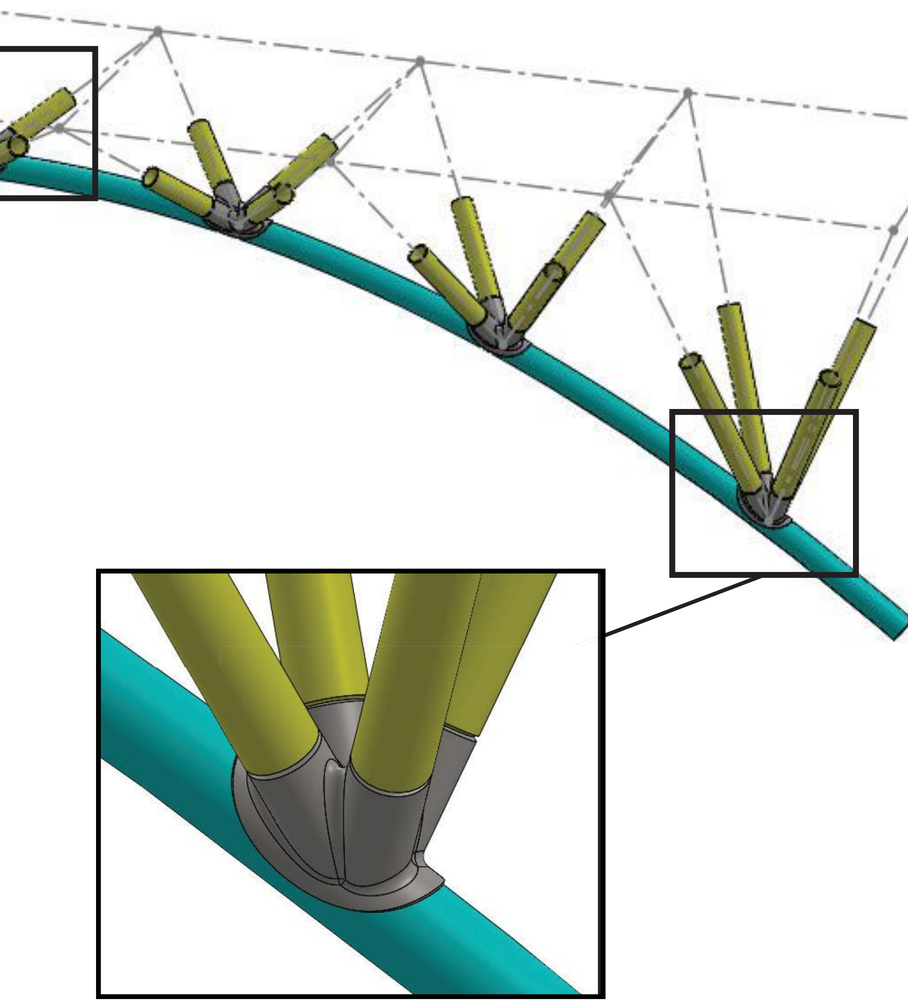
Material costs for rolled steel shapes and plate materials are less than material costs for both cast steel materials and AM wire feedstock. Castings come out ahead when fabrication labor prices comprise a large portion of the entire connection cost. Cast connections can offset some labor costs with their advantages in overall structural efficiency and performance, geometric precision, and aesthetics, creating many instances where they provide the most economical and highest value solution for a connection.

With AM, the cost per ton of carbon and low-alloy material is higher than that of equivalent cast steel grades, but creatively leveraging AM's technical advantages can make AM the best value and most economical solution for many situations. Moreover, AM being better suited to one-off designs extends range of commercial applicability for freeform structural steel components beyond casting's scope.

AM's direct-to-production capability also offers shorter lead times than casting manufacturing, creating many opportunities for AM to produce replacement parts for aging in-service steel castings in various industrial and manufacturing applications. It can also be an alternative to casting for structural applications when the casting lead time associated is prohibitive for the project's construction schedule.

As with castings, AM parts must be designed simultaneously for their end use and for the manufacturing process. Designers considering leveraging AM are best advised to work closely with AM product manufacturers or turnkey suppliers to develop manufacturable solutions for their projects. In these early stages of AM adoption, prototyping and physical testing parts may be warranted, and all parties should be prepared to rely heavily on first principles and engineering judgement as a key part of the design process. Early discussions with owners and authorities having jurisdictions about using AM on a project may help ensure the requisite buy-in of all stakeholders.

The interest in AM and prevalence of welding standards have sparked publication of various standards for AM fabrication, and



Cast Connex

many more are in development. The American Petroleum Institute Standard 20S provides requirements for qualification of the manufacturing process, production, marking and documentation for additively manufactured metallic components.

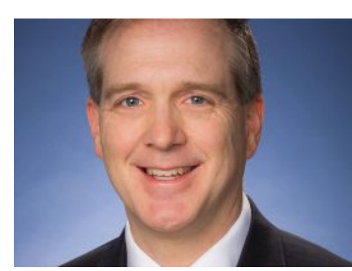
AWS D20.1 also has a specification for the design, qualification, fabrication, inspection, and acceptance of additively manufactured components. However, API and AWS see value in separating wire-based AM processes from powder-based. Both have established sub-committees to draft wire-based AM specific standards. Additionally, the International Institute of Welding, American Society of Mechanical Engineers, and U.S. Navy have sections and technical publications devoted to wire-arc AM.

Here to Stay

AM's upside is evident and intriguing, and it's not going away. Its place in the structural steel marketplace, though, is still forming, even as its advantages over casting in certain situations become clearer.

AISC has a task group considering how to best implement and promote AM in the steel industry. Its efforts funding research examining the performance of AM parts subjected to both static and fatigue loading. Additionally, AISC Task Committee 10 (Materials, Fabrication, & Erection) has established a working group that aims to develop language for the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) that will acknowledge AM in the code and provide general guidance on how to implement AM parts into steel structures.

Designers who want to take the AM plunge don't have to wait. Lincoln Electric's Additive business unit produces components for use in demanding applications in a variety of industries. Cast Connex, with its expertise in freeform steel connections, is actively seeking customers with projects that may be a good fit for AM production.



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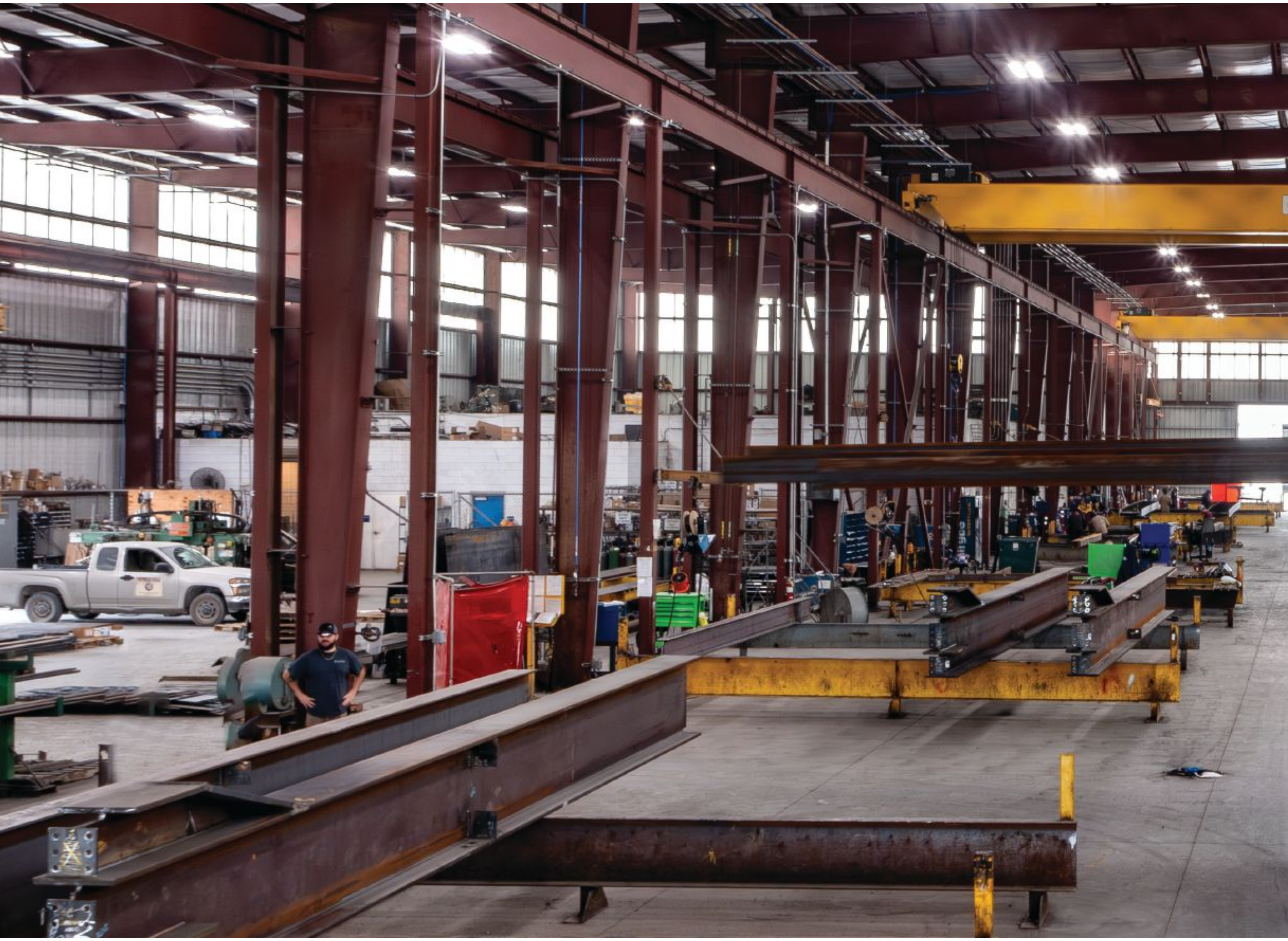

Growth Challenges

BY RICKY HORTON

MANY FABRICATION BUSINESSES started as a one-man operation, perhaps even in a garage. A barebones operation like a garage shop poses plenty of challenges, but it's also the easiest for an owner to oversee. In those initial stages, the owner is involved in every aspect of running the business, and management is straightforward because there are fewer moving parts and fewer people.

However, those businesses grow if managed well, and growth introduces complexities that will eventually stretch an owner too thin if he or she continues to handle all tasks and decisions. Some owners struggle when they reach that point, hindering their company's growth. Managing the shift from a small to a medium to a large fabricator poses difficulties, but ones that are navigable with the right plan. Effectively handling growth ensures that the growth experience remains enjoyable.

I have visited fabricators of varying sizes across the country and observed several challenges in maintaining efficiency during growth from a small operation to a business with triple-digit employees or multiple locations. My three steps for maintaining efficiency during a period of growth fit into a tidy acronym: ACT.

Greg Folkins Photography

Maintaining efficiency while growing from small operation to large company is often a fabrication business's biggest hurdle.

Automate Processes

If a fabrication business wants to compete with larger fabricators as it grows into one itself, it must consider automating fabricating processes in the shop. For growing companies, that generally involves purchasing new machinery and implementing it on the shop floor. The structural and miscellaneous steel fabrication industry has traditionally lagged in technology adoption, but an explosion of technological advancements over the past five to 10 years offers significant opportunities for improved speed and volume.

The automation boom has helped fabricators increase capacity while decreasing hours per ton produced. It has become even more crucial as workforce scarcity has tightened its grip on the industry.

Today's trade labor shortage makes employees harder to find, which in turn is leaving more positions unfilled in fabrication shops across the country. If those jobs are empty, reaching peak efficiency is challenging, if not impossible. Automation can often fill those gaps and help reach targets without a full staff.

Control Information

Efficiently managing fabrication shops requires that you effectively manage information. Leaders at fabrication companies must gather, analyze, and use the information to make decisions quickly. An inability to even gather the information will prohibit analyzing it and making effective decisions.

One of my clients at Fabrication Information Systems recently said he was constantly fielding calls concerning the fabrication status of projects in the shop. He was frequently running out of his office to talk with machine operators, fabricators, and material handlers to assess how much material had been completed and what remained. That means multiple people were pulling away from their work just to figure out what they had already done. Even then, he couldn't get a full accounting of every piece.

That's a good example of the information controlling the people rather than people controlling the information. The client needed help implementing a system that could quickly and accurately provide the information he needed with minimal user effort. In other words, he needed to gain control of the information.

Systems must be in place to disseminate the information so it is not siloed in one department or person. These systems are normally software and procedures that dictate flow of information throughout the organization. I have found that most businesses have them in place, but don't use them to their potential. Fabricators need to emphasize the power and simplicity of their systems to their workforce, which will save time and money. Chaos decreases as control of the information increases.

Trust Your People

Trusting the people within your organization is a challenging yet essential aspect of successful growth. Owners accustomed to making every decision may may need help delegating responsibilities as the company expands. It's crucial to allow new employees independence to perform their jobs without the fear of being micromanaged and undercut by the owner. Trust that your hiring process will bring in workers who will appreciate and meet expectations.

Expanding from a small to a medium-sized fabricator is hard enough, and it's even more difficult when growing into a large company. As the company grows, owners must distribute authority and delegate responsibilities throughout the organizational hierarchy rather than concentrate decision-making power within a limited group of individuals as if the business were still small.

Simply put, if the shop supervisor must seek input from the president or vice president before making decisions, inefficiencies and chaos will likely persist. However, fostering efficiency and order requires a shift towards delegating responsibilities and focusing on managing processes rather than micromanaging individuals.

*This article is a preview of the 2024 NASCC: The Steel Conference session "Maintaining Efficiency While Growing Your Fabrication Business." To learn more about this session and others, and to register for the conference, visit **aisc.org/nascc**. The conference takes place March 20–22 in San Antonio, Texas.*

Ricky Horton (**rhorton@fabricationis.com**) is the President of Fabrication Information Systems.

Builder Friendly

BY CLIFFORD SCHWINGER, PE

Constructability should be an important consideration during design.

ENGINEERS WILL QUICKLY earn the praise of fabricators, detailers, and steel erectors if they design structures with the construction process in mind.

While designing structures to meet building code requirements is paramount, considering constructability is also crucial—and failure to contemplate it could compromise strength and stability.

The more constructable a structure is, the more economical it will be. There are four core tenants of constructability:

- Simplicity equals economy.
- Less weight does not always equal less cost.
- The fewer the number of pieces, the more economical the design.
- Efficient and constructable connection details equal economical design.

More than 500 years ago, Leonardo Da Vinci said, "Simplicity is the ultimate sophistication". It was true then and is true today. Structural engineers should always be aware of and strive for simplicity in their designs.

Engineers should also strive to minimize the need for member reinforcement at the connections, which usually consists of multiple small pieces of steel, such as stiffener plates or web doubler plates. This steel—sometimes called the "gingerbread"—is often time-consuming, labor-intensive, and expensive to fabricate and install. Designers should instead consider reconfiguring the framing or upsizing members to minimize the need for member reinforcement at connections with an eye towards simplicity.

Two other constructability rules are important to remember during design. The fewer members in a structure, the fewer members to detail, fabricate, ship, erect, and connect. And the fewer pieces attached to a member (such as plates or stiffeners), the fewer pieces to detail and fabricate. While it may not be possible to eliminate all member reinforcing at connections, designers should be mindful of costs incurred due to member reinforcing and weigh it against the cost of using heavier members without member reinforcing. Likewise, there is always a trade-off between money saved by using fewer floor framing members (for example, using floor beams spaced further apart) versus the additional cost incurred by using thicker floor slabs.

Closely associated with constructability is the concept of connection designability. Designers can model anything, but can the model be easily built, and can the connections be designed? Many constructability problems are related to connection issues. In some cases, connections might not be unconstructable, but could be better described as constructability challenged. While those connections may be designable, they may be less cost-effective than an alternative framing configuration.

Because a significant percentage of a steel-framed structure's cost is related to the cost of connections, designers should consider allowing fabricators to use their preferred (and most economical) connections. That process is described in the *Code of Standard Practice for Steel Buildings and Bridges* (AISC 303-22), which provides a roadmap to help engineers navigate the challenges of designing more constructable structures.

Section 3.1(b) of the *Code* points designers to *Specification for Structural Steel Buildings* (ANSI/AISC 360-22) Section A4, which lists information they must consider and show on the design documents. Section A4, Item (j) states that engineers must consider "Any special erection conditions or other considerations that are required by the design concept," and identification and documentation of complex or difficult connections falls in that category.

For example, an analysis and design model might show eight beams framing to a column. Designers can model this condition, but designing and detailing the connections would be difficult. The engineer of record (EOR) is responsible for finding constructability-challenged connections in the model and resolving them during design.

The *Code* provides various options to assist the owner's designated representative for design (ODRD)—usually the EOR—in navigating the options for dealing with connections. ODRD's may choose any one of four options for designing and documenting connections on their projects:

- Option 1: (*Code* Section 3.2.3(1)): The ODRD designs the connections and provides complete details for both the connection design and the design of member reinforcement at the connections (where required) on the design documents.
- Option 2: (*Code* Section 3.2.3(2)): The ODRD designs and details the member reinforcement at the connections (where required), and provides sufficient connection detail information (typically connection tables, details, or both) to the fabricator to allow experienced detailers to select the appropriate details to complete the connections. Detailers are not expected to do any connection design. They choose pertinent connection details provided by the ODRD and detail the connections based upon that information.
- Option 3a (*Code* Section 3.2.3(3) & 3.2.4(2)(a)): The ODRD designs and provides details for all member reinforcement at the connections (where required). The ODRD also provides project-specific concept connection details showing the required conceptual configuration of the connections. The ODRD then delegates design of the connections to a licensed connection design engineer working for the fabricator.
- Option 3b (*Code* Section 3.2.3(3) & 3.2.4(2)(b)): The ODRD provides concept connection details for all member reinforcement at the connections (where required) and provides a bidding quantity for each member reinforcement detail. The ODRD also provides concept connection details for the connections. The ODRD then delegates connection design and design of the member reinforcement at the connections to a licensed connection design engineer working for the fabricator.

  If the ODRD neglects to provide concept connection details for the member reinforcement at the connections or does not provide bidding quantities for each member reinforcement detail, then bidders are not required to consider the cost associated with the member reinforcement. See Code Section 3.2.4(2)(b).

Code section 3.2.3 requires the ODRD to indicate which option applies to each connection. While the four Code connection design options vary in how the ODRD is required to deal with connection design and details, all deem the ODRD responsible for ensuring that connections will be constructable and designable.

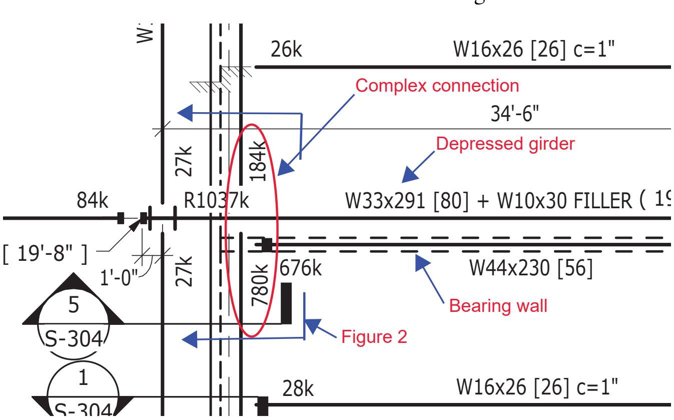

Fig. 1: Podium framing with constructability challenged connections.

Continual refinement of the *Code* has resulted in a document intended to help EORs produce complete, coordinated, and constructable designs. Designers must review their models to identify constructability problems and connection designability issues before design drawings are issued for construction and contracts are awarded. Ultimately, the ODRD is responsible for the safe design of all connections, even when delegating connection design.

The following are three examples of constructability and connection designability issues:

Figure 1 shows a partial framing plan of a second-level podium floor supporting ten levels of bearing wall construction. The podium level was modeled using widely used modeling software. A limitation of most modeling software used today is that the software considers neither constructability nor connection designability.

Figure 2 shows a designable but complex beam-to-girder connection detail (from the framing in Figure 1) requiring significant member reinforcing at the connections. The engineer potentially could have simplified the detail by altering the framing.

Figure 3 shows another section on a podium floor where connection constructability issues not flagged by the model necessitated reframing. In this case, connections interfered with each other, and several parallel framing members were too close together to allow bolt installation.

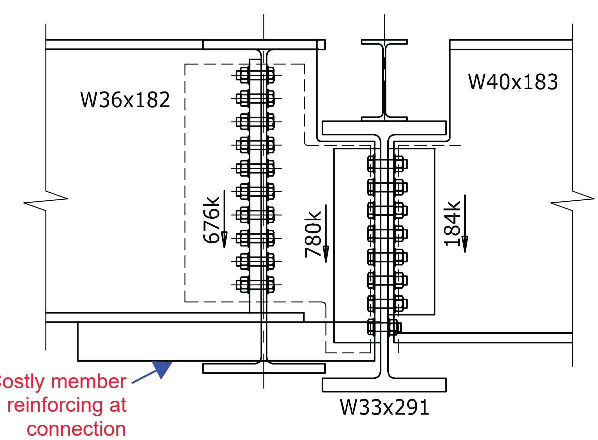

Fig. 2: A section through a girder that could be simplified.

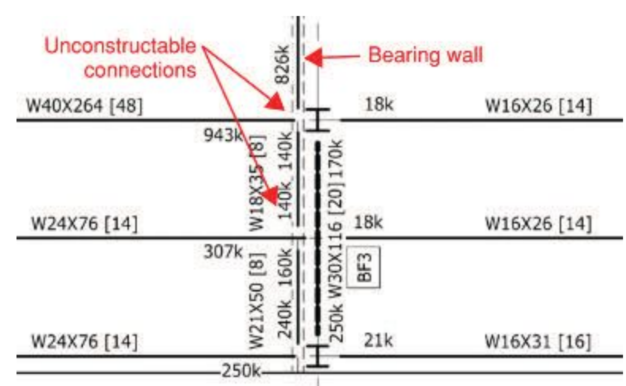

Fig. 3: Podium floor framing with unconstructable connections.

Fig. 4: A complex yet constructable transfer girder detail where the girder steps down.

Figure 4 shows a complex but constructable transfer girder detail requiring a step in the girder. The EOR provided a constructable concept connection detail on the contract documents. The fabricator's estimator accurately estimated the cost of the connection, and the connection design engineer designed and detailed the connection based on the concept connection details provided by the EOR.

Finding and resolving constructability and connection designability issues during design eliminates wasted time, needless arguments, potential serious design flaws, and reduces RFIs and change orders.

Artificial intelligence may one day identify and resolve constructability and connection designability issues during modeling. Until then, designers must closely examine framing members and their connections and clearly design and document connection details per *Code* Options 1 or 2 (with connections being completed by an experienced detailer for Option 2). Or, they must provide designable concept connection details and delegate the connection design per *Code* Option 3a (per Sections 3.2.3(3) and 3.2.4(2)(a)), using EOR fully designed details of member reinforcement at the connections per Option 3a, or by delegating design of the connections and member reinforcement at the connections for Option 3b (per Sections 3.2.3(3) and 3.2.4(2)(b)). Following these procedures will result in more constructable and problem-free projects that come together as quickly and efficiently as possible. ■

This article is a preview of the 2024 NASCC: The Steel Conference session "50 Tips for Improving the Constructability of Steel-Framed Building Structures." To learn more about this session and others, and to register for the conference, visit ***aisc.org/nascc****. The conference takes place March 20–22 in San Antonio, Texas.*

To hear more from Cliff, check out the August 2023 Field Notes column "Quality Time" and listen to the associated podcast at ***modernsteel.com/podcasts****.*

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Constructability Tips

1. Show the reactions—including transfer forces. (Do not simply refer to Table 3-6.)
2. Consider delegating connection design per AISC 303-22 to a licensed engineer working for the fabricator.
3. Frame girders to column flanges and beams to column webs.
4. Size columns to eliminate the need for stiffeners.
5. Do not arbitrarily specify CJP welded moment connections.
6. Minimize the number of skewed connections.
7. Frame members with large reactions square to supporting columns.
8. Frame no more than one member to any one side of a column.
9. Head off steeply skewed beam-to-girder connections.
10. Avoid skewed columns in braced frames.
11. Favor round HSS columns and posts over rectangular HSS when skewed members frame to the HSS.
12. Look for connections clashing with other connections.
13. Verify that bolts can be installed and welds can be made.
14. Understand EOR connection design responsibilities per AISC 303-22
15. Use beams with flanges at least 6 in. wide that have flange-bolted moment connections.
16. Limit the tension yield strength ratio to 0.75 when sizing tension members.
17. Communicate and coordinate with other consultants.
18. Look for unreasonable or excessive beam web penetrations.
19. Avoid indicating arbitrary CJP welds.
20. Avoid specifying unnecessary all-around fillet welds.
21. Favor fillet welds over groove welds.
22. Configure diagonal brace slopes between 35 and 55 degrees to facilitate connections.
23. Configure floor framing to minimize the number of beams.
24. Maximize slab span to minimize the number of beams.
25. Use R=3 for seismic design whenever possible.
26. Orient columns in moment frames to bend about strong axis.
27. Run heavy moment-connected girders continuous through columns.
28. Run cantilevering roof beams continuous over tops of columns.
29. Minimize the small pieces of steel known as "gingerbread."
30. Avoid horizontally skewed beam-to-column moment connections.
31. Avoid full-depth stiffeners where possible.
32. Orient wide flange columns on transfer girders with webs parallel to girder webs.
33. Simplify baseplate and anchor rod details.
34. Use angle hangers when possible—avoid HSS hangers.
35. Avoid torsion in W-Shapes.
36. Always think about the connections during design and modeling.
37. Strive to keep connections square.
38. Identify where member reinforcement at connections is required—and seek to minimize it.
39. Communicate and coordinate.
40. Understand local fabricator preferences regarding preferred connection details.

Bridging the Gap

BY ED AVERY, JIM FOREMAN, SE, PE, WYATT SOEFFING, AND
ALEX STONE, SE, PE

Even thought thermal break pads in steel construction are not yet incorporated in building codes, they're a smart way to avoid a costly serviceability issue.

A PESKY TYPE OF BRIDGE has the design community's attention, and it doesn't involve spans or abutments.

Expectations for energy efficient buildings are rising as jurisdictions set new targets and establish green standards, and the design and construction communities have important roles to play in providing energy efficient buildings. Over the last 10 years, designers have become more focused on eliminating thermal bridges from building envelopes.

Designers have multiple options for thermal bridging avoidance. In some cases, thermal bridges can be eliminated with thoughtful detailing. In other cases, designers are incorporating thermal break products into the structural system, a common method in some regions that requires careful analysis and design.

Thermal bridging has considerable consequences. Unmitigated thermal bridging can account for 20% to 70% of heat flow through a building envelope, according to Applied Building Technology Group. And a significant concern with thermal bridges is condensation, which can result in mold, damage to interior finishes, and structural deterioration.

At exterior walls, steel thermal bridges can be created by balconies, canopies, and sign supports (Figure 1, left). At roofs, thermal bridges include platforms or dunnage supporting mechanical systems, screen wall posts, and fall protection or façade access anchors (Figure 1, right).

Some thermal bridging conditions can be improved with thoughtful structural and architectural detailing. Otherwise, thermal bridges can be mitigated by interrupting the continuous steel member and creating a bolted splice connection with a thermal break pad, or TBP (Figure 2).

There are, however, some structural issues that must be considered. The AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) references the 2020 RCSC *Specification for Structural Joints Using High-Strength Bolts*. Section 3.1 of the RCSC *Specification* states, "Compressible material shall not be placed within the grip of the bolt." Also note that the commentary to Section 1.1 of the RCSC *Specification* states, "These provisions do not apply when material other than steel is included in the grip…" The designer of a joint utilizing a thermal break material should ensure that the

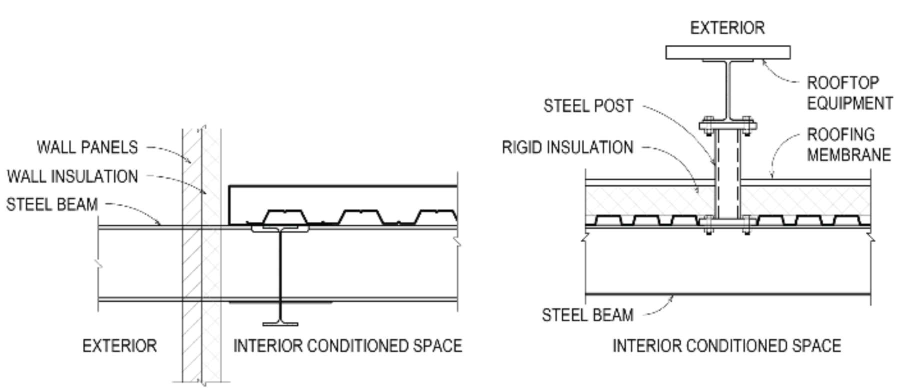

Fig. 1. Examples of thermal bridges: beam penetration (left) and column penetration (right).

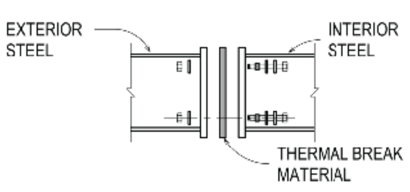

Fig. 2. Typical thermal break assembly.

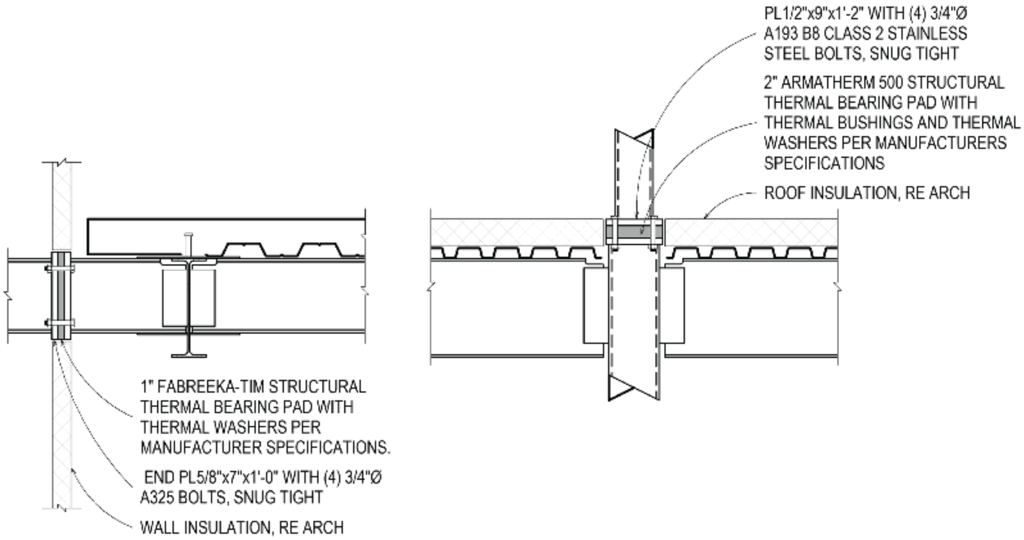

Fig. 3. Example detail callouts for TBPs.

connection including the fasteners will perform as expected with the thermal break in place in the grip.

TBPs are a part of the structural system and need to be clearly specified in the structural documents. At a minimum, TBP size and thickness should be identified, as well as any special fastener requirements. Thermal break requirements can be shown in detail (Figure 3), described in plan notes, or addressed in the specifications. Among the things to consider when implementing breaks:

Thickness: Ideally, the project architect or building envelope consultant should determine the thermal break pad thickness. The structural engineer needs to specify the pad thickness in their drawings and account for that thickness in the connection design. Research has shown effective TBPs are at least 1 in. thick, with additional thickness improving thermal performance.

Including a thermal break pad at the splice improves the condition. But, in an interesting twist, if the pad is too thin, the overall thermal condition may become worse than if the beam were continuous. A University of Alaska Anchorage experimental study and a Morrison Hershfield analytical study concluded that simply splicing a steel beam with a bolted end plate connection (but without a TBP) increases the heat flow rate through the member, resulting in a more significant thermal bridge than the continuous beam condition.

Washers and bushings: Thermal washers and bushings are not necessarily required for every thermal break connection. If they are included, the washers and bushings need to be clearly specified on the project drawings or in the project specifications. Furthermore, some TBP manufacturers recommend special washer detailing when bushings are used.

Published studies indicate mixed conclusions as to the effectiveness of thermal bushings and washers. A Morrison Hershfield 2020 study indicated significant improvement in thermal performance, while a Northeastern University study from 2017 concluded the impact is negligible. Designers are left to balance cost, benefits, and added complexity of specifying thermal washers and bushings.

Bolts: Bolts are another crucial consideration in thermal breaks. Stainless steel is roughly three times less conductive than carbon steel, meaning stainless steel bolts in lieu of carbon steel bolts at a thermal break connection will improve the break's effectiveness.

However, stainless steel bolts are significantly more expensive than A325 bolts and can have less strength than carbon steel bolts. Per AISC Design Guide 27: *Structural Stainless Steel*, specifying stainless steel requires special considerations such as galvanic corrosion, a bolt tightening qualification procedure if bolt pretension is specified, and lubrication of bolts to prevent galling of the threaded surfaces.

Inspection: Thermal breaks should be included in special inspection. Special inspections of steel construction conform to *International Building Code (IBC)* Chapter 17 requirements, which reference AISC *Specification* Chapter N to define what quality assurance inspections are completed by the third-party special inspector.

The current AISC *Specification* does not reference connections with thermal breaks, so the designer should consider adding to the statement of special inspections within the construction documents. Items that may be listed within the required inspections include thermal break pad size, thickness, material specification, thermal bushing and washer installation (if applicable), and bolt size, grade, and installation.

Structural Design

Currently, formal building codes do not provide guidance for engineers designing connections with TBPs. The RCSC *Specification* discusses thermal breaks in the Section 1.1 commentary. The commentary clarifies that the RCSC *Specification* does not apply to connections using thermal breaks and states that thermal break joints are not intended for primary load resisting systems.

Regardless of the code limitations, thermal break pads are being used in steel construction, and engineers must address them in their connection designs. There are several design methods to evaluate these connections that have been published by thermal break suppliers and academic researchers.

Steel-to-steel connections using thermal break pads are predominantly achieved using bolted end plate connections. Any design approach must address shear and moment transfer through the thermal break pad.

Bolt Bending: One approach is to transfer the shear force through bolt bending, with the shear force resisted by single curvature bolt bending, neglecting the presence of the TBP. The bolt will more likely behave in double curvature due to the clamping of the bolt head and nut, but published research currently recommends the single curvature bending model. AISC *Specification* section J4.5 can be used to design the bolts for single curvature bending.

The magnitude of the bending moment in the bolt is the bolt shear force times the TBP thickness. In connections transferring moment due to a cantilevered member, the moment is resolved by forming a compression and tension force couple, where the tension is resisted by the bolts and the compression is resisted in bearing between the end plates and through the TBP (Figure 4). Bolts are designed for the shear, tension and moment using the combined forces equations in AISC *Specification* Chapter H.

Filler Plate: Shear can also be transferred when the TBP is treated as a filler, which is a common design methodology in Europe but one the AISC *Specification* does not address. The bolts can be designed according to AISC *Specification* section J5.2, Fillers in Bolted Bearing-Type Connections. In this case, the shear is transferred using the shear capacity of the bolts, multiplied by a reduction factor based on the TBP thickness. For this method, it is recommended that the thickness of the pad should not exceed $4d/3$, where d is the nominal diameter of the bolt.

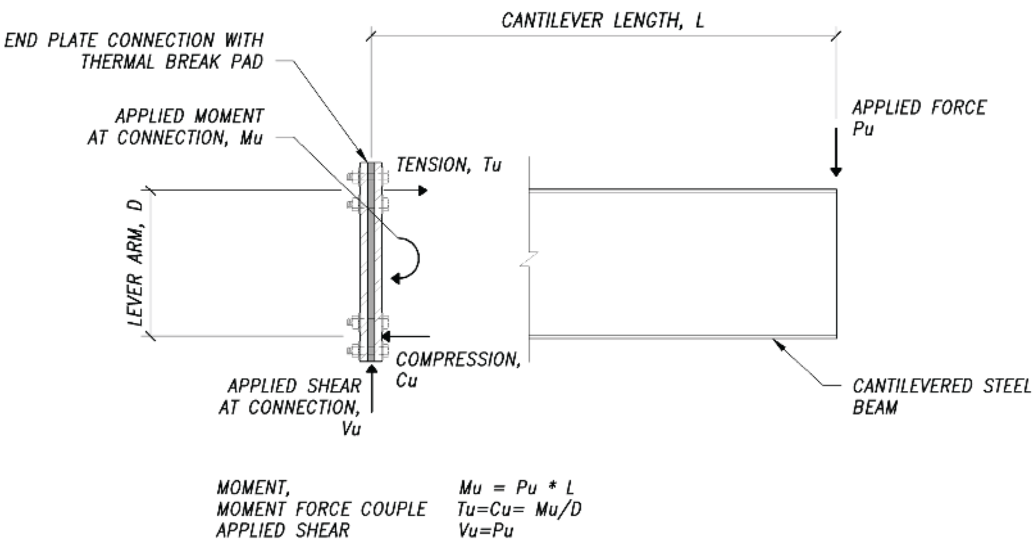

Fig. 4. Free body diagram and force transfer with cantilevered beam and end plate connection.

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Clamping and shear friction: The other mechanism to transfer the shear relies on the compressive force of the moment couple and the frictional coefficient between steel and the TBP to provide shear resistance across the TBP. The manufacturer's product data provides a frictional coefficient—a value of between 0.25-0.30 can be expected. If pretension in the bolts is specified, it can create an additional clamping force. But caution should be exercised when using bolt pretensioning due to lack of research and potential loss of pretension due to pad deformation (including creep).

Another structural design consideration is the compressive force of the moment force couple, which passes through the TBP and creates a compressive stress within the TBP. Compressive stresses in TBPs should be limited to 35% or less of the compressive strength of the material to limit creep.

Most proprietary TBPs have compressive strengths above 30 ksi for applications that transfer bending moments, though the compressive modulus is significantly lower than that of steel. Pad deformation, and more importantly creep, are important considerations when evaluating the additional beam rotation that can occur at thermal break connections. This is especially important in cantilevered members where additional beam rotation causes more deflection at the end of the beam.

Fig. 5. Thermal washer failure at bolt with regular oversized washer.

Fig. 6. Thermal washer failure at bolt with regular oversized washer.

Fabrication and Erection

The main challenge for fabricators and erectors is properly estimating the cost of including thermal breaks in project bids. Clear details and specification of the TBPs and assemblies in the contract documents are critical for pricing. Thermal break assemblies also impact erector installation costs because they take more time and labor to install. A typical four-bolt end plate connection can be assembled by just one iron worker. Adding thermal washers, bushings, and a pad to the connection complicates assembly: an improved thermal break connection can require two iron workers to keep all components in place during installation.

A common field issue with assemblies is thermal washer failure during the bolting process, specifically when bushings are used due to the use of much larger holes in the end plates to accommodate bushings. To avoid failure (Figures 5 and 6), manufacturers suggest using USS Grade 8 steel washers on both sides of the thermal washer to prevent crushing.

Shop and field modifications of TBPs can be difficult. Special safety precautions are required to cut, drill, or modify TBPs due to their composition leading to potential respiratory damage and special equipment may be required.

In general, thermal break materials should be considered a final product rather than material that can be modified. With building codes unlikely to catch up soon due to long code cycles, the design and fabrication communities must exercise judgment when using TBPs. Understanding their function and the limits is critical for designing and building successfully with them. ■

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new products

This month's New Products section features two updates to Computations & Graphics softwares, a Dlubal software updated to incorporate AISC 360-22, and an A1A rigging software.

Computations & Graphics Real3D v22 and sCheck

Real3D is an innovative structural design and finite element analysis software tailored for engineers of all expertise levels. This powerful tool seamlessly combines reliability, user-friendliness, and affordability, prioritizing accuracy and simplicity. Real3D stands out with its unique quad-precision solver, adept at handling numerically challenging structures. Boasting a rapid sparse solver based on the Intel (R) Math Kernel Library, it effortlessly manages models with millions of degrees of freedom.

The software offers convenience through spreadsheet input and output, intuitive command line input, and detailed reports in both Microsoft Word and PDF formats. Its visually compelling graphics, built on OpenGL(R) standards, enhance the user experience. Its latest iteration, Version 22, integrates the most up-to-date steel code with the *Specification for Structural Steel Buildings* (ASNI/AISC 360-22).

sCheck is a simple yet powerful Windows tool that analyzes and designs steel shapes according to the *Specification*. Its key features include:

- Shape compatibility: sCheck assesses the capacity of various AISC shapes: W, M, S, HP, C, MC, L, WT, MT, ST, 2L, HSS, Pipe.
- Optimal design: sCheck selects the best AISC shapes for your project based on load effects.
- Moment magnification: sCheck can account for moment magnification in non-sway conditions.
- Professional documentation: sCheck generates incredibly detailed calculation procedures in Microsoft Word and PDF formats.

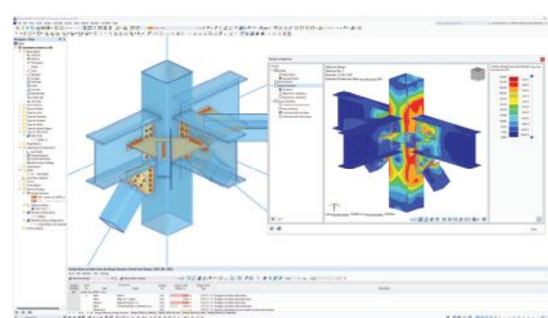
Visit **www.cg-inc.com** to download a 15-day trial of the full versions.

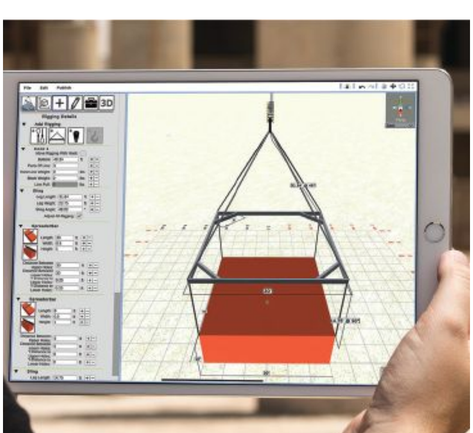
Dlubal RFEM6

Dlubal Software has enhanced its RFEM 6 finite element analysis (FEA) structural analysis and design software to incorporate the *Specification for Structural Steel Buildings* (ANSI/AISC 360-22) connection design. This design surpasses conventional analytical models by internally generating an FEA model, enabling the creation of unique or non-standard connections.

Users can select from a diverse library of predefined steel connection templates or create their own. Complete bolt, weld, and plate checks with AISC design formulas and equations are given. A buckling sub-model is automatically generated to identify various buckling failure mode shapes of plate elements.

The program includes comprehensive design of the global structure, such as AISC member design checks. All member end forces are incorporated directly into the steel connection design within a single model. Eliminate data loss during the transfer of member end forces when switching between external programs, and experience RFEM's all-in-one design solution. For more information, visit **www.dlubal.com**.





A1A Rigging Designer

A1A Software LLC, developer of 3D Lift Plan, has released a new application-based tool for anyone who needs to create rigging plans. The stand-alone application, available in the A1A Product Suite, provides detailed documentation for pre-lift planning, while also being easy to use on the go for planning in the field.

Rigging Designer is pre-loaded with 3D equipment, such as mobile cranes, forklifts, overhead cranes, or gantries. A library of more than 1,300 3D objects includes common loads lifted in construction and industrial applications and common buildings. Users can also create their own objects by entering the dimensions.

Rigging Designer features easy-to-use drag and drop functionality and mirror/duplicate settings. Users can add notes regarding the lifting equipment to be used. Print your plan for use by rigging crews and field personnel, or create digital records that can be imported into the full version of 3D Lift Plan. For more information, visit **www.a1asoftware.com/products.**

IN MEMORIAM

Engineering Icon Charles Thornton Passes Away At 83

Thornton Tomasetti

Engineering icon Charles Thornton died December 12 following a brief illness. He was 83.

Thornton's career in structural engineering spanned more than 60 years. Its highlights included serving as chair and CEO of Thornton Tomasetti and founding the ACE Mentor Program of America. He retired from Thornton Tomasetti in 2004, but held an advisory role for several years afterward. He also served as chairman of the Charles H. Thornton Company, LLC, a management and strategic consulting firm, following his retirement from Thornton Tomasetti.

"Charlie Thornton was driven to succeed," said AISC President Charles J. Carter, SE, PE, PhD. "He did so in many ways, but the most poignant one for me comes from the story he told me about how he used to dislike public speaking. He didn't just overcome that—he became a generational voice in our profession."

Under Thornton's leadership, Thornton Tomasetti designed prominent skyscrapers, airports, entertainment venues, transportation hubs, and special and long-span structures all over the globe. That work included some of the world's first supertall towers, including Taipei 101 in Taiwan. He was also regarded as an expert in collapse and structural failure analysis.

"Charlie was a truly visionary structural engineer," said Degenkolb Engineers COO and Senior Principal Jim Malley. "From leading the design of some of the world's iconic structures to growing a small New York City firm into a global engineering powerhouse to founding the ACE Mentor Program, Charlie's life and career were transformative and inspirational."

Founded in 1994, the ACE Mentor Program now introduces more than 10,000 high school students each year to potential careers in architecture, engineering, and construction—with the guidance of more than 4,000 mentors. In 2011, President Barack Obama honored it with the Presidential Award for Excellence in Science, Mathematics and Engineering Mentoring.

Thornton earned several awards and accolades during his career. He earned an AISC Lifetime Achievement Award in 2010. He was inducted into the National Academy of Engineering and the National Academy of Construction. The American Institute of Architects (AIA) and the American Society of Civil Engineers (ASCE) each named him an honorary member, and ASCE also gave him its Outstanding Projects and Leaders Award. The Franklin Institute honored him with the Benjamin Franklin Medal in Civil Engineering. He and Richard Tomasetti earned the Council on Tall Buildings and Urban Habitat's Fazlur R. Khan Lifetime Achievement Medal.

Thornton was a devoted husband, father, grandfather, and great-grandfather who enjoyed sailing and painting.

People & Companies

Gerdau Long Steel North America (Gerdau) announced the completion of a solar farm next to its steel mill in Midlothian, Texas. Built in partnership with 174 Power Global, a leading solar energy company, and TotalEnergies, a global multi-energy company, the plant started providing power to the steel mill this summer.

Built adjacent to the Midlothian mill and equipped with 187,000 solar panels, the complex has the capacity to generate 80 megawatts (MW), the equivalent of the annual energy consumption of 14,000 Texas homes. The project also utilizes more than 3,000 tons of Gerdau beams, produced at the Midlothian mill.

"This is an example of the circular economy at work: we recycled scrap metal to produce world-class steel, which was then utilized in a renewable energy project that will improve our future environmental performance," said Scott Meaney, Gerdau vice president of sales and marketing. "Gerdau beams have the lowest embodied carbon in the U.S., based on Environmental Product Declaration data. This project is further reducing our emissions, allowing us to supply our customers with a cleaner product.

The Midlothian project is the result of a 20-year energy purchase agreement between Ellis Solar, LLC and Gerdau. TotalEnergies and 174 Power Global each hold a 50% stake in the Ellis Solar, LLC joint venture.

Nous Engineering announced the addition of two new principals: Jeff Roi, SE, who joins from Degenkolb, and Jon Buckley, SE, from Miyamoto. Roi brings specific expertise in lab, biotech, and healthcare seismic retrofits and tightly-calibrated new builds. Buckley brings a strong track record of providing innovative structural solutions to K-12, community college, and higher education campuses, along with essential services facilities where critical operations cannot be disrupted.

AWARDS

AISC Milek Fellowship Award Recipient Announced

AISC has awarded the 2024 Milek Fellowship to a researcher working toward bringing automation to the steel design process.

Mohannad Z. "M.Z." Naser of Clemson University has earned the fellowship, which is given out annually by the AISC Committee on Research, for his research proposal "SteelGPT: Automating Structural Design of Steel Structures." Naser aims to use artificial intelligence (AI) and machine learning (ML) to create a virtual assistant named SteelGPT that will help enhance the steel design process.

Naser is not creating SteelGPT to replace the design process. Rather, he hopes it will aid designers and fabricators by flagging potential improper designs or assumptions and offering suggestions for how to improve design. It's intended to be a resource to help design engineers develop an optimized and safe design while speeding up the overall process.

SteelGPT will be trained on and incorporate multiple AISC publications as data sources, including the *Specification for Structural Steel Buildings* (ANSI/AISC 360), *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341), and the *Steel Construction Manual*, as well as answers provided by the Steel Solutions Center.

Naser plans to develop SteelGPT by repurposing popular AI assistant ChatGPT's open-source Large Language Model (LLM) to structural steel design. His research team will explore and tune ChatGPT's intricate layers and the attention mechanisms that allow it to generate text. The research will encompass ChatGPT's knowledge breadth and depth, focusing on the parameters and layers that provide ChatGPT with its high-end comprehension capabilities.

The research team plans to use well-known ML techniques, such as tokenization and embedding, to convert raw data into a machine-compatible format. Much of the research effort will focus on filtering out redundancies or outdated information. SteelGPT will be a live program that will accept future editions of AISC technical documents and other relevant industry standards.

Naser is a professional engineer and assistant professor at the Clemson University School of Civil and Environmental Engineering and Earth Sciences. He's also a faculty member of the AI Research Institute for Science and Engineering (AIRISE), the chair of the American Society of Civil Engineers (ASCE) Advances in Information Technology committee, and a voting member of several national and international engineering institutes. He has authored more than 140 peer-reviewed publications and also made the Stanford University-Elsevier list of the world's top 2% most cited scientists each of the last two years.

The Milek Fellowship is a four-year, $75,000-per-year award presented annually to a promising non-tenured university faculty member. It's named for former AISC Vice President of Engineering and Research William A. Milek, Jr. The 2023 winner was Georgia Tech professor Ryan Sherman, whose research project is titled, "Additive Manufacturing for Structural Steel Applications." To learn more about Sherman's project, read the Field Notes column in the December 2023 issue of *Modern Steel Construction* or listen to the Field Notes podcast at **modernsteel.com/podcasts**.

SSBC

2024 Student Steel Bridge Competition Rules Announced

Students around the country are gearing up for another exciting Student Steel Bridge Competition—and AISC and the American Society of Civil Engineers have announced the challenge that they'll solve by designing, fabricating, and erecting a scale-model steel bridge.

In this year's hypothetical scenario, Lincoln Parish Park in Ruston, La., is considering adding a disc golf course with a non-invasive, man-made river as a water hazard. And where there's water, there must be a bridge for players, park employees, and maintenance vehicles to get to the other side. Lincoln Parish Park is one of the most popular in the nation because of its aesthetic pleasure. The bridge in it must be aesthetically pleasing as well.

This year, teams will be allowed to install and use temporary barges in the man-made river to facilitate construction—but doing so will increase a bridge's cost. The full rules and updated resources for teams and faculty advisors, are available at **aisc.org/ssbc**.

"Students consistently tell us that the Student Steel Bridge Competition is not only a highlight of their college experience but provides an invaluable hands-on perspective that informs their careers in the real world," said AISC Senior Director of Education Christina Harber, SE, PE. "I am so excited to see what this year's competition holds."

The Student Steel Bridge Competition is an annual competition that challenges student teams to develop a scale-model steel bridge. Each team must determine how to fabricate their bridge and then plan for an efficient assembly under timed construction at the competition. Bridges are then load-tested and weighed. The bridge must span approximately 20 ft, carry 2,500 lb, and must meet all other competition rules and specifications. Bridge aesthetics are also judged.

The 20 regional competitions will take place at ASCE Student Symposia throughout the country in the spring, and the top teams will gather at Louisiana Tech University for the national finals on May 31 and June 1, 2024.

AISC and ASCE would like to thank this year's sponsors: Nucor, W&W | AFCO Steel, Bentley Education, Trimble, the American Galvanizers Association, Atlas Tube, GWY, LeJeune Bolt Company, SpaceX, St. Louis Screw and Bolt, and Unytite, Inc.

advertiser index

Dave Soulier (center) with his daughter Cameron (left) and AISC President Charles J. Carter.

Manual Mania

DAVE SOULIER collects editions of the AISC *Steel Construction Manual* like other intrepid gatherers collect baseball cards. Soulier, a senior civil engineer at Cajun Engineering Solutions in Baton Rouge, La., has all 16 editions of the *Manual* in his bounty. He has snatched some older ones away from pesky potential purchasers in eBay auctions. His collection features pre-AISC steel books that date back to the 19th century. More recently, he bought the 16th edition *Manual* when it was released in August.

This fall, Soulier saw the prize that would complete his collection—the *Manual* equivalent of the Honus Wagner T206 baseball card.

AISC gave away 16 specially bound copies of the 16th Edition, numbered and signed by AISC President Charles J. Carter. To claim signed copy number one, though, Soulier needed to concoct an expression of his *Manual* affinity grand enough to win a contest.

His efforts bested the other *Manual* enthusiasts, but only with the help of his daughter Cameron. Soulier and his daughter wrote a parody of Toto's "Africa" that trumpeted his desire to add the special edition to his collection, which Cameron sang on video while holding various older versions of the *Manual*.

Carter hand-delivered the special edition to Soulier and his family over lunch in Baton Rouge. Soulier professed how much he appreciated the *Manual*'s organization and information, especially the tabs for major sections. Cameron earned an AISC microphone trophy for her efforts.

While you can't get your own special edition, the 16th edition *Manual* can be had with a simple online order. It's jammed with all the same information and updates as Soulier's prize. You can read more about its features in this month's and last month's SteelWise articles. Order your copy of the gold standard in steel design and construction at **aisc.org/publications**.