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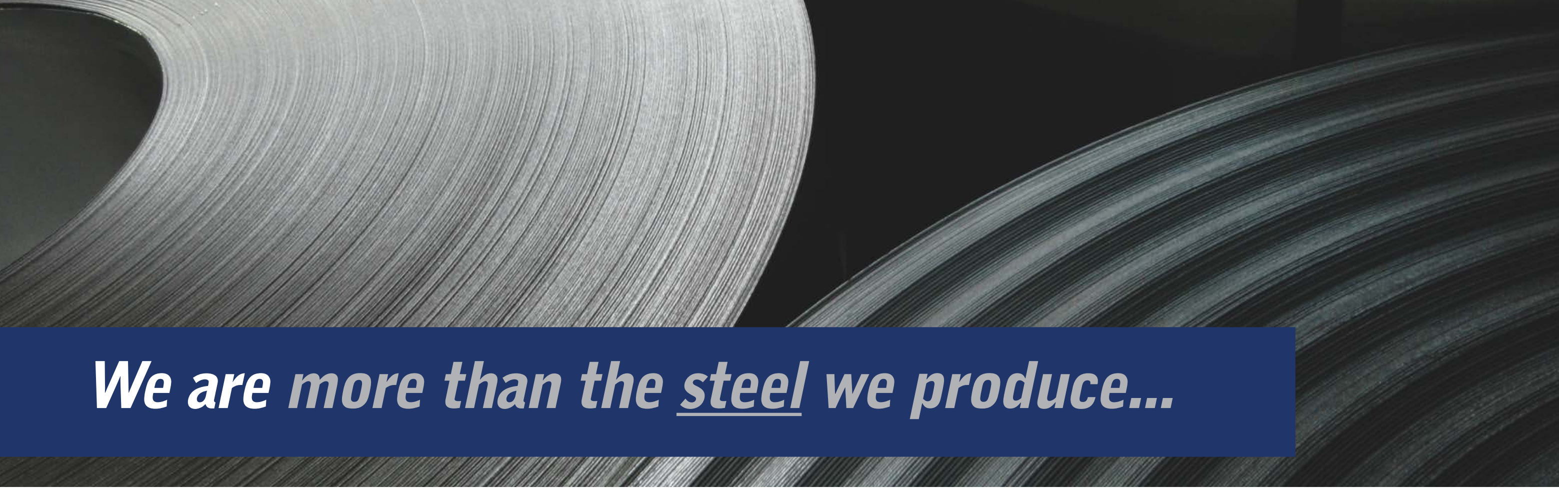

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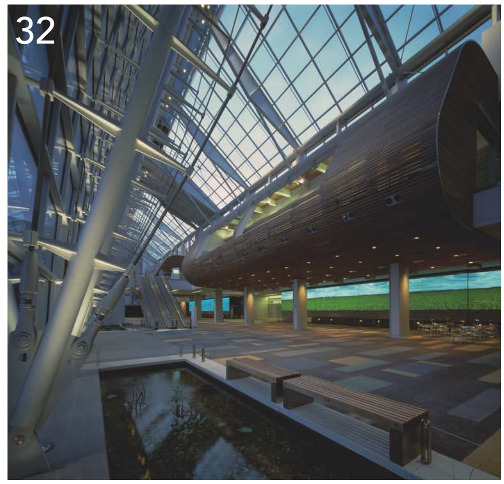

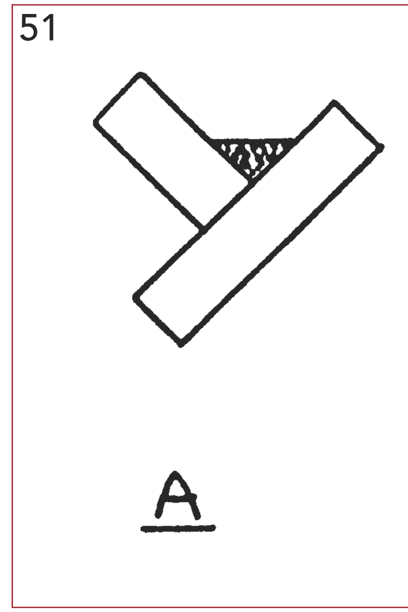

ON THE COVER: The Walter P Moore-designed Phantom Theatre at the Venetian Resort Hotel Casino in Las Vegas.

MODERN STEEL CONSTRUCTION (Volume 47, Number 10). ISSN 0026-8445. Published monthly by the American Institute of Steel Construction, Inc., (AISC), One E. Wacker Dr., Suite 700, Chicago, IL 60601. Subscriptions: Within the U.S.—single issues $3.50; 1 year, $44; 3 years $120. Outside the U.S.—single issues $5.50; 1 year $88; 3 years $216. Periodicals postage paid at Chicago, IL and at additional mailing offices. Postmaster: Please send address changes to MODERN STEEL CONSTRUCTION, One East Wacker Dr., Suite 700, Chicago, IL 60601.

editor's note

I USED TO THINK THAT JUGGLING WAS ONE OF THOSE NEARLY IMPOSSIBLE SKILLS THAT ONLY A FEW INCREDIBLY COORDINATED PEOPLE COULD ACCOMPLISH. Like everyone else, I had picked up three balls countless times, hurled them in the air, and watched them fall to the ground.

However, during my family vacation this summer to the Delawana Inn on the shores of Lake Huron, I attended a brief juggling clinic (I had wanted to see the instructor the night before, but that's a different story involving late-night shenanigans and misbehaving children). But after a simple 10-minute lesson (and plenty of sleep since I didn't stay up late for the juggling show—yes, I'm bitter), I could keep three balls in the air, at least for a few moments. I'll never be an accomplished juggler (okay, now *that* takes some serious skill), but basic juggling ended up being pretty easy.

It turns out that being "green" is a bit like juggling: The basics are easy; mastering the subtleties requires skill and practice.

On the most basic level, steel is inherently green. Today's wide-flange members come from 95% recycled material, and steel is completely recyclable. The mills have cut their energy usage by more than a third and reduced their carbon emissions enough to even exceed the Kyoto protocol. (If you want to know more about steel's green qualities, visit **www.aisc.org/sustainability** or the Steel Recycling Institute at **www.recycle-steel.org**.)

But a recent press release I received from the good folks at Side Plate Systems reminded me that there's a lot more to being green. The release talked about the need to design buildings that not only reduce material quantities, but also reduce the manpower needed to erect the structure. It explained that while steel is the greenest structural material, there's an ecological balance between reducing material and increasing manpower. That there's an even greater impact on the environment by the workers erecting a building than there is by the building material itself. And that the worst environmental impact is from the workers on-site (while shop workers, who usually live closer to the workplace, have a lesser impact). In other words, the press release reminded me of many of the advantages of structural steel construction—advantages that go far beyond just the amazingly high recycled content of structural steel.

Of course, being green can go even further. My friend Sylvie Boulanger (or Dr. Sylvie, as her fans know her) from the Canadian Institute of Steel Construction told me that some fabricators have taken being green to heart to such an extent that they've built a cogeneration plant to provide cleaner energy and minimize environmental impact.

And while it's not uncommon for companies to have light sensors that turn on lights only when a room is occupied, some companies are looking at skylights to reduce the need for artificial lighting. Others are instituting water management programs. And the best part? These techniques are not only good for the environment, but also for a company's pocketbook.

So don't just be content with knowing that being part of the steel design and construction industry by definition makes you a green consumer. Go beyond the obvious and look at your processes and designs. Think beyond just getting your LEED points and consider what will truly make a difference. Even if you only switch from incandescent bulbs to compact fluorescent bulbs, it will make a difference.

Scott Melnick

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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.

Penetration into the Base Metal

I am concerned with a fabrication shop that is using E70C-6M. When inspecting heavy columns, I noticed the welds at the gusset plate have little or no penetration into the base metal. My wire book is saying the wire is good on carbon steel up to 70 ksi. Is the gas-shielded metal-cored wire, which I see as having little penetration, acceptable on structural steel because the drawing calls for the use of E70XX filler metal?

Duane Miller of Lincoln Electric provided his thoughts:

The "key" to welding is fusion, not necessarily penetration. The two often get confused.

Fusion is achieved when filler metal and base metal are melted together and fused together. Penetration refers to a specific amount of melting into the base metal. Typically, when penetration is achieved, fusion is assured. However, if penetration is minimal or nonexistent, then fusion is often uncertain (as appears to be the case in this situation).

To your question regarding acceptability, the answer is yes; the filler metal meets all the requirements of AWS D1.1:2006, Table 3.1. The AWS classification of E70C-6M meets the requirements shown on the drawings for E70XX. However, I hasten to add the caveat that, as with all welding, the welding parameters (amps, volts, travel speed, preheat, material cleanliness, etc.) must be such that proper fusion is achieved. With your report of "little or no penetration," you should not assume that fusion is being achieved, even though the electrode meets the AWS D1.1 requirements.

There is no inherent problem with the use of this particular classification of electrode, but as is the case with all electrodes, improper procedures and techniques can result in poor quality welds. If there is no fusion, there are major procedural problems that must be addressed. Short-circuiting transfer associated with GMAW welding, low currents, small diameter electrodes, and other factors can all lead to conditions wherein fusion is not achieved.

Rather than focusing on the electrode, the concern should be on the welding parameters and quality. Make sure the WPS meets the requirements of AWS D1.1 and that the electrode is operated within the manufacturer's recommended parameters. If there are remaining concerns about the suitability of the welding electrode and the welding procedure parameters being used, it would be appropriate to perform some mechanical testing on welded connections. Simple fillet weld break tests, which are normally used for tack welder qualification, can be used to identify fusion problems. See AWS D1.1 Figure 4.35 for an example of such tests. A good weld will fail through the throat. If fusion problems exist, failure will be along the face of the base metal.

Section Properties

I am trying to calculate the I_x values found in Section 1 of the 13th edition AISC manual. The numbers that I get are close, but don't quite match what is printed. Are you including the fillets in the calculation? What shape do you assume they are? Where is the thickness of the flange equal to t_f when shapes have an inner flange surface slope? Is it at the location half way between the face of the web and the end of the flange?

Yes, the fillets are included in the calculation of the section properties listed in the AISC manual tables. The fillet radii used are based on surveys we conduct of fillet practices used by producing mills, and the calculations are based upon a parabolic fillet in a right angle as shown on page 17-40 of the 13th edition AISC *Steel Construction Manual*. The t_f of a shape with a sloping flange depicts the average flange thickness taken between the tip of the flange and the transition point to the fillet.

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

PJP Weld Capacity

When AISC gave a seminar on the new code, they gave out laminated "short-cut" cards that identify the ASD tension capacity of a PJP groove weld as $0.32F_{EXX}A_w$. I am having trouble coming up with the origin of the 0.32 factor. For ASD I have always used an allowable shear stress of 0.3 times the nominal tensile strength of the weld metal. Can you explain the difference?

I think that your confusion stems from the distinction between the directions of applied load with respect to the axis of the weld. You will note that the laminated cards give a value of $0.30F_{EXX}A_w$ for shear on the weld, but $0.32F_{EXX}A_w$ for tension on the weld.

Available strength of welded joints is covered in Table J2.5 of the AISC specification (a free download at **www.aisc.org/2005spec**). Note that if you are looking at *tension* normal to the weld axis, the nominal strength is $0.60F_{EXX}$, and Ω = 1.88. If you are looking at *shear*, the nominal strength is $0.60F_{EXX}$, and Ω = 2.00. It will thus follow that the allowable ASD coefficient for the tension case would be 0.60/1.88 = 0.32, while that of the shear case will be 0.60/2.0 = 0.30.

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

Single-Angle Bending

Upon reviewing a condition I have encountered of an unequal-leg single angle bent about a geometric axis (X-X in this case) with no restraint against lateral-torsional buckling along its length, I do not see in AISC specification Section F10-2 any subsection that encompasses this condition. Is it advisable to assume that since this condition is not explicitly covered with a definition for M_e, that there is no case in which LTB will cause the failure of an unequal-leg single angle bent about a geometric axis? If not, which equation would be applicable to determine the LTB criteria for this condition?

No, lateral-torsional buckling does apply. Refer to the commentary of section F10 at the top of page 16.1-282 for your condition.

The biaxial bending can be treated using the provisions in Chapter H to combine the forces around the principal axes. Leg local buckling may also limit the flexural strength for noncompact angle legs as per Section F10.3

Amanuel Gebremeskel, P.E.

Slip-Critical Surface Classes

AISC 341 lists only Class A surface preparation for bolted connections that are part of the SLRS. This would appear to eliminate galvanizing as a corrosion control method, since, when roughened, it is considered a Class C surface. Is this correct?

No. There is no longer a Class C surface designation in the AISC specification. Class A surfaces include hot-dip galvanized with roughened surfaces. See Section J3.8 of the AISC specification (a free download at **www.aisc.org/2005spec**) for guidance.

Note that the RCSC specification has not been updated since the release of the 2005 AISC specification, and, therefore, the current 2004 release still contains the Class C reference.

For further discussion on the subject of combining the formerly separate Classes A and C into a single Class A, see the Commentary to Section J3.8 of the 2005 AISC specification.

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

Edge Distance for Anchor Rod Holes

What and where are the requirements for edge distance for the recommended maximum-size holes for anchor rods listed in table 14-2?

AISC does not directly address edge distance requirements for anchor rod holes in base plates. This is a matter of engineering judgment because only the designer can anticipate what types of loads the hole is likely to see. See FAQ 7.1.7 at **www.aisc.org/faq** for further discussion on the subject.

It is generally not recommended (see AISC Design Guide 1, *Column Base Plates*), but if shear is being transferred through anchor rods and the engineer can anticipate which anchors can reasonably be expected to bear against the oversized holes, the provisions of section J3.4, J3.10, and J4 may be helpful.

Amanuel Gebremeskel, P.E.

Historic Steel Beam Designation

We are renovating a job dated 1914. Beams are called out as 9×21. Is this an "I-shape," 9 in. deep at 21 lb. per ft? Is there a source for the old beam properties?

Yes, it is very likely an I-shape, and yes, there are sources that contain information on old beam shapes. The early 1900s era preceded the formation of AISC and the publication of the AISC manuals. However, AISC has Design Guide 15, which is a reference for historic shapes and specifications. This guide includes information on many shapes that predated the AISC manuals. The document is available at **www.aisc.org/epubs**. AISC also has a shapes database (v13.0 for contemporary shapes and 13.0H for historic shapes) available on the same web site.

It is likely that the 9×21 shape of your inquiry was an "American Standard Beam" produced by one of the mills of the time. Today these are called S-shapes and are listed as such in the design guide. There are multiple listings for shapes with the S9×21 designation because there was more than one producer of the shape and the properties may not have been identical. Despite the variations in cross-section, the S9×21 would be nominally 9 in. deep and weigh 21 lb per ft.

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

Single-Plate Connections with Axial Loads

Is it acceptable to design single-plate connections for shear and axial loads? If so, what are the design criteria for the axial load?

The tabulated values for shear connections in the AISC manual do not address beam connection with axial loads. However, yes, it is acceptable to design single-plate connections for shear and axial loads. The same limit states that are covered in Chapter J of the AISC specification (such as bolt shear, bearing strength at bolt holes, tension, shear, block shear, etc.) would be applicable.

The extended single-plate connection procedure provided on page 10-102 is intended as a starting point for cases that deviate from the limitations imposed on the more traditional conventional configuration. It is used in combination with consideration of axial force in several examples in the AISC *Seismic Design Manual*. As one example, see Example 5.2.

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

Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is AISC's chief structural engineer, and Lou Geschwindner is AISC's vice president of engineering and research.

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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.

This month's Steel Quiz was developed by AISC's Steel Solutions Center. Sharpen your pencils and go!

1 **True or False:** When checking plates that are loaded in compression, the designer needs to consider the limit state of local buckling.

2 How many methods of second-order analysis are included in the 2005 AISC specification?

3 Does AISC provide a method for designing single angles?

4 Which document specifies the design, installation, and inspection requirements for joints that make use of high-strength bolts?

5 What is prying action and how is it accounted for?

6 In composite floor framing, what defines the minimum required flexural strength of the steel section alone for unshored construction?

7 Are channel shapes available as Grade 50?

8 **True or False:** An HSS beam to HSS column moment connection in a lateral force resisting frame is not prequalified for use in a special moment frame (SMF).

9 When is the use of tension-only bracing permitted in Seismic Design Categories D, E, and F?

a. Tension-only bracing is not permitted for use in SDCs D, E, and F.

b. Tension-only bracing is permitted for use only in special concentric braced frames (SCBF)

c. Tension-only bracing is permitted for use only in ordinary concentric braced frames (OCBF)

10 **True or Flase:** The AISC *Code of Standard Practice* permits the use of architectural, electrical, and/or mechanical design drawings to show requirements for quantities, sizes, and locations of structural steel where it pertains to those trades.

TURN PAGE FOR ANSWERS

steel quiz

ANSWERS

1 False. Chapter E of the 13th edition AISC specification (available at www.aisc.org/2005spec) checks global buckling of such members. Since the mechanism is the same for local buckling in a plate, local buckling need not be checked.

2 Three. The effective length method and the first-order analysis method are included in Section C2, and the direct analysis method is included in Appendix 7. These three methods are explained in greater detail beginning on page 2-10 in the 13th edition AISC *Steel Construction Manual*.

3 Yes. Section F10 of the 2005 AISC specification and its commentary provide methods for analyzing and designing single angles with equal and unequal legs. The section considers both geometric and principal axis loading for both torsionally restrained and unrestrained conditions.

4 The 2005 AISC specification (Section J3) does, although there are many references from this specification to the 2004 RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*. The latter document is published by the Research Council on Structural Connections and is available as a free download at www.boltcouncil.org. The 13th Edition AISC manual (available at www.aisc.org/bookstore) also includes a copy of this document.

5 Prying action is a phenomenon in bolted construction that affects bolt tension forces. When a fitting, such as an angle, has a bolt that passes through it while transmitting tension, deformation of the fitting can increase the tensile force in a bolt above that due to the direct tensile force alone. Proper design for prying action includes the selection of bolt diameter and fitting thickness such that there is sufficient stiffness and strength in the connecting element and strength in the bolt. The procedure for this design is found in Part 9 of the 13th edition AISC manual starting on page 9-10.

6 Per Section I3.1c of the 2005 AISC specification, the steel section alone shall have adequate strength to support all loads applied prior to the concrete, attaining 75% of its specified strength f'_c.

7 Yes. While channel shapes are still rolled most predominantly to meet the ASTM A36 standard, it is becoming more common to find channels rolled to the ASTM A572 grade 50 (or ASTM A992, in some cases) standard. One can check availability by contacting a steel fabricator, steel service center, or mill. For a listing of steel mills, service centers, and material availability, visit www.aisc.org/availability.

8 True. There are no HSS beam to HSS column moment connections that are prequalified for use in SMF in AISC 358-05 (available at www.aisc.org/aisc358), or in FEMA 350. If desired, such a detail can be qualified by testing per Appendix S.

9 c. Tension-only bracing is permitted for use only in OCBF; see Section 14.2 of the 2005 AISC *Seismic Provisions* (available for free download at www.aisc.org/2005seismic).

10 False. Section 3.2 of the AISC *Code of Standard Practice* (available for free download at www.aisc.org/code) specifies that all quantities, sizes, and locations of structural steel must be shown or noted in the structural design drawings. It is permitted to use the other trade drawings to define detail configurations and construction information of structural steel.

AISC to Offer Online Education Series

Beginning October 1, AISC will introduce a monthly series of online audio and video short-courses at **www.aisc.org/onlineseminars**. These 1- to 1.5-hour courses can be viewed for free, and after passing a test on the course content, a CEU form valid in all states can be purchased for only $25.

The scheduled courses currently include the following:

Release Date	Author	Title
Oct. 1, 2007	Ted Galambos	Shakedown Behavior of Steel-Framed Structures
Nov. 1, 2007	Stan Rolfe	Fatigue and Fracture Control in Steel Structures
Dec. 1, 2007	Geoff Kulak	High-Strength Bolting: The Essentials
Jan. 1, 2008	Ron Ziemian	Basic Introduction to Nonlinear Analysis
Feb. 1, 2008	Shankar Nair	A New Approach to Design for Stability
March 1, 2008	Greg Dierelein	Seismic Design and Behavior of Composite RCS Frames
April 1, 2008	Mike Engelhardt	Basic Concepts in Ductile Detailing of Steel Structures

SSTC Fall Seminars

The Steel Structures Technology Center has announced three one-day, seven-hour seminars and a two-hour evening seminar. All four seminars are conducted in cooperation with the International Code Council (ICC).

Structural Steel and Bolting Inspection includes International Building Code (IBC) special inspection requirements, steel materials, steel fabrication and erection, and high-strength bolting (one day).

Structural Welding Inspection includes IBC special inspection requirements and welding inspection under American Welding Society (AWS) structural welding codes (one day).

Inspection of Seismic Steel Frames includes AISC, IBC, and AWS requirements for connection details, welding, bolting, inspection, and nondestructive testing for steel buildings designed to the AISC *Seismic Provisions* (one day).

Plan Reading for Steel Construction includes structural design and shop drawings (two-hour evening format).

For seminar dates and more information, visit **www.steelstructures.com**.

IABSE 2008 Call for Papers

The International Association for Bridge and Structural Engineering has issued its call for papers for the 2008 IABSE Annual Meetings and Congress. The deadline for abstracts is October 31, 2007. Full papers based on accepted abstracts (acceptance will be verified by December 31) will be accepted until February 29, 2008. The annual meeing will take place September 14-19 next year in Chicago. Themes and topics include design challenges, learning from experiences, creative design and construction processes, and engineering as a global profession. For more details about paper topics and for more information about the conference, visit **www.iabse.org/chicago08.**

2007 World Steel Bridge Symposium Coming to New Orleans

The 2007 World Steel Bridge Symposium marches into New Orleans in just a couple of months. Hosted by the National Steel Bridge Alliance, a division of AISC, the annual event will take place December 4-7 at the Sheraton New Orleans Hotel. WSBS will host steel bridge owners, designers, and contractors from around the world to discuss all aspects of steel bridge design and construction. The exhibit hall will be full of products and services to advance the state-of-the-art of the steel bridge industry, and attendees will learn about the latest innovations in steel bridges.

Program features will include:

- Short-span bridges
- Intermediate-span bridges
- Case studies featuring the use of high-performance steel
- Modular and accelerated bridge construction
- Restoration, rehabilitation, and re-use
- Fabricating, constructing, and erecting
- Innovative bridge designs
- Inspection and maintenance

If you are interested in exhibiting or sponsoring at the 2007 WSBS, please contact Jody Lovsness at 402.758.9099 or lovsness@nsbaweb.org. For more information on the symposium, visit **www.steelbridges.org**.

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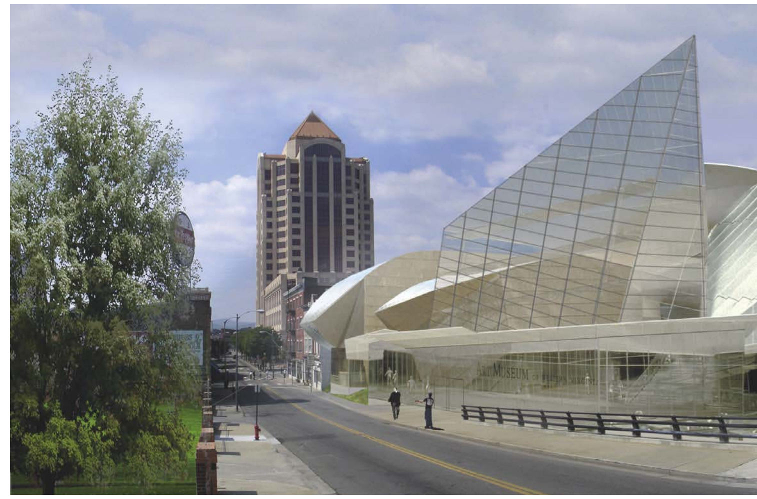
PROJECTS

Art Museum of Western Virginia Tops Out

This summer, the Art Museum of Western Virginia, located in downtown Roanoke, celebrated the topping out its new museum building. The new 81,000-sq.-ft facility is being built to better accommodate the Art Museum's growing collection and enable it to meet the demands for its education and outreach programs.

The building will be a dramatic composition of flowing, layered forms in steel, patinated zinc, and high-performance glass, paying sculptural tribute to the nearby Blue Ridge Mountains.

Construction remains on schedule. Installation of the metal panels that form the exterior wall enclosure has begun, and numerous ZEPPS panels (Zahner Engineered Profiled Panel System), which are used to provide the protruding, cantilevered edges of the building's undulating roof, have been installed. Once all of the panels and the remaining roofing substructure are in place, the structure will be ready for the stainless steel roof application.



Designed by Los Angeles architect Randall Stout, the Art Museum's steel was fabricated by Superior Steel, Knoxville, Tenn. (AISC Member). The new facility is scheduled to open in the fall of 2008.

Correction

The fabricator of record for the new Dallas Cowboys Stadium was listed incorrectly in last month's issue. The prime steel contractor was W & W Steel, which subcontracted some of the secondary support trusses to Prospect Steel. AISC regrets any confusion caused by the error.

letters

Structure-after-design

Most contractors will agree that cost is designed in. Customer requirements and site conditions, blended with architects' willingness to surpass themselves, often result in projects that seem to be reaching new heights in cost. That said, as steel detailers we recommend that the customers not only discuss their projects with the architects, but also with the structural engineering group. Get a sound structural design recommendation first and then walk it though with the architectural firm in order to get the body and shape onto the skeleton.

I really enjoyed "Banking on Sustainability" (July MSC, p 26). This is exactly what we recommend: a fresh look at the dynamics of doing things. I congratulate Mr. Christensen on his approach and I hope that it confirms a trend in the industry. I'd like to add that you can *bank* on the experience of the structural engineering group every time.

As steel detailers we are quite far down the food chain and have little say in major projects. We leave this to the structural engineering group, but in many cases they too have their hands tied. Let's change that.

Mario Lapointe
North American Steel Detailing

On a Misleading Note?

Zoruba and Liddy did an excellent job of outlining the specifications relevant to current structural steel (SteelWise, March 2007). But in one of their listed items, namely Direct-Tension-Indicator Washers, ASTM F959, they offered a note "e" to their table that stated: "Washers that express colored dyes when compressed are not covered by ASTM."

This note may be misleading because our Squirter DTI washers are produced to the requirements of ASTM F959 and installed according to the corresponding procedures in the RCSC specification. The orange silicone itself is not covered by ASTM, but a DTI having this feature can still be approved on jobsites and used exactly as a non-squirting DTI is used.

We believe the squirt feature, when calibrated on Skidmores on bolts at jobsites, as the manufacturer recommends, can enable the bolt installers and inspectors to be better and more efficient, and therefore improve the constructability of steel structures.

Chris Curven
Applied Bolting Technology Products

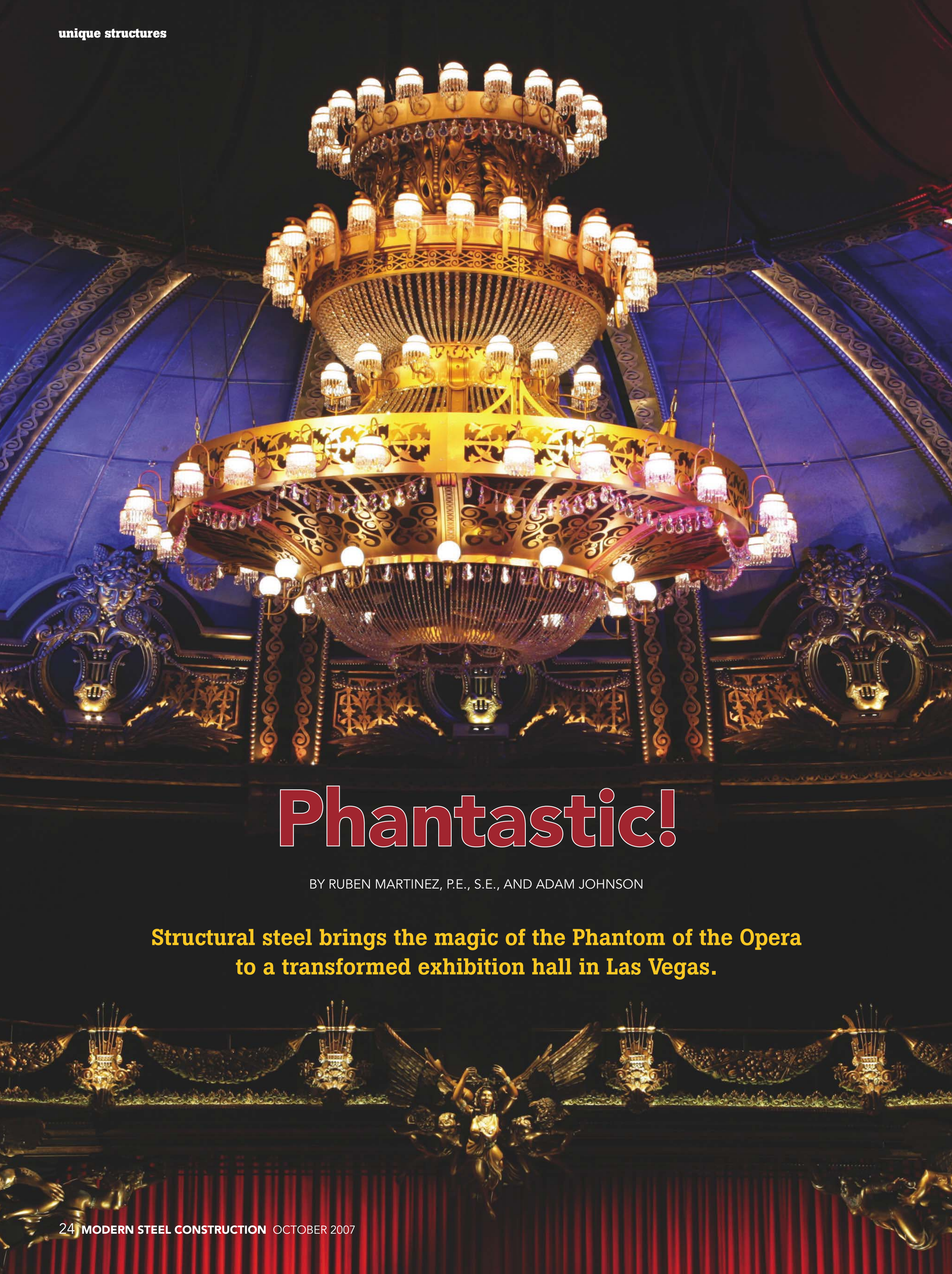
Charlie Carter, P.E., S.E., chief structural Engineer with AISC, responds:

Thank you for helping us to clarify our intent. We should have said that, when washers that express colored dyes when compressed are used, they are used following the same procedures as conventional DTIs.

No Mixing, No Welding

I would like to augment the answer that was given to the first question in the August Steel Quiz, which asked about the difference between "filler metal" and "weld metal." The term filler metal refers to the chemistry and physical properties of the welding metal by itself. The term weld metal refers to the chemistry and physical properties of the weld deposit. This metal is a combination of the effects of the welding process, the chemistry of the filler metal, and the chemistry of the base metal. The answer, as given, did not include the effect of the base metal. If there is no mixing of the filler metal and base metal, the joining process is either brazing or soldering, not welding.

D. Robert Lawrence II, CWI, CWE



Phantastic!

BY RUBEN MARTINEZ, P.E., S.E., AND ADAM JOHNSON

Structural steel brings the magic of the Phantom of the Opera to a transformed exhibition hall in Las Vegas.

“PHANTOM—THE LAS VEGAS SPECTACULAR,” A RECREATION OF THE FAMED MUSICAL “THE PHANTOM OF THE OPERA,” RECENTLY OPENED TO RAVE REVIEWS AND SOLD-OUT CROWDS AT THE VENETIAN RESORT HOTEL CASINO. The lavish $40-million, 1,800-seat Phantom Theatre is the result of painstakingly tailored design and use of structural steel to transform an existing exhibition hall into the show’s Paris Opera House setting.

The outcome is magnificent—beautiful finishes, an impressive domed ceiling, and thrilling performances and special effects, particularly an operable 2,100-lb Baroque-style chandelier that transforms from a dilapidated state back to its original grandeur while gliding up to the ceiling.

Scene 1: Laying the Foundation

Micropiles were the foundation system best suited for the constrained project site that offered limited access for construction equipment. Grouted shafts were drilled up to 75 ft deep. Under steel columns, pile caps utilized groups of three or four micropiles. The tops of the micropiles—which were designed to carry downward forces as well as uplift and lateral seismic forces—were simply cast into the walls to support the heavy loads from the concrete stage house. Using a smaller-than-typical drill rig, the design team placed column foundations and walls in very the tight spaces created by the existing structure and its neighbors.

Scene 2: Stage House Box

Most of this spectacle’s magic happens in the stage house. At the east end of the theater, the massive concrete stage house is 125 ft long by 45ft wide by 112 ft tall. At the front of the stage house is the 55-ft-wide by 35-ft-tall plenum that opens to the theater and through which the audience watches the show. At the roof level, the 18-in.-thick concrete walls support steel roof framing, which in turn supports an intricate system of pulleys capable of flying over 80 tons of scenery.

To resist the large downward and lateral forces imparted by the pulleys at the counterbalance point, a built-up steel beam—consisting of a W36×230 stiffened by a horizontal W36×160 welded to the web and stiffened with ⅝-in. plates—was required. A steel gridiron hangs from the roof steel and provides the crew’s platform, almost 80 ft above the stage. Below the gridiron are two more intermediate fly gallery platforms from which smaller scenery pieces can be hung. At the stage level, new openings were cut in the existing concrete floor to accommodate temporary floors supported at the basement slab below.

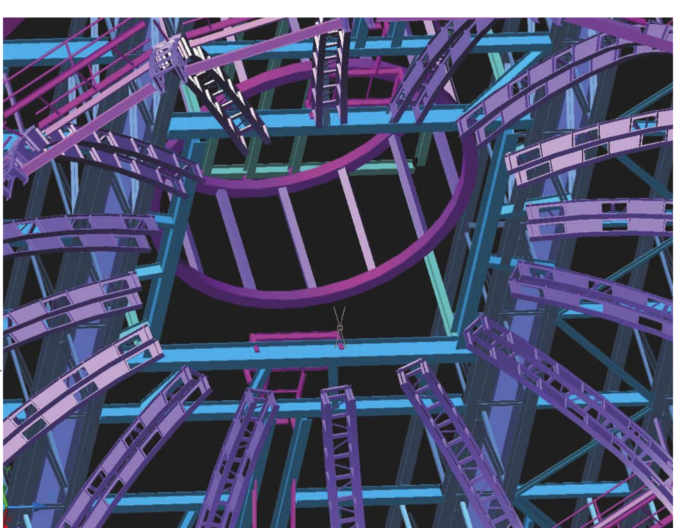
Backstage, the new stage house had to coexist with and incorporate the neighboring structure. The existing Guggenheim Hermitage Museum used a 59-ft-wide by 69-ft-tall steel-framed “megadoor.” To save demolition costs, the design team moved the door to the open position and locked it permanently into place, where it serves as the exterior wall for the loading dock. The 36-in. pipe column that served as the hinge for the door was filled with concrete and incorporated into the stage house wall.

Scene 3: From Museum to Theater

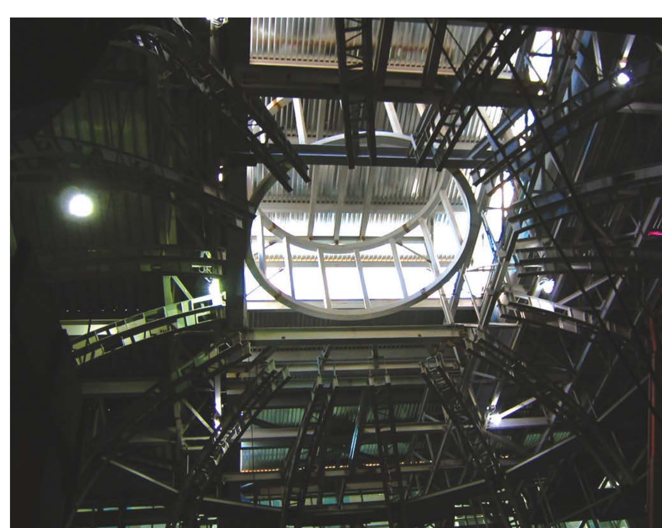
Some interesting transformations were required to nest the intricately decorated theater within the confines of the existing structure, which had a heavy concrete structure for the exhibit floor that withstood live loads of 250 psf. This structure would have definitely been able to support a theater usage—if not for a 32-ft-wide trench right down the middle! The beams at the trench edge came equipped with continuous steel embeds that previously supported a removable floor that spanned from one side to the other. The existing embeds were used to weld connections for new (and permanent) composite steel beams that would close the trench for good and provide a continuous structure to support the Orchestra seating above.

At the rear entrance to the theater, the elevated Parterre lobby is comprised of composite beams framing into wide-flange steel columns. The theater’s stepped seating area is framed with composite beams supported by concrete walls that rest on the existing floor below. The seating area floor system consists of a minimum 2½ in of normal-weight concrete over 1½ in composite steel deck.

Column-free viewing and vibration control were important considerations for patron enjoyment. Patrons seated at the Balcony Level arrive via the Grand Stair from the level below. The


Fisher Technical Services, Inc.

3D rendering of steel structure supporting the Phantom chandelier (including tracks, winches, and equipment).


Walter P Moore

Construction photo of the intricate structural steel backbone of the ornate theater ceiling.

elevated Balcony Level is supported primarily by built-up steel plate girders that provide column-free views for the patrons below. The plate girders had to cantilever the required 23 ft and provide enough stiffness to control objectionable vibrations to balcony patrons, and be shallow enough to not affect the view from the Parterre below. To accomplish these goals, the plate girders taper from 42 in. deep at the supports to a slim 8 in. at the cantilever tip.

The theater is laterally stabilized by four braced frames that were strategically located by the design team so as to not obstruct any views, while also providing the necessary resistance in the event of an earthquake. With the site falling under Seismic Design Category C, according to IBC 2000, ordinary steel concentrically braced fames provided an economical solution. The braced frames consisted of wide-flange columns and beams with either double-angle or rectangular HSS diagonal members.

Scene 4: A Supportive Ceiling

Like any theater production, Phantom requires numerous catwalks and equipment rooms above the theater from which technicians control lighting and other operations. For this production, however, the designers also envisioned an intricate dome ceiling that houses a 2,100-lb mechanized chandelier that can move about the theater. But, the existing roof structure did not have the capacity to support such a structure. To solve this challenge, the design team devised a long-span solution to take advantage of the museum's crane rails, which were supported by beams 125 ft apart. The design team utilized four steel box trusses supported by posts from the crane rail support beams. The box truss was necessary for stability because the top chord would not be braced by the roof or ceiling structure.

The steel fabricator was involved early in the design phase in order to provide the most structurally efficient and economical trusses possible. The fabricator constructed the trusses on the ground and lifted the whole truss into place, and also welded all of the connections—a situation that lent itself well to the configuration that was ultimately chosen. The chords were web-vertical W14 shapes while the web members consisted of double-angle tension diagonals and HSS compression posts forming panel points at approximately every 12 ft. The web-vertical bottom chords allowed for standard shear connections for the supported ceiling beams and flexibility in the location of these beams without subjecting the chords to minor-axis bending. The two faces of each box were then connected at the top and bottom chords with double-angle laces and battens.

Scene 5: The Chandelier's Big Scene

Exactly how do you support a chandelier that designers described as "exploding" and is so integral that it is thought of as a main character? Early design discussions included a turntable structure, a chandelier that broke into several pieces, and a chandelier that needed to move from its static position at the center of the domed ceiling to the stage itself—with the Phantom in it! Again, structural steel proved to be the answer.

The mechanized chandelier is introduced to the audience in a grand fashion. The production begins as a dilapidated chandelier ascends from the stage toward three other pieces of chandelier and reassembles before gracefully rising to its static position in the theater ceiling. Later in the show (and to the surprise of those sitting close to the stage), the Phantom and the chandelier descend from the ceiling. Then, in a flash, the chandelier rises back up to its at-rest position.

Design requirements for the chandelier were coordinated between the structural engineer and a specialty contractor; these included the weight allowances for the chandelier, the tracks, and the various winches required. In order to support the tracks on which the chandelier traveled, 16 curved "pocket beams" were designed using two W21x50 beams with detailed web openings and laced together at the top. The bottom flanges could not be laced, as this would have literally stopped the chandelier in its tracks and prevented the travel of the cables and ultimately the chandelier. The curved pocket beams were in turn supported by the ceiling structure (steel beams and box trusses). Because the chandelier was from the onset conceived to travel in pieces and throughout most of the ceiling space, the engineer had to model, analyze, and design for the chandelier weights at various different locations within the ceiling. This in turn gave the chandelier's specialty contractor a great deal of flexibility in tailoring its path in the show.

Music of the Night

If and when you ever find yourself in this Las Vegas theater waiting for the Phantom overture to begin, take a few moments to admire the building's magnificent structure and intricate interior. A "phantastic" performance by the design team will take you back to the Paris Opera House in all its splendor. On with the show! MSC

Ruben Martinez, P.E., S.E., principal, and Adam Johnson, graduate engineer, are based in the Austin office of Walter P Moore.

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Production Architect
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Steel Fabricator/Erector
SME Steel, West Jordan, Utah (AISC Member)

Chandelier Contractor
Fisher Technical Services, Inc., Las Vegas

Theater Consultant
Fischer Dachs Associates, New York

Engineering Software
SAP2000 and ETABS

The Suite Life

BY ERIC ROTH, P.E.

An innovative steel system is seeing increased use in university residence halls, as illustrated by a current dorm project in Pennsylvania.

IN LATE 2005, A RESIDENCE HALL PROJECT AT KUTZTOWN UNIVERSITY WAS IN THE MARKET FOR A STRUCTURAL SYSTEM. As the design team, we were determined to employ a system that would not only be cost-effective, but would also exceed expectations for speed and possible winter construction.

We ultimately chose the Girder-Slab System. Often schedule-driven, with the need to meet occupancy requirements for incoming students, the student housing market has increasingly turned to this system. To date, it has been utilized on nine student housing projects at colleges and universities throughout the U.S. It will also be used for a project at the University of North Florida, currently in design, and was recently chosen for a 200,000-sq.-ft mixed-use North Quad student housing facility at the University of Michigan.

Why Girder-Slab?

The new steel and precast composite 258,000-sq.-ft residence hall at Kutztown U. (in Kutztown, Pa.) is scheduled for occupancy by August 2008. It will include retail space and meeting rooms in addition to providing 856 beds and several apartment-style units.

When the project was awarded to our firm, I immediately knew what system we should use (see the sidebar "Discovering D-Beams"). The challenge was convincing our chief structural engineer, the project manager, and the client that Girder-Slab was the way to go, given that we had no track record with this system. What finally convinced everyone was the erection speed. Girder-Slab indicated that with sufficient erection forces and completed foundations, the building frame, with floors in place, could be erected in 40 days. In the student housing market, if the building isn't in place at the start of the school year, a school can suffer huge revenue losses.

Getting to Know the System

The Girder-Slab System, like any other steel frame, can use several different lateral load resisting systems, but of course some work better than others. Moment frames can be used, but D-Beams—the heart of the Girder-Slab System—are very shallow, only 8 or 9.645 in. deep, and therefore do not have very high moment capacities. Columns in moment frames often are required to be deeper, making it more difficult to bury them in the walls of a housing unit. Shear walls can be used for lateral loads, but bringing another trade on board only slows down the erection.

We found that the Girder-Slab System works best with braced frames; the issue is finding places within a housing unit to place bracing. Due to the short column-to-column distances, the braces tend to be steeper than optimum, and brace forces in mid-rise structures can be very large. Bracing locations need to be addressed early in the project since they often require thicker walls to conceal the bracing, which needs to be worked into the overall architectural floor plan.

The Girder-Slab web site (**www.girder-slab.com**) offers several different types of typical connections of D-Beams to columns, including shear tabs, end plates, and column trees. Our drawings were set up to allow the fabricator a choice of connection types. The fabricator opted for end plate connections. The Girder-Slab standard details we used show a typical end plate connection with bolts to the column outside of the beam flanges.

In retrospect I would opt for end plate connections with the bolts inside the beam flanges. Having the bolts outside the beam flanges brought up the questions of whether this was truly a pinned connection and whether we were introducing unintended moments into the column. Also, since the end plates and the bolts on the top side of the connection extend above the precast plank and topping, we have had unanticipated issues with the plates and bolts being in the way of wall framing. Moving the bolts inside the beam flanges on end plate connections avoids these issues. Using the 9.645-in. D-Beam, there is room to tighten the bolts. The FAQ technical section of the Girder-Slab web site suggests two bolts above and two bolts below the bottom flange. Girder-Slab's new design guide will also suggest this.

Finding a Way

Several places in the building required spans that were larger than the 15-ft column bays typically used with Girder-Slab System. One of these was a lower level vehicle pass-through. In this case we were able to use the column tree-style connection. Each column branch was extended 3½ ft, allowing us to increase the span at this one location to 22 ft. Column branches were used on each side of the columns in question in order to balance the loads into the column and reduce the amount of moment introduced into the column. As with all longer spans the engineer must be aware of the amount of dead load deflec-

Steel erection at Kutztown University occurred this summer. Lateral bracing is provided by chevron braces strategically located to avoid window locations.

Discovering D-Beams

The main structural elements for the Kutztown University project are D-Beams, the heart of the Girder-Slab System. Our choice to go this route was actually due to our experience on a project prior to the Kutztown building. This first project was a seven-story housing unit for another university, and the owner had requested a block-and-plank building. During the gravity portion of the design, as our engineer's design reached the bottom floor for the bearing walls, he found that he simply could not get the block to take the dead and live loads from the floors above.

As in many housing units, there were numerous doors along the center hallway bearing walls. Thicker walls or multiple widths were not an option, as reduction in room size or expansion of the building width was deemed unacceptable. Going to a conventional steel frame system would have increased the building's floor-to-floor height and its overall height of the building—another unacceptable scenario. At this point, the design ground to a stop, as we had no solution for bearing walls given the constraints on the project. Unfortunately, our structural group was removed from the project, and we were made aware that an outside engineering firm had come up with a solution.

Professional curiosity being what it is, we inquired which system and engineer were completing the project. The engineering firm, O'Donnell & Naccarato, Inc., Philadelphia, was actually the engineering consultant hired by Girder-Slab Technologies to oversee the original laboratory and full-scale testing program.

Through further research we discovered D-Beams, which were used for this project. A D-Beam is a castellated tee beam with a flat bar welded to the web to form the upper flange. The D-Beam initially supported the plank weight and construction loads—on its bottom flange—and then was doweled and grouted to form a composite section for post-erection loads. I was left thinking, "We could have done that if we had only known." Lesson learned!

—Eric Roth, P.E.

tion in the system. This is especially true with the shallow D-Beam members. Even in lightly loaded members, deflections can become excessive.

A typical detail for attaching the plank to the framing is to use weld plates in the bottom of the plank to connect it to exterior spandrel beams. Attaching the D-Beams to the plank is typically accomplished by running reinforcement through the beam castellation and grouting the gap to make the beam composite with the plank. During erection, multiple floors of plank may be set before grouting the plank. The erector will often plumb the structure using cables and come-alongs. We soon found that as the cables were tightened during the plumbing operation, the columns were being pulled closer together since there is no physical attachment of plank to the D-Beams prior to grouting. Angles and shims had to be added between the plank and the columns to prevent the plank from any movement on the bottom flange of the D-Beam. In the future I would require planks bearing on D-Beams that are adjacent to the columns to have weld plates installed, thus preventing movement.

The Right Choice

Was Girder-Slab the right system for this project? Absolutely! While the Girder-Slab System has span limitations and requires numerous columns, its speed of erection and clear underside of structure are big pluses when dealing with housing units or hotels and low floor-to-floor heights. There are some detailing items that must be addressed during the design as mentioned above, and as with any new system there's a bit of a learning curve. But the system itself can be analyzed just like any other steel frame system. **MSC**

Eric Roth is chief structural engineer with STV, Inc., Douglassville, Pa.

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Ocean View

BY WILLIAM BAKER, P.E., S.E., ROBERT SINN, P.E., S.E., AND BRADLEY YOUNG, S.E.

INSPIRED BY ITS LOCATION ON THE ATLANTIC COAST, the recently opened Virginia Beach Convention Center's architectural design is based on nautical themes and seagoing vessels, with a strong emphasis on exposed structural support related to such endeavors.

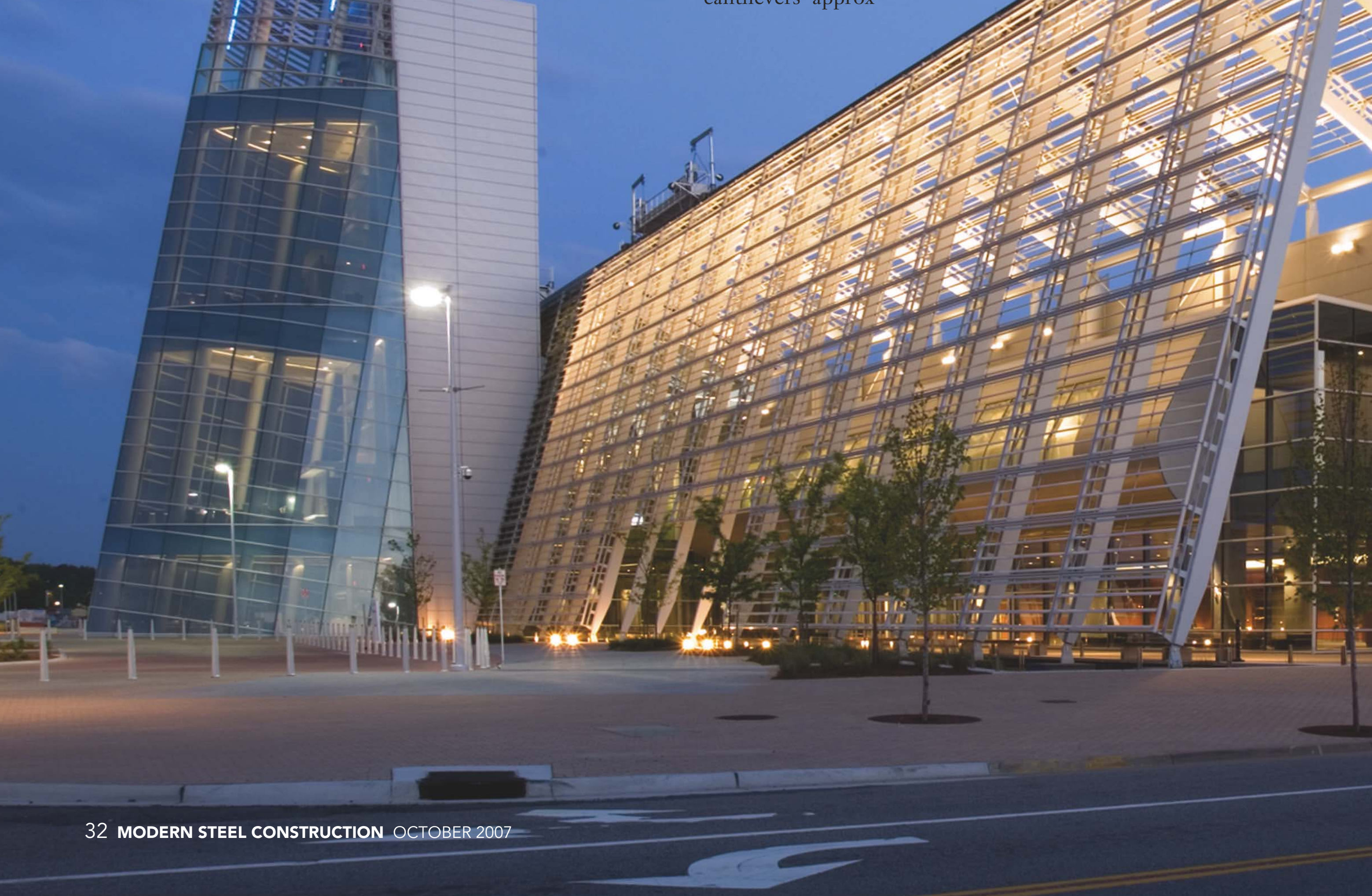
Images such as sails, yacht fittings, lighthouses, waves, and water all find their way into the design vocabulary for the facility. A monumental steel-framed glass curtain that forms the front entry to the center resembles a wind-filled sail and appears to float in pools of water. Visitors walk across wooden boardwalks to make their entrance into the building, where they stroll on carpet that evokes images of seaweed, beach towels, and sand. The steel and glass observation tower is reminiscent of a lighthouse, offering breathtaking views of the ocean and the beach. Exposed structural steel elements are used throughout the convention center as a common and unifying theme while supporting the architectural metaphor of modern, sleek, ocean-going vessels.

The overall footprint of the convention center is 880 ft by 430 ft in plan. The project was built in two phases. The adjacent, existing convention center was used during construction of the first half of the new facility. Once this phase was completed and operational, the existing convention center was demolished and the adjoining second phase was completed. The full facility opened in December 2006.

Exhibition Halls

The convention center contains a total of 516,000 sq. ft of space, 150,000 sq. ft of which is dedicated to column-free exhibition halls. Structural steel Pratt trusses with arched top chords were designed to span 256 ft between bearing supports, also creating a 40-ft-tall unencumbered exhibition space. These open hall spaces, economically covered by the structural steel roof structure, are seen as a distinct marketing advantage in terms of heightened flexibility by the convention center operators; Virginia Beach is in direct business competition for exhibitions and conventions with other facilities in nearby cities located along the central eastern seaboard.

The long-span roof trusses over the ballroom and exhibition hall spaces are spaced at 30 ft on center along the length of the convention center, and are 22 ft deep between the top and bottom chords at the roof high point and 9 ft at the north bearing. The roof cantilevers approx-



Wide open spaces, made possible by long spans, are part of a coastal theme at the new Virginia Beach Convention Center.

imately 60 ft beyond the north support and over the facility's 21 loading docks.

The exhibition hall trusses were shipped to the site in individual pieces, assembled on the floor of the hall in the lay-down position with specified cambers, and then lifted into place as an entire assembly (approximately 60 tons total) with one fixed-leg and two crawler cranes without temporary support towers. ASTM A913, Grade 65 yield steel was used for all long-span roof truss top and bottom chords. In addition to reducing the total quantity of steelwork for the roof, the use of 65 ksi grade steel for the truss chords also required somewhat less crane capacity for the truss lifts. The arched top chord was fabricated in straight-line segments between bolted field splices. Diagonal and vertical members of the roof trusses were specified as ASTM A992 Grade 50 ksi yield steel. All W14-series roof truss chord and web members are oriented in bridge-type arrangement with flanges aligned and double gusset plates at the joints.

Field bolting at the gusset plates is accomplished with 1⅛-in.-diameter A490 bolts designed in bearing within standard holes. Truss camber ordinates up to 3½ in. were supplied on the construction documents at each chord splice location. The roof trusses span between reinforced concrete structures to the north and south of the exhibit hall. The southern support is formed from a fixed manufactured pot bearing, whereas the north bearing is of guided expansion type.

All structural steel trusses, ceiling and roof purlins, diaphragm bracing, and roof decking are left entirely exposed in the final built condition in the exhibition halls, staying consistent with the notions of exposed structure and inherent strength throughout the facility. All structural steel received a multi-coat paint system including sand-blasted substrate, zinc-rich metal primer, and final finish coats. A consistent birch finish color was used throughout to tie all exposed steel elements into the overall architectural design.

Pre-function Area

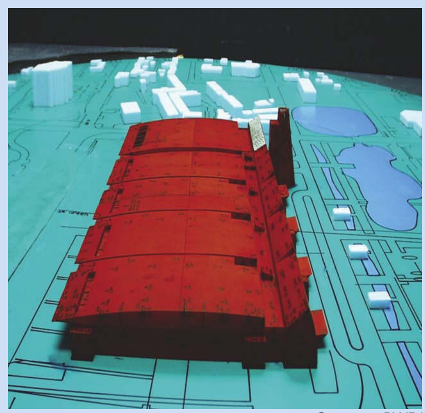
Housing the convention center's pre-function space is a striking 80-ft-tall glass window wall, subtly curved along its vertical surface, emulating the profile of a boat's sail in the wind. This glass façade is thermally and visually separated into three individual modules by reinforced concrete stair towers. Each pre-function module is approximately 120 ft long in plan and consists of nine vertically spanning cable trusses spaced at 15 ft on center. Each cable truss is ground-supported at its base and supported at its top by a structural steel frame, which is anchored to the reinforced concrete frame of the main convention center. In addition, each truss consists of an 8-in.-diameter double-extra strong structural steel pipe with rigidly connected structural steel "arms" of cruciform cross-section extending out at various lengths from opposite sides of the 8-in.-diameter central "spine" column. Following the profile created by these cruciform arms, two 1⅝-in.-diameter structural steel cables flank the central spine and give the trusses their "fishbone" profile. The glass panels at the building façade span vertically between supporting rectangular HSS that span horizontally between fishbone trusses. Each pre-function module is diagonally braced at its roof and back face, and is tied into the main structure R/C frame to provide lateral stability to the overall system.

The primary function of the vertically spanning fishbone trusses is to resist the high design wind loads imposed upon

Wind Tunnel Testing

The Virginia Beach Convention Center is located less than a mile from the Atlantic shoreline in a region prone to hurricanes and very strong winds. Therefore, engineering for wind loads was clearly a critical component in the design of the structure and its external cladding. The "fishbone" cable trusses that support the pre-function window wall, as well as the long-span roof trusses over the exhibition hall and ballroom, are relatively flexible, lightly damped, low-frequency structures. An understanding of these elements' deformations under the applied loads, as well as any inertial forces that may be generated through their dynamic response to wind, needed to be understood. These structural characteristics, along with the complex characteristics of hurricane force wind events, meant that code-prescribed wind loads alone would not be appropriate in this instance for developing the design wind loads; specialized wind tunnel testing would be required.

The design team turned to Rowan, Williams, Davies & Irwin (RWDI, Guelph, Ontario, Canada) to provide wind tunnel testing and consulting services for the project, and a 1:400 scale model was created for the testing. Due to the limited overall height of the convention center and the importance of obtaining accurate loads for cladding design, a pressure model was created, utilizing the high-frequency pressure integration method for obtaining equivalent static design wind loads from measurements taken from more than 700 pressure taps at the model surface.



Courtesy RWDI

A 50-year return period, three-second gust design wind speed of approximately 130 mph, which includes an importance factor of 1.15, was used for testing. The objectives of the wind tunnel testing program were to determine the peak local winds for cladding design and overall structural wind loads for the design of the primary lateral load resisting elements. Wind loads for secondary structural elements of the building, including the long-span roof trusses over the exhibition hall and ballroom and the fishbone trusses at the pre-function space, were also determined.

Wind pressures for the design of these elements are discovered through a combination of external pressures measured directly from the scale model in the wind tunnel, and internal pressures determined through a combination of numerical techniques along with the wind tunnel tests.

the structure from the coastal environment. Under the action of wind, the central pipe acts as the compression chord of the truss, while the cables act as the tension chords.

The cruciform arms linking the central pipe with the cables act to link the compression and tension elements of the fishbone truss. Because the trusses are pin-supported at each end, the moment is at a maximum at mid-height. An initial pretension of 80 kips was introduced to each cable in order to offset the loss of tension in the cable and ensure that the cables remained in tension under wind loads. The main compression chord of the fishbone truss is extremely slender when considering its height between its pin-ended support points. This central spine relies upon the adjacent cables in tension to maintain its stability. Under lateral loads, the fishbone truss central spine develops destabilizing axial compression while the cable chord develops stabilizing axial tension. Essentially, the individual fishbone trusses are self-stabilizing mechanisms; the compression chord's tendency to buckle is resisted by the cable's tensile stabilization forces. The cruciform arms act to link these balancing mechanisms, and therefore serve as brace points along the height of the compression spine.

Vertical "fishbone" trusses (at left in photo)—consisting of round HSS, cruciform cross-section arms, and tension cables—support the glass enclosure of the pre-function space.

The performance of the fishbone trusses is dependent upon the fitting and connection detailing to ensure system behavior consistent with the intent of the engineering design and analysis. The trusses were fabricated and tensioned at Josef Gartner & Co.'s fabrication plant in Gundelfingen, Germany, where the long main pipe and cruciform arms were placed in a setting frame and the cables were stressed to the specified pretension using a hydraulic jack anchored at the fishbone truss end plate.

The setting frame ensured that the cruciform arms did not restrain the elongation of the cables through the end fittings during the cable-tensioning operation, avoiding locked-in stresses and deformations in the cruciform arms and non-uniform force distribution in the cable trusses.

Once the specified tension had been achieved, the cable end was locked into place and the cable fittings at the cruciform arm ends were fastened around the cables. The fishbone trusses were mounted as fully assembled and tensioned units within a supporting frame to ensure that the transport of the assemblies did not induce loading conditions and stresses that were not intended in the design of the trusses. The trusses were then transported to Virginia Beach for erection.

Towering Above

The 150-ft-tall observation tower serves as a focal point and meeting place for patrons of the convention center. In addition to a publicly accessible rooftop deck, the tower houses two special 40-ft-tall meeting rooms that may be used for special functions, convention center meetings, and more intimate receptions. Overall, the $202 million facility is an example of architects, structural engineers, and multiple steel fabricators and erectors working together to exploit the potential for a wide variety of structural steel shapes and assemblies within an overall composition of exposed steel design. MSC

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Turner Construction, Chicago

Steel Detailer
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Steel Fabricator
Cives Steel Company, Winchester, Va. (AISC Member)

Structural Engineering Software
SAP2000
S-FRAME

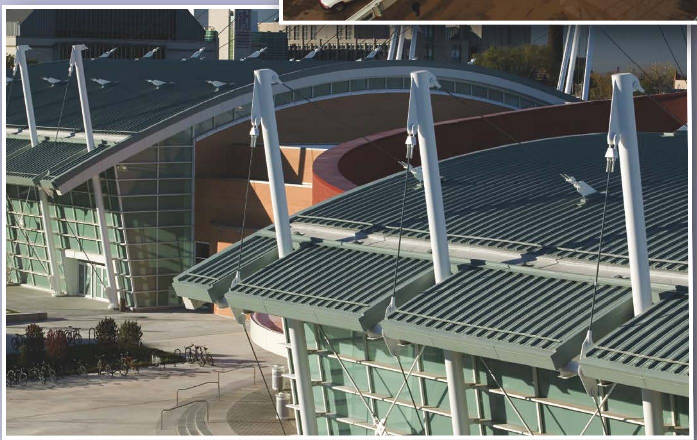
Winners Choose Chicago Metal to Curve Steel

2002 EAE Merit Award – 400 tons of 16" OD Pipe curved for JFK International Airport Terminal 4. Jamaica, NY.

2003 IDEAS National Winner – 300 tons of 5" square curved 45º off-axis for the Kimmel Center. Philadelphia, PA.

2007 IDEAS National Winner – The University of Phoenix Stadium.

400 tons of 12" square tubing curved for the retractable, lenticular roof trusses. Phoenix, AZ.



2004 EAE National Winner – 310 tons of beams bent including reverse curves for the Gerald Ratner Athletics Center. Chicago, IL.

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A Tale of Two Projects

BY MICHAEL BOLDUC, P.E., JOHN THOMSEN, P.E., AND JOSEPH ZONA, P.E.

Teams from two health-care projects offer some insight into choosing between field-welded and field-bolted moment connections.

WHETHER IT'S A 5,000-SQ.-FT ADDITION OR A $185 MILLION FREE-STANDING BUILDING, MOST OF THE HEALTH-CARE FACILITIES WE'VE DESIGNED DURING THE PAST DECADE SHARE THE SAME LATERAL FORCE RESISTING SYSTEM: A STEEL MOMENT FRAME.

This preference is the result of special design requirements for "essential facilities," as well as the unique combination of a need for long life and renovation flexibility in health-care construction.

Most hospital buildings are considered essential facilities, which the 2003 International Building Code defines as "buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes."

At the same time, it's typical for health-care buildings to be kept in service longer than the typical 50-year life cycle. However, medical campuses frequently morph and change—constructing new facilities, demolishing outdated buildings, and renovating and adding on to existing buildings—all in an effort to keep up with the constantly evolving "state-of-the-art" for modern medicine and clinical practices.

Because of this "ever-changing" mentality, owners and architects prefer to use structural systems that provide as much future flexibility as possible.

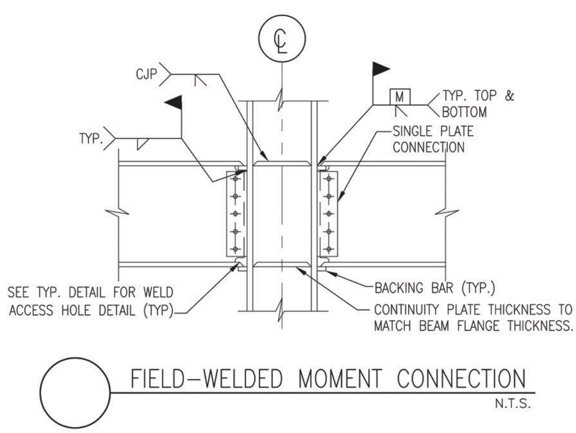

Figure 1. Typical field-welded moment connection detail.

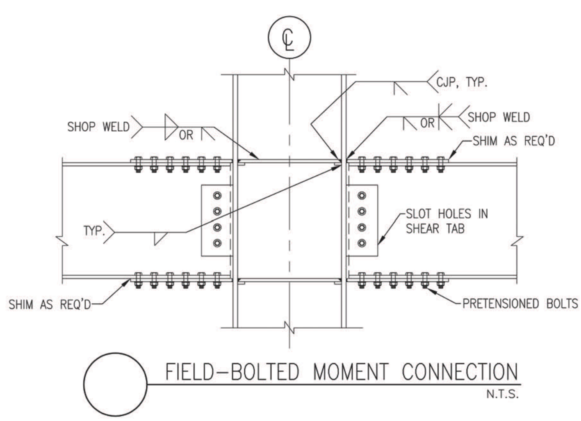

Figure 2. Typical field-bolted moment connection detail.

Two Hospitals, Two Moment Connection Details

For ordinary steel moment frame (OSMF) buildings located in the eastern half of the country, we typically provide two options for moment connections: field-welded (Figure 1) and field-bolted (Figure 2). The fabricator is then able to choose the moment connection most appropriate for the project.

By comparing two projects after completion (both in the eastern half of the U.S., with one using a field-bolted flange-plate moment connection and the other a field-welded moment connect), we can objectively look at lessons learned and challenges faced during the detailing and construction phases.

The Heart Hospital at SwedishAmerican Health System in Rockford, Ill. is a four-story, 130,000-sq.-ft facility that was designed to accommodate a future two-story vertical expansion. The fabricator on this project, Zalk Joseph Fabricators, went with flange-plate moment connections using all field-bolted connections.

The second project, the Women and Infants Building at the Maine Medical Center in Portland, is a five-story, 210,000-sq.-ft facility that was also designed to accommodate a future two-story vertical expansion. The fabricator, Novel Iron Works, chose field-welded moment connections for this project.

A Seismic Note

Both projects are located in regions of low or moderate seismic activity and were designed as OSMFs. The contract documents specify that the detailing of the moment frames conforms to AISC *Seismic Provisions* *for Structural Steel Buildings*, which requires the moment connection to develop the full plastic moment capacity of the moment girder.

During the construction phase of the Heart Hospital, the detailer asked if they could design the moment connections using a seismic response factor R = 3, consistent with a steel system not specifically detailed for seismic resistance. This is common practice in the low-seismic region of the Midwest. However, since the building was designed for a future vertical addition, and because it is uncertain how future code revisions may restrict the use of the R = 3 design and detailing practice, the OSMF system was retained.

Choosing the Proper Connection Type

During interviews with the construction managers (CMs), fabricators, and erectors for these two projects, five key criteria were factored into the selection of either welded or bolted moment connections:

Historical regional common practice. Past experience with steel moment frame projects and the prevailing common practice in the local geographic regions played large roles in both projects, in terms of the decision to use moment connections. According to Novel, approximately 90% of typical moment connections in New England are field-welded and 10% are field-bolted. In comparison, Zalk Joseph estimates that over the past decade approximately 50% of the steel moment frame structures they've fabricated used field-welded connections and 50% used field-bolted.

Labor cost: shop labor vs. field labor. Field-bolted moment connections typically require more shop fabrication time to shop-weld large flange plates to the columns with complete joint penetration (CJP) welds. However, the amount of field labor should be considerably less, because installing bolts is faster than field-welding.

In the Rockford area where the Heart Hospital is located, the local ironworkers union requires a two-person team (one welder and one helper) to work on all field-welded moment connections, which effectively doubles the amount of manpower—with certified welders at a premium—and increases the amount of welding equipment needed. Zalk Joseph chose to shop-attach the flange plates, anticipating that the reduction in field labor would compensate for the increases in required shop hours, resulting in a net savings.

The flange plates were fabricated with an extra ¼-in. gap—to accommodate beam depth tolerances—which was filled with full-coverage shim plates with matching bolt holes after erecting the girder. Because the flange plates were designed to develop the full plastic moment capacity of the moment beams per the AISC *Seismic Provisions*, the fabricator considered the physical size of the flange plates to be burdensome. The flange plate thickness (up to 2 in.) and length (up to 3 ft 6 in.) made coordination troublesome with the architectural precast façade connections bearing on the exterior moment frame girders near the moment connections. The thickness of the plates also required additional flange extension plates to support the metal deck near the ends of the beams.

Weather concerns. Weather is another critical factor to consider as it can impact the erection schedule. CMs for both projects mentioned that the anticipated time of year for construction—and the related weather patterns—plays a role in selecting a moment connection detail. If steel erection was to be scheduled for the winter, they recommend bolted flange plate connections, since bolts can be installed in most weather conditions. However, if the weather was to be favorable during the erection phase, both CMs preferred using field-welded connections since the erection tolerances are not as strict. (Field-welding is significantly more weather-dependent than field-bolting, especially in sub-freezing winter climates and during rainy weather.)

Preheating top flange of moment girder before welding in cold weather conditions at Maine Medical Center.

Erecting Heart Hosptial's column "trees" with shop-attached flange plates for bolted moment girder connections.

A completed bolted moment girder connection with top and bottom flange plates at Heart Hospital.

SwedishAmerican Health System's Heart Hospital in Rockford, Ill. is a four-story bolted steel moment frame with capacity for two additional stories.

Since much of the Women and Infants Building's steel erection occurred during winter months, the welding schedule was significantly impacted by severe cold, wet, and windy weather. William A. Berry & Son, the CM, said they would seriously consider using a bolted moment connection for future projects where steel erection was scheduled for winter months.

Conversely, because the construction for the Heart Hospital occurred mostly during warmer months, the CM on that project said that if they had to do it all over again they would likely go with field-welded connections to avoid much of the coordination problems between the bolted connections, precast supports, and metal deck. Zalk Joseph said that in hindsight, the savings they received on the field labor during steel erection was not adequate to offset the increase in shop time spent making flange plate connections.

Construction schedule. Both CMs indicated that the construction sequence, schedule, and logistics are always driving factors in selecting a moment connection design. Erecting a complete steel-framed building involves several different subcontractors working in concert with each other, with the CM serving as the conductor, directing the overall performance to ensure that each piece is constructed in the proper sequence.

On the erection side, the common industry assumption that field-bolted moment frame erection is faster than field-welded may not hold true in all cases. Novel said that the steel erection schedule typically drives the decision to use field-welded connections, noting that crane time is a primary component of the erection cost and that many erectors would rather take time after the pieces are erected to finish welding the connections rather than sacrifice crane time for troublesome erection.

Field-welded moment girders can be erected in the same manner as typical gravity framing, with a few bolts in the webs on each end of the girder to hold it in place. Once the shear connection is started, the steel erection crew can move on with the next piece of steel while the bolt-stuffing crew follows behind.

During erection of the bolted flange plate connections at the Heart Hospital, the steel erector, J.P. Cullen, commented that they had to line up not only the bolt holes in the beam web, but also several of the bolts in the top and bottom flange plates so that all of the bolt holes were aligned, before the crane could be "cut loose" to erect the next piece of steel.

Although erection of the bolted flange plate moment frame takes longer than the welded moment frame, once the steel is erected and in place, the bolted moment connections can typically be completed much faster than the field-welded moment connections. The bolted connections simply need the remainder of the bolts to be installed and tightened, and if tension-controlled (TC) bolts are used, the visual bolt inspections can be done quickly and easily.

In comparison, the welded moment frames need the remaining bolts in the web connection to be installed and tightened (often, slip-critical bolts), and then welding of the flanges can begin. Typically, several certified welding crews are required simultaneously on each project to ensure that the CJP welds can be completed in a timely manner. After the welding is completed, CJP welds must be ultrasonically tested by a certified weld inspector. Since the field-welding and inspection process is time-consuming, poor weather conditions can cause significant delays to the overall construction schedule.

Coordination with subsequent trades. Only after the moment connec-

Maine Medical Center's Women and Infants Building in Portland, Maine (scheduled for completion in 2008) is a five-story welded steel moment frame with capacity for two more stories.

tions are completely installed and inspected can the other trades follow. The metal decking crew will lay out the metal deck and fasten it to the steel framing, and then the rebar and concrete crews will place the reinforcing steel and the concrete slabs.

The metal decking crew must coordinate closely with the welding crew so as not to cover up incomplete steel connections, or the decking must be cut open to allow the welding crew to finish the incomplete connections. William A. Berry & Son stressed that the erection flow should properly schedule erection, welding, inspections, decking, and slab pours. The number one issue from the CM's standpoint is keeping all trades moving in unison. This involves making sure that connections are accessible for welders while the decking crew is installing deck and that the welds and inspections are completed prior to moving on with the construction sequence.

In Retrospect

OSMFs continue to be a preferable lateral force resisting system for health-care facilities in low and moderate seismic zones because they provide expansion capability and floor plan flexibility. Field-welded and field-bolted moment connections each have their own respective places in the construction industry. The dominant factor in deciding which connection to use turned out to be the anticipated weather conditions during the bulk of the steel erection phase. If erection is scheduled for spring, summer, or fall, welded connections would likely be the prevailing choice. And if steel erection will occur during winter months, serious consideration should be given to a field-bolted moment frame. Whenever possible, this decision is best left in the hands of the construction team. MSC

Michael J. Bolduc is a staff engineer, John H. Thomsen is a senior project manager, and Joseph J. Zona is a principal and national head of the Structural Engineering and Engineering Mechanics Group with Simpson Gumpertz & Heger Inc.'s Waltham, Mass. office.

Heart Hospital at SwedishAmerican Health System

Owner
SwedishAmerican Health System, Rockford, Ill.

Structural Engineer
Simpson Gumpertz & Heger, Inc., Waltham, Mass.

Architect
Perkins + Will, Inc., Boston

Construction Manager
Turner Construction Co., Chicago

Fabricator
Zalk Joseph Fabricators, LLC, Stoughton, Wis. (AISC Member)

Women and Infants Building at Maine Medical Center

Owner
Maine Medical Center, Portland, Maine

Structural Engineer
Simpson Gumpertz & Heger, Inc.

Architect
TRO-Jung|Brannen, Inc., Boston

Construction Manager
William A. Berry & Son, Danvers, Mass.

Fabricator
Novel Iron Works, Inc., Greenland, N.H. (AISC Member)

Certifying the Certifier

BY ROBERTA MARSTELLAR, P.E., WITH SHEILA ALEGRIA

Over a three-year period Quality Management Company improved itself from the inside out—and became ISO certified in the process.

THOMAS FULLER SAID, "HE DOES NOT BELIEVE WHO DOES NOT LIVE ACCORDING TO HIS BELIEF." For years we at Quality Management Company, LLC (QMC)—the AISC subsidiary charged with conducting steel fabricator and erector quality management system audits—had been pushing our program participants to do something that we were not committed to doing ourselves.

When I took the reins in 2004, I wish I'd known the vast improvements that implementing the ISO 9001:2000 system would bring to QMC. I realize now that it's not about the audit, but rather a way of doing business every single day. The procedures we've set in motion and now adhere to have directly lead to improvements in our communication, problem-solving, customer satisfaction, and more. We incorporate ISO elements, such as reviews of customer complaints and internal corrective actions, into our staff meeting agendas. As a group we discuss the root cause of an issue and what we can do in the future to prevent a reoccurrence. We compile the feedback we receive from our satisfaction surveys and review it on a quarterly basis, searching for trends and meaningful ways to improve the level of service we provide to our program participants.

The road to an effective quality management system, and ultimately our ISO 9001:2000 certification, was long and challenging, but in the end it proved to be invaluable. The following story—warts and all—is an internal, firsthand account of how QMC has evolved over the past three years. Hopefully, it will serve as inspiration to any fabricator or erector who is considering becoming certified but doesn't know where to begin or what to expect. The quick answer is to take things in steps. As we often say around the office: "How do you eat an elephant? One piece at a time!"

The following journal entries chronicle QMC's sometimes-painful path towads ISO certification:

July 1, 2004: Day one as responsible party in charge of QMC and AISC Certification

Since accepting the job a month ago I've learned that:

- Certified companies are upset because engineers aren't maintaining requirements for AISC Certification in their specifications.
- AISC just introduced a new certification program for building fabricators, an apparently unwelcome change for fabricators resistant to change. And efforts to educate the specifiers about the new program were weak at best.
- The number of certified fabricators is decreasing.
- The number of complaints regarding QMC is on the rise.

I love a challenge!

July 2, 2004: Day two

The manager of administration just gave her one-week notice. That's more of a challenge than I'd hoped for. She doesn't have anything written down that would give me or her replacement any indication of what to do or how to do it. She has two direct reports. One started three weeks ago—enough said. The second has accepted an offer in another department. It's my second day and the team is already shrinking.

July 6, 2004: Technically still in my first week

The president just walked out of my office. He received what he described as his weekly call from a disgruntled program participant. He's worried. So am I, but I assured him it would be fine. On the bright side, we have our first goal: Keep the president's phone from ringing regarding certification and audits.

July 9, 2004: A lesson in customer service

I spent the afternoon discussing customer service with a QMC staffer after I overheard him telling a fabricator participant, "Yeah, your materials are in my stack…I'll get to it when I get to it." This isn't his first offense. He's one of two original team members remaining, but we're considering "voting him off the island." I'm held hostage knowing that beyond his rudeness toward customers, I have little understanding of his role and responsibilities.

October 14, 2004: Days turn to weeks turn to months

Captain Helpful was indeed voted off the island. There is something liberating in knowing that things really can't get any worse. In three months we've assembled a brand new team and are starting to find our way. I feel like a dog in a butcher shop: everywhere I turn there is an opportunity for improvement. Unfortunately, we

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**.

Lessons Learned the Hard Way

MAKE THE TIME! If you spend more time being proactive, it will save you reactive time down the road, and you'll have far fewer fires to put out.

PEOPLE! As Jim Collins suggests in his best-selling book *Good to Great*, get the right people on the bus. People will pleasantly surprise you. Don't underestimate them!

CHANGE IS GOOD! Don't be afraid to change the way you do things. It just might be for the better.

INVOLVE EVERYONE! You waste time trying to guess what people in your organization do. So, ask them or have them write it down with the understanding that not everyone dreams of being a technical writer. You can always fine-tune it later.

PRIORITIZE! Focus on what is customer-critical for your organization. Don't waste valuable time on procedures that fall into the "would be nice" category.

WRITE DOWN WHAT YOU REALLY DO! In the future you have the opportunity to revise your procedures as you identify gaps and inefficiencies. In the beginning consider only what you currently do, not what you'd do in your "perfect world."

KEEP IT SIMPLE! A well-written procedure should be two pages maximum. Any more than that is probably a combination of procedures. As for your quality manual, if it can double as a doorstop, you've gone too far. Less really is more; any necessary updates and revisions (yes, you will have revisions) will be minimized.

A PICTURE IS WORTH A THOUSAND WORDS! If a flow chart can clearly communicate the steps of your procedure, use it. A flow chart with enough information is a procedure.

FIGHT IT OUT! If you have the "right people on the bus" and they're passionate about what they do, you can expect some disagreements. Don't fret; constructive disagreements are healthy. Keep an open mind and make room for other perspectives. This will also help you get buy-in from everyone involved.

YOU NEED BUY-IN! You don't want people going through the motions just because the boss said so. This is a system that will only be as strong as its weakest link. You can use the company grapevine to find out where your weak links are. If they are the right people, it may be as simple as enhancing communication. In a worst-case scenario, they may not prove to be a good fit for your organization after all.

GO BEYOND WHAT'S REQUIRED! Certification programs like ISO 9001 and the AISC *Certification Standard for Steel Building Structures* require certain specific procedures. However, odds are you will need more procedures to accurately describe the processes you use in your business. Keep thinking, "If we had to start with all new people tomorrow, would we have the tools in place for a new team to pick up where the old team left off?"

REVIEW PROCEDURES TOGETHER! Consider it a valuable training opportunity; you'll learn a lot about the jobs people do. Look for gaps and overlaps. Remember to keep attendance lists; they will serve as your documentation of training.

DON'T INSIST ON PERFECTION! Procedures are living documents. If you're committed to continuous improvement and the evolution of your quality management system, know that tweaking your procedures will be an essential part of that evolution.

EXPECT CORRECTIVE ACTIONS! If you're convinced that no quality management system is perfect, then you shouldn't be surprised by the need for corrective action. Don't despair! These are valuable learning experiences and an essential part of the continual improvement process. They will help you identify gaps in your procedures, training needs, and more.

TRAINING, TRAINING, AND MORE TRAINING! Be sure everyone involved has a clear understanding of their role in the organization's quality management system. Remember, this may be new for them as well.

BE PATIENT! This will take time, especially if you are starting from scratch.

CELEBRATE! No one said it would be easy. Make time to celebrate with your team. They work hard. They have a lot on their plates and still make time to contribute toward developing your quality management system. Be sure to thank them and recognize their contributions.

are still in a highly reactive environment with little time or resources available to devote to strategic planning and process improvement. Things fall through the cracks everyday: lost payments, certificates issued to the wrong company (or to the right company with the wrong date or address), etc. On the bright side, the president is spending less time in my office.

November 17, 2004: Ouch

I'm starting to understand why our participants are so angry. We introduced a new program to more than 500 AISC Certified building fabricators. We gave them three years to adopt the new program, offered a few early seminars on the new criteria, and then turned our backs on them. They have until the end of 2005 to be audited to the new criteria, and out of the 500, we've had approximately 75 early adopters. It's starting to look like 2005 will be a tough year too.

January 5, 2005: ISO?

We are setting goals! At today's staff meeting we optimistically discussed the notion of becoming ISO 9001:2000 certified. I don't know what's involved, but I'm in favor of anything that would formalize and improve how we do business. The first few months felt as though we were recreating the wheel. We have a good team with a passion for helping our customers and the industry; that's a great start.

March 14, 2005: Are these the worst of times?

Reality check: The final year of transition to the AISC *Certification Standard for Steel Building Structures* is as bad as we expected. In spite of the resources and FAQs we've compiled on our web site and our new "let us help" customer-centric attitude, it appears it's "too little, too late" for some. Companies are struggling to make sense of the criteria and apply it to how they currently do business. We hear a lot of "I've been doing it this way for 30 years and…" More important, many of these companies are struggling to keep their doors open in the wake of steel price volatility. They are bidding on jobs at the 11th hour only to find that they are one of 11 bidders. It's apparent that becoming AISC Certified isn't the first thing they think about when getting out of bed in the morning. Yes, doing quality work is important to them, but like QMC they are putting out fires and struggling to find the resources to devote to overhauling their quality management system.

QMC is pretty much in the same boat when it comes to our own quality management system and ISO certification: It'll have to wait. We don't have the time this year for anything extra. We have fewer than five procedures currently documented. Our customer service program is entirely reactive. Bottom line: The squeaky wheel is getting the oil.

January 2006: Ready, Set… ISO!

Today we set our goals for the year. This will be the year for ISO!

April 11, 2006: A step in the right direction

The ISO books have arrived! I just opened up the Q9001-2000, *Quality Management Systems – Requirements*. It is written in English, but it really seems like a foreign language to me. "Product Realization"—what is that? Oh, this is going to take some getting used to…

May 24, 2006: My first ISO lesson

ISO requires that you implement a quality management system (QMS) that incorporates eight principles: customer focus, leadership, involvement of people, process approach, system approach to management, continual improvement, factual approach to decision making, and mutually beneficial supplier relationships. In turn, you develop a quality manual and set your quality policy and objectives. There are also six required procedures that must be set into place: control of documents, control of quality records, internal audit, control of nonconforming products, corrective action, and preventive action. Whew, this is going to be a lot of work for a company starting from scratch with only six employees!

August 21, 2006: Quality Policy

We finally agreed on our Quality Policy. I thought this would be the easy part, but I was wrong. It was an agenda item at no fewer than three staff meetings. Each time, we found ourselves debating words and their meaning. We felt the policy would become the cornerstone to our journey, so we took it seriously. Not unlike a mission statement we wanted something that everyone could get behind and support. And in the end:

It will be the Policy of Quality Management Company to ensure and improve the quality of steel fabrication and erection through audits and certification.

October 1, 2006: Customer satisfaction

Today we launched our satisfaction survey. It is our first vehicle for proactively collecting customer feedback, and it's long overdue.

October 27, 2006: Identify procedures and prioritize them

At today's staff meeting we made a list of the procedures required to conduct business. These procedures are above and beyond those required by ISO. We came up with more than 20! Then we prioritized the list and assigned authors. A priority of 1 indicates the item is customer-critical and therefore essential to how we do business. A 3 represents a procedure that is useful, but perhaps not invoked on a regular basis.

November 1, 2006: Training for internal auditors

Today's training for internal auditors was long and tedious, but I learned a lot.

At times it felt a bit daunting because it's so far from anything I've ever done in the past. I suspect I won't feel comfortable with the notion until I've actually done it.

November 10, 2006: Writing our first procedure

Yet another lesson! I thought writing procedures would be an exercise of simply listing the steps in a process, but there is more to it than that. You need a scope, assignment of responsibility, related records, and a revision history. I was surprised that we could still fit most of our procedures onto a single page.

November 29, 2006: Writing more procedures

We've been meeting as a team once a week and taking turns helping each other write procedures. We pick one procedure and allow no more than an hour. Today we discussed the procedure for scheduling of annual audits. Some jobs are a lot more complicated than I realized.

January 18, 2007: The internal audit, part one

As it turns out, the challenge of conducting the internal audit is a function of who is involved in the audit. Our client services coordinator was tasked with interviewing me about the subject management responsibility for her portion of the internal audit. We were both nervous! She didn't like being the one to put the boss under the microscope. As the team leader I felt pressured to have all the right answers. We both survived and learned a lot in the process.

January 24, 2007: Our first management review meeting

We reviewed our QMS in its entirety for the very first time. We discussed the areas required by ISO such as our quality policy, goals, customer feedback, and the results of our internal audit. (We had ten corrective actions!)

February 19, 2007: Training each other on our procedures

It was an uncomfortably long meeting and we only made it through ten procedures. The good news is that we discussed a lot of gaps and areas of overlap.

April 9, 2007: Day one of our ISO 9001:2000 audit

For many years now we have required our fabricators and erectors to go through a rigorous audit in order to obtain their AISC certification. Today, the tables were turned and QMC was put under the microscope.

April 10, 2007: Day two of the audit

It's finally over! Total corrective action requests: four. All in all, it was a very positive experience. Our auditor had a lot of useful suggestions for how we could streamline our processes. It felt like he was a part of the team. I hope that our own auditors instill the same feeling in our program participants.

April 12, 2007: CARs closed

We closed our third and final corrective action today!

April 20, 2007: It's official!

We received our ISO 9001:2000 certificate today! This was a rewarding teambuilding exercise. And now it's time to plan a party for the team; they've worked so hard!

Today: Leading by example

We invested a lot of time and energy in building an effective quality management system and obtaining ISO 9001:2000 certification. Many people have asked, "Was it worth it?" The answer is, "Absolutely!" ISO isn't perfect, but it gave us a logical framework for developing our system, and in the end it has paid off. We continue to strive for excellence with the understanding that there is always room for improvement. Yes, we still make mistakes. Things still fall through the cracks, but this happens much less often these days. ISO has taken us to a level that might not have been attainable a few years ago. We take great pride in our accomplishment and look to the future with an open mind.

If you are considering AISC Certification as a means of enhancing your business, let us help you find your way. We learned the hard way, but you don't have to. For more information call 312.670.7520 or e-mail us at certinfo@aisc.org. MSC

Roberta Marstellar is vice president and Sheila Alegria is client services coordinator with Quality Management Company.

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Engineering Value into Your Project

BY DAVID T. RICKER, P.E., UPDATED BY CHARLIE CARTER, P.E., S.E.

Design economy is a topic that never grows old. Here's an update of a classic MSC article outlining ways to keep your steel projects on time and on budget.

Dave Ricker has been retired and enjoying his explorations in Payson, Ariz. for some time now. His sage advice and years of experience live on, however, in AISC *Engineering Journal* papers, which we often reference when answering questions that come into the AISC Steel Solutions Center.

Recently, while searching for an old article in the archives of MSC, I happened across an article Dave wrote on how to engineer value into a project. It read almost as if he had written it yesterday, since so much of the information remains perfectly applicable today.

Following is a slightly updated version of Dave's April 2000 article. It is interesting how much of the spirit of what Dave recommended years ago is emboldened in a document the Council of American Structural Engineers (CASE) wrote more recently: CASE 962D *A Guideline Addressing Coordination and Completeness of Structural Construction Documents*.

Perhaps the best way to implement Dave's time-honored recommendations is to build a relationship of teamwork among project participants. Everyone brings something to the table that can help the others. Start by talking about Dave's below recommendations and see what other ideas can be harvested.

—Charlie Carter

SUCCESSFUL PROJECTS REQUIRE A TEAM EFFORT, WITH THE OWNER, DESIGNER, FABRICATOR, AND ERECTOR WORKING TOGETHER TO CREATE THE FINISHED STRUCTURE. Each of the key team members have specific roles, and with these roles come responsibilities to the other team members. For example, one of the fabricator's and erector's key roles is to correctly interpret and comply with the designer's instructions. In order to accomplish this goal, however, he or she requires loads, dimensions, and member sizes to be summarized as outlined below:

✔ Beam end reactions for gravity, axial, and torsion loads, as well as moments, should be shown. Likewise, the designer should indicate if live load reductions have or can be taken. And, the designer should indicate whether or not the reactions given are LRFD loads or ASD loads.

✔ Column loads—not only axial, but also shear loads at splices and at the base, plus any moments at beam ends, brackets, and splices—can be shown on the column schedule.

✔ The designer should indicate diagonal axial loads and whether they are in tension, compression, or both. If the designer has preferences for bracing work-point locations or bracing connection design methods, they should be shown.

✔ The fabricator and erector both need to know all special floor and roof loads and point loads for special equipment or service requirements, such as beams supporting construction equipment storage areas or jump cranes, during erection. Such items should be discussed at the pre-construction conference.

✔ The fabricator and erector need to know which beams, if any, are subject to vibration loads such as from machine rooms and elevator beams.

✔ Reactions for special load conditions—such as cantilevered members, two- and three-span beams, beams with both uniform and concentrated loads, and beams with non-uniform snow-drift loads—should be shown.

✔ Specific column stiffener and doubler plate requirements should be shown—including sizes and locations. However, designers should consider the oftentimes more economical option of increasing the column size to eliminate the need for stiffening.

✔ If painting or galvanizing is required, the fabricator and erector need to know the specific requirements, such as surface preparation, which members are to be painted, the type of paint, etc. This information should be expressed using standard SSPC notation.

✔ Special attention should be given to details where steelwork structurally interacts with the work of other trades, such as web openings, support for fascia panels, support for metal deck, etc.

One of the most perplexing situations for fabricators and erectors is when designers don't share the information developed during the design process. During the design process, the structural engineer develops all of the information required to fabricate and connect the structural steel members, including loads, reactions, stiffening requirements, special conditions, etc. But when it comes to the design drawing, the engineer all too often merely shows the member sizes.

Skimping on the design drawing always comes back to haunt the designer in the form of questions, higher bids, change orders, arbitrating disputes, a slower review/approval process, and a dragging construction schedule. If it is a question of time, then the designer is fooling himself or herself. The time the fabricator spends deriving all of the needed information is passed back to the owner in the form of higher fees. And the engineer's approval reviewer has to spend additional time analyzing the questions and change orders.

The solution is greater teamwork and a consciousness of the importance of value engineering. The team member with the greatest impact on the economic success of the project is the designer. The team members all live or die with the engineer's design.

The following is a checklist of items designers should consider while designing a steel project:

Capitalize on steel's strengths. Steel offers good weight-to-strength ratio, efficiency of pre-assembly, speed of delivery and erection, strength in three directions, and ease of modification/renovation.

Keep current on the cost and availability of the various steel products. A steel fabricator can supply basic steel prices and guidance if any of the non-usual grades of steel applicable to a given product should be considered (see Figure 1 and the related information in Part 2 of the 13th Edition AISC *Steel Construction Manual*). A designer also should be aware of where the money is spent on steel construction: approximately 30% on material, 30% on shop costs, 30% on erection, and 10% of other items such as shop drawings, painting, and shipping. Labor is more than 60%!

Consider using partial composite design of floor beams—something in the range of 50% to 75%. Full composite design is often inefficient and uneconomical. The cost of one shear stud in place equals the cost of approximately 10 lb of steel. Unless this ratio can be attained, the addition of more studs will prove uneconomical.

Table 2–3

Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series									
												HSS		
					W	M	S	HP	C	MC	L	Rect.	Round	Pipe
Carbon	A36		36	58-80[b]	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▒	
			50	62								▒		
	A501		36	58								▒	▒	
	A529[c]	Gr. 50	50	65-100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70-100	▒	▒	▒	▒	▒	▒	▒			
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65[d]	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60[e]	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65[e]	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618[f]	Gr. I & II	50[g]	70[g]								▒	▒	
		Gr. III	50	65								▒	▒	
	A913	50	50[h]	60[h]	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50-65[i]	65[i]	■	▒	▒	▒	▒	▒	▒			
Corrosion Resistant High-Strength Low-Alloy	A242		42[j]	63[j]	▒									
			46[k]	67[k]	▒			▒			▒			
			50[l]	70[l]	▒	▒	▒	▒	▒	▒	▒			
	A588		50	70	▒	▒	▒	▒	▒	▒	▒			
	A847		50	70								▒	▒	

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

[a] Minimum unless a range is shown.
[b] For shapes over 426 lb/ft, only the minimum of 58 ksi applies.
[c] For shapes with a flange thickness less than or equal to $1\frac{1}{2}$ in. only. To improve weldability a maximum carbon equivalent can be specified (per ASTM Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM Supplementary Requirement S79).
[d] If desired, maximum tensile stress of 70 ksi can be specified (per ASTM Supplementary Requirement S91).
[e] For shapes with a flange thickness less than or equal to 2 in. only.
[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
[g] Minimum applies for walls nominally $\frac{3}{4}$-in. thick and under. For wall thicknesses over $\frac{3}{4}$ in., $F_y = 46$ ksi and $F_u = 67$ ksi.
[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM Supplementary Requirement S75).
[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory in ASTM A992.
[j] For shapes with a flange thickness greater than 2 in. only.
[k] For shapes with a flange thickness greater than $1\frac{1}{2}$ in. and less than or equal to 2 in. only.
[l] For shapes with a flange thickness less than or equal to $1\frac{1}{2}$ in. only.

Figure 1

Take advantage of live-load reductions if governing codes permit.

Select optimum bay sizes. An exhaustive study by John Ruddy, P.E., formerly of Structural Affiliates International in Nashville and now with AISC (AISC *Engineering Journal*, Vol. 20, No. 3, 1983), indicated that a rectangular bay with a length-to-width ratio of approximately 1.25 to 1.50 was the most efficient. The filler members should span in the long direction with the girder beams in the short direction (see Figure 2).

Tailor the surface preparation and the painting requirements to the project conditions. Do not overdo or under-do the coating requirements. An extensive examination of a multitude of aged structures with steel frames indicates that the presence or absence of a shop primer is immaterial as long as the structural steel is kept dry (see the AISC *Specification* Commentary Chapter M). These same studies indicate that shop primer alone affords very little protection if a structure develops a serious leak.

In recent years, the trend has gone toward not painting. There are many side benefits to be gained by the omission of paint: no masking around bolt holes, better adhesion for concrete and/or fire proofing, easier weldability, ease of inspection, ease of making field repairs/alterations, etc. If shop painting is necessary, bear in mind that a shop coat is by definition a temporary coat—usually serving less than six months in duration. As such, there is little justification that the coat be perfect (i.e., of uniform thickness with no drips, runs, or sags).

Show all necessary loads on the design drawing to avoid costly over-design of connections or—worse yet—under-design. The designer who provides a complete design will find that the subsequent review and approval process of shop drawings will be much quicker and more positive.

Make sure the general contractor or construction manager indicates who is responsible for any "gray areas" such as loose lintels, masonry anchors, elevator sill angles, elevator sheave beams, fastenings for precast concrete spandrel beams, etc. Unless the responsibility is specifically delegated, it is likely that the cost of these items will be included in the bids of multiple contractors, which means the owner will pay more than once for the same article.

Don't require the steel subcontractor to perform work normally done

by other trades, such as installing masonry anchors, ceiling hangers, toilet partition supports, window wall supports, etc. Information required to perform this work is often slow to develop, resulting in needless delay to the fabricator. The most efficient steel jobs are those on which the fabricator and erector are allowed to concentrate on the steel frame while unencumbered by the intricacies pertinent to other trades. This reduces coordination requirements and allows the steel framework to be turned over to the other trades in far less time than would otherwise be possible.

Consider the use of cantilevered rafters and purlins to save weight on roof design (see Figure 3).

Do not design for minimum weight alone. The savings in material cost will often be negated by the need for more members, more connections, and more costly shop work and field erection.

Excessively stringent mill, fabrication, and erection tolerances beyond state-of-the-art construction practices will reduce the number of bidders and raise the cost of the project. ASTM A6 tolerances and those established by AWS and AISC have served the industry well for many years and should be adhered to except under extraordinary circumstances where some special condition dictates a more strict treatment.

Design the proper type of high-strength bolt value. The correct application of each type (snug-tightened, pretensioned, and slip-critical) is well documented in the current AISC and RCSC specifications. Do not specify "slip-critical" bolt values for the purpose of obtaining an extra factor of safety. The trend in recent years is toward the use of snug-tightened bolts and bearing values.

Allow the use of tension control (twist-off) high-strength bolts. These bolts are as reliable as other methods of pretensioned installation and save labor costs in both shop and field.

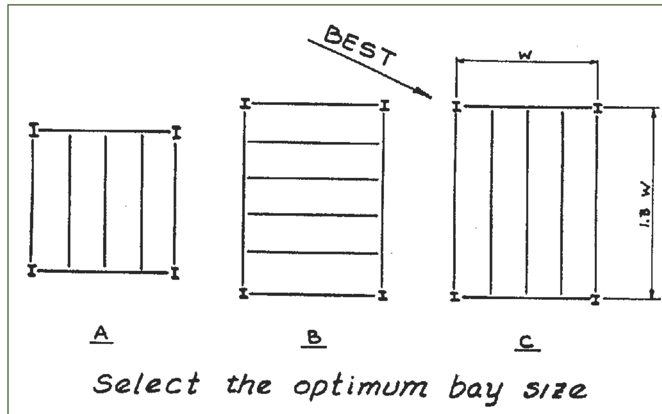

Figure 2

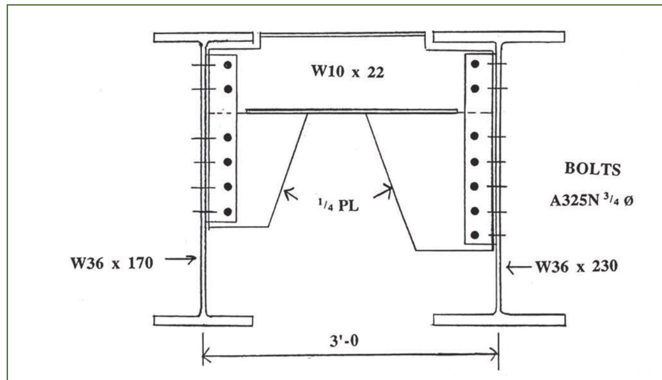

Figure 3

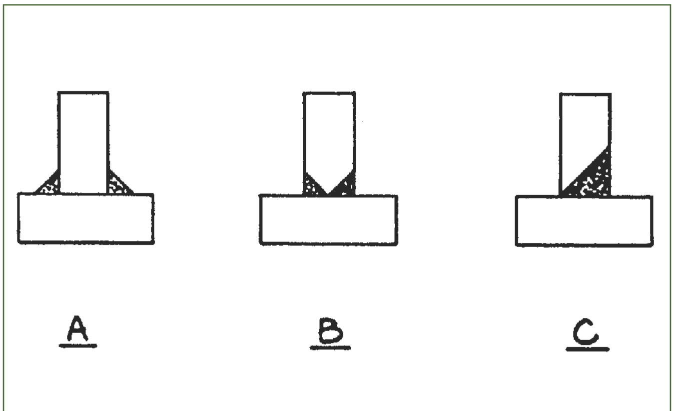

Figure 4

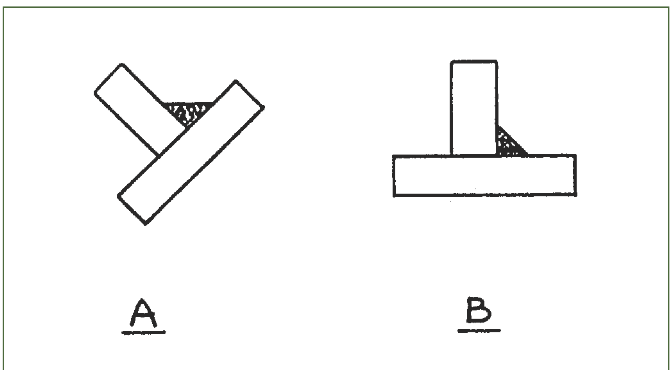

Figure 5

Where possible, specify fillet welds rather than groove welds. Groove welds are more costly because of the joint preparation required and the generally greater volume of weld (see Figure 4).

Use single-pass welds where possible. This involves keeping fillet welds to a maximum of 5⁄16 in.

Favor the horizontal and flat welding positions. These welds are easier and quicker to make, and are generally of high quality (see Figure 5).

Don't specify more weld than is necessary. Over-welding creates excessive heat, which may contribute to warping and shrinkage of the members resulting in costly straightening expense.

Grant the fabricator the option of eliminating some column splices. The cost of one column splice equals the cost of approximately 500 lb of A992 steel. However, the fabricator should study the situation carefully before deciding to omit the column splice, as the resulting column may be too long for safe erection. Multi-tier columns should be designed to have splices every two or four floors. Three-floor columns are to be avoided due to erection difficulties. The higher up in a tall building, the less desirable it is to use four-floor columns due to higher wind speed and difficulties in guying.

Avoid designing column splices at mid-story height. These are often too high for the erector to reach without rigging a float or scaffold. If the splice can be located no higher than 5 ft above the tops of the steel beams, it saves the expense of the extra rigging and still will be in a region of the column where bending forces are relatively low (see Figure 6).

Except where dictated by seismic considerations, do not design column splices to "develop the full bending

strength of the governing column size." Seldom is the splice located at the point of maximum bending and seldom do the bending stresses result in a condition that would require a full-strength splice. The column has axial compression stresses. The excess capacity is allotted to bending stresses that occur as compression in one flange and tension in the other. The compression forces are added to each other at one flange while at the other flange the tension force is subtracted from the compression force. Seldom does this other side of the column ever go into tension and almost never into full allowable tension of the magnitude that would require a full-strength splice. Thus, except in high-seismic construction, there often is little justification for requiring a full-strength column splice (see Figure 6).

Consider using a heavier column shaft to eliminate the need for web doubler plates and/or column stiffeners opposite the flanges of moment-connected beams. One pair of stiffeners installed costs approximately the same as 300 lb of A992 steel if the stiffeners are fillet welded. If they are groove welded, the cost skyrockets to the equivalent of 1,000 lb of A992 steel. The cost of one installed doubler plate is about the same as 350 lb of A992 steel (see Figure 7). Considering that for an average two-floor column there could be as many as four pairs of stiffeners and two or more doubler plates, at least 1,900 lb of A992 steel could be sacrificed in order to save the time and expense of making the lighter shaft compliant. For more information, see AISC *Design Guide 13: Wide-Flange Column Stiffening at Moment Connections*.

Avoid designing heavy or awkward members in remote, hard-to-reach portions of the structure. This may eliminate the need for larger, more expensive hoisting equipment.

Reinforce beam-web penetrations only where necessary.

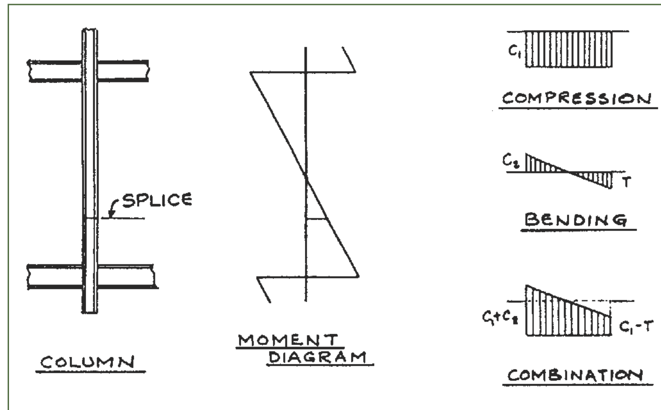

Figure 6

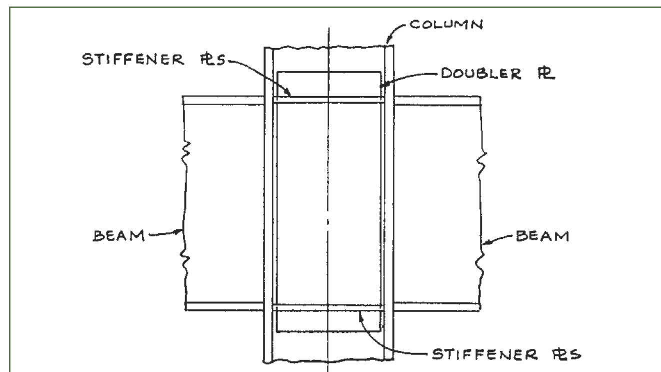

Figure 7

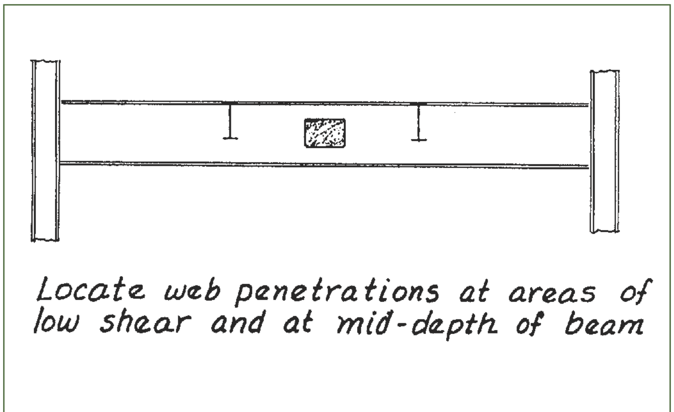

Figure 8

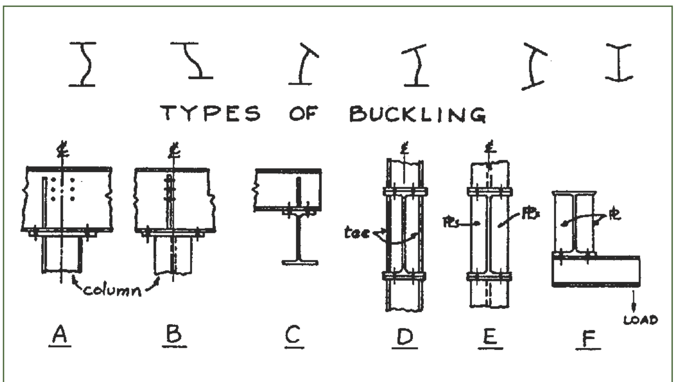

Figure 9

It may be less costly to use a beam with a thicker web, to move the opening to a less critical location, or to change the proportions of the opening to something less demanding (see Figure 8). To help in designing web openings, AISC published *Design Guide 2: Design of Steel and Composite Beams with Web Openings*. AISC also offers a software program, Webopen, to help in designing web openings.

Allow the prudent use of oversized holes and slots to facilitate fit-up and erection. They may eliminate or reduce the need for costly reaming of holes or modification of connection parts in the field.

For ordinary structures, do not specify that connection material be of one type to the exclusion of other types. Allow the fabricator to use stock materials to good advantage. However, the fabricator should recognize that certain structural situations require specific types of steel. The designer should identify these special conditions.

Avoid calling for the indiscriminate use of stiffeners. Allow partial-depth stiffeners where applicable. Stiffeners are required to prevent local deformation or to transfer load from one part of a member to another (see Figure 9). If the main members are capable of taking care of themselves, then the cost of stiffeners can be saved.

Avoid odd sections that may not be readily available or which are seldom rolled, since this could result in costly delays. Consult with a fabricator concerning the availability of specific shapes.

In areas subject to snow drift loading, arrange the purlins parallel to the drift and space the purlins closer together as the drift load increases so the same gage roof deck can be used throughout (see Figure 10).

Space floor beams so as to avoid the necessity for shoring during the concrete pour. The cost of shoring is relatively expensive and can easily be offset by varying the span, gage or depth of the floor deck.

Avoid the "catch-all" specification that reads something like this: "Fabricate and erect all steel shown or implied necessary to complete the steel framework." The bids will undoubtedly be padded to cover whatever might be "implied." Or worse: arguments and extras!

Avoid the "nebulous" specification calling for stiffeners, roof frames, reinforcing of beam web penetrations, etc., "as required." The fabricator and erector are rarely furnished with enough information at the bid stage to determine what is or is not required and therefore will include in the bid an allowance for investigating and furnishing the questionable items whether they're needed or not.

Avoid overly restrictive specifications. The more restrictions listed in the steel specifications, the greater the chance that no one will be able to meet them all. This will eliminate some competition and result in higher bids.

Design for duplication of beam sizes where possible, since this results in economies of scale. For example, in a mezzanine the edge beams often carry less load and could be made smaller but for the sake of duplication make them the same.

Likewise, design for duplication of connections. For example, if most of the filler beams on a job can be connected using a four-bolt shear plate but a few require only a three-bolt shear plate, make them all four-bolt connections. This miniscule "give-away" is more than made up by the efficiencies of duplication—both for the shop and field. Connection material rarely exceeds 5% of the job total weight. Trying to save a tiny percentage of 5% is not cost-effective if it leads to special handling, marking, sorting, and other special treatment of the members in question. MSC

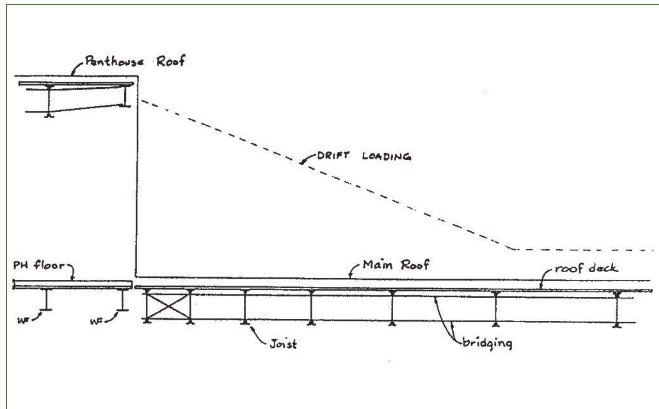

Figure 10

David T. Ricker, P.E., began his career in the American Bridge Division of U.S. Steel and later moved to Berlin Steel (Berlin, Conn.), eventually becoming the company's chief engineer and then its vice president of engineering. Now retired, he lives in Payson, Ariz.

Charlie Carter is AISC's chief structural engineer.

Primary Concerns

Primary research involving input from clients and industry experts should be a key element in developing your firm's marketing strategy.

BY ANNE SCARLETT

WHEN MARKETING YOUR A/E/C FIRM TO POTENTIAL CLIENTS, RESEARCH IS AN INTEGRAL PART OF THE STRATEGIC PLANNING PROCESS. It helps shape the direction of your efforts—what markets you will enter, expand within, or exit. It also assists in best reaching subsector groups within markets, because you learn the nuances that will most appeal to the decision makers.

Primary research is the most important type of research to your marketing endeavors. Its purpose is to answer your company's specific questions, and it is typically gleaned from direct exposure to sources through focus groups, interviews, surveys, observation, and testing.

As technical leaders within your firm, you can play an integral role in facilitating primary research, especially within your existing markets. Why? Because you are the ones regularly engaged with your clients. By working alongside your marketing/business development team, you can collaborate on research to collect the data that will best guide your firm's future business decisions.

Asking the Right Questions

Part of the research process is asking questions that are specific to your firm and its marketing efforts. Some very beneficial insights might come out of asking your clients questions such as these:

- ✓ What trends do you anticipate over the next three years for your company? How do you see things changing, and why?
- ✓ What imminent economic, social, or political issues may impact your company, and why?
- ✓ What are the biggest challenges that your company/industry is currently facing?
- ✓ How might an A/E/C firm help you to address these challenges? Are there additional services that you would suggest?
- ✓ How do you see your personal role in your company changing in the next three years?
- ✓ What are the top three decision making criteria that your company uses when selecting an engineering firm?
- ✓ What forms of marketing does your firm consider most effective? What kind of information would you find most helpful, and in what form would you like it communicated?

Meeting Planning

Once you determine the data you need, it's time to get people talking, sharing, and brainstorming. Here are four solid techniques to get the most out of your primary research:

Integrate your research within a client perception discussion. You need to get client feedback anyway, so why not ask for industry insights at the same time you are discussing their perception of your firm and its performance? When you are dealing with clients with whom you hope to have an ongoing (even lifetime!) relationship, my suggestion is to hold a focus group upon the project's completion. Invite an array of people representing various segments of the project: your day-to-day client contact, project owner, contractor/construction manager, sub-consultant(s), real estate entity, etc.

The cool thing about multidisciplinary focus groups is that attendees will remind one another of what occurred during the project. Of course, these meetings can backfire if the project has experienced some bloopers; only hold them if you're willing to professionally manage potential conflict.

> In a well-run focus group, every participant can learn something.

Once you've received feedback regarding the project—which will likely include some fantastic testimonials for future marketing strategies—you can then move on to facilitate a dialogue around industry insights. If you have ever participated in a well-run focus group, then you know that it's actually quite fun to hear one another's perspectives. A well-run focus group will leave participants feeling that their time was well spent—not just as an obligatory clearinghouse for feedback, but in terms of learning something new.

Emphasize the value of the client's expertise by including their input in an article or white paper. Do you write articles for publications, craft white papers to entice prospects, and submit abstracts to speak at conferences? I hope your answer is yes, and if so, why not involve a client or pros-

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SCARLETT CONSULTING

pect in any or all of these efforts? By letting them know your intentions for picking their "expert" brains, they will be more than happy to grant you an interview and share their thoughts. Not only will you have new data to support or guide your firm's marketing strategy, you'll also have information to add to your position within the article, white paper, or abstract, and you will be building a stronger rapport in the process! It's win-win-win situation.

Stress that having clients and experts help you can very well help them. Even in business-to-business contexts, people love to talk about themselves. It may even be their favorite subject. So why not stress the fact that their willingness to share their perspectives and industry knowledge could quite possibly help you to help them in their professional roles, even if you are dealing with those at the ownership level? In fact, owners may reap more benefits than anyone, since they are (often) more vested than anyone in the growth of the company at large. By sharing challenges and trends with you, there is the potential that your firm could, in turn, evolve and innovate to better serve their business purpose in the future.

Host a private roundtable forum. Just as professional organizations hold private, invitation-only roundtables, so can your firm. Many A/E/C firms organize special events for clients and prospects. It's not always easy to coordinate these parties, but it can be done, especially when you are bringing your own existing clients together. Let's say that once a year, you invite representative, non-competing clients (just a few, not all of them) to your firm as a thank you for their business. Perhaps your agenda involves a program where you discuss big-picture external issues that have varying impacts on the attendees. Attendees can share case studies and lessons learned in relation to centralized themes (e.g., budget approvals, internal workflow/processes, technology, etc.). Perhaps you wrap up the business portion of the program by pairing people up to brainstorm on one another's business challenges. You then conclude with time for socializing and networking over a meal; good food is a divine motivator!

Be the Baker

Primary research is like cookie dough. Once you've collected and mixed all the ingredients—all of the insights from your sources—you can roll it out and cut it into whatever shape best suits your company's marketing plan. Just make sure it isn't half-baked. MSC

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Joist Cause

BY P.J. MORAN, P.E., AND ERIC JENSEN, P.E.

Providing the proper information and checking the computer's work will keep the joist portion of any construction project on track.

Open web steel joists can be adapted to a variety of geometric configurations.

OPEN WEB STEEL JOISTS—OR BAR JOISTS, AS THEY ARE COMMONLY CALLED—HAVE BEEN USED IN BUILDING CONSTRUCTION SINCE THE EARLY 1920S. Their original intended purpose was to provide an economical lightweight truss to be utilized as a single-span, uniformly loaded member that supported dead loads, including roofing materials and other permanently affixed materials and equipment, and live loads. Standard details were easy to apply; and the designer could easily enter the economical joist selection guide and pick a standard joist.

Today, however, steel joists have evolved into structural support components with complex geometries and can be designed for numerous loading conditions and load combinations. And thanks to their structural efficiency, steel joists are a sought-after solution for complicated framing conditions. Although many joist manufacturers have the ability to design and build joists for many different conditions, the costs and fabrication lead times for specialty products have increased significantly. In today's fast-track environment, keeping things simple and providing all required information will help manufacturers stay on schedule.

Most structural steel framing members are designed and specified by the project's engineer of record, and solid web beams and columns are typically noted on the structural drawings. The steel fabricator and detailer develop material cut sheets and produce shop drawings directly from these drawings, while the steel joist manufacturer develops its own erection drawings and bills of material for the joists, joist girders, and bridging prior to final design and fabrication. In addition to the physical dimensions required to fabricate the joists and girders, the joist manufacturer must acquire all of the specific loading information in order to complete the designs. Unfortunately, much of the information required for special design requirements for the joists and joist girders is not contained on the structural drawings and must be coordinated between the joist supplier, steel fabricator, general contractor, and engineer.

Below are some tips to make the overall process of selecting, designing, and manufacturing joists more efficient. Following them will allow manufacturers to provide economical and quality products that meet project design requirements and schedules.

Compute, Then Check. Advances in computer technology and proprietary software programs have increased the ability of steel joist manufacturers to design and accommodate a large variety of conditions that meet the structural framing requirements for very complex buildings. Multiple software packages are now available that help manufacturers select standard designation joists. However, software cannot apply engineering judgment; it may select joists that are individually economical, but that might not be economical on a collective basis. For example, it is often more economical to not change joist sizes within a given bay or when the bay is skewed. There is a net loss of economy when joist designations are changed, even though a specific member has been optimized. The joist selection software may not account for bridging or special erection requirements. It is therefore always prudent to review the results of software-generated joist selections and make adjustments as necessary.

Keep KCS in Mind. KCS series joists (a Steel Joist Institute designation) are usually heavier than standard or load-per-foot designation joists. However, these joists provide excellent versatility and flexibility when loading requirements are unclear or if future changes are anticipated. Over the long term, KCS may be the most economical solution due to additional time and costs that result from design changes; design changes to existing joists often result in expensive joist modifications. Overall costs may be lower because KCS joists are "standard" joists with no special engineering by the joist manufacturer, no special load coordination issues, and no special detailing requirements. It is advisable to use KCS joists wherever possible to minimize the use of "SP" joists and special load diagrams. Remember, KCS joist selections are all-inclusive. In other words, all loads must be included in the KCS designation, except for net uplift. Do not specify a KCS joist with additional concentrated loads or other types of additional loads.

Mechanical Matters. Another method to simplify design is through the use of mechanical zones. Often, the final size, location, and weight of mechanical units are unknown when the joist manufacturer begins to detail the job. Therefore, the best solution may be to design whole bays or areas for variable mechanical unit placement. Instead of showing and instructing the joist manufacturer to design each joist for every individual mechanical unit, which may change before the designs are final, instruct them to design for an additional concentrated load at any panel point on all joists within the mechanical zone. This may be a better solution than the use of KCS joists. For example, an envelope solution for a mechanical zone may be to specify each joist to be designed for a 1,000-lb concentrated load at any panel point. This is an economical compromise that still provides needed flexibility and avoids expensive retrofits or joist replacements when changes occur after joist fabrication.

Don't Double. In the past, it was common to use double joists when extra capacity was needed to accommodate additional loads such as mechanical units. As previously described, it is more economical and practical to use mechanical zones or KCS joists, or to specify additional loads to take care of additional capacity requirements. In addition, if bolted cross bridging is required, it is not possible to install the bridging and still meet OSHA erection requirements when joists are doubled.

Space Out. Often, joists are spaced based on the designer's past experience, using rules of thumb or simply dividing the girder length by some arbitrary number. It is almost always more economical to maximize the span of the specified deck when determining joist spacing. (If applicable, remember to check Factory Mutual requirements for maximum deck spans.) Fewer joists result in lower erection costs. In addition, fewer, heavier joists may require less rows of bridging, which will also reduce erection costs.

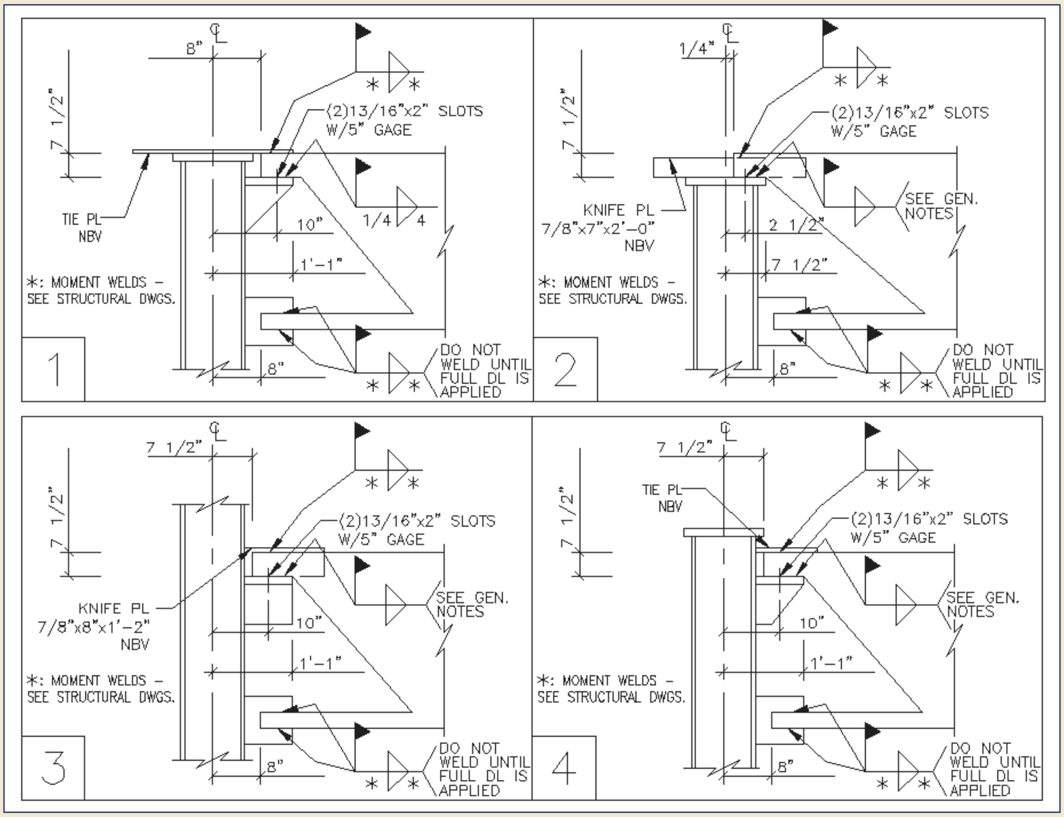
Camber Carefully. Oftentimes, disputes arise over mismatched elevations due to standard joist camber. This problem can be avoided by not mixing elements of different stiffness within a bay or interrupting spans of horizontal framing members with columns (e.g., if a bay is framed with 80-ft-long joists, avoid framing the end span with an intermediate column and two 40-ft beams). Do not place joists directly adjacent to walls (masonry or concrete) running parallel to joists. Instead, provide a detail for the deck to be supported on the wall or by an angle attached to the wall, and then provide a full joist space to the first joist adjacent to the wall. Standard camber will not usually be a significant problem because the deck can be easily pushed down and attached at the wall.

Standard joist camber may be excessive at spans around 80 ft to 90 ft. Consideration should be given to the load conditions at which the joists should be level. If camber is a concern, the manufacturer should be given specific instructions to provide special camber and indicate the loads at which the joists are expected to be level. The joist manufacturer will calculate the deflection under those loads and provide camber accordingly. There may be an additional expense for special camber, but special camber is not nearly as expensive as the remedies that are required to correct undesirable camber problems.

Making Ends Meet. It is not uncommon for structural drawings to show 3/16-in. or ¼-in. fillet welds for end anchorage. Many joist bearing seats are fabricated using material that is only ⅛ in. thick. If possible, specify ⅛-in. fillet welds with sufficient length to meet the anchorage requirements. If required, bearing seat thickness of 3/16 in. or ¼ in. can be supplied at an additional cost.

The Rule of Four. Standard top chord extensions were developed in order to give designers an idea of the capacity of top chord extensions on standard joists. For example, published capacities for R4 type extensions are capacities expected for extending the top chord and bearing seat angles of K4 type joists. Therefore, specifying an R10 extension on a K3 joist may present a mismatch problem—an extension and a joist that are incompatible. A good rule of thumb is to limit the numerical difference between the designation size and extension size to *four*. (For example, don't specify an R8 extension on a K3 joist, as the numerical difference is five.) Contact the joist manufacturer for help with other options when this limit is exceeded. Also keep in mind that deeper bearing seats may provide additional capacity for long or heavily loaded extensions

Lightening the Load. Even though there are published capacities for standard joist girder bearing seats subjected to



Suggested details for transferring lateral forces into columns.

lateral loads, a detail that transfers lateral loads directly from the joist or girder top chord to the supporting structural member is recommended (see above figure). Published values illustrate that there is very little additional capacity available in bearing seats to transfer lateral loads. In addition, the deformation of standard bearing seats under lateral loads may have a negative impact on frame drift.

Bearing it All. K and KCS series joists require 2½-in. minimum bearing on steel, while LH series joists require 4-in. minimum bearing on steel. Sufficient bearing length must be provided to meet required minimums when joists bear on wide-flange beams or other structural support members. A typical problem occurs when joists oppose each other on small wide-flange beams. To resolve this problem, use 6-in. minimum flange widths for supporting members for opposing K series joists and 9-in. minimum flange widths for supporting members for opposing LH series joists. Staggering joists over narrow beams requires additional detailing, complicates the deck layout, and increases the difficulty of joist and deck erection. MSC

Eric Jensen is an engineering manager and P.J. Moran is an engineer with Nucor Vulcraft-Texas in Grapeland, Texas.

products

STEEL JOIST PRODUCTS

Company	Products Offered	Product Description
All View System www.allviewsystem.com **787.786.9878**	**JoistLab v1.0a + Girders Module**	Windows-based software to analyze, design, and detail steel joists and joist girders, following SJI design specifications. First and only ready-to-use software with such capacities available in the market.
Canam Steel Corporation www.canam.ws **301.874.5141**	**Steel Joist Institute open-web steel joists**	Short-span, mid-span and long-span, K, KCS, LH, and DLH series joists in accordance with Steel Joist Institute specifications.
	Joist girders and special trusses	Spans to 120 ft, depths to 120 in., per SJI specifications; custom steel trusses at plants certified by AISC, and welders certified by AWS and CWB.
	Purlins/girts	Cold-formed using high-strength steel to minimize weight while maximizing capacity.
	Long-span and complex joists	Longer than SJI from 144 ft to beyond 200 ft; barrel, bowstring, scissor, and double- and triple-pitched, with engineering of field-bolted splices and complex connections.
CMC Joist & Deck www.cmcjd.com **800.643.1577** (Hope, Ark.) **800.763.0026** (Eastover, S.C.) **800.706.6476** (Starke, Fla.) **888.643.1577** (Fallon, Nev.) **888.290.6875** (Iowa Falls, Iowa) **570.568.6761** (New Columbia, Pa.) **915.298.5050** (Juarez, Mexico)	**Steel joist products, including the K, LH, and DLH series and girders**	The CMC Joist & Deck Division produces long-span steel joists, deep long-span joists, and joist girders. With seven strategically located fabrication plants providing nationwide service, CMC Joist & Deck's goal is to build products that exceed customer expectations and provide on time delivery, every time, with "NO EXCUSES." The in-house Joist Test Lab demonstrates our commitment to improving products and practices through research and development. CMC Engineering Centers offer detailing and engineering services in addition to construction solutions.
The D.S. Brown Company www.dsbrown.com **800.848.1730**	**Expansion joints, bearing assemblies, and specialty products**	Bridge Products: D.S. Brown's extensive product line includes: Steelflex Modular Expansion Joint Systems, Versiflex Bearing Assemblies, Cableguard Elastomeric Wrap, Exodermic Bridge Deck, Fiberbond FRP (Fiber Reinforced Polymer) System, and other specialty products. D.S. Brown is fully integrated, performing and controlling all requirements of a project internally: research and development; engineering design/CAD detailing; rubber compounding, mixing, extruding, and molding; and custom steel fabrication and machining.
Gooder-Henrichsen Co. gooder@sbcglobal.net	**Open-web steel joists, long-span joists, joist girders, and specialty joists**	A long-standing member of the Steel Joist Institute and associate member of AISC, Gooder-Henrichsen has been in business since 1927 and producing joists since the 1960s. With a newly upgraded 110,000-sq.-ft facility near Chicago, Gooder-Henrichsen has expanded its product line to include not only all SJI specified products, but also specially engineered composite joists, gable joists, scissor joists, bow-strings, barrel joists, and super-long-spans (exceeding 200 ft long). Our customers enjoy friendly and responsive service, as we establish a reputation for the fastest production cycle in the industry.
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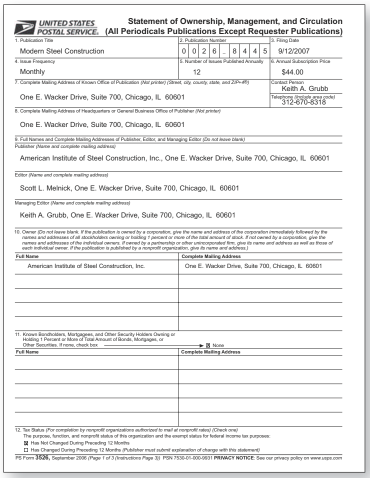
Nucor Vulcraft Group www.vulcraft.com **435.734.9433** (Brigham City, UT) **607.529.9000** (Chemung, NY) **843.662.0381** (Florence, SC) **256.845.2460** (Fort Payne, AL) **936.687.4665** (Grapeland, TX) **402.644.8500** (Norfolk, VA) **260.337.1800** (St. Joe, IN)	**Steel joist products**	Nucor Vulcraft Group is the nation's largest producer of steel joists, joist girders, and steel deck. Vulcraft also produces highly engineered products, such as composite floor joists. Vulcraft's seven facilities across the United States produce roughly 800,000 tons to 1 million tons of joist and deck each year. A variety of joist and joist girder configurations are available for architectural consideration, including arched chord, bowstring, scissor, and single- and double-pitched designs. Vulcraft's engineers are experienced with customized applications. Vulcraft products, made from more than 90% recycled materials, have been essential elements in green buildings.
Quincy Joist Company www.quincyjoist.com **850.875.1075**	**Open-web steel joists and joist girders**	Quincy Joist Company manufacturers all types of open-web steel joists and joist girders in compliance with Steel Joist Institute standards and specifications. Open-web steel joists are manufactured with the highest quality standards in the industry to meet our customers' requirements. We have the engineering capability to design and manufacture joists for all your specialty projects or fast-track requirements.
Steel Joist Institute www.steeljoist.org **843.626.1995**	**Standards, specifcations, load tables, and weight tables catalog**	The 183-page reference is ANSI-certified.
	Industry organization	Besides setting standards for the steel joist industry, the Steel Joist Institute works closely with major building code bodies throughout the country to develop code regulations regarding steel joists and joist girders. The Institute also invests in research related to steel joists and joist girders and offers a library of publications, ongoing educational seminars, and research aids such as a steel joist identification service.

STEEL JOIST PRODUCTS

Company	Products Offered	Product Description
Valley Joist, Inc. **www.valleyjoist.com** **800.633.2258**	**K, LH, and DLH joists and joist girders**	Valley Joist is a manufacturer of steel joists (K, LH, and DLH and joist girders) and steel deck (roof deck, composite deck, and form deck). Valley Joist also fabricates a wide range of non-standard joists and joist girders including bow strings, rainbow joists, single-pitch, double-pitch, scissor joists, and gable joists. Valley offers customers one-stop shopping and provides quick service. Valley offers full customer service, beginning with a computerized detailing department. Complete AutoCAD placement drawings are produced for each customer's building. Valley's fleet of trailers and modern tractors deliver products the morning construction begins. Valley Joist has delivered buildings as far north as Alaska, as far south as Puerto Rico, and as far west as the Marshall Islands.



UNITED STATES POSTAL SERVICE

Statement of Ownership, Management, and Circulation (All Periodicals Publications Except Requester Publications)

1. Publication Title: Modern Steel Construction	2. Publication Number: 0 0 2 6 _ 8 4 4 5	3. Filing Date: 9/12/2007
4. Issue Frequency: Monthly	5. Number of Issues Published Annually: 12	6. Annual Subscription Price: $44.00

7. Complete Mailing Address of Known Office of Publication *(Not printer) (Street, city, county, state, and ZIP+4®)*: One E. Wacker Drive, Suite 700, Chicago, IL 60601 — Contact Person: Keith A. Grubb — Telephone *(Include area code)*: 312-670-8318

8. Complete Mailing Address of Headquarters or General Business Office of Publisher *(Not printer)*: One E. Wacker Drive, Suite 700, Chicago, IL 60601

9. Full Names and Complete Mailing Addresses of Publisher, Editor, and Managing Editor *(Do not leave blank)*

Publisher *(Name and complete mailing address)*: American Institute of Steel Construction, Inc., One E. Wacker Drive, Suite 700, Chicago, IL 60601

Editor *(Name and complete mailing address)*: Scott L. Melnick, One E. Wacker Drive, Suite 700, Chicago, IL 60601

Managing Editor *(Name and complete mailing address)*: Keith A. Grubb, One E. Wacker Drive, Suite 700, Chicago, IL 60601

10. Owner *(Do not leave blank. If the publication is owned by a corporation, give the name and address of the corporation immediately followed by the names and addresses of all stockholders owning or holding 1 percent or more of the total amount of stock. If not owned by a corporation, give the names and addresses of the individual owners. If owned by a partnership or other unincorporated firm, give its name and address as well as those of each individual owner. If the publication is published by a nonprofit organization, give its name and address.)*

Full Name	Complete Mailing Address
American Institute of Steel Construction, Inc.	One E. Wacker Drive, Suite 700, Chicago, IL 60601

11. Known Bondholders, Mortgagees, and Other Security Holders Owning or Holding 1 Percent or More of Total Amount of Bonds, Mortgages, or Other Securities. If none, check box → ☒ None

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12. Tax Status *(For completion by nonprofit organizations authorized to mail at nonprofit rates) (Check one)*
The purpose, function, and nonprofit status of this organization and the exempt status for federal income tax purposes:
☒ Has Not Changed During Preceding 12 Months
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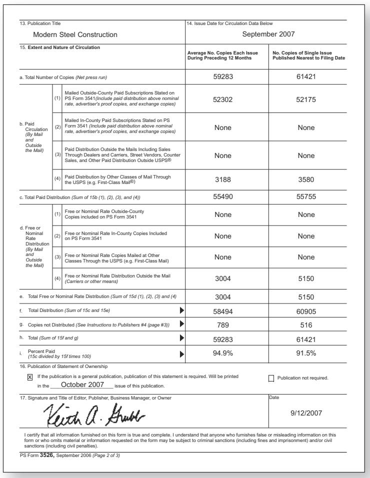

13. Publication Title: Modern Steel Construction — 14. Issue Date for Circulation Data Below: September 2007

15. **Extent and Nature of Circulation**	**Average No. Copies Each Issue During Preceding 12 Months**	**No. Copies of Single Issue Published Nearest to Filing Date**
a. Total Number of Copies *(Net press run)*	59283	61421
b. Paid Circulation *(By Mail and Outside the Mail)* (1) Mailed Outside-County Paid Subscriptions Stated on PS Form 3541 *(Include paid distribution above nominal rate, advertiser's proof copies, and exchange copies)*	52302	52175
(2) Mailed In-County Paid Subscriptions Stated on PS Form 3541 *(Include paid distribution above nominal rate, advertiser's proof copies, and exchange copies)*	None	None
(3) Paid Distribution Outside the Mails Including Sales Through Dealers and Carriers, Street Vendors, Counter Sales, and Other Paid Distribution Outside USPS®	None	None
(4) Paid Distribution by Other Classes of Mail Through the USPS (e.g. First-Class Mail®)	3188	3580
c. Total Paid Distribution *(Sum of 15b (1), (2), (3), and (4))*	55490	55755
d. Free or Nominal Rate Distribution *(By Mail and Outside the Mail)* (1) Free or Nominal Rate Outside-County Copies included on PS Form 3541	None	None
(2) Free or Nominal Rate In-County Copies Included on PS Form 3541	None	None
(3) Free or Nominal Rate Copies Mailed at Other Classes Through the USPS (e.g. First-Class Mail)	None	None
(4) Free or Nominal Rate Distribution Outside the Mail *(Carriers or other means)*	3004	5150
e. Total Free or Nominal Rate Distribution *(Sum of 15d (1), (2), (3) and (4)*	3004	5150
f. Total Distribution *(Sum of 15c and 15e)*	58494	60905
g. Copies not Distributed *(See Instructions to Publishers #4 (page #3))*	789	516
h. Total *(Sum of 15f and g)*	59283	61421
i. Percent Paid *(15c divided by 15f times 100)*	94.9%	91.5%

16. Publication of Statement of Ownership
☒ If the publication is a general publication, publication of this statement is required. Will be printed in the October 2007 issue of this publication. ☐ Publication not required.

17. Signature and Title of Editor, Publisher, Business Manager, or Owner: Keith A. Grubb — Date: 9/12/2007

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AISC Engineering & Research Dept.
Senior Engineer

How would you like to help write the next AISC Manual? If you are looking for the next step in your career, have excellent people skills, and like new challenges, AISC is the place for you. AISC is now seeking applicants for a Senior Engineer position in the AISC Engineering & Research Dept. This is a unique opportunity to apply your design experience in developing AISC's Specifications, Manuals, Design Guides, and other technical publications. You will work with the top engineers, educators and leaders in the North American steel design and construction industry, and will be in a position to establish yourself as an expert in steel design. The position will allow you to use your technical expertise and background to develop tools that benefit the entire structural engineering community and structural steel industry.

Applicants should have a BS degree with structural emphasis in architectural or civil engineering, with an MS or M. Eng. preferred, and a minimum of 3 to 5 years of design experience is required. AISC provides a great working environment offering professional development opportunities, flexibility, excellent resources and a competitive salary with an excellent benefit package.

Send resumes to Cynthia Duncan at **duncan@aisc.org**.

BIM Modeler

Progressive structural engineering firm seeks a structural 3-D Building Information Modeler interested in an opportunity to join an innovative team involved with adopting a more advanced technological approach to designing, documenting and constructing building facilities across the country.

Responsibilities will include working with design and construction teams in a 3D environment to design and coordinate multi-discipline building design projects and assist the interoperable flow and use of 3D models to the construction team. Qualifications include 3-5 years Tekla detailing or Revit modeling experience and fluency in Autocad.

Submit resume and project biography to Structural Consultants, 3400 E. Bayaud Ave. #300, Denver, CO 80209, fax 303-333-9501. Email: joe@sci-denver.com.