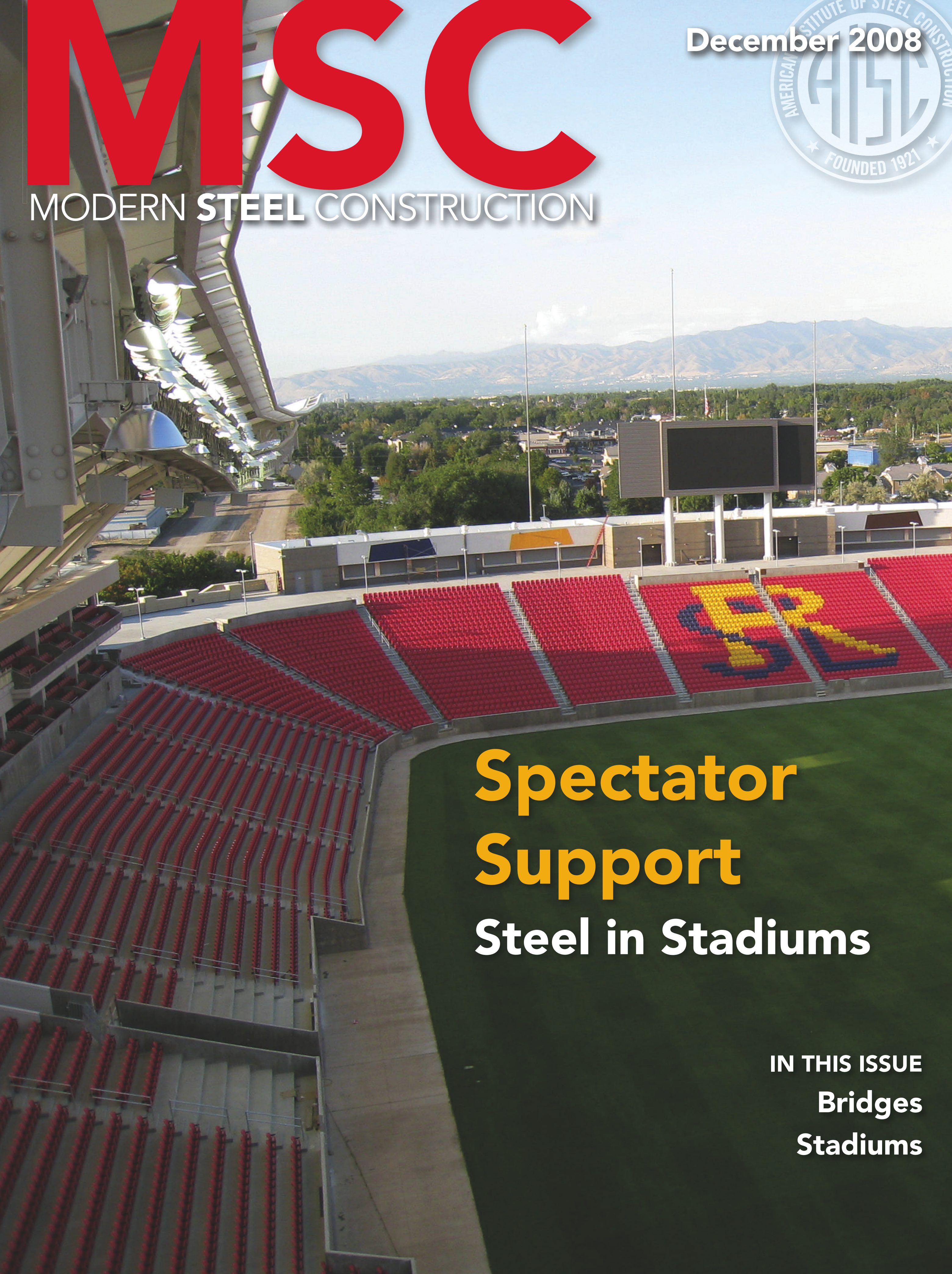

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December 2008

Spectator Support
Steel in Stadiums

IN THIS ISSUE
Bridges
Stadiums

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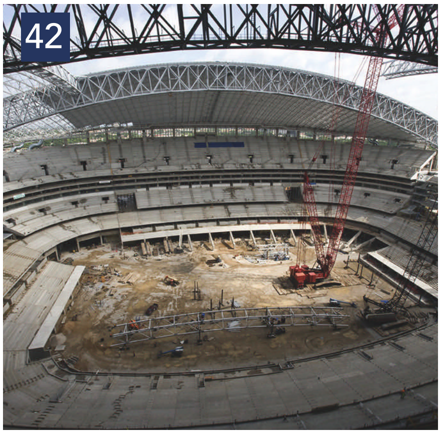

ON THE COVER: Rio Tinto Stadium, Sandy Lake, Utah. (Photo courtesy of Rio Tinto Stadium)

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editor's note

HOLDING A SOCCER TOURNAMENT IN THE CHICAGO SUBURBS IN NOVEMBER IS ALWAYS IFFY, BUT THIS YEAR THE STARS ALIGNED AND THE WEATHER WAS ABSOLUTELY BEAUTIFUL. And even though it was a U12 tournament, we had full referee teams on each field—a rarity for our region. The first two games I worked the lines, but the third game I was the center ref. As I stood at the center circle chatting with my referee team, one of the players came up and asked if I was refereeing the upcoming game. I said I was; her face lit up, she exclaimed "Great!" and she actually high-fived me. I was a bit dumbfounded and stammered to the other referees that I've refereed a lot of U12 games this year and I guess I'm a bit popular with some of the players. But when I recounted the story to my wife later, I was actually quite proud; it's nice to be recognized for your efforts.

Too often structural engineers and other members of the design and construction team are not recognized for their contributions. And they should be. That's one of the reasons behind AISC's Innovative Design Engineering and Architecture with Structural Steel awards (IDEAS^2).

I love this program because it doesn't just honor the megaprojects that everyone already knows about; rather it offers separate prizes for big and small projects. So what's the criteria? Judges look for creative solutions, exciting aesthetics, and bold use of advanced technology. Some projects win because they look great; others because of an innovative structural solution. Some win for their use of innovative design methodology; others for efficient construction technology.

Of course, there's also a more highfalutin explanation of the competition: "The design and construction industry is growing due to the value of coordination, collaboration and teamwork in the successful accomplishment of a project's program. In active support of this trend, AISC has brought together previously separate architectural and structural engineering award programs that focused on a single aspect of the building project, into a single program designed to recognize excellence and innovation in the use of structural steel on a comprehensive, project basis."

The winning projects will be featured at next year's NASCC: The Steel Conference (visit **www.aisc.org/nascc** to view the advance program for this must-attend conference), an upcoming issue of *Modern Steel Construction*, and during award ceremonies held around the country at the project sites.

To see a full description of the competition and learn how to enter, visit **www.aisc.org/ideas**.

I look forward to reading about your winning project!

SCOTT MELNICK
EDITOR

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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.

ASTM A500 Rounds

A steel subcontractor on our project says he has a U.S.-made A500 Grade B round HSS. I believe that this material only comes in square and rectangular shapes. Can you confirm the availability of these shapes?

Round HSS are routinely produced to meet the requirements of ASTM A500 Grade B; it is in fact the usual material specification for round HSS in the U.S. The mechanical properties are slightly different than rectangular/square, but it is still A500 Grade B.

For square/rectangular shapes the minimum yield stress for ASTM A500 grade B is 46ksi, and the minimum tensile stress is 58ksi.

For round HSS in ASTM A500 grade B, the minimum yield stress is 42ksi, and the minimum tensile stress 58ksi.

This can be verified by consulting Table 2-3 of the 13th edition AISC *Steel Construction Manual*. Availability can also be verified by using the availability database on the AISC web site, which lists several domestic producers: **www.aisc.org/availability**.

As a general rule of thumb, ASTM A500 HSS cross sections that match up with ASTM A53 pipe cross sections are available. For other cross sections, check with a fabricator, steel service center, or mill.

Martin Anderson

A325SC Bolts?

How are ASTM A325SC bolts designed and installed? In past projects we typically used A325N and we are now using A325SC. Do both of these types of bolts need to be checked for bearing?

When dealing with high-strength bolted connections, it is probably best to consider the difference in terminologies pertaining to the bolt type versus that of the connection type. The A325 example refers to the bolt type, while the SC or N refers to required installation details for the connection type and geometry respectively.

Bolt Types

The most common types of bolts used in structural steel applications meet either the ASTM A325 or A490 Standard. There are also twist-off types of bolts, which are equivalent to these bolt types: ASTM F1852 equivalent to ASTM A325, and ASTM F2280 equivalent to ASTM A490. The property requirements are the same for a particular bolt type regardless of the type or details of the connection in which it is to be used.

Connection Types

There are three basic connection types used in structural steel applications: Snug-Tightened, Pretensioned, and Slip-Critical. Descriptions of these connection types and installation requirements can be found in the RCSC *Specification* (a free download at **www.boltcouncil.org**). In all joint types the connection is required to be checked for bearing, which could occur at some time during the life of the structure.

Snug-Tightened Connections: In this connection type, it is required that the faying surfaces of the connection be brought into firm contact. While some pretension of the bolts is required to bring the surfaces into firm contact, there is no specific requirement for a level of pretension that must be induced into the bolts. Thus, while there is some level of clamping force in the connection as a result of the installation requirement to bring the surfaces into firm contact, this type of connection is assumed to provide the least level of safety against slip. The bolts in these connection types are always assumed to be in bearing against the base material, and thus the connection is defined as a bearing connection.

Pretensioned Connections: This type of connection generally is just like a snug-tightened joint, except it also requires that a specified pretension be applied to the bolts in the connection, once the firm contact of faying surfaces has been achieved. It is still a bearing-type joint, and the shear strength design parameters are identical to those of the snug-tightened joint.

Slip-Critical Connections: This connection type is just like a pretensioned connection with the addition of surface preparation requirements to provide the required level of slip resistance. To achieve this goal, there is a specified level of friction coefficient required for the faying surfaces in the connection. After the faying surfaces are brought into firm contact, the bolts are required to be installed to a specified level of pretension, which is the same as that required for a pretensioned connection. Thus, the bolt installation procedure for slip-critical connections is identical to that required for pretensioned connections, the difference being in the preparation requirements for the faying surfaces of the respective connection types.

Threads In or Out of the Shear Plane

One parameter that we have not discussed to this point is the N designation cited in your example. This designation is an indication of where the threads of a bolt are assumed to be located with respect to the shear plane(s) in a connection. The N assumes that the threads of the bolt will be located within the shear plane; an X assumes that the plies are detailed to exclude the threads from the shear plane. The shear strength is reduced if the plies are detailed such that the threads are located within the shear plane.

Kurt Gustafson S.E., P.E

Connecting a Cambered Truss

We have a 120-ft-long, 12-ft-high truss made up of wide-flange beams for the upper and lower chords. The truss will be fabricated with a 5-in. camber. At the worse loading condition, the truss will still be 1 in. above flat. How is the connection made from the member end to the column? Will the fabricator detail the member so the bolt holes in the cambered member are vertical and in line to the connection plate or angles holes, or do they offset the bolt holes? If the

truss is to rotate downward 4 in., should oversized holes be used in both the plate and member to allow for rotation?

The truss will be detailed taking into account the camber that you specify. The connection to the column will be detailed such that the holes follow the geometry of the cambered truss. The connection of the bottom chord to the column (assuming this truss member is essentially a zero force member at the column) is often made using long slotted holes. The bolts often are left loose until all of the dead load is applied and much of the camber has come out. The bolts are then tightened. What I have just described is a common approach, but you should ensure that this approach is suitable to the specifics of your case. Also, whatever decision is made, you need to make sure your intention is clearly conveyed to the fabricator and erector.

Larry S. Muir, P.E.

Welding Anchor Rods to Rebar

Can anchor rods be welded to the reinforcing bars to be embedded in the slab to be cast? I cannot find anything in the codes in reference to this subject. Can you help?

The AISC *Specification* does not address the subject of welding of anchor rods to reinforcing steel. However, if for some reason the responsible design professional intends to weld the anchor rods to rebar, it first would be advisable to make sure that a weldable grade of rebar and anchor rod is used. The weld details used probably would have to be qualified, but this is possible and not entirely uncommon today. You may need to reference a combination of the provisions in AWS D1.1, *Structural Welding Code—Steel* and AWS D1.4, *Structural Welding Code—Reinforcing Steel*.

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

Base Metal/Fastener Compatibility

Where can I find information pertaining to compatibility when using different types of base metal and fasteners in a connection?

Table 2-6 in the 13th edition AISC *Steel Construction Manual* provides information pertaining to *Metal Fastener Compatibility to Resist Corrosion*, which you may find helpful.

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

Beam "Sweep"

Where can I find information for the allowable amount of beam sweep in an erected building?

The term "sweep" implies curvature about the weak axis of beams. This generally is a tolerance that is addressed in mill requirements, rather than after erection, and is covered by tolerances given in ASTM A6/A6M. A summary of this information is included in the 13th edition AISC *Steel Construction Manual* on page 1-117.

The tolerance limitations for beams that are erected are given in the AISC *Code of Standard Practice for Steel Buildings and Bridges*. However, these limitations deal more with alignment tolerances between work points rather than actual "sweep" between working points.

David Wickersheimer, S.E.

Double-Concentrated Forces

Section J10 of the AISC *Specification* addresses double-concentrated forces. Could you please explain the meaning, or give an example of a double-concentrated force as it applies to this section?

Double-concentrated forces is defined in the AISC *Specification* as "two equal and opposite forces that form a couple on the same side of the loaded member." A moment connection of a beam-to-column flange, where one of the beam flanges is applying a tension force while the other flange is applying a compression force, would be one example.

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

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

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

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

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

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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 Where in the 2005 AISC *Specification* (**www.aisc.org/2005spec**) can a design engineer find the requirements for filler metal strength level?

2 Can the strength of a weld group that is composed of segments that are parallel and transverse to the direction of loading be determined by summing the individual strengths of the weld segments?

3 Where cyclic loading requires design for fatigue resistance, does the grade of steel affect the likelihood of fatigue cracking?

4 True/False: According to the 2005 AISC *Specification*, evaluation of fatigue resistance is not required if the stress range results in compressive loads only.

5 What type of cracking is addressed by stress category F in the fatigue provisions in Appendix 3 of the 2005 AISC *Specification*?

6 True/False: Longitudinal backing bars are permitted to remain in place in assemblages that will be subject to significant cyclic loading.

7 What is the primary cause of weld cracking?

8 How can the design engineer reduce shrinkage stresses in welded connections?

9 True/False: When structural steel is cooled rapidly, a microstructure that is susceptible to cracking can be formed.

10 True/False: The American Welding Society publishes a document that specifically addresses welding in Seismic Lateral Resisting Systems.

TURN TO PAGE 14 FOR ANSWERS

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steel quiz

ANSWERS

1 Requirements for welding consumable are given in 2005 AISC *Specification* sections A3.5, J2.6, and J2.7. Permissible filler metal strengths are shown in Table J2.5, based on matching filler metals shown in AWS D1.1 Table 3.1 (**www.aws.org**).

2 Yes, if the directional increase factor for the transversely loaded segments is ignored. Alternatively, Equation J2-9b can be used with the directional increase.

3 No. Fatigue resistance is dependent on the sensitivity of the details to fatigue, the stress range, and the number of loading cycles.

4 False. According to Appendix 3 of the 2005 AISC *Specification*, evaluation of fatigue resistance is required even when the entire stress range results in compressive stress only.

5 Stress category F addresses fatigue cracks that form within the weld rather than in the base metal.

6 True. According to Section 3.5 of Appendix 3 in the 2005 AISC *Specification*, backing bars that run longitudinal (parallel) to the direction of the load are permitted to remain in place. These backing bars must be continuous or joined with complete-joint-penetration butt joints, if spliced.

7 Shrinkage stresses associated with the hot expanded weld and base metal creates residual stresses that remain after the material cools. This can result in cracking when there is insufficient ductility to accomodate these deformations.

8 Section 5.5 of AISC Design Guide 21 (available at **www.aisc.org/epubs**) lists measures that can help reduce shrinkage stresses in welded connections. Among them is reducing the volume of weld metal used, and the design engineer plays a key role in making sure that this can be done by specifying appropriate weld types, sizes, and loads.

9 True. Rapid cooling around welds can increase the possibility of the development of a sensitive heat-affected zone (HAZ), particularly with a chemistry that is high in carbon and some other alloying elements. HAZ cracking is one of three weld crack types addressed in Chapter 5 of AISC Design Guide 21. Methods for reducing cooling rates, when necessary, also are presented in the same chapter.

10 True. AWS D1.1 addresses general welding requirements, and AWS D1.8 specifically addresses the special requirements when specifying welding in seismic frames that are designed for ductility. The requirements in AWS D1.8 are referenced in the AISC *Seismic Provisions*.

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

Steel Solutions Center

AWARDS

Donald W. White Wins 2009 T.R. Higgins Award

Donald W. White, Ph.D., a professor at the School of Civil and Environmental Engineering at the Georgia Institute of Technology, is the recipient of AISC's 2009 T.R. Higgins Lectureship Award. White is being honored for his papers on stability analysis and design and the flexural provisions of the 2005 AISC *Specification for Steel Buildings* published in the ASCE *Journal of Structural Engineering* and AISC's *Engineering Journal*.

The T.R. Higgins Lectureship Award is presented annually and recognizes an outstanding lecturer and author whose technical paper(s) are considered an outstanding contribution to the engineering literature on fabricated structural steel. The award, which includes a $10,000 prize, will be presented at the 2009 NASCC: The Steel Conference in Phoenix in April.

"Don's work forms the foundation of many of the new provisions in the 2005 AISC *Specification*," commented Charles J. Carter, S.E., P.E., AISC vice president and chief structural engineer. "He's a particularly productive researcher, author, and lecturer. We're fortunate to have him as a contributor to the advancement of fabricated structural steel and as a member of the Committee on Specifications. He's an outstanding selection as the T. R. Higgins Lectureship Award winner."

White has been a member of the Georgia Tech faculty since 1997. Prior to joining Georgia Tech, he served on the faculty at the Purdue University School of Civil Engineering from 1987 to 1996. He received his doctorate in Structural Engineering from Cornell University in 1988 and attended North Carolina State University for graduate school.

He's a member of the AISC Committee on Specifications as well as various other AISC technical committees. He's also a member of the American Iron and Steel Institute (AISI) Bridge Research Task Force and the Structural Stability Research Council (SSRC). In addition to his contributions to the 2005 AISC *Specification for Structural Steel Buildings*, he served as a major contributor to the 2004 update of the American Association of State and Highway Transportation Officials (AASHTO) Load and Resistance Factor Design Specification provisions for curved and straight steel bridge design.

White received the 2005 Special Achievement Award from AISC for his research on design criteria for steel and composite steel-concrete members in bridge and building construction, and the 2006 Shortridge Hardestey Award from ASCE for his research on advanced frame stability concepts and practical design formulations. He's an associate editor of ASCE's *Journal of Structural Engineering* and serves on the Editorial Board of the *Journal of Constructional Steel Research*.

To view a few recent samples of White's writing, visit **www.aisc.org/donaldwhite**.

IN MEMORY

A Half-Century Dedicated to the Steel Industry

It is with deepest sadness that I report that our good friend and colleague Bill Liddy passed away on October 29. Bill was a proud representative of the steel industry for more than half a century. Working for American Bridge and U.S. Steel in his early professional years, after graduating from college in the early 1950s, he remained as an ambassador for steel construction until his passing. Bill culminated his long career in the steel industry working for AISC. He formally retired in the spring of 2007 and was awarded a Lifetime Achievement Award at the 2007 NASCC at the time of his retirement. He was also recently honored for his long service to the steel industry by the AISC Board of Directors (see related story on pg. 20).

Bill's constant smile and friendly disposition made him a friend of multitudes associated with the steel, design, and construction industries, as well as those he met in his personal life. Those of us that have known and worked with Bill for many of those 50-plus years, as well as those that had the opportunity to work with him for only a few months, shared the feeling that Bill was a true friend.

– Kurt Gustafson

Former AISC Board Member Dies

Former AISC Board of Directors member (1974-1976) David Kingsnorth Patterson, Sr. passed away at his home in Binghamton, N.Y. on October 26 from complications of Parkinson's Disease. He moved to Binghamton with his family in 1955 to join Binghamton Steel and Fabricating, Inc., where he eventually became president. He was 85.

NSBA Reorganizes, Strengthens Mission to Grow Steel Bridge Market

The National Steel Bridge Alliance, a division of AISC, today announced a reorganization that balances its mission between marketing activities to promote the use of steel bridges, legislative activities, and technical support for bridge designers, owners, and fabricators.

"During the past six years, the NSBA has spent an increasing amount of its resources on legislative action," explained Jack Klimp, chairman of the NSBA executive committee and general manager of Cianbro Fabrication and Coating Corporation in Pittsfield, Maine. "While we still need to continue our legislative activities, we also need to achieve a greater balance between our legislative, technical, and marketing work. In addition, we can achieve more cost-effective results by working more closely with other trade

Continued on page 18.

news

Continued from page 17.

groups with legislative initiatives compatible with ours, as well as strategic use of Washington-based consultants."

Under the reorganization, the NSBA will continue its regional activities and will also integrate its technical activities into the popular AISC Steel Solutions Center. The AISC Steel Solutions Center is a one-stop source for technical information and conceptual solutions for the steel construction industry. The team of structural engineers and consultants answer more than 200 technical inquiries each week at no charge to designers, owners, and fabricators. In addition, they provide conceptual studies comparing steel solutions to designs in alternate materials. The conceptual solutions include framing system comparisons, as well as cost and scheduling information. A direct link to the AISC Steel Solutions Center for bridge-related questions and studies will soon be available at **www.steelbridges.org/solutions**. (For more information about the AISC Steel Solutions Center, please visit **www.aisc.org/solutions** or call 866.ASK.AISC.)

The NSBA is a unified industry organization of businesses and agencies interested in the development, promotion, and construction of cost-effective steel bridges. Activities include development of the *Steel Bridge Design Handbook*, hosting the World Steel Bridge Symposium, and fostering the AASTO/NSBA Steel Bridge Collaboration, a group dedicated to achieving quality and value in steel bridges by standardization of design, fabrication, and erection and the sharing of resources.

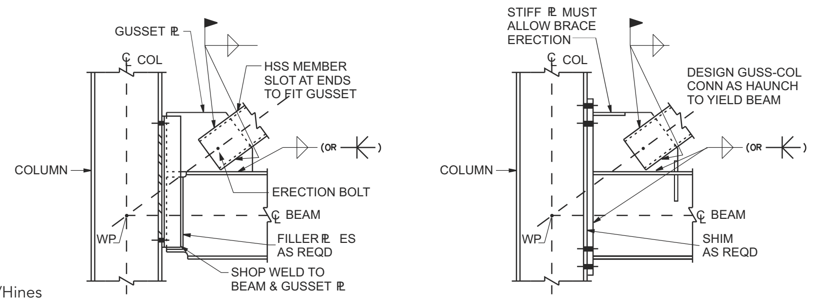
RESEARCH

Gusset Plate Behavior

As $R = 3$ lateral systems gain popularity in moderate seismic regions, the need for rigorous assessment of these systems' collapse performance grows stronger. In particular, $R = 3$ concentric braced frames depend on the inherent reserve capacity of their buildings' gravity framing systems to maintain stability during a maximum considered earthquake event. The role that gusset plate connections play in such reserve systems after brace fracture, if better understood, could be leveraged to ensure adequate reserve capacity with minimal cost increases. The below figure compares a typical gusset plate connection (a) to a possible modified end-plate connection designed for higher ductility levels (b). Design measures for gusset plates that act as ductile, haunched beam-column connections after brace fracture are the subject of current collaborative research between Tufts University and the University of Illinois, Urbana-Champaign (UIUC).

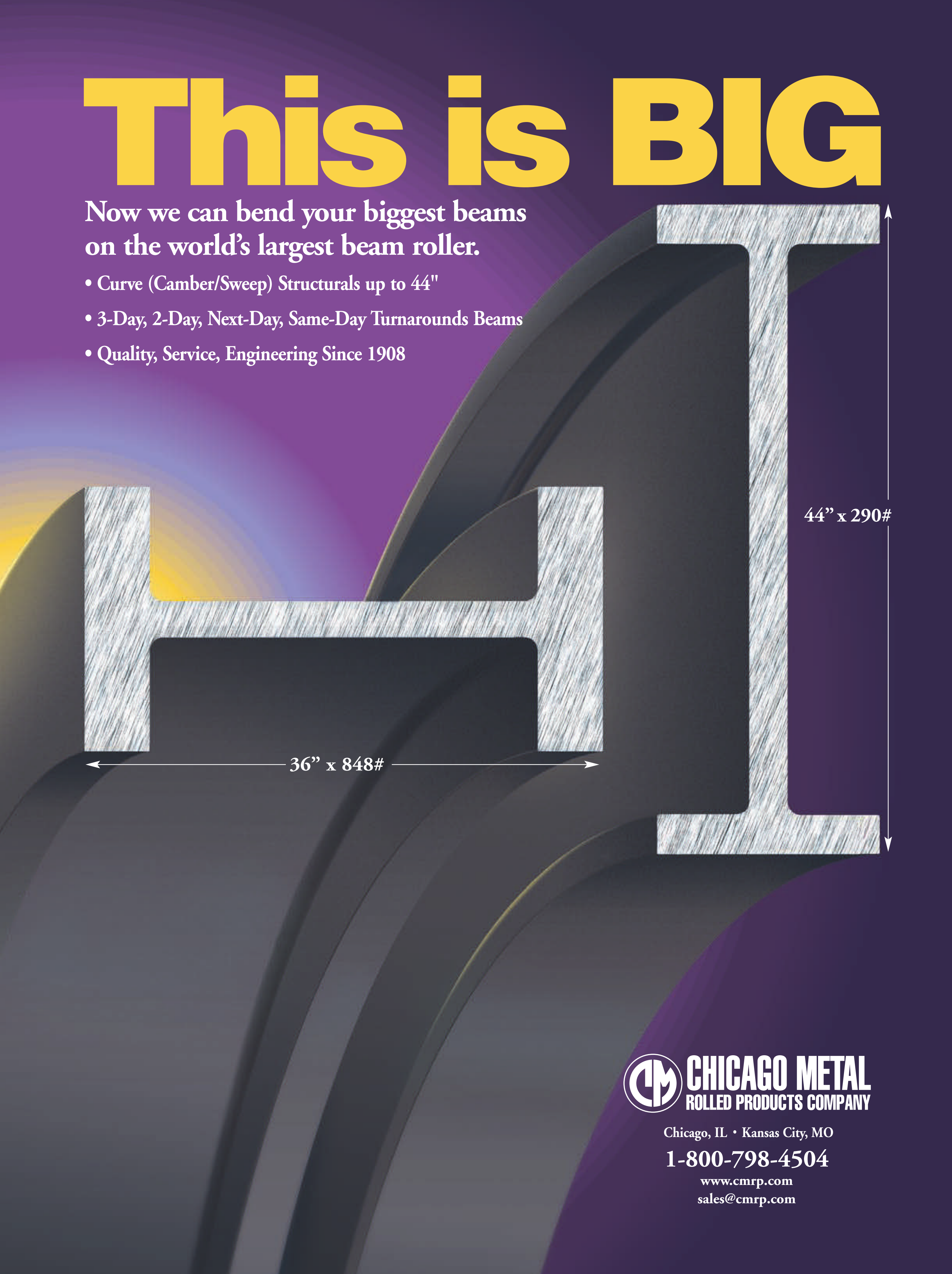
Based on innovative lateral system design concepts developed by LeMessurier consultants for a 1,200-bed, 600,000-sq.-ft dormitory at Northeastern University, a suite of full-scale tests was developed to evaluate gusset plate behavior at large drift levels. The test units were designed by UIUC researchers in collaboration with researchers at Tufts University and engineers at LeMessurier consultants. The test units were donated by Novel Iron Works, who fabricated them at their main shop in Greenland, N.H. Complete inspection services were provided by Briggs Engineering of Rockland, Mass. AISC sponsored transportation costs to UIUC as well as part of the test set-up costs.

The $R = 3$ concept was advanced by AISC in its 1997 *Seismic Provisions* as a minimum seismic design standard for those structures in Design Categories A, B, and C not required to adhere to the *Provisions*. The approach requires no specific seismic detailing and relies on the inherent reserve capacity of steel structures to ensure stability during major earthquakes in moderate seismic regions.



Cheever/Hines

(a) TYPICAL (b) MODIFIED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PROJECTS

L.A.'s First Steel-Plate Shear Wall High-Rise

AISC recently launched a new web site and portal, **www.aisc.org/LA-LIVE**, that features the latest information on the L.A. Live Hotel and Residences project, the first steel-plate shear wall high-rise building in Los Angeles.

With eye-catching images of the project site and a live webcam of the structure, the L.A. Live web site showcases the hotel and residences building as the centerpiece of the L.A. Live development, a 4 million-sq.-ft, $2.5 billion downtown Los Angeles sports, residential, and entertainment district development adjacent to the Staples Center and Los Angeles Convention Center. The 56-story structure will house 1,001 hotel rooms and 224 luxury condominiums. The total development cost is pegged at about $1.0 billion for 2 million sq. ft of space.

L.A. Live Hotel and Residences broke ground on November 2007, and structural steel erection is expected to be completed by the end of 2008, which is about two months ahead of schedule; the expected opening time frame is early 2010.

The idea for the L.A. Live Hotel and Residences structure was born in March 2006, when Nabih Youssef Associates reviewed the conceptual design and suggested replacing the heavy 30-in. concrete shear walls with light ¼-in. to ⅜-in. steel-plate shear walls to free valuable real estate space; eliminate 35% of the weight of the structure; and reduce seismic design forces and foundation sizes. As a result, the proposal compressed the construction schedule and budget while allowing for more simplified and efficient construction. Nabih Youssef Associates was then hired by the development group, AEG, to convert the 56-story concrete shear wall design to a steel-plate shear wall system solution.

For more information on the steel-plate shear wall system, please contact the AISC Steel Solutions Center at 866.ASK.AISC or **solutions@aisc.org**.

AEG

news

AWARDS

AISC Recognizes Outstanding Achievements at Annual Meeting

AISC's Lifetime Achievement and Special Achievement Awards were presented at the 2008 AISC Annual Meeting in Colorado Springs, Colo., held September 11-13.

AISC's Lifetime Achievement Award honors living individuals who have made a difference in the structural steel industry's success. This year's award was presented to Bill Liddy and Bert Cooper.

Bill Liddy spent nearly 60 years in the steel industry, first promoting structural steel for the mills, then as a regional engineer with AISC, and finally as an advisor in the AISC Steel Solutions Center. He worked closely with the fabrication industry, especially in the Midwest, and was very respected by both the fabrication and design community. He also acted as a mentor for younger staff at AISC. Sadly, Bill passed away in October (see pg. 17).

Bert Cooper is a long-time contributor to the structural steel industry as both an AISC board member and as the owner of a leading fabrication firm, W&W/AFCO Steel. He's contributed substantial time and financial resources to support steel industry research activities.

AISC's Special Achievement Award gives special recognition to individuals who demonstrated notable singular or multiple achievements in structural steel design, construction, research or education. This award honors living individuals who have made a positive and substantial impact on the structural steel design and construction industry. The 2008 Special Achievement Award was presented to William W. Brown and Joseph J. Hunt.

Brown is the president of Ben Hur Construction Co. and Hunt is the general president of the International Association of Bridge, Structural, Ornamental and Reinforcing Iron Workers. They were awarded this year's Special Achievement Award for their work in developing and nurturing I.M.P.A.C.T., which is a labor/management partnership designed to bring together local unions and their signatory contractors to address mutual problems and create solutions to those problems.

For more information on AISC's Individual Awards and past recipients, please visit **www.aisc.org**.

BOLTS

Know Your Bolts

In the past few months, photographs of bolts with what appear to be raw material seams have been circulating on the Internet. These photographs have generated concerns in the industry about what is done or should be done to prevent similar bolts from being used on structural steel construction projects. This has prompted concerns from owners and engineers regarding the quality levels of fasteners manufactured to ASTM standards, particularly overseas.

Over the years, and primarily due to the requirements of the Fastener Quality Act, a detailed infrastructure has been developed to assure quality in fasteners. Manufacturers must produce bolts under an acceptable quality management system; ISO 9000 would be an example of such a system. Acceptable quality management systems require many in-process quality control inspections. Manufacturers also conduct quality assurance inspections on a randomly selected sample of the fasteners they produce. These inspections are defined in the individual fastener specifications and include dimensions, hardness, thread fit, wedge tensile, and surface defect inspections. The surface defects shown in the pictures that have brought this issue to the attention of the industry are defined in ASTM F788/F788M. This standard indicates that seams such as those shown in the pictures do have an acceptable limit, but without the supporting results of a comprehensive evaluation, it is impossible to say whether the bolts in the pictures complied with the specification.

In the U.S. construction industry, the fasteners may also undergo one of two other tests. In the building industry, the installer conducts Preinstallation Verification Tests on bolts in connections requiring full tensioning. Bolts from each lot are tightened in the tension-measuring device to the minimum required pretension to ensure the bolt assembly and installation method can develop the required clamp load. Additionally, all galvanized structural bolts, as well as all structural bolts used on bridges, are Rotational Capacity Tested. The Preinstallation Verification Test is described in the Research Council on Structural Connections' *Specification for Structural Joints Using A325 or A490 Bolts* (2004). There are two versions of the Rotational Capacity Test. One is described in the ASTM A325/A325M standard. The other is in the AASHTO specification.

Many years ago there were concerns about fastener quality. In response, quality systems were developed to provide assurance that fasteners were manufactured in compliance with standards. The recent incidents are a good reminder that owners and suppliers should be acquiring fasteners from manufacturers that have implemented acceptable quality systems. This can be done by knowing your supplier and asking about the quality systems their manufacturers use. Suppliers should be able to tell you reasonably quickly whether the manufacturer is certified to ISO or some other recognized quality system. This should be a simple step, and when performed, should be sufficient to verify the quality of the bolts. *- Chad Larson, Vice President, LeJeune Bolt Company*

letters

The Right People for the Job

I've been subscribing to MSC for a few years now and I've always enjoyed Scott Melnick's editorials.

His last one, relaying Howard Putnam's comments [on hiring practices], really struck a chord with me, as much of my time over the last three years has been trying to hire the right people. These are all excellent points.

I just wanted to let you know that your work is relevant and appreciated. Great job, and please keep up the good work!

Sam DeFranco
Engineering Authority
BP America, Inc.

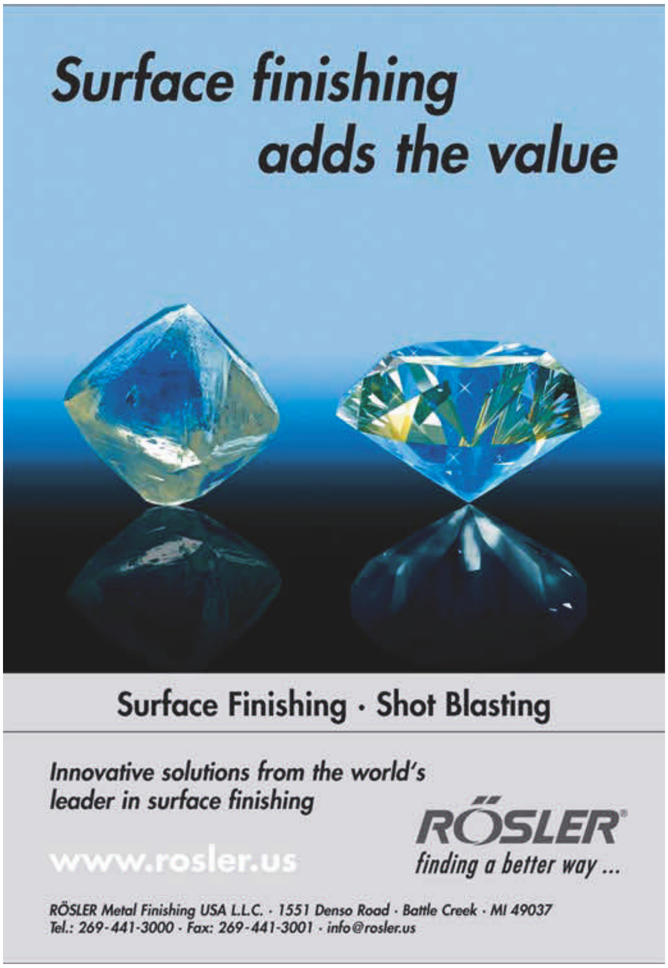
People and Firms

• Structural Engineering firm **Thornton Tomasetti** last month opened two new offices in the United Arab Emirates of Dubai and Abu Dhabi to accommodate the firm's expanding business development and extensive involvement in more than a dozen current, high-profile projects in the Middle East. Kyle Krall, a principal with Thornton Tomasetti, will relocate to the region in January to operate both offices.

Thornton Tomasetti also announced the following promotions at the firm's New York office: Erleen Hatfield, P.E., LEED AP, Ling-En Hsiao, Ph.D., Gary Mancini, P.E., LEED AP, and Michael Squarzini, P.E., have been promoted to senior vice president/principal. Hi Sun Choi, P.E., and Jeffrey Schreier have been promoted to principals. Jan Kalas, AIA, was promoted to senior vice president. And Eli Gottlieb, P.E., and Libero Petrella, P.E., were both promoted to vice president.

• Members of the **Structural Engineers Association of California** elected Reinhard Ludke, S.E., to serve as president of the association and lead the nine-member Board of Directors. Ludke has been practicing structural engineering and involved in construction since 1969. He is the principal structural engineer with Creegan + D'Angelo Infrastructure Engineers' transportation, water, buildings, public works, and earthquake-performance projects.

• **Design Data** announced that Jim Dager recently retired as company president. Dager will maintain ownership of Design Data and will continue to be involved in monthly management meetings. Damon Scaggs, who currently serves as Design Data's executive vice president, will take over as company president and chief executive officer. Barry Butler, senior development manager, will take on the role of executive vice president.



• Management consulting and research firm **ZweigWhite** has identified the 200 fastest-growing architecture, engineering, and environmental consulting firms for its annual ranking, *The Zweig Letter* Hot Firm List. This annual list features the design and environmental firms that have outperformed the economy and competitors to become industry leaders. The top 10 firms on the list are:

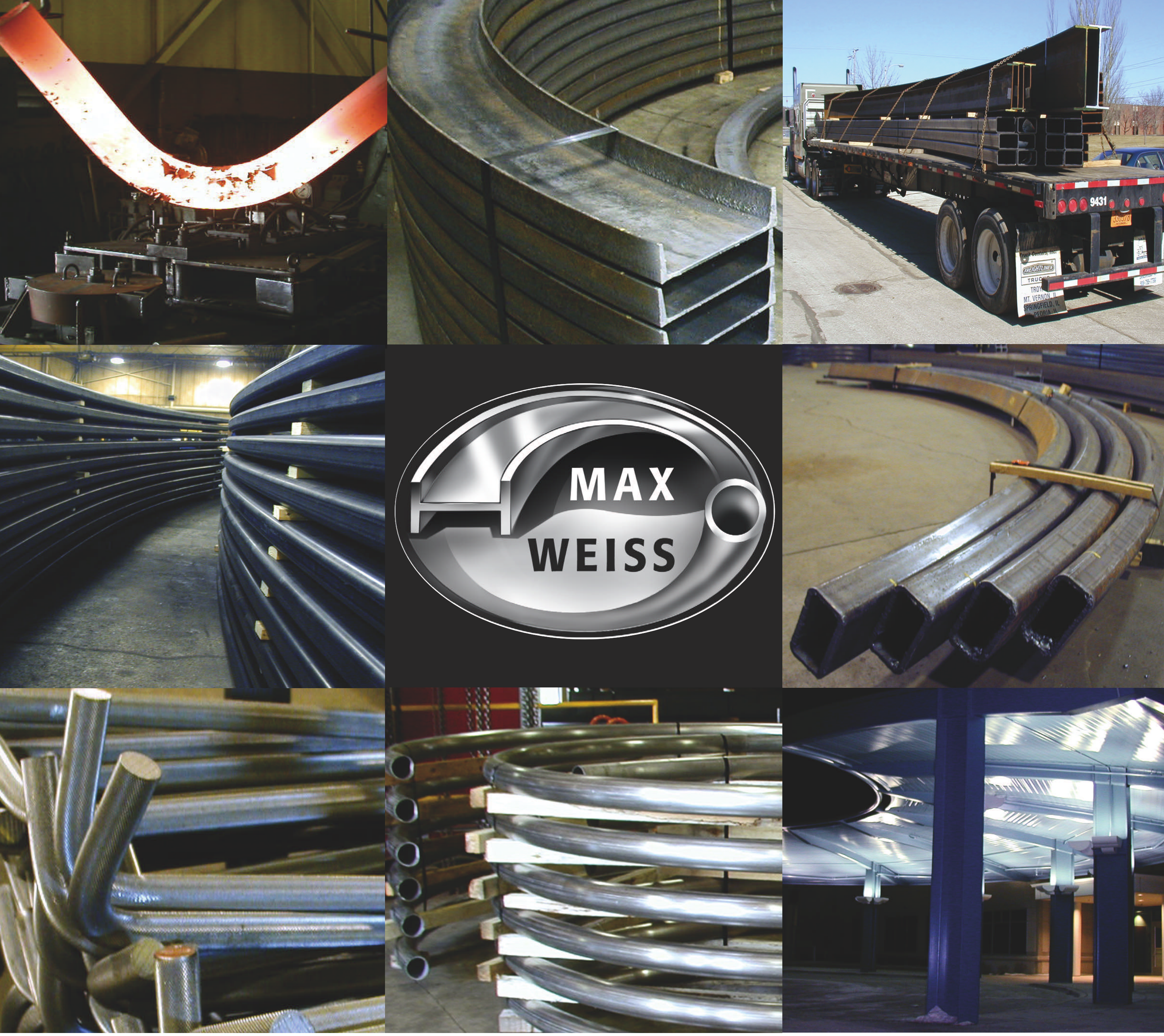
1. WSP Group (USA), New York, N.Y.
2. Hill International, Inc., Marlton, N.J.
3. ARCADIS U.S., Inc., Highlands Ranch, Colo.
4. NELSON, Philadelphia, Pa.
5. Natural Resource Group, LLC, Minneapolis, Minn.
6. TolTest, Inc., Maumee, Ohio
7. Stantec Consulting Inc., Irvine, Calif.
8. X-nth, Maitland, Fla.
9. ITAC Engineers and Constructors, Chester, Va.
10. Trow Global Inc., Brampton, Ontario, Canada

A complete list of the 200 fastest-growing architecture, engineering, and environmental consulting firms was published in the November 3, 2008 issue of *The Zweig Letter*. Visit **www.zweigwhite.com**.

• **Hobart Brothers Company** recently announced the expansion of its manufacturing operations into a new 65,000-sq.-ft facility in Troy, Ohio. The new facility will be used to increase production capacity for the company's welding consumables. The company anticipates that an additional 40 people will be added to its workforce once the new plant is at capacity. The new building is scheduled to be completed by this coming spring.

• **Tennessee Galvanizing** was recently recognized by the Tennessee Chamber of Commerce and Industry for outstanding environmental accomplishments at the annual Tennessee Chamber Environmental Conference. The company was awarded certificates for Hazardous Waste Management and Environmental Excellence.


NUCOR-YAMATO
STEEL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Steel Bridge NEWS
NATIONAL STEEL BRIDGE ALLIANCE
DECEMBER 2008

Taking Flight

BY TIM RICHEY, AIA

Lara Swimmer Photography

An aviation museum's new pedestrian bridge pays homage to flight via a complex yet elegant design.

SEATTLE'S TIES TO THE AVIATION INDUSTRY ARE WELL KNOWN; THE CITY IS THE BIRTHPLACE OF BOEING AND STILL THE CENTER OF THE COMPANY'S ASSEMBLY OPERATIONS. So it's not surprising that Seattle's Museum of Flight, located adjacent to Boeing Field in Tukwila, Wash., is one of the largest air and space museums in the world. Besides its jumbo-sized exhibits, the museum will soon boast another eye-catching attraction: a striking 340-ft steel pedestrian bridge linking the current museum to a remote exhibit space and future development site across a busy traffic arterial.

Vapor Trail

A conventional, utilitarian public works bridge design would have been possible but inadequate to convey the spirit of the museum and the area's aviation history. Instead, the bridge's design is inspired by the phenomenon of a contrail, a stream of crystallized vapor created in a plane's wake. The metaphor is carried out in the bridge's unusual tube-shaped truss design, made of crossing circular steel pipes surrounding an inner glass enclosure. The bridge interior includes exhibit panels describing aviation history in the area, as well as colored LED lights along its path and a sound installation by local artist Paul Rucker with audio sampling from aerospace history and nature.

Precast structural shapes and cast-in-place concrete solutions were studied, but the narrow aperture through which the structure needs to pass—above the roadway clearance and below the power lines—limited the amount of structural depth that could be accommodated below the bridge deck. Using a steel truss allowed the structural depth to surround the partially enclosed interior space and also maintained consistency with the existing museum's architecture.

Not Your Ordinary Truss

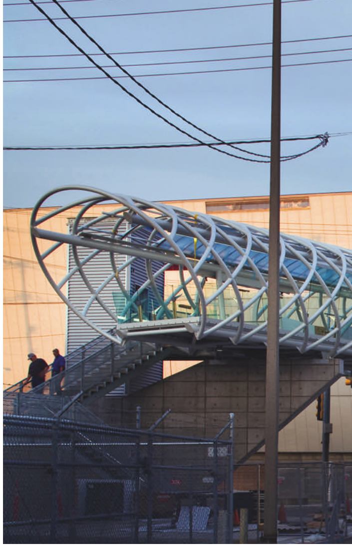
The unique structure of the bridge evolved from the design collaboration between the architect, structural engineer, and steel fabricator. By bringing these parties together as early as possible in the design process, the team was able to push the design beyond a conventional solution toward something extraordinary.

Through a series of creative charrettes, architect SRG Partnership and Magnusson Klemencic Associates, the engineer, devised a unique structural design that didn't rely on conventional truss webs, but instead distributed the vertical shear in the bridge structure through a matrix of curving steel pipes. The cross-section of the bridge is widest at the center of the span, tapering at its ends as if the contrail was dissolving into the sky. This exciting and dynamic form, however, had the potential to be overly complex and unachievable within the project budget.

Jesse Engineering was chosen as the fabricator, based on their experience with another pedestrian bridge—at a shopping center in Bellevue, Wash.—that was admired for its craftsmanship. But it was expertise in 3D computer modeling from MKE Detailing and Jesse Engineering's capacity to cut complex pipe curves, or "fish mouth" shapes, that allowed the design form to be broken down

SRG Partnership

Lara Swimmer Photography



into simple components. Jesse Engineering worked with the design team to detail the truss structure, using mostly standard steel shapes with constant-radius pipe bends and repetitive connection types, to achieve the complex bridge form economically. Casey Carver of Jesse Engineering explained, "The challenge of a project like this for a fabricator is starting with basic concepts and growing it into something to work with. Normally, fabricators only get involved after design has already been established." According to MKA principal Jay Taylor, "Our intention in this structural design was to take simple shapes and use them in unique ways."

Tapered and Tubular

The result is a complex yet elegant 340-ft-long bridge made of crossing circular steel pipes, spanning 140 ft across a major road. The main tube-shaped truss, measuring 200 ft in length, is composed of a series of crossing 5-in.-diameter pipe hoops tilted at 45°. The radiused bend of the hoops varies from 22 ft at the center of the span to 19 ft at the tapered ends. Although the curvature of the hoops is a true radius, when tipped at an angle an elliptical interior space is created. In total, the project uses approximately 10,000 linear ft of steel pipe weighing a total of 190 tons.

Within the truss, the semi-enclosed environment protects pedestrians from Seattle's infamous gray and rainy weather. Overhead, a translucent polycarbonate roof suspended beneath the overhead steel pipes filters direct sunlight while glass panels on the south blocks the winter winds.

The bridge deck was originally specified as cast concrete over metal deck, but the weight

3

4

Photos 3 & 4: Lara Swimmer Photography

1 A matrix of curving steel pipes replace the conventional truss webs.

2 A large waterborne crane lifts the tube truss onto a barge in Tacoma's industrial waterways.

3 Inspired by the phenomenon of a contrail, the 340-ft-long pedestrian bridge stretches across the street to connect museum visitors to a remote commercial aircraft display.

4 Blue LEDs give the structure a nighttime glow.

of the material remained a problem over the longest bridge span. Once an extruded aluminum deck plank was identified, the significantly lower dead load allowed the steel to be reduced in weight, resulting in a savings to the project and a more elegant design.

Minimal Closure

One construction challenge was navigating through a dense network of below-grade infrastructure and overhead power lines that remain critical to Boeing's local research and manufacturing facilities. However, general contractor Sellen Construction helped accelerate the project schedule by compressing the on-site utilities and foundation work with the off-site truss fabrication at Jesse Engineering. Although unusual, several subcontractors, including electrical, lighting, and glazing, worked together off-site at Jesse's shop, enabling a greater degree of prefabrication and less on-site construction over the busy roadway.

Steel fabrication began in the spring of 2008, and by early July the two trusses were welded, painted, and ready to be transported to the job site, along with the aluminum floor system, the electrical conduit, lighting, and some glazing mounts already in place on the truss. Jesse Engineering's location along the industrial waterways of Tacoma allowed the truss to be shipped by barge 40 miles north through Puget Sound and down the Duwamish River in Seattle, and unloaded just a few hundred yards from the final erection point. To install the bridge over the traffic arterial, the construction team negotiated with the City of Tukwila to divert traffic for a full day, then raced against the clock to erect the two trusses, weld each in place, and remove the crane equipment within a 24-hour window. The planning paid off, as the roadway was clear and open for traffic by 6 a.m. the following morning. The bridge, already a winner of the AIA Washington Council's 2008 Civic Design Awards program as an unbuilt project, opened in October. MSC

Tim Richey is a senior associate with SRG Partnership.

Architect
SRG Partnership, Seattle

Structural Engineer
Magnusson Klemencic Assoc., Seattle

Steel Fabricator
Jesse Engineering Co., Tacoma, Wash. (AISC/NSBA Member)

Steel Detailer
MKE Detailing Service, Inc., Seattle (AISC/NISD Member)

General Contractor
Sellen Construction, Seattle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the Arch Reinvented

BY SUSAN MATZAT, P.E.

Craig Shaw, Stratus Imaging

As Ithaca, N.Y. has grown over the years, one of its most significant spans needed to keep up.

"ITHACA IS GORGES." Ezra Cornell and his associate Andrew Dickson White capitalized on this now-trademarked phrase to bring students to their new university in 1868. Recognizing the significance of the setting and reputation of Cornell University, the City of Ithaca and the New York State Department of Transportation (NYSDOT) implemented a first-of-its-kind design to retain a bit of history in combination with a bit of invention for the rehabilitation of Cornell's primary link over the Fall Creek Gorge.

The 181-ft-long crossing serves more than 34,000 students, faculty, and staff, but severe congestion was causing pedestrians to walk in the travel lanes and cars and buses to be held up at the approach intersection. To solve the problem, induction-bent hollow structural section (HSS) arches were tied to two existing arches to widen the Thurston Avenue "gateway" between the residential and academic campuses of the university.

Saving the Arches

The original bridge was constructed more than a century ago. When the Ithaca Street Railway Co. decided to extend its trolley service from downtown Ithaca to the Cornell University campus in 1897, a steel arch bridge was built over the Fall Creek Gorge.

After the trolley service stopped, the city took ownership of the bridge, converting it for vehicular and pedestrian use. They replaced the bridge in 1960 with a two-rib steel box-deck arch structure. By 2001, pedestrian and bicycle counts revealed that the university and the surrounding area had significantly outgrown the bridge, which was operating at maximum capacity and at an extremely low level of service. Improved function, safety, and movement through this critical corridor were necessary, and the bridge could not accommodate these needs without major rehabilitation or replacement.

Sam Juriga, LaBella Associates.

Existing and new steel framing elements were painted light green and dark green, respectively, to provide differentiation between these elements.

Courtesy of the City of Ithaca, N.Y.

The original bridge in 1897.

During preliminary design of the new bridge, the existing 1960 span was found to be included in NYSDOT's Inventory of Historic Bridges due to the rare type of the existing arches and specific "character-defining" features, including curved floor beam ends and vertical picket bridge railings. This designation extensively increased the level of alternative analysis required to reach a solution that would accommodate the needs of multiple interests.

The solution was to widen the bridge 12 ft by adding new induction-bent HSS arches at each fascia to provide for 10-ft-wide sidewalks and 5-ft-wide bicycle lanes across the bridge and its approaches. The new arches were elevated so that the existing arches remained visible, and existing and new steel framing elements were painted light green and dark green, respectively, to provide differentiation between these elements. The paint system included shop- and field-applied zinc-rich epoxy primer and urethane intermediate and finish coats.

The final parabolic curvature of the new arches was designed to meet constraints posed by a number of factors. The location of existing floor beams for column and hanger connections helped determine the locations where the arches rise above the deck. The height of the crown was determined by the owner's desire to allow views to the gorge—and to discourage climbing.

LaBella developed computer-generated design models using MIDAS structural modeling software to produce a complete and detailed 3D model of the existing and widened structures. The basis for the structural analysis was that the proposed design was in essence two independent structures that only interact to support the bridge floor beams. The existing bridge remained as designed and originally constructed, and was only altered by lengthening the floor beams.

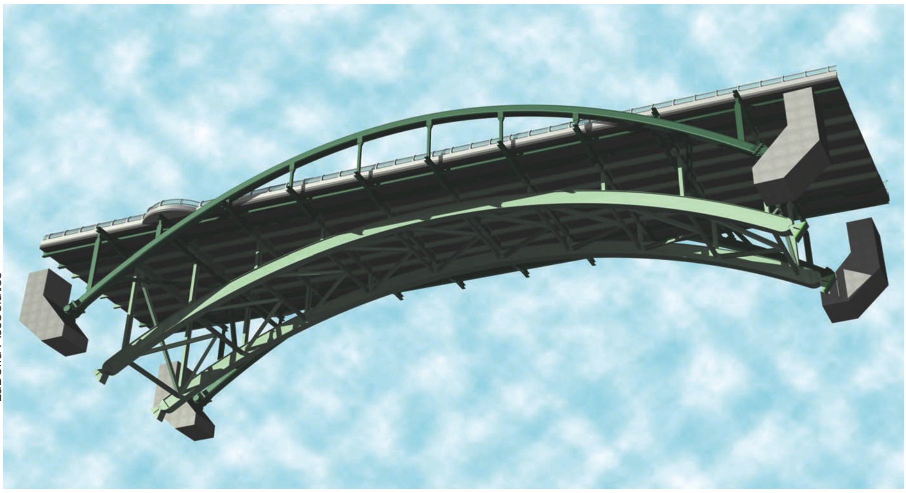
Structural Independence

Our analysis showed that the existing structure was stable and structurally adequate for current loads but could not support any additional vertical loading. The new arch structure was designed as an independent structure capable of supporting all vertical and lateral loads independent of the existing bridge. Both structures were analyzed independently for self-weight, with utility loading applied only to the existing structure.

The columns and hangers were then connected between the models and analyzed by applying the deck dead load, superimposed dead loads, and live loads. Analysis results were then combined for each member and connection to obtain the total additive effects and ensure that the structures were compatible. Several additional models were produced to check the design for all stages of construction and the additive effects of loading based on the sequence of construction.

Bearings for the new arches were designed and detailed with stainless steel pins to allow for rotation. Floor beam ends were detailed with slotted openings in the top flange to facilitate bolted column and hanger connections to the floor beam web. Bolted connections (versus welded connections) reduced the duration of field erection, allowed movement to occur while the deck and other items of work were being finalized, and provided better long-term fatigue resistance.

The 32-in. by 30-in. by 1-in.-thick tubular shape we were considering was larger than any standard tube section produced in the U.S. and would have to be custom fabricated. The tubes also had to be bent into a parabolic curve and had to incorporate field-welded splices to maintain a continuously smooth appearance for the entire length of the arch rib. Our search for a fabricator with the capability to produce the arches led us to BendTec, Inc.,



LaBella Associates

Induction-bent HSS arches were tied to two existing arches to widen the Thurston Avenue "gateway" between the residential and academic campuses of Cornell University, in order to relieve congestion caused by major increases in foot and vehicle traffic.

whose extensive welding experience and induction bending capabilities were vital to the advancement of the HSS arch design.

Creating the New Arches

The process of creating the tubes began by cold-bending two 1-in.-thick flat plates into two "U" shapes with 6½ in. outside-radius corners (5 in. inside). The arches are very visible to people crossing the bridge, which made it necessary for the dimensional tolerances of all of the U sections to be held very close. To maintain the shape and consistency of the sections, BendTec made special dies for press-breaking the outside radius of the tube corners in one step.

The fabricator then machined the edges and welded the two U shapes together with complete joint penetration seam welds to create 20-ft tube sections. The sections were then fed through BendTec's induction bending machine.

The process involves the pieces being fed into a rolling guide assembly, which drives the piece through an induction coil. The electric-powered coil heats a 1-in. cross section of tube to 1,850 °F as it is pushed through at 1.5 in. per minute. The yield strength of the material (50,000 psi in this case) is reduced to less than 10% of the yield strength of the cold material as it is heated, to facilitate forming the curvature.

A clamp assembly attached to a swinging arm at the end of the induction coil works with the guide drive and the "plastic hinge" in the heated tube section to induce a bending moment and form the intended curvature. The bending occurs as the tube passes through the heating zone while the material strength is reduced. The material is quenched upon exiting the induction coil by spraying 60 gallons of water per minute on the tube.

Each 20-ft section is compressed by 2% of its length as it passes through the bending machine. This required careful calculation of initial lengths to achieve accurate final section lengths. During bending, the welds shrink differently than the base material. To provide a smooth, uniform outside profile, BendTec machined the tubes and the outside surface of the seam welds.

They also developed detailed shop drawings for and fabricated all of the new steel members, including two 181-ft by 38-ft rise tubular arches, four arch bearings, 20 15-ft curved-flange floor beam extensions, eight columns, 12 hangers, 44 stringer beams, and all of the new bracing struts.

To ensure that all pieces would fit properly during erection and that all internal stiffeners and connections plates were properly located, BendTec welded the 20-ft bent sections together to create two end sections and one crown section per arch, and assembled each full-length arch, including the columns, hangers, and bracing, in its shop.

The internal stiffeners and connection plates for each of the support points were not welded in until after the sections were disassembled and the welds between the 20-ft sections were cut back out. After the internal welds were tested, the 20-ft sections were welded back together to create the final end and crown sections for each of the two new arches.

HSS arch ribs curved by the induction bending process had not been designed or constructed by NYSDOT before. To ensure that details, initial and final material properties, and fatigue resistance were achieved as expected, they imposed stringent certification and testing requirements. Shop and field testing and inspection of the fabrication and bending process, welding procedures, and erection procedures were required.

Plate bends were magnetic particle tested and all welds received radiographic and ultrasonic testing in accordance with the *New York State Steel Construction Manual*. The induction bending process followed TPA-IBS-98 *Recommended Standards for Induction Bending of Pipe and Tube* by the Tube and Pipe Association, International. Surface hardness, tensile strength, and Charpy V-notch tests were required for each arch extrados, intrados, each side, and all rib corners.

Longitudinal and circumferential butt welds between individual tube sections met AWS D1.5 requirements after induction bending and post-bend heat treatment. A test bend was done to ensure the material properties met the required specifications. The impact property achieved for the tube sections was 25 ft/lb at 40 °F.

Bringing New and Old Together

As fabrication was progressing, demolition of the existing sidewalks and construction of the new arch footings were taking place. The bottom shoe of each bearing was installed on the new footings, and the curved ends of the existing floor beams were cut. New bracing struts were fed through the existing structure and held in place with cables attached to the existing steel until they could be bolted to the new arches.

Once delivered, each arch was erected by setting the end pieces first, followed by the center piece with one crane on the bridge and one additional crane at each approach. The splice ends were fabricated with a backing tube that allowed the crown section to be dropped in without springing the two end sections.

The cranes held the arch sections in place for approximately 16 hours until temporary stand-offs and new bracing struts were connected to each arch and complete joint penetration butt welds at the splices were finished and tested. The splice welds were completed using the shielded metal arc welding process and received radiographic and ultrasonic testing.

When the existing floor beams were cut, a portion of the bottom flange was maintained, straightened, and connected to new curved extension sections with bolted splices. The columns and hangers were then attached to the floor beams and arches. Initial connections used erection bolts only. Final connections were not completed until the deck had been removed and replaced.

Each arch was filled with pressurized nitrogen gas. The gas was pumped into the arch, replacing all of the air inside, and then sealed with a slightly positive pressure to provide an internal corrosion protection system. Permanent pressure gauges ensure that pressure loss does not occur.

On (but not Over) the Edge

An additional distinctive feature of the rehabilitation is the custom steel bridge railing. The railing system integrates a crash-tested 16-in.-high concrete "brush curb" with a continuous round steel rail at 28 in. high for vehicular protection, with a pedestrian railing component designed to meet AISC's architecturally exposed structural steel requirements.

The pedestrian component was designed as a vertical extension of inwardly curved bridge rail posts and 1-in.-diameter vertical pickets. The railing, which was galvanized then powder-coated, includes an aluminum top rail at 55 in. high that incorporates embedded LED strips to light the sidewalks for pedestrian safety. In addition, staged construction was used so the four-way intersection at the southern bridge approach could be reconstructed.

The challenges of the $8.1 million project spurred the design team and the City of Ithaca to pay attention to—and truly appreciate—the uniqueness of the arch structure. And it's paid off in more ways than one; in addition to state and local awards, the project just received the Award of Excellence for the Federal Highway Administration's 2008 Excellence in Highway Design Biennial Awards in the category "Structures Costing Less than $10 million."

Opportunities to create distinctive designs and apply new fabrication methods, as experienced with the Thurston Avenue Bridge project, are invaluable to expanding our knowledge base and moving the industry forward. MSC

Susan Matzat is a senior structural engineer and project manager with LaBella Associates, P.C.

Owner
City of Ithaca, N.Y.

Bridge Design
LaBella Associates, P.C., Rochester, N.Y.

Steel Fabricator and Detailer
BendTec, Inc., Duluth, Minn.
(AISC/NSBA Member)

General Contractor
Economy Paving, Inc., Cortland, N.Y.

Early Adopter

BY JOHN W. LITZINGER, P.E.

Photos: Courtesy of HNTB Corporation

A new steel bicycle and pedestrian bridge in Cupertino is the first cabled-stayed bridge crossing an Interstate highway in California.

THE TERMS "INNOVATIVE" AND "CUTTING-EDGE" HAVE BEEN ASSOCIATED WITH SILICON VALLEY FOR DECADES. Typically referring to the region's role in computer and software development, they now also apply to a new bicycle and footbridge in the area.

Scheduled to open in early 2009, the steel Mary Avenue Bicycle Footbridge is the first cabled-stay bridge crossing an Interstate in the state of California. The 500-ft-long by 16.3-ft-wide connector spans I-280 in Cupertino and closes a gap in a regional bike route joining suburban communities to work centers such as De Anza Community College, Homestead High School, and multiple technology companies in Cupertino and Sunnyvale. Complete with a structural steel girder-and-beam superstructure and a 13.5-ft-wide precast concrete panel deck, it is supported by 44 locked coil-stay cables, suspended from two 90-ft steel towers, with a clear span of 325 ft over the eight-lane Interstate and adjacent ramps. There are roughly 240 tons of steel in the project split evenly between the towers (W shapes) and superstructure rolled sections.

Twelve, 3-ft-diameter, 86-ft-deep cast-in-drilled-hole (CIDH) piles, with concrete footings and pile caps, make up the abutments and tower foundations. Nearly 12 acres of native landscaping, complete with sound-wall reconstruction, complement the bridge, as does 2,000 ft of lighted bicycle and pedestrian trails. The total project budget is $14.8 million, with $9.07 million going directly to bridge construction.

The bridge was originally designed with concrete, but this design bid out substantially over the planned budget. As such, the design team conducted a value engineering study to develop a budget responsive to alternatives and concluded that changing the pylons and superstructure to structural steel was the best option for multiple reasons, including speed of erection and seismic performance.

"In terms of seismic performance, the reduction in superstructure and pylon mass leads to large reductions in lateral forces and therefore foundations size," says Ted Zoli, Vice President and Technical Director-Bridges, with the bridge's designer, HNTB Corporation. "In the transverse direction, the superstructure is designed using the deck as a shear wall and the non-composite steel edge girders and floor beams as an eccentrically based frame. This is similar to a hybrid-coupled shear wall system used in seismically resistant building design."

Lightness and aesthetics were also factors. Terry Greene, an architect for the City of Cupertino, notes that the bridge weighs 1,700 lb per linear ft compared to the concrete design, which would have weighed 4,300 lb per linear ft.

"[The bridge's] lack of mass uses little airspace, which is important for visual impact," he explains. "Steel gives a lot of support and capacity for the airspace used. In terms of actually using it and enjoying it, what one sees is the structural capacity of steel being used in its very best way. It's not intrusive on the landscape and is very gentle in the way it populates the visual environment."

In addition, adds Zoli, the diamond-shaped towers optimize the bridge's torsional performance—ideal for edge girder-type cross sections—while adhering to the visual characteristics of the original design. This configuration also assisted in fabrication and erection, as it allowed the towers to be fabricated, transported, and erected in two pieces spliced only at the strut and the top. The geometry of the half-tower was designed so that it would be vertical in its free-standing condition, eliminating the need for temporary or shore bracing.

With the bridge being built over a busy freeway, lane closures were, of course, a concern. Fortunately, the steel design also resulted in a reduced

The diamond-shaped towers optimize the bridge's torsional performance while adhering to the visual characteristics of the original design.

construction schedule, which in turn resulted in fewer lane closures.

"Fewer, shorter lane closures were part of our intent to develop a project that is cost-responsible and efficient," Zoli says. "That's the idea behind the pylons and superstructure. Each tower can be erected in a single night and complete superstructure erection in three nights, compared to a cast-in-place concrete alternative that would take months."

Seismically Sound, Wind Resistant

Without question, a long-span bridge in this region must be designed for seismic considerations and wind stability. The Mary Avenue Bridge was designed to behave elastically under the design-level seismic event, and the unique cross section proved to have excellent wind stability, far exceeding design requirements.

According to Zoli, the gap between edge girder and deck proved to be extremely effective at enhancing aeroelastic behavior, both for flutter and for vortex shedding excitation. The section is also stable up to wind speeds exceeding 120 mph.

The pylons are W-shapes with both web and flanges tapered, and they bolt together at only two locations to minimize the number of field connections. The superstructure's floor beams and edge girders are comprised of W14 rolled sections and are proportioned to optimize seismic performance. Adina finite element structural software enhanced the HNTB team's seismic analysis and was supported by an in-house service software referred to as T187.

Reconnection

Beyond being the first cabled-stayed bridge crossing an Interstate in California, the bridge is also a symbol of reconnection in a day and age when major highways interrupt the flow of neighborhoods.

"Pedestrian bridges in urban areas serve to reconnect neighbors the Interstate system has divided," summarizes Zoli. "The premise that we don't build pedestrian bridges because they're too expensive isn't a solution for pedestrian safety. The engineering and construction industry should continue to develop elegant and inexpensive pedestrian bridges. It's up to us to ensure that the number of these types of opportunities continues to grow." MSC

John Litzinger is the office leader for HNTB Corporation in San Jose, Calif. His current focus is on rail and other transportation-related projects.

Owner/Client
California Department of Transportation/ City of Cupertino, Calif.

Designer
HNTB Corporation, San Jose, Calif. and New York

Steel Fabricator
Oregon Iron Works, Vancouver, Wash. (AISC/NSBA Member)

Steel Detailer
Tensor Engineering, Indian Harbor Beach, Fla. (AISC/NSBA Member)

Expanding History

BY MICHAEL W. CARROLL, P.E.

Photos: Harrington & Cortelyou, Inc.

A replacement span over the Missouri River provides much-needed extra room, scenic overlooks, and historic reverence.

FOR EIGHT DECADES, THE STEEL TRUSS BRIDGE SPANNING THE MISSOURI RIVER IN HERMANN, MO. had served its community well. But like many older bridges, it began requiring continual maintenance later in its life. In addition, the roadway was only 20 ft wide, and wide vehicles such as trucks, buses, and farm and emergency equipment had difficulty crossing the bridge against oncoming traffic; the bridge deck was often littered with broken side mirrors.

The replacement span, the new 2,244-ft Christopher S. Bond Bridge (named for the current U.S. Senator from Missouri), has expanded the roadway width to 44 ft and also includes an 8-ft protected bikeway, which provides a safe river crossing for access to the nearby Katy Trail. Three scenic overlook areas were provided on the bikeway of the $30-million bridge to allow pedestrians to look out over the river without obstructing the bikeway.

Span and Piers

The four-span main river unit of the bridge is composed of parallel-flange steel plate girders spanning up to 460 ft across the river. The long spans safely accommodate barge traffic on the river and helped minimize construction of costly piers. The river unit cross section is composed of six girder lines spaced at 9 ft, 8 in. with an 8.5-in. concrete deck. The steel plate girders have a web depth of 11.5 ft and were designed to use the maximum depth that could be handled and reasonably transported to the site. Girder depths were also sized to avoid the use of expensive horizontally welded web splices.

High-performance steel (HPS) was used for the negative moment flange plates over the piers; Grade 70W HPS helped economize the girder sections and allowed the use of parallel flanges instead of expensive haunched webs over the piers. Flange plates up to 3.25 in. thick and 40 in. wide were necessary for the long spans. Grade 50W steel was used for the positive moment flanges, web plates, and cross frames.

Steel erection in the Missouri River and handling large steel girders with barge cranes presented complex challenges. Design of the girder sections took into account the desirable lengths, weights, and stability of the individual steel segments. Field sections were sized for ease of shipping by truck and for handling during erection. The girder design was also optimized for ease of fabrication, which provided additional cost savings. In addition, thicker web plates eliminated the need for expensive welding of web stiffeners. Foundations for the river piers include 7-ft-diameter concrete drilled shafts that extend up to 80 ft into sound bedrock; the massive base shafts are designed to resist barge impact loads.

Ornamental pedestrian fence and handrails made from hollow structural sections adorn the new Christopher S. Bond Bridge.

High-performance steel was used for the negative moment flange plates over the piers.

Preserving History

An essential part of the project was to provide aesthetic enhancements that would allow the new bridge to blend with the historic character of Hermann. The bridge is located within the Historic Hermann District and many notable buildings are located near the bridge. The contractor was required to use special construction techniques and had to monitor ground vibrations at nearby structures so as not to threaten them. Driven piling was not allowed on the south side of the river within the city limits. End Bent 1, Bent 2, and Pier 3, all on the south side of the river, used drilled shaft foundations. In addition, no blasting was allowed on the south side of the river for any rock excavation or for removal of the existing bridge.

Ornamental pedestrian fence and handrails made from hollow structural sections adorn the new bridge. The fence and handrail provide protection over the Missouri River and Union Pacific Rail tracks, but also provide a visual impact, and decorative roadway and pier lighting were also put in place.

The pedestrian fence used 6-in. x 6-in. structural steel tubes for the posts, 2-in. x 6-in. tubes for the lower rails, and 3.5-in.-diameter pipe for the top rail. The handrail on the barrier used 4-in. x 4-in. steel tubes for the posts, 2-in. x 4-in. tubes for the lower rail, and 2.5-in. pipe for the upper rail. Pickets are 0.75-in.- and 1-in.-square steel bars.

The black steel railing, limestone-block concrete form liner for the piers, and decorative lighting all project a vision of strength, character, and pride that reflects well on the area. The new bridge serves as a vital transportation link over the river and also as a magnificent gateway to the historic town of Hermann. MSC

Michael Carroll is a director and project manager with Harrington and Cortelyou.

Owner
Missouri Department of Transportation

Designer
Harrington and Cortelyou, Kansas City

Steel Fabricator
Stupp Bridge Co., Bowling Green, Ky. (AISC/NSBA Member)

Steel Detailer
Stupp Bridge Co., St. Louis (AISC/NSBA Member)

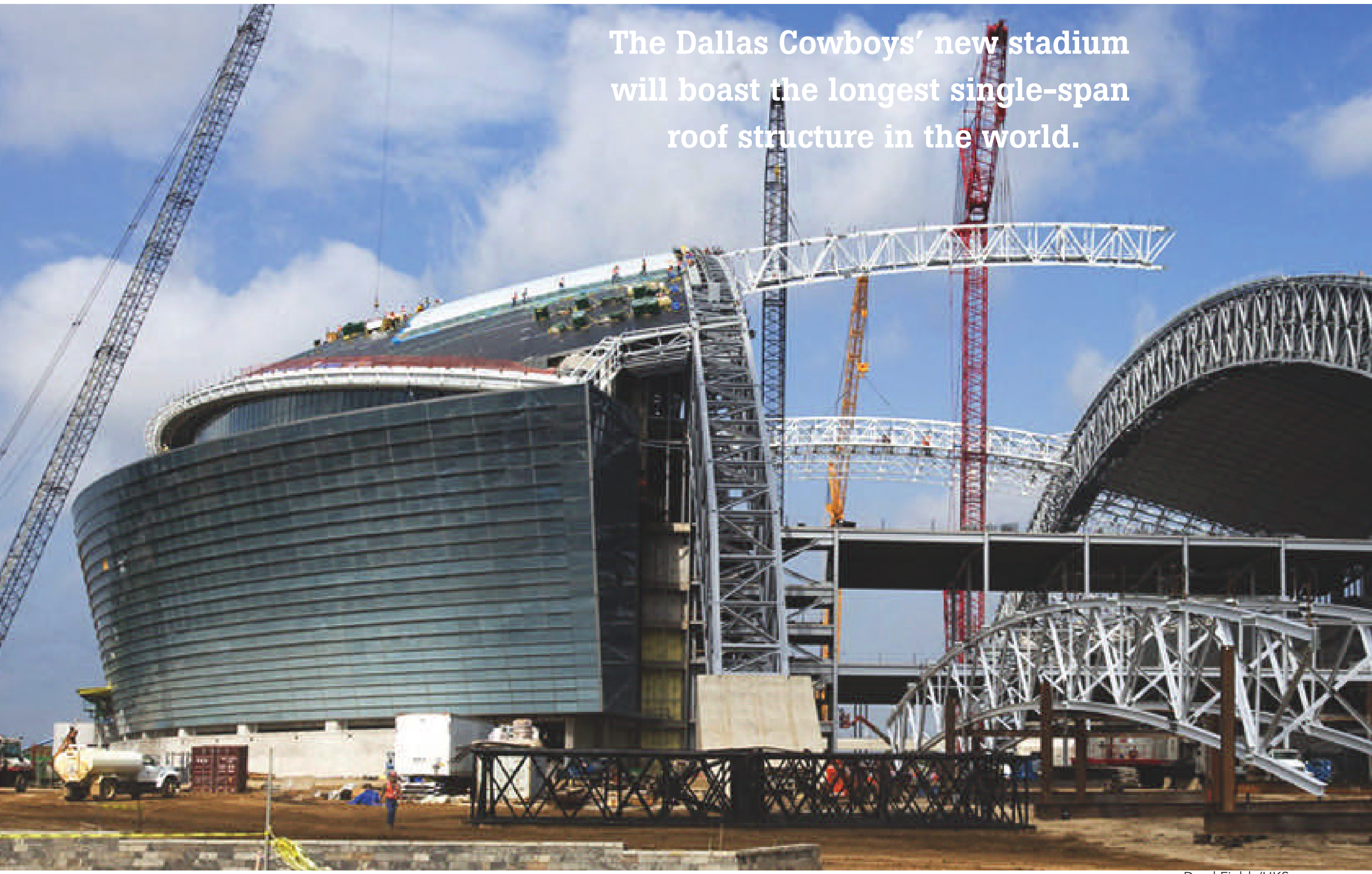
Contractor
Jensen Construction, Des Moines, Iowa


NATIONAL CHAMPIONS 1985
NATIONAL CHAMPIONS 2000
NUCOR
VULCRAFT GROUP

GOING LONG

BY R. JOHN ANIOL, P.E., S.E., JOSEPH DOWD, P.E., AND DAVID PLATTEN, P.E.

The Dallas Cowboys' new stadium will boast the longest single-span roof structure in the world.



Daryl Fields/HKS

THERE'S A JOKE THAT'S POPULAR AMONG DALLAS COWBOYS FANS: WHY IS THERE A HOLE IN THE ROOF OF TEXAS STADIUM? SO GOD CAN WATCH HIS FAVORITE TEAM PLAY.

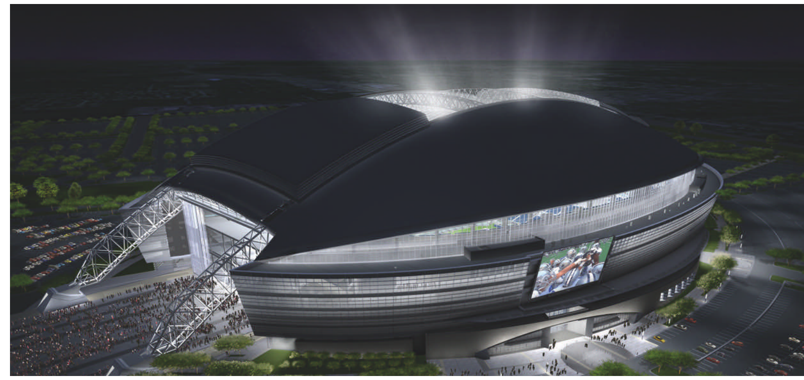
Despite the well-known "hole in the roof," the Cowboys' home since 1971 no longer exemplifies the team's winning tradition. Hence the Cowboys' move to a new stadium, one that does reflect the organization's commitment to success but doesn't compromise Texas Stadium's characteristic design elements or the Cowboys brand. When it opens in June 2009 in Arlington, Texas, the new $1.1-billion, 3-million-sq.-ft Cowboys Stadium will take its place among the world's premier sports venues, establishing several world records:

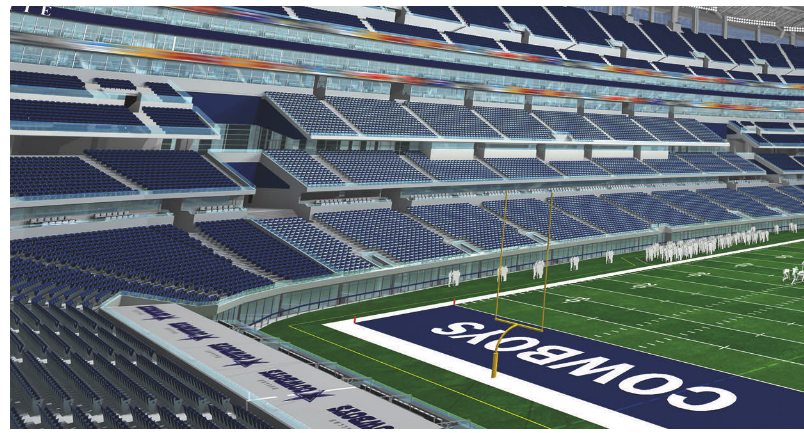
- The world's longest single-span roof structure. This will be supported by soaring twin arch box trusses that span 1,225 ft between abutments. At 660,800 sq. ft, the stadium's roof will be one of the largest domed structures in the world. The "hole" will remain but can now be closed when necessary, as the new stadium's roof is retractable. Formed by two bi-parting mechanized panels, each measuring 63,000 sq. ft, it can open or close in 12 minutes.
- The world's largest (at 25,000 sq. ft) center-hung high-definition video display board. Suspended 90 ft directly over the 50-yard line, the structure measures 72 ft high and 186 ft wide.
- The world's largest operable glass doors—each measuring 180 ft wide by 120 ft high—located at each end of the stadium.

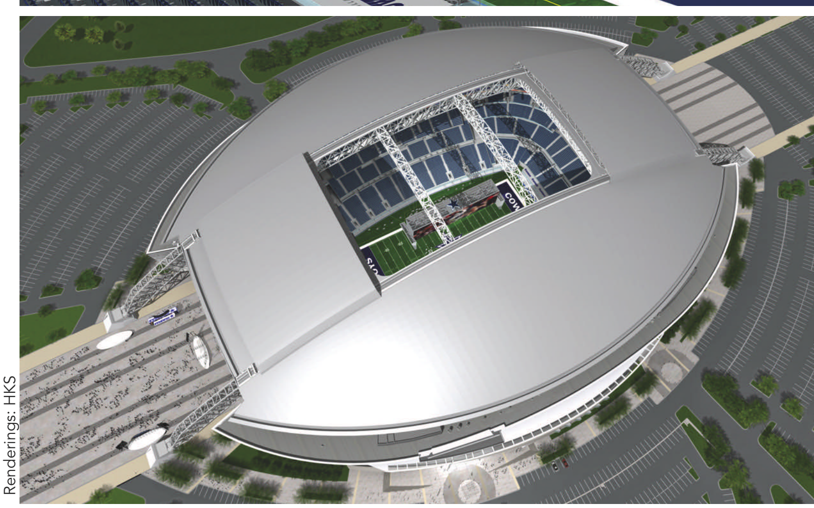
Monumental Arch Trusses Go the Extra Yard(s)

The record-setting arch box trusses have a radius of 1,025-ft. The apex of the arches occurs 292 ft above the field below, providing enough clearance for the Statue of Liberty below. Each 17-ft-wide by 35-ft-deep arch truss weighs 6.5 million lbs, the equivalent of 20 Boeing 777s. The four truss chords of each box truss are comprised of ASTM A913 Grade 65 steel, with sizes

A crane hoists a segment of one of the six box trusses into position between the soaring twin arch trusses.







Renderings: HKS

ranging from W14×311 to W14×730, made by ArcelorMittal in Luxembourg.

The project's structural engineers were able to realize an approximate 25% increase in yield strength and subsequent steel tonnage savings by minimizing the arch truss chord slenderness ratios ($33 < KL/r < 40$). (For more detailed information on the use of ASTM A913 steel, refer to "High-Strength Steel in the Long-Span Retractable Roof of Reliant Stadium" by L. Griffis et al, from the 2003 NASCC *Proceedings*, available at **www.asic.org**.) According to the project's steel contractor, W&W Steel, the cost of ASTM A913 group 4 and 5 shapes is virtually the same as ASTM A992. This saved the owner approximately $3 to $4 million.

The arch trusses feature a Quadrangular Warren web configuration, creating a Scottish argyle (diamond or checkerboard) pattern of web members along each vertical side, one of the world's most ancient three-dimensional spacing patterns. A Warren configuration was also used for the top and bottom chord horizontal bracing (lacing between each chord). After analyzing numerous web configurations, the engineers determined that employing the Quadrangular Warren configuration (repeating-X) along each vertical side would not only reduce the stress on the heavy chord members, but also enable them to stay within the W14×730 shape limit, thereby eliminating the need for costly built-up shapes or the use of more than four truss chords.

Each of the four 64,000-lb cast solid-steel arch-pin bearing assemblies support a 19-million-lb thrust reaction. The design-build arch pin bearing assembly, consisting of ASTM A148 Grade 50 and made by Uni-Systems, was cast in a custom sand mold and sits atop a 25-ft by 11-ft solid concrete thrust block column that launches out of the ground at a 32° angle from the horizontal to receive the ends of each arch. The real enormity of the system lies hidden below ground, as the thrust block column is anchored to a slurry wall box abutment that transfers the thrust into the surrounding soil.

The fixed side roof and fixed roof over the end zone seating areas are held aloft with planar trusses oriented to remain perpendicular to the roof's curving surface. This cant's variation matches the radial


Walter P Moore

Ironworkers tighten bolts in one of the slip-critical end plate splice connections that link the 56-ft long segments of the arch truss chords.


Walter P Moore

A 64,000-lb cast steel pin assembly touches down on one of four massive concrete abutments that transfer 19 million lb of thrust from an arch truss into the ground.

configuration of the arch truss, creating an elegant, sweeping structural system that emphasizes both rhythm and repetition.

A Really, Really Big-Screen TV

In addition to supporting the stadium's roof, the arch trusses permanently hold aloft the world's largest center-hung, high-definition video board—weighing 1.2 million lbs—plus an additional 200,000-lb show-rigging allowance that can be placed in dozens of configurations.

The 72-ft-tall center-hung video board, with over 25,000 sq. ft of video displays, looms over the football field in an enormous I-shaped plan that extends from nearly one 20-yard line to the other. The structural design team created a 72-ft-tall steel structure that contains a ten-level network of catwalks and is clad on all four sides with video displays. Behind this dynamic cladding lie full-height trusses that carry the total gravity load to the video board's support cables.

ASTM A586 structural spiral strand steel cables (3-in.-diameter) grip each end of the video board structure's I-shaped plan and extend vertically upward toward the opening in the roof. The cables are tethered to 14-ft-wide by 17-ft-deep steel box trusses that span the 256-ft opening between the twin arch trusses.

Detailing and Fabrication Execution

The project team held a series of collaborative coordination meetings during the design development and construction document phase to discuss design, fabrication and erection issues including roof geometry, connection design, erection sequencing, thermal movements, tolerances, and fit-up. The arch was fabricated in 56-ft segments, which enabled the basic geometry of each segment to be the same, reducing detailing and fabrication costs. To facilitate fabrication and erection preferences, W&W Steel proposed an alternate arch truss connection that consisted of a gusset plate shop groove welded to the toe of the chord flange and field bolted to the W14 diagonal and vertical members. An end-plate bearing chord splice connection was specified, resulting in significant connection tonnage savings. Regardless, each arch required 46,500 ASTM A490 bolts for assembly.

Roof and Video Board Erection

To facilitate more efficient construction, Manhattan Construction and Derr Steel proposed an alternate roof erection sequence to what was originally shown on the contract documents. The revised sequence consisted of erecting each arch on six temporary shoring towers, beginning at one abutment then progressing to the 50-yard line, where two bracing trusses were installed before moving to the other abutment where the sequence was repeated. Erection of a 56-ft keystone segment at the 50-yard line completed each arch. Upon completion of the south arch, shoring towers were moved to the north arch and the entire sequence proceeded again. Each arch truss took approximately five months to erect; the fixed roof and roof deck installation followed. The center fixed roof, six box trusses, and retractable roof will complete the roof erection for a total of 17 months.

Barnhart Crane and Rigging Co. will use several strand jacks to lift the completed video board steel structure from the playing field level to its final position, 90 ft above, then the high-definition video display panels will be installed on the completed structure.

Post-Game Recap

When Cowboys Stadium opens in 2009, innovation will truly score big in this world record-setting, monumental structure. The stadium's sleek and brilliant form, efficient function, robust structure, and fleet movement will team together to evoke the definitive Dallas Cowboys' image. With the roof ready to roll, the team and its spectators will experience games and events in an environment that champions originality and creativity. MSC

R. John Aniol is a Walter P Moore principal and served as Cowboys Stadium's structural project manager. Joseph Dowd is a senior associate with the firm and served as the structural project engineer for the roof. David Platten, a senior principal with Walter P Moore, served as the structural principal-in-charge. All three work in the firm's Dallas office.

Owner
City of Arlington, Texas

Developer
Blue Star Development/Dallas Cowboys

Architect
HKS, Inc., Dallas

Lead Structural Engineer
Walter P Moore, Dallas and Austin

Associate Structural Engineer
Campbell and Associates, Inc., Dallas

Roof Steel Fabricator
W & W Steel, LLC., Oklahoma City (AISC Member)

Roof Steel Erector
Derr Steel Erection Company, Euless, Texas (AISC Member)

Roof Steel Connection Engineer
W & W Steel
Computerized Structural Design, S.C., Milwaukee

General Contractor
Manhattan Construction Company, Dallas

Mechanization Consultant
Uni-Systems, LLC, Minneapolis

Video Board Structure Lifter
Barnhart Crane and Rigging Co., Memphis, Tenn. (TAUC Member)

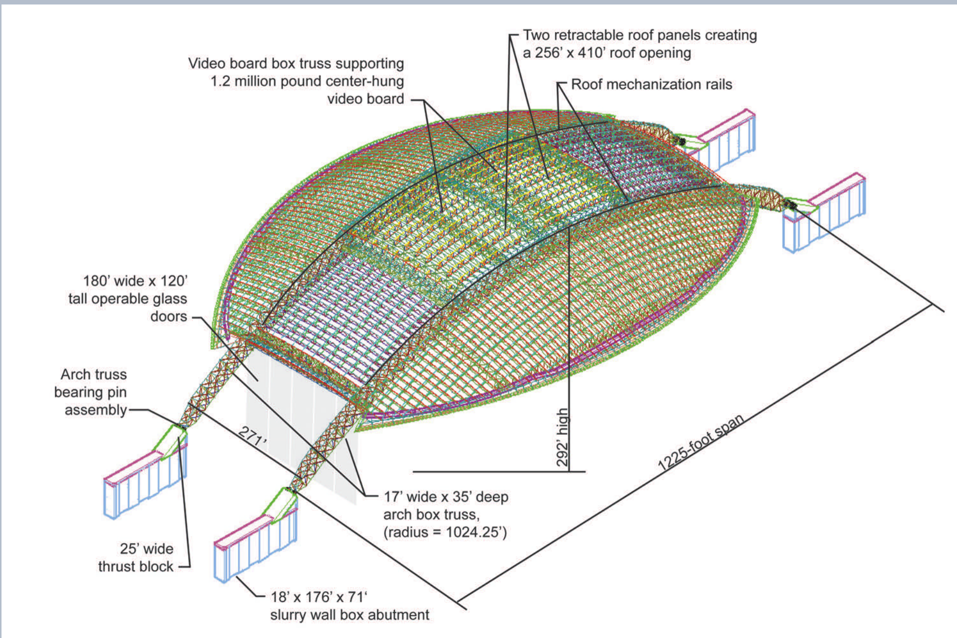
Software
SAP 2000
Tekla Structures and AutoCAD 3D

Rolling Roof Plays to Win

While it was important to bring the "hole" design from the old stadium to the new one, another key design goal was having a fully enclosed stadium roof to protect spectators from inclement weather and enable the structure, which can be configured to seat 100,000, to function as a multi-purpose venue capable of hosting NCAA Basketball Final Four Tournaments, the Cotton Bowl, concerts, and even the Super Bowl. The design team accommodated both of these seemingly mutually exclusive desires with a sleek, bi-parting, retractable roof, which can open or close in 12 minutes.

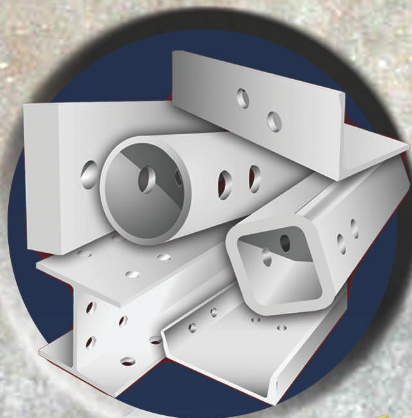
The two translucent retractable roof panels—using Birdair cladding—each measuring 290 ft by 220 ft, travel along the length of the arches to meet at the 50-yard line. The panels are clad with a Telflon PTFE-coated fiberglass tensile membrane (Sheerfill I–HT by Saint-Gobain Performance Plastics Corp.). The mechanization system, designed by Uni-Systems in close coordination with Walter P Moore, is the first application of a rack-and-pinion retractable roof in North America. Each of the roof's 128 motors (32 per roof quadrant) powers a pinion that eases the operable panels down 328 ft of toothed steel rack permanently attached to the arch trusses. As the moving roof progresses downhill, the slope becomes more dramatic as the arches maintain their constant radius curve.

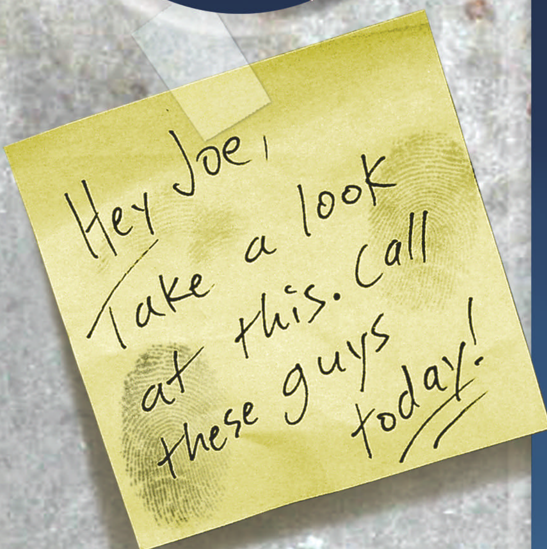
When the two roof panels are parked in the open position to reveal the iconic roof opening, the rack-and-pinion drive system holds the roof panels at a slope of 23°. While this feat is impressive enough, the drive system must also power the retractable panels back into the closed position, conquering this 23° slope in the uphill direction. The efficiency of this system is demonstrated in the modest power requirements to move the panels' 3.5 million-lb weight back up the slope. Each of the 128 motors produces 7.5 horsepower, making the 960 horsepower required to close the roof roughly equivalent to only three Ford Mustang GT engines.



Walter P Moore

A computer analysis model with more than 30,000 frame elements helped structural engineers conquer the stadium's distinctive challenges.





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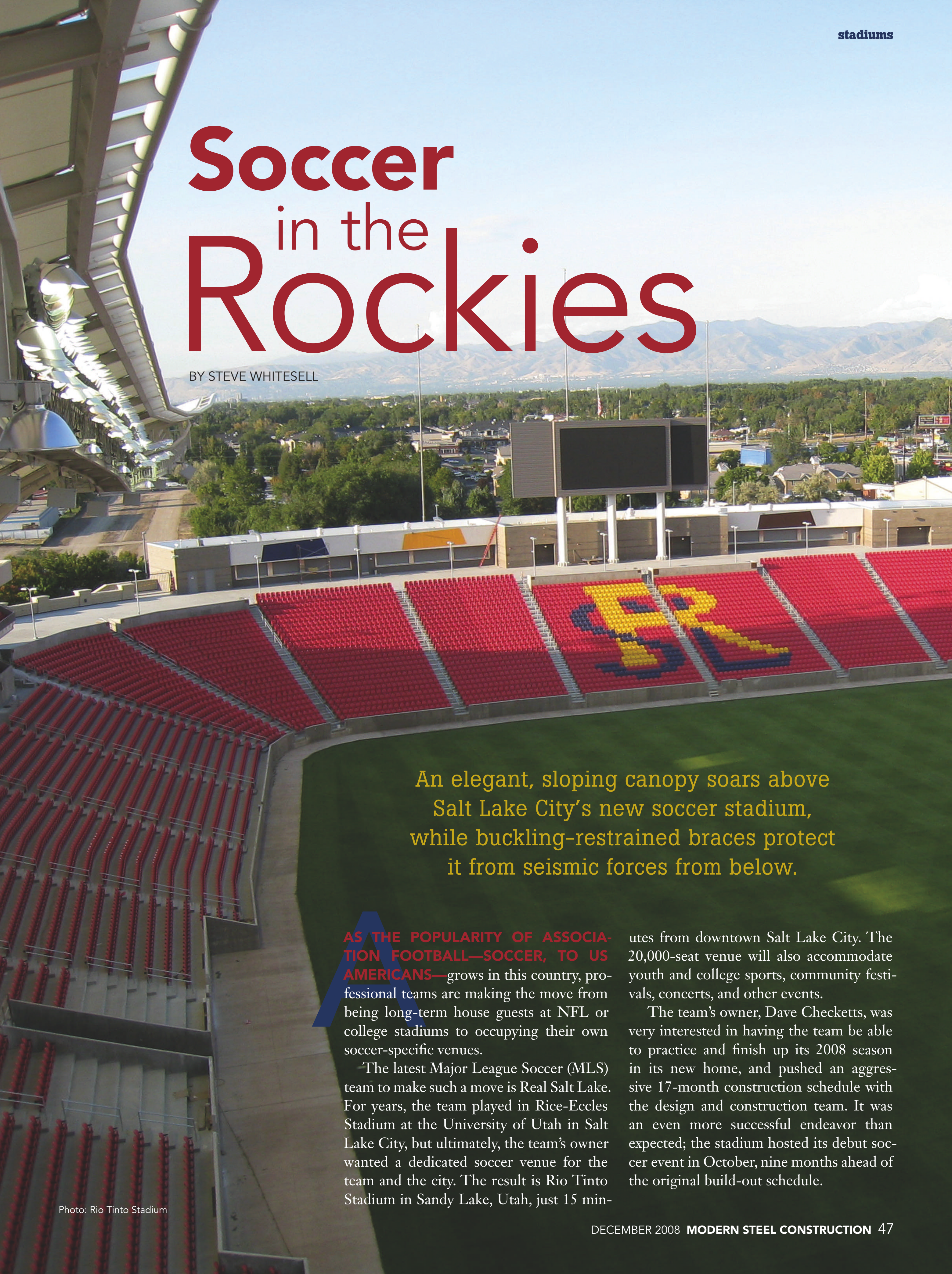

Soccer in the Rockies

BY STEVE WHITESELL

An elegant, sloping canopy soars above Salt Lake City's new soccer stadium, while buckling-restrained braces protect it from seismic forces from below.

AS THE POPULARITY OF ASSOCIATION FOOTBALL—SOCCER, TO US AMERICANS—grows in this country, professional teams are making the move from being long-term house guests at NFL or college stadiums to occupying their own soccer-specific venues.

The latest Major League Soccer (MLS) team to make such a move is Real Salt Lake. For years, the team played in Rice-Eccles Stadium at the University of Utah in Salt Lake City, but ultimately, the team's owner wanted a dedicated soccer venue for the team and the city. The result is Rio Tinto Stadium in Sandy Lake, Utah, just 15 minutes from downtown Salt Lake City. The 20,000-seat venue will also accommodate youth and college sports, community festivals, concerts, and other events.

The team's owner, Dave Checketts, was very interested in having the team be able to practice and finish up its 2008 season in its new home, and pushed an aggressive 17-month construction schedule with the design and construction team. It was an even more successful endeavor than expected; the stadium hosted its debut soccer event in October, nine months ahead of the original build-out schedule.

Photo: Rio Tinto Stadium

Roof framing before canopy installation.

Photos: Rio Tinto Stadium

Erection of the canopy framing.

Multi-Jointed Collaboration

Facilitating the accelerated schedule was general contractor Turner Construction's (in a joint venture with Layton Construction) prior experience with another MLS stadium, for the Colorado Rapids in Denver. For both projects, Turner used a design-build consortium, which for the Rio Tinto Stadium project included an erection and fabrication joint venture between erector LPR Construction and buckling-restrained brace (BRB) manufacturer Star Seismic. (Star Seismic's patented BRB designs incorporate pinned or welded connections, radiused copes, material consistency, and higher capacity systems to deliver maximum structural survivability while using less steel, welding, and crane time in the erection process. They also exceed all AISC provisions, and independent analysis shows that incorporating BRBs reduces building costs by up $2.40 per square foot.)

This collaborative effort was made possible by the understanding of Gene Fatur, Turner's executive project manager, that specific steel management and fabrication excellence would pay dividends to the owner. Working in concert, the LPR-Star-Group and Layton-Turner ventures used the expertise of structural engineer John A. Martin and Associates to meet the architectural vision—and bring it to fruition well ahead of schedule.

The primary objective for this joint venture was to procure the structural steel in advance, thereby avoiding costly steel material increases during the design process, and work in conjunction with the design team to implement the shapes into the design. A mill order was placed when the design development was only 50% complete, and was updated throughout the 12-week rolling cycle, enabling the team to not only stay on budget, but also improve the schedule. Opening nine months early not only allowed the team to finish out the season in its new facility, but also avoided costly winter construction.

Of the total original design budget, the structural engineering and fabrication costs were reduced by 9.1%, saving well over $1 million. Of this amount, nearly $250,000 was related to the incorporation of Star Seismic's WildCat BRBs. By using 191 WildCat braces in all, the LPR/StarGroup was able to reduce brace connections by 90% and allow the engineer to use smaller beams and columns. According to Star Seismic principal Argan Johnson, using a BRB system replaced the eccentric braced frame system, which allowed the team to save on materials, fabrication, welding, and bolting. In sum, 162,000 cubic yards of earth was removed, 5,000 cubic yards of concrete was poured, and 720 tons of steel was managed.

Rio Tinto Stadium

An aggressive construction schedule allowed Rio Tinto Stadium to open nine months early.

Wind and Snow

One of the key architectural elements of the stadium is a sloping fabric canopy whose white steel framing and fabric capture the essence of the surrounding snowy peaks. The canopy was designed to garner attention from afar, and the round curved beams at the canopy edges intend to resemble a "smooth sky trail on a snowy mountain top."

Because the Salt Lake area receives almost 5 ft of precipitation and multiple winter storms every year, the canopy had to withstand several forces simultaneously and drain effectively. Layton-Turner worked with wind engineering consultant RWDI (Guelph, Ontario, Canada) regarding wind and snowfall analysis. While several design options were discussed, a geometric steel frame design with tension fabric was selected to meet both the design criteria and engineering requirements of the sloped canopy.

Working with the project's structural engineer, John A. Martin Associates, the LPR/StarGroup joint venture was able to order materials before the snow study was complete. In essence, the team was able to develop a lighter structure with better load paths to reduce the total steel costs at a time when material costs were high. MSC

Steve Whitesell is president of the Whitesell Group, a public relations firm in Park City, Utah.

Architect
Rossetti Architects, Los Angeles

Structural Engineer
John A. Martin Associates, Los Angeles

Steel Erector
LPR Construction, Loveland. Colo.
(AISC Member)

Buckling-Restrained Brace Supplier
Star Seismic, LLC, Park City, Utah
(AISC Member)

General Contractor
Turner Construction and Layton Construction (a joint venture), Salt Lake City

Scoring with the Fans

Rio Tinto Stadium can seat 20,008 for soccer games, but for concerts, festivals, and community events, can accommodate up to 25,000. With a total of four levels, the stadium contains locker rooms, utility services, team offices, 32 private luxury suites, a presidential suite, a press lounge and press box, camera stations, and mechanical rooms.

The state-of-the-art audio/visual system includes a 40-ft by 22-ft scoreboard and a crystal-clear sound and paging system. To enhance the fan experience, the first row of seating is only 2 ft above the playing field, putting the fans as close as possible to the excitement. The natural grass turf and drainage system allows for year-round use, even at an elevation of 4,450 ft.

Rio Tinto Stadium is situated in a north-south direction, replicating the alignment of the Rocky Mountains just a few miles to the east. The stadium's placement has left room for commercial development just to the north that will include a hotel, a water park, restaurants, and retail space.

Outside Looking In

BY MANUEL HERNANDEZ, ASSOCIATE AIA

Photos: Tony Soluri Photography

Steel rooftop bleachers give Cubs fans a sturdy perch for watching the game from across the street.

WHILE THE CHICAGO CUBS' 2008 SEASON didn't end quite the way the club and its fans hoped and expected it to, there's one thing the Cubs faithful take enduring pride in: Wrigley Field.

And as anyone who has watched the Cubs on TV or has been to the park knows, not all of the seating is in Wrigley Field itself. Rooftop bleachers on the buildings across the street on two sides of the stadium have been a prominent part of the game-day experience for several years now.

Older stadiums like Wrigley Field require renovation or expansion from time to time, and the same goes for rooftop bleachers. Until recently, the three-story brick residential building at 1010 Waveland Ave.—the left-field side of the park—hosted a small rooftop prefab bleacher system, with a seating capacity of 100 people, that only took up one portion of the roof. This system has now been replaced with a new steel bleacher system, the top of which is 64 ft above street level, that seats 200. The bleachers occupy the west side of the roof while the east side now contains new amenities including bathrooms, a bar, tables,

From across the street from Wrigley Field, 200 fans can get a clear view of the action (right).

and food service counters. The entire bleacher structure was made of galvanized W-shape beams and columns. Hollow structural shapes were used for the railings and bar tray tabletops. Shapes included HSS4×4×5/16, HSS6×6×3/8, and W12×40. The entire project cost $2.5 million and used 40 tons of structural steel.

The beams and columns of the system, using mostly bolted connections, sit on the existing load-bearing masonry walls. A new concrete roof deck had to be poured over the existing wood-framed roof—2 ft above it—to allow for utilities to run underneath and service the space. The concrete deck was framed out with wide-flange steel beams spanning from bearing wall to bearing wall. As it was challenging to fit the structural system onto the roof walls, laser surveying of the roof was performed. The existing roof was reinforced with W18×35 and W16×26 shapes for the most part.

The stair systems span from the roof to the upper-most bleacher level and have new footings and foundation walls. The structure also includes steel catwalks from one level of the roof to the other, given the irregular exterior walls of the building and the internal courtyards that provide natural light to the residences facing inward.

While the Cubs' dreams of a World Series didn't come true this year, come spring, fans will no doubt flock to the Friendly Confines of Wrigley Field—and the rooftop bleachers just beyond—to watch their team give it another try. MSC

Manuel Hernandez is vice president of kutlesa+hernandez architects, inc.

Owner
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The bleacher system's beams and columns sit on the building's existing load-bearing masonry walls and a new concrete roof (left).

Updating the Seismic Provisions

BY JAMES O. MALLEY, S.E.

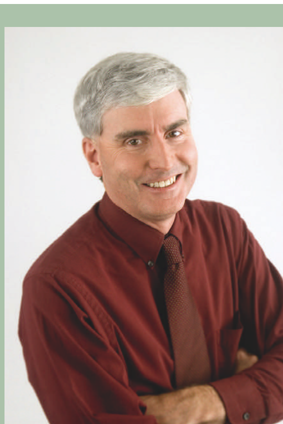
The AISC Seismic Provisions are the result of input from several experts—including you.

AISC TASK COMMITTEE 9 ON SEISMIC DESIGN is made up of a large group of hard-working professionals that span the breadth of the structural steel industry, with practicing engineers, researchers, and fabricators/erectors all well represented. Since 1995, I have had the honor and privilege of chairing this committed, highly productive group of experts in seismic design.

TC9 is responsible for developing AISC Document 341, *Seismic Provisions for Structural Steel Buildings*. The AISC 341 Provisions provide the detailed system requirements for the seismic design of all steel buildings and non-building structures in jurisdictions that adopt the International Building Code through their adoption by reference in ASCE 7. An extensive Commentary is also written by AISC TC9 to provide the background for the Provisions of AISC 341.

It should be noted that the work of AISC TC9 is done in conjunction with the ASCE 7 Seismic Subcommittee and the Building Seismic Safety Council's Provisions Update Committee. These two committees are responsible for defining the system design parameters (R, C_d, and Ω_0) and height limits for the various Seismic Design Categories for all structural systems defined in ASCE 7, including the incorporation of new structural systems or changes to the design parameters of existing systems. Close coordination is required between ASCE 7 Seismic Subcommittee and standards-writing committees for the different building materials (such as AISC TC9 for structural steel) to ensure that the system design parameters and height limits are consistent with the detailed member and connection design and detailing requirements.

Since their initial publication in 1992 (under the direction of then-chair Professor Egor Popov), the AISC Seismic Pro-



James O. Malley is a Senior Principal with Degenkolb Engineers in San Francisco and is the chair of AISC Task Committee 9 on Seismic Design.

visions have been constantly reviewed, updated, and hopefully improved. The 1997 edition of the Seismic Provisions was a major revision that tried to incorporate many of the early lessons learned in the aftermath of the Northridge earthquake damage. In both 1999 and 2000, TC9 published two brief amendment documents in an attempt to stay as current as possible with the large influx of new information being generated by the FEMA/SAC Steel Program and many other studies. In 2002, a complete update to the provisions was completed to fully integrate this information. The most recent edition, completed in 2005, included two new structural systems (Buckling-Restrained Braced Frames and Special Plate Shear Walls), a major expansion to the requirements for project documentation and quality control and assurance, and a number of other updates to the various sections of the document.

Next Edition

The next edition of AISC 341 is planned for publication in 2010. This will be done in conjunction with the main steel specification, AISC 360, *Specification for Structural Steel Buildings*. It is intended that this and all future updates will also be on a five-year cycle to properly coordinate with the ASCE 7 and IBC publication cycles. The present schedule calls for the 2010 edition of AISC 341 to be incorporated into the 2012 edition of the IBC.

Our TC 9 Committee is hard at work on the 2010 edition, with the first ballot versions to be reviewed by the AISC Committee on Specifications this fall. We are working on a revised format that will more closely correlate with AISC 360 and incorporate the composite construction provisions more directly into the document (previous editions placed the composite provisions into a separate part of the document) in addition to the typical technical updates that incorporate the latest information and thinking of the committee. As with all the previous editions, we hope that the 2010 edition will be more transparent and easier for the structural engineers that design and detail steel buildings to use.

Make Yourself Heard

Here's where those you that use AISC 341 on a regular or semi-regular basis can help: In our quest to continually improve the document, we want to encourage questions, comments, suggestions, and even complaints about it. While our group attempts to write provisions that are clear, concise, and complete, we understand that the design process is different from that of developing specification language. Comments and suggestions received from engineers that use the specifications to design structures on a daily basis are the best gauge of how well the document is serving the profession and the steel industry. I promise you that every comment received by AISC will be duly considered and addressed during our deliberations. Comments specific to system design parameters that are the responsibility of ASCE 7 will be forwarded for the consideration of that committee.

So, please send your comments to Cindi Duncan, AISC's Director of Engineering and the staff person in charge of TC9's activities. Her e-mail address is **duncan@aisc.org**.

Finally, we would like you to review and provide comments on the public ballot versions of the next edition of AISC 341, which will be published sometime early in 2009. Please keep an eye out for that public ballot as another means of helping TC 9 to produce the best set of seismic provisions possible. MSC

A Skewed Perspective

BY FRED BECKMANN, BOB CISNEROS, RONNIE MEDLOCK, AND DON WHITE

Starting with girders out-of-plumb is the way to go for skewed bridge construction.

THE IDEA OF BEAMS BEING TWISTED DURING ERECTION might seem a bit alarming to the average person. But in the case of skewed bridges, beams, in fact, *should* be twisted out of shape during erection.

Specifically, steel girder webs on straight skewed bridges must be out-of-plumb during construction if they are to be plumb at the end of the construction. This is because the individual girders and overall bridge cross section twist, or displace torsionally, as the concrete deck is placed; if the girders are made plumb before erection, they will twist out-of-plumb as the deck is placed. In other words, the girders can be plumb in only one configuration. When this phenomenon is not understood, delays, rework, claims, or compromised performance can result. (This article is limited to straight bridges with intermediate cross-frames that are normal to the girders, as opposed to parallel to the skew angle.)

Deflections

Engineers generally understand that girders deflect under load and that this deflection varies along the length of the girders. For example, on a simply supported girder, the deflection is greatest near the mid-span and varies to zero at the supports.

During the bridge construction sequence, the girders deflect a certain amount under their own weight, then deflect further when the deck is placed. On a square (non-skewed) bridge, the deflections across any section of the bridge due to the deck weight are roughly the same; for example, at the mid-span on a square bridge, the girders all deflect approximately the same amount as the deck is placed.

By contrast, on a skewed bridge, the deflections are not the same across the width of the bridge since the girders are longitudinally offset from each other by the skew—i.e., there are differential deflections between the girders across any section of the bridge. However, the girders cannot realize these differential deflections without twisting because they are tied together by relatively rigid cross-frames. As the dead load is applied, the change in the shape of the cross-frames is relatively minor compared to the deflection of the girders. Prior to their connection to the cross-frames, steel I-girders are torsionally flexible; the skewed girders tied together with cross-frames twist due to differential deflections.

Girder Twist and Layover

For straight skewed bridges, it is recommended that the girders should be plumb (within a reasonable tolerance) when the construction is complete. Therefore, as stated above, the girders must be out-of-plumb so that as the deck is placed, they will deflect and untwist into the plumb condition. Detailing and fabricating the cross-frames such that the girders will be plumb under the application of the total dead load facilitates the alignment of the adjacent deck segments at expansion joints and avoids potentially visible layover of the girder webs relative to substructure components.

The natural question, then, is just how out-of-plumb the girders should be prior to the deck placement—i.e., what is the required layover prior to deck placement? This number is not difficult to calculate, but doing so is unnecessary.

Consider how girders and cross-frames change shape during erection: The girders begin in a twisted, out-of-plumb state before the deck placement, then twist to plumb as the concrete is added. However, the cross-frames stay effectively rigid (within their planes). Therefore, the simple way to set the girders to the proper web layover is to detail and fabricate the cross-frames to their final desired geometry and to fit the girders to the cross-frames in the field.

The AASHTO/NSBA Steel Bridge Collaboration Standard G12.1, *Design for Constructability*, Figure 1.6.1.B, describes an effective way to erect girders on a skewed bridge using this approach (note that this example is just one way of erecting a bridge):

- The girders are first set in place in a plumb condition, without intermediate cross-frames added (at this stage, the girders must be properly supported to ensure stability).
- As intermediate cross-frames are added, the girders are twisted—i.e., forced into position—to fit the frames. In the G12.1 example, this is accomplished by first suspending the frames from the top two corners and then pushing the girders as needed to fit the bottom two corners. The bolts are tightened (a must), and then the girder condition is set and ready for the deck placement; the girders have now been set to the proper layover by the attachment of the cross-frames to the connection plates.

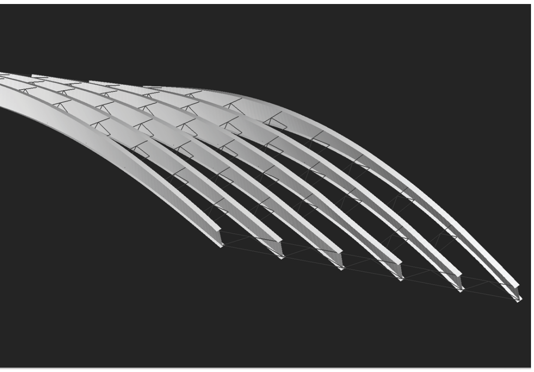
The erector chooses a girder and cross-frame erection sequence to align and twist each subsequent girder to fit the previously erected girder during construction.

Analysis

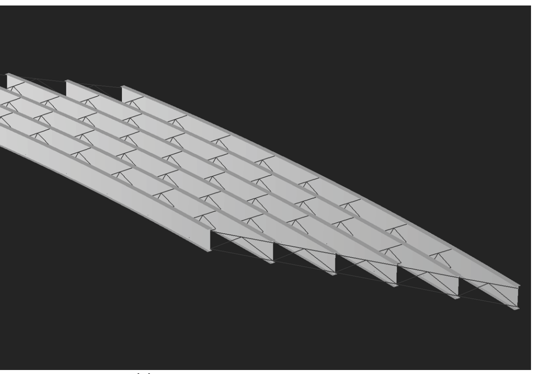
What twist forces must be accounted for in the design to accommodate the girder layover and rotation to plumb? For straight, skewed I-girder bridges, the answer is simple: none. The girders are twisted out of plumb during cross-frame installation (by use of come-alongs or other force), but then they "untwist" back into the plumb position once the deck is placed. Though the girders experience a certain amount of stress during the initial twisting, this stress is relatively small and is largely released when the girders untwist into their final and proper orientation during the deck placement.

Plumb before Placement?

It is certainly possible to set girders up plumb prior to casting the deck, though we don't recommended this practice for straight skewed bridges. To achieve this condition, the girders and cross-frames must be detailed and fabricated accordingly, and the fabricator must know this prior to de-



Skewed beam geometry, before deck pouring.



Final beam geometry.

tailing and, preferably, prior to bidding the job.

However, not only does setting girders up plumb prior to deck casting result in a final out-of-plumb condition, but also the final twisted shape will induce some additional stresses in the girders and cross-frames in the final condition. Again, using the recommended approach, the stresses due to the initial twisting of the girders during installation of the cross-frames largely offset the stresses due to the twisting of the girders in the opposite direction under the action of the dead load.

Curved Bridges

The behavior of curved bridges, especially skewed curved bridges, is similar but more complex than straight skewed bridges. Although the concepts discussed in this article also apply to a certain extent to curved bridges, other considerations not discussed here are necessary for these structure types.

Bearing Line Diaphragms or Cross-frames

Bearing line diaphragms or cross-frames, which tie the girders together at the bearing lines, are a special case and, depending on the framing arrangement, can be difficult to install, especially at abutments. The construction rotations must be accommodated in the bearing design.

Unlike intermediate cross-frames, abutment cross-frames always follow the skew of the bearing line. Since the vertical deflections are zero at the bearing line, differential deflection twist effects are avoided. However, the abutment cross-frames introduce a twist associated with longitudinal girder rotation. Since the abutment cross-frames or diaphragms are relatively stiff in their own plane compared to the stiffness of the girders, they force a twist into the girders such that compatibility with the girders is maintained. The AASHTO/NSBA Steel Bridge Collaboration Standard S10.1, *Steel Bridge Erection Guide Specification's* first sample of erection procedures illustrates one method of installing the abutment cross-frames such that the girders are initially rotated out-of-plane at the bearing lines but rotate back to plumb under the action of the dead load.

Fasteners

Fasteners that connect the girders to the cross-frames must be installed and tightened before the deck is placed. Otherwise, the cross-frames cannot maintain the proper girder orientation during deck placement. Local exceptions to this rule may be instituted in some cases at end cross-frames, where the interaction with adjacent intermediate cross-frames may make the installation of one of the frames difficult. Also, in parallel staged construction, where superstructures are built in separate longitudinal units with a longitudinal joint between them, exceptions to this rule are common at the longitudinal joint between the superstructures. However, in each of these cases, the girders and cross-frames are well connected within the individual units such that the deflected geometry can be reliably calculated and controlled.

One last consideration of fundamental importance: Successful skewed bridge erection, with the least impact of the skew on the design, requires an experienced bridge project team that understands—and applies—the principles addressed in this article. MSC

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Under Foot

BY SUSAN BURMEISTER, P.E., AND WILLIAM P. JACOBS, P.E.

Horizontal floor diaphragm load effects on composite beam design.

ONE OF THE MOST NEGLECTED ELEMENTS in the design of buildings is the horizontal floor diaphragm and its interaction with the lateral load resisting systems. Most multi-story structures depend on the floor slab and roof systems to act as horizontal diaphragms to collect and distribute the lateral loads to the vertical framing members, which provide the overall structural stability.

In steel structures, floor diaphragms are most commonly constructed using composite steel deck with concrete fill, although other systems, such as pre-cast planks, formed reinforced concrete, or concrete on non-composite steel deck, may also be used. While there are numerous references that discuss the design of the diaphragm itself, there is little guidance available on the transfer of diaphragm forces into the lateral load resisting system. In addition, the specific issues related to beam design for members collecting lateral loads in composite floor systems has gone largely undocumented. The intent of this article is to help "fill in the gaps" on these issues through a discussion of the effect of diaphragm forces on the supporting steel beam behavior, as well as through practical detailing guidelines.

General Diaphragm Behavior

Before delving into the specific issues associated with the transfer of diaphragm forces to the supporting framing, it is necessary to understand general diaphragm behavior and how assumptions made affect the detailing required in establishing a robust load path. Figure 1 depicts a floor plan for a typical steel building. Braced frames are provided adjacent to stairwells near each end of the building to resist the lateral loads, and the diaphragm strength is assumed to be adequate to transfer the shear around the openings. In a simple analysis, the floor diaphragm is idealized as a continuous cantilevered beam and the braced frames are treated as beam supports. Due to the symmetry of this example and assuming the braced frames have the same geometry and stiffness, the diaphragm force at each braced frame will be equal to 50% of the total applied lateral load.

Depending on the magnitude of lateral load to be transferred to the braced frames, the designer can detail the force transfer to occur uniformly along the entire frame line between grids *A* and *D* on the grid lines where the braces occur, or they may elect to concentrate the load transfer to a segment of this length, such as the beam in the braced frame between grids *B* and *C*.

In the first scenario, the load distribution is proportional to the overall available transfer length, and beams *A-B* and *C-D* each collect 35% of the total force while beam *B-C* collects 30% of the total force. Beams *A-B*, *B-C*, and *C-D* are all crucial members for getting load to the lateral load resisting system, and the connections of these beams to the columns at grids *B* and *C* must be designed for a horizontal force equal to the axial load being transferred through the column joint to the braced frame plus a vertical shear force resulting from the eccentricity of the diaphragm relative to the beam centerline. These member forces will occur simultaneously with the vertical beam shear reactions due to the gravity loads. Design and detailing of these joints for the combined forces is often overlooked.

In the second scenario, beam *B-C* collects 100% of the force. The distribution of axial, shear, and flexural member forces due to the applied lateral load for this beam will depend on the specific braced frame configuration. Once defined, these forces can be transferred into the braces with standard braced frame connections. Figure 2 illustrates the shear flow associated with this scenario.

Tension and compression chord forces are developed at the perimeter of the floor diaphragm due to the lateral loads. Typically, the floor slab concrete can resist the compression chord forces. Tension chord forces can be resisted by the spandrel steel beams, continuous steel closure plates, or by reinforcing steel within the concrete slab. In order to use the spandrel steel beams as the tension chord, the diaphragm chord forces must be transferred into the steel beams, and the steel beam connections at the columns must have sufficient strength to transfer the beam forces through the column joints. Again, this is a condition that often is overlooked, where the beam connections must be designed for the combined effects of vertical shear loads and horizontal axial loads.

Once the basic distribution of horizontal forces is understood, the effect of these forces on the design of the composite beams can be examined.

Figure 1. Typical building floor plan

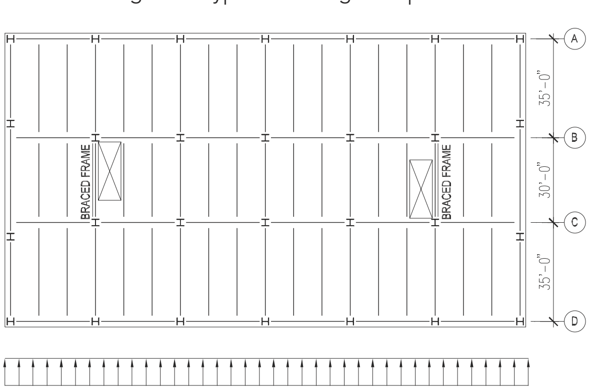
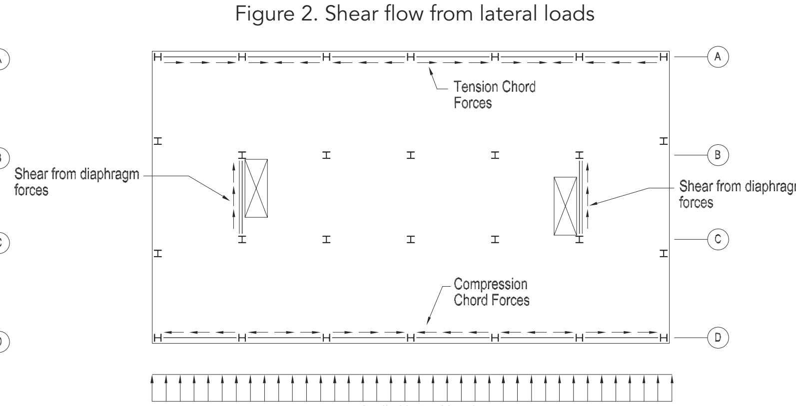


Figure 2. Shear flow from lateral loads



Additional Shear Connection?

The mechanical connection of the floor system to the supporting steel beams in a composite beam system is achieved using headed steel studs or hot-rolled channel shear connectors. Often, the beams are designed as composite members for the gravity loads applied to the floor system. Traditionally, if the beam also serves as a collector element, additional shear studs are added to account for transfer of the superimposed horizontal load into the beam. However, this practice is not always necessary for two primary reasons.

First, the quantity of shear studs selected for a composite beam is usually determined based on a gravity load combination, such as 1.2*D*+1.6*L* (LRFD) or 1.0*D*+1.0*L* (ASD). When lateral loads are applied in conjunction with the gravity loads, the load combinations of ASCE 7 reduce the live load levels. Under these reduced live loads, the shear studs provided to develop the composite action required for the gravity loads will be under-used and thus have additional capacity available for the transfer of the diaphragm forces.

Second, the interaction of the shear flow from the different loading conditions is additive for some studs but opposite for others. The distribution of horizontal shear from beam flexure is assumed to flow in two directions from the point of maximum moment to the point of zero moment. For a typical simple-span composite beam with uniform gravity loads, this shear flow is as indicated in Figure 3. While the beam shear is greatest at the ends of the beams, it is common practice to assume that the shear studs will deform and redistribute the shear uniformly to all studs.

Conversely, lateral loads induce shear in only one direction. When these beams are used to collect the diaphragm forces, the shears due to the lateral loads are superimposed on the horizontal shears due to the gravity loads, as indicated in Figure 4. On one side of the beam, the lateral loads increase the horizontal shears over the gravity-induced values, while on the other side of the beam, the lateral loads oppose the gravity-induced horizontal shears.

Assuming the shear studs have sufficient ductility to distribute the horizontal shears evenly along the beam, a composite beam can transfer a horizontal shear due to lateral loads between the floor diaphragm and steel beam that is equal to the summation of the strengths of all the shear studs on the beam regardless of demand on the shear studs from the gravity loads.

"Non-Composite" Composite Beams

Designers encounter many conditions where steel beams are designed as non-composite members under gravity loads. Shear studs placed on these beams for transfer of lateral forces still will be subjected to horizontal shears due to flexure from gravity loads. This is unavoidable. Therefore, in order to ensure the anchors are not overloaded under the gravity loads, it is recommended that all beams that transfer diaphragm forces to the lateral load resisting systems have enough anchors to achieve a minimum 25% partial composite action. When less anchors than this are provided, large deformations of the studs may occur under the gravity load case, inhibiting the ability of the beam to function as intended under lateral loading.

To Phi or not to Phi?

The nominal strength of the individual shear studs, Q_n, can be determined from the equations in the AISC *Specification*. The current shear connector strength equations generally are used as nominal strength equations in composite beam design where the anchors are part of a composite system; the ϕ and Ω come in later in the calculation of the beam flexural strength. However, when the shear stud strength is checked as a connection between the diaphragm and beam, a resistance (or safety) factor should be applied to the shear stud nominal strength. Based on preliminary results of ongoing research at the University of Illinois at Urbana-Champaign, ϕ = 0.65 (LRFD) or Ω = 2.30 (ASD) are recommended. These values are in line with similar recommendations by PCI and ACI.

Secondary Shears and Moments

Once the designer deals with the transfer of force from the floor diaphragm into the supporting steel beam, the effect of the diaphragm forces on the design of the beam and its connections to the remainder of the lateral load resisting system must be considered. Of particular concern is the effect of the vertical offset (eccentricity) between the plane of the diaphragm and the centerline of the supporting beam as indicated in Figure 5. Intuitively, one would anticipate additional moments imposed on the beam as a result of the eccentricity. However, this is not the case.

As an example, consider a simple-span beam with uniform horizontal shears from the lateral loads and resulting reactions as shown in Figure 5. For this scenario, assume the member is connected to the lateral load resisting system at the left end of the beam only.

The free body diagram in Figure 6 shows the internal member forces that result from this applied uniform load. The axial load in the beam will increase linearly toward the end of the beam designed to transfer the collected force to the lateral load resisting system, but the internal moment due to this applied lateral load, even considering the *d*/2 eccentricity, will be zero. The member should be designed as a beam-column, considering the combined effects of the axial forces due to the lateral loads and the flexural forces due only to the gravity loads. The shear is a constant value and must be considered in the connection design at both ends of the member.

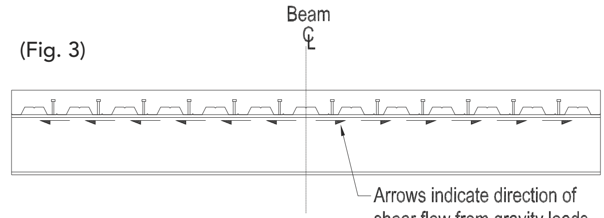
Force Interaction

The rigorous design of composite beams for combined axial force and flexure is complex. As a reasonable simplification for design purposes, it is acceptable to use the non-composite axial strength and the composite flexural strength in combination using the interaction equations in the AISC *Specification*, Chapter H. Note that for compressive loading, this type of composite beam-column is generally considered unbraced for buckling between braced points about the major axis, and fully braced by the composite diaphragm for buckling about the minor axis.

As with all structural systems, there is an element of engineering judgment involved in the proper design and detailing of horizontal diaphragms and composite beam interaction. Careful consideration should be made to provide a continuous load path. The designer must account for the required axial forces and shears to be transferred at the end connections of all beams. Though there are many aspects to consider for the design of composite beams subject to horizontal diaphragm forces as reviewed in this article, their implementation is straightforward, thus allowing the composite beams to be used as an economical and efficient component of the lateral force resisting system. MSC

Susan Burmeister is an associate with Cagley and Associates, Inc., Rockville, Md., and William P. Jacobs is a design engineer with SDL Structural Engineers, Atlanta.

(Fig. 3)



(Fig. 4)

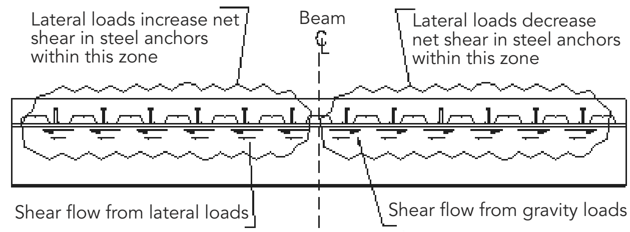
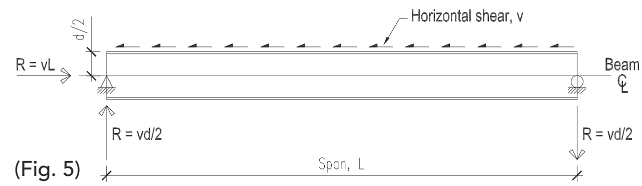
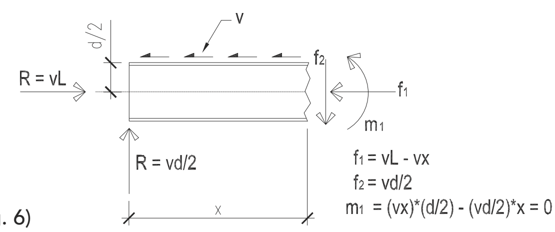


Fig. 3. Shear flow due to gravity loads only
Fig. 4. Shear flow due to gravity and lateral loads in combination



(Fig. 5)



(Fig. 6)

Fig. 5. Analytical model of simple span beam
Fig. 6. Free body diagram of beam segment

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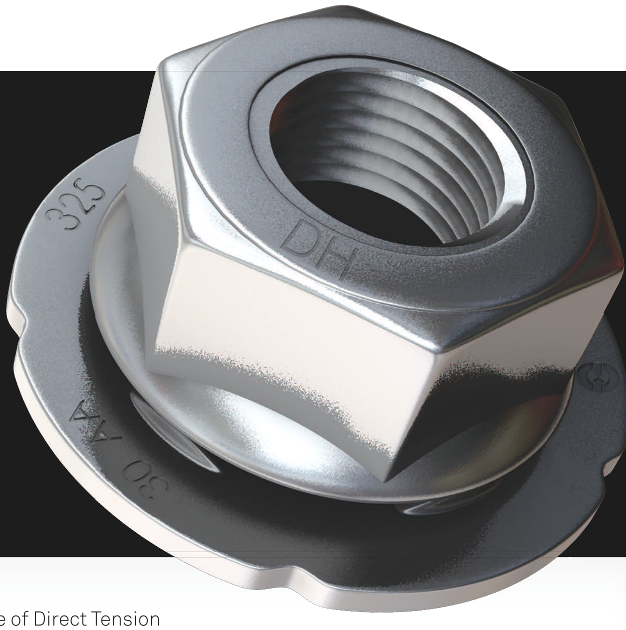

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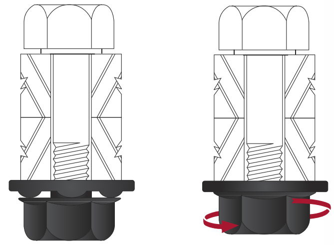

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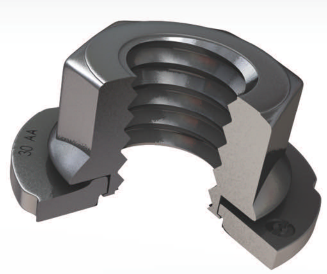

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A Long Fall

BY JOHN P. CROSS, P.E.

The question, in terms of the economy and specifically the construction market, is *how* long?

IF ANY PREDICTION OF THE FUTURE can be certain, it will be that the months of September and October of 2008 will be the focus of hundreds if not thousands of books and scholarly studies over the next fifty years.

A period of eight weeks witnessed the collapse of the investment banking system, an unprecedented tightening of credit, the failure or merger of dozens of banks, over 20% of residential properties being worth less than their underlying mortgages, wholesale intervention in the financial market by the federal government, lightning-fast action by a Congress known for inaction, record losses (and gains) on Wall Street, and the acknowledgment of the long-held suspicion that the United States economy had settled into a recession—all occurring in the middle of a contentious Presidential campaign and just in the United States. Globally, the old axiom that "when the United States sneezes the whole world catches cold" still seemed to hold, with plunging commodity prices led by a 50% drop in oil prices, even larger percentage drops on global financial markets, and the economically powerful countries rushing to prop up their own financial institutions and currencies.

The experts will debate the causes and ultimate results of the "Fall 2008 Fall" for years, but the more immediate question is how these events will impact domestic business conditions and, more specifically, the construction marketplace. The simple answer is that *if the rescue plans work*, construction volumes on a square-footage basis will continue their current decline through 2009, flattening in 2010 with no significant upturn expected until 2011. It would be easy to say that the current downturn is simply the result of tightening credit and the economic shocks experienced over the past two months, but reality paints a different picture.

Domestic non-residential and multi-story residential construction volume, measured in square footage, peaked in 2006 at 1.8 billion sq. ft, with 2007 down 2% compared to 2006. That decline has steepened in 2008 with a projected decrease of 14% compared to 2007. Construction economists vary on their projections for 2009, but most predictions seem to center around an additional loss of 10%, with 2010 remaining level with 2009. This would mean that construction levels in 2009 and 2010 would be in neighborhood of 1.35 billion sq. ft, a total decline of 25% compared to 2006 levels. Some of this reduction can be traced to the impact of an overbuilt single-family residential market and the delayed impact of that overbuilt market on building types that follow new residential developments such as retail, commercial, small office space, religious facilities, and elementary schools. Also contributing to the early stages of the slowdown was a tightening of lending standards that can be traced back to early 2007 that limited the level of speculative construction activity. Clearly, the majority of the impact is a result of an overall slowing of the domestic economy.

Short-Term Immunity

During 2007 and most of 2008 the structural steel industry seemed immune from the impact of the declining construction marketplace. Producers, service centers, and fabricators maintained active order books, attractive margins, and healthy backlogs. Several factors contributed to the overall heath of the industry, including the effectiveness of broad-based industry marketing efforts that raised the market share of structural steel by 7 points between 2000 and 2008; growth in the non-building demand for fabricated structural steel, particularly focused in the energy and process industries; and a weak dollar, discouraging imports of both mill and fabricated material.

However, the impact of the credit crisis and the continuing contraction of the construction marketplace will be felt by the structural steel industry over the coming months. Even when increases in market share are taken into account, the demand for fabricated structural steel for non-residential and multi-story residential buildings is projected to be down 15% from 2008 levels when building type and height are taken into account. Non-building construction may offset a limited portion of this loss, but those segments of the economy are being impacted by decreasing oil prices, challenging the financial viability of some energy projects and the lack of available credit. At the same time, the current strengthening of the dollar will encourage the growth of imports of both mill products and fabricated material.

The reality of this downturn is being demonstrated in a sharp reduction in the architectural billing index, reports of layoffs at structural engineering firms, a dramatic increase in the number of projects reported by AISC member fabricators being cancelled or placed on hold, an increase in the number of competing fabricators bidding on projects, a reduction in producer and service center orders, and a continuing decrease in construction employment.

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More than the Markets

But not all is gloom and doom. It is easy, in light of the press coverage of the current credit crisis and the loss of value of residences and investment accounts, to assume that this will be a long, deep trough and that it is the financial markets that drive construction activity. Construction activity is actually driven by a wide range of factors, and even though the lack of credit availability can temporarily choke off construction activity the demand for construction actually will help drive the economy into recovery. Several factors will drive a construction-based recovery.

Unlike the single-family residential market, non-residential construction is not in an overbuilt condition. Prior to the drying up of credit markets, there was a growing demand for non-residential construction based on declining vacancy levels and square footage rental rates increasing at a rate greater than the base level of inflation. This indicates that there will be a pent-up demand for new buildings that will accelerate the rate of recovery as the economy comes out of the recessionary period.

The fundamental relationship between population and building inventory, when taken with the average rate of building replacement, continues to indicate a long-term square-footage growth rate for construction of 3.5% between 2008 and 2030. While the construction marketplace will always experience cyclical growth cycles, the long-term prognosis is good.

Investment in Infrastructure

There is an increasing level of discussion in government circles of an infrastructure-based financial stimulus program as opposed to the earlier consumer-based stimulus program. While this program would primarily fund non-building-based transportation projects, funds may also be made available for institutional and public building construction. Past trends indicate that an increase in infrastructure funding will have a trickle-down effect in the building market. Certainly any governmental or private sector encouragement of energy projects, whether they are renewable or fossil fuel-based, will increase demand for fabricated structural steel.

Increasing emphasis on productivity improvements involving alternative project delivery methods, sustainability, lean construction, and building information modeling (BIM) will allow an overall reduction in project cost and a better platform for managing risk. Increasing productivity, reducing project costs, controlling risks, and the desire for high-performance, green buildings and the higher lease rates they command will result in more projects being financially viable increasing construction levels. The structural steel industry is already recognized as a leader in each of these developing trends.

At the same time, the drop in commodity prices will have a direct impact on construction materials and the cost of construction projects. Over the past several years, the increase in cost for new buildings has been nearly twice the increase in the consumer price index. The result was that fewer projects were financially justifiable. As commodity prices drop, construction costs may still increase, but that increase should be less than the base inflation rate, resulting in more projects being financially viable.

Ready, Willing, and Available

The availability of construction materials will remain high, with structural steel shapes been readily available in the marketplace either through service centers or directly from producers. Recent capacity expansions among structural steel producers will provide excellent availability of material as the market recovers.

And finally, it must be remembered that building construction is an integral part of the DNA of the United States. We are by our very nature designers, engineers, and builders. At the same time, Americans are creatures of habit. We may not build the same buildings we were building three years ago, but as a people we will continue to build. This point was driven home in a recent conversation I had with a salesperson who works in the paper bag industry. When asked how the economy was treating his business, the answer was surprising: Their sales are at record levels because sales of bags for french fries have skyrocketed. Americans are in the habit of eating out—only the places they eat out have changed. We may change what we build and even where we build it, but we will continue to build.

The Near Future

So what does 2009 hold for the design, construction, and structural steel industries?

- A decrease in design activity of about 7% on a dollar basis through the end of 2009
- A decrease in non-residential construction starts of 10% on a square-footage basis
- A decrease in multi-story residential construction of 4% on a square-footage basis
- An increased interest in the use of structural steel, based on opportunities for productivity enhancement through early fabricator involvement, lean construction techniques, focused on off-site fabrication, and increased use of BIM
- A recovery in the design industry during the second half of 2009, slightly after a recovery in the general economy
- A recovery in construction starts in the second half of 2010, lagging the design industry by about three quarters

Next quarter's topic (previously planned for this issue) will address the issue of achieving cost and schedule savings by implementing collaborative design methodologies. MSC

Stop Giving Cold Calls the Cold Shoulder

Strengthen your abilities to make initial calls to prospects.

BY ANNE SCARLETT

ASK A TECHNICAL PROFESSIONAL about their *least* favorite marketing task, and you'll find that cold-calling often tops the list. I can relate to why many of you put telephone prospecting at the bottom of your to-do list. Even with almost 20 years of warm-/cold-calling experience, I am still not a big fan.

But let's get something straight: You cannot use your technical background as an excuse for lackluster cold-calling abilities. I myself have an interior architecture degree, and my colleague John Ross, business development director at Affiliated Engineers, Inc., is an engineer—and we make cold calls. So, now that we've eliminated that excuse, let's take a look at some techniques I have compiled, with John's additional insights.

What's your hook?

Share your hook within the first 20 seconds of the call. "Wow" factors about your firm's performance may work, such as remarkable statistics relating to project results. But if they are not compelling or if you can't support your claims with facts, then go for a different type of attention-getter: Name a mutual colleague, reference an organization to which you are both involved, inquire about news regarding the contact's organization or industry, or make an observation on what you've read about the prospect from a reliable source, such as LinkedIn. John finds that mentioning a few highly respected existing clients seems to pique immediate interest. One of my own favorite hooks involves expressing my desire to hear the prospect's opinion on relevant subjects. People like to feel valued; they will likely engage in a comfortable rapport when they realize that you seek their perspective. Each call is different, so select a pre-developed hook in order to capture attention from the get-go, and brainstorm with colleagues about various hook options.

You've got their attention; what do you want from them?

Clearly state your purpose up-front. Time is tight, and meandering and mindless chit-chat will not be welcomed. This is not to say that you can't be playful and relaxed as the conversation evolves, but put yourself in their shoes; it's more palatable to have clarity on what someone wants from you than to be suspicious of hidden agendas. Further, by directly stating your intention, you will confirm whether you have reached the right person or if you need to speak with someone else. If you are passed along, keep track of everyone you spoke with so that you can map out an organizational chart. You never know when those other people may come in handy.

Match their communication style

It's important to match the delivery of the prospect, at least during the first couple of minutes. If they answer the phone in a curt manner, be somewhat brisk in your own response. Just last week I received a call from a prospect; she had found me on the web. Yes, it was easier because she initiated the call, but I still needed to be at my best in order to promote my services. Interestingly, I caught myself a couple of times showing an unbalanced level of energy compared to her tone and rate of speaking. While highly articulate and content-rich, she was soft-spoken in her delivery style. She was thoughtful, pensive. On the one hand, I wanted to demonstrate my high-energy personality, which would be fitting for what she needed (marketing training for her technical staff). On the other hand, I did not want to alienate her, even subconsciously, because of differing styles. After I hung up from that first 30-minute call, I vowed that during the next conversation I would be extra mindful to be in-line with her. Sure enough, our next call felt much more balanced. (And incidentally, I was awarded her business!)

Agree on the next step

I personally like to agree upon a next step fairly quickly within the call and then sign off. Conversely, some business development colleagues see value in building a longer rapport by phone. Either way, never conclude a call without agreeing on a next step. For me, my next desired step is typically to schedule a face-to-face meeting, although there are times when I'm simply calling to remain top-of-mind. When possible, keep the ball in your court. If you agree to send information, state that you'll follow up within a specified time frame. John likes to send a subtle "no-pressure" message after an initial call. When the prospect asks, "Why don't you check in with me again in a couple of

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SCARLETT CONSULTING

months?" John will suggest a date that is about four months out. Often, the client will back it up and say specifically that they'd like to hear from him again in one to two months.

Show respect; ask about a preferred form of contact

First, get permission to follow up by e-mail immediately after the call so that they will have all your contact information. (Make sure your e-mail signature includes your mailing address, web site, and direct phone number.) Then, ask if they prefer to correspond via e-mail or by phone. Some people have strong opinions one way or another; they will be appreciative when you respect their preferences.

Voicemails: to leave or not to leave

Many people suggest avoiding voicemail messages. Instead, try to obtain the direct dial digits and attempt calls until you connect. Many leaders, aka decision-makers, have Type-A personalities, so you may also want to try them on off-hours such as early mornings or weekends.

My approach is different. I actually prefer to leave a voicemail as a first form of phone contact. Here's why: A well-executed voicemail allows you to express your point with succinct confidence, minus any sense of resistance from the other end of the line. By practicing your voicemail delivery in advance to ensure an interesting, appropriately dynamic delivery, you can be sure that your message will be heard fully. At the end of the message, recap the nature of your call, repeat your full name, and state your phone number twice. Although you may not receive a return call, take comfort in knowing that when you call back again—don't leave another message until you reach the person live—the contact will already have a cursory knowledge of who you are and what you want with them. In my view, leaving a good voicemail can work just like a direct mail piece because it prompts a familiarity with your brand. Take note: Sometimes you will be able to redo your message. Every phone system is different, but often if you hit the pound sign you will have an opportunity to listen to—and re-record—your message. Once you figure out the coding for the prospect's phone system, include it within your calling notes for future reference.

As you can see, there are varying views on the "right" way to successfully execute cold/warm calls. What's most important is that you find your own right way. As you hit upon success—no matter how large (winning a new project) or how small (securing a meeting)—you will be on the way to becoming a skilled caller. MSC

*For more advice from Anne on cold calls, see her article "Warm up to cold calls; Reassess the value of telephone prospecting" at **www.annescarlett.com**.*

ON STEEL
Arnold

STARTING OUT "I wanted to be an architect. I started working in an engineer's office and enjoyed the speed at which projects went through my office. There were 10 projects on and off my desk in a week, or sometimes in a day. Engineering is very exciting. Architects make a building beautiful and interesting, but engineers are the people who make them stand up."

Barry Arnold. Principal. Vice President. ARW Engineers, Ogden, Utah. Started his career with ARW in 1985 as a drafter. Received master's degree in engineering in 1991. Received the 2007 Engineer of the Year award from Utah Engineers Council. Loves nature for its structures. Uses steel to create what he sees.

FLEXIBILITY "ARW works on a large variety of projects, but my greatest interest and affection is in steel design. Every designer finds that, despite their best efforts, no project can be perfect; problems happen. Steel provides the simplest and easiest solutions to fix any problem. With steel there's always an easy solution to any problem. If a beam's a little short, you can weld something on. If you need to move a column 10 feet, with steel, it's easy. If your concrete beam is short or a column needs to be moved – you've got a big problem with no easy solution. Steel keeps the projects flowing and going, no matter what type of building it is. I'm happiest when I'm designing in steel… Steel is not nearly as frustrating as other materials – there's always an economical solution in steel."

GREEN "My love of steel wasn't a huge epiphany, it was a growing appreciation of its characteristics and qualities, you know, the nature thing. It only takes working on one or two projects in other materials to make you wish you were designing in steel. You know the design would have been so much easier with steel; it's just so much more predictable. Steel allows for expression in combination with simplicity of design. If an owner is thinking long-term about the environment and building flexibility, steel's the only answer. With everything going green, steel is a natural choice because it's revered as a recyclable material. LEED® is making an impact now, and in years to come, it will be a significant driving factor. With steel, it's easy to make LEED points and points with your clients."

LEARNING "The inspiration I get personally comes from when I attend AISC seminars or go to AISC conferences. There's a plethora of new ideas and innovation available through AISC. Information is presented in a neat, orderly format. You can come back to your office and use the ideas and information immediately. It's always applicable to the projects you're working on today. AISC gives you all the backup and support you need. If you ask a question, AISC responds very quickly."

TEAMWORK "Teamwork is very important… Engineers can be very opinionated. If you ask 20 engineers how to solve a problem, you'll get 20 different answers and that's a good thing. They're all slightly different answers, but they're all correct. You have to keep options open. We tend to gravitate toward what we've done before and many times, that turns out to be a solution that includes steel.

Everyone has that 'manual' in their head of how to do things and that's okay. The young engineers like to test the old engineers as much as the old engineers like to test them, but one thing we all seem to end up having in common is a deep appreciation for what steel can do that other materials can't. We review lessons learned on projects weekly in our office. Everybody has a say. We talk freely and openly without egos getting in the way. We're one unified company, with 20 different people thinking about the options. You get to pick one answer. And most of the time the answer you pick will center around steel and its seemingly unlimited capabilities."

PRIDE "I have no dreams about a special project that I'd like to do one day. I've devoted myself to being proud of every single job I worked on – regardless of whether it's big or small, or designing the whole building or a few connections. I do what needs to be done every day. I don't put my professional ego on display and say, 'look at all these buildings we've done.'"

INSPIRATION "I have a huge appreciation for the environment. In fact, I can see the Wasatch Mountain Range from my office. Being outdoors helps me appreciate my responsibility and obligation to future generations. Engineers have an ethical obligation to protect our natural resources; it's your way of contributing to all of mankind. My work directly affects the environment – I'm humbled by that fact. I find inspiration when I'm in our National Parks and take in the grandeur and majesty of it all, and understand that we all have an obligation to preserve these spaces and our resources for future generations. You get a much bigger perspective out there."

STEEL "If you look at modern steel construction, you will see some exciting innovation going on. I've seen a lot in my career but I know the best is yet to come. Steel is like a good friend – reliable, strong, tested and trustworthy – that has supported me, as I interpreted the architects' concepts to make their dream a reality. That's what young engineers really need to know. That's the power of steel."

Correcting Your Corrective Actions

BY DAN KAUFMAN AND SHEILA ALEGRIA

Knowing what a corrective action is and when to employ one can help you tighten up your quality management system and use it to its full potential.

IF YOU'RE ONE OF AISC'S CERTIFIED FABRICATORS, there are two kinds of corrective actions that apply to you. As a fabricator, you will write a corrective action when you find a problem that needs to be addressed in your system; this is the first type. The second is an AISC Audit corrective action, which is, as the name implies, potentially written as a result of an AISC/QMC certification audit.

The first one is the most important, as it leads to problem resolution within your own organization and results in documentation that shows that your quality system is working. It also shows that management is actively involved in the quality system. The AISC audit corrective action, on the other hand, might happen only once a year as a result of the "snapshot" view of your company taken by the onsite auditor on an AISC certification audit. While an audit corrective action should not be alarming, it does indicate that an outside entity was required to point out a problem in your system. Bu while AISC is an outside entity, having the issues come up during the AISC audit is better than having an issue arise for one of your customers and possibly lower *their* quality evaluation of your company.

Annually, we compile the statistics of the audit corrective actions written by the QMC auditors and publish the results in *Modern Steel Construction*. These statistics show the summarized reasons for the corrective actions, hopefully giving fabricators a look at where typical soft spots are found. Fabricators looking to become certified probably gain the most from the summary, as it points to where they can concentrate their efforts in creating a good quality management system. (That's a *management* system that is of higher quality, not a system limited to only product quality.)

Dan Kaufman is Manager of Operations and Sheila Alegria is Client Services Coordinator with Quality Management Company.

Every year with this analysis, it seems apparent that there are some fabricators who are not using their internal corrective action systems correctly, and therefore are not receiving the full benefit of the system. Errors in executing corrective actions typically start with a nonconformance—or according to the American Society for Quality (ASQ), a "nonconformity." Nonconformities are typically discovered in the inspection process. Some fabricators believe that any repair of a nonconforming piece is a "corrective action." It is not. A repair of a nonconforming part is, very simply, a repair. When a situation involving nonconforming pieces or processes occurs, a determination must be made to decide if the event is serious enough to warrant a bona fide corrective action. For example, a clip angle placed off the mark can be removed and replaced easily. Yes, time was lost, but a review of inspection records and nonconformance summaries can show how often it is happening. Similarly, if clips have to be removed and replaced several times a day, this is a situation that deserves some investigation. It is the investigation, solution, and implementation of the solution that comprise the corrective action process.

Another example might be a case where the most expensive piece ever handled in the shop had to be scrapped because it was cut too short. It's only one piece but it's a high cost, and nobody wants that to happen again, not even once. The repair of the piece is still not the corrective action. Again, investigation of the error, finding a solution to prevent that error from happening again, and the successful implementation of the solution are what make up the corrective action.

Making the decision when to initiate the corrective action process is not always clear. Remember that a procedure can be changed if it isn't working. If you are spending too much time investigating simple problems, or not doing any investigations, a change in the plan is indicated, although neither of those situations is desirable. Looking at a record of past nonconformances or inspection findings will normally indicate a pattern of problems. A review of that pattern will show problems that can be considered low-, medium-, or high-impact. You will also see a frequency of problems—again low-, medium-, or high-frequency. A simple chart (following page) can be made to easily show how events can be grouped, and limits chosen indicating when to trigger corrective actions. In this example, the green area—low-frequency and low-impact—does not warrant a corrective action. However, the red zones, such as high-impact situations—like losing a customer—with varying frequency should trigger a correc-

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**.

		Frequency	
Impact	Low	Medium	High
High	Replacing a customer		
Medium			
Low	Replacing a clip		Replacing 10 clips a day

tive action. Likewise, if a low-impact problem reaches a high frequency, then it will merit additional attention as well.

When to Implement a Corrective Action

Another source for an internal corrective action is a concern written during an AISC certification audit. These concerns are normally described during the closing meeting of the audit and recorded in the audit report; it is expected that certified companies use their own system to address the concerns. These issues can be handled internally, without becoming AISC audit corrective actions the following year.

One more source for internal corrective actions can be a process or procedure failure. If, during your internal audit, you find that a procedure is not being followed—or a procedure is actually causing problems—that issue can be documented and resolved by the use of a corrective action.

When a decision is made to implement a corrective action, the root cause of the problem must be found. A very frequently applied "root cause" is to train or retrain employees. While training is valid on occasion, it should not be over-used. If you find your organization using this excuse too often, you are dealing with a symptom. The disease is the fact that there are so many employees needing so much training! Either the training isn't any good, or employees are being placed where they shouldn't be working. In some locations employee skill levels are not as abundant as the market requires. Your cure will have to include a really good trainer.

Chasing a root cause can be daunting, but it can be eased by employing a root cause analysis model. This may sound complicated but it doesn't have to be. It also can be a one-time effort. One model used widely is the "Five Whys." Start with the first response to "why" a nonconformity happened, and when an answer is given ask "why" again. This will likely annoy the life out of the recipient of the questions, but it's a good way to drill down to a root cause. Categorization of possible causes by personnel, machine, materials, or methods can help narrow down the possible causes. Pick a method and make it your "official" root cause analysis plan. When everybody knows what is expected in root cause analysis, it will happen faster. There are many books and Internet articles on root cause analysis. (There are also lots of consultants ready to tell you that they are the only ones qualified to show them to you.) There is free help at **www.qmconline.com** and also at **www.asq.org**.

Using a corrective action system is a very important part of your management system. Not performing corrective actions is like announcing that key individuals don't believe in using quality tools to make your business better. Knowing how—and when—to perform a corrective action is good, but even more important is the will to put the system into action. MSC

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

Modular and Movable

The QX Modular Crane Package by R&M Materials Handling offers a pre-engineered, modular crane package intended to maximize performance. Designed to expedite simple assembly, ease of operation, and reduced maintenance, the QX is the perfect solution for most industrial crane applications. It simplifies assembly and ensures operational safety. Plug-in electrical cabling (available for most packages) facilitates simple, fast installation and allows you to control movements from a convenient push-button station. The package comes with standard safety features, including an electro-mechanical weight overload limit switch that prevents lifting loads in excess of 110% of the rated capacity. It also includes end trucks capable for CMAA Class C or Class D crane duty. The modular design of the bridge drives includes a gearbox, motor, and brake specially designed for crane applications. The bridge controls feature a standard inverter that offers two-speed or two-step infinitely variable speed control. Additionally, the crane includes festoon systems, equipped with CSA-approved PVC flat cables, and plug connections on cables are available for most packages.

For more information, visit www.rmhoist.com or call 800.955.9967.

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

Connecting Analysis and AutoCAD

Structural Desktop provides a simple connection between analysis programs and AutoCAD drawings, which can be used as the basis for analytical models. Users can quickly assign member sizes, types, and materials to these members, then export an analytical file with member and element information intact. The software package includes definitions of light-gage members, joists, joist girders, and double-tee concrete panels that can be used with your analytical file and in production of 2D drawings, 3D models, and material reports. In addition, it does not require the engineer to learn AutoCAD to produce the major portion of his or her drawings. It draws precast concrete sections, AISC sections, LGSI and AISI light-gage sections, steel joists, wooden beams and studs, and others. It creates solid models for rendering and 2D drawings for contract documents that are 100% compatible with AutoCAD.

For more information, visit www.structuraldesktop.com or call 866.269.8394

CSI Software

Computers and Structures, Inc. has released SAP2000, version 12. SAP2000 users who have a current SUM (Support, Upgrade, and Maintenance) agreement are eligible to receive the upgrade to version 12 free of charge. Special notice for SAP2000 version 9 users: A special discounted price is being offered to all users of version 9.xx to upgrade to version 12.

For more information, visit www.csiberkeley.com.

Story begins on page 78.

My next contention is that the image of past generations of engineers working exclusively in an "ivory tower" of first principles must be a myth. Automation and "shortcut" methods have been part and parcel of structural engineering for more than a century, and the computer is no more inherently evil than its predecessors. In his excellent, thoughtful essay "Don't Blame the Computer for Mistakes!" Bashar Altabba "vividly remembers the days when similar arguments [about computers] were being made about handheld calculators, back when these were first introduced. Some schools even banned their use… At that time, the proposed solution for complex calculations was a simple one: Just use a slide rule like 'real engineers' do! Does anyone today still hold this view about handheld calculators?"

Was not the slide rule itself introduced to cut down longer pure-hand calculations?

I do not deny that over-reliance on computer output without proper care and consideration (such as a senior engineer checking the results) can be detrimental, even gravely so. However, I do question whether design automation is truly an entirely new problem. One of my superiors says that no computer program, ultimately, is anything more than a "glorified spreadsheet."

I next take issue with our elders' fear of declining competence and intelligence in young engineers. I will not argue against individual anecdotal claims such as, "I have seen engineers with eight or more years of experience with no engineering intuition or common sense."

There are, have been, and will be good engineers and bad engineers, just as there are, have been, and will be good doctors and bad doctors, good lawyers and bad lawyers. And like any other business, the engineering "org" chart is a triangle with few at the top and many at the bottom. I'm sure the harbingers of doom know at least a handful of good young eggs, and might these be the few to ultimately succeed those at the top? (And isn't that the way it has always been?) The senior people at my firm think of recent graduates as apprentices, with the idea that one's first office has the obligation to provide that link between the university and the workplace.

On a deep philosophical level, it is not surprising for our elder engineers to fear the future. One professor at the University of Buffalo has noted that "it is natural that older engineers have a lack of confidence in younger engineers." Ours is a serious and difficult profession to protect both the public's safety and the client's money; this responsibility should instill a sense of pride and self-confidence. Like King Lear, we want to see our realm passed on to proper hands, and we hope for a brighter outcome than he found. It is easy to fear that one's successors may be unprepared if they do not follow exactly in one's footsteps. However, difference does not imply inferiority.

What Should We Do?

Many have offered solutions, perhaps the most well-defined being ASCE's policy 465, which proposes to expand and deepen civil engineering education at the university level. In theory this will bolster the engineering student's body of knowledge to a level certainly not yet on par with, but closer to that of, a medical or law student.

While there are countless outstanding engineers who never pursued a master's degree (as well as the inverse) there may be no tangible way to demonstrate to the lay public the educational rigor of the engineering profession (besides drastic salary increases) without raising the bar of degree attainment. I support the policy 465 initiative.

Previously I touched on the rise of the unconventional, computer-enabled, "funky" architectural schemes with which we structural engineers are compelled to work. I contend that structural engineers must "take back the funk." We must lead in this geometric revolution, on equal if not superior footing to the starchitects, because we ultimately hold the keys to the realities of strength and stability. Are the works of Frank Gehry and Zaha Hadid any more inspired than those of Eduardo Torroja and Eladio Dieste? Besides Santiago Calatrava's projects, I fear that engineers have fallen into the complacency of merely reacting to the architects' dreams, while it rarely occurs to us to have the dream first.

Another suggestion, made publicly by NCSEA President Ed Huston, is to dig up—out of books, notes, and individual experience—all the "rules of thumb" and "reality checks" engineers have acquired over the years and circulate them among peers both young and old. I agree with this sentiment. No matter how complicated an analysis becomes, it is practically guaranteed that at some point in the process you will need to "prove" your design succinctly, in the space of a single page, to someone—a client, a colleague, a contractor, a senior or junior coworker, or, above all, your own conscience.

I encourage employers to ponder the true nature of our profession. Does anyone really start with intuition, or is this cultivated slowly over time? Is the computer really evil, or does it in fact *help* the engineer develop understanding because it challenges one's conventional thinking? Should an engineering firm be a hierarchy of those who "have" knowledge and those who simply run simulations, or should it be a place of continuing education between masters and apprentices? Even if we do "clean our own house," how do we deal with architects who produce designs in CAD that cannot be built, and construction managers who churn out schedules from Primavera without any intuition of their own? And what about the declining fees for our services?

As a final illustration, let us recall the story about William LeMessurier re-analyzing the entire Citicorp Building by himself in a cabin during the post-construction crisis concerning bolted versus welded connections, under the previously unconsidered effects of quartering winds. While his application of first principles in hand calculation is magnificent, the more important moral of this story is LeMessurier's global thinking, humanistic conscience, creative problem solving, and having the right priorities.

Let us too have the right priorities. Look inward and march forward! MSC

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Are Young Engineers Unprepared? A Young Engineer Answers

BY EYTAN SOLOMON, P.E., LEED AP

A critical look at the concerns of our elders.

DEBATE OVER THE perceived inadequacies of structural engineering graduates has reached a fever pitch. Some say that young engineers today are not technically competent, that they have no engineering judgment or intuition, and that these deficiencies will manifest through poor designs into an increase in structural failures and collapses. To these ominous claims I offer not quite a rebuttal, but a reasonable continuation of the discussion from the viewpoint of that brash young engineer whom everyone fears.

How Have Things Changed?

Few will deny that the engineer of today is faced with more information than 100, 50, or even five years ago. High-strength steel, high-strength concrete, prestressed concrete, fiber-reinforced concrete, and structural glass are just a *few* of the new construction materials of our generation; finite element analysis (FEA), building information modeling (BIM), and sustainability are just a *few* of the new design paradigms; and globalization, intelligent technology, and digital fabrication are just a *few* of the new industry standards.

Similarly, the codified laws by which we create structures have also expanded and sharpened. The bureaucratic, legalistic, rule-fixated demeanor of our society—a character that does not necessarily yield negative results—has given rise to building codes and design guidelines that are voluminous and complex without precedent.

Clearly, structure geometries today are more complicated than before. Increasing sophistication in computer hardware and software is both a cause and effect of the ever-more "funky" designs that come across our desks. I once sat with the president of my firm, an engineer who has seen it all in his 50-odd years of practice, to look over the latest fantastical proposition from a certain "starchitect." "Why do they want to do this?" he implored sincerely before we both realized the answer: "Because they can."

In the Frank Gehry age of architecture, it is impossible to design many buildings without a computer and, in fact, it is impractical without a tremendous reliance on computer analysis. We feel sorry for our architect friends who log endless hours on AutoCAD, but many engineering students come out of school to work as "desk monkeys" on Revit, RISA, or SAP models for geometrically complex projects. How much "intuition" can one really attain in such an assembly-line environment? Does this inherently cause a disconnect between the "first principles" learned in school versus a young engineer's day-to-day practice?

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When our elders went to school, the truss and beam designs of steel and concrete were perhaps closer to what they would actually work with after graduation. Now the 3D modeling program is absolutely essential to an engineer's ability to analyze and design complex structures efficiently and is very often linked directly with drawing production and construction logistics as well.

With the increased complexity in materials, codes, and geometries, engineering educators find themselves scrambling to catch up with the pace of industry, while at the same time struggling to retain the fundamental courses in mechanics, analysis, and design. A special education committee for ASCE recently noted that civil engineering students today, on average, earn at least 20 fewer credits—including 18 fewer credits for engineering topics—than did their counterparts in the 1920s.

Today's engineering schools must, out of necessity, adapt to the times. Many offer classes with more direct preparation for industry practice, such as computer design and drafting or group work and project presentations. Some schools have increased the time to complete the engineering degree from four years to five years. And some programs, though they must compose a minority, have resisted additional computer-oriented courses so that the undergraduate curriculum can concentrate on fundamentals of analysis and design.

How Have Things Stayed the Same?

Despite all of the new challenges, I contend that the same timeless principles of engineering, experience, and management apply as much to our generation as ever before. Some structural engineers young and old hold a preconceived notion that the way to gain engineering judgment is by performing long hours of calculations by hand. But while the ability to do hand calculations is undeniably important, it is equally necessary to cultivate engineering judgment and intuition by walking construction sites, arguing with—and teaching—architects, hearing war stories from contractors and older engineers, and seeing how project after project is "solved" with different materials. A legendary professor at Columbia University once said, "The best engineer is the one with grease under his fingernails." With a constant objective of educating oneself, every moment of every day can be a learning experience.

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