MSC

MODERN **STEEL** CONSTRUCTION

July 2009

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
AISC
FOUNDED 1921

Global Perspective

International Projects

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At CMC Joist & Deck, we understand the importance of saving our environment; that's why we manufacture products that are green building friendly. Commercial Metals Company (CMC) is a leader in the recycling industry and a member of the U.S. Green Building Council. At CMC Joist & Deck, we typically can provide LEED® (Leadership in Energy and Enviromental Design) Materials and Resources credits in the areas of recycled content and regional materials.

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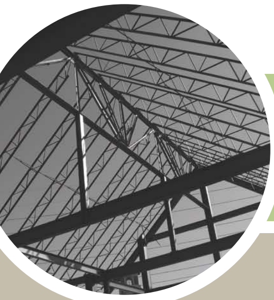

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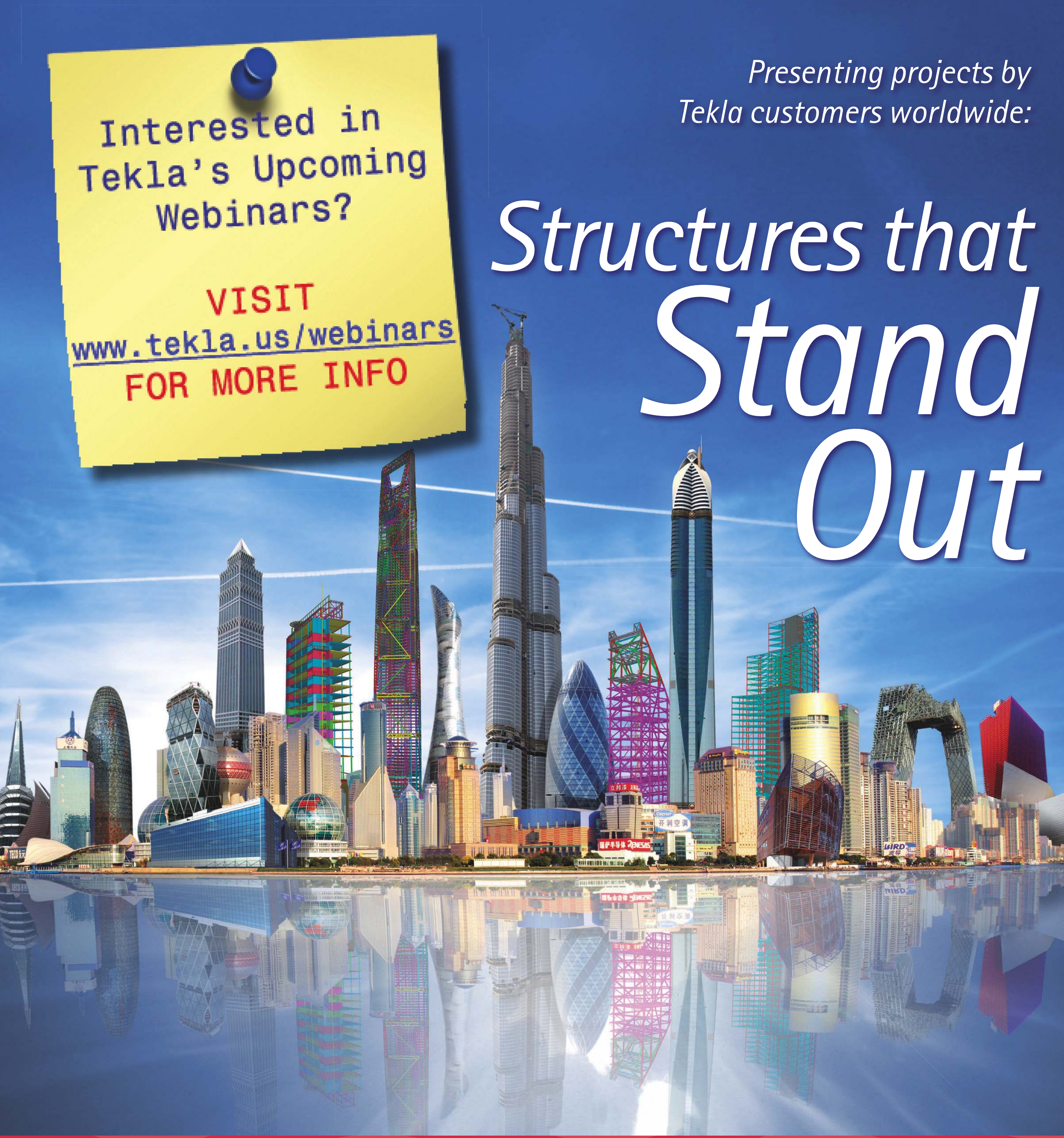
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ON THE COVER: Art Gallery of Alberta in Edmonton, Alberta, Canada. Photo: Randall Stout Architects

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editor's note

THE OTHER MORNING I ASKED MY DAUGHTER IF SHE COULD NAME ANY FABRICATORS. After a moment's thought, she came up with Bob Owen and Terry Peshia. It was a remarkable pairing, since the AISC Board of Directors had voted just a few days earlier to award them the Robert P. Stupp Leadership Award, the Institute's highest honor (along with its sister awards, the J. Lloyd Kimbrough Medal for designers and the Geerhard Haaijer Award for educators).

The Stupp Award is given in special recognition to individuals who have provided unparalleled leadership in the steel construction industry and have had an outstanding impact on advancing the use of structural steel in the construction industry.

(A little truthiness: I really expected my daughter to answer the question exactly as she did thanks to a wonderful trip my family took to the Canadian Institute of Steel Construction Annual Meeting five years ago. At the time, Bob Owen's company owned a plane but was in the process of selling it. Bob, an avid and skilled pilot, was also going to the meeting and offered me a "ride." I politely declined, since I was bringing my whole family, but he said to bring them all—"I love kids" was his response. He was also giving Terry Peshia and his lovely wife Connie a ride. Terry spent much of the flight talking with my daughter and at one point pulled out a Chinese Checkers board and taught her to play. Upon our return to Chicago, Terry gave the set to my daughter. She attempted to decline, but he explained that he had only bought the set to play with her on the flight. It's just the type of people Bob and Terry are.)

Of course, Bob and Terry are not receiving this important honor just because my daughter likes them. Each have brought an important focus to their involvement with AISC.

Bob will always be thought of for his international involvement. He pushed us to reach out and learn from our counterparts in the United Kingdom and other countries. He made us think about the metric system. And he reminded us that others may do things not just differently from us, but also better. Of course Bob wasn't just focused internationally. He served on numerous committees (and also served a term as Chair of AISC's Board of Directors) and was always willing to devote time to advance the domestic structural steel industry. Just as importantly, he allowed his staff to devote valuable time and resources as well. He is well aware that AISC's success depends on the commitment of its many invaluable volunteers and he has always supported and aided that effort.

Terry also brought something unique to the table. For many years, he has been the go-between in AISC's relationship with the erection community—and that expertise proved essential in the development and advancement of the Erector Certification program. Terry also is in the unique position of being located in relatively close proximity to AISC's headquarters. So when we need a fabricator on short notice, or when we need to show new staff a fabrication shop, he's always been available. He too has served on numerous committees, including being very active in AISC's University Relations efforts (and, as with Bob, served a term as Chair of AISC's Board of Directors). An early supporter of the steel sculpture program, Terry has donated more than one sculpture and usually has one ready to go for any university in need (I think I noticed one in front of his shop the other day when I was driving my daughter to a dance competition in Aurora, Ill., Garbe Iron's hometown).

But for all their individual accomplishments, what I most appreciate about both Bob and Terry is their unofficial status as wise men. Every organization needs these elder statesmen who bring perspective and wisdom to a discussion. I always enjoy the spirited debates at board meetings, and I know when the discussion becomes too emotionally charged either Bob or Terry will step in and present a reasonable compromise.

The Stupp Awards will be presented at the AISC Annual Meeting on September 24-25. This meeting, which is open to all members of the Institute, typically attracts 200 leading fabricators. Unlike the Steel Conference, which offers technical programs and a large exhibit hall, the Annual Meeting focuses on business issues and allows for networking in a more intimate setting. (For more information, visit **www.aisc.org/annualmeeting**.)

I hope to see many of you there to help AISC and the steel industry honor Bob and Terry.

SCOTT MELNICK
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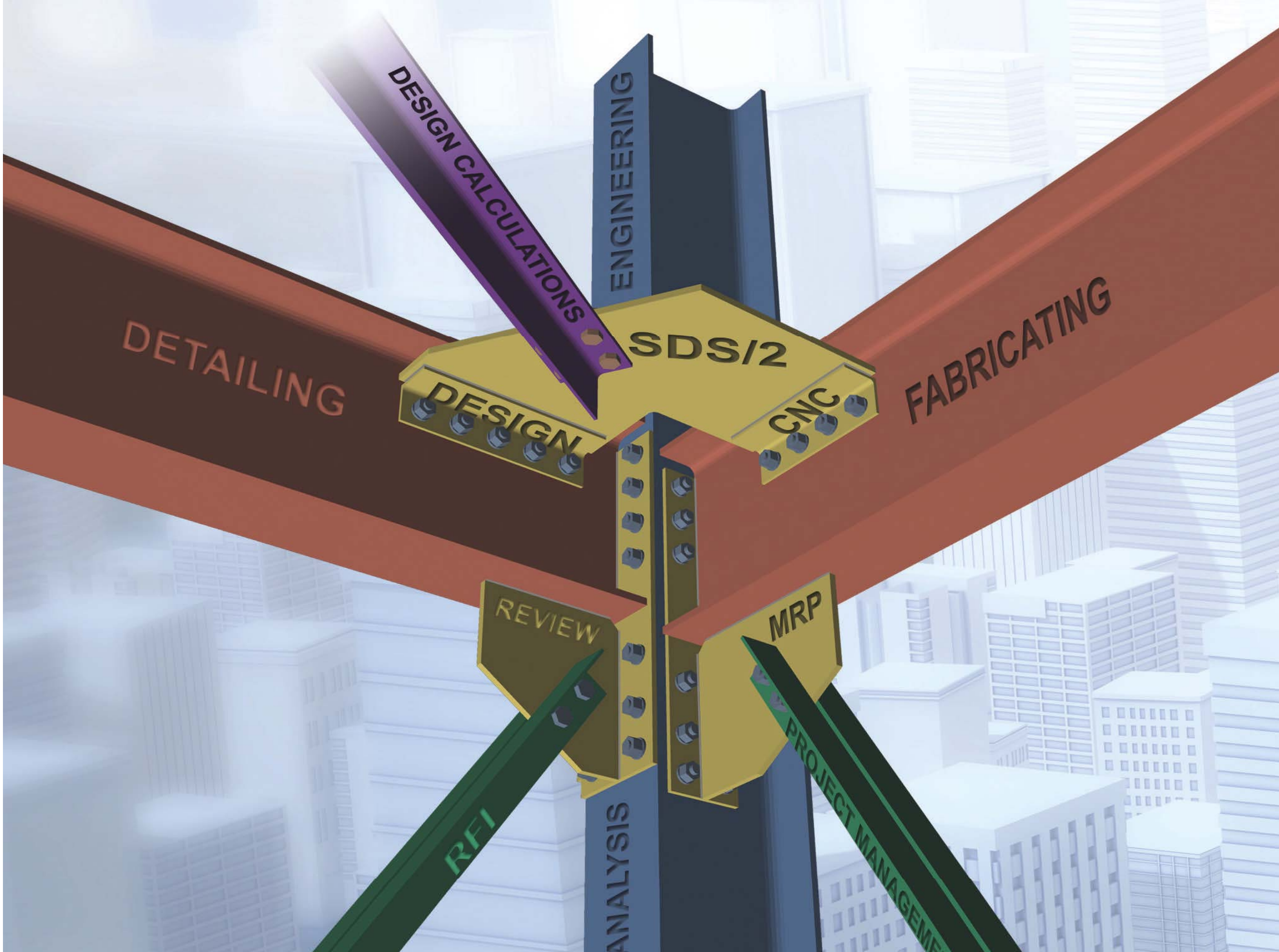

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IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to solutions@aisc.org.

Width-Thickness Limits for S-Shape with Cap-Channel

I am designing a monorail beam, which is an S-shape with a cap channel. I'm having trouble determining the limiting width-thickness ratios for strong-axis bending per Table B4.1 of the AISC 13th Edition *Manual*. For strong-axis bending I am checking three components:

- **channel web between two fillet welds per Table B4.1 Case 12**
- **channel web between channel top flange and fillet weld as stiffened elements per Table B4.1 Case 12**
- **S-shape beam flange as unstiffened element per Table B4.1 Case 2**

Am I doing this correctly?

From your description, it is assumed that your beam has the top flange in compression and that the cap channel is connected to the top flange.

For the first two checks, you are correct that the channel web between the two fillets and between the fillet and channel flange should be checked using Table B4.1 Case 12. This situation is similar to "flange cover plates between lines of fasteners or welds."

For the last check, it is conservative to check the S-shape flanges as unstiffened elements, per Case 2. However, it is justified to consider the S-shape flange to be a stiffened element and check it per Case 12 with "*b*" equal to half the flange width.

There is another element to check: the S-shape web, although it will be compact for standard North American S-shapes with F_y not exceeding 65 ksi. Case 11 is the correct case for the S-shape web, because the presence of the cap channel moves the elastic and plastic neutral axes toward the top of the section as shown in the figure for Table B4.1 Case 11. Please note that if the crane beam is subjected to axial "tractive" forces, uniform compression width-to-thickness ratios should be checked as well.

Brad Davis, Ph.D., S.E.

Column Buckling

I am reviewing an existing built-up column. The section is singly symmetric (symmetrical about the weak axis). The column is subject to combined axial force and flexure about the strong axis. Does the web element for uniform loading fall under Table B4.1, Case 14 of the 2005 AISC *Specification*? While checking the limit states of flexural-torsional and torsional buckling, I am using Equation E4-5 for singly symmetric members. Is this the correct equation when the axis of symmetry is the weak axis?

Yes, either Case 10 or Case 14 in Table B4.1 works for this case. I am assuming that the shape is not tapered.

When checking for axial strength, either flexural bucking (E3) or flexural torsional buckling (E4) can control the design, depending on the bracing details. In those cases, buckling about the weak axis typically controls. If the web is slender as per Table B4.1, the provisions of Section E7 must be applied.

After checking the flexural strength about the strong axis as per Chapter F, the interaction equations in Chapter H then can be used to determine the strength of the member for the combined effects.

Amanuel Gebremeskel, P.E.

Countersunk Bolts

I am trying to find the preferred material specification for countersunk high-strength bolts. Building codes are virtually silent on the subject of countersunk bolts for structural applications, yet there are occasions where, because of interference, a regular hex-head A325 or A490 bolt will not work and a countersunk bolt is needed. Is this addressed anywhere in the AISC *Steel Construction Manual*?

The AISC *Specification* does not address the use of countersunk bolts. These are ASTM A307 (or similar soft material) bolts and used only in bearing connections. These are not generally used as primary structural connections. There is a short discussion in Part 7 of the 13th Edition AISC *Steel Construction Manual* pertaining to checking the available bearing strength at such bolt holes.

Kurt Gustafson, S.E., P.E.

Prequalified and Qualified High-Seismic Moment Connections

Table 2-2 of FEMA 350 allows bolted flange plate (BFP) moment connections as prequalified moment connections for OMF and SMF in high-seismic applications. ANSI/AISC 358 makes no mention of this type of connection. Is the use of BFP moment connections still permissible in high-seismic applications?

The AISC *Seismic Provisions* (ANSI/AISC 341-05) does not limit the special moment frame connection types to those shown in ANSI/AISC 358-05. As covered in Section 9.2b of AISC 341-05, using ANSI/AISC 358-05 is one of three methods permitted to provide conformance demonstration. The three options are: use of a connection that is prequalified, like those in ANSI/AISC 358; use of a connection that is qualified based upon available test results; and use of a connection that is qualified based upon project-specific testing.

Although not all FEMA 350 connections have yet been adopted into AISC 358, more types of moment connections are being added, as the necessary testing and review is performed. It often is possible to use the testing behind the connections that are included in FEMA 350 to justify their use. The Commentary to Section 9.2b of the AISC *Seismic Provisions* discusses the published testing, such as that conducted as part of the SAC project, and reported in FEMA 350 and 355 may be used to satisfy this provision.

Kurt Gustafson, S.E., P.E.

HSS Seismic Connections

Prequalified seismic moment connections only include W-shape beams. Can HSS beams be used for IMF? How can the seismic requirements for this type of connection be met?

The prequalified connections do not provide for the use of HSS beams. In order for these to be used, the connections must be qualified in accordance with Appendix S of the AISC *Seismic Provisions*. Alternatively, an OMF can be used.

Larry S. Muir, P.E.

steel interchange

Design Using the 2005 *Specification*

I have been using the ASD 9th Edition *Manual*. I am trying to learn how to use the 13th Edition. I am having a hard time finding the allowable stresses for different members, such as tension members, compression members, and members in flexure just to name a few. Is the bending stress for flexural members still 0.66F_y and 0.6F_y, depending on my unbraced length? Where are these located?

The 2005 AISC *Specification* is based on a strength format rather than stress, but strength equations can always be formatted as stress by dividing out the appropriate section property. While many of the limit states are similar to those used in the old ASD specifications, there may be slight variations. You will find the nominal limit state capacities for tension in Chapter D, for compression in Chapter E, for flexure in Chapter F, and so on.

For flexure, a compact shape is handled somewhat differently in the 2005 *Specification* than in the older ASD specifications. It is now permitted to use the actual shape factor for the section—instead of the lower bound shape factor of 1.1 for W-shapes, which was incorporated in the older ASD specification provisions:

$M_n = F_y Z_x$

Using ASD: $\Omega = 1.67$, therefore $M_n / \Omega = 0.60 F_y Z_x$

The shape factor = Z_x / S_x, which ranges from 1.1 to 1.3 for W-shapes. If one uses the shape factor = 1.1 as assumed in the old ASD specifications:

$M_n / \Omega = 0.60 F_y Z_x = 0.60 F_y (1.1 S_x) = 0.66 F_y S_x$

Most other cases are more straightforward in that they do not require mathematical manipulation to compare the new to old. For example, tension yielding has $F_y / \Omega = 0.6 F_y$.

Do these look familiar?

Kurt Gustafson, S.E., P.E.

Rivet Head/Shaft Diameter Relationship

We are doing a project involving inspections of truss bridges, most of which were built in the early 1900s and are connected together with gusset plates attached with rivets. We have not been able to locate any literature relating the diameter of the head of the rivets to the shaft diameter. Is there any reference material that denotes the relationship of the diameter of the head to the actual shaft diameter?

There was a general relationship for driven rivet heads as a function of the diameter of the rivet published in the Fifth Edition AISC *Steel Construction Manual:*

Diameter of Head = 1.5 × Diameter of Rivet + ⅛ in.

There was also a general published relationship for manufactured heads as a function of the diameter of the rivet of:

Diameter of Head = 1.5 × Diameter of Rivet + $\frac{1}{32}$ in.

I am not sure if this applied to both hand- and power-driven rivets, but I would surmise that it was fairly standard. The manufactured head equation was published in the manuals of the 1950s but not in earlier ones of the 1920s. Therefore, there could have been a change in this standardization depending on the era of the rivets. You may want to sample a few rivets to see if this relationship is accurate for the specific project.

Kurt Gustafson, S.E., P.E.

Single-Plate Shear Connections to HSS

Is there, or will there be, an update of the 1997 *Hollow Structural Sections Connections Manual*? I am particularly interested in finding information pertaining to single-plate shear connections to HSS.

There are no plans to develop another HSS connections manual, but there is an HSS connections design guide that is soon to be printed (see SteelWise on page 53 for more on this design guide). The HSS Connections Manual was based on the stand-alone AISC *LRFD HSS Specification*. Much of that information has now been included in the 2005 AISC *Specification*, with Chapter K covering HSS connections. See the User Note to Equation (K1-10) in the 2005 *Specification* for discussion of the yielding (punching) check on the wall of the HSS tube. Also, there is information on shear connections to HSS beginning on page 10-156 in the 13th Edition AISC *Steel Construction Manual*.

Kurt Gustafson, S.E., P.E.

Shear Lag

Could you please explain the term "shear lag?"

Shear lag is the phenomenon discussed in Section D3.3 of the 2005 AISC *Specification* (a free download at **www.aisc.org/2005spec**). The bottom figure on page 16.1-252 of the *Specification* Commentary provides a good example. Using that example, away from the connection, the stress is uniform across the entire angle. However, because the horizontal leg is bolted to the support, and the vertical leg is not, the total load must transition to being only in the horizontal leg and transferred to the support along the length of the connection. With enough distance to accomplish this transition, the tension rupture strength will not interrupt this transition. But the shorter the connection is, the more abrupt the transition is. The effective net area concept is how this phenomenon is addressed, and this is accounted for by using the *U*-factor of Section D3.3.

Brad Davis, Ph.D., S.E.

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

Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Larry Muir and Brad Davis are part-time consultants to AISC.

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

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

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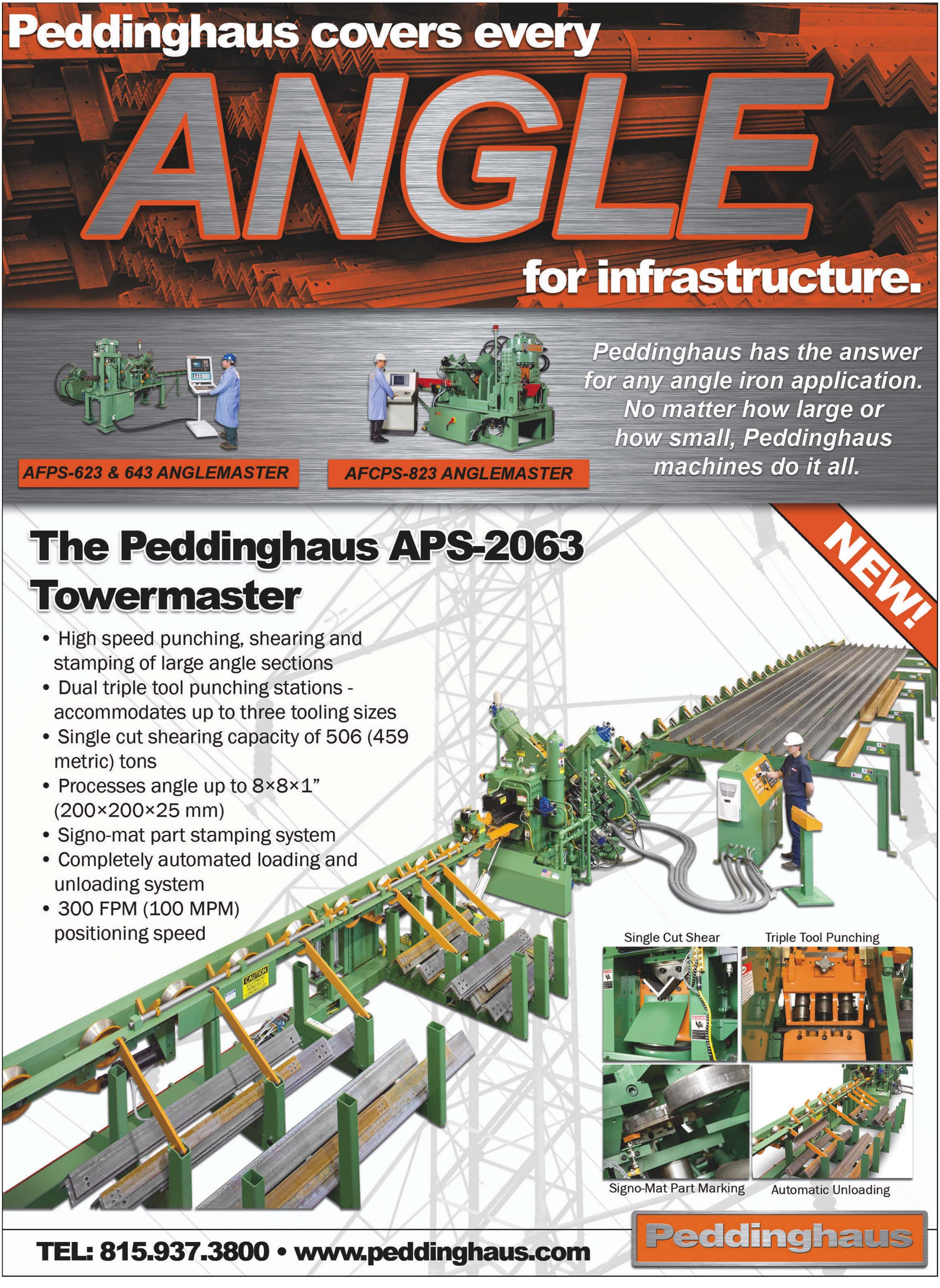
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steel quiz

LOOKING FOR A CHALLENGE? *Modern Steel Construction*'s monthly Steel Quiz tests your knowledge of steel design and construction. Most answers can be found in the 2005 *Specification for Structural Steel Buildings*, available as a free download from AISC's web site, **www.aisc.org/2005spec**. Where appropriate, other industry standards are also referenced.

1 True/False: In the seismic design requirements in ASCE 7, a response spectrum is developed based upon assumed dynamic properties of a representative building.

2 Into which of the following ranges of period do most earthquake accelerations fall?
(a) 0.2 – 1.0 seconds
(b) 0.5 – 1.2 seconds
(c) 1 – 2.2 seconds
(d) None of the above

3 Which buildings are more likely to have a fundamental period that is resonant with most earthquakes?

4 True/False: A seismic load resisting system designated as ordinary always has less strength than one designated as special.

5 What is the response modification coefficient, *R*, in ASCE 7-05, for a steel special moment frame?
(a) 3.5
(b) 4.5
(c) 8
(d) 10

6 Is it correct to say that the overstrength factor, Ω_o, is used when designing elements that are intended to remain nominally elastic in the design earthquake?

7 When designing a moment connection for a special moment frame, which of the following approaches can be used?
(a) A prequalified connection can be selected from AISC 358
(b) A connection can be qualified based upon existing tests available in the literature per AISC 341 Appendix S
(c) A connection can be qualified based upon project-specific tests per AISC 341 Appendix S
(d) All of the above

8 How many moment connections are prequalified in AISC 358?
(a) 1
(b) 3
(c) 6
(d) 9

9 Does AISC 358 permit the use of a concrete slab in special moment frames with prequalified bolted extended end-plate moment connections?

10 True/False: According to ASCE 7-05, *R* = 3 systems are permitted in low-seismic applications (Seismic Design Categories A, B, and C), and the requirements of the AISC *Seismic Provisions* need not be applied.

TURN TO PAGE 14 FOR ANSWERS

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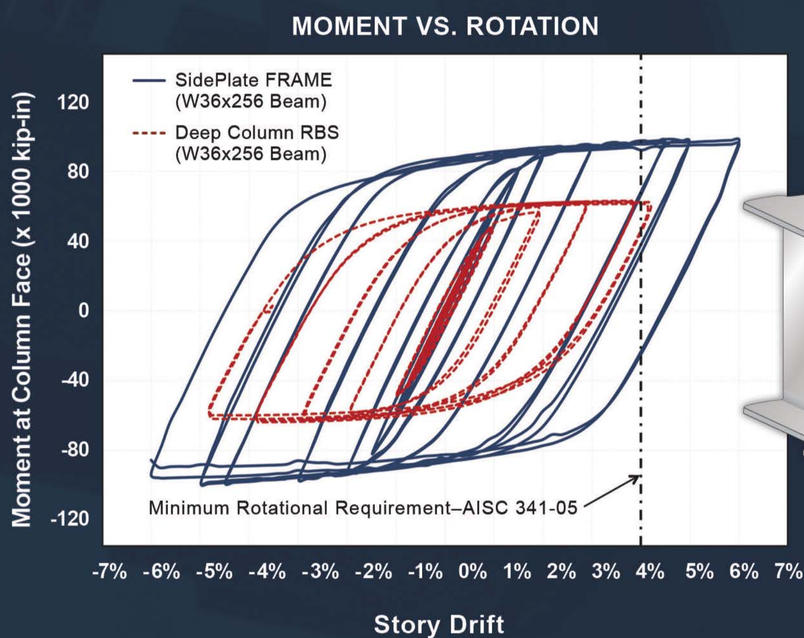

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steel quiz

ANSWERS

1 False. The seismic design requirements in ASCE 7 are based upon the use of a response spectrum, which represents the characteristics of the design earthquake. This response spectrum is used for the design of buildings with varying dynamic properties.

2 (a) Most earthquake accelerations fall in a natural period range of 0.2 to 1.0 second. See Part 1 of the AISC *Seismic Design Manual* for more information on this.

3 (b) A typical two-story building has a fundamental period of vibration of about 0.2 seconds, while a typical 10-story building has a period of about 1.0 second. Therefore, buildings in this range are more likely to be resonant.

4 False. A seismic load resisting system designated as ordinary is detailed to meet ductility and redundancy requirements that are not as stringent as those of a similar system classified as special. Strength-wise, the comparison depends upon the member sizes and connections that are used, and it is not an absolute as to which system will have a higher strength.

5 (c) The response modification coefficient, R, in Table 12.2-1 of ASCE 7-05, for steel special moment frames is 8. Sections 14.1 and 12.2.5.5 of ASCE 7-05 provide more information on this.

6 Yes. Seismic load resisting systems rely on dissipation of earthquake energy through some varying level of inelastic behavior in specifically chosen elements in the structure. That is, specific components in each system are designated for such ductile behavior in order to protect the others that are not. The overstrength factor, Ω_o, is used to amplify the seismic force for elements that must be designed to remain nominally elastic.

7 (d) Subject to the approval of the authority having jurisdiction, an engineer may use any moment connection in an SMF that satisfies the testing requirements of AISC 341 Appendix S. This can be done with existing testing or new testing. Often, however, one of the connections that has been prequalified in AISC 358 by a panel (CPRP) according to Appendix P can be used. This prequalification means that the available testing is already known to meet the requirements in Appendix S.

8 (c), though you can claim partial credit if you said (b). Until recently there were three prequalified moment connections in AISC 358: two bolted extended end-plate moment connections and one reduced beam section moment connection. Three additional moment connections recently have been added by supplement: a welded unreinforced flange moment connection with a welded web, a bolted flange plate moment connection, and a proprietary connection called the Kaiser bolted bracket connection. This supplement will be available at **www.aisc.org** shortly.

9 Yes, though don't feel bad if you said no. Until recently, Section 6.2 of AISC 358 prohibited the use of SMF systems with concrete slabs when using prequalified extended end-plate moment connections. However, the same supplement that added additional connection options also relaxed this limitation, where a 1-in. gap is provided between the concrete and column faces. This is commonly accomplished with rigid insulation to maintain the required compressible gap.

10 True. R = 3 systems allow an approach for low-seismic applications in which seismic loads are treated in a manner similar to wind and gravity loads, without the capacity design or structural fuse requirements found in the AISC *Seismic Provisions*. This approach depends upon the normal ductility, redundancy, and robustness present in steel framing.

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

Steel Solutions Center

AWARDS

T.R. Higgins Lectureship Nominations Due August 1

Each year the T.R. Higgins Lectureship Award recognizes an outstanding lecturer and author whose technical paper or papers, published during the eligibility period, are considered an outstanding contribution to the engineering literature on fabricated structural steel.

AISC encourages everyone involved with steel construction to submit nominations by August 1. Nominations should include the following information:

- Name and affiliation of person nominated for Lectureship
- Title of paper or papers to be named in nomination with publication citation
- In case of multiple authors, identify the principal author
- Reasons for nomination

A copy of the paper, as well as any published discussion, must accompany the nomination. The author must be a permanent resident of the United States and available to fulfill the commitments of the award. The paper or papers must have been published in a professional journal within the five-year period from January 1, 2004 to January 1, 2009. The 2010 award winner will give a minimum of six presentations of the lecture on selected occasions during the year.

The award will be made to a nominated individual on the basis of two criteria: (1) his/her reputation as a lecturer and (2) the jury's evaluation of the paper or papers named in the nomination. The papers will be judged for originality, clarity of presentation, contribution to engineering knowledge, future significance, and value to the fabricated structural steel industry.

A framed certificate will be presented to the lecturer at the 2010 NASCC: The Steel Conference in Orlando, Fla. Co-authors of the paper or papers named in the successful nomination will also be recognized at the award presentation. In addition, the winner will receive a $10,000 cash award.

Send your nominations for the T.R. Higgins Lectureship Award to:

T.R. Higgins Award Nomination
c/o Janet T. Cummins
Engineering and Research Coordinator
AISC
One East Wacker Drive, Suite 700
Chicago, Ill. 60601

Donald W. White (right), this year's T.R. Higgins Award winner, with AISC vice president and chief structural engineer, Charlie Carter.

ASSOCIATIONS

Galvanizing Essay Contest Winners

The American Galvanizers Association (AGA) has announced the winners of its 2009 *Galvanize the Future: An Edgar K. Schutz Scholarship* essay contest. Three students were selected from more than 40 applicants based in architecture, civil engineering, or other engineering programs in North America. The winners are:

- First Place: Anna Bruce, Texas Tech University, Lubbock, Texas, for her essay "The Galvanized Community"
- Second Place: Stephanie Grannetino, Philadelphia University, Philadelphia, for her essay "What has the Steel Construction Industry Seeing Green?"
- Third Place: Jenny Joe, Columbia University, New York, N.Y. (beginning this fall) for creating a course outline to teach students about corrosion management and the role of hot-dip galvanized steel

For more information on next year's program, visit the AGA's scholarship page at **www.galvanizeit.org/scholar**.

NASSPA Expands its Membership

The North American Steel Sheet Piling Association (NASSPA) has expanded its membership to companies and individuals that are involved with accessories, equipment, design, and specification of steel sheet piling (SSP) systems.

In keeping with the mission of NASSPA to provide a forum where the users of steel sheet piling technology can interact and discuss best practices, the Board of Trustees approved two new membership categories into NASSPA. Associate membership is offered to firms engaged in the manufacture, distribution, and/or supply of equipment, material, accessories, or services to the hot-rolled steel sheet piling industry in North America. Technical affiliate membership is offered to firms engaged with the design or in teaching the art and science of design and installation of hot-rolled steel sheet piling in North America.

STANDARDS

Revision Results in Requirement Reversion

AISC distributes *Selected ASTM Standards for Structural Steel Fabrication*, which includes verbatim copies of ASTM material standards. The most recent AISC document, dated 2008, contains the ASTM standards available at the time, generally dated 2007.

The 2007 version of ASTM A709 revised the Charpy V notch requirements for Grade HPS50WT, increasing the minimum energy value and decreasing the test temperature to be the same as for Grade HPS70WT—i.e., 25 ft-lbs at -10 °F. It was subsequently determined that this change was unnecessary, and the requirement was revised in the 2008 version of ASTM A709, reverting back to 20 ft-lbs at +10 °F for Grade HPS50WT.

If a bridge has been designed using A709-07 HPS50WT, it is recommended that the engineer and DOT consider and permit the use of A709-09 for the production of this material.

People and Firms

- Minneapolis-based law firm **Fredrikson and Byron** recently launched a Construction Group to help clients navigate the current challenges of the construction industry.
- Plate technology manufacturer **W.A. Whitney** has unveiled its redesigned website at **www.wawhitney.com**.
- **Dennis Jang, P.E., S.E.**, senior vice president and district director with **T.Y. Lin International**'s San Francisco office, was recently selected as the 2009 recipient of the National Taiwan University Alumni of the Class of '78 Award.
- **Robert W. Santillo** was elected president of The Association of Union Constructors, and **James Mirgliotta** was awarded the Spirit of Union Construction Award, both at the **TAUC Leadership Conference** in May.
- **Peddinghaus Corporation** recently named **James Magnuson** as vice president of research and development. Also at Peddinghaus, **Jim Sutliffe** will assume full departmental responsibilities as vice president of engineering.
- Engineered connector manufacturer **MiTek, Inc**. has acquired **SidePlate Systems Inc.**, a provider of proprietary high-performance steel-frame connection technologies.

SEMINARS

Jack Miller to Retire after Farewell Tour

Construction industry speaker Jack Miller has announced that he is retiring from his popular construction seminar series. Miller, who has lectured for 42 years, will conduct a final tour, offering his three seminars—Marketing/Sales, Design/Build/Lease/Financing, and TQM: Total Quality Management—in Denver, Nashville, Chicago, and Orlando before hanging up his lectern.

A civil engineer by training, Miller has worked in the industry for more than half a century—as a construction laborer, foreman, field engineer, sales engineer, sales manager, director of marketing, subcontractor, erector, fabricator, GC, and owner of commercial and industrial real estate.

For additional information on the Jack Miller Seminars, visit **www.jackmiller.com**. Below is the final tour schedule:

Denver	TQM	Aug. 17-18
	Marketing/Sales	Aug. 20-21
	Design/Build/Lease/Financing	Aug. 24-25
Nashville	Marketing/Sales	Oct. 5-6
	Design/Build/Lease/Financing	Oct. 8-9
	TQM	Oct. 12-13
Chicago	Marketing/Sales	Nov. 9-10
	Design/Build/Lease/Financing	Nov. 12-13
	TQM	Nov. 16-17
Orlando	Marketing/Sales	Jan. 25-26
	Design/Build/Lease/Financing	Jan. 28-29
	TQM	Feb. 1-2

The Debate Clearly Continues

With reference to Charlie Carter's article "Connection Design Responsibility: Is the Debate Over?" (5/09, available at **www.modernsteel.com**), any changes to the code of standard practice are welcome, especially in the grey area of connection design. However, this doesn't seem to go far enough. As a fabricator, we have to bid projects from the plans and specifications. If the design is complete and all connections are shown, our estimates are accurate and we can price competitively. If the connections are not shown (options 2 and 3 in the article), we have no way of knowing what they may be unless we have an engineer run test calculations before we bid. Very few fabricators have the staff, time, or money to do this.

This is something I discussed last year with Mr. Carter, who said AISC was aware of the problem but had no real solution. The connection types on a particular project we discussed were shown on the drawings with notes such as "web doublers and stiffener plates (if required)." At the bid stage, we added something to the price but could not have anticipated the extent. Almost every column required doubler plates (usually 1 in. thick on both sides) and stiffeners plates and every connection. The amount of additional material required was of little significance compared to the huge amount of welding and shop time. Every connection had to be engineered (no "one-size-fits-all"), which added to the costs and delays. Very few general contractors understand these issues and often view them as excuses for delays. So because the engineer couldn't be bothered to do his job properly, the fabricator loses money, delays are caused, and relationships are strained. It is my opinion that some engineers show vague information, so the job is under-bid.

The problem with methods 2 and 3 is that the fabricator can't accurately bid the project without connections, but also can't afford to engineer them at the bid stage. As usual in this business, the lowest bidder gets the job. He probably missed something, which is why he's the lowest bidder.

Peter Officer
Tamburri Associates, Inc.

Stepping Back in Steel Time

I appreciated Scott Melnick's May editorial [on factory tours]. I just returned from visiting my sister in Birmingham, Ala., where I had the opportunity to visit Sloss Furnaces, one of the last "old" blast furnaces to operate (it shut down in the early 70s). It has been preserved and interpreted as a museum by the city. Talk about whirring devices (all silent now)! Fabulous equipment dating from the mid-1920s with some from 1900. The site consists of two 400-ton blast furnaces and some 40 other buildings.

Sloss Furnaces is now a National Historic Landmark, and admission is free. The best part is that there are very few areas you can't go into; most of it is open to the public (watch your step). I spent two hours walking around and could have spent another four. More info can be found at **www.slossfurnaces.com**.

Warren Stewart, S.E.
Seal Beach, Calif.

Full House

STORY AND PHOTOS BY GEOFF WEISENBERGER

Future engineers pack UNLV's Thomas and Mack Center for the annual National Student Steel Bridge Competition.

LAS VEGAS IS TRADITIONALLY PACKED on Memorial Day weekend. The usual four-hour drive from Los Angeles can take more than twice that long, the airport is mobbed, and the taxi lines are a major hassle—although quite entertaining as well.

Away from the chaos and revelry of the Strip, a different type of intensity was on display this past Memorial Day weekend in Vegas—and it was focused on something a bit more productive than the activities typically associated with Sin City. While everyone else was in town on vacation, college students—around 550 of them—were doing something constructive. More specifically, they were building steel bridges.

The occasion for such prolific, focused activity in such a leisure-oriented locale was the National Student Steel Bridge Competition (NSSBC), which took place at the University of Nevada, Las Vegas' Thomas and Mack Center. "We truly came into this determined to be like no other, and we exceeded all expectations," said Vik Sedhev, UNLV engineering student and 2009 NSSBC student director.

In all, 46 teams of university-level civil engineering students from the U.S. and Canada assembled, displayed, and tested their creations in the annual contest. The teams are narrowed down from nearly 200 teams that participate in 18 conference competitions around the country.

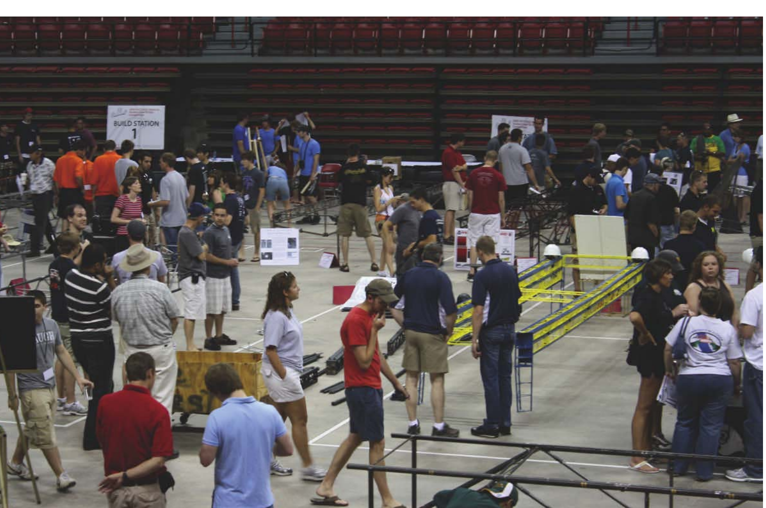
"At this level, they really know what they're doing," said John Parucki, who has been the head judge of the competition for the past 15 years. "We get the cream of the crop every year, and they get to compete against each other. You can't get any more real-world than this."

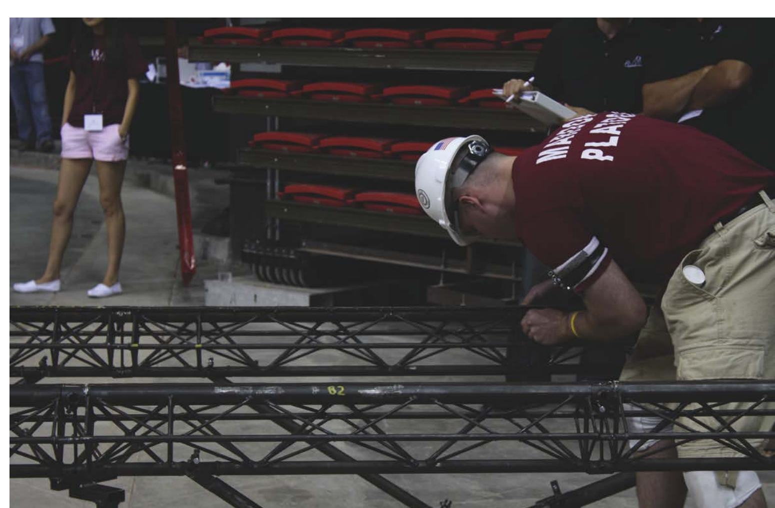
NSSBC is a joint effort between AISC and the American Society of Civil Engineers. It started as a regional competition in the upper Midwest in the mid-1980s and grew into a national competition by 1992. Generally, the top three teams from each conference competition make it to the national level. And improvement between the two levels is the norm more than the exception.

"Once the top teams get back from regionals, they really get to work to improve their scores," explained Scott D. Schiff, professor of civil engineering and director of the Wind and Structural Engineering Research Facility at Clemson University. "Most teams can cut 10 to 25% off of their construction time by improving their connections, developing new assembly schemes, and just practicing for countless hours so that every movement is memorized."

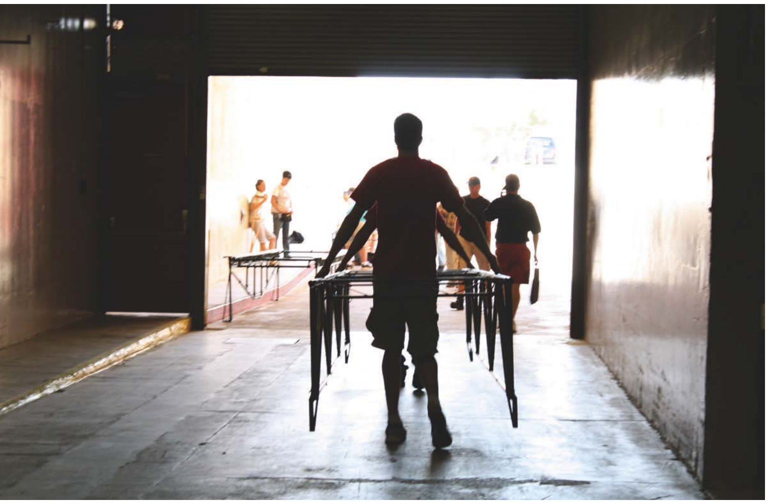
Three teams at this year's competition built their bridges in under four minutes, and several others weren't too far behind; the majority of the field finished in under 15 minutes.

But construction speed is only one of six categories in which the bridges are judged. Stiffness, lightness, economy, display, and efficiency are also assessed, and the best combined score across all six categories wins. Every year, the design parameters change slightly to meet the Problem Statement, which this year called for teams to create a scale model of an attractive and functional replacement for a century-old highway bridge spanning a scenic river. In past competitions, above-deck steelwork was part of the program, but this year everything had to remain below the deck. Also, this year's bridges were required to be 20 ft long and capable of carrying 2,500 lb.

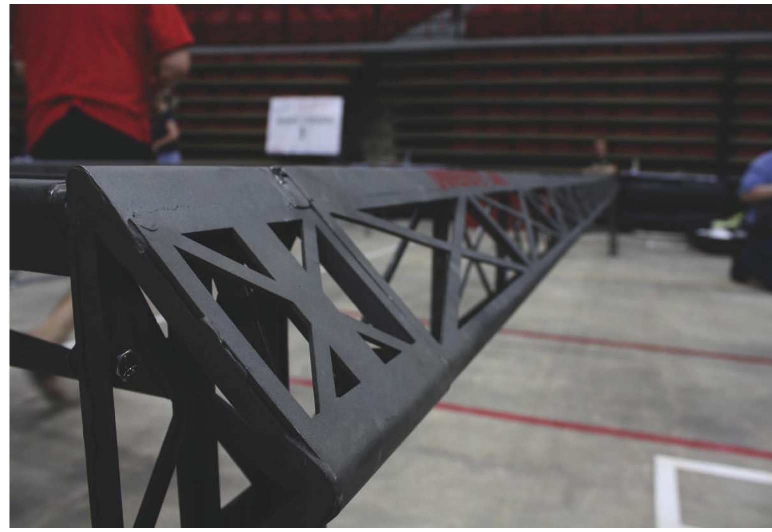




The Lafayette College team wore their fasteners on their sleeves—with a little help from magnets.



The bridge has left the building (top). Kansas State's team performing load testing (bottom).



Gray and flat with X-bracing was just one of many bridge styles (top). The University of California, San Diego's team in action (bottom).

Prep Work

Students design and build the bridges themselves and begin the whole process months in advance. The assembly is practiced over and over until it is perfected; in many cases, teams will assemble their bridges more than 100 times.

"We design our bridge in the fall semester, we fabricate it for one week over winter break, and practice construction in the spring semester," explained Alex Pschorr, a co-chair for the University of Wisconsin–Madison team. "We tried to determine how many times we practiced putting in together, and we counted over 120 construction runs (dress rehearsals)."

In some cases, the design changes at the last minute—before the regional competition and sometimes even between the conference and national competitions. "We actually had our bridge built a month before regionals, then decided to scrap the entire truss and throw it away," noted Eric Gunderson, North Dakota State University's co-captain; NDSU has won the competition five times in the last 10 years. "We designed and fabricated a new truss for regionals in less than two weeks. That bridge got us to Vegas and with a few more minor changes after regionals, we were ready to compete at nationals. It took us two bridges to get it right, but in the end we got what we wanted."

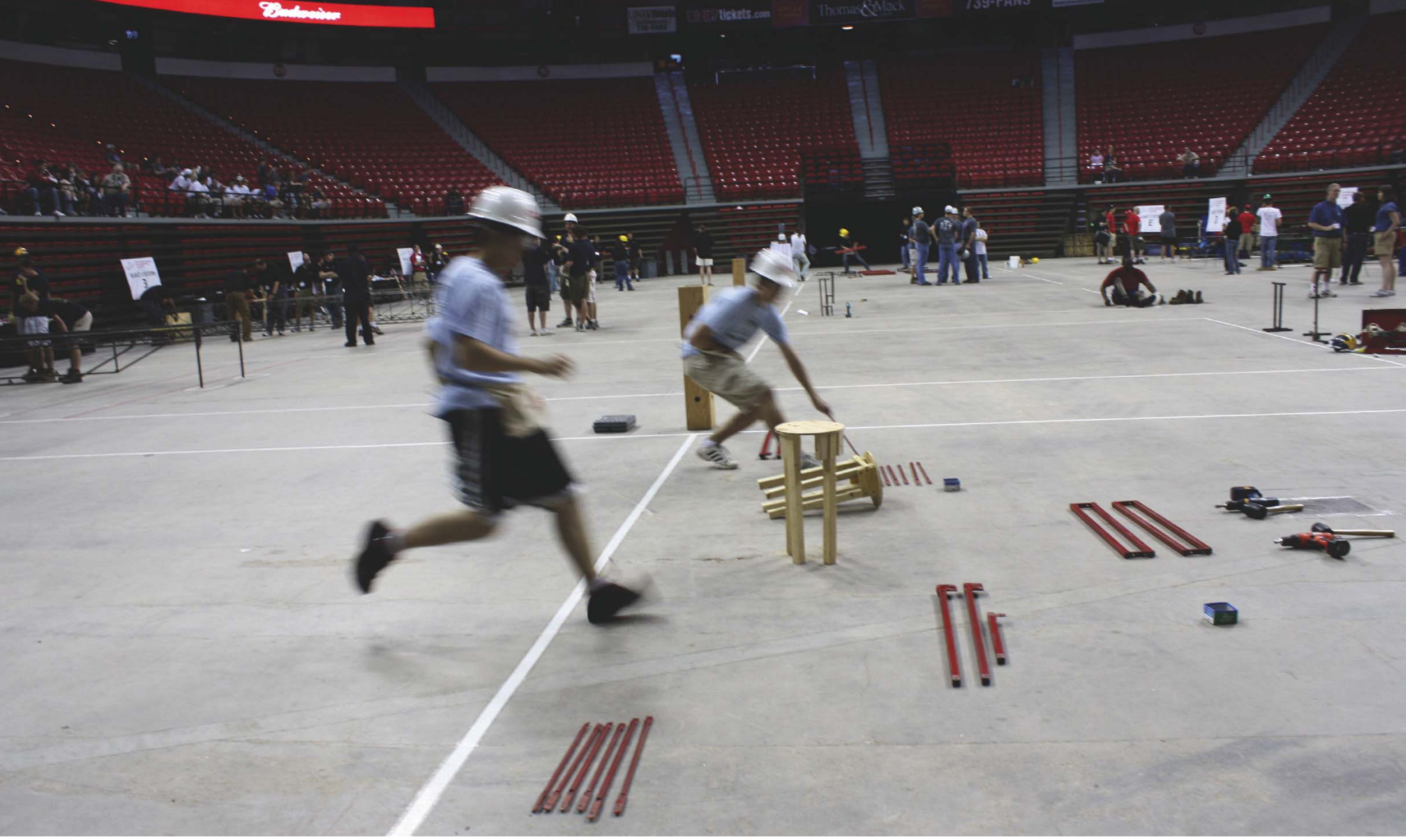
One team, California Polytechnic – San Luis Obispo, put approximately 1,500 hours into their bridge design and construction. "We redesigned the entire bridge after regionals, when we realized the design flaws the bridge had," said Mike Ginther, the team's captain. "The construction team spent the last three days before the competition practicing, going through 15 to 20 run-throughs building the bridge."

In fact, Ginther was so involved with the project that it became inescapable, even in sleep. "Most of my ideas for the bridge came to me while I was sleeping," he said. "The last six months, all my focus was on the bridge."

It's On

The two-day competition began on Friday, which involved the most arbitrary segment, the display judging. The Rules Committee—made up of 10 volunteers from the steel industry and academia—made their rounds and decisions on which entries they found most aesthetically pleasing. (So did I.)

Walking amongst the entries was like walking through a museum of bridge design. The sheer variety of colors, styles, and designs was amazing, especially given the parameters to which the teams must adhere. Several bridges were painted; many were decked out in school colors, while the University of Hawaii at Manoa's bridge was metallic purple. Bridges were constructed with a variety of framing types, including joists, trusses, box trusses, HSS, or any combination thereof. Some were Spartan while others were elegant; some were simple while others were complex. And of course, there was flare. The University of California at San Diego's entry sported silver tridents, and Kansas State University's name plate (every bridge is required to display the school's name) featured the school's well-known wildcat logo.



Teams raced against the clock in the build portion of the competition.

From Museum to Racetrack

While Friday offered an opportunity to look over the bridges at a leisurely pace and observe the students in a somewhat relaxed setting, Saturday was a different story and featured the most exciting part of the competition: the timed construction of the bridges. Whatever preconceived notions I had about a bunch of engineering students building bridges were replaced by what felt more like a swim meet—and the venue, a college basketball arena, only added to the atmosphere. Students raced back and forth between their material staging areas and the bridges in an effort to beat the clock. They yelled encouragement and directions to one another—as did "coaches," from the sidelines. And many teams even had their very own cheering section in the stands, typically comprised of the rest of the team.

Here's how it works: Teams are compiled of 10-20 members, although only four or five get to build. The judges—there are almost 50, many of them local and all involved in the steel industry in some form or fashion—referee all areas of the competition except for the aesthetics portion.

The competition takes place in a designated (by tape) area, the build station; there were five build stations in all, so at any given point, you could watch five teams competing at once. Teams—who must wear safety gear such as hard hats and construction boots throughout the competition—lay out their bridge materials at one end of the build station, the staging area. At the other end of the station is the assembly area. Once the clock starts, the runners (there are one or two) run the members across an open area, one by one, to the assemblers. As the assemblers put the bridge together, the runners go back and forth between the assembly and staging areas until the bridge is complete. The action is much like that of a relay, except instead of handing off the baton, the runners are handing off steel. Each runner has to wait outside of the assembly area until the assemblers finish connecting the previous piece, before handing over the next piece; it can be a waiting game on both ends. The ideal assembly scenario is when a runner hands off his piece and the assembler has it in place and is ready for the next piece right when the runner returns with it, in a continuous fluid process.

Verbal encouragement isn't only motivating, it can also be crucial. Shouts of: "Watch that pier!" "Check the bottom chord bolts!" "Bolt in the water!" and similar guidance can be heard throughout. "In the heat of battle, it's easy to forget things," said Mike Engestrom, a member of the NSSBC Rules Committee and technical marketing director with Nucor-Yamato Steel, one of the event's sponsors.

As this year's competition featured a "river" (also designated by tape), the team members were not allowed to step into it and were penalized if they did so. Fasteners had to be held by the assemblers in a pouch. There was a lengthy discussion over what constituted a "pouch" at the team captains' meeting, which took place the night before. Two teams came up with the idea of taping magnetized strips to their arms in order to have easier access to their fasteners.

When the bridge is complete, the clock stops. This year's fastest time was delivered by State University of New York (SUNY) Canton, which came in at just over three minutes. However, in some ways, the clock doesn't stop with the construction portion. Additional time may be added due to penalties given during the load test, much like a hurdler being penalized for knocking down a hurdle even if he crosses the finish line first. Violations include items such as a nut falling off its bolt during transport to the load testing area or a nut not being fully engaged—or again, stepping or dropping something in the river. Hence, while teams strive for the fastest assembly, they must also account for a *quality* assembly. (Erection time plays a factor into another of the competition's categories, construction economy, which also is determined by the number of builders and the number of temporary piers used.)

Surveying Strength

Following the construction portion, teams put their bridge's strength to the test at the load stations, where lateral and vertical load testing is performed. Safety supports are placed below the bridge, should one happen to collapse. For the lateral test, a load of 75 lb is placed on one side of the bridge and a "sway target" is established on the other side, then a 50-lb lateral pull is applied at the sway target and the sway is measured. Sway must not exceed 1 in., or the bridge does not pass the test.

Vertical load testing begins by having the team members place two decking units near opposite ends of the bridge and adding 100 lb to each of them. From here, 1,150 lb is added to one unit. Two targets are established longitudinally at the center of the decking unit, on either side of the bridge. Downward vertical deflection is measured at both targets. Next, 1,150 lb is placed on the other decking unit. There's only one target at this end. (It too is established longitudinally at the center of the decking unit, but only on one side.) The absolute value of vertical deflection at this target that occurs from when the load is added to the first unit to when it is added to this one, is measured.

Unfortunately, even at the national level, failures can occur. It happened to one of this year's teams when a weld failed during the load testing. Factors such as a bridge's design changing between the conference and national levels can introduce last-minute mistakes that prove costly during the moment of truth. While discouraging in a competition setting, mistakes can be learned from and provide motivation and caution for future competitions and, eventually, the real world. As Parucki put it, "Failures can be 'eureka' moments."

Weighing In

The last step for the bridge is to undergo a weight test. Simply put, the lightest bridge wins this category (although penalties can be assessed based on factors from the other portions of the competition). To weigh the bridges, they are placed on what could be described as a four-part scale—one for each footing.

Weight also plays into the final category, structural efficiency; aggregate deflection from the vertical load test also factors into this category.

Final Results

In the end, the sum is the whole of its parts. Sacrifices in one area might lead to advantages in others. While timing and cost are important, "Being able to construct the design—that's what's most valuable," said NSSBC judging veteran T. Bartlett Quimby, an associate vice provost at the University of Alaska Anchorage.

SUNY College of Technology at Canton, after placing first in two categories last year, won the overall competition this year. NDSU took second, while Lakehead University came in third.

While it's certainly nice to win, the competition is really about preparing future engineers for the real world. According to UC Davis team member Tyler Hickox, "I have learned much from my experience with [the competition] and have incorporated many new ideas into what I will make my senior thesis."

"The competition is an invaluable part of my college career," said Eric Michal, project manager for the University of California – Berkeley team. "Not only are we able to apply the classroom knowledge we learn, but working and managing a group of individuals is greatly beneficial for what is to come in the real world—not to mention an unbelievable and unforgettable experience." MSC

For the full results of the overall competition and the individual categories, visit www.aisc.org/steelbridge. Also, the 2010 NSSBC will be hosted by Purdue University May 28-29 in West Lafayette, Ind. The 2010 rules will be posted at the above link this August.

Northern Lights

BY DERRICK D. ROORDA, P.E., S.E., LEED AP, AND LISA T. MINAKAMI, P.E., S.E., LEED AP

Edmonton pays hommage to its northern locale with a shining new addition to the Art Gallery of Alberta.

Randall Stout Architects

EDMONTON, ALBERTA has for several decades centered on the arts—literally; the Art Gallery of Alberta (AGA) is located in Sir Winston Churchill Square, the city's main civic and public arts square.

However, the museum, a Brutalist concrete structure, has not been taking full aesthetic advantage of its high-profile location. But that's about to change. The new Gallery, which opens next year, includes an addition/renovation component that upgrades the existing below-standard facilities and adds new celebratory public event areas that will bring a new architectural vitality to Edmonton's urban core. The project, 84,000 sq. ft in total, will add 27,000 sq. ft of new public spaces and galleries and will include approximately 24,000 sq. ft of interior exhibition space.

In the new building, designed by Randall Stout Architects, Inc. (RSA), the museum's public spaces are reinvented in a language of sinuous stainless steel surfaces, peeling off of one another, creating opportunities for generous views and natural light within the building. The design was inspired in part by the aurora borealis, the night sky phenomenon that is most readily observed in the northern region that is home to Edmonton and its new Art Gallery.

The overall project is comprised of a renovation of the existing concrete building, a two-story vertical addition above the existing building to contain additional gallery space and administrative office space, and the addition of the atrium space, including the borealis elements.

Steel forms a "snowcone" at the Art Gallery of Alberta.

Randall Stout Architects

The form of the building evokes the northern lights.

Vertical Expansion

Structural steel was the obvious choice for the new vertical addition, as it was imperative to minimize the impact on the existing structure below, minimize loads at the foundation level, and provide a column-free interior within the new gallery. The entire addition, comprising 10,000 sq. ft at each level, is supported by only six columns located on the north and south perimeter of the volume. These columns each thread down through the existing structure and bear on pile caps supported by 40-ton screw piles, which were installed inside the existing basement. One-story-deep trusses span between and cantilever past the columns to provide support for W33 and W36 beams that span across the space. The resulting gallery addition is completely unimpeded with structure in order to maximize curatorial flexibility for both large and small exhibitions.

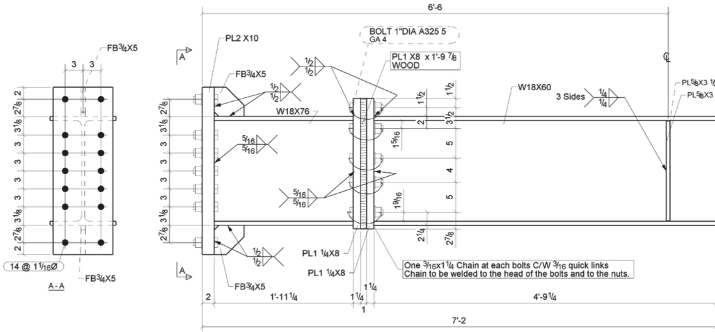
Atrium and Borealis Elements

The angular, transparent glazing planes forming the building envelope of the atrium, and the curving, reflective metal-clad borealis surfaces that penetrate it at multiple locations, work together to animate the building, expose the activities within, and engage people and art at multiple levels on both the interior and exterior. Due to the complex geometries, steel was again the clear choice for structural support, as it could be bent and curved into the required forms.

Wide-flange members as large as W14×370, hollow structural sections up to HSS16×16×0.500, and custom box shapes (18 in. by 18 in.) comprised of 1.5-in. plates create the unique forms. These uncommon shapes were chosen due to strict geometric limitations to define the surfaces of the borealis. Careful consideration was given to connections of these elements, including the transfer of large torsion forces at critical locations.

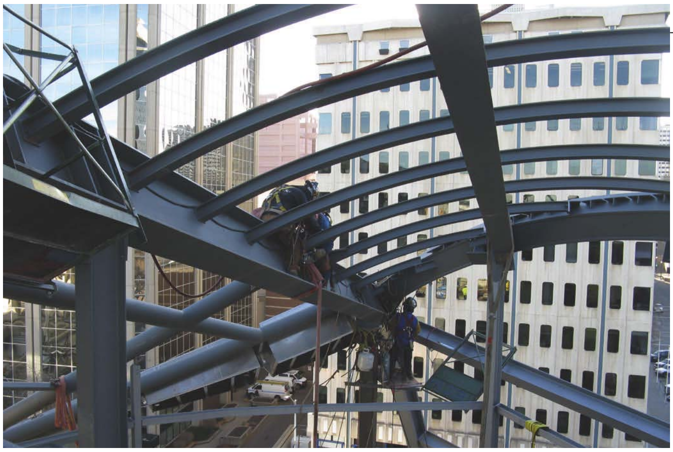
Thermal Breaks

The design team recognized very early that thermal affects



Empire Iron Works

A detail drawing for the thermal break test.



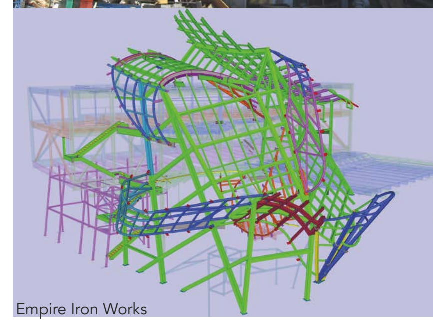
Empire Iron Works

Steel members of all types and sizes bring the unique structure together.


Randall Stout Architects

Wide-flange members as large as W14×370, hollow structural sections up to HSS16×16×0.500, and custom box shapes (18 in. by 18 in.) comprised of 1.5-in. plates create the borealis elements.

Randall Stout Architects


Empire Iron Works

The atrium and borealis element were modeled with Tekla Structures.

would play a significant role in this project. Winters in Edmonton frequently bring lows approaching -50 °F, while interior temperatures are maintained at roughly 70 °F. Of paramount concern were the multiple locations where the curving borealis elements penetrated the building envelope of the atrium.

At such locations, it is crucial to provide a complete thermal break between steel elements that are on opposing sides of the building enclosure. Failure to do so allows the warm interior steel to lose heat to the much colder exterior steel. As the interior steel cools, moisture can begin to condense. On exposed steel shapes such as those in the atrium, this can result in noticeable condensation and even dripping water inside the building. On steel shapes that are enclosed by cladding materials, this moisture can cause problems with coatings, and can lead to mildew and other undesirable non-structural issues.

The laws of physics, however, demand that a cantilevering structure be provided with a backspan in order to remain stable. When the cantilever extends through the building envelope, some amount of continuity must be maintained. The team chose to solve this problem by developing a special moment connection comprised of two steel face plates separated by a single 1-in.-thick block of wood. Wood, in this case oak, was selected as it has a very low coefficient of thermal conductivity, while also having a relatively high compressive strength. All resulting tension forces through this connection are transmitted by high-strength 1-in.-diameter A325 bolts. While the bolts bridge the thermal envelope, it is estimated that this connection prohibits 90% of the heat that would be transferred with a typical moment connection. A testing program conducted at the University of Alberta concluded that the connection is capable of transmitting the required forces so long as the bolts are pretensioned per the standard requirements of CSA-S16-01, the Canadian steel design code.

BIM Workarounds

The curving borealis forms and angular atrium geometries necessitated the use of a 3D modeling program in order to accurately convey the required member work points, both among the design team and ultimately to the construction team. DeSimone and RSA had extensive experience using Rhinoceros (Rhino) NURBS software by McNeel for this purpose (see “A Model Museum,” 09/08, available at **www.modernsteel.com**) and continued to use this tool throughout the design phase.

With the steel design work progressing in Rhino, a detailed Revit model was created of the existing concrete structure. As design development began, the team incorporated the new steel structure into the Revit model in order to take advantage of the software’s parametric capabilities. The then-current version of Revit Structure, Version 3, however, was unable to accommodate the curving shapes of the borealis, as well as the leaning steel shapes of the atrium. It also did not provide the geometric precision that was required by the architect. As the team still wanted to take advantage of Revit where possible, a decision was made to keep all rectilinear elements in Revit, including the new vertical addition, and use Rhino for the geometries that could not be accommodated in Revit. In order to produce a complete set of 2D plans using Revit, 2D cuts were made in Rhino and referenced into the Revit model at proper elevations to insure they would display properly. This workaround was cumbersome, but necessary in order to complete the documentation, due to limitations in the selected BIM platform.

Ultimately, the Revit model was not used during the construction portion of the project. Instead, a structural Rhino model with wire and solid modeling of each steel member was provided to the construction team along with conventional 2D structural drawings.

Construction

The steel detailer, Empire Iron Works (EIW), created shop models using Tekla Structures Version 13.0. In addition to the primary steel, which was modeled by referencing the structural Rhino models, EIW also modeled the borealis cladding panels and support clips, which were obtained from A. Zahner Company (AZCO), the cladding subcontractor. EIW also used reference models from AZCO to ensure that connection plates and other miscellaneous pieces of steel did not "daylight" through the enclosure surfaces. Shop models and shop drawings were then submitted simultaneously to the design team for review.

The borealis members were fabricated using hollow structural sections (HSS) that were rolled to the geometry generated by the Tekla model. Individual pieces were joined by fitting laser cut ends and fillet welding the joints. EIW created tight joints by exporting information from the Tekla model to the laser cutter's software in IGES format to cut 4-axis bevels.

The alignment of the support clips for the architectural panels that formed the nosing of the borealis components was critical to the final shape of the structure. The Tekla model was used to develop jigs that could be rotated to rectilinear coordinates, allowing the shop to use traditional fitting methods to fabricate the borealis truss components. The support clips were positioned in the shop using data points that were downloaded into a Spectra Precision Optical TS415 Total Station directly from the Tekla model. The data was referenced to an origin relative the global model position to ensure that all structural components aligned in the final structure.

Once the structure was erected the support clips were surveyed by using the site benchmark and model coordinates to verify alignment. The site data points were plotted in the Tekla model and a reference model was issued to AZCO to confirm that their cladding panels could be installed properly. Except for minor deflection issues, all support clips were within the specified tolerances. MSC

Lisa Minakami is a senior project engineer and Derrick Roorda is a senior associate principal, both with DeSimone Consulting Engineers and both AISC Professional Members.

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Art Gallery of Alberta, Edmonton, Alberta, Canada

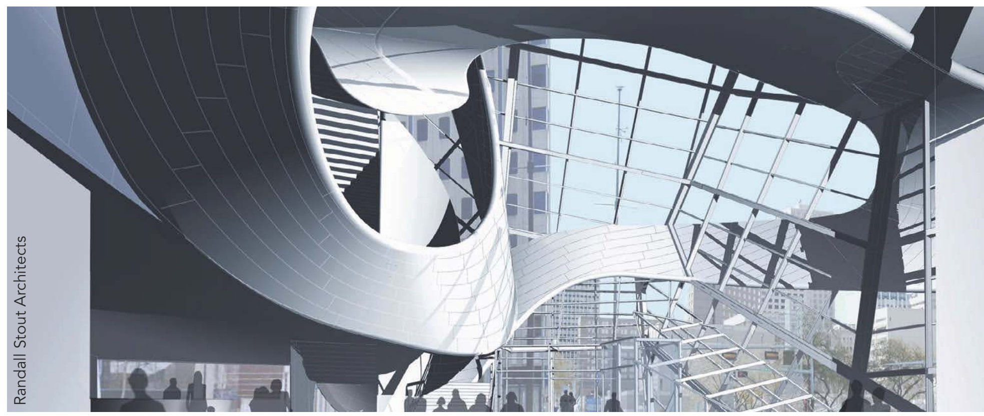
Architect
Randall Stout Architects, Inc., Los Angeles

Structural Engineer
DeSimone Consulting Engineers, San Francisco

Structural Engineering Subconsultant
BPTec-DNW Engineering Ltd., Edmonton

Construction Manager/General Contractor
Ledcor Construction Ltd., Edmonton

Cladding Subcontractor
A. Zahner Company, Kansas City



Randall Stout Architects

The project, 84,000 sq. ft in total, will add 27,000 sq. ft of new public spaces and galleries and will include approximately 24,000 sq. ft of interior exhibition space.

Standing Tall In Madrid

A visually light Vierendeel frame wraps Spain's tallest building.

BY GREG LAKOTA, P.E., S.E.

MAJOR CHANGES CAN, AND USUALLY DO, occur during a building's construction process, but project ownership isn't typically one of them. The new corporate headquarters for Caja Madrid, one of Spain's largest banks, was originally designed to be the corporate headquarters for Repsol YPF, Spain's largest oil company. However, during construction the increase in value of the building was so great that Repsol YPF decided to build their headquarters elsewhere and sell their new building to Caja Madrid.

Located in Madrid on the former training grounds of the Real Madrid football (soccer) team, the tower is part of a new business park, Cuatro Torres, which includes three other new office buildings. At 820 ft tall and nearly completed, Caja Madrid Tower is now the tallest building in Spain, and the new owner will be moving in within the year.

Structural engineer Halvorson and Partners of Chicago collaborated with architect Foster and Partners of London to design an "iconic building," as directed by Repsol YPF, to consolidate the oil company's many smaller offices into one central location. The tower consists of five below-grade parking levels and 34 office floors that are divided into three distinct office blocks of 11, 12, and 11 floors. Below each office block is a two-story space dedicated to mechanical equipment that services the floors above. Within these spaces are two pairs of perpendicular two-story steel trusses, one pair spanning between the two end cores and supporting a transverse pair that align with the steel columns above. This truss system transfers all gravity loads to the reinforced concrete cores at the building ends, which are the only vertical load-carrying elements that extend to the foundation. The result: sufficient gravity load on the cores to counter tensions due to wind overturning as well as a dramatic, column-free lobby.

The typical office floor plate, about 14,530 sq. ft, is framed in structural steel and is unique in that the four corners of the floor cantilever roughly 27½ ft from the core and 23 ft from the nearest exterior column through the use of an aesthetically light

A progressive collapse resistance diagram of the building. Halvorson and Partners steel Vierendeel frame that wraps the perimeter of the building. The building uses approximately 9,000 tons of structural steel in all.

Floor Framing System

All the steel floor framing in the tower is S355 K2G3/G4 steel (approximately equivalent to ASTM A992). The typical office floor slabs have 3-in. deck plus 3 in. of lightweight concrete. The floor slabs at levels 1, 12 and 24, which correspond to the top chords of the two-story truss system, and the mechanical level slabs, which correspond to the bottom chords of the truss system, are 3-in. deck plus 6 in. of normal-weight concrete; the thicker slab is primarily required to minimize sound transmission from the mechanical equipment. The floor framing at each level is supported by the two reinforced concrete cores and by four interior and four exterior HD400 (W14) steel columns.

Long-Span Transfer Trusses

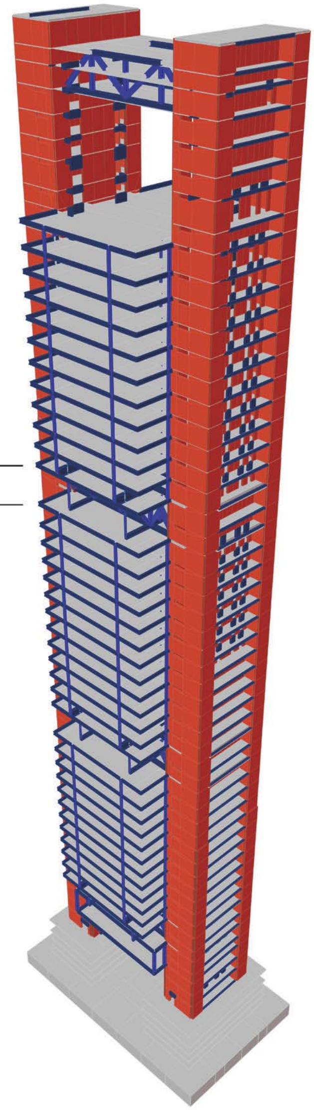
The north-south lateral loads are resisted by pure cantilever action of the two cores, and with gravity load for the entire building supported only by the cores, there is no uplift or tension in the core walls, even with an aspect ratio of 11 to 1. For east-west lateral loads the cores are too narrow to provide adequate strength and stiffness as pure cantilevers, and therefore are linked together at the three truss levels such that the core and truss system behaves like a large moment frame to resist lateral forces.

The three truss levels not only serve as a primary component of the lateral system, they also transfer the eight gravity load columns (four interior and four exterior) from each office block to the two cores. At each of the three truss levels the system of trusses consists of the following: two "primary" trusses that span 105 ft between the cores and two "secondary" trusses that cantilever 33 ft to the north and south past the primary trusses and transfer the four exterior columns back to the primary trusses.

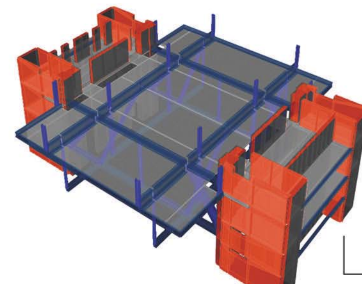
Ideally, the primary trusses would have been designed as simple span between the cores. However, since the primary trusses also interact with the cores to resist lateral loads, the top chord of the truss had to be connected to the core. With the truss top chord connected to the core walls, negative bending moments due to gravity loads would develop in the truss, resulting in large top chord tensions at the connection to the core. In order to minimize the gravity load negative moments in the truss, the top chord connection of the primary trusses to the core has been detailed to allow horizontal movement, and this connection was not pretensioned until the full structural dead load of the office block it supported had been applied to the truss. So, in the permanent condition, top chord tensions in the truss only result from floor live loads and east-west lateral loads.

Since the top chord connections of the primary trusses were not pretensioned, they would act as simple-span trusses during erection of each office block, developing large tension forces in the bottom chords, which would be resolved as a horizontal thrust against

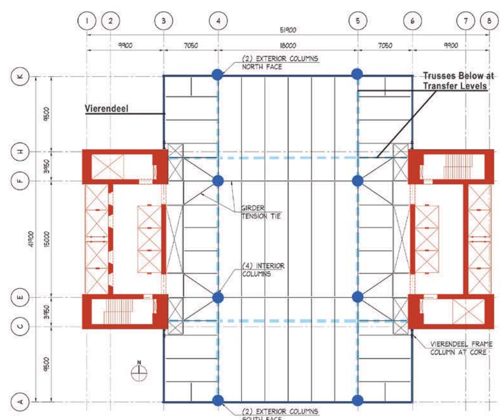
Typical and transfer levels of Caja Madrid.



Transfer Level



Typical Level



Images: Halvorson and Partners

The typical floor area in the building is approximately 14,530 sq. ft.


Foster + Partners

Caja Madrid Tower, at 820 ft, is the tallest building in Spain.

the cores. As the cores try to resist the thrust and move outward, away from the floor plate, tension forces are introduced into the floor framing members on multiple floors above and below the truss bottom chord levels. In essence, the floor framing above and below the truss levels is trying to hold the cores together as the thrust from the trusses tries to push the cores apart, which was an undesirable design solution. To eliminate the trusses' thrust against the cores, post-tensioning tendons are provided along the bottom chord of the primary truss and anchored to an embedded column in the cores. In addition to minimizing the axial thrust on the core, the post-tensioning provides a level of redundancy for the connection of the bottom chord truss to the core.

At each level within the core, where the truss top and bottom chords attach, a 6.2-ft-thick slab is provided. The thick slabs, along with a two-way system of multi-strand post-tensioning tendons and reinforcement, provide the required strength and a means of engaging the full cross-section of the core to resist the truss chord forces. The connection of the bottom and top chords of the primary trusses to the core are critical, since these 12 connections transfer all gravity loads and east-west lateral loads to the cores, and ultimately both the top and bottom chords of the truss will transmit tension and compression forces to the core. Once the bolts of the top chord connection to the core are tightened, live loads on each office block will induce negative bending in the truss and thus tensions in the top chord; east-west wind loads will induce load reversals in the truss chords. So, robust positive connections of the truss to core are provided by embedding two built-up steel columns within each core. The column is in turn anchored with post-tensioning rods into the 6.2-ft slab, providing adequate strength to transmit truss forces into the core cross-section.

Given the critical nature of the trusses and their connections to the core, redundancies were built into the entire system. Each set of primary trusses and connections to the core have sufficient strength to resist service loads from two office blocks, should a catastrophic failure of a single truss level occur. For instance, if a failure in the lowest level truss occurred, the first 11-story office block could hang from the truss level above. One might think that designing each truss level with enough capacity to support two office blocks is an inefficient; however, the premium to build in this redundancy was marginal. The truss member sizes were not controlled by strength to resist gravity and lateral loads forces; they were sized to provide adequate stiffness for lateral drift in the east-west direction. With the trusses sized for stiffness, an increase in the material strength from S355 to S460 (50 ksi to 65 ksi) was sufficient to provide the necessary strength to support service loads from two office blocks on one truss level.

Although the truss system was designed with robustness and redundancy for a catastrophic failure, the core walls themselves did not have this additional capacity to resist the negative bending forces that would develop, and the building would still be susceptible to a progressive collapse. The solution to protect the core walls was to design a "structural fuse" for the top chord-to-core wall connection, which essentially proportioned the splice plates such that they would yield prior to any overstressing of the core walls.

Vierendeel Frame

The architectural design intent was to minimize the number of exterior columns on the typical office floors and eliminate corner columns. This was achieved by providing only two columns on the north and south faces of the building; the columns are spaced 59 ft apart, with a 23-ft cantilever to the east and west of each column. To eliminate the columns from the corners, spandrel beams on the east and west side of the building would cantilever from the cores out to the 23 ft cantilevers on the north and south faces of the building.

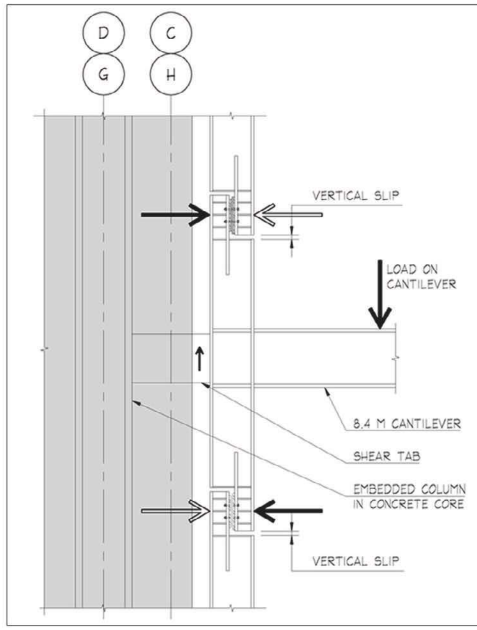
The two exterior columns on the north and south sides are supported directly on the secondary trusses at the three truss levels. To minimize the depth of the 23-ft cantilevers, the spandrel beams on the east and west are moment connected to the core. A moment connection of the steel spandrel beam to the concrete core wall would have been difficult to erect, so a steel column was placed 6 in. from the core wall to provide moment fixity for the spandrel beam at the core. The column adjacent to the core is connected with a simple shear connection, a simpler detail to construct. The perimeter spandrel beams and exterior columns form the Vierendeel frame, minimizing the depth and weight of the frame and helping to control deflections.

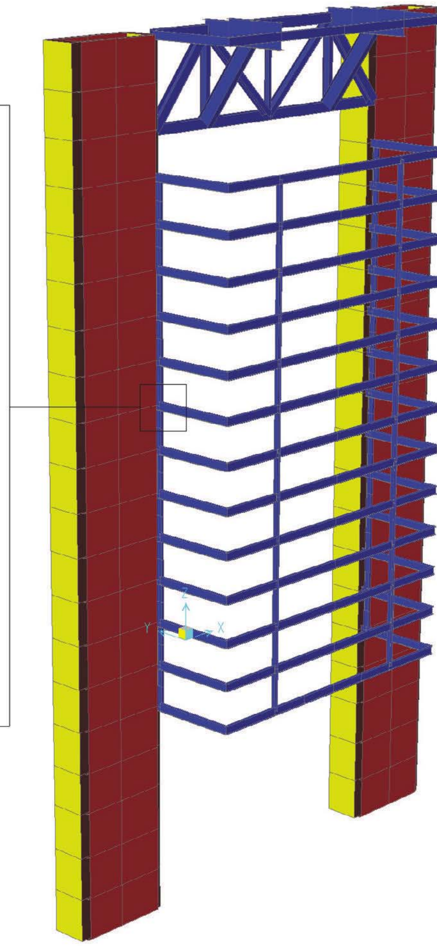
With a steel column located just 6 in. from the core wall, the effects of creep and shrinkage of the concrete core had to be addressed. Since the steel column would not creep or shrink with the concrete core, the core would be transferring axial load to the column over time and overstressing the column and the connection between the column and the core. Since the adjacent steel column is only required to provide bending stiffness for the cantilevered spandrel beam, the axial loads could be released, allowing the core to creep and shrink as it wants to without overstressing the columns. A vertical slip detail was provided at the mid-depth of the column, at approximately the inflection point. The slip detail still allowed for a shear transfer such that the column could provide bending stiffness for the cantilevered spandrel beam of the perimeter Vierendeel. MSC

Greg Lakota, AISC Professional Member, is a principal with Halvorson and Partners and was the lead project engineer for the Caja Madrid Tower.

Architect
Foster + Partners, London

Structural Engineer
Halvorson and Partners, Chicago





A detail of the connection between the Vierendeel exterior framing and a concrete core.

Halvorson and Partners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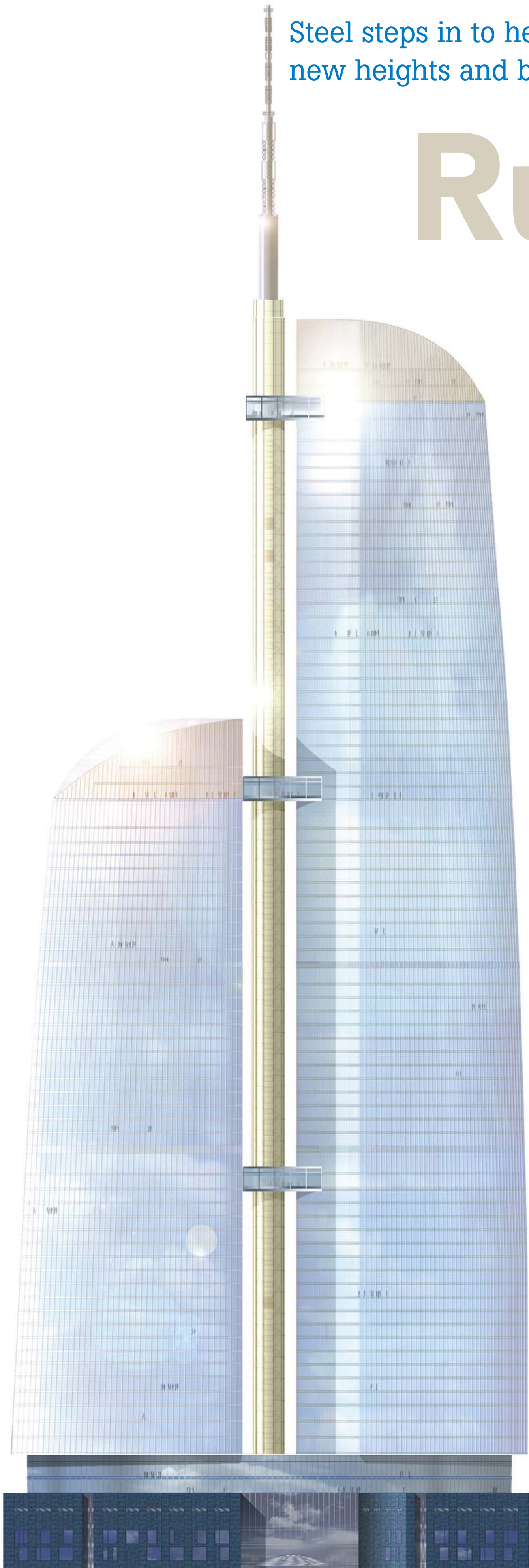

Rendering courtesy of NPS-Tchoban-Voss.

Steel steps in to help Moscow's Federation Tower reach new heights and become Europe's tallest building.

Russia Rising

BY BRAD MALMSTEN, P.E.

WHILE THE NEVER-ENDING RACE for world's tallest building has shifted to the Middle East for now, tallest on the continent is nothing to sneeze at. Upon its completion next year, Tower A of the Federation Tower complex in Moscow, Russia will be the tallest building in Europe at 1,181 ft (its architectural central spire will reach 1,470 ft).

The 93-story Tower A is mainly concrete (as is the 62-story Tower B), but major steel framing at multiple crucial areas of the tower—including an atrium at the top—was essential to bringing the tower together and allowing it to vie for the title of Tallest in Europe. Roughly 5,500 tons of structural steel in all made this happen.

Tower A Structure

Outrigger levels—complicated structural interchanges involving column transfers, shifting of wind loads from core to exterior, and rebalancing of building weight—are the hubs of the tower's load network. These hubs are built in steel, and the result is open space for equipment, access to fresh outside air, and a system of trusses capable of supporting fully reversed loading.

The trusses link all of the exterior columns together into one giant perimeter "belt" and outrigger trusses link the concrete central core to this belt. This linkage manifests the network of load paths that is the cornerstone of the building's plan to maximize the resiliency of the structure as a whole, stabilize it against lateral loading, and mitigate the potential for progressive collapse.

The steel truss systems are placed at the one-third and two-thirds heights of Tower A and share floor space with mechanical rooms and refuge areas. These areas naturally separate the building into distinct fire protection zones as well as occupancies. Steel framing also allowed clear space—that would have been impossible to match with concrete walls—facilitating the MEP design and the layout of refuge floors.

The complex geometry of the tower also made these particular areas more conducive to steel framing. The extraordinary amount of reinforcing steel that would be required and the curvature of the exterior belt made rebar detailing difficult and execution cumbersome, and thus prone to non-conformance. On the other hand, angle changes were easily handled by steel truss work, with gusset plates carefully detailed and shop-fabricated to bend around the column flanges to make the required curvature.

At about 20 ft deep, the 33rd-floor outrigger system is shallower than the designers would have preferred due to other design constraints, but even more reason to take advantage of the ability of steel to handle

The Federation Tower mixed-use complex in Moscow. Tower B on the left and Tower A on the right flank a 1,470-ft-tall mast structure.

Photo by urban-photos.com

Goin' to Kansas City, Kansas City here we come.

Photo by urban-photos.com

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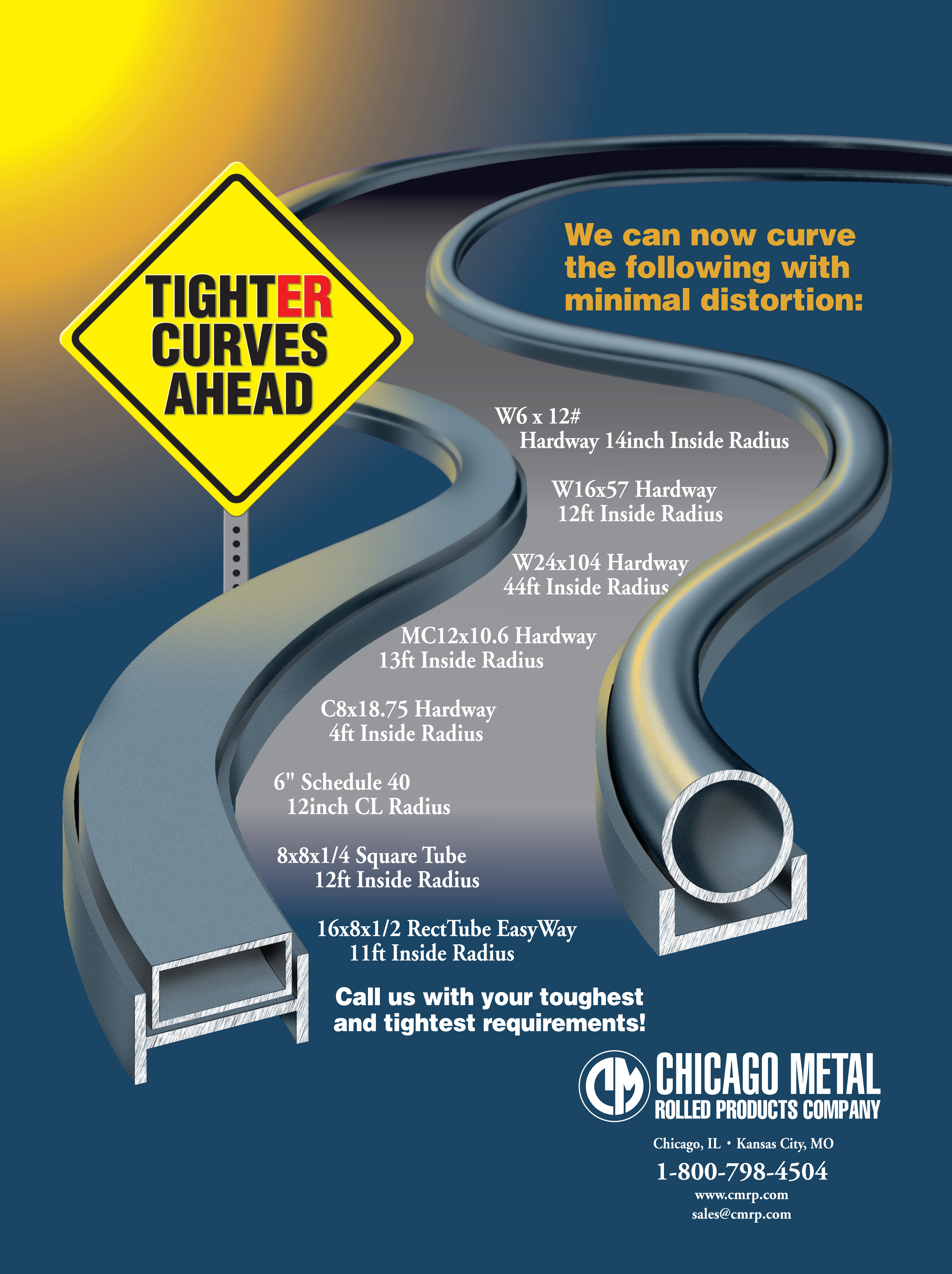

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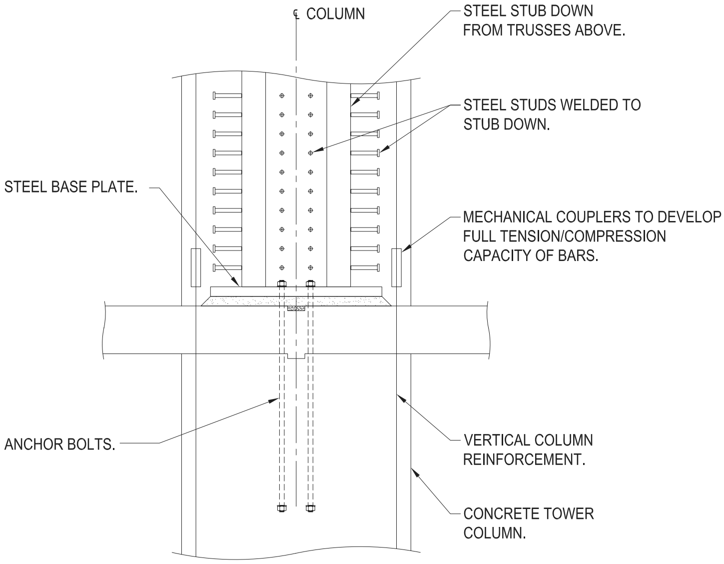
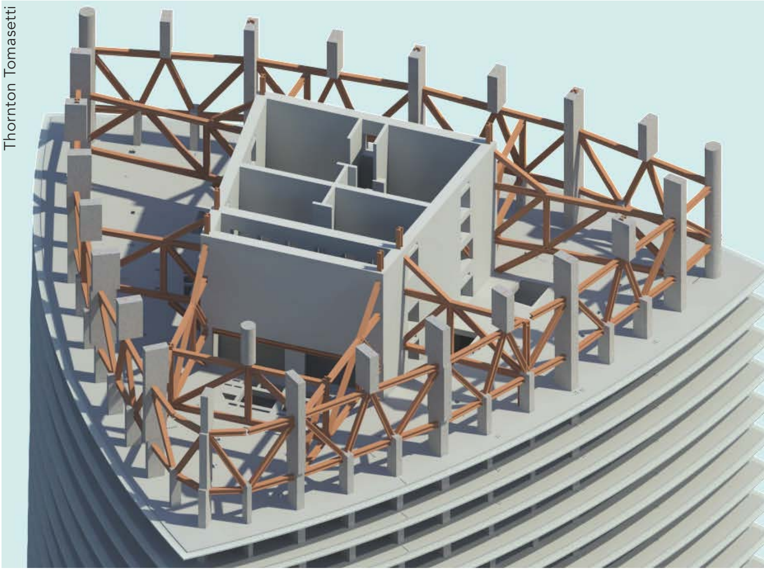


Base of a vertical stub down from steel a outrigger truss.


Thornton Tomasetti

Tower A's 61st-floor outrigger level.

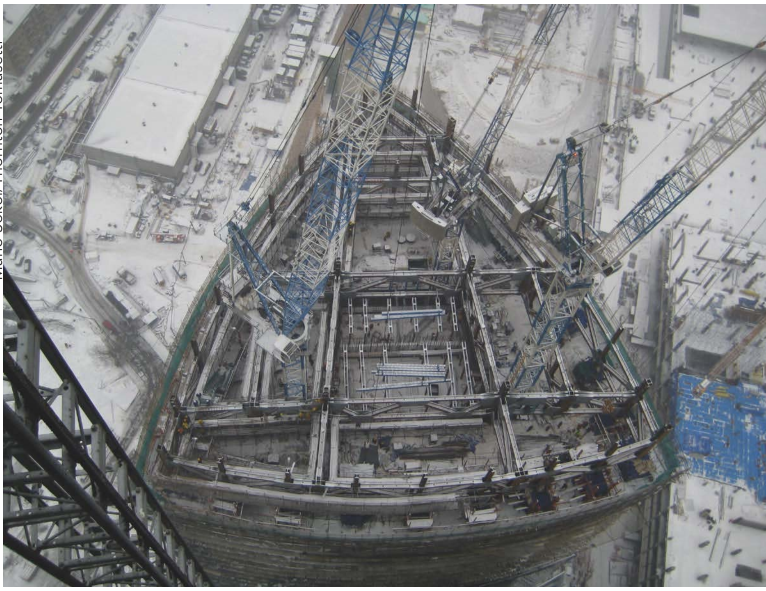
incredibly large forces in relatively small space. Some outrigger diagonals see design axial forces as high as 3,500 tons, and the belt truss members are designed for up to 1,700 tons. The entire truss system weighs roughly 2,000 tons and is comprised almost entirely of W14 sections connected by flange-bolted gussets. Where trusses intersect, built up box sections were used in order to simplify the connection details.

The 61st-floor truss system is about 43 ft deep and doubles as a transfer truss for half of the perimeter columns. At this level, outrigger diagonals see axial forces as high as 5,400 tons and the belt truss members up to 3,500 tons. This group of outriggers and belt trusses weighs in at about 3,000 tons. Again, W14 sections were used where possible, and the most extremely loaded pieces were built up from 5-in.-thick, 30- to 34-in.-wide plates.

Load Network

The term "load network" is used above to indicate the level of interconnectivity of the structural elements of Tower A. As in all buildings designed to resist progressive collapse, creation of alternate loads paths was desired. In the case of the Federation Tower, steel trusses allow two levels to be force hubs in the overall load network of the structure, providing not just one load path and one potential alternate load path, but an unlimited number of potential load paths.

Continuity and ability to handle reversed loading in all of the structural elements in the building is a major theme of the design. With this in mind, the steel trusses were not only encased in the concrete core walls and columns within height of outriggers, but also truss verticals were extended one floor up and down into the concrete structure above and below. This ensures the best behavior of the building as a whole in common or unusual circumstances, as well as ensures that each steel piece is loaded in a manner consistent with the design intent. Stubbing up and down allows the concrete to unload smoothly into the steel trusses, and for the trusses to unload smoothly into the concrete columns and walls.


Mario Sokol/Thornton Tomasetti

Looking down on Tower A's 33rd-floor outrigger and belt trusses during the final phases of steel erection.

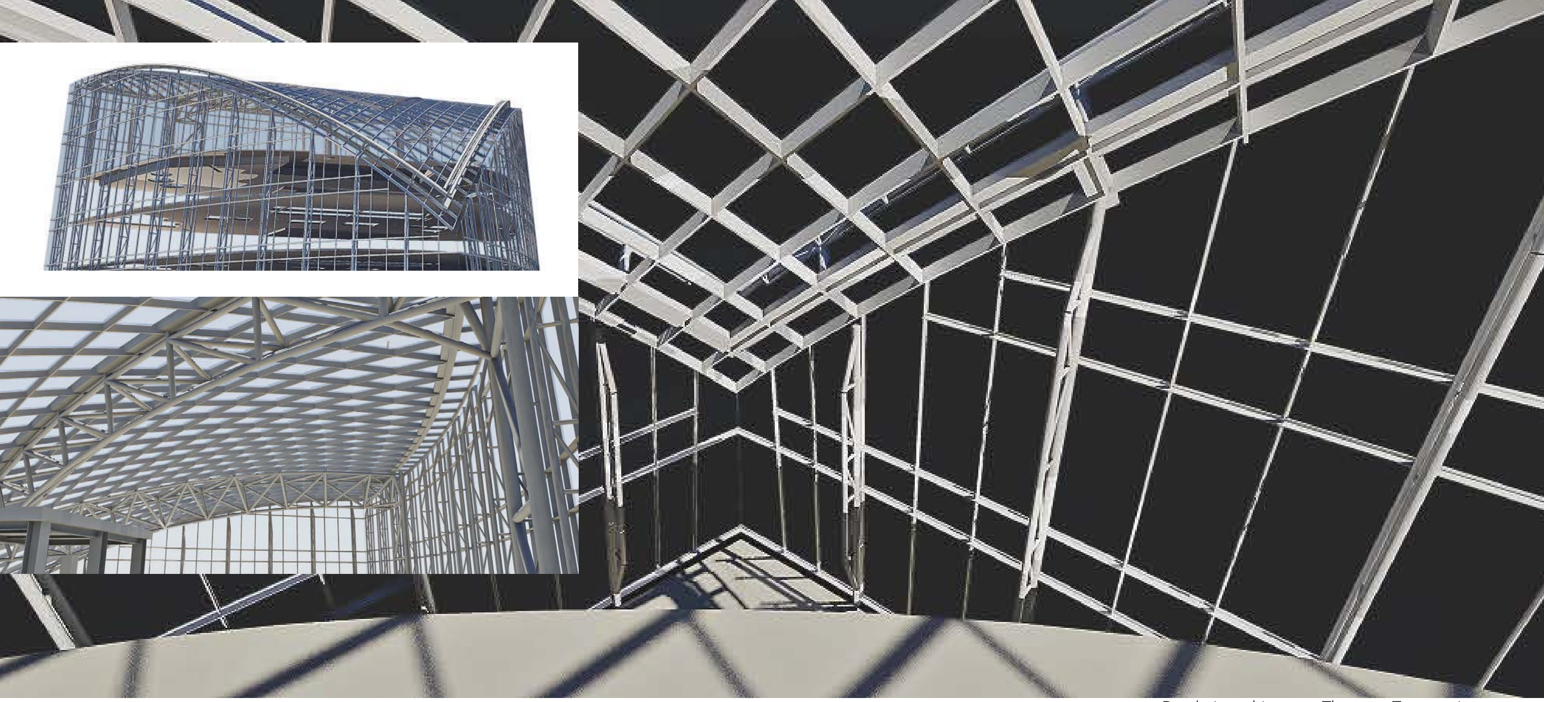
Cap Structure

The Tower A cap was destined to be a highlight of the project. Curved sides and roof meet above a five-star hotel lobby and exclusive VIP space, and the opportunity to do something special called. Steel—about 500 tons of it—provided the answer in the form of an integrated, transparent crown.

The stunning glass box that caps Tower A is designed so that the glazing mullions also act as part of the load bearing structure. Integrating the structure, glazing, cleaning, drainage, shading, fire protec-


Sofia Pechorskaya/Thornton Tomasetti

The west section of the 33rd-floor outrigger trusses, linking the belt truss on the left to the central core on the right.



Renderings this page: Thornton Tomasetti

Multiple 3D renderings/views of the Tower A cap.

tion, and MEP systems results in maximum transparency and allows the occupant to realize the openness and beauty of the space instead of its many components, most of which will be fully exposed. The 16,000-sq.-ft open space features built-up façade mullions, solid-plate roof mullions, two central super columns built from 4-in.-thick plates, and exposed triangular super trusses.

Perhaps no building in the world so clearly demonstrates the benefits of hybridization—or how even in a concrete building, there are still areas where steel is the only option. MSC

Brad Malmsten is an associate with Thornton Tomasetti and an AISC Professional Member.

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Thornton Tomasetti, New York

Architect
NPS Tchoban Voss, Berlin, Germany

Keeping the

The Pulse Bar at the Atlanta Marriott Marquis.

Benjamin Lipford

New space requirements call for a repurposed lobby and ballroom areas in an Atlanta hotel.

Party Going

BY ROBERT M. WEILACHER, P.E., S.E., LEED AP, AND DR.-ING. OLAF U. FABER, P.E.

A RECENT HOTEL RENOVATION in Atlanta not only repurposed a significant amount of the interior, it also saved large portions of the building from demolition. This $138,000,000, 302,740-sq.-ft renovation of the Marriott Marquis Hotel presented a number of structural challenges in several areas of the building.

Ballrooms

The most significant portion of the retrofit was the addition of a new roof above the existing atrium level, which in turn became the floor of the new Atrium Ballroom. The new roof is supported by long-span structural steel trusses spanning from 90 ft to 135 ft (WT18 top and bottom chords, double angles for diagonals). The trusses are supported by concrete corbels attached to the existing shear wall structures, using reinforcing steel that was dowelled with epoxy.

For a second ballroom, the Marquis Ballroom, the existing 34,260-sq.-ft. roof structure was converted into the floor for the Atrium Ballroom directly above. With spans of up to 97 ft, 6 in., significant alterations to the trusses were required to satisfy the load capacity and stringent vibration criteria dictated by the new occupancy, as well as limited structural depth due to architectural desires and highly variable steel elevations in the existing secondary framing (see Fig. 1 for a structural cross section of both ballrooms).

Initial 2D models (created with RAM Advanse and several custom applications) and hand calculations determined that the dynamic behavior of the ballroom floor was unacceptable, with computed accelerations in excess of four times the allowable values. Due to the complexity of this problem, a full 3D dynamic model of the structure was made using SAP.

During the structural optimization process, one strengthening concept that was developed was to engage vertically offset the concrete and steel elements to act in composite action. To achieve this Uzun and Case used concrete stem walls, shear studs, and shear friction rebar to engage steel trusses with two different layers of concrete stabs and some vertically offset steel beams.

Extensive benchmark studies were performed to calibrate the structural behavior of the composite system, which lead to a finite element model suitable for static and short-term dynamic analysis. Shear deformations were included to increase the accuracy, and the finite element model yielded deflections deviating less than 0.5% from theoretical results, using the transformed moment of inertia method.

For final static code checks, a staged construction analysis was performed. Non-composite stresses from the construction phase and composite states after completion were superimposed and compared to load carrying capacities of the modified composite ballroom floor trusses. The final solution used WT×325 members with overall lengths ranging from 40 to 60 ft to reinforce the bottom chords of the existing trusses. The top chords were modified to engage the existing and new floors slabs in creating composite action for added rigidity and strength. The welds for the WT shapes were designed for the shear flow along the interface with the existing truss bottom chord (see Fig. 2).

The conversion of the existing roof structure into a composite floor system was accomplished by using a new reinforced concrete slab over structural foam infill divided by a grid of supporting short reinforced concrete walls, which transmitted forces between slab and truss layers using new welded shear head connectors on the top chords of existing trusses. Transverse vibrations were controlled by introducing knee braces at the secondary beams. The new concrete floor slab had a thickness between 5 in. and 8 in. and balanced the simultaneous gain in stiffness and undesirable increase in mass. The resulting non-homogeneous mass and stiffness distribution, bridging the voids between the existing floor truss structure and new slab, was modeled in the dynamic 3D model.

For the spatially defined activity zones on the new ballroom floor, loading conditions for the "dancing" and "concert" scenarios were generated from available spectral representations. The structural response was computed and evaluated in the time domain by means of direct integration, and the response was validated in the frequency domain using spectral analysis methods. The calculated accelerations were mapped over the entire floor system for each loading condition under consideration. Computed acceleration peaks for the modified floor system were within the intervals recommended in AISC Design Guide 11 *Floor Vibrations Due to Human Activity*.

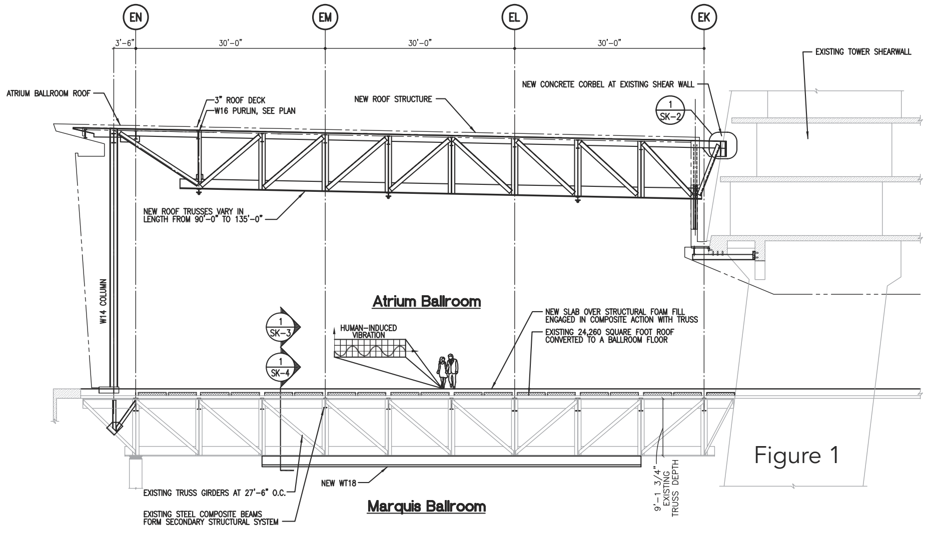
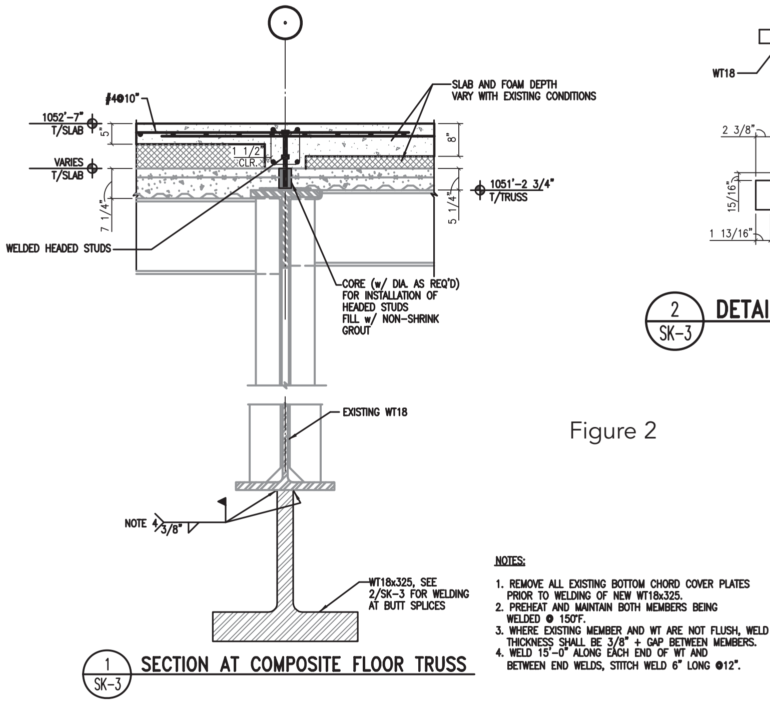


Figure 1



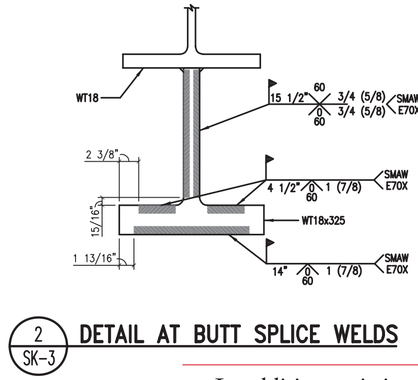


Figure 2

photos this page by Benjamin Lipford

The renovation includes two ballrooms, one on top of the other.

Roof trusses for the Atrium Ballroom.

In addition, existing structures that were above the new floor elevation—concrete columns, stairs and trellis structures—were cut down. Concrete infill slabs on metal deck and steel beams bolted into the existing concrete were used to fill the openings.

Lobby

In the lobby area, a new 45-ft-tall steel and acrylic paneled bar—the Pulse Bar—was added, consisting of curved 8-in. hollow structural sections and rod bracing supported on four HSS columns. Holes were drilled in the existing concrete floor, and the new columns were connected to the floor by steel collars and post-installed anchors.

Also in the lobby area, the existing escalator was expanded to service more floors; openings were cut into the existing concrete floor to make room for the new escalators. The loss of continuity in several cut beams necessitated the addition of steel beams to support the concrete floor. These varied between W12 and W24, more due to the geometry of the space rather than strength. One escalator opening required the removal of an existing concrete tension tie that resisted the "scissor" forces caused by the building's shear walls. The existing tension tie system was replaced by a pre-tensioned, high-strength steel rod system placed below the floor before the existing tie's removal.

Besides escalators, several new stairs also had to be installed in the hotel to satisfy increased building code egress requirements. Floor openings were cut into the existing structure, sometimes to three levels down, requiring the reinforcement of existing steel beams and the creation of new framing using a combination of steel beams and cover plates. The project used approximately 1,025 tons of structural steel in all.

Thanks to some creative reconfiguration work, the Marriott Marquis' prime gathering areas enjoy enhanced use and life, all while keeping one of Atlanta's major hotel and convention destinations intact. MSC

Robert M. Weilacher is a senior associate and Olaf U. Faber is an associate, both with Uzun and Case Engineers and both AISC Professional Members.

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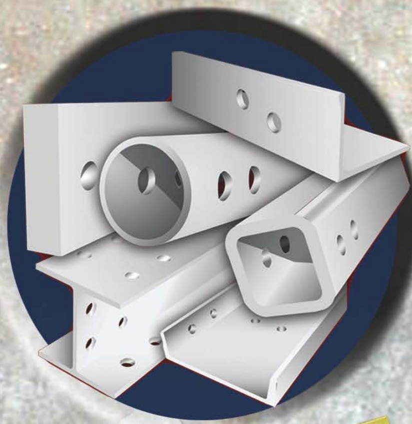

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Under Glass

Photo by The Harman Group

BY MALCOLM BLAND, P.E., LEED AP, AND CHRISTOPHER CONN, P.E., S.E.

AT 2,400,000 SQ. FT, GAYLORD NATIONAL is the largest privately financed combined hotel and convention complex on the East Coast.

The complex is the cornerstone of the newly developed National Harbor development overlooking the Potomac River in Prince George's County, Md. The hotel contains 2,000 guest rooms, meeting space, and two ballrooms. The 800,000-sq.ft convention center portion includes a 180,000-sq.-ft exhibition hall, two ballrooms of 50,000 sq. ft and 35,000 sq. ft, and approximately 400,000 sq. ft of flexible meeting space.

Structural steel was used in many areas of the project, such as the hotel and convention center ballrooms, theater space, mezzanines, façade support, canopies, an 8,000-sq.-ft pool building, and a porte cochere, for a total of 5,400 tons.

Marquee Space

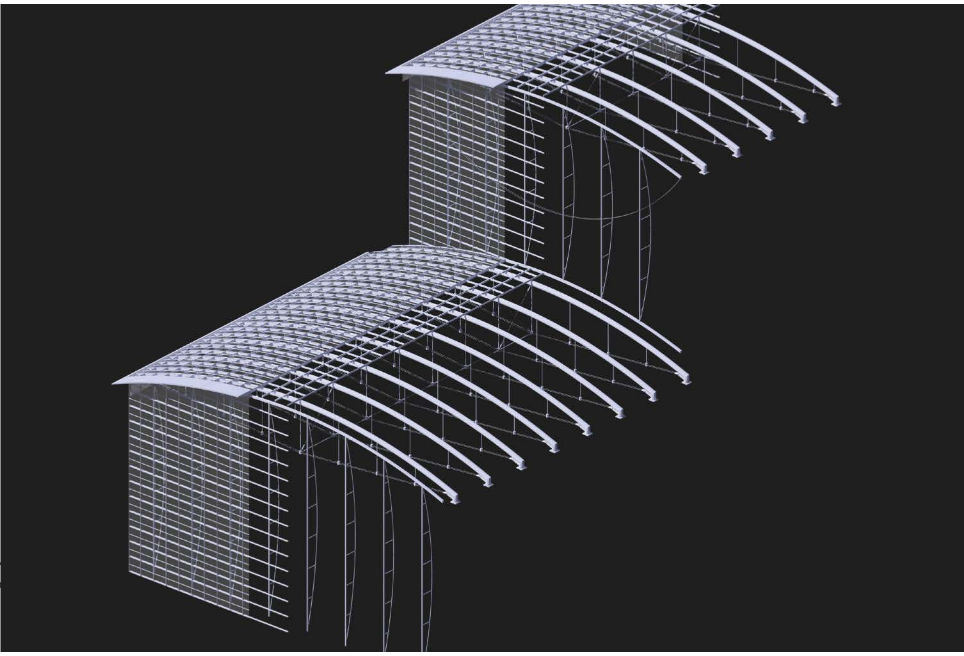
The centerpiece of project is the 2,400-ton structural steel bow-string truss roof skylight structure over the 1.6-acre hotel atrium. Providing 120,000 sq. ft of glass walls and roofs, the atrium connects the wings of the central hotel tower and the two lower hotel towers. As the feature space of the project, and in deference to the view to the Potomac River, the objective for owner Gaylord Entertainment was that the skylight must have less structure and more openness than its previous projects, thus maximizing the benefit of the project's premier location. The design team developed multiple schemes for the structure to achieve Gaylord's goals, eventually selecting the bow-string truss structure, as it achieved both the aesthetic and functional requirements for the atrium.

The final result was a design in which the horizontal surfaces of the atrium are spanned with exposed bow-string (tied arch) trusses and hollow structural section (HSS) purlins, while the vertical curtain wall façades that span between the upper and lower roof and from the lower roof to the ground were supported using Vierendeel vertical trusses at the walls with HSS girts, and a hung catenary upper wall.

The bow-string trusses consist of curved 40- and 42-in.-diameter pipe top chords, vertical pipe web members and double 3-in. and 4-in. rod bottom chords. The trusses at the upper atrium roof are 28 ft deep at mid-span and span 195 ft, while trusses at the lower atrium roof are

2008© Photo by Alan Karchmer, courtesy of Gensler

A glass-and-steel-encased atrium meets the goals of less structure and more openness, and provides great views of the Potomac River in the process.



Rendering by Gensler

opposite: The 19-story atrium is topped with a 2,400-ton steel bow-string truss roof skylight structure.

top of page: The Gaylord National contains 2,000 guest rooms and a 180,000-sq.-ft exhibition hall.

mid-page: This cutaway of the atrium depicts the hierarchy of framing.

32 ft deep at mid-span and span 220 ft. Stability of the trusses, which include round HSS king posts, is attained with the use of bridging located at the center king post.

The HSS purlin frames that support the glazing system connect to the top of the bow-string truss top chord and provide lateral bracing to the trusses and act as the diaphragm for distributing lateral forces to the hotel towers. Lateral wind forces were determined through boundary wind layer testing performed by Rowan Williams Davies & Irwin, Inc. (RWDI) by combining mean wind load distributions with appropriate fluctuating dynamic load distributions, including an allowance for dynamic amplification resulting from inertial effects. An IBC Exposure Category of D was used as the basis of design due to the proximity to the Potomac River, which is 2½ miles wide at the location of the complex.

Efficiency of Design

To increase efficiency, fabricator Banker Steel Company suggested using spiral welded steel pipe at all but the two heavily loaded trusses. A mock-up of the top chord was prepared to determine the acceptability of the spiral seam. Gensler determined that while the seam was visible at close range, it did not need to be ground smooth, given that the trusses are viewed from a distance. However, at splice locations, the seams had to be within the top 20 degrees so the seam would appear continuous from below. One significant challenge for Banker was curving the 40- and 42-in.-diameter top chord members. A shop-performed heat curving procedure, developed by Banker, accomplished this successfully. The process involved building a track device so that gas "rosebud" heat torches would gradually move along the pipe and heat the steel so that it would curve under its own self-weight.

In order to eliminate structural columns impeding the use of the atrium floor, the roof structure was designed to be supported directly on the top of the hotel's structural framing. The layout of the hotel wings, with a U-shaped 19-story building and two 8-story sections—all separated with expansion joints—required careful consideration of slide bearing supports for the atrium structure and hinging of the upper wall. Thoughtful design of slide bearing connections, plates, and bearing pads at each end of the trusses, capable of transmitting the

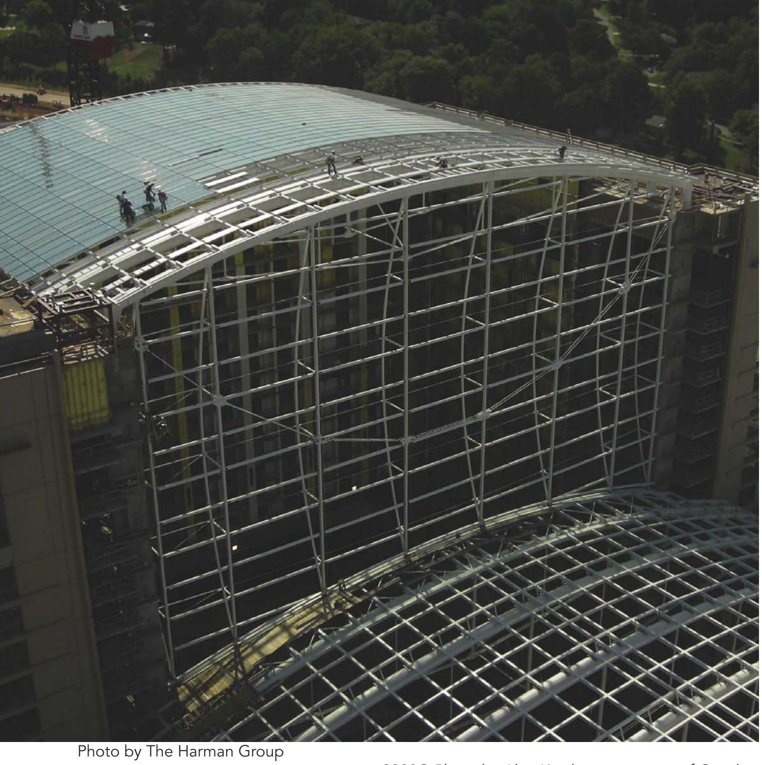


Photo by The Harman Group

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The upper atrium wall is designed as a catenary truss hung from the top of the U-shaped 19-story hotel tower.

gravity loads and resisting lateral loads induced by wind or seismic events, allow the hotels tower to move independently.

To accommodate the differential movement between the towers, slide bearing connections were detailed to allow movement in certain directions while resisting forces in other directions. This allowed adjacent towers to move out of phase as much as 6 in. The vertical glass walls were detailed in a similar manner; thus only one tower resists the lateral load in the direction parallel to the truss span. Stop blocks and uplift preventers were added to the end connections to control the action of the atrium structure should the movement at the slide bearings exceed the anticipated ranges.

Rather than using a large truss at the lower atrium roof for the support of the upper wall, which would have hindered the openness of the atrium, the upper wall is designed as a catenary hung from the top of the U-shaped 19-story central tower, similar to a dog flap. Double 2.5-in.-diameter rods with standard clevises in an approximate parabola form a deep catenary truss with a rectangular HSS top chord. Lateral loads acting perpendicular to the plane of the wall are resisted by vertical Vierendeel trusses that span from the upper to the lower atrium roof. Due to the expansions joint locations in the hotel towers, the connection of the vertical trusses to the lower atrium roof is designed to allow movement in the north-south direction, as well as to allow vertical movement and rotation while resisting movement in the east-west direction.

In order to reduce erection time and cost, the HSS purlins forming the diaphragm of the upper and lower roofs were fabricated into modules, and all connections were designed to be field bolted. Careful consideration of ease and speed of erection, erection tolerances and adjustability, and glass system deflection limitations drove the connection design of the HSS purlin frames to the tops of the bow-string trusses.

Clevises were used to connect the rod bottom chord of the trusses to the gusset plates at each connection point. Standard and heat-treated cast #8 clevises were used for trusses with 3-in. rod bottom chords, and machined custom clevises, designed and fabricated by fastener manufacturer Wecall, were used for the 4-in. rod bottom chords; machined clevises were required at two heavily loaded trusses.

Material strengths for the clevises included ASTM A668 Class A, F_u = 47 ksi for the standard clevises; ASTM A668 Class F, F_u = 90 ksi for the heat treated clevises; and ASTM A322 Grade 4140, F_u = 98 ksi for the machined clevises. Both machined and heat treated clevises were full scale tested, to failure, at Lehigh University to determine their ultimate capacity. Based on the test results, the clevis capacities for the heat-treated and machined clevises were approximately 850 kips and 1,400 kips, respectively. Due to their small size, their clevises provided aesthetic and construction advantages over connections of other types. The use of the clevises allowed truss erection to be expedited, thanks to reduced field welding and reduced quantities of bolts.

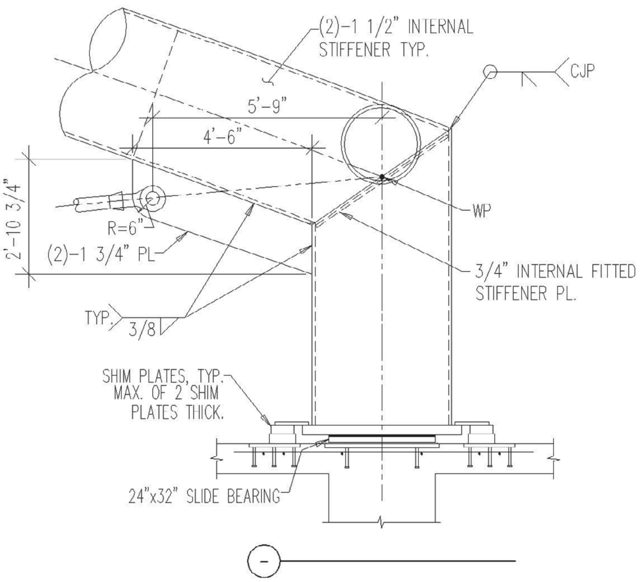
Erection Logistics

Each truss was fully shop assembled to insure proper fit-up of the field welded joints and to confirm all dimensions of each truss. Once delivered to the field in pieces, the trusses were fully assembled in a vertical position on precisely located rig platforms. The pipe top chord was shipped in three pieces for most trusses and five pieces for the two heavily loaded trusses, and full-penetration welded back together on the ground. The overall dimensions were checked again, as were the elevations of the purlin seated connections. In regards to the vertical wall, the trusses were again shop and field assembled into position following the approved erection procedure. With all of the complexities involved in this project, there were only minor fit-up issues in the steel-to-steel connections.

The exposed steel is coated with a Sherwin-Williams direct-to-metal (DTM) urethane.

Photo by The Harman Group

Erection of one of the bow-string trusses; the trusses used for the atrium were either 195 or 220 ft long.



A detail drawing of the truss end bearing. The Harman Group

The layout of the hotel wings greatly impacted erection logistics. To gain access to install the bow-string trusses of the upper atrium roof, the lower roof could not be placed. This required holding the start of erection until the center hotel tower had topped out, which further dictated the sequencing of construction of the hotel towers. Since the largest bow-string truss weighed 77 tons and also due to the tight site constrictions, a Liebherr LR 1400/2 crane—assembled in a wheeled ringer configuration and with a 450-ton capacity—was used for erection. Trusses were picked up from the atrium floor and rotated in the air, then the crane advanced into the center of the U-shaped center hotel tower to place the trusses on the roof of the hotel.

Erection of the bow-string trusses proceeded westward until the upper atrium roof was installed and the first two 220-ft-long lower atrium bow-string trusses and purlin frames were installed. Construction outriggers were fitted to the lower roof to temporarily support the vertical Vierendeel trusses of the upper wall. The temporary supports remained in place until the catenary rods and girts of the upper wall were fully installed. When the temporary supports were removed, putting the catenary into tension and hanging the upper atrium wall from the center hotel tower roof, the wall deflected down approximately ⅜ in., within 10% of what was estimated. Once the upper Atrium wall structure was completed, erection of the remaining lower Atrium roof trusses and purlin frames and lower wall Vierendeel trusses continued to the west. MSC

*Malcolm Bland is a vice president with the Harman Group and principal in charge of the structural engineering of the entire hotel and convention center complex, and can be reached at **mbland@harmangroup.com**. Christopher Conn is an associate with the Harman group and directed the design of the atrium roof structure. He can be reached at **cconn@harmangroup.com**. Both are AISC Professional Members.*

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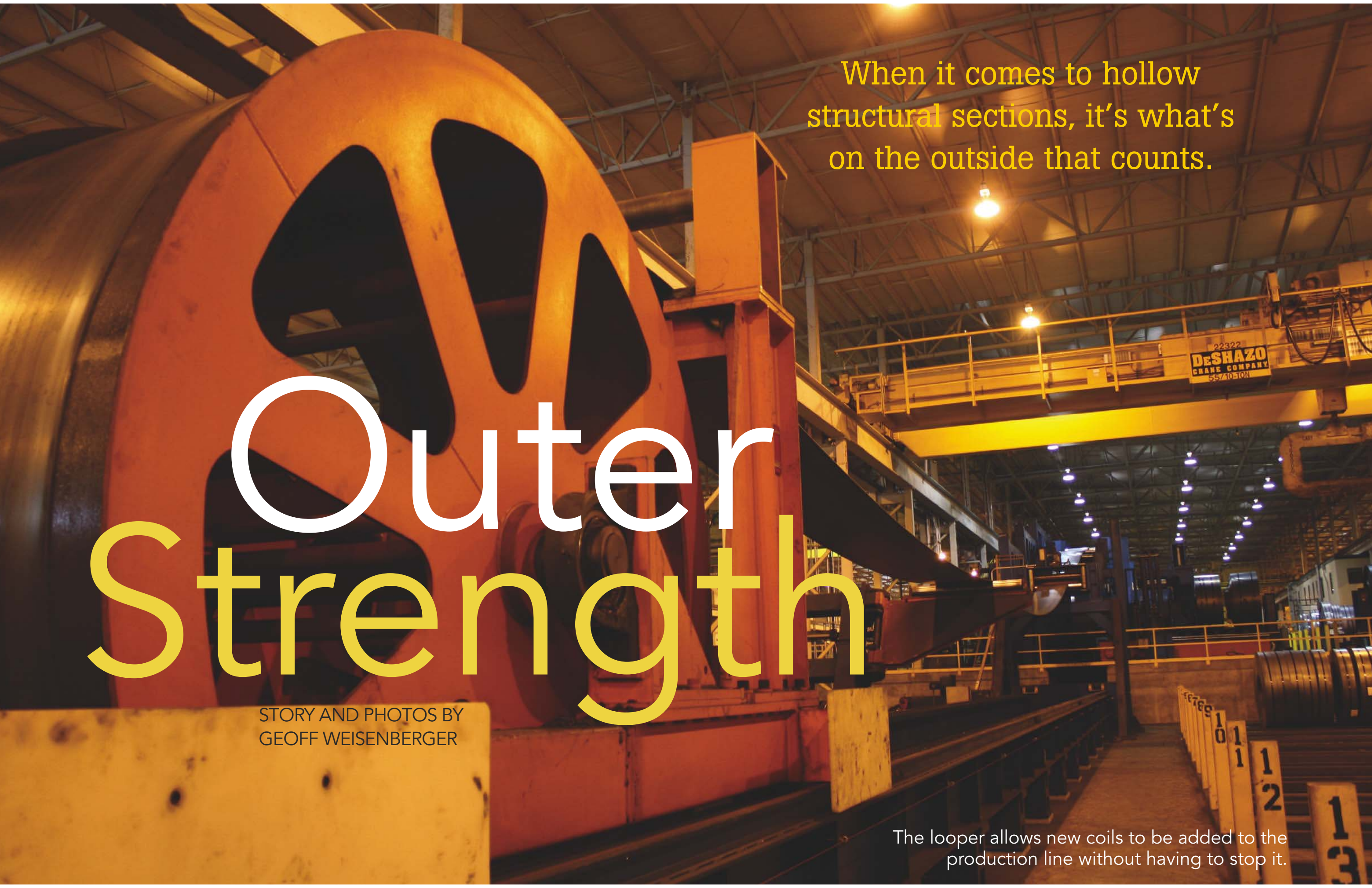


When it comes to hollow structural sections, it's what's on the outside that counts.

Outer Strength

STORY AND PHOTOS BY GEOFF WEISENBERGER

The looper allows new coils to be added to the production line without having to stop it.

THE FIRST IMAGE THAT POPS TO MIND when thinking about structural steel is usually a wide-flange member, with its familiar "I" end profile. But lately, circles, squares, and rectangles have been making their own case for steel symbolism; hollow structural sections (HSS) are a major part of the structural steel presence these days. Roughly 3 million tons of HSS were produced in North America in 2008, compared to approximately 5.6 million tons of W-shapes.

A visit to the country's newest HSS mill, in Decatur, Ala., provided some insight on how hollow structural shapes come to be. The plant, which opened in 2006, belongs to Independence Tube Corporation and is one of the company's three facilities; the others are in Chicago and Marseilles, Ill. This newest facility produces hollow rounds and shapes for structural fabricators and steel service centers; the other two facilities also produce structural sections, as well as steel tube for agricultural machinery. Independence is currently installing another mill in Marseilles, which will allow them to produce more rounds from 1.66 in. OD through 4 in. OD; it will be operational this fall.

As Independence's newest plant, Decatur is also the most automated. However, division manager John Helinski notes that machines can never completely replace people in the HSS industry. "I don't think you can ever take people out of the tube-making process," he says. "Automation can only take us so far. There is a lot of assumption on the part of newer employees is that the machine can do it all on its own. Machines can reset themselves back to an original position for size setup, but that's based on sensors. But what if the sensors are wrong? People need to calibrate them. Also, if the sensors fail completely, operators still need to know how to set up the equipment manually."

In terms of output, custom orders provide the bulk of the Decatur plant's work, although having product in stock has become increasingly important. "Customers want it now," says Helinski.

The plant operates its weld mill on two shifts instead of three. Helinski explains that this is because the company believes strongly in ongoing preventive maintenance as opposed to running machinery into the ground before replacing it. There are two full-time maintenance personnel on site for three shifts, allowing one full shift to be used for preventive maintenance. Downtime can be used to service the machinery as necessary, and Independence ships tubing on all three shifts at each location.

A walk through the 310,000-sq.-ft facility almost immediately reveals that Independence practices what it preaches: HSS is used everywhere, from the entrance canopy to the main structure and even the heavy tube-making machinery. Obviously, the facility didn't build itself; the HSS used to frame it was made in the Marseilles and Chicago plants. Of course, the mill is now capable of making its own HSS and can create product from 2½ in. square to 12 in. square, and 6⅝-in., 8⅝-in, 10¾-in, and 12¾-in. round (OD); wall thickness capabilities range from ⅛ in. to ⅝ in. In terms of length, the plant can produce sections up to 80 ft long.

Coiled Up

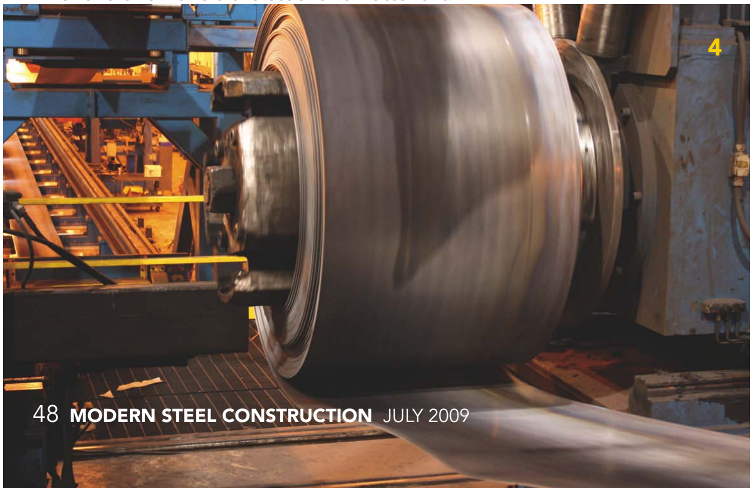
The interesting part is how these sections, whether 80 ft long or 20, come to be. HSS arrives in its final format in a much different manner from wide-flange steel. Where the latter is formed via the pouring of molten steel into dog-bone casts and rolled into its distinctive I

The slitter, creating two new widths of steel; sheared scrap (right) is recycled.

Coils in the staging area, following the slitting operation.

A steel coil, ready to be turned into HSS.

The leveler unravels the steel and flattens it.



shape, HSS is created from sheet steel that is hot-rolled into shape. Here's an over-simplified analogy: Think of taking a piece of paper and rolling it up, then taping the ends together. The process of making HSS is a high-tech, much more complex version of this, but you get the idea.

Here's how it works: Sheet steel arrives from the mill in coils, which look, to me, like huge rolls of metal toilet paper. Independence buys steel from multiple mills, and the Decatur facility receives most of its steel via barge—it's conveniently located along the Tennessee River—although it can also arrive by truck or rail.

Slit to be Rolled

Once it shows up at the plant, the sheet steel goes through several specialized machines before it ends up as tube. The first is the slitter, which, as its name suggests, slits the steel. Steel coil is fed into one end, where it unraveled. Next, the edges are slit off by round circular knives, which brings the coil to the proper width for the size of tube that is to be formed, and also creates smooth edges; the slitter is adjustable up to 74 in. wide. The steel is then recoiled onto another roll. The edge scrap is "balled" into separate collectors and eventually recycled. The operating station has a shield in front of it should the edge scrap, which is razor sharp, snap (although this has never happened here). The coil, now at the correct width, is staged until it is ready to be put through the mill.

The Never-Ending Ribbon

The mill stretches practically from one end of the building to the other. When in operation—i.e., when it's not shut down for breaks, shift changes, maintenance, or when the plant is closed—the mill essentially hosts a never-ending ribbon of steel.

A steel coil, after being sheared to the proper width in the slitter, is picked from storage via an overhead crane and fed onto a roll in the leveler. Like the slitter, the leveler—the first step of the mill—lives up to its name. As the coil is unraveled, the leveler draws it through rollers into a flattened state, working out any bumps or waviness. Next, the coil is butt-welded to the end of the last coil, although not all the way across—just enough so that it can be drawn along with the rest of the steel ribbon, which, again, never stops.

Buffer Zone

What makes this never-ending strand of steel possible is the looper or buffer, which is physically located at the near end of the mill, before the leveler (with the far end being final product). The looper works by creating a buffer or queue in the steel ribbon when a new coil is being added. This allows the mill to keep operating while a new coil is welded in place to again fill the looper (or queue) to capacity. There are different types of loopers in the tubing business. The one the Decatur plant uses is essentially a large drum that moves horizontally on a set of tracks in the opposite direction of the production process; a similar drum is at the far end of the leveler. So, when a new roll is welded into the line, it travels forward, loops overheard backward via the looper, then moves forward down through the rest of the mill proper. The process is repeated every time a new coil is added. A control screen shows how much time the looper

Flat sheet steel becoming rounder…

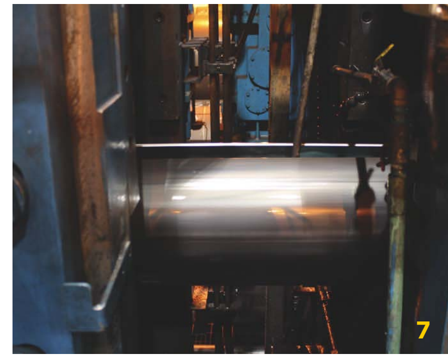
has and what length of steel is in the "storage queue" before the mill would need to be shut down. The transition is much like that made by club DJs segueing from one record to the next. A switch brings down the volume from the first record while gradually bringing it up on the second record. At the same time, the DJ can slow down or speed up either turntable so that the beat synchs up between the two songs, thus creating a seamless transition and the illusion of one continuous song. And like the HSS

…and rounder…

mill, he only has a certain amount of time to make it happen.

From Flat to Round

Once it's added into the ribbon, from the looper, the steel is pulled into the forming section. From this point forward, it is bathed in a synthetic coolant/lubricant along the line in order to dissipate heat and prevent "pickup" or steel adhesion to the tooling. The sheet is fed through a series of tooling that gradually forms the flat strip into a circle. (The forming section tooling is permanent and remains in place, but is adjusted for size made; the tooling in the remainder of the mill is changed for each tube diameter produced.) The edges are then welded together to complete the circle. Actually, "welded" is a misnomer in this case.



…and rounder.

They are actually forged together with heat and pressure. Ferrite is suspended inside the partially formed tube to direct the electrical current to the forge point. The outer-diameter weld slag (or "squeeze-out," as it is called) is scraped or "scarfed" from the tube to form a smooth seam (the other Independence facilities also perform inside diameter scarfing, which will be done at Decatur in the future). The strand of scarfed steel, much like the edge scrap cut via the slitter, is wound up into a separate roll, discarded, and eventually recycled.

The edges are heat-forged together to form a complete circle.

From Round to Square

From here, the newly formed round tube travels through a cooling trough before going through another series of tooling that will form it into a rectangular or square shape. (For tube that will end up as round, less tooling is required and part of this process is skipped.)

This "squaring section" uses a progression of three different sizes of tooling to press the round tube down on all sides into shape.

Newly formed tube enters the cooling trough before being cut or formed into a rectangular shape.

After each of the three points, you can see the tube gradually becoming closer and closer to being square. These dies are much smaller than the first set that forms the sheet into a round. Where the forming process uses both convex- and concave-shaped rolls that force the steel into shape, those in the squaring section press into the round at four distinct points, applying a precise level of pressure, to gradually "shape"

Steel is lubricated throughout the production line.

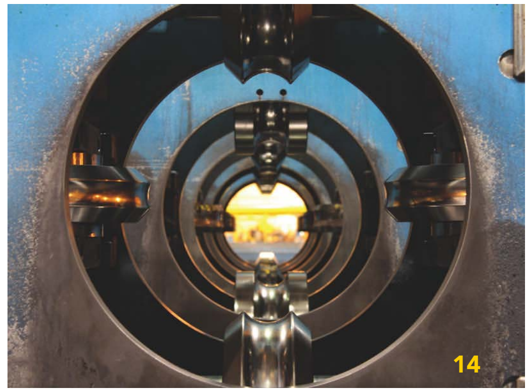
the round into a square or rectangle. As with the rollers that gradually bend the steel into round tube, these dies can be changed out to accommodate different sizes of square and rectangular product.

It should be noted that all tooling in all mill sections is hydraulically operated and adjusted. The mill also has the ability to repeat setups exactly, thanks to the computerized controls.

Round tube…

…is gradually pressed into…

...square tube...

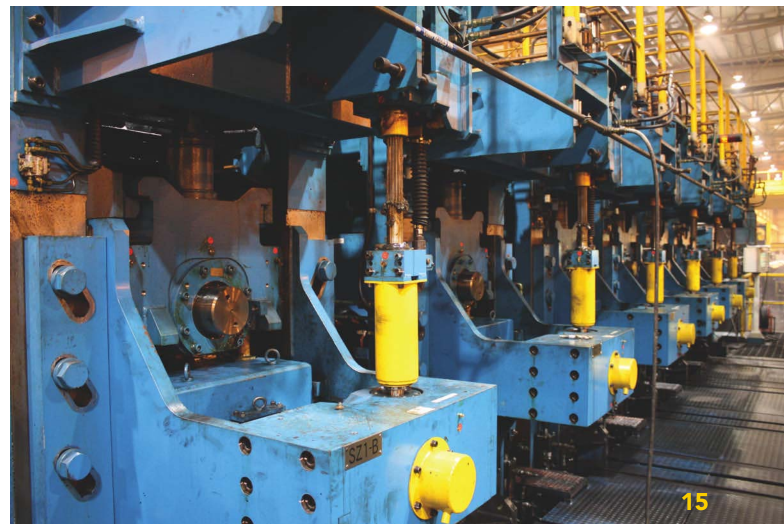


...using interchangable dies...

Making the Cut

At this point, the forming is complete and the tube is now ready to be friction-saw cut. The friction cutoff, which uses a 65-in.-diameter blade, is a giant circular saw that moves with the tube by temporarily attaching itself to it—so as not to stop the continuous ribbon—and making the cut on the fly, so to speak. The effect is somewhat like watching two space vessels docking with one another, then detaching, while both are on the move—or watching a cartridge in an office printer moving back and forth.

Once the cut is made, the saw detaches and retracts to its starting point, ready for the next cut. Then the steel, still steaming, rolls onto a cooling rack/conveyor. Once it reaches the end, it is lifted by magnets, stacked into bundles to meet the customer specification, and banded together by workers. From here, the bundles are transported via another conveyor into the storage area, ready for shipping. As with incoming coils, truck, rail, and barge are used to deploy finished HSS product, with 90% of product being shipped via truck.



...on this machine.

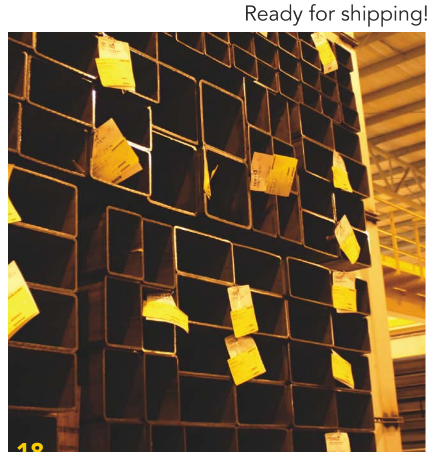
The saw attaches to the tube and cuts individual sections.

Ladders Too

In the end, what was once flat is now round—or square or rectangular, depending on the order to be filled. Whatever its shape or size, each member comprises an efficient, aesthetic part of a sturdy framing system. And a versatile one too; even the portable ladders in the plant were made from tube! MSC

A recently cut section, on its way to packaging.

Ready for shipping!



For updated information on HSS shape availability, visit **www.aisc.org/steelavailability**. And for all issues related to HSS, visit **www.aisc.org/hss**.

Also, several domestic HSS mills are opening their doors on AISC's inaugural SteelDay, which takes place September 18. For more information on SteelDay and a map of planned events, visit **www.steelday.org**.

September 18

September 18
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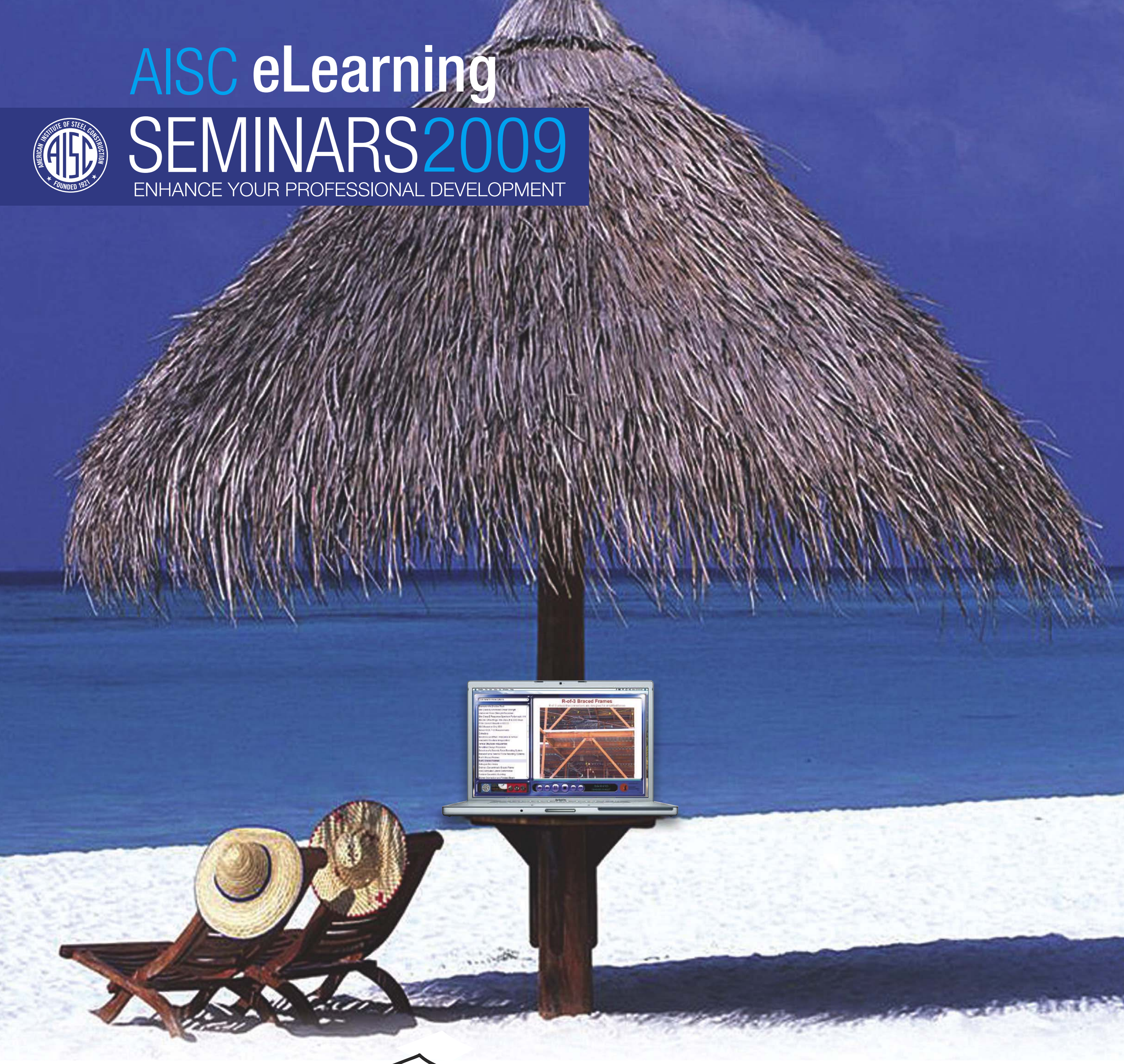

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A Strong Connection to HSS

BY M. THOMAS FERRELL AND ERIN CRISTE

An upcoming design guide expands AISC's library of resources on HSS connections.

ENGINEERS THAT INCORPORATE hollow structural sections (HSS) into their designs—or that are thinking about doing so—will soon have another resource on the subject. AISC *Design Guide 24* (with a working title of "HSS Connections"), to be released soon, supplements the information on HSS connections in the AISC *Manual*. Add in the 2005 AISC *Specification*, and designers will have a trifecta of go-to manuals for information and recommendations on designing with HSS.

For starters, *Design Guide 24* discusses the ASTM properties of round and rectangular HSS, noting the differences that exist between the properties and yield strengths of the various shapes so that engineers will know which members to specify in any given situation (see Table 1).

The Guide also provides a discussion of some of the 2005 AISC *Specification* provisions for HSS design. For example:

1. Rectangular and square sections that exceed the periphery limit in ASTM A500 of 64 in. are classified as box-shaped members, and the section properties must be obtained from the manufacturer.
2. Information in Chapter K of the 2005 AISC *Specification*, which provides the design criteria for the various forces in the HSS members and framing systems, is illustrated.
3. The nominal wall thickness of HSS members is not the design thickness for HSS, due to manufacturing tolerances. The 2005 *Specification* and tables in the *Manual* and *Design Guide* provide for 0.93 times the nominal wall thickness for HSS. This reduction applies to electric resistance welded (ERW) manufactured shapes, but not submerged arc welded (SAW) HSS.

Also of note, the 2005 *Specification* incorporates both ASD and LRFD, and the design examples in *Design Guide 24* are calculated using both ASD and LRFD as well.

Consider Cost

For economical HSS construction, the designer must consider the cost of the connections. *Design Guide 24* explains the various types of connecting devices, including welding and mechanical fasteners. The AISC *Manual* provides design guidance and examples for simple shear connections; this information is not repeated in the *Design Guide*. Generally speaking, Figure 1 (page 55) shows some comparisons of more and less economical connections. Always consult a fabricator, though, as the right conclusion on economy often depends upon the specifics of the project!

Table 1. Round and Rectangular HSS and Pipe

Name	HSS Rectangular	HSS Square	HSS Round	Pipe
Designation	HSS6x5x¼	HSS5x5x¼	HSS5.563x0.258	P5,P5X,P5XX
Usual material	A500 Grade B[1]	A500 Grade B[1]	A500 Grade B[1]	A500 Grade B[2]
Min. yield strength	46 ksi	46 ksi	42 ksi	35 ksi
Max. wall thickness	⅝ in.	⅝ in.	⅝ in.	Not limited
Max. periphery dimension	64 in.	64 in.	64 in.	Not limited
Alternative hot-formed grades	A501[3] and A618[4]	A501[3] and A618[4]	A501[3] and A618[4]	N/A
Alternative weathering grades	A847	A847	A847	N/A

1. The Guide describes the ASTM specifications for HSS and notes that ASTM A500 Grade B is the typical and preferred HSS designation for cold-formed members with an electric resistance welded (ERW) continuous seam. ASTM A500 Grade C HSS, which has higher yield strength of 50ksi for rectangular/square and 46ksi for round HSS, is becoming increasingly more available but its availability should be confirmed prior to specifying Grade C products. It is important to check with the manufacturer or fabricator for the availability of certain shapes and sizes before designing and specifying HSS (see **www.aisc.org/availability** for HSS manufacturers and availability).
2. ASTM A53 products are available only in round cross-sections.
3. ASTM A500 (cold-formed) does not contain notch toughness requirements but ASTM A501 Grade B (hot-formed) does have these requirements.
4. ASTM A618 is the designation for hot-formed high-strength (yield strength of 50ksi) structural tubing. This specification is used in high strength low-alloy applications for square, rectangular, round and special shape tubing.

M. Thomas Ferrell is president of Ferrell Engineering, Inc. He is an AISC Professional Member and a member of the AISC Committee on Manuals and Textbooks, the AISC TC6 Connections Committee, and the ASCE Committee on Design of Steel Buildings. Erin Criste is an AISC Steel Solutions Center advisor and can be contacted at ***criste@aisc.org****.*

Truss connections often involve directly welded HSS-to-HSS connections, and these are joints that can only be welded from one side. Economy requires that the designer consider this when selecting member wall thicknesses and specifying welds for these connections. Fillet welds are the most economical welds and should be used in HSS connections whenever possible. The *Design Guide 24* summarizes weld types that may be used in these joints and the limit states that must be considered by the designer. The 2005 *Specification* requires that both weld metal and base metal be checked for appropriate design limit states, and the new guide introduces the concepts of branch loads on HSS to determine connection nominal strength, including the principle limit states that apply. Several of these are illustrated for a rectangular HSS gapped K-connection, as illustrated in Figure 2.

Several types of mechanical fasteners used for HSS connections are listed in the new guide, and design examples are provided to illustrate design principles as outlined in the *Specification* for fasteners. These fasteners typically include:

- ➜ Through-bolts
- ➜ Screws
- ➜ Threaded studs
- ➜ Blind bolts
- ➜ Flow-drilled bolts
- ➜ Nails

Fasteners are generally categorized as subjected to shear or tension loading (although a combination of both can occur). Some specialty products exist for these applications, including Lindapter Hollo Bolt fasteners shown in Figures 3 and 4.

Another proprietary connection product available for HSS applications, bracing connections in this case, is the Cast ConneX High-Strength Connector, shown in Figures 5 and 6.

For more information on the Lindapter (**www.lindapter.com**) and Cast ConneX (**www.castconnex.com**) products, as well as other HSS connection products, please contact our Steel Solutions Center at **solutions@aisc.org**.

End Plates

The new design guide also discusses moment connections and tension/compression connections, including end-plate design. For moment connections, it lists types of connections and design examples for connections between W-shape beams and HSS columns in several configurations: continuous beams over HSS columns, through-plate

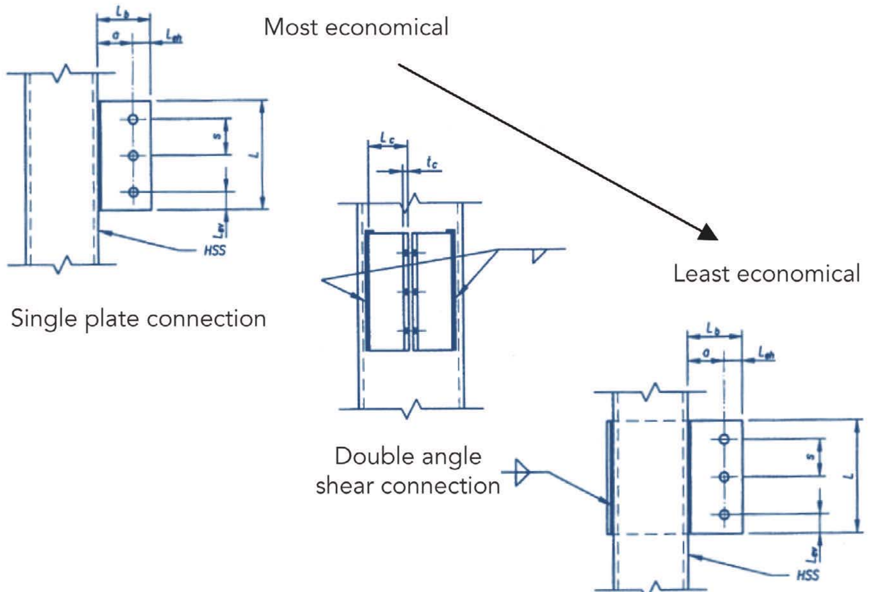


Figure 1: Typical simple shear connection for HSS.

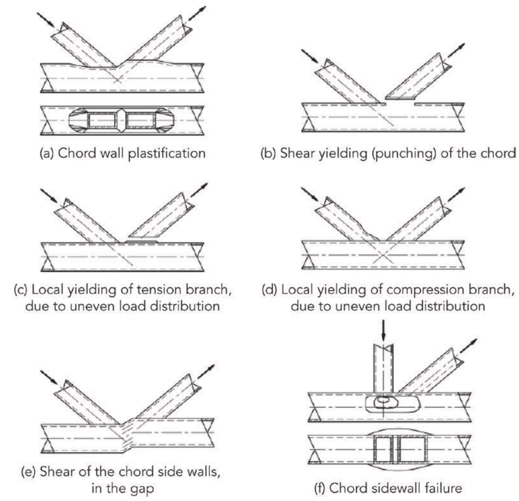
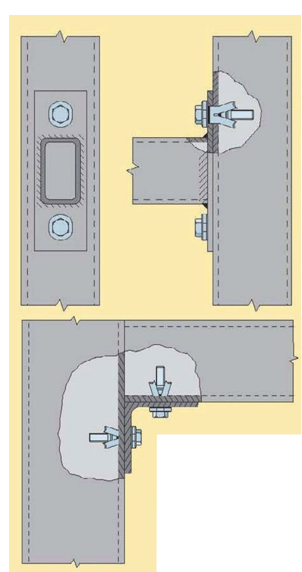


Figure 2: Typical limit states for HSS-to-HSS truss connections.

connections, and directly welded connections. It provides guidance for consideration of the effect of line loads and concentrated forces on the walls of HSS; HSS frequently are used as bracing components in bracing systems that are subject to tension and compression loads. In addition, the guide covers topics pertaining to end-tee, slotted-gusset, and end-plate connections with design examples provided. Figure 7 shows typical connection details for HSS.

In the coming months, check **www.aisc.org** for an announcement on when AISC *Design Guide 24* will be released (when it is, it'll be a free download at **www.aisc.org/epubs** for AISC members). And while you're at the AISC web site, take a look at **www.aisc.org/hss** for additional HSS resources. And again, you can always contact AISC's Steel Solutions Center via 866.ASK.AISC or **solutions@aisc.org**. **MSC**



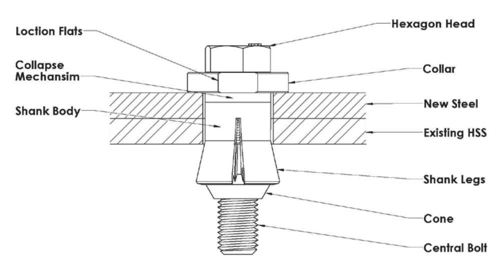


Figure 3: Hollo Bolt connection details.

Figures 3 and 4 courtesy Lindapter

Figure 4: Picture of Hollo Bolt connection details.

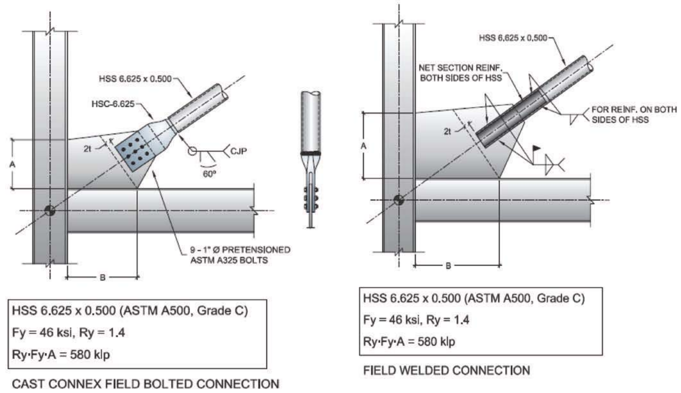


Figure 5: Comparison of Brace Detail.

Figures 5 and 6 courtesy Cast ConneX

Figure 6: Installed Cast ConneX brace.

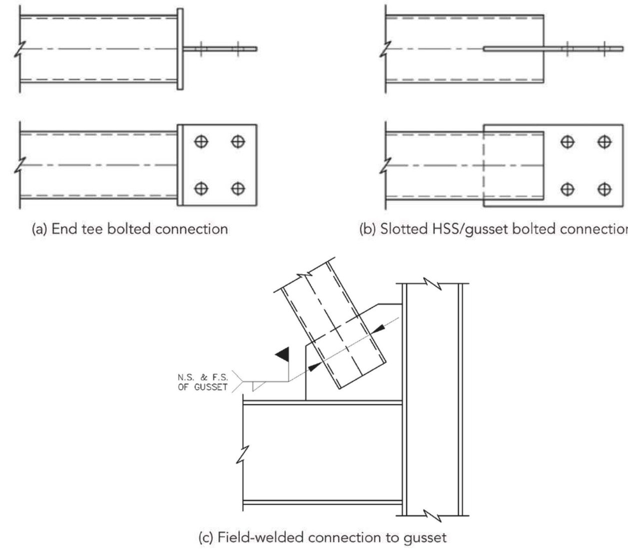


Figure 7: Bracing connections with bolts or welds in shear.

More on HSS

Experimental and theoretical studies conducted on HSS members and their connections since the 1970s form the basis for many of the current HSS requirements. CIDET (International Association for the Development and Study of Tubular Construction) offers HSS design guides to promote the use of HSS members.

In addition, the 1997 AISC *Specification for the Design of Steel Hollow Structural Sections* and the AISC *HSS Connections Manual* sparked significant growth in the use of HSS in building construction in the United States. Current requirements for the use of HSS in building design and construction have been integrated into ANSI/AISC 360-05 *Specification for Structural Steel Buildings* (the AISC *Specification*), which is available for free download at **www.aisc.org/freepubs**. The 13th Edition AISC *Steel Construction Manual* contains guidance and design aides for use in the design of HSS members and connections.

Checklists? You've got to be Kidding!

BY MARK W. TRIMBLE, P.E.

Checklists shouldn't be the entire quality program, but they shouldn't be left out of the program either.

BEFORE LEARNING TO FLY AIRPLANES, the creative side of my personality believed that being asked to follow checklists was demeaning and limited my creativity. To justify my resistance, I claimed to be looking for ways to improve the process through constant reevaluation. (Ha!)

Really, though, I just wanted to reduce the boredom of repetition—plus following someone else's way of doing things left the "me" out of the equation. It's amazing, though, how an out-of-routine experience can cause a profound change in thinking.

Learning to Fly

My first flying lesson was like that. Normally, during an introductory flight lesson, your instructor will have you doing tasks that you really don't feel ready to perform. In my case, these tasks included using my feet instead of my hands to steer the airplane during taxi and then trying to coordinate my feet and hands to accomplish smooth climbs and turns. During later sessions, I learned some very counterintuitive characteristics of airplanes, such as "sometimes when you want the airplane to go down, you point its nose up" and "the higher you fly, the faster you go—even if the airspeed indicator reads the same".

During this very intense learning environment, I felt very overwhelmed, wondering if I would be able to pull it all together into a cohesive understanding. My instructor kept reminding me to use my checklist, but I was so used to pulling information from memory, I seemed unable to think of the checklist as a tool. But, in an effort to minimize my stress and speed up my understanding, I learned to let the checklist be my friend. I stopped trying to remember every detailed step and just tried to remember to follow the checklist. Soon, with consistent use of the checklist, everything made better sense and I could relax—a bit.

In aviation, checklists serve two important purposes. For a student pilot, checklists are "what to do" lists, providing a systematic, proven way of ensuring that all life-protecting tasks are completed. These checklists have been prepared by professional aviators who, through experience and training, have determined the best way to help pilots and their passengers are safe. As the student pilot gains experience, the second purpose surfaces as he or she becomes more familiar with the "flow" of each process and begins to use the checklist in a different way. The pilot now knows what steps to take, remembers to take them, and uses the checklist only as a "did I do it?" list to confirm that all vital tasks have been completed.

Replacing Experience

So, how does this high-flying example relate to my company and yours? All companies are faced with employee turnover and the resulting knowledge vacuum that occurs when experienced people leave and inexperienced people are hired. Those of us who are the "experienced pilots" in our companies have a lot to offer those "student pilots" that we have just hired. The typical way we share this knowledge is to put the new hire in the "copilot" seat to observe the way that our seasoned pilot magically performs his or her duties. Through observation, the student will begin to figure out what the pilot is doing and then mimic the behaviors. The student may have the foresight to take notes for future use, but during the first few "flights" is generally overwhelmed with sensory overload and will likely overlook some key steps. Many, many flights are necessary before the student has an ingrained knowledge of what to do and is able to proceed without supervision. This is the first frustration: The student wonders what step to perform next and just can't seem to remember what the instructor did under those same circumstances.

As experience increases, the new employee gains confidence, but is consistently forgetting to do important steps. This becomes the second frustration: The instructor wonders, "Why does the student continue to forget? We have been over this so many times!" During solo flight, this lack of consistency can lead to an accident. In our businesses, this delay in learning can be costly to

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Quality Corner is a monthly feature that covers topics ranging from how to specify a certified company to how long it takes to become a certified company. If you are interested in browsing our electronic archive, please visit **www.aisc.org/QualityCorner**.

quality and profitability. Part of the blame for the student's inconsistency should be directed toward us, his instructors.

We seasoned folk often take for granted what we know. We forget that what we now consider to be common knowledge, we learned through many years of trial and error. This oversight is why we often question the actions of our new employees and we are disappointed when they do not make better decisions.

We cannot expect our new hires to know what we know, so why do we behave as if they do? If you want someone to make decisions like you would, then you must create an effective way for that knowledge to be transferred. The first step in that knowledge transfer should be creation of the checklist. Creating a checklist has benefits similar to other up-front planning activities; if you put most of the planning effort and decision-making at the beginning, the rest of your project will run much smoother.

Not a Measure of Quality

Before some of you get the impression that I am a supporter of checklist-based certification audits, let me take my aviation analogy a step further. A student pilot must be able to show compliance with the FAA's Practical Test Standards (PTS) before being allowed to become a licensed pilot or carry passengers. The PTS, in a manner similar to AISC's Certification Standard, provides an expectation of "quality" that the examiner compares the student's performance against. A checklist is not a measure of quality nor does it replace the Standard; it is simply an aid to use for training on existing processes and to provide your employees with a tool to verify that all steps in a process, procedure, or work instruction have been followed. Here are some things to consider when you create a checklist:

- Take the time to document each process you want to delegate
- Start with a simple list of questions that need to be answered before decisions are made
- Keep each checklist short. It is better to have several short checklists than one long one
- Group checklist items in small, easily memorized "chunks" of four to six items each
- Include all necessary decision-making and action steps
- Make your checklists outcome-based by including confirmation/verification steps
- Provide a place for the user to place a check mark and date when the step is completed

After some initial training, most anyone who has access to your checklist can follow your steps, just like you would. The result: You will become more comfortable with delegation, and your new employee will become more effective a lot sooner.

One of the major obstacles to creating an effective checklist is that the person creating the list (often you) believes that the decision-making process is far too complicated to document. Don't let that stop you! This roadblock can be overcome when the process is broken down into smaller sub-processes where the number of variables is greatly reduced. You will know when the process is subdivided in enough detail, because the checklist will just fall into place.

The other major problem with checklists is that, to be effective, the checklist must actually be used. A simple way to ensure use of checklists is to require that they be initialed and then attached to any transfer of documents or fabrication components. This way if a process is not followed, the only reasonable cause is that the person did not follow the checklist through to completion. Then, you can remind the offending party of proper checklist use.

A convenient way to avoid these two roadblocks is to involve others in the checklist creation process. You probably don't have all the answers, so ask some others to help. You may be surprised what great ideas will result. A likely result of this collaboration will be a checklist with shared ownership and a much greater chance of being used. By the way, not all checklists need to be printed on paper. There are some very inexpensive (some free) web-based applications for checklist input and documentation.

Creative Outlet

If, after reading this article, the creative side of your brain is still not convinced that checklists are for you, think on this. Try leading a checklist creation team. Let all of that creative spirit that's bottled up inside you flow through the creative steps of your checklists. You and your company will greatly benefit. MSC

Go Green or Go Home

BY TIMOTHY R. JOHNSON

Keeping with the green movement is crucial to recruitment and retention.

MOST HIRING FIRMS HOPE FOR that extra edge when looking to take on new talent. Sometimes that edge is a curve that firms need to keep ahead of, and knowing the growing trends is always important. As a recruiter for the building and design industry, I commonly ask recruited candidates, "What are you looking for in a new opportunity?"

More and more often lately, I've heard the response: "To work more with green building and sustainable design." With the baby-boomer generation retiring, and a shortage of new architects and engineers entering the industry, what exactly does going green mean for building and design firms in need of top-talent professionals? I'll break it down for you.

The Times, they are a Changin'

First, let's look at this in terms of keeping with the times. I'll quickly reiterate what is no doubt a commonly used, but highly appropriate, theme: The professional services sector is similar to, say, the business of product technology. Do you think a company that got its start selling AM radios over fifty years ago would survive today if it stubbornly refused to expand its offered products and manufactured only outdated audio products in a world of iPods and MP3 players? Likewise, as green building and sustainable design become the new hot trend in the building and design industry, it behooves firms that offer such services to keep with these trends.

Instead of attracting consumers to purchase products, put recruiting top-talent professionals under the keeping-with-the-times lens. What does this have to do with going green? More and more work in green building and sustainable design presents itself to the AE industry as time goes on. According to *Greener Buildings*, a report by market research firm SBI finds that "the booming green building market will continue its rapid expansion through 2011," increasing to more than $4.7 billion in that time. The report also notes that, while the growth may slow slightly in percentage—still remaining over 15% annually—the green building boom will continue despite the current building slowdown in the U.S. These findings suggest that continued work for those focused on green building and design will not only produce revenue—which is always a great thing, of course—but will also go far in attracting prospective employees.

One of the primary concerns recruited candidates raise, especially during an economic slowdown, centers on the workload of the hiring firm and uncertainty over a lack of work, which often leads to layoffs. Firms that focus on the growing market sector of green building and sustainable design will be more attractive to top-notch candidates for reasons of recognized workload availability, progressive growth, and employment stability.

> "...roughly 80% of the new wave of professionals entering the workforce wants jobs that have a positive impact on the environment..."
>
> —Monster Trak survey results

Environmental Interest

According to a recent survey, conducted by Monster Trak, the majority of college graduates today are seeking employment opportunities with companies that help the environment. The survey revealed that roughly 80% of the new wave of professionals entering the workforce wants jobs that have a positive impact on the environment, including work with green building and sustainable design. And if you want to take a moment to talk dollar signs, you might be interested to note that some respondents to the survey stated that they would work for less money if it meant being part of a more environmentally conscious organization.

In his book *101 Ways to Turn Your Business Green*, Rich Mintzer explains that, "as a green company, you will be in a far better position to attract top talent," due to the increasing number of environmentally conscious professionals entering the workforce.

Now I'm not saying that you need to start serving your employees all-organic food or go out and stock up on a two-year supply of carbon offsets, but what I am saying is that there is an increasing amount of available work out there in green building and sustainable design, a focus that continues to grow.

What can firms in building and design do to implement green practices and further develop services to include sustainability? A recent interview I conducted with ASHRAE (American Society of Heating and Refrigeration Engineers) Distinguished Lecturer, Vincent Sakraida, reveals some key points to consider.

Sakraida, a LEED Accredited Professional engineer and recognized leader with international engineering firm Jacobs Engineering, has been involved in green building design for more than twenty years. He breaks down the best approach to bringing a firm up to speed on green building and sustainable design services into four simple steps:

First, firms can become involved in prominent green building associations, such as the United States Green Building Council (USGBC). Step two requires the assessment of internal employee and overall firm experience with green building work and calls for the creation of a focused green building and sustainability group within the firm.

The next step is to implement a formal introductory training program for all employees, with an overview seminar on green building, sustainability, and LEED (Leadership in Energy and Environmental Design). Education programs should be established, as well, to provide continuous training and keep employees up to date on evolving trends.

The last step is to require professionally registered employees to becoming LEED Accredited.

Filling the Gap

Over the course of the next two decades, thousands of baby boomers per day will reach retirement age. Currently the number of new professionals entering the building and design industry is not enough to fill in all these gaps. The new professionals, perhaps more so than ever before, have their eyes open for opportunities in green building and sustainable design, recognizing it as an option of security and growth, the future of their industry. Indeed it is no coincidence that more than 50,000 people have become LEED Accredited Professionals since the certification began in 2001.

Firms in building and design that keep with the times and seek out more green focused work will not only do well to increase workload and revenue, they will also vastly improve their ability to recruit and retain top-talent professionals at a time when doing so becomes increasingly difficult. MSC

*Timothy R. Johnson in a principal at GCA International, an executive search consulting firm focused on the green building and sustainable design industry (**www.gcaintl.com**). He can be reached at **tjohnson@gcaintl.com**.*

Down South

BY ROB KINCHLER, P.E.

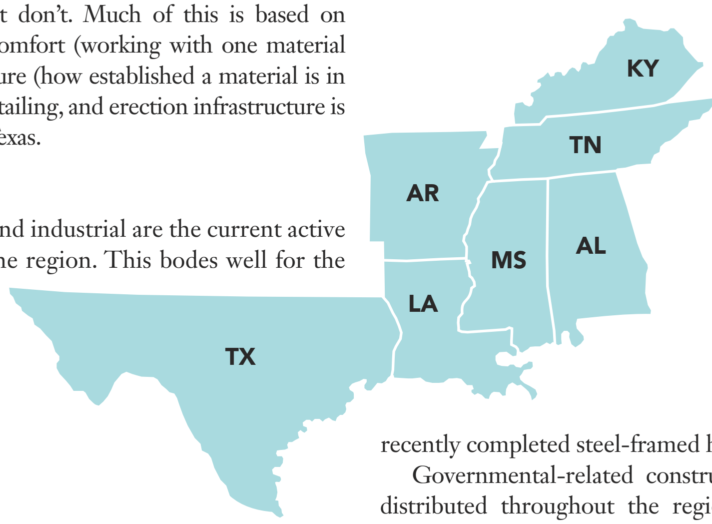
The South Central United States: Manufacturing a smart and healthy construction market.

FROM DEEP IN THE HEART OF TEXAS to the Bluegrass State's finish line at Churchill Downs (that's Louisville), the South never experienced the uncontrollable growth that benefited and now plagues other parts of the country. Over the past 10 years population growth in AISC's South Central region has been steady, fueled by northern migration south as well as immigration. This is not to say the region hasn't scaled back construction activity recently, but the downshift hasn't been as dramatic as in other parts of the country.

The South Central region, like all others, has pockets that prefer steel and pockets that don't. Much of this is based on regional preference due to comfort (working with one material over another) and infrastructure (how established a material is in the area). Steel fabrication, detailing, and erection infrastructure is very strong in Alabama and Texas.

Strong Markets

Health-care, education, and industrial are the current active and influential markets in the region. This bodes well for the steel industry, as we tend to do well in these markets. The steel market share in the region continues to climb rising by 7 points over the past 5 years. The large increase in market share may be due to the shift in the types of construction projects currently being built, but however you slice it the relative use of structural steel is increasing in the region.



The non-residential and multi-story residential building construction market in the region is currently running at an annual rate of 140 million sq. ft, down from 240 million sq. ft last year. School construction is currently the dominant market for the region, as 32% of all construction projects moving forward in the market are education-related. However, the industrial market should start to increase significantly in the coming months. Several industrial facilities are in the design phase throughout the region, particularly in the power industry. There are also heavy machinery plants, battery plants, steel plants, aircraft plants, solar plants, auto manufacturing facilities, and petroleum refineries scheduled for design and construction—even in this economy.

Elementary and secondary schools are being built all across the region and are keeping many fabricators busy. Alabama and Texas lead the region in new school construction, while Louisville, Ky. just passed a bond to allow for the construction of several new schools. In addition, Arkansas recently received $340 million dollars of stimulus money for school construction, and Louisiana's Central School District in Baton Rouge just passed a $55 million tax increase for school renovation and construction.

College and university construction also forges ahead. The University of Texas at Austin is currently spending $1 billion on construction projects. Some of these are being constructed now and many more projects are in the planning stages. Also in the area, Austin Community College is currently spending $200 million on a new campus in Round Rock.

Health-care remains strong, with several new steel-framed hospitals recently completed or under construction in the region and several others slated to start. Hillcrest Baptist Medical Center in Waco, Texas; Our Lady of the Lourdes in Shreveport, La.; The Baptist West Hospital in Jackson, Miss.; Norton Healthcare in Louisville; and the Women's Hospital in Baton Rouge, La. are just a few examples of recently completed steel-framed hospitals.

Governmental-related construction remains steady and is distributed throughout the region. New convention centers for Louisville and Nashville are being developed. BRAC (Base Realignment and Closure Act) work at military bases in the region is moving forward at Ft. Campbell in Ky.; Redstone Arsenal in Huntsville, Ala.; Ft. Bliss in El Paso, Texas; and Ft. Sam Houston in San Antonio.

Everything's Bigger in Texas

It's safe to say that the region is dominated by Texas. The Lone Star State makes up well over 50% of the construction market in the region and is currently the largest construction market in the U.S., with 11.5% of the total recently passing both Florida and California. Other hot construction spots in the region include northwest Arkansas, Nashville, and Huntsville, Ala.

Texas continues to build thanks to its population growth, proximity to the coast, and petro-

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leum and medical industries. Houston, the largest city in the state and region and fourth largest city in the United States, leads the way, as it hosts a huge petroleum industry and is a major U.S. port. In addition, the city's Texas Medical Center is a hotbed of research and patient care. The Medical Center's "central business district" of skyscrapers rivals that of many other U.S. cities' downtown skylines.

Austin is a dynamic city, with the University of Texas leading the construction parade. Austin's motto is "Keep Austin weird." And it certainly is unusual in the current economic climate. You wouldn't know there was an slowdown in this city; construction continues at a breakneck pace. There are still many condominium developers trying to move forward with new residential projects. They believe the demand is there, just not the financing. Austin is redeveloping its riverfront, building its educational infrastructure, and starting new solar plants. This is a high-tech, vibrant city.

While Texas has a strong construction market, it also has a strong concrete industry. This is one of the challenges the structural steel industry faces in promoting the use of steel. Thankfully, Texas also has a strong steel fabrication infrastructure, which can be mobilized to educate the community on how steel construction brings an as-yet-to-be-considered enhanced value to a project.

Northwest Arkansas is home to three Fortune 500 companies: Wal-Mart, J.B. Hunt, and Tyson. It is also home to the University of Arkansas. The towns of Springdale, Rogers, Bentonville, and Fayetteville have experienced large population growth over the last decade and continue to build to meet the needs of the community, as well as these large companies.

In Tennessee, Nashville is home to Hospital Corporation of America and Community Health Systems, the largest health-care providers in the United States, which build and operate hospitals all over the country—and they're currently working on building several more. Nashville's Vanderbilt University is also planning significant construction with its medical center and other areas of the university. The hot spots in town consist of Cool Springs (a mixed-use commercial/residential/office development south of town), the West End, and downtown. There are also still several residential developments on board that could be released for construction soon, and AISC has been making an effort to switch some of these projects from concrete to steel. The big-ticket item planned for Nashville is the new convention center.

In Alabama, Huntsville is heavily invested in aerospace, defense, biotechnology, and technology in general and has a very large technical/engineering workforce. More than 4,000 jobs are coming to the region as a result of BRAC, which is spurring development in and around Huntsville. In addition, the area's Cummings Research Park has several large office and technology projects planned.

What is AISC doing?

Sometimes, owners/developers are not even presented with a steel alternative. Whether steel brings a cost and/or time savings to the project cannot be determined unless it is given consideration from the beginning and an accurate estimate developed. AISC's regional engineers work with the AISC Steel Solutions Center—our in-house technical and conceptual solution assistance center—to help engineers find solutions that are beneficial to a project. We have assisted with multi-story residential, parking, office, and other project types. In my experience, the majority of the conceptual studies we provide clearly demonstrate the viability and economic benefits of using structural steel as compared to framing systems using other materials.

Typically if a comparison is made between steel and another material, it is only based on a typical bay and considers only the fabricated/erected price of steel including fire protection if required. But the benefits of steel construction extend far beyond the steel package price itself. Using steel on a project can bring significant savings in foundations, general conditions, cladding and curtain walls, equipment rental, and operational costs, as well as lower floor-to-floor heights and LEED credits.

We also help the project team find ways to get the steel delivered to the site as soon as the foundations are ready to accept them, employing resources such as service centers, early involvement, and building information modeling (BIM).

Recently in my region, I have been encouraging fabricators, service centers, and mills to become active in both AISC and their local fabrication associations. The idea is for the industry to better support one another, get better educated, and have a common message to present to the design community and to the project owners. I am also working on getting our industry to develop relationships with the project team and to become more proactive in a project, so that the project team realizes the benefit steel fabricators, erectors, detailers, service centers, and mills can bring to a project early on.

As I roam around the region enjoying meat and threes (that's Southern for "your choice of meat and three sides"), Krispy Kreme doughnuts, sweet tea, and southern hospitality in general, feel free to call me, and we can discuss how steel can be the material of choice for your next project. To learn more about my future travels and to contact me directly, please visit me on the web at www.aisc.org/myregion or e-mail me at kinchler@aisc.org. MSC

Each month MSC's product section features items from all areas of the steel construction industry. In general, these products have been introduced within the past six months. If you're looking for a specific product, visit MSC's online product directory at **www.modernsteel.com/products**. You can browse by product category or search on any term to help find the products you need, fast.

A One-Two Facial Protection Punch

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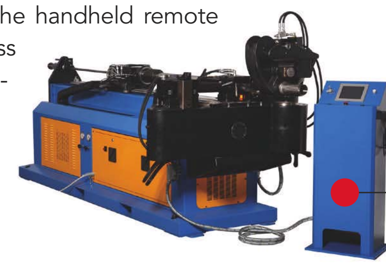

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All products submitted are considered for publication, and we encourage submittals related to all segments of the steel industry: engineering, detailing, fabrication, and erection. Submit product information via e-mail to Geoff Weisenberger (weisenberger@modernsteel.com). To be included in MSC's online products directory, contact Louis Gurthet (gurthet@modernsteel.com).

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Contract Auditor wanted to audit Quality Management systems, to AISC steel fabrication and erection certification criteria for Quality Management Company. Applicant must be willing to travel. Typical workload is 10-14 audit days per month, not including travel time. Audits are mostly in North America (90%), but also throughout the world (Asia, South America, Europe, Middle East). Prefer applicant with background in steel construction (fabrication or erection) and Quality system auditing, ASQ/CWA/CWI certifications a plus. International travel experience and multi-lingual abilities preferred but not required. Auditors must be fiscally responsible.

Please send cover letter and resume to:
Pat Thomashefsky, patt@qmconline.com. No phone calls please.

Marketing Director - NSBA

Are you a strategic and innovative marketer looking for an exciting challenge? AISC is looking to hire an accomplished marketing professional to take charge of its bridge division: the National Steel Bridge Alliance (NSBA). NSBA is the technical and marketing arm of the steel bridge community and is dedicated to increasing the market share of steel bridges.

The successful candidate will work with an oversight committee to develop and implement a strategic plan, including success metrics, incorporating three key areas: marketing, technical activities, and governmental action.

A degree in marketing plus a minimum of 10 years of experience with engineering or construction companies is preferred. Excellent presentation and management skills are critical. This position is based in our Chicago office. The ability to travel is required.

We offer an excellent salary and benefits package and numerous professional development opportunities. If you are interested in applying for this position, please forward your resume and cover letter, including your desired salary requirements, to Cathy Becker at: HR@aisc.org

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AISC Intermountain West Regional Engineer

Are you an engineering professional looking for a new challenge that's different and exciting? Are you good at promotion, overcoming road-blocks, working with diverse teams, juggling multiple tasks, and managing numerous high-level relationships? Are you ready to take the lead in growing the construction market for structural steel in the Mountain West?

AISC is looking for the right person to take the lead in promoting the use of structural steel in Montana, Idaho, Wyoming, Nevada, Utah, Colorado, Arizona, and New Mexico. Your success will depend on your abilities to develop and maintain relationships with key influencers, pursue and influence projects, work with owners and architects, make presentations, conduct seminars, and assist structural steel fabricators with promotional and business development programs. The ideal candidate will be a civil or structural engineer, or will have an architecture background, with a minimum of five years experience in building design, construction, and/or fabrication and a passion for consultative marketing. Strong communication and computer skills are a must.

We offer an excellent salary and benefits package and numerous professional development opportunities. If you are interested in applying for this position, please forward your resume and cover letter, including your desired salary requirements, to Cathy Becker at: HR@aisc.org

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NSBA - Southeastern Regional Director

AISC is looking for an engineering professional to take the lead in representing and promoting the steel bridge industry in the southeastern region of the U.S., using their understanding of bridge design and construction and relationships with Departments of Transportation, bridge consultants, and others to help facilitate the selection of steel as the preferred material for bridge construction.

The ideal candidate will have a Bachelor's degree and a minimum of 5 years experience in bridge design or construction. Experience with AASHTO Bridge Design Specifications is preferred. The ability to travel extensively in the region and attend industry events is required.

We offer an excellent salary and benefits package and numerous professional development opportunities. If you are interested in applying for this position, please forward your resume and cover letter, including your desired salary requirements, to Cathy Becker at: HR@aisc.org

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If You Want it Done Right, Do it Yourself

BY MATT THOMAS, S.E.

How hand-checks create the right balance, even in the digital age.

WE (ENGINEERS), both as a society and as a profession, have become dependent on our computers. They do everything we can imagine. They track our budgets, they enable us to draw and visualize, they allow us to communicate quickly with one another, and they perform design operations for our buildings.

As structural engineers, we use them in analysis and design to take over some of our fairly tedious, repetitive tasks like designing beams and columns. Theoretically, this allows us to design more efficient structures. But sometimes it seems that engineers, especially younger engineers, rely on computers too blindly, treating their results as gospel when using hand analysis could potentially show inefficiencies in their designs, and even catch costly design errors. We should endeavor to use these programs more cautiously.

One argument for caution in the use of computer design is that, especially for more inexperienced engineers, it doesn't allow you to "feel" the analysis and design of a building. That may seem like an oddly vague argument, but repeatedly doing hand calculations enables an engineer to gain invaluable knowledge about how a design is progressing and help develop that ever-elusive skill: sound engineering judgment.

Do enough calculations by hand, and you start to learn almost by instinct what loads a W12×19 will take or whether a W8×15 is a good "guess" size for a 30-ft-long girder (it isn't). These instincts will serve an engineer well. Oftentimes throughout design and construction meetings or site visits, architects, contractors, or engineers from other disciplines will ask if a beam or column size can be changed to accommodate a mistake, an enlarged duct, or some other design element. Being able to give an educated guess (to be confirmed by calculation later, of course) as to the answer is a valuable skill, which requires a thorough understanding of how loads will act on a member, as well as member capacities. It's tough to hone this skill by simply inputting loads into a spreadsheet or structural modeling program, which is, to some extent, a "black box"—you input the numbers, and the result spits out, with little knowledge of the steps involved in coming to that answer.

Matt Thomas is an engineer with Holabird & Root in Chicago and an AISC Professional Member.

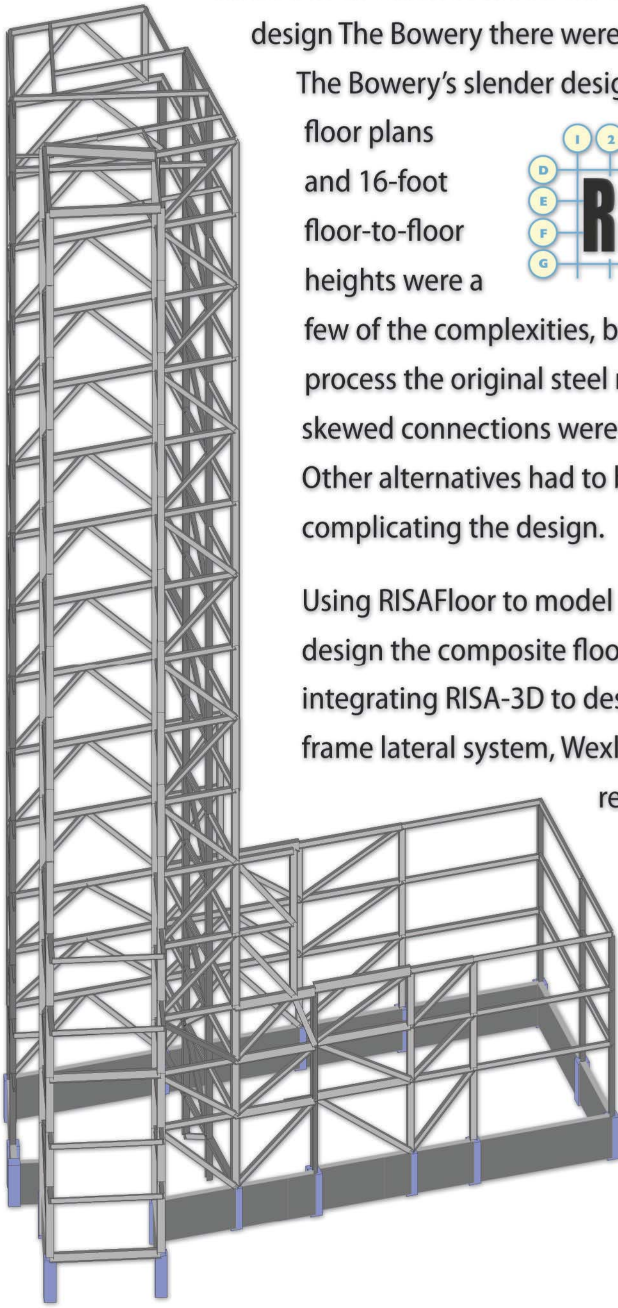
Additionally, computers can miss important design checks. You might build a model and run an analysis and design program on it, assuming that unless the model shows up red (indicating failed members), you're in the clear. But you might not know that the program won't check, say, torsion, unless you specifically ask it to. And the option to check torsion might be buried within a sub-menu. You could potentially design and issue documents for an entire building this way, without realizing you need stiffening for torsion or some other important piece of the puzzle. But if you're checking things by hand, you would (hopefully) know to calculate all the forces you'd need to check every time you analyze a member and after a while, you'd get a good sense of whether torsion, shear, moment, or anything else is going to be a problem for your particular design.

Consider, as a lesson, the Hartford Civic Center. Its cutting-edge, space-frame roof design required the engineers to use design assistance from computer programs. Many assumptions were made in the computer program, which, in reality, did not hold true. The assumed dead load was 20% too low, the true unbraced length of some members was double what was shown in the computer programs, and many brace connections had a true eccentricity that was not assumed in the computer analysis. These errors, along with a lack of proper oversight during construction, doomed the roof, which collapsed in 1978. A thorough hand check of the computer assumptions and calculations should have revealed the errors inherent in the design. How? Because a good hand analysis should force the engineer to consider the assumptions at each stage of design. At a less dramatic level, we've all seen projects where, because the computer's output wasn't thoroughly checked by engineers, costly reinforcement or more framing had to be added during the construction phase.

Conversely, computers can occasionally miss certain helpful design provisions, rendering a design overly conservative. For example, some computer programs ignore compression steel when calculating the maximum allowable reinforcement in a concrete beam. This results in an error from these programs, which would require you to make your section larger rather than simply being able to add compression steel to achieve the desired result. A hand check of the results may show you some avenues for providing an efficient design that might not be immediately apparent from a computer design.

I'm not trying to minimize the usefulness of computers in the design process. Computers aren't evil. They are incredibly useful tools that provide great benefits—when used properly. However, we must always temper their use with checks and balances. Remember that you weren't hired for your data entry skills, but for your sound engineering judgment. MSC

Have an opinion you'd like to share in "Topping Out"? Send your feedback to Geoff Weisenberger, senior editor, at weisenberger@modernsteel.com.

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