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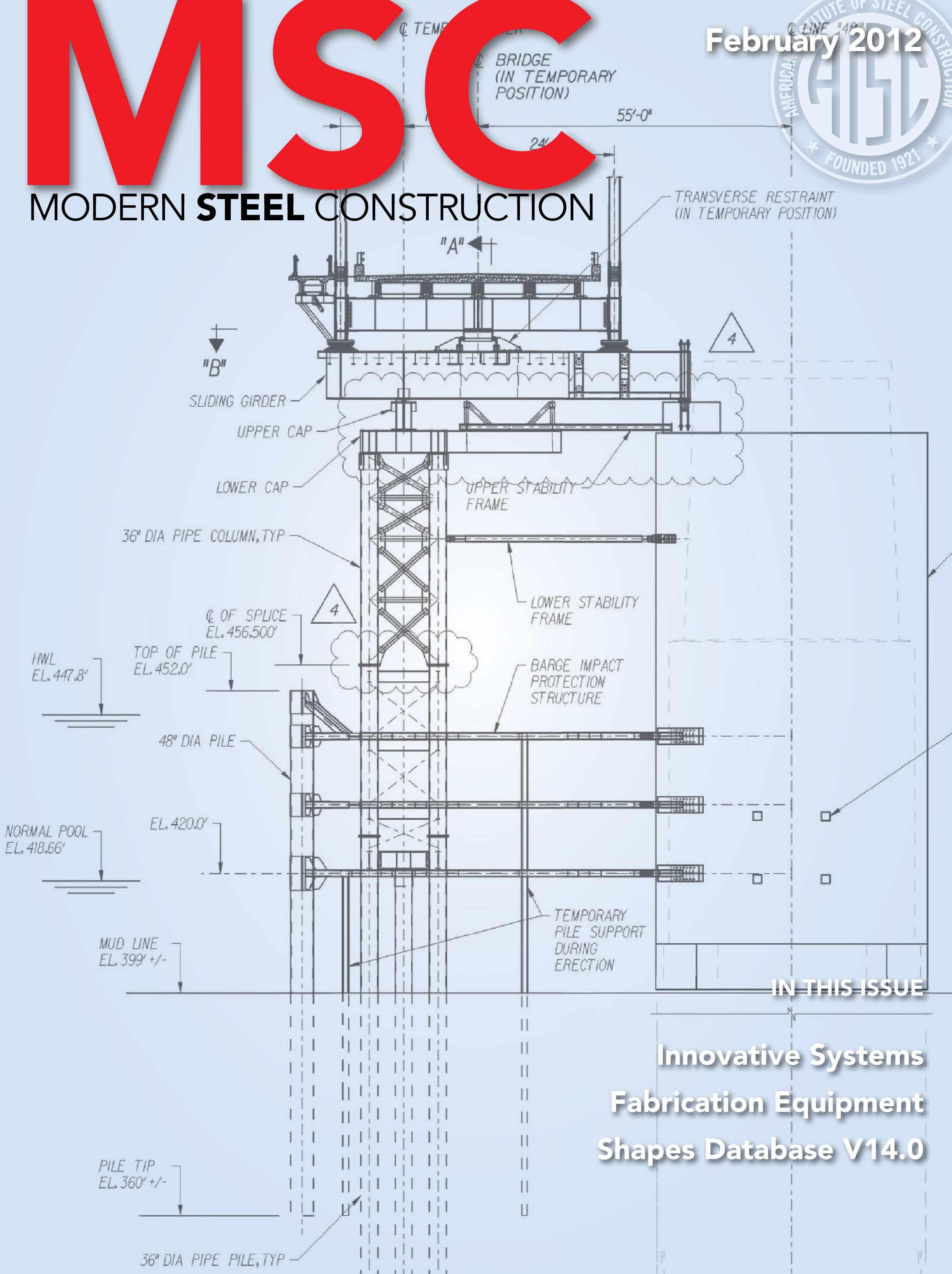
MODERN **STEEL** CONSTRUCTION

February 2012



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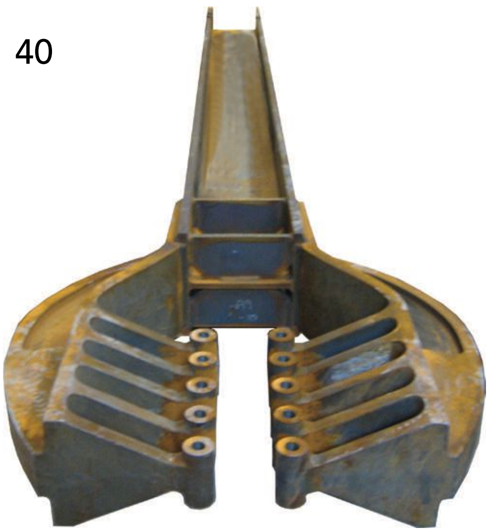

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ON THE COVER: One of the temporary piers for the Milton Madison Bridge replacement project, courtesy of Buckland & Taylor Ltd.

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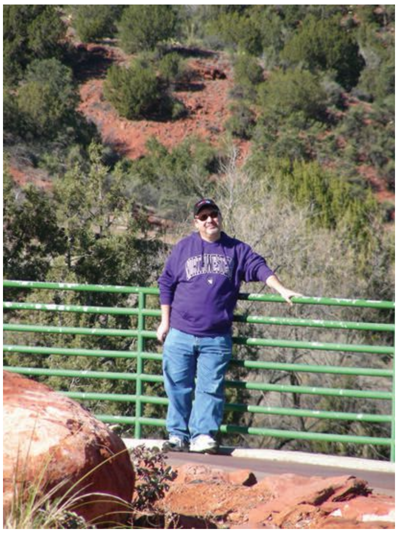
editor's note



I JUST RETURNED FROM AN ABSOLUTELY fabulous vacation—and it wasn't just that the January sunshine in Arizona completely trumps the snow in Chicago.

The vacation started, as so many do, at the airport. The flight was typical and we ended the first leg of our journey at Enterprise Car Rental, which had the lowest fees of any rental company I could find online. As we approached the rental booth, two employees came up to us and asked if we would like some water. The day was already warm and as we nodded our appreciation, they presented us (gratis) with five bottles of icy cold water. One of the men looked at our contract and then escorted us to the car area. Given the amount of driving we anticipated and the number of passengers (somehow three kids take up more room than any number of adults), I had requested a minivan. Amazingly, we were given our choice of three—a Sienna, a Grand Caravan and a Town and Country.

What a wonderful experience! The low-cost provider didn't simply provide the minimum required; rather, they provided an exceptional level of customer service. (Ask yourself what reaction a customer or client has when they first interact with your business. Do you provide a level of service so superior that they not only want to use your services again, but they also tell everyone they know about you?)

As great as the trip began, it only got better. Our hotel was beautiful, our drive to Sedona spectacular. But the highlight—for me and my entire family—was a visit to Canyon Creek Ranch. Our guide, Joe, was the perfect host. He made us feel comfortable on our horses, entertained with his tails and explanations of the trails and scenery, and taught us skills ranging from roping to shooting (though we were all extremely nervous to put a loaded gun in the hand of our rambunctious nine-year-old). Looking back at the experience, it was hard to figure out what made it so perfect. The ride wasn't much different than one my wife and I had taken in Colorado a few years earlier. The difference, I think, was the employees. Whether we were asking about helmets for our kids or laughing at my attempts at axe throwing, everyone made us feel welcome and comfortable.

In contrast, I knew my vacation was over as soon as I got home. While my wife and I started unpacking, one of the kids turned on the TV. It didn't work. I called Comcast and couldn't get a live person but did reach their automated system. And fairly quickly, I resolved the problem. Within 20 minutes, TV and internet service was restored. But I wanted to know why the problem had occurred and I couldn't get an answer. While Comcast resolved the problem, they left me irritated and considering alternatives to their service. Yes, they were competent. But they were not enjoyable to deal with.

If you read anything by Malcolm Gladwell or his ilk and think about their message of what makes a great company great, it really boils down to the corporate culture and the employees. And it all deals with how the company relates to its customers and clients.

What experience do your clients have when they work with your company? How do your customers view your company? Are you Joe…or are you Comcast?

(By the way, if you haven't registered yet for this year's NASCC: The Steel Conference, what are you waiting for? The registration fee goes up each week! So visit **www.aisc.org/nascc** and register today.)

Scott Melnick

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

steel interchange

Application of Q_f

When the variable Q_f is used in AISC 360-10 Section K1, it is typically applied as a multiplier outside of the bracketed portion of the equation. As such, it has an effect on the entire nominal strength calculation. However, Equations K1-12 and K1-13 for the limit state of wall plastification have Q_f inside the bracketed portion of the equation. Is it correct for these two limit states that Q_f only applies to a portion of the nominal strength equation?

I actually had the same question when I was reviewing this section of the AISC *Specification*. Q_f is in the correct location (inside the brackets) in 2010 AISC *Specification* Equations K1-12 and K1-13. The equation is based on a yield line approach. Because the force in the member has a greater effect on the strength of the yield lines transverse to the chord axis and little effect on the yield lines parallel to it, Q_f is applied to only those portions where it has a significant effect. This approach predicts a strength consistent with test results.

Larry S. Muir, P.E.

Weld Access Holes

It seems that the industry standard for end-plate moment connections is to not have weld access holes at the beam flange to end-plate CJP groove welds. This is different from the directly welded flange moment connection, which requires a weld access hole for the beam flange CJP groove welds. Why are weld access holes not used in end-plate moment connections?

The behavior of bolted end-plate connections has been shown to differ from directly welded flange connections in physical tests and in finite element models. In end-plate connections, the flange force is partially resisted by the bolts inside the flange. The weld access hole interrupts the stress flow to these bolts and causes a stress-riser that promotes flange fracture early in the inelastic range. Accordingly, weld access holes are not recommended in these types of connections.

This is briefly mentioned in the AISC *Manual* in Part 12 on end-plate moment connections. It states, "As reported by Meng and Murray (1997), use of weld access holes can result in beam flange cracking. If CJP welds are used, the weld cannot be inspected over the web; however, because this location is a relative 'soft' spot in the connection, it is of no concern." To quickly find this reference online, or any of the others in the 14th Edition *Manual*, go to **www.aisc.org/manual14** and follow the link to the Interactive Reference List.

Heath Mitchell, S.E., P.E.

Single Angles in Compression

Do the effective slenderness ratio provisions of AISC 360-10 Section E5 apply to the design of a concentrically loaded, compact single angle?

No, the slenderness modifications only apply to eccentrically loaded single angles that meet the specific criteria outlined in Section E5. The charging language of AISC 360 Section E5, states, "The *nominal compressive strength*, P_n, of single angle members shall be determined in accordance with Section E3 or Section E7, as appropriate, for axially loaded members. For single angles with $b/t > 20$, Section E4 shall be used. Members meeting the criteria imposed in Section E5(a) or E5(b) are permitted to be designed as axially loaded members using the specified effective slenderness ratio, KL/r." In other words, the slenderness modifications in Section E5 allow the eccentricity to be neglected in some eccentrically loaded single angles by designing them as axially loaded members with an effective slenderness ratio.

Heath Mitchell, S.E., P.E.

Extended Shear Tab Design

Consider a beam-to-girder extended single-plate connection that has a depth equal to the full-depth between, and is welded to, the flanges of the supporting girder. Is the eccentricity used in designing the bolt group taken as the distance from the bolt line to the beam web or can it be taken from the bolt line to where the single plate has full support (supporting beam flange tips)?

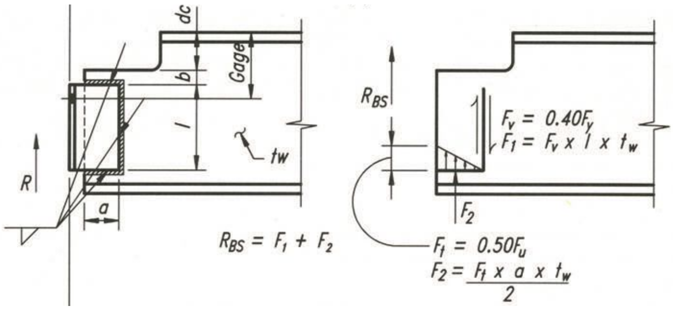
The procedure provided in the 14th Edition AISC *Steel Construction Manual* assumes an eccentricity on the bolt group from the face of the support to the center of the bolt group. The *Manual* also specifically allows the use of other rational methods. One of these might be to consider the "balancing" effect of having connections to both sides of the girder web. However, if this is done only the persistent dead loads should be considered when evaluating the countering effects of the additional connection.

If there is a connection to only one side of the girder, this is a difficult matter. Because wide-flange sections are generally weak in torsion, it would not be advisable to assume that the beam itself resists any of the moment caused by the eccentricity. This is essentially what you would be doing if you were to take the eccentricity from the toe of the support beam flange to the bolt group.

Larry S. Muir, P.E.

Block Shear of Welded Single-Angle Connection

What shear area should be used when checking the block shear strength of a top coped beam with a welded clip angle connection? The most recent design guidance I have found is in the AISC *Manual – Volume II Connections, ASD 9th Edition and LRFD 1st Edition*, which includes Example 9 and Figure 3-23 (see below). This shows that the shear area is the length of the connection angle multiplied by the beam web thickness and the portion of exposed beam web above the connection is ignored. Is it an AISC *Specification* requirement that the portion of exposed beam web beyond the connection angle be ignored in the block shear strength calculation?



I do not think it was ever AISC's intent to limit the length of the shear area to the length of the welded clip. This certainly does not need to be done to satisfy the AISC *Specification*.

It is sometimes convenient to make this kind of conservative assumption when the vertical location of the clip angle relative to the cope is not known or could vary, such as when the cope depth varies but the punch down remains constant. In practice, assuming the length of the shear area equal to the length of the clip would allow a general calculation to be conservatively applied over a range of connections, only performing a more exact calculation where the strength predicted from the conservative length does not satisfy the contract loads.

The full available dimension to the cope can be used in the calculation.

Larry S. Muir, P.E.

Correction of Errors

What is the intent of "moderate amounts of reaming" as used in 2010 AISC *Code of Standard Practice* Section 7.14? Does it mean that the erector should expect to ream at every connection?

The Commentary to AISC *Code* Section 7.14 states, "As used in this Section, the term 'moderate' refers to the amount of reaming, grinding, welding or cutting that must be done on the project as a whole, not the amount that is required at an individual location. It is not intended to address limitations on the amount of material that is removed by reaming at an individual bolt hole, for example, which is limited by the bolt-hole size and tolerance requirements in the AISC and RCSC *Specifications*."

In other words, it is not intended to mean that the erector should expect a moderate amount of reaming on each hole or connection. Rather, the erector should expect that some reaming may be necessary on the project. The need to correct minor misfits should be the exception, not the norm. In addition, the RCSC *Specification* tolerances on bolt holes still apply to the final, reamed hole.

Heath Mitchell, S.E., P.E.

T-1 Steel

I am doing research on bridges built prior to 1960. Does "T-1" steel fall under the ASTM A7 standard?

No. T-1 was the trademarked name used by U.S. Steel for high-strength, quenched and tempered 100 ksi steel, which is not the same material as ASTM A7. The product that U.S. Steel called T-1 is currently covered by a variety of similar standards: AASHTO M270 Grade 100, ASTM A514 or A517, and ASTM A709 HPS 100.

AISC *Steel Design Guide No. 15, AISC Rehabilitation and Retrofit Guide*, is a reference for historic shapes and specifications. It is available as a free download for AISC members, and for purchase by others, at **www.aisc.org/dg**. Table 1.1a lists the historic specifications. You will see the history of A7 and other steels as well as their yield and tensile properties.

Erin Criste, LEED GA

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**.

Heath Mitchell is director of technical assistance and Erin Criste is staff engineer, technical assistant at AISC. Larry Muir is a consultant to AISC.

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

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

Many of the answers to this month's steel quiz can be found in the AISC *Specification*, *Seismic Provisions* and *Prequalified Connections*, all of which are available as free downloads at **www.aisc.org/epubs**.

1 What slip coefficients for Class A and Class B surfaces, respectively, are recognized by the 2010 AISC *Specification*?
a) 0.20 and 0.40 b) 0.30 and 0.50
c) 0.33 and 0.50 d) 0.35 and 0.50

2 True/False: The minimum radius for bent plates differs depending on whether the bend is perpendicular to or parallel to the direction of final rolling.

3 True/False: When determining the torsional capacity of a weld around the perimeter of a round HSS, an "*Mc* over *I*" approach can be used.

4 True/False: All moment frame connections in seismic lateral force resisting systems must be prequalified per ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings*.

5 True/False: Castellated and cellular beams are proprietary systems within the United States.

6 What is the maximum thickness of fillers that is permitted in bolted connections without having to address the presence of the fillers?
a) 1/8 in. b) 3/16 in.
c) 1/16 in. d) 1/4 in.

7 True/False. A bracing member must have both sufficient strength and stiffness in order to provide a braced point to a column, beam or beam-column.

8 True/False: The only applicable limit state for rods with threaded ends that are subjected only to tension is the tensile strength of the threaded end as given in AISC *Specification* Section J3.6.

9 The 14th Edition AISC *Steel Construction Manual* is available in which of the following formats?
a) Electronic format
b) Print format
c) On a DVD
d) Both A and B

10 True/False: AISC 360-10 Chapter N, "Quality Control and Quality Assurance," includes inspector qualifications, responsibilities, inspection tasks and frequencies.

TURN TO PAGE 14 FOR ANSWERS

1 (b) Two surface classes, Class A and Class B, are recognized by the AISC *Specification*, ANSI/AISC 360-10, and correspond to slip coefficients of 0.30 and 0.50 respectively. This is discussed in Section J3.8 of the *Specification*. The slip coefficient for Class A surfaces has been reduced from that found in the 2005 *Specification* based on recent research that shows a greater variation than previously considered in the performance of mill-scale surfaces.

2 True. Table 10-13 in the 14th Edition AISC *Steel Construction Manual* provides this information. The values shown in the table are based on bend lines perpendicular to the final rolling direction. However, footnote 1 states that when bend lines are parallel to the final rolling direction, the values in the table should be multiplied by 1.5. The principle is that you can achieve a tighter bend when the bend line is perpendicular to the final rolling direction.

3 True. The 14th Edition *Manual* gives guidance on this matter on pages 8-12 and 8-13. If the torque is represented by the symbol M_t and the radius of the weld group is R, the shear per linear inch of weld, r_m, is given by $r_m = M_t R / I_p$, where $I_p = I_{xo} + I_{yo} = 2\pi R^3$. With this formulation, r_m is the shear per linear inch of weld around the complete perimeter.

4 False. R=3 moment connections and those in Ordinary Moment Frames do not need to be prequalified. Intermediate and Special Moment Frames are permitted using either connections prequalified in accordance with ANSI/AISC 358-10, *Prequalified Connections*, or ANSI/AISC 341-10, *Seismic Provisions*, Section K1, or connections qualified in accordance with Section K2 of the *Seismic Provisions*. For more information see ANSI/AISC 341-10, Sections E2.6 and E3.6.

5 False. These types of members are no longer proprietary in the U.S.

6 (d) AISC *Specification* Section J5.2 allows for fillers (or shims) up to ¼-in. thick without any reduction in bolt strength. Otherwise, one of the four requirements given in Section J5.2(a) through (d) must be satisfied to address the presence of thicker fillers.

7 True. Appendix 6 of the AISC *Specification* defines the requirements for bracing that is provided to stabilize individual columns, beams and beam-columns.

8 False. AISC *Specification* Section J3.6 is the appropriate section for determining the rupture strength of the threaded section of the rod. However, the unthreaded length of the rod also must be checked for yielding per AISC *Specification* Section D2.

9 (d) AISC has augmented its classic *Steel Construction Manual* with a Digital Edition. This Digital Edition is a secured PDF file of the entire *Manual* . To learn more about the 14th Edition *Manual*, or to purchase a copy of the *Manual* in either print or digital format (or both), go to **www.aisc.org/manual14**.

10 True. Chapter N provides a template to follow for inspection criteria for steel structures.

Steel SolutionsCenter

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**.

steelwise

INTRODUCING THE AISC SHAPES DATABASE V14.0

BY JIE ZUO

This is the first installment of a three-part series on the new companion materials—all available free online—that have been prepared to complement the 14th Edition *Steel Construction Manual.*

WHEN IT COMES to structural steel design and construction, the most comprehensive resource is the AISC *Steel Construction Manual.* One of its most basic elements is the collection of tables that make up Part 1, which contain commonly used dimensions and properties of nearly all of the structural shapes and sizes available today. The Part 1 data include properties such as gross area, moment of inertia, and width-to-thickness ratios that are used in various design equations and formulas. These—and many more dimensions and properties beyond those provided in the tables in Part 1 of the *Manual*—also are included in electronic format in the AISC Shapes Database.

The AISC Shapes Database was first assembled by Ray Tide, P.E., in the late 1970s in conjunction with the 8th Edition *Manual*, although it was not publicly available until the mid-1980s. Today the database includes less frequently used and more shape-specific properties, such as normalized warping function, W_{no}, for wide-flange shapes and channels, and reduction factors, Q_s, for slender unstiffened compression elements. The AISC Shapes Database is essentially an expanded and more comprehensive electronic version of the tables in Part 1 of the *Manual*, and engineers who are aware of this tool can use it to quickly locate and apply the dimension or property for a specific structural shape being considered in design. It is also useful in developing design and analysis software or creating other steel-related spreadsheets. The updated AISC Shapes Database Version 14.0 was released in September 2011, just a few months after the release of the 14th Edition AISC *Steel Construction Manual.*

Version 14.0 offers a number of improvements and additions. Some additions are a result of new shapes added to the *Manual.* These added shapes include a few new C and MC sizes, a series of smaller double angle sizes (with both short legs back-to-back and long legs back-to-back configurations) and a series of larger HP sizes, up to HP18×204.

In addition to the new shapes, new properties have been introduced to the database as well. The elastic section modulus and moment of inertia about the principal axes of single angles have been provided to facilitate flexural design. The section modulus is computed for three different points about the w and z axes on a single angle, as shown in Figure 1.

Properties to assist with torsional analysis of channels also have been added. These include the warping statical moment, S_w, about three points on a channel's cross-section (see Figure 2).

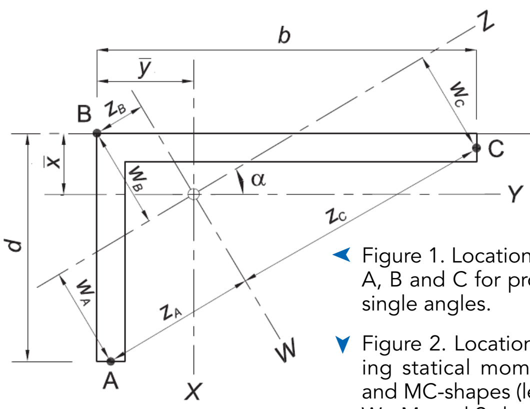

Figure 1. Locations of points A, B and C for properties of single angles.

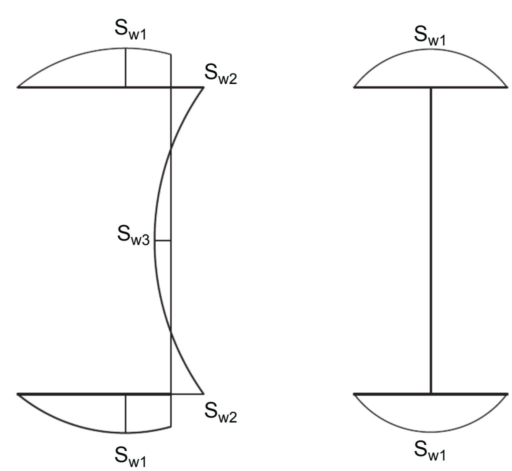

Figure 2. Locations of warping statical moment for C- and MC-shapes (left) and for W-, M- and S-shapes (right).

Jie Zuo is a staff engineer with AISC, Chicago.

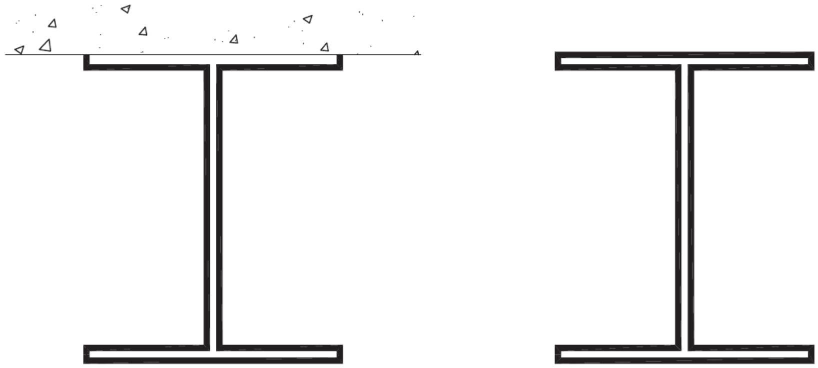

Figure 3. Shape perimeter. Case A is shape perimeter minus one flange width. Case B is full shape perimeter.

Surface area of members is included to help in evaluating how much paint or fireproofing (or any other material applied to the surface) will be needed. The shape perimeter of wide-flange sections has been calculated for two different cases as shown in Figure 3. Case A depicts a scenario where a concrete slab rests on top of the steel section, therefore, reducing the total steel section perimeter by a flange width. Case B depicts a full section perimeter without the flange width deduction. Metric values have been provided in previous versions, but V14.0 is the first to support metric units for all dimensions and properties. These additions highlight the major changes to the new AISC Shapes Database V14.0.

Another tool available is the AISC Historic Shapes Database V14.0 and a "Readme" file. The historic database contains dimensions and section properties from every AISC *Steel Construction Manual* and many of the producer handbooks published since 1873 that preceded the *Manual*. It does not contain any current values, which should be looked up in the AISC Shapes Database. There are 10 past publications listed, the most recent being the 13th Edition *Steel Construction Manual*, which was released in 2005. In the past, steel producer information, which showed specifically the shapes that were produced by each major mill, was included in the *Manual*, and the historic database reflects that up to the 8th Edition *Manual*. After that time, there was more uniformity in production practices and it was no longer necessary to track this.

The Readme file provides important background information and is essential to those who have not previously used the database. It contains definitions for all the dimensions and properties, as well as figures and references for additional guidance. Some properties in the database have been generated for use with information in AISC *Steel Design Guides*, and the Readme document will point you to the design guide pertaining to that property. It also lists the historical publications included in the historical database, and gives directions on how to use that resource.

The AISC Shapes Database V14.0 is a wealth of information and can be used in a variety of ways, ranging from automating simple design calculations to building powerful design tools. It is provided in a Microsoft Excel file format and can be stored and moved electronically with ease so you can always have a little piece of the *Manual* handy in your back pocket. The AISC Shapes Database Version 14.0 is available as a free download at **www.aisc.org/manual14**. MSC




steelwise

ARE YOU PROPERLY SPECIFYING MATERIALS?

BY MARTIN ANDERSON AND CHARLES J. CARTER, S.E., P.E., PH.D.

Keeping tabs on ASTM specifications will help you make the right steel shape choices when designing and building your projects.

THE MATERIALS AND PRODUCTS used in building design and construction are almost universally designated by reference to an appropriate ASTM specification. This simplifies the design and construction process because you can define all the characteristics of a specified product. However, with dozens of ASTM specifications applicable in steel building construction alone it can be a challenge to keep the standard designations used in contracts current.

This article provides a summary of the common ASTM specifications used in steel building design and construction, including structural shapes, plate products, fastening products, and other products. This information is based on similar and more extensive information in the 14th Edition AISC *Steel Construction Manual*. You may also find it convenient to use the AISC publication *Selected ASTM Standards for Structural Steel Fabrication*, a compilation of more than 60 steel-related ASTM standards. (Both the AISC *Manual* and *Selected ASTM Standards* are available for purchase online at **www.aisc.org/bookstore**.)

Note that ASTM standards routinely include a section on ordering requirements that lists the variables in each standard that should be specified in a complete order or specification for the material. This is routine for the purchasing department at the local fabrication company, and may be of great interest to others as well.

Structural Shapes

See Summary in Table 1.

➤ W-Shapes

The preferred material specification for W-shapes is ASTM A992 (F_y = 50 ksi, F_u = 65 ksi). The availability of W-shapes in grades other than ASTM A992 should be confirmed prior to their specification. W-shapes with higher yield and tensile strength can be obtained by specifying ASTM A572 Grade 60, or 65, or ASTM A913 Grades 60, 65 or 70.

W-shapes with atmospheric corrosion resistance (weathering characteristics) can be obtained by specifying ASTM A588 Grade 50 or ASTM A242 Grade 42, 46 or 50. Other material specifications applicable to W-shapes include ASTM A36, ASTM A529 Grade 50 and 55, ASTM A572 Grade 42 and 50, and ASTM A913 Grade 50.

➤ M-Shapes and S-Shapes

The preferred material specification for M-shapes is in transition. ASTM A36 (F_y = 36 ksi, F_u = 58 ksi) remains common, but 50 ksi grades increasingly are being used, including ASTM A572 Grade 50, ASTM A529 Grade 50 or ASTM A992; each of these 50 ksi grades has F_y = 50 ksi and F_u = 65 ksi for these shapes. The availability of M-shapes in grades other than A36 should be confirmed prior to their specification.

M-shapes with a higher yield and tensile strength can be obtained by specifying ASTM A572 Grades 55, 60 and 65, ASTM A529 Grade 55, or ASTM A913 Grades 60, 65 or 70. M-shapes with atmospheric corrosion resistance (weathering characteristics) can be obtained by specifying ASTM A588 Grade 50 or ASTM A242 Grade 50. Other material specifications applicable to M- and S-shapes include ASTM A529 Grade 42, ASTM A572 Grade 42 and ASTM A913 Grade 50.

➤ Channels

The preceding comments for M-shapes apply equally to channels.

Note the MC12×14.3 that appears in the current ASTM A6 listing of standard shapes. Think of this new channel shape as a stair stringer—it has a 2⅛-in. flange width, which is wide enough to accept the handrail pipe and fillet weld around it.

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➤ HP-Shapes

The preferred material specification for HP shapes is ASTM A572 Grade 50 (F_y = 50 ksi, F_u = 65 ksi); the availability of other grades should be confirmed prior to specification.

HP-shapes with atmospheric corrosion resistance (weathering characteristics) can be obtained by specifying ASTM A588 Grade 50 or ASTM A242 Grades 46 or 50. Other material specifications applicable to HP-shapes include ASTM A36, ASTM A529 Grades 50 or 55, ASTM A572 Grades 42, 55, 60 and 65, ASTM A913 Grades 50, 60, 65 and 70, and ASTM A992.

Note the new HP18- and HP16-series shapes that have been added to ASTM A6.

Table 1

Applicable ASTM Specifications for Various Structural Shapes

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

[a] Minimum unless a range is shown.
[b] For shapes over 426 lb/ft, only the minimum of 58 ksi applies.
[c] For shapes with a flange thickness less than or equal to 1.5 in. only. To improve weldability a maximum carbon equivalent can be specified (per ASTM Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM Supplementary Requirement S79).
[d] If desired, maximum tensile stress of 70 ksi can be specified (per ASTM Supplementary Requirement S91).
[e] For shapes with a flange thickness less than or equal to 2 in. only.
[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
[g] Minimum applies for walls nominally ¾-in. thick and under. For wall thicknesses over ¾ in., F_y = 46 ksi and F_u = 67 ksi.
[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM Supplementary Requirement S75).
[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory in ASTM A992.
[j] For shapes with a flange thickness greater than 2 in. only.
[k] For shapes with a flange thickness greater than 1.5 in. and less than or equal to 2 in. only.
[l] For shapes with a flange thickness less than or equal to 1.5 in. only.

● Preferred material specification.
○ Other applicable material specification, the availability of which should be confirmed prior to specification.
(blank) Material specification does not apply.

➤ Angles

The preferred material specification for angles is in transition. ASTM A36 (F_y = 36 ksi, F_u = 58 ksi) remains common, but 50 ksi grades increasingly are being used, including ASTM A572 Grade 50, ASTM A529 Grade 50 or ASTM A992; each of these 50 ksi grades has F_y = 50 ksi and F_u = 65 ksi. The availability of angles in grades other than ASTM A36 should be confirmed prior to their specification.

Angles with higher yield and tensile strength can be obtained by specifying ASTM A572 Grades 55, 60 or 65, ASTM A529 Grade 55 and ASTM A913 Grades 60, 65 or 70. Angles with atmospheric corrosion resistance (weathering characteristics) can be obtained by specifying ASTM A588 Grade 50 or ASTM A242 Grades 46 or 50. Other material specifications applicable to angles include ASTM A529 Grade 42, ASTM A572 Grade 42 and ASTM A913 Grade 50.

➤ Structural Tees

Structural tees are split from W-, M- and S-shapes to make WT-, MT- and ST-shapes respectively. For the preferred material specifications, as well as other suitable material specifications for structural tees, refer to the preceding sections on W-, M- or S-shapes as appropriate.

➤ Rectangular (and Square) HSS

The preferred material specification for rectangular hollow structural sections (HSS) is ASTM A500 Grade B (F_y = 46 ksi, F_u = 58 ksi), although ASTM A500 Grade C (F_y = 50 ksi, F_u = 62 ksi) is very common. The availability of rectangular HSS in grades other than ASTM A500 Grade B should be confirmed prior to their specification.

Rectangular HSS with atmospheric resistance (weathering characteristics) can be obtained by specifying ASTM A847. Other material specifications applicable to rectangular HSS include ASTM A501 Grades A and B and ASTM A618.

➤ Round HSS

The preferred material specification for round HSS is ASTM A500 Grade B (F_y = 42 ksi, F_u = 58 ksi), although ASTM A500 Grade C (F_y = 46 ksi, F_u = 62 ksi) is very common. The availability of round HSS in grades other than ASTM A500 Grade B should be confirmed prior to specification. Generally speaking, only round HSS with the same cross-sectional dimensions as steel pipe are stocked and available. See the sidebar below "12 Tidbits" for further information.

Round HSS with atmospheric corrosion resistance (weathering characteristics) can be obtained by specifying ASTM A847. Other material specifications applicable to round HSS include ASTM A501 Grades A and B and ASTM A618.

➤ Steel Pipe

The material specification for steel pipe used in structural frames is ASTM A53 Grade B (F_y = 35 ksi, F_u = 60 ksi). In some regions, ASTM A53 material is more readily available than ASTM A500 for round cross-sections. See the sidebar "12 Tidbits" for further information.

Plate Products

See Summary in Table 2.

➤ Structural Plates

The preferred material specification for structural plates is in transition. ASTM A36 (F_y = 36 ksi for plate thickness equal to or less than 8 in., F_y = 32 ksi otherwise; F_u = 58 ksi) remains common, but 50 ksi grades increasingly are being used, including ASTM A572 Grade 50 (F_y = 50 ksi for plate thickness equal to or less than 4 in.; F_u = 65 ksi) and ASTM A529 Grade 50 (F_y= 50 ksi for plate thickness equal to or less than 1 in.; F_u = 70 ksi). The availability and cost effectiveness of structural plates in grades other than ASTM A36 should be confirmed prior to their specification. Note also that the availability of grades other than ASTM A36 varies through the range of thicknesses as shown in Table 2.

12 Important Tidbits for 2012

1. When in doubt, check it out. Have questions about availability? Contact a fabricator or the AISC Steel Solutions Center (**solutions@aisc.org**). Either one can keep you swimming in available steel.

2. Times change. ASTM A992 originally was introduced covering only W-shapes. A later revision to this ASTM standard expanded its scope to include other hot-rolled structural cross-sections, including channels, angles and M-shapes, allowing them to be made to ASTM A992. Nevertheless, A992 still is not common in shapes other than W-shapes.

3. Round HSS ≠ Steel Pipe. Know the difference between ASTM A500 and ASTM A53. ASTM A500 is for HSS (F_y = 42 ksi for Grade B; 46 ksi for Grade C). ASTM A53 is for steel pipe (F_y = 35 ksi).

4. Round HSS are similar to steel pipe. Know the similarity between available round HSS (ASTM A500) and steel pipe (ASTM A53). Generally speaking, only round HSS with the same cross-sectional dimensions as steel pipe are stocked and available. So avoid specifying a round HSS with a cross-section that does not match up to one of the steel pipe cross-sections. This is a lot easier than it sounds—just use round HSS with non-zero numbers after the decimal point. For example, HSS5.563×0.258 has the same cross-section as a Pipe 5 Std. And it will generally be available, while HSS5.000×0.250 is an HSS-only product and will require a mill-order quantity to obtain.

5. Properly designate your HSS. A round HSS is designated by nominal diameter and wall thickness, each expressed to three decimal places—for example, HSS5.563×0.258. A square or rectangular HSS is designated by nominal outside dimensions and wall thickness, each in rational numbers—for example, HSS5×3×⅜.

6. Properly designate your steel pipes. Use nominal pipe size (NPS) designation through NPS 12—for example, Pipe 5 Std., Pipe 5 x-strong or Pipe 5 xx-strong. Note that this notation has commonly been abbreviated as follows for the examples given: P5, PX5 and PXX5 respectively. Above NPS 12, use the format "Pipe" followed by diameter x nominal wall thickness, each expressed to three decimal places. For example, a standard-weight NPS 14 is designated Pipe 14.000×0.375. The latter format also applies to any steel pipe size smaller than NPS 12 that does not have an NPS size.

7. Don't confuse anchor rods with bolts. Do not specify your anchor rods as ASTM A325 or A490. The ASTM A325 and A490 standards cover headed bolts, with limited thread length, generally available only up to 8 in. in length, and governed by provisions for steel-to-steel structural joints only. You say you've always specified your anchorage devices this way and it's never been a problem? Well, the reality is that your fabricator has been awfully nice to not embarrass you by pointing out that you've specified a product that does not come in the length you likely specified—or as a hooked or longer-threaded rod. Use ASTM F1554, which covers hooked, headed and threaded/nutted rods in three strength grades.

8. Have all the information at your fingertips. More extensive information can be found in the 14th Edition AISC *Steel Construction Manual* and the AISC publication *Selected ASTM Standards for Structural Steel Fabrication*. Both are available at **www.aisc.org/bookstore**.

9. Remember to specify the alternate core location CVN requirement when you have heavy shapes or plates; see AISC *Specification* Sections A3.1c and A3.1d for further information.

10. When specifying weathering steel, ASTM A242 material typically is more difficult to acquire than ASTM A588 material.

11. Use the new MC12×14.3 for stair stringers. The handrail pipe sizes will fit—as will the fillet welds used to connect them on this new channel with a wider flange.

12. When in doubt, check it out. Oh wait, this is number 1. Well, it is important.

Table 2

Applicable ASTM Specifications for Plates and Bars

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Plates and Bars									
					to 0.75 incl.	0.75 to 1.25 incl.	1.25 to 1.5 incl.	over 1.5 to 2 incl.	over 2 to 2.5 incl.	over 2.5 to 4 incl.	over 4 to 5 incl.	over 5 to 6 incl.	over 6 to 8 incl.	over 8
Carbon	A36		32	58–80										■
			36	58–80	■	■	■	■	■	■	■	■	■	
	A529	Gr. 50	50	70–100	▒	▒ b	▒ b	▒ b	▒ b					
		Gr. 55	55	70–100	▒	▒ b	▒ b							
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒	▒		
		Gr. 50	50	65	▒	▒	▒	▒	▒	▒				
		Gr. 55	55	70	▒	▒	▒	▒						
		Gr. 60	60	75	▒	▒								
		Gr. 65	65	80	▒	▒								
Corrosion Resistant High-Strength Low-Alloy	A242		42	63				▒	▒	▒				
			46	67		▒	▒							
			50	70	▒									
	A588		42	63								▒	▒	
			46	67							▒			
			50	70	▒	▒	▒	▒	▒	▒				
Quenched and Tempered Alloy	A514[c]		90	100–130						▒	▒	▒		
			100	110–130	▒	▒	▒	▒	▒					
Quenched and Tempered Low-Alloy	A852[c]		70	90–110	▒	▒	▒	▒	▒	▒				

[a] Minimum unless a range is shown. [b] Applicable to bars only above 1-in. thickness. [c] Available as plates only.

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

Structural plates with higher yield and tensile strength can be obtained by specifying ASTM A572 Grades 55, 60 or 65, ASTM A529 Grade 55, ASTM A514 Grades 90 or 100, or ASTM A852. Structural plates with atmospheric corrosion resistance (weathering characteristics) can be obtained by specifying ASTM A588 Grades 42, 46 or 50 or ASTM A242 Grades 42, 46 or 50. Other material specifications applicable to structural plates include ASTM A529 Grade 42 and ASTM A572 Grade 42.

➤ Structural Bars

The preceding comments for structural plates apply equally to structural bars, except that ASTM A529 Grade 50 provides for bars up to 2½ in. thick and neither ASTM A514 nor A852 are applicable.

➤ Raised-Pattern Floor Plates

ASTM A786 is the standard specification for rolled steel floor plates. As floor-plate design is seldom controlled by strength considerations, ASTM A786 "commercial grade" is commonly specified. If so, per ASTM A786 Section 5.1.3, "the product will be supplied with 0.33% maximum carbon and without specified mechanical properties." Alternatively, if a defined strength level is desired, ASTM A786 raised-pattern floor plate can be ordered to a defined plate specification such as ASTM A36, A572 or A588; see ASTM A786 Section 5.1.3 and Section 8.

➤ Sheet and Strip

Sheet and strip products, which generally are thinner than structural plate and bar products, are produced to such ASTM specifications as A606, A1008 or A1011. Previously A570 and A607 were listed; these standards have been withdrawn and the materials covered by them are now in A1008, A1011 and thicker materials in A1018. These are "umbrella" standards with many types and grades; the structural steel type is designated "SS" and the standards provide for grades from 25 or 30 to 80. Availability should be checked before specifying the grade.

Fastening Products

See Summary in Table 3.

➤ Conventional Bolts

The preferred material specification for conventional (heavy hex) high-strength bolts in steel-to-steel connections is ASTM A325, although ASTM A490 can be specified when higher strength is desired. In either case, Type 1 is the most commonly specified (medium-carbon steel). When atmospheric corrosion resistance is desired, Type 3 can be specified. While still formally permitted in the AISC *Specification*, the use of other material specifications in steel-to-steel bolting applications has become quite uncommon.

➤ Twist-Off Type Tension-Control Bolt Assemblies

There are two preferred material specifications for twist-off type tension-control bolt assemblies. ASTM F1852 offers a strength equivalent to that of ASTM A325 bolts and ASTM F2280 offers a strength equivalent to that of ASTM A490 bolts.

➤ Nuts

The preferred material specification for heavy-hex nuts is ASTM A563. The appropriate grade and finish is specified per ASTM A563 Table X1.1 according to the bolt or threaded part with which the nut will be used. For steel-to-steel structural bolting applications, the appropriate grade and finish is summarized in the RCSC *Specification* Section 2.4. If its availability can be confirmed prior to specification, ASTM A194 Grade 2H nuts are permitted as an alternative, as indicated in the RCSC *Specification* Table 2.1.

➤ Washers

The preferred material specification for hardened steel washers is ASTM F436. This specification provides for both flat and beveled washers. While standard ASTM F436 washers are sufficient in most applications, there are several specific applications when special washers are required. The special washer requirements in RCSC *Specification* Section 6 apply when oversized or slotted holes are used in the outer ply of a steel-to-steel structural joint. In anchor rod and other embedment applications, hole sizes generally are larger than those for steel-to-steel structural bolting applications. Accordingly, washers used in such applications generally are larger and might require design consideration for proper force transfer, particularly when the anchorage is subject to tension. Where anchor rods are used in holes larger than 1⁄16 in. bigger than the rod, ASTM F844 washers are permitted and they have a larger diameter than F436.

➤ Compressible-Washer-Type Direct-Tension Indicators

When bolted joints are specified as pretensioned or slip-critical and the direct-tension-indicator pretensioning method is used, ASTM F959 compressible-washer-type direct-tension indicators are specified. Type 325 is used with ASTM A325 high-strength bolts and Type 490 is used with ASTM A490 high-strength bolts. The use of these devices must conform to the requirements in the RCSC *Specification*, which provides detailed requirements for pre-installation verification (Section 7), installation (Section 8) and inspection (Section 9). The RCSC *Specification* also permits alternative washer-type indicating devices subject to the provision in Section 2.6.2.

➤ Anchor Rods

The preferred material specification for anchor rods is ASTM F1554, which covers hooked, headed, threaded and nutted anchor rods in three strength grades: 36, 55 and 105. ASTM F1554 Grade 55 is most commonly specified, although Grades 36 and 105 are normally available. ASTM F1554 Grade 36 may be welded, while Grade 55 may be welded if it is ordered with

Table 3

Applicable ASTM Specifications for Various Types of Structural Fasteners

ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Diameter Range (in.)	High-Strength Bolts: Conventional	High-Strength Bolts: Twist-Off-Type Tension-Control	Common Bolts	Nuts	Washers	Direct-Tension-Indicators	Threaded Rods	Shear Stud Connectors	Anchor Rods: Hooked	Anchor Rods: Headed	Anchor Rods: Threaded & Nutted
A108		—	60	0.375 to 0.75 incl.								●			
A325[d]		—	105	over 1 to 1.5 incl.	●										
		—	120	0.5 to 1 incl.	●										
A490[d]		—	150	0.5 to 1.5	●										
F1852[d]		—	105	1.125		●									
		—	120	0.5 to 1 incl.		●									
F2280[d]		—	150	0.5 to 1.125 incl.		●									
A194 Gr. 2H		—	—	0.25 to 4				○							
A563		—	—	0.25 to 4				●							
F436[b]		—	—	0.25 to 4					●						
F959		—	—	0.5 to 1.5						●					
A36		36	58–80	to 10							●		○		○
A193 Gr. B7[e]		—	100	over 4 to 7							○		○		○
		—	115	over 2.5 to 4							○		○		○
		—	125	2.5 and under							○		○		○
A307 Gr. A		—	60	0.25 to 4			●								
A354 Gr. BD		—	140	2.5 to 4 incl.							○		○	○	○
		—	150	0.25 to 2.5 incl.							○		○	○	○
A449		—	90	1.75 to 3 incl.	c						○		○	○	○
		—	105	1.125 to 1.5 incl.	c						○		○	○	○
		—	120	0.25 to 1 incl.	c						○		○	○	○
A572	Gr. 42	42	60	to 6							○		○		○
	Gr. 50	50	65	to 4							○		○		○
	Gr. 55	55	70	to 2							○		○		○
	Gr. 60	60	75	to 1.25							○		○		○
	Gr. 65	65	80	to 1.25							○		○		○
A588		42	63	over 5 to 8 incl.							○		○		○
		46	67	over 4 to 5 incl.							○		○		○
		50	70	4 and under							○		○		○
A687		105	150 max.	0.625 to 3							○		○		○
F1554	Gr. 36	36	58–80	0.25 to 4									○	○	○
	Gr. 55	55	75–95	0.25 to 4									●	●	●
	Gr. 105	105	125–150	0.25 to 3									○	○	○

● Preferred material specification.

○ Other applicable material specification, the availability of which should be confirmed prior to specification.

Material specification does not apply.

— Indicates that a value is not specified in the material specification.

[a] Minimum unless a range is shown or maximum (max.) is indicated.

[b] Special washer requirements may apply per RCSC *Specification* Table 6.1 for some steel-to-steel bolting applications and per Part 14 for anchor-rod applications.

[c] See AISC *Specification* Section J3.1 for limitations on use of ASTM A449 bolts.

[d] When atmospheric corrosion resistance is desired, Type 3 can be specified.

[e] For anchor rods with temperature and corrosion resistance characteristics.

Supplement S1 and the carbon equivalent formula in Section S1.5.2.1. Grade 105 may not be welded, as the heat will detrimentally affect performance. Several other ASTM specifications also may be used. For applications involving rods that are not headed, ASTM A36, A193, A307, A354, A449, A572, A588 and A687 can be specified. Note that the ASTM A307 Grade C "anchor bolt" has been deleted from ASTM A307 and replaced by ASTM F1554 Grade 36. For applications involving headed rods A354 and A449 can be specified.

Threaded Rods

The preferred material specification for threaded rods, whether provided with plain or upset ends, is ASTM A36. Other material specifications that can be specified include ASTM A193, A307, A354, A449, A572, A588 and A687. Note that ASTM A354 Grade BC and A449 are permitted to be used for bolts when the size required is outside the range of ASTM A325. ASTM A354 Grade BD is permitted when the size required is outside the range of ASTM A490. These standards are material standards, not bolt standards, so the desired dimensions have to be specified as per ANSI ASME B18.2.6 heavy hex Class 2A.

➤ Shear Stud Connectors

The preferred material specification for shear stud connectors used for the interconnection of steel and concrete elements in composite construction is ASTM A29 provided in a condition defined in ASTM A108. The mechanical requirements are stated in AWS D1.1 Table 7.1 for Type B (F_y = 50 ksi, F_u = 65 ksi).

➤ Forged Steel Structural Hardware

Forged steel structural hardware products, such as clevises, turnbuckles, eye nuts and sleeve nuts are occasionally used in building design and construction. These products are generally provided to AISI material specifications. AISI C-1035 is commonly used in the manufacture of clevises and turnbuckles. AISI C-1030 is commonly used in the manufacture of steel eye nuts and steel eye bolts. AISI C-1018 Grade 2 is commonly used in the manufacture of sleeve nuts. Other products, such as steel rod ends, steel yoke ends and pins, cotter pins and coupling nuts are provided generically as "carbon steel." The dimensional and strength characteristics of these devices are described in the literature provided by their manufacturers. Note that such information may be provided as a safe working load and based upon a factor of safety as high as five, assuming that the product will be used in rigging or similar applications subject to dynamic loading. If so, the tabular value might be overly conservative for permanent installations and similar applications subject to static loading only. In these applications, a factor of safety of three is more common.

➤ Filler Metal

The appropriate filler metal for structural steel is summarized in ANSI/AWS D1.1 Table 3.1 for the various combinations of base metal specification and grade, and electrode specification. A tensile strength level of 70 ksi is indicated for the majority of the commonly used steels in building construction.

Other Products

➤ Steel Castings and Forgings

Steel castings are specified as ASTM A27 Grade 65-35 or ASTM A216/A216M Grade WCB with supplementary requirement S11 Grade 80-35. Steel forgings are specified as ASTM A668.

➤ Crane Rails

Crane rails are furnished to ASTM A759, ASTM A1 and/or manufacturer's specifications and tolerances. Rail is designated by unit weight in units of pounds per yard. Dimensions of common rail are shown in the AISC 14th Edition *Manual* Table 1-21. Most manufacturers chamfer the top and sides of the crane rail head at the ends unless specified otherwise to reduce chipping of the running surfaces. Often crane rails are ordered as end-hardened, which improves the crane rail ends' resistance to impact from contact with the moving wheel during crane operation. Alternatively, the entire rail can be ordered as heat-treated. When maximum wheel loading or controlled cooling is needed, refer to manufacturer catalogs. Purchase orders for crane rails should be noted "for crane service." Light 40-lb rails are available in 30-ft lengths, standard rails in 33-ft or 39-ft lengths, and crane rails up to 80 ft. Consult manufacturer for availability of other lengths. Rails should be arranged so that joints on opposite sides of the crane runway will be staggered with respect to each other and with due consideration to the wheelbase of the crane. Rail joints should not occur at crane girder splices. Odd lengths that must be included to complete a run or obtain the necessary stagger should be not less than 10-ft long. Rails are furnished with standard drilling in both standard and odd lengths unless stipulated otherwise on the order. MSC

product expert series

THE EVOLUTION OF FABRICATION

BY LYLE MENKE

Steel fabrication facilities in the U.S. have undergone remarkable changes and today benefit greatly from technology's increased role.

THE STEEL FABRICATION SHOP is one area of the steel construction industry that has seen an evolution of technologies paralleling any manufacturing or commercial enterprise in the world.

But unlike common manufacturers, structural steel fabrication does not require mass production. In contrast to Henry Ford's assembly line, which was designed to produce millions of the same "widgets" annually, structural steel fabricators are in the business of building skylines.

Structural steel fabricators must manufacture to specific drawings: one-of-a-kind construction, if you will. All the while, the fabricator must adhere to exacting engineering criteria, safety standards and building codes.

The very uniqueness of the shop fabrication process has led to innovation. While perhaps not as dynamic as the assembly line for making automobiles, technology certainly has played an important role in advancing the steel construction industry.

A Look Back

Less than 40 years ago, the typical structural fabrication shop had a much different look. The structural work was very labor intensive, using equipment like portable magnetic drills, portable C-frame punch presses and radial arm drills.

No CNC or automated tool measuring in those days; it was the era of measuring tape, soapstone and tee square. Welding was predominant, and much of it performed on site during the erecting process.

Crane use was mandatory to move almost every structural shape or section. Preliminary functions such as the lengthy process of sawing a section to length required exacting measurement and employed cold saws or oxy/fuel torches, which required significant grinding to clean the cut.

Then came the innovative tool called the "Beatty Punch," which employed hydraulic presses in a semi-automated (numerically controlled, or NC) system. This enabled a beam profile to be moved to the tool for hole making, rather than moving the tool to the beam profile. The first seeds of automation had been planted.

A CNC "Systems Approach" to Fabrication Changes Everything

As the steel construction industry moved forward into the 1970s and 1980s, systems technology enveloped the structural fabrication shop. New electronic motors were employed, which provided precise positioning of even large columns. The first computers were seen on the shop floor, pioneered by the HP 85 and 9815 models on the first Peddinghaus drill lines. Thus, the era of computer numeric control (CNC) technology was ushered in.

What exactly did CNC technology, or a systems approach to fabrication, bring to the table? In a nutshell, it enabled the hole making process to be fully automated—and more importantly—increased accuracy and repeatability.

The bottom line for steel construction: welded connections could now be replaced by bolted connections, which were still as strong but faster and easier to fabricate and erect. Steel construction was beginning to boom as a result of this technology.

New electronic technologies streamlined old manual, labor-intensive methods to make the structural fabrication shop more productive, with the benefit of increased CNC accuracy.

Meeting Challenges

As we focus on fabrication shop floor "manufacturing," it is evident that new technologies have always provided a "better mousetrap" for the industry. A quick review of some fabrication shop challenges that were solved with today's technologies shows how positively the industry has been affected.

Consider these three technologies that have effectively changed the face of fabrication shop production.

➤ **Problem:** Today's rolled steel sections are much higher tensile strength—exceeding capacities on a typical beam punch line; additionally, oversized fabricated beams are used in some applications.

Lyle Menke is the vice president of marketing for Peddinghaus Corporation, Bradley, Ill., and has more than 30 years experience in the steel fabrication industry.

➤ **Solution:** Drill line technology, such as the first TDK models, introduced electronic measuring and positioning that enabled fast, accurate positioning, as well as faster hole making via drilling rather than punching the hole.

Fast forward to today where carbide drilling is employed on structural drilling machines. Holes that once were drilled in nine seconds are now processed in three seconds with carbide bits. Today's carbide drill lines capitalize on precision electronic technology to dictate superior positioning speed, precise hole locations, and determine any mill tolerance deviations in just tenths of a second. Today's modern drills also tap, countersink, and create slotted holes via milling.

➤ **Problem:** Today's steel construction methods require many copes, interior cuts, weld access holes, dog bone configurations and other shapes that require thermal cutting. The previous manual method required about 10 steps including laying out the pattern with a measuring tape and soapstone, lighting a torch, manually cutting, and then grinding it clean.

➤ **Solution:** Employing electronic principals of crisp beam positioning, thermal processing, and dynamic software, the automated beam coping system was born.

Fast forward and the 10 steps were eliminated, the quality was perfect, and the production was increased one hundredfold. Plasma units, such as Peddinghaus' Ring of Fire machine, employ plasma technologies for all-thermal cutting including copes, internal cuts, notches, cut offs, miters and even hole making in a one-pass process.

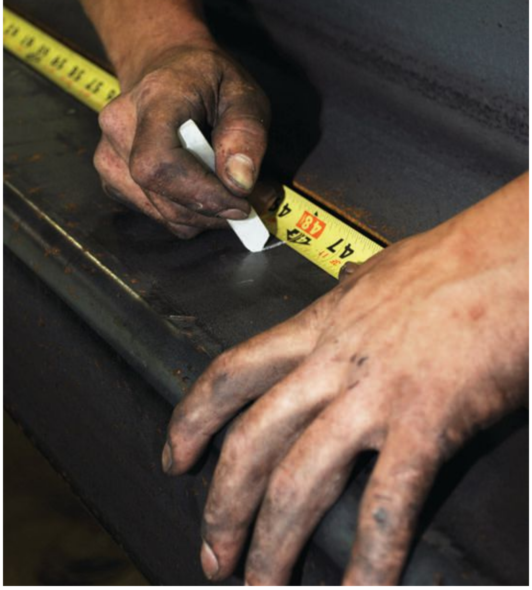
A quick review of some fabrication shop challenges that were solved with today's technologies shows how positively the industry has been affected.

➤ **Problem:** Laying out the beam detail onto any structural section always required a skilled craftsman, accurate drawings, a correct measuring tape—and a sharpened soap stone. This job was always done manually and always created a production bottleneck in the fit-up area. Multiple craftsman were required to keep up with production requirements.

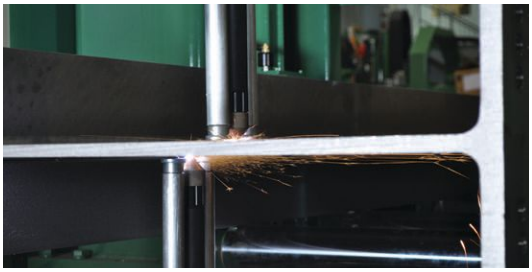
➤ **Solution:** Today's modern BIM software carries all the detail for each component of the building. The location for every connection plate and clip angle, identification numbers and weld symbols are normally included in the data, which can be downloaded to the fabrication equipment via a normal DSTV file.

Fast forward to 2011 and NASCC: The Steel Conference in Pittsburgh where two machine tool technologies were introduced that provide automated solutions to the age old problem of beam layout and fit-up. In one, carbide scribe technology can be employed on all four beam surfaces in a drilling machine. The data is etched onto the steel beam. It is effective, but can delay hole processing productivity on your machine.

Newer technology employs plasma arc writing of all the same layout/fit-up data onto all surfaces of the beam. This technology employs a low voltage plasma arc that provides a clear, legible mark in seconds, not minutes.



Unlike the bygone days when multiple craftsmen were required to lay out each fabrication and keep up with production requirements, today's equipment uses data directly from the building information model.



Today's carbide drill lines use precision electronic technology for quick and accurate positioning, also determining and accounting for any mill tolerance deviations in just tenths of a second. Many can also tap, countersink and create slotted holes via milling.

Give Credit Where Credit is Due

The structural steel fabrication shop is the Rodney Dangerfield of the steel construction industry. It just doesn't get any respect!

- The shop floor is not as glamorous as the architect's drawings/renderings, which create all of the buzz and excitement about a new project.
- The shop floor does not generate the accolades from the structural engineers and the detailers that the "techno glitz" of the BIM models do.
- The shop floor does not receive the admiration and awe from spectators when the steel is erected at the job site.
- All the structural fabrication shop does is make or break a project, usually meaning the difference between financial profitability and loss.

Don't you agree that a little respect is in order?

These three examples show how modern technology has changed the entire face of steel construction on the shop floor. With today's new equipment, one operator working in a safe, quality controlled environment can produce tonnages that in years gone by required multiple laborers.

Fabrication shop floor technologies—spearheaded by the demand for higher tonnage production at lowered costs—continue to improve the odds for steel in the battle of selecting a building material. Today's advanced software and modern machine tools are the "weapons of mass construction" that bring competitive numbers when bidding projects. Fast fabrication leveraged by new technologies that further facilitates fast erection is setting the stage for increased market share, and that's good news for the construction industry. MSC

Automated beam coping systems have eliminated many of the steps previously required to cope beams, including much of the cleanup.

economics

THE ECONOMICS OF INNOVATION

BY JOHN CROSS, P.E., LEED AP

Consider an active approach to driving down construction costs, rather than passively waiting for demand to increase.

OVER THE PAST SEVERAL YEARS the U.S. economy has been subject to a variety of innovative economic interventions. Regretfully those interventions have had little long-term impact in generating a recovery in the building construction sector. Many explanations have been given as to the lack of a rebound in building construction, but the bottom line is actually quite straightforward—the required investment is not justified by the financial return that the building will generate.

The lack of a satisfactory rate of return is not just an issue with respect to spec or rental buildings. It applies to all building construction. Even when organizations are considering new facilities for their own internal use project costs must be weighed against rental rates in the vicinity of the proposed project. In many areas of the country rental rates for office, warehouse, retail and industrial space are depressed as a result of oversupply. At the same time, the general uncertainty relative to the economy pushes up the level of the required rate of return as a means of accounting for the external economic risk of the project.

The flip side of this scenario can currently be found in apartment construction. Demand for apartments has been increasing as a result of foreclosure displacement and a current preference for renting over buying. This increase in demand has resulted in increasing rental rates as noted in the January 5, 2012, *Wall Street Journal*.

- The nation's apartment-vacancy rate in the fourth quarter fell to its lowest level since late 2001 [and rents climbed], according to data firm Reis Inc.
- The vacancy rate fell to 5.2% from 6.6% a year earlier and 5.6% at the end of the third quarter, according to Reis.
- Nationwide, landlords raised asking rents an average of 0.4% in the fourth quarter.
- Just 8,865 units were delivered in the quarter, the second-lowest quarterly figure since Reis began publishing quarterly data in 1999. The strength of the market hasn't been lost on developers who are racing to move plans off the drawing boards. More than 173,000 units were likely started in 2011 and some 225,000 and 280,000 starts are expected nationwide in 2012 and 2013, according to Zelman & Associates.

These higher rental rates provide the required rate of return to justify the construction of new apartment facilities. None of this should be surprising as the construction market is simply behaving according to basic laws of supply and demand.

Does that mean other types of building construction will remain in the doldrums until there is an increase in demand for space? Clearly demand for office space will not increase until office employment returns to pre-recession levels. Demand for retail and warehouse space will not rebound until consumer spending accelerates. Industrial space demand is dependent on the balance between imports, exports and domestic production. So the answer is yes, building construction activity is dependent on the vitality of the overall economy.

But that doesn't mean construction activity is completely captive to the overall economy and the hope that innovative economic intervention might accelerate a recovery. It is not innovative economics that will provide an impetus to construction, but rather it is the economics of innovation that can help move construction forward.

The financial justification of a project is not based solely on the actual or equivalent income that a building will generate. Income is only half of the equation. The other half is construction cost. As construction costs are reduced, lower levels of pro forma income are required to justify the project.

According to the U.S. Bureau of Labor Statistics, the cost of new construction dropped from a 2008 peak to a trough in 2009 and then began to again increase to a current level above that of 2008. This drop and recovery in construction costs was a function of the decrease in, or elimination of, overhead and profit margins triggered by a drop in demand and then an ultimate re-stabilization of supportable costs on the part of general and specialty contractors. It is important to note that the actual cost of construction probably did not change; rather, the level

John Cross, P.E., LEED AP, is an AISC vice president.

of compensation received by construction firms was artificially depressed.

The actual reduction of the costs associated with building construction can come in one of two ways. The project designer can investigate innovative new systems that can reduce the traditional cost of construction or innovative approaches for project delivery can be implemented to improve construction productivity thereby lowering construction costs. The economics of innovation result in lower project costs, which means that more projects will meet the required levels of return.

Various innovative structural steel based systems are currently available in the marketplace. Girder-Slab, Peikko, ConX-Tech, SidePlate, Smartbeams and Versa-Floor along with many others regularly highlighted in *Modern Steel Construction* are all proprietary systems with a demonstrated record of significantly reducing project costs. Other systems, such as the TTG One Story High Rise system, are on the verge of their initial projects moving forward. The proprietary nature of these systems should not deter designers or owners from exploring and pricing them. Licensing costs are the justifiable return on the investments made by these organizations to develop these systems and are easily identified in the economic assessment of the project.

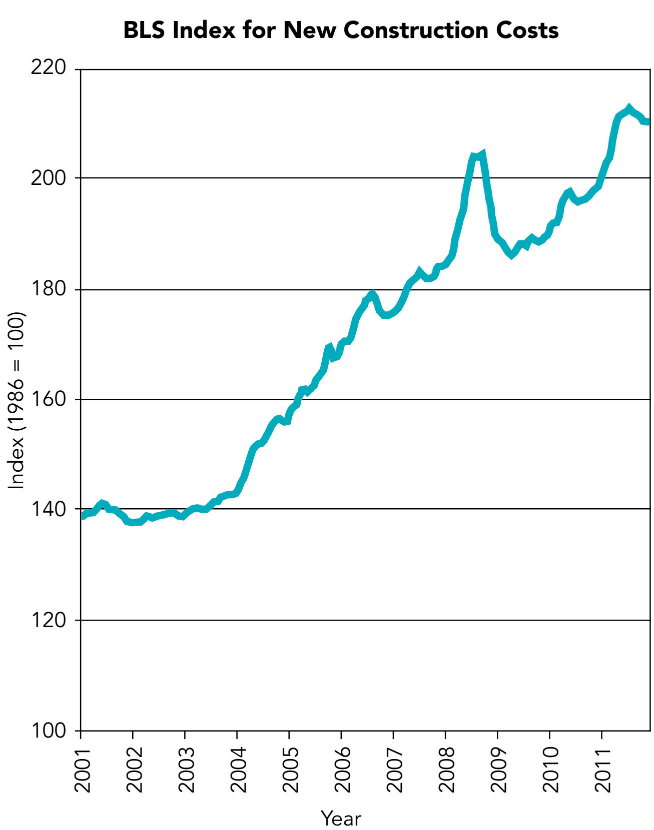
Other innovative approaches that can reduce project costs, such as steel plate shear walls, staggered truss applications for multi-story residential projects and replaceable steel elements in seismic designs, are not proprietary and can be easily evaluated by designers. New approaches including a greater reliance on modular construction techniques should also be considered as a means of improving productivity and reducing project costs.

When construction costs decrease, additional projects become viable and overall construction activity can increase.

Such systems are only half of the innovative solutions that can reduce project costs. Much has been written about how the implementation of building information modeling (BIM) and the various forms of integrated project delivery that also can significantly reduce the final project cost. Numerous case studies exist documenting project cost savings in the range of 10% or more when technology and collaboration are allowed to improve design and construction project productivity. A 10% change in construction costs can swing a project from failing to meet its required financial return to being a viable project.

Even on projects not implementing BIM or still being delivered on a design-bid-build basis, costs for the structural steel portion of the project can be reduced by simply engaging the structural steel fabricator early in the design process.

The bottom line is that the economics of innovation can serve to drive down construction costs while still allowing construction firms and specialty contractors to generate a necessary level of return on their efforts. When construction costs decrease, additional projects become viable and overall construction activity can increase. MSC



Although the actual cost of constructing things probably did not change between its 2008 peak and 2009 trough shown on this graph of BLS statistics, owners saw a substantial drop in costs that reflected the decrease in, or elimination of, overhead and profit margins triggered by a drop in demand.

business issues

BRINGING SANITY TO THE WORKPLACE

Applying a bit of psychology can help address the root cause of unsafe behavior.

BY BARRY NATHAN, Ph.D.

IF IT SEEMS that some employees seem to be more accident prone, or have more workers' compensation claims than others, you're right: Personality matters when it comes to safety.

A growing body of research is proving that personality plays a big part in how safely or unsafely someone will work. As a result, companies are turning to safety-based personality assessments to identify potential and current employees who are more likely to take risks and experience accidents and injuries.

A 2008 study by the Liberty Mutual Research Institute for Safety found that injured employees cost organizations nearly $1 billion per week in direct and indirect costs, despite engineering and environmental interventions, and policies and procedures specifically designed to increase workplace safety. Clearly, safety training is not enough. One safety consultant noted that if an organization doesn't have insight into how employee personalities contribute to the safety climate, even the most extensive safety program will have limited success.

Predicting Unsafe Employees

Tulsa, Okla.-based Hogan Assessment Systems is one of the leading workplace personality assessment firms in the world. The firm is best known for its leadership assessments, but also has conducted extensive research on safety. Going back over 30 years, Hogan has identified six safety-based personality attributes that predict a range of safety-related outcomes, including workers' compensation claims, accidents and injuries:

- **Compliant:** An inclination to adhere to rules and policies.
- **Strong:** Able to effectively manage stress under pressure.
- **Cheerful:** Able to maintain control over one's emotions and not lose one's temper.
- **Vigilant:** Remaining focused when performing routine or mundane tasks, not becoming distracted.
- **Cautious:** Not being inclined to take risks.
- **Trainable:** Willingness to accept new ways of doing things.

Hogan's research results are impressive. One West Coast transportation firm found that individuals with a high-safety/low-risk profile had 22% fewer accidents, 40% fewer rule violations and 25% fewer workers' compensation claims.

In a Midwest manufacturing firm, 63% of individuals with below average safety scores filed workers' compensation claims, compared to only 28% of those workers with above average safety scores. That is a 40% difference in workers' compensation claims.

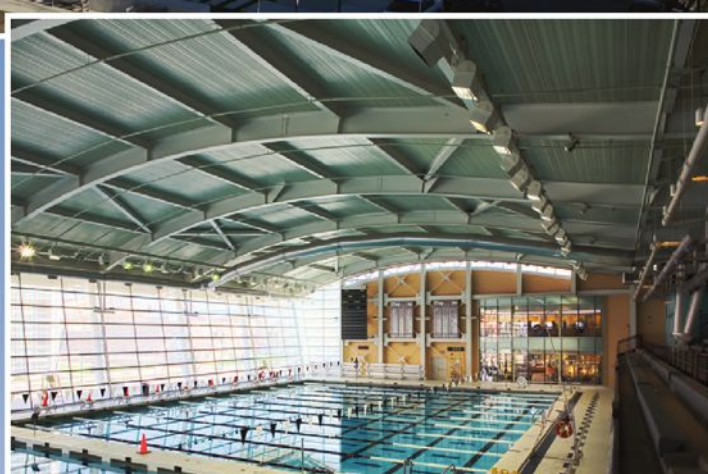
Among employees in a national postal and parcel delivery organization performing jobs that involved receiving, transporting, and delivering packages, employees with above average safety scores had 25% fewer citations for "unsafe work behaviors" compared to those with below average safety scores. By hiring only individuals with above average safety scores, the company could have reduced its number of citations by 13%.

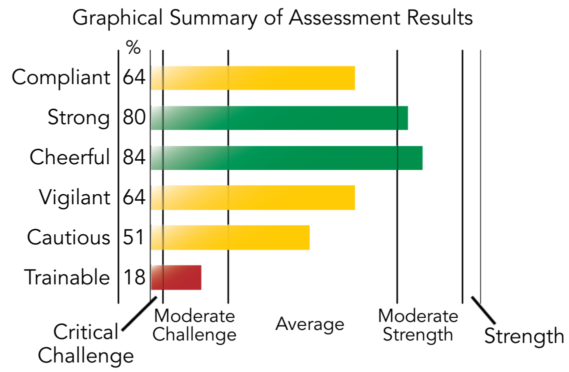
Improving Safety Among Current Employees

While some employers are turning to personality assessment to avoid hiring potentially unsafe employees, others are using personality assessment as a preemptive way to address current employees' unsafe tendencies *before* accidents happen. Using personality assessments can be an effective way for supervisors to coach their employees about safety concerns.

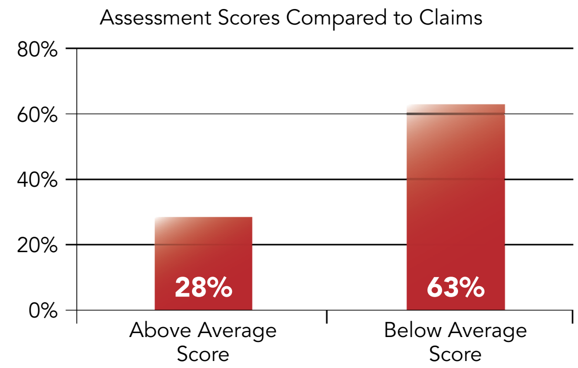
Once trained in how to interpret the assessment results and how to use this information to coach, supervisors review each subordinate's results and develop personalized Safety Improvement Plans with each employee based on their assessment-determined tendencies. Coaching helps employees recognize personal tendencies and anticipate problems before they occur. Coaching based on personality assessment makes employees aware of their blind spots and helps them develop personalized strategies for how to deal with them.

Barry Nathan, Ph.D., is an industrial-organizational psychologist and president of Pittsburgh-based Leader Business Coaching. He can be reached at **Barry.nathan@LeaderBusinessCoaching.com**.





Graphical representation of the assessment results show an individual's areas of strength in green while areas that could be improved are in yellow or red.



Portion of one firm's workers who filed workers' compensation claims, based on whether they scored above or below average on the personality assessment.

Employers have found an additional benefit from training supervisors to become personality-based safety coaches: Supervisors become better coaches in general. Typically, coaching focuses on performance issues; too often it occurs after an incident has occurred. Often supervisors are reluctant or uncomfortable confronting employees about performance problems, and employees often become defensive. In these situations, the coaching experience is often distasteful, not always effective, and occasionally dysfunctional.

> Personality assessment provides a pre-emptive way of addressing unsafe tendencies *before* accidents happen.

In contrast, when the focus of coaching is on safety assessment results rather than an incident, the discussions tend to be more strategy focused. There is less reason to be defensive, and more reason to be collaborative. Coaching becomes an opportunity to drive safety commitment, rather than a discussion about safety compliance.

Hogan's SafeSystem Assessments are delivered online. The coaching program for supervisors is provided by a Hogan-certified trainer, but the safety improvement materials and personalized lessons for employees are provided as online modules. The SafeSystem assessments and safety reports have been translated into 22 languages and dialects.

Not Just for Technical Employees

Most companies are using SafeSystem in technical and transportation divisions, but at the 2011 NASCC: The Steel Conference in Pittsburgh, the program piqued the interest of a one firm's vice president of sales. "I have a great sales staff," he said, "but they're risk-takers, nonconformers, easily distracted, and in some cases, a bit arrogant; everything you say makes for an unsafe employee. They'd jump on a moving girder if it meant making a big sale! I love their commitment, but sometimes their enthusiasm scares me to death. This would be a great on-boarding program!" I told him that I couldn't agree more. MSC

Structural Steel "Flat Plate" Construction

BY JANIS VACCA, P.E., AND CLIFFORD SCHWINGER, P.E.

Using a system of custom-fabricated steel beams to support hollow-core plank enables quick and economical construction of low floor-to-floor height structures.

FLAT PLATE FLOOR SYSTEMS offer several advantages over other floor framing systems—the primary one being the ability to build structures with low floor-to-floor heights. Low floor-to-floor height is a standard requirement for most residential projects such as apartments, condominiums and university residence halls. Flat plates are usually associated with materials other than structural steel; however, when Girder-Slab Technologies developed the D-Beam® 10 years ago, it opened the door to using steel framing in a flat plate system.

D-Beams are shallow, structural steel beams custom-fabricated from wide-flange shapes and steel plate and configured to support precast hollow-core plank in a manner that buries them within the depth of the plank (see Figure 1). The resulting floor system is essentially a "flat plate" where few, if any, steel beams protrude below the bottom of the plank.

The D-Beam is the heart of the Girder-Slab® System, and also is the way by which Girder-Slab licenses fabrication of D-Beams to fabricators. Steel fabricators bidding Girder-Slab projects request a license fee from Girder-Slab when preparing their bids. The fee is added to the bid price and the successful bidder pays Girder-Slab Technologies the license fee, which allows them to fabricate D-Beams and to assemble the beams in a manner that achieves composite action with the hollow-core plank.

The composite action between the D-Beams and the grouted hollow-core provides an in-slab girder with flexural strength much greater than that of the steel beam acting alone. It is both the D-Beam and its composite action with the plank that have been patented by Girder-Slab Technologies.

The company has developed six different D-Beam sizes as well as a design procedure for use by designers for sizing the members. The design procedure is explained in a design guide that is available at no cost on the Girder-Slab website, **www.girder-slab.com**.

A Recent Project

Rutgers University
Camden Housing Project Details

Tower square footage:	161,653 sq. ft
Number of floors:	12
Steel tonnage:	600 tons (7.4 psf)
Construction started:	April 2011
Steel erection started:	July 2011
Steel topped off:	September 19, 2011

One example of a project using Girder-Slab construction is on Rutgers University's Camden, N.J., campus. When the university recently was looking to build a new residence hall, several alternative structural systems were investigated. The goal was to select a framing system with the following characteristics:

- Economical
- Non-proprietary system
- Fast construction
- Low floor-to-floor height
- Accommodation of other trades to permit fit-out on lower levels while building above
- Easy accommodation of alternative facade systems
- Open space on ground
- No transfer girders
- Flexibility to permit architectural expression

A structural steel frame with Girder-Slab D-Beams supporting hollow-core plank met all of the requirements. Once the system was selected, the design team worked closely with the construction manager, the steel fabricator and Girder-Slab Technologies to optimize the design to best suit the fabricator's fabrication and erection preferences.

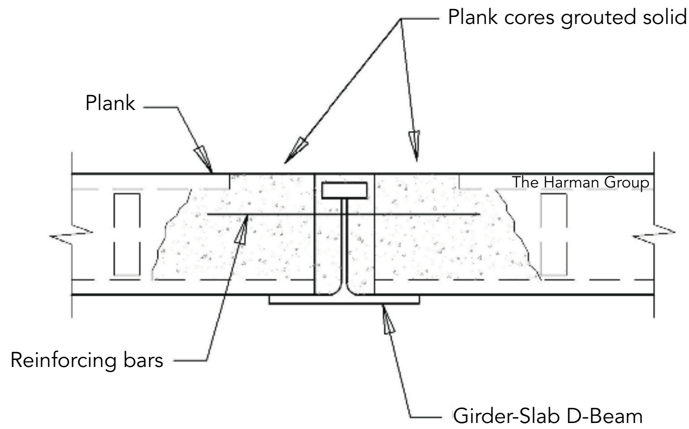


Figure 1: Section through a Girder-Slab D-Beam.

Simple but critical issues included omitting column stiffeners, designing and documenting all connection details on the contract documents, eliminating perimeter spandrel framing parallel to the plank span and coordinating between the structural system and mechanical systems to eliminate issues between the two that often arise during construction. Coordination and attention to detail by the design team permitted construction to progress at a rate of 16,000 sq. ft per week (see Figure 2).

As with any framing system, the key to economical design is familiarity with details and constraints. The structural engineer, the Harman Group, was familiar with the Girder-Slab System, having previously designed five Girder-Slab projects. The steel fabricator, Berlin Steel Construction Company, also was experienced in using this system. The Rutgers-Camden project was its 13th Girder-Slab building. Additionally, Dan Fisher, Sr., the managing partner of Girder-Slab, provided excellent support throughout the project. A steel fabricator for more than 30 years prior to starting Girder-Slab Technologies in 2002, Fisher's expertise in steel fabrication and erection was an invaluable resource to the team during design.

The Rutgers Camden project was delivered on budget and ahead of schedule. The versatile combination of structural steel framing with Girder-Slab D-Beams provided Rutgers University the perfect answer to its need for more student housing.

More About the System Efficiency

Girder-Slab-framed floors are extremely economical. The basic weight of the D-Beams on the typical floors in the Rutgers project was about 1.5 lb per sq. ft. That allowed flexibility in the project budget for some remarkable architectural expression, such as a cantilevered corner in the northeast quadrant of the building.

Erection with this system is quick. All-bolted construction permits the steel to go up fast. After the floor framing is in place the hollow-core plank is placed, reinforcing steel is installed through the D-Beams into the ends of the plank and the plank ends are grouted (Figure 1).

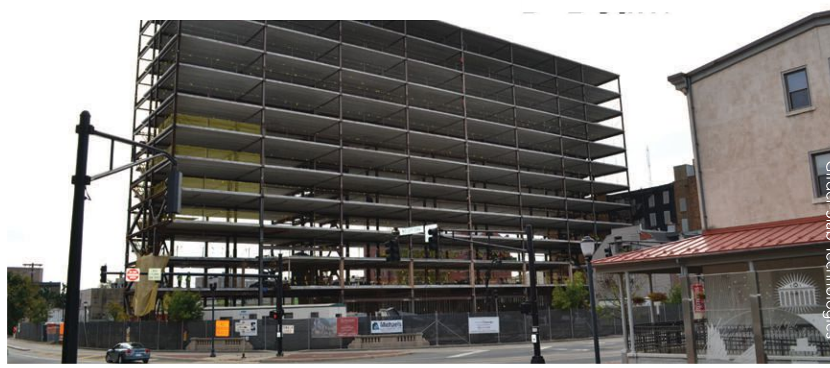
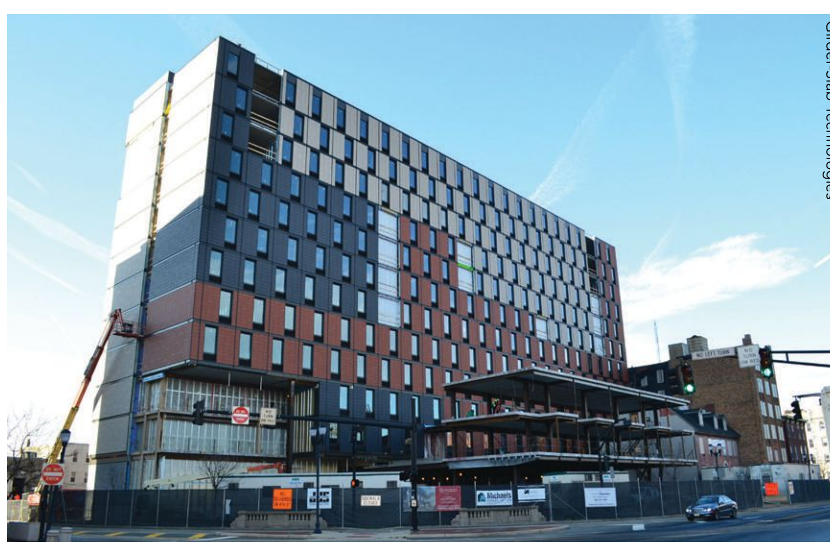


Figure 2: The fully erected and planked structure in mid-September 2011.

Figure 3: By the first week of January, 2012, the structure was almost fully enclosed.



Janis Vacca, P.E., and Clifford Schwinger, P.E. are structural engineers and principals at The Harman Group, King of Prussia, Pa. Both are AISC Professional Members.

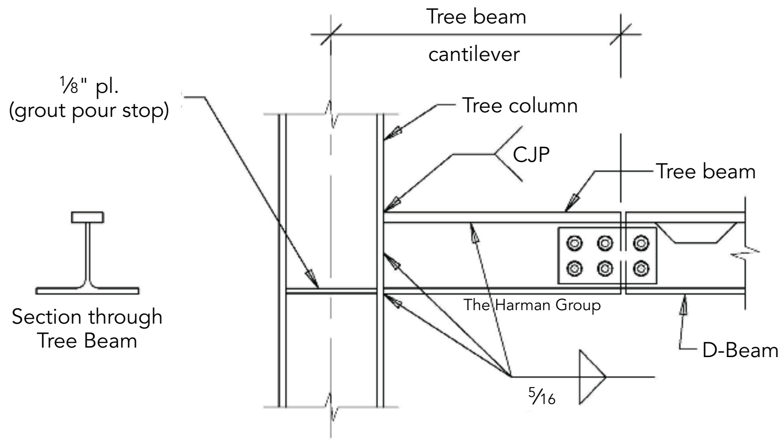


Figure 4: Tree columns are used to increase column spacing.

Figure 5: Tree columns in place.

Efficiently Configuring D-Beam-Framed Floors

The practical span limitation is about 18 ft for 8-in.-deep D-Beams supporting 8-in. hollow-core plank spanning up to 30 ft. Column spacing can be increased by shop welding cantilevered beams to the columns, commonly referred to as "tree columns" (see Figures 4 and 5). The cantilevered "tree beams" on the tree columns have the same configuration as the D-Beams, but are non-composite members resisting the negative bending moment. When tree columns are used, column spacing can be increased to about 23 ft, although span limits vary depending on member sizes, plank span and loads. D-Beams are used only where required. Conventional W-shape framing should be used where possible, such as at spandrels, on braced frames and at other locations where the use of W-shapes can be accommodated.

Lateral Load Resisting System and Floor Diaphragm

Lateral loads in steel-framed structures with hollow-core plank floors typically are resisted with braced frames and/or moment frames. Braced frames and moment frames must be configured to work within the span limitations of untopped hollow-core plank diaphragms. Typically, this requires placing braced frames in every second or third bay perpendicular to the plank span. This arrangement limits diaphragm spans to manageable dimensions and better mobilizes the dead load of the structure in resisting the overturning moments from lateral loads. The greater the number of braced frames used, the smaller the net tension forces will be in the columns and foundations. Single strut 4-in.-wide HSS diagonal braces should be used when possible because these narrow members fit well within standard walls.

On the Rutgers-Camden project a single braced frame was configured parallel to the long direction of the building. A continuous drag strut was installed parallel and adjacent to the corridor to tie the building together and to collect lateral loads along the length of the building to transfer them to the braced frame. An alternative to the single longitudinal braced frame would have been to provide W12 spandrel beams and moment frames along the exterior column lines.

The use of a 2-in. structural topping slab can greatly increase the spanning capability and strength of the floor diaphragm. Topping slabs are required on projects where seismic forces are of a magnitude that precludes the use of untopped plank diaphragms. The disadvantage of a structural topping slab, however, is that it adds several dollars per sq. ft to the project cost and may introduce issues related to pouring the topping slab during the winter months. When a topping slab is not used, a ¾-in.-thick self-leveling material should be specified in order to achieve a level floor finish. Significant camber can occur in hollow-core plank and the leveling material will serve to provide a more level floor. The self-leveling material can be installed after the building is enclosed.

Spandrel Beams

Spandrel beams spanning parallel to the span of the hollow-core plank may be eliminated when the plank is designed to support the facade, or when the facade is panelized and designed to span column to column. Eliminating the spandrel beams will reduce steel tonnage, however attention must be paid in providing continuous chord reinforcing steel within the plank floor diaphragm that would have otherwise been provided by the spandrels.

On the Rutgers-Camden project the spandrels were eliminated. The plank was designed to support the facade and continuous chord reinforcing steel was provided in the edges of the plank.

Conclusion

Steel-framed structures using Girder-Slab D-Beams to support hollow-core plank floors often provide the most economical solution for mid-rise residential construction. The system's low weight, compared to cast-in-place concrete construction or masonry bearing walls, results in lower foundation costs. As a structural system, it offers low floor-to-floor heights and fast, year-round construction. The hollow-core planks provide good sound attenuation and a flat soffit that eliminates conflicts with mechanical systems above ceilings. Because the system is patented but not proprietary, fabricators pay a project-specific license fee to fabricate D-Beams in an arrangement that benefits owners by allowing all steel fabricators to bid on Girder-Slab projects. MSC

Developer
Michaels Development Company dba Camden Student Housing, LLC, for ownership by Camden County (N.J.) Improvement Authority

Architect
Erdy McHenry Architecture, Philadelphia

Structural Engineer
The Harman Group, Inc., King of Prussia, Pa.

General Contractor and Construction Manager
Joseph Jingoli and Son, Inc., Lawrenceville, N.J.

Steel Fabricator
The Berlin Steel Construction Company, Malvern, Pa. (AISC Member)

Steel Erector
JL Erectors, Inc., Blackwood, N.J. (IMPACT Member)

The Matter of Cost

Comparing the cost of one project with the cost of another is difficult. Differences in location, economic conditions and the specific geometry of projects make dollar comparisons misleading. For example, some additional steel cost on the Rutgers-Camden project went into the cantilevered corner at one end of the building. However, had this been a plank and masonry bearing wall project we would not have been able to cantilever the corner as we did on this project. Likewise a masonry bearing wall building would have required costly transfer girders below the second floor to provide the required open space on the first floor.

Structural engineers often are judged by the "pounds per square foot" of steel on the project. Averaging 1.5 psf for basic floor framing on this project is extremely low, as is 7.4 psf overall. But even with such good structural efficiency, structural steel would not even have been considered were it not for the low floor-to-floor heights achievable with the Girder-Slab system.

Easily replaceable after a seismic event, this brace is equally well suited to new construction and retrofits.

A New Brace Option for Ductile Braced Frames

BY MICHAEL GRAY, CONSTANTIN CHRISTOPOULOS, PH.D., P.ENG., JEFFREY PACKER, PH.D., P.ENG., AND CARLOS DE OLIVEIRA, P.ENG.

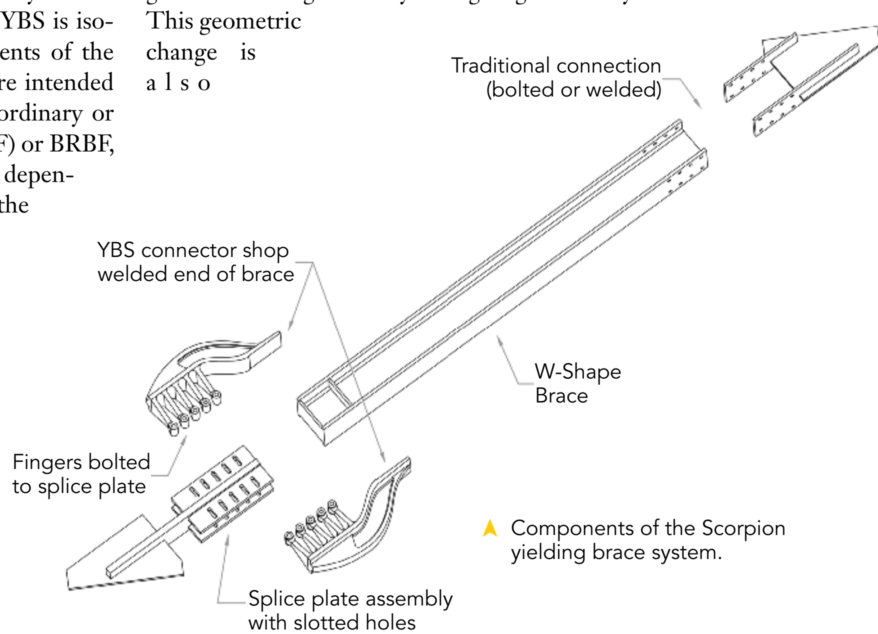
THE BUCKLING RESTRAINED BRACED FRAME (BRBF) is one of the more innovative new technologies to have been developed in the construction industry in recent years. Introduced to the U.S. engineering community in 1999, it became a codified system in less than a decade. Because thinking outside the box is the best way to solve complex problems, it's no surprise that the wheels of science and innovation have continued to churn, resulting in a new system. The industry is now being offered a completely novel yielding brace for braced frames—the Cast ConneX® Scorpion™ Yielding Brace System (YBS).

The YBS, developed at the University of Toronto by Constantin Christopoulos, Jeffrey Packer and Michael Gray, provides enhanced seismic performance for concentrically braced frames. In an earthquake, non-linear response in a YBS is isolated to the brace assembly while the other elements of the frame, such as beams, columns and gusset plates, are intended to remain predominately elastic. So, just as in an ordinary or special concentrically braced frame (OCBF or SCBF) or BRBF, the performance and response of a YBS frame is dependent primarily on the performance and response of the bracing elements comprising it.

The Scorpion Brace

A brace in a YBS frame consists of two cast steel connectors connected to the end of a W-shape or other structural member. Each connector is made up of an arm and a series of fingers and is produced from a highly ductile, notch-tough cast steel grade. In a brace assembly, the fingers of both connectors are bolted to a splice plate which is connected to the beam-column intersection through a traditional gusset plate connection. The other end of the brace is connected to the beam-column intersection with a conventional gusset-to-W-shape connection that can either be field welded or bolted. The Scorpion moniker comes from the brace's resemblance to a scorpion, with the connectors forming the creature's claws.

Energy dissipation in the brace is provided through the flexural yielding of the connector's fingers. Like the 1993-vintage triangular added damping and stiffness system (TADAS), the fingers of the YBS connectors are triangular in shape to promote the spread of plasticity along their entire length. As the fingers are expected to undergo large deformations during a design level earthquake, the ends of the yielding fingers are bolted to long slotted holes in the splice plate, which accommodate the geometric change in the yielding fingers as they are deformed. This geometric change is also



Components of the Scorpion yielding brace system.

responsible for the system's distinct post-yield increase in strength and stiffness at large deformations.

Full-Scale Structural Testing

After the system was conceived, a prototype connector assembly was designed for a 250 kip yield load and the assembly was component tested as well as tested in a full-scale braced frame at the University of Toronto Structural Testing Facility. Two prototype brace assemblies successfully completed a displacement protocol in the braced frame based on the requirements of Section K of ANSI/AISC 341-10, with a design level brace elongation, Δ_{bm}, of 1.5 in. The resulting hysteresis exhibited very full, symmetric loops with the distinct post-yield stiffness at large deformations.

The prototype that was designed and tested represents only one potential connector configuration. Yielding finger geometry can be altered and the number of fingers varied, which together provide the ability to achieve a variety of combinations of elastic stiffness and yield load in the connector. Additional information on the initial development of the connectors and the subsequent testing is available in the conference proceedings from the 2010 U.S./Canadian Conference on Earthquake Engineering. The proceedings are available for purchase at **http://2010eqconf.org**.

Scorpion Connector Series

The YBS technology was specifically conceived to be well suited as an off-the-shelf line of products. With the successful testing of the prototype having been completed, an initial series of connectors has been developed for a range of yield forces (see table at right); additional connectors are planned for assemblies having yield forces exceeding 500 kips. The initial connector series is based on five unique connector designs that can fill a range of 12 different yield forces by removal of up to four yielding fingers in the five base assemblies. Because the device is a connector, a single YBS design can work for nearly any unique building geometry without requiring a significant change in the typical details.

Performance Benefits

A main drawback of a BRBF is that its stiffness is linked to its yield force. The primary method of increasing stiffness is increasing core area, which also increases the yield strength of

Device*	Nominal Yield Force[1] [kips]	k_{device}[2] [kips/in]
YBS-50-6	50	834
YBS-100-6	75	1,182
YBS-150-6	90	1,336
YBS-100-8	100	1,576
YBS-150-8	120	1,776
YBS-215-6	129	1,542
YBS-150-10	150	2,221
YBS-215-8	172	2,061
YBS-310-6	186	2,044
YBS-215-10	215	2,575
YBS-310-8	248	2,729
YBS-310-10	310	3,415

* Each color represents a unique connector design, each with a different number of yielding fingers. Shaded cells highlight the five unique connector geometries.

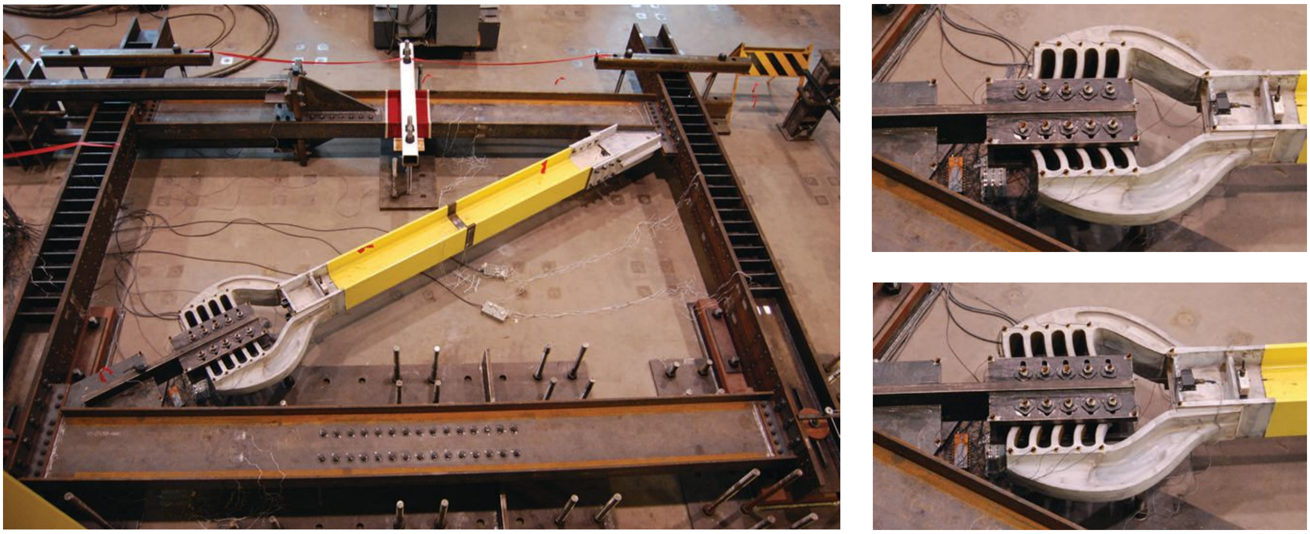
[1] Cast ConneX recommends using a resistance factor, ϕ, of 0.9 in conjunction with the Nominal Yield Force when designing for strength requirements.

[2] The axial stiffness of the Scorpion brace assembly is calculated by combining the stiffnesses of the YBS device and that of the brace member per the following equation:

$$k_{assembly} = \frac{1}{\left(\frac{1}{k_{device}}\right) + \left(\frac{L}{AE}\right)_{brace}}$$

➤ The Scorpion yielding brace system.

Michael Gray is a doctoral candidate at the University of Toronto. Constantin Christopoulos, Ph.D., P.Eng., is an associate professor of civil engineering and the director of structures laboratories at the University of Toronto. Jeffrey A. Packer, Ph.D., P.Eng., is Bahen/Tanenbaum Professor of Civil Engineering at the University of Toronto. Carlos de Oliveira, P.Eng., is president and principal structural engineer at AISC member firm Cast Connex Corporation (**www.castconnex.com**).



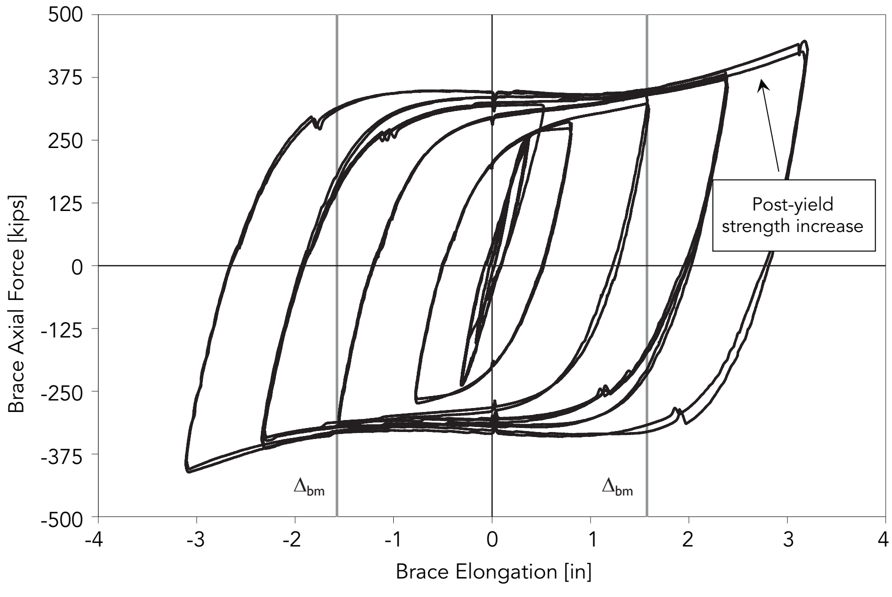
The full-scale test frame at the University of Toronto with the Scorpion brace in the neutral position. A short video of the frame being tested aired on the Discovery Channel and can be viewed at **http://bit.ly/gvji3U**.

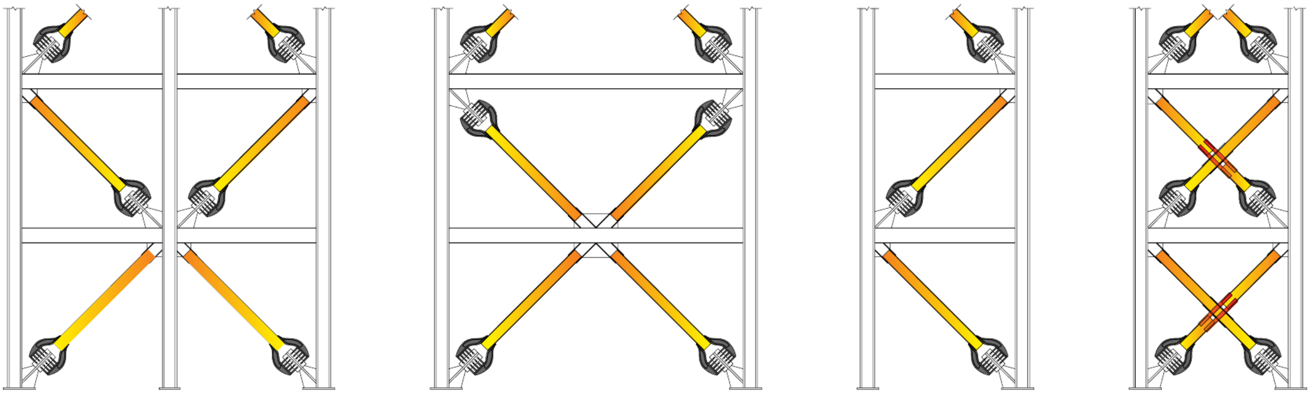
the brace. Consequently, taller buildings and buildings with strict drift requirements often are designed with stronger than necessary braces and thus these structures do not take full advantage of the available ductility of the system. When capacity designing the other structural members of the frame, overdesign of the braces can lead to significantly larger members and thus elevated tonnage and cost.

Engineers employing Scorpion braces avoid this problem because the devices have relatively high elastic stiffnesses in the brace axis direction. Depending upon brace length and the brace section selected, Scorpion braces can be as much as twice as stiff as a buckling-restrained brace of the same activation force. A designer need only select the appropriate connector size to satisfy strength requirements and then use capacity design principles to select an appropriate brace member. The stiffness of the Scorpion brace assembly is easily computed by adding the flexibilities of the connector and the brace, just as one combines stiffnesses for any structural elements in series. If the resulting design does not provide the required stiffness, the designer has the freedom to select a brace member that is stiffer, which will increase the assembly stiffness without changing the yield force of the brace. Thus, stiffness and strength can be independently adjusted to provide the desired structural performance.

An additional benefit of the YBS is its unique post-yield response. As previously noted, at large deformations the YBS exhibits post-yield strengthening and

A graph of the test hysteresis based on a 1.5-in. brace elongations, Δ_{bm}, shows very full, symmetric loops with distinct post-yield stiffness at large deformations.





Various possible YBS configurations.

stiffening due to second-order geometric effects. This stiffening behavior allows for a better distribution of yielding in braces over a building's height at large drift levels. In the event that deformations begin to collect in a single story, the system's post-yield stiffening and strengthening will cause braces in adjacent stories to be activated, thereby reducing the likelihood of the formation of a "soft-story". Neither of these advantages is available in systems that exhibit little or no post-yield stiffness. Further, after a significant seismic event, the yielding fingers of the connectors are fully accessible and can be easily inspected for signs of severe inelastic deformations or damage.

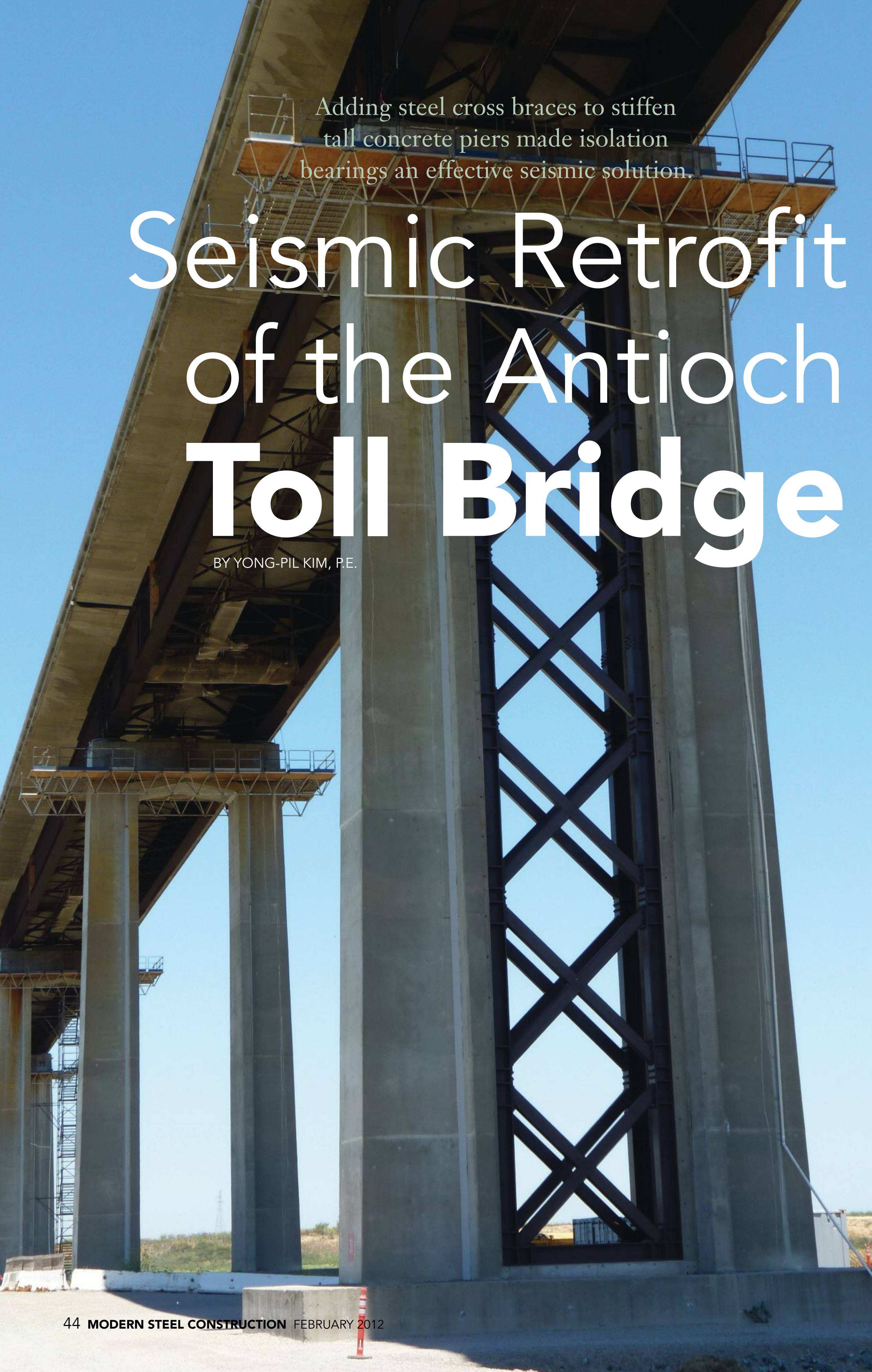
Scorpion connectors are produced through a highly controlled steel casting process that ensures excellent quality control and a high repeatability of geometry and mechanical properties. It is envisioned that each unique connector from the series will be subjected to full-scale structural testing to confirm the mechanical response of the standardized component. Then, the performance of subsequent production components can be assured through non-destructive examination of each part and the physical and chemical testing of steel produced from every heat.

Also For Retrofit

The combination of high stiffness and low activation force also makes the YBS ideal for retrofitting seismically deficient structures. The high elastic stiffness of the system can reduce the drift levels of older building frames while lower activation forces reduce costly remediation of members, connections and foundations. In fact, the YBS is currently being considered for use in the seismic retrofit of a school in the Charlevoix region of Quebec, Canada's most severe seismic region.

Furthermore, because all of the inelasticity in the brace is confined to the Scorpion connectors, engineers using the system have the flexibly to employ a wider range of brace configurations than can be accommodated with other yielding brace systems. For example, a single-bay, single-story X configuration can be employed to reduce the number of frames which are lost or obstructed. In the case of retrofit, this configuration can reduce the cost of the removal and replacement of building envelope and finishes.

For more information regarding the Scorpion YBS including typical details, recommended design level deformations for each connector, or for design assistance, contact AISC member Cast ConneX (www.castconnex.com). MSC



Y.P. Kim

Adding steel cross braces to stiffen tall concrete piers made isolation bearings an effective seismic solution.

Seismic Retrofit of the Antioch **Toll Bridge**

BY YONG-PIL KIM, P.E.

Steel braces were added to stiffen the columns of the 20 tallest piers as part of a seismic retrofit on the Antioch Toll Bridge over the San Joaquin River in Northern California.

THE SEISMIC RETROFIT of the Antioch Toll Bridge in Northern California consists of replacing the existing bearings at all 39 piers and at the abutments with seismic isolation bearings. In order to make the isolation bearings work effectively, it also was necessary to install steel bracing in the tall piers to make the pier portal frames stiffer.

Caltrans owns and operates Antioch Toll Bridge, but the funding came from the Bay Area Transit Authority (BATA), which also managed oversight of the retrofit construction. The total steel used for the cross bracing was 1,850 tons, all of which was fabricated and prime painted by Brooklyn Iron Works, Inc., Spokane, Wash. Eighty-two single-surface friction pendulum isolation bearings were supplied by Earthquake Protection Systems, Inc., Vallejo, Calif.

The main structure is 8,650-ft long with 40 spans arching over San Joaquin River. The midsection of the bridge rises as high as 147 ft to allow for ship passage. The superstructure consists of two weathering steel plate girders that are continuous over the piers. The girders are in excellent condition, having formed the expected uniform protective outer coating with no degradation in structural capacity.

Antioch Toll Bridge is one of the last two toll bridges to be retrofitted in Northern California. It was constructed in 1978, so the lessons learned from the San Fernando Earthquake of 1971 were implemented in the original design. For this reason, the bridge was long considered to have sufficient earthquake resistant features and deemed safe. However, reevaluating the bridge based on the latest seismic design criteria and an extensive geotechnical investigation, Caltrans concluded that the bridge needed to be retrofitted.

The bridge's average daily traffic is 15,000, a relatively small number compared to other toll bridges in Northern California. However, because it crosses the San Joaquin River, which is an important navigational channel, its seismic retrofit is based on the Safety Evaluation Earthquake criteria with a 1,000-year return probability. The project-specific SEE design criteria are based on "No Collapse" with permissible damages in parts of the pier pile groups and the deck expansion joints.

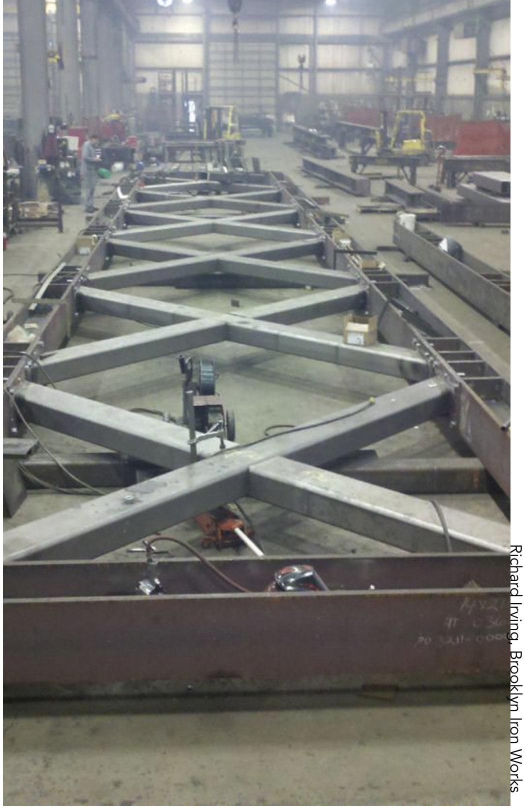
The analysis of the existing bridge exposed several deficiencies. First, there is a possibility of shear failure in the existing columns and the bent caps. Second, the existing rebar couplers at the base of the columns could fail prematurely. Third, the existing pile foundation system could fail undermining the stability of the bridge. In addition, the existing pin hanger hinges could fail due to possible misalignment of the girders.

Although the existing superstructure carries only two traffic lanes and is relatively light , isolating the superstructure proved to be an effective solution. Single-surface friction pendulum isolation bearings were selected for the design due to the restricted vertical clearances. Two sizes were used in order to accommodate different magnitudes in loading conditions. The larger bearings are 7.2 ft in diameter, 9.2-in. thick and have 23 in. of maximum displacement capacity. The smaller ones are 5.8 ft in diameter, 7.2-in. thick and have 20 in. of maximum displacement capacity.

By isolating the superstructure, the base shear at the piers dropped between 23% and 79%. Similar reduction in shear demand in the bent caps was observed. In addition, it reduced the tensile forces in the column vertical rebar. This will eliminate concerns about premature failure of the existing rebar splices by keeping the forces in the rebar within the yield limits. Although the retrofit reduces forces going into the pile foundation, some pile failures are still expected. Most of the pile failure will be in the exterior battered piles that will form multiple plastic hinges. Some interior piles will fail, in some piers, but based on the project-specific "No Collapse" criteria the performance of the substructure is defined as acceptable. This not only reduces the construction cost but also saves the existing river environment from any disturbance.

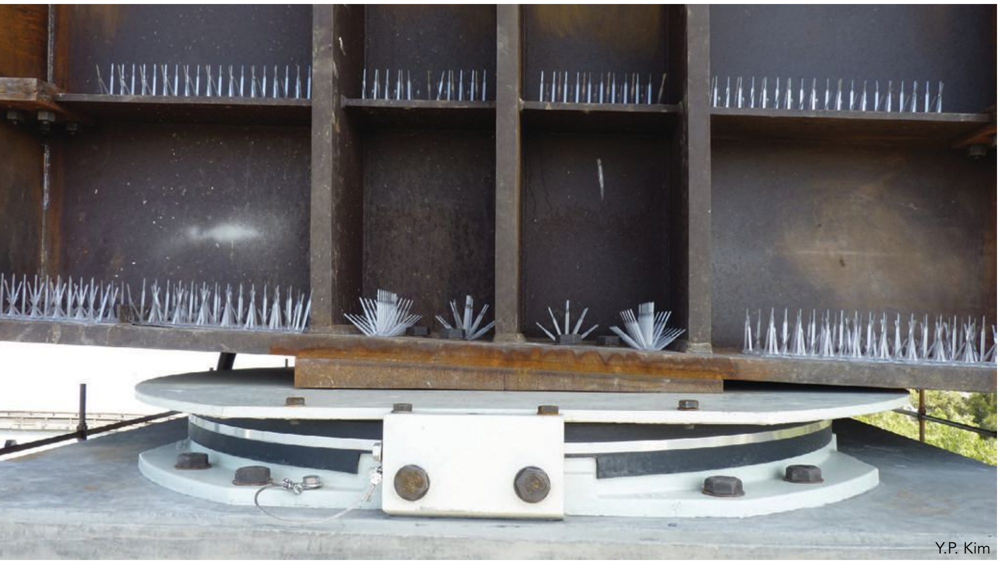
Yong-Pil Kim, P.E., is a senior bridge engineer at Caltrans. He has 23 years of bridge design experience and is the project engineer for the design of seismic retrofit the Antioch Toll Bridge project.

Two sets of steel cross bracing were installed to stiffen each of the taller piers, then painted brown to match the weathering steel superstructure.



Far left: The main diagonal cross braces are rectangular HSS welded at the cross joints.

Left: All frames were trial fit in the Brooklyn Iron Works shop.



Allowance had to be made for the bevel plate above the isolation bearing in addition to the thickness of the bearing itself.

Steel braces were added between the columns for 20 of the tallest piers in the mid-portion of the bridge, which range in height from 82 ft to 147 ft. The piers consist of portal frames with two hollow concrete columns linked by a hollow concrete bent cap. Because the original, unbraced frames were flexible under lateral loading, it was necessary to make them stiffer for the isolation beatings to be effective. Accomplishing this through the use of bracing also reduced the seismic loading in the columns. Steel braces were the obvious solution, because of their relatively light weight and ease of installation.

The main diagonal cross braces consist of HSS 12×8⅝ welded at the cross joints. There are two sets of bracing per pier. Each set of braces aligns with the webs of the hollow columns in the transverse direction to make the concrete and steel bracing work integrally in resisting shear. The cross braces are connected on each side to vertical W14×211 wide-flange beams, ASTM A709, Grade 50W, which in turn are connected to the columns through a cast-in-place concrete pedestal. The combination of rebar attached to the side of the existing concrete pier and the shear studs attached to the beam flange cast within the concrete pedestal will solidly link the two elements. Connection plates welded to the ends of the braces are bolted onto the inside flanges of the wide-flange beams.

Field Installation

Jacking of the girders was carried out with live traffic on the bridge deck. Only temporary road closures were necessary when lowering the bearings from the deck. A maximum of ½ in. of lifting of the girders was necessary to release the existing bearings. On many of the piers the jacking system was supported on two solid steel cylinders that were inserted into holes cored in the concrete bent cap. Simultaneous jacking was carried out at four points on each pier to unload the existing bearings.

Even though relatively thin bearings were selected it was still necessary to remove some existing concrete at the top of the bent cap to accommodate the bevel plate over the bearing and the grout pad underneath and still keep the same vertical profile of the existing bridge deck. This was accomplished by using a cable saw to cut and remove as much as 4.5 in. of the top of the existing concrete bent caps. Because this process either cuts through the exiting top transverse bent cap reinforcements or weakens their bonding, additional post tensioning was installed in transversely cored holes and lightly stressed. This will preserve the moment capacity of the bent cap. Even though some of the bent caps were up to 32-ft wide, coring through them could be achieved fairly accurately.

The bridge has four intermediate hinges that were retrofitted with an internal shear key system in order to prevent any possible transverse misalignment of the girders with respect to each other across the hinges. The hinges are connected with a pin hanger system. Any out of plane bending would make the hinge vulnerable and although there are stay plates attached to both top and bottom flanges they are not strong enough to resist in a major earthquake.

The seismic retrofit of the Antioch Toll Bridge based on isolating the superstructure

Steel cylinders inserted through holes cored in the pier cap provided a base for hydraulic jacks, which lifted the girders to allow replacement of the original bearings with isolation bearings.

Pile Group Performance

Even with the isolation of the superstructure there will be partial failures in the existing pile groups under the design earthquake. Because the project-specific design criteria are based on preventing the collapse but not immediate functionality of the bridge, the partial failure of the pile groups after a major earthquake is acceptable as long as the bridge can still support its own weight.

All the piers have battered exterior piles, which will absorb much of the seismic forces and likely form plastic hinging in the piles. The interior vertical piles will deflect and ride out the seismic forces more easily. Due to the exterior battered piles, the pile group rigidly linked with the pile cap will not only translate laterally but also will rotate. In order to analyze the maximum displacement capacities of the pile groups, the displacement and rotational capacities of each pile group had to be calculated. This was compared with different stages of pile group failure to ascertain their ultimate capacities.

The as-built condition was analyzed with a global dynamic analysis based on acceleration response spectra (ARS) with an equivalent 6x6 matrix stiffness for the pile groups at each piers. The retrofitted bridge was analyzed with a global dynamic analysis using time histories. SAP2000 was used for the dynamic analyses.

is a simple but effective solution. Implementing this scheme by adding steel cross braces to the concrete pier frames was an ideal match. Shop fabricated segments of the steel braces were field assembled with bolted connections and the bracing can be easily integrated to the existing concrete frame by connecting the two different elements through a cast-in-place concrete pedestal. Due to steel's light weight, the additional weight of the bracing could be accommodated within the capacity of the existing foundation. Not requiring a foundation retrofit meant big savings in the construction cost and also minimized the disturbance to the sensitive environment. MSC

Owner and Engineer
California Department of Transportation (Caltrans)

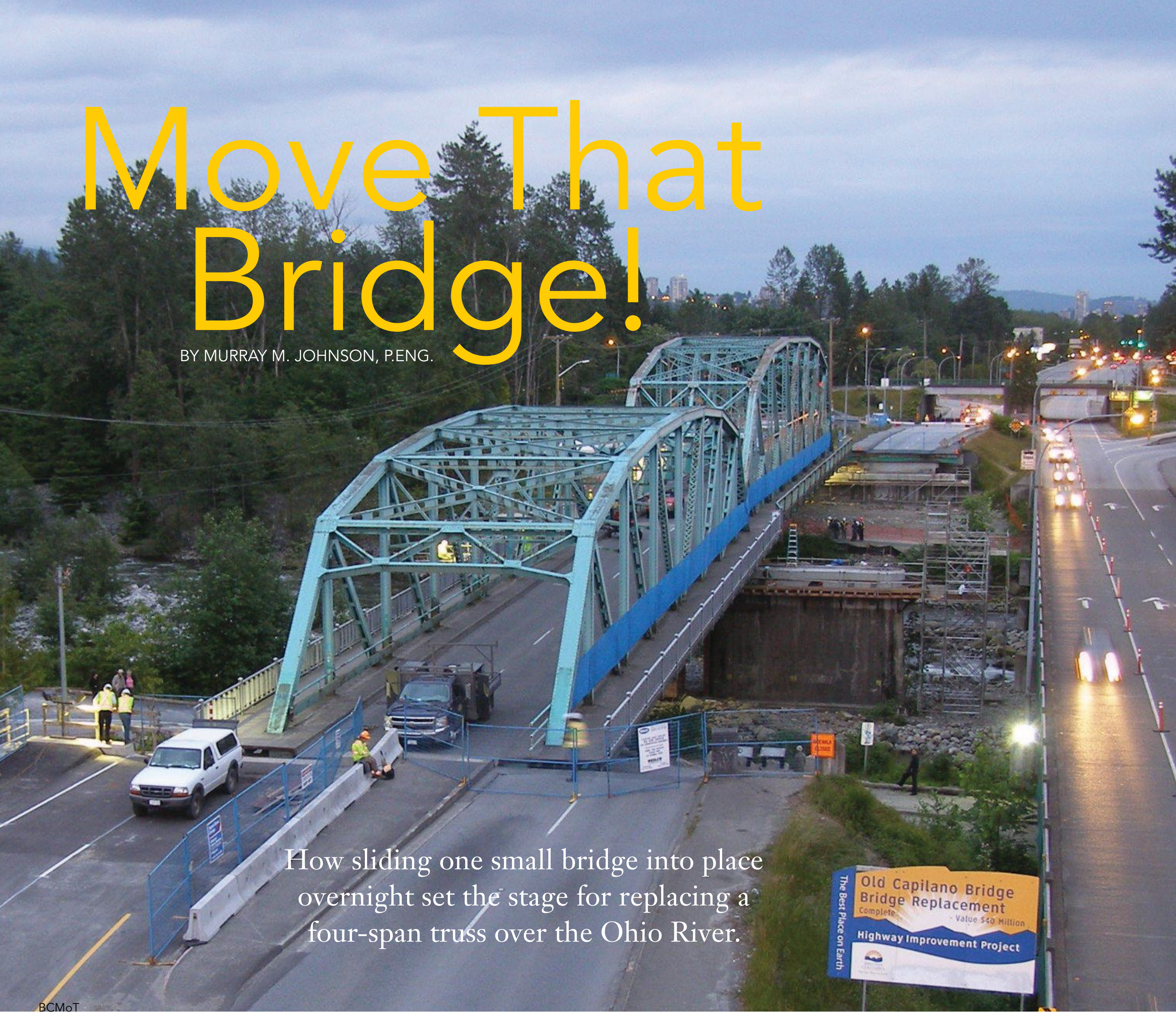
Steel Fabricator
Brooklyn Iron Works, Inc., Spokane, Wash. (AISC Member)

General Contractor
California Engineering Contractors, Inc., Pleasanton, Calif.

Move That Bridge!

BY MURRAY M. JOHNSON, P.ENG.

How sliding one small bridge into place overnight set the stage for replacing a four-span truss over the Ohio River.



BCMoT

▲ The Old Capilano River Bridge halfway through the sliding operation (moving right to left in photo). Note one lane of westbound traffic being accommodated on normally eastbound bridge at far right.

Murray Johnson, P.Eng., is an executive engineer with Buckland & Taylor Ltd., North Vancouver, British Columbia, specializing in bridge construction engineering. He can be reached at **mmj@b-t.com**.

REPLACING A VITAL BRIDGE carrying busy traffic loads over water, with limited or onerous alternate routes, is a serious undertaking. It generally requires either a new alignment, some form of staged construction that maintains partial traffic while the bridge is replaced around it, or construction of a temporary detour bridge. Existing road connections, built-up urban areas, other infrastructure, right-of-way limitations, or other site constraints often preclude a new alignment. Staged construction usually disrupts traffic and is always more expensive than building the new bridge in one pass, and not even structurally feasible for some bridge types. Sometimes a detour bridge may be built with a lesser length or to a lower standard, such as over a seasonally variable watercourse, but often the scale of the detour bridge required matches that of the permanent bridge itself. This can dramatically increase the project cost and delay or even prevent an owner from proceeding with it due to funding limitations.

One innovative technique for providing the detour for a bridge replacement, while keeping the original alignment, involves using either the old bridge or the new superstructure as the detour and

➤ The Old Capilano River Bridge during sliding, with bridge halfway from old river pier onto temporary pier. A one-minute time lapse video of the slide is available on YouTube at **http://bit.ly/zT6dZ5**.

employing lateral sliding of the superstructure during construction to reposition it during a short closure. Lateral sliding of bridge superstructures has been used as a construction technique on smaller girder bridges such as highway overpasses, but is less common on larger spans.

In June 2010, the 80-year-old Capilano River Bridge, in West Vancouver, British Columbia, was slid sideways onto a temporary pier and abutments during an overnight closure and reopened to traffic in the morning, becoming an instant, low-cost detour while a replacement bridge was built in the original location. Near the end of 2012, the new Milton Madison Bridge, spanning the Ohio River between Kentucky and Indiana, will be closed for just a few days while it is slid sideways, from the temporary piers upon which it is being constructed onto rehabilitated and enlarged original piers, after serving as the traffic detour while the old bridge superstructure is demolished.

In both cases, the sliding technique allows the projects to be built while minimizing disruption to traffic, accelerating construction, and reducing costs considerably. The difference is one of magnitude: The two-span, 430-ft, 1,280-ton Capilano River Bridge slide will be scaled up dramatically at Milton Madison, where four steel truss spans measuring 2,430 ft and weighing 15,260 tons will be slid into place.

The Capilano River Bridge

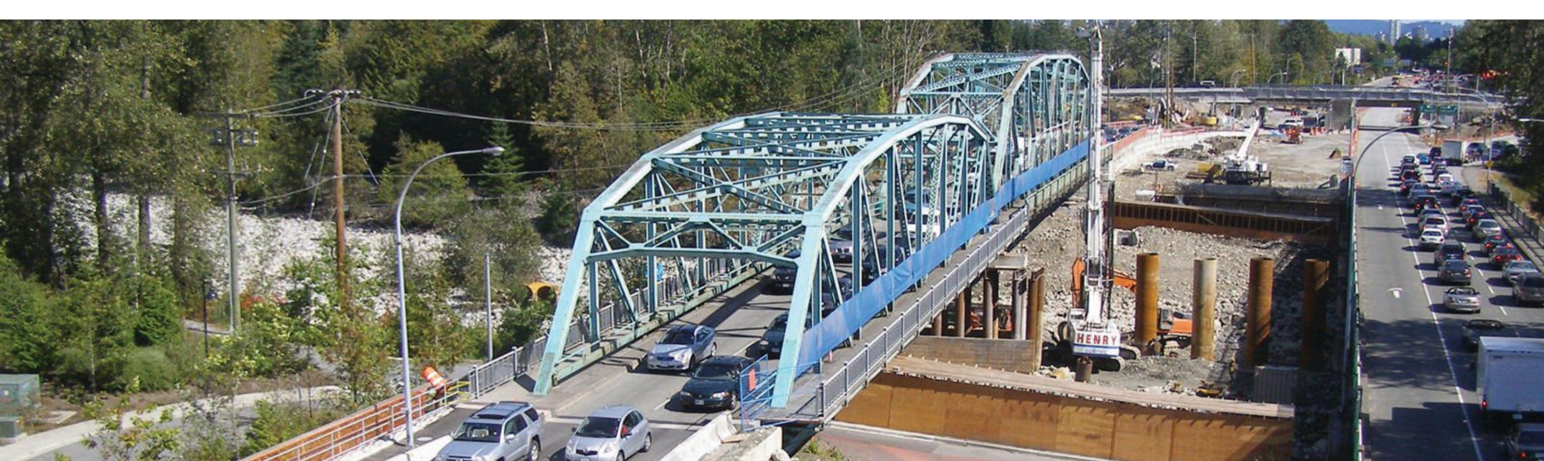
The Capilano River Bridge carries all westbound traffic on Marine Drive from North Vancouver and off the iconic Lions Gate Bridge from Vancouver, over the environmentally-sensitive Capilano River. Originally built in 1929 as a single 250-ft steel truss span with short jump spans at each end, a second 180-ft steel truss was added after a 1949 flood washed out the west bank, abutment and jump span and widened the river.

By 2009, the two shoulderless narrow lanes of the bridge were a bottleneck for the more than 25,000 vehicles using it each day. Pedestrian and cyclist accommodation was poor, transit improvements were needed, and the bridge was deemed to be functionally obsolete. The bridge owner, the British Columbia Ministry of Transportation and Infrastructure (BCMoT), had longer-term plans for replacement of the bridge when funding help was suddenly offered under the Canadian federal government's infrastructure stimulus program.

▲ Workers monitor progress, keeping a close eye on tight clearances, as the Old Capilano River Bridge slide continues.

➤ Truss sliding runway at the Old Capilano River Bridge piers during the sliding operation.

▼ The Old Capilano River Bridge in operation as the detour while new bridge substructures start to take shape. Wood hoardings at both ends of site accommodated pedestrian traffic through the bridge site throughout construction.



New steel girders being launched from the Capilano River's east bank, over the river pier and closing in on west abutment.

The completed New Capilano River Bridge.

The catch: in order to receive the funding, project completion was required in less than two years, including design and construction. This tight deadline was further complicated by additional schedule constraints related to the upcoming Winter Olympics, during which roadwork was banned, and very limited in-stream working windows due to the salmon-bearing values of the river.

The BCMoT dove into planning for the project. With the help of a study by Buckland & Taylor Ltd. (B&T), international bridge engineers headquartered in North Vancouver, British Columbia, the solution chosen was to slide the old bridge superstructure upstream onto temporary supports to become the construction detour, thereby exposing the original alignment for demolition of the old abutments and pier and construction of a new, wider bridge. Less than three months from the conception of the project, construction of detour approaches was well under way and tenders had been called for building a temporary pier.

Two months later, in September of 2009, the temporary pier had been finished within the short "fish window" period, and left for the main contractor to slide the bridge onto in 2010. This pier, which would be the sliding runway as well as support the bridge in temporary service, used a forest of steel pipe piles drilled into the river bed, topped by steel W-shape cross-caps and a heavily reinforced concrete cap/sliding beam which was directly connected to the old concrete pier.

Design of the new bridge continued through the winter while the temporary approaches, using modular concrete blocks and geogrid-reinforced fills, were completed. A construction contract was awarded and sitework began on April 1, after the Olympics shutdown.

Schedule again was a factor, as the old bridge had to be slid out of the way in time to allow in-stream work on the new bridge to take place during the summer "fish window." To speed the process, the required design for the sliding was included in the tender documents with only details, equipment and work procedures to be added by the contractor.

Moving Old Trusses

The old steel trusses were supported on pinned shoes at each bearing location, some fixed, some on steel roller nests. After installing steel sliding tracks along each runway, the trusses were jacked up during short night closures and sliding shoes—steel plates with PFTE pads—inserted under each bearing. Old rollers were removed and replaced with stacks of steel plates. Although the two truss spans were structurally independent, the 1949 truss shoehorned onto the pier around the existing 1929 truss. That meant the bearings at the pier were nested and had to be dealt with together, and a common sliding shoe was inserted here.

The bridge was rotated in plan as it was slid, to suit the required alignment of the detour, so a different pulling speed was required at each of the three support lines. Moving the bridge was done using pairs of hydraulic jacks pulling on high-strength threaded bars, cycling up to 6 in. at a time.

The sliding design eliminated vertical jacking requirements at the conclusion of the slide, saving money and especially time. The detour approach roadways had been carefully positioned so that when the bridge arrived in the new location, it was vertically aligned and traffic could flow as soon as the deck joints were covered. The PTFE sliding elements that were the lateral sliding element became the longitudinal sliding bearings for the bridge in its new service.

Prior to the scheduled sliding date, a test slide was required, moving the bridge 1 in. then stopping, in order to test equipment, communications and control. After a few minor adjustments, one Saturday evening traffic on the bridge was diverted and the bridge closed for the sliding operation. The steel tracks were greased and the bridge was moved along its curved path, held on course by a single guide track at the pier, arriving in the detour location less than 6 hours later. The remainder of the night was spent installing restraints at the bearings, covering the deck joints, and repositioning roadway barriers. The bridge was reopened to traffic the following morning, with many drivers hardly aware that it had been moved. With the site now available, construction started immediately on the new bridge.

The most economical design for the new bridge was determined to be a two-span continuous steel plate girder structure with a cast-in-place composite deck and integral abutments, without deck joints. The single pier in the river and both abutments are supported on steel pipe piles. Because the profile of the roadway over the bridge includes a symmetric vertical curve with a rise of about 2 ft, by keeping the girder bottom flange horizontal the girder depth varied from 5 ft at the abutments to 7 ft over the pier, neatly accommodating the higher bending moment demands over the pier. Five steel plate girders of 50 ksi weathering steel at 12-ft spacing support a 57-ft-wide cast-in-place concrete deck carrying three lanes of traffic, shoulders and a wide shared pedestrian and cycle path. The contractor chose to launch the steel girders across the river from one bank, bolting two of the three sections together, pushing them partway out, then bolting on the remaining section in the limited site space before finishing the launch.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Kicking it Up a Notch With a Design-Build Innovation

Design-build bids were solicited for the Milton Madison Bridge replacement over the Ohio River near Madison, Ind., in June 2010. The joint owners of the bridge, the Kentucky Transportation Cabinet and the Indiana Department of Transportation, had studied the bridge replacement issue extensively with their engineers and had arrived at the concept of rehabilitating most of the existing piers and replacing the superstructure. Although the existing steel truss bridge, built in 1929, is very narrow, without shoulders or sidewalks, deteriorating, and functionally obsolete, it is the only crossing in a 72-mile stretch of the Ohio River and is vital to the communities it serves.

Bidders were to replace the entire superstructure with a new wider four-span continuous steel truss bridge with new concrete girder approach spans. Four of the five piers supporting the new main span's superstructure would be rehabilitated existing piers. The roadway and sidewalk requirements, as well as the general dimensions and some design features of the truss, were pre-established to meet the pier constraints as well as the requirements of a public consultation process for replacing the historic but deteriorated bridge. Main truss spans would be 600 ft, 600 ft, 727 ft, and 500 ft, with 48-ft center-to-center of trusses.

Bid documents included a formula to establish the effective bid price. To the contractor's construction price would be added an amount equal to $25,000 per day for every day the bridge was closed, limited to a maximum of 365 days. In addition, two completion dates, September 2012 or May 2013, were allowed, with a deduction of $3.75 million from the effective bid price for committing to the earlier date. A round-the-clock ferry would have to be operated for the closure period of the bridge.

Walsh Construction Ltd., Crown Point, Ind., teamed up with Burgess & Niple, Inc. (B&N), Columbus, Ohio, and Buckland & Taylor to bid the project. B&N would design the approaches and the pier rehabilitation while B&T would perform both the design and the construction engineering for the steel main spans. In addition to coming up with efficient designs for the new permanent structure, the challenge for the bid design team was finding an innovative solution that would eliminate the need for a long bridge closure and reduce construction risk associated with schedule.

The team developed a bold solution building on B&T's recent success with sliding the Capilano River Bridge. The existing bridge would remain open to traffic while beginning the pier rehabilitation, while the new bridge superstructure would be completely constructed alongside on temporary piers. Traffic would then be diverted onto the new structure and the old superstructure and pier tops demolished to make way for the completion of the new pier caps. Temporary access ramps for traffic would allow the new approaches to be completely built in their final position. Finally, when all was ready, the bridge would be closed for a few days at most and the entire new superstructure slid into final position.

With a bid price of $103.7 million, a bridge closure bid of only 10 days, and the earlier completion date, Walsh was the successful bidder. The design of the new steel truss bridge, a 2,430-ft continuous truss built using 8,200 tons of high-performance 50 ksi and 70 ksi steel, with a continuous concrete deck on floating bearings, is remarkable but beyond the scope of this article.

Tons of Temporary Steel

The temporary works associated with the plan are extensive and involve a total of some 3,200 tons of steel piling and fabricated steel. In accepting the concept of operating public traffic on a bridge on temporary supports, the owners required that the design criteria be essentially the same as for a permanent structure. The most significant consequence of this is related to the ship impact

Deborah Crawford

➤ New Span 2 trusses for the Milton Madison Bridge being assembled on barges at Kentucky bank, shown one-third complete.

▼ Overall view, looking from Indiana to Kentucky, of the existing Milton Madison Bridge (left). New Span 2 trusses being assembled on barges at Kentucky bank, shown halfway complete.

Murray Johnson

▼ New Span 2 trusses for the Milton Madison Bridge being assembled on barges at Kentucky bank.

Murray Johnson

◄ Welded steel barge impact frames for Temporary Pier 3 on the Milton Madison Bridge project, doubling as pile driving template.

► Welded steel box beams for the Milton Madison Bridge temporary piers.

loadings required for the temporary piers, because long trains of heavily loaded barges operate on the Ohio River. Complicating this is a highly variable water level at the bridge location. These factors result in a temporary pier design with massive steel barge impact frames at three levels, heavily connected to the strengthened permanent pier stems and protecting the six 36-in.-diameter steel pipe piles supporting the temporary towers. These will include 1,250 tons of steel in barge impact frames and temporary pier towers, as well as 1,400 tons of steel pipe piles.

The two truss spans over the main river channel are being assembled on barges against the Kentucky shoreline, and will be floated out one at a time and lifted some 80 ft into place using strand jacks. To accommodate the lifting, three of the temporary piers will have lifting towers added on top, while the end of one span will be lifted by jacks perched on top of the new truss top chord. Once lifted, the truss spans will be set on heavy steel box girders, 101-in. deep and 78-in. wide, which double as the top caps for the temporary pier supported on two levels of sub-caps and, eventually, as sliding runways. Hillsdale Fabricators, St. Louis, is producing some 520 tons of these temporary girders, using plate up to 3-in. thick as well as associated support steel.

Once the two central truss spans are lifted and secured, the side span trusses will be erected piecemeal, using cranes on barges and land, cantilevering toward the river banks. Intermediate erection bents will take the load near river's edge and jack the trusses to allow them to land on the two end temporary piers. Padgett, Inc., New Albany, Ind., is fabricating 75 tons of temporary sliding girders and pier caps for these end piers. Following completion of steel erection, the new bridge superstructure will be completed with concrete deck, sidewalk, barriers, and temporary expansion joints, and linked to the temporary approach ramps to start carrying traffic.

Sliding

The sliding will come after demolition of the old bridge superstructure and completion of the new pier caps. Similar to the Capilano River Bridge, the new bearings and the sliding process have been designed so that on completion of sliding, no vertical jacking is required. When the PTFE element built into the truss bearing for lateral sliding arrives in its final position, it will simply be fastened to the embedded bearing plate.

The sliding track at each pier will include the top flange of the temporary pier girders, the masonry plates, and sliding plates set on the pier concrete between the bearing plates. Bearings that in permanent service will be sliding bearings will be temporarily locked together for the lateral slide, and also as for Capilano, the entire structure will be guided along a path at the center pier, allowing thermal movements to occur in both directions during the course of the slide.

On sliding day, the entire superstructure will be moved 55 ft upstream to its new position, pulled by strand jacks linked to a computerized, displacement-monitoring control system. One adjacent approach span will also be separately slid into place, and then expansion joints will be completed at the bridge ends and the bridge reopened to traffic in its permanent position.

In meeting the goals of accelerating bridge replacement, providing efficient detours for unrelenting traffic, and building cost-effective new bridges, lateral sliding of bridge superstructures, longer and heavier than ever, is one more tool that designers and builders can employ. The success of the Capilano River Bridge project has helped develop the techniques that are now being used on a much larger bridge, and undoubtedly will be used on many more future bridge projects, as owners and contractors try to meet the demand for faster, more efficient, less disruptive and more sustainable construction. MSC

Capilano River Bridge

Owner
British Columbia Ministry of Transportation & Infrastructure

Structural Engineer
Buckland & Taylor Ltd., North Vancouver, British Columbia

General Contractor
Neelco Construction Inc., Chilliwack, British Columbia

Milton Madison Bridge

Owners
Indiana Department of Transportation and Kentucky Transportation Cabinet

Owner's Engineers
Michael Baker Jr., Inc., Louisville, Ky., and CDM Smith, Indianapolis

Structural Engineer (Main Spans Design and Construction)
Buckland & Taylor Ltd., North Vancouver, British Columbia

Structural Engineer (Pier Rehabilitation, Approach Spans, and Temporary Ramps)
Burgess & Niple, Inc., Columbus, Ohio

General Contractor
Walsh Construction Ltd., Crown Point, Ind.

Steel Detailer (New Bridge)
Tensor Engineering, Indian Harbor Beach, Fla. (AISC Member)

Steel Fabricator (New Bridge)
High Steel Structures Inc., Lancaster, Pa. (NSBA/AISC Member)

Steel Fabricators (Temporary Works)
Hillsdale Fabricators, St. Louis (AISC Member) and Padgett Inc., New Albany, Ind. (NSBA/AISC Member)

Box-shaped beams offer thin floors, flat ceilings, long spans and quick erection.

Steel Support for a Slim Floor System

BY MARK JOHANSON

AN UNUSUAL STEEL BEAM SYSTEM recently had its first installation in the U.S., providing the solution to a headroom problem at the Dwell Bay View project being constructed in Milwaukee. Using Peikko's Deltabeam, the project team was able to increase the clearance in the vehicle pathway and enable access for extra-high "accessible" type vans without having to rework any footing elevations.

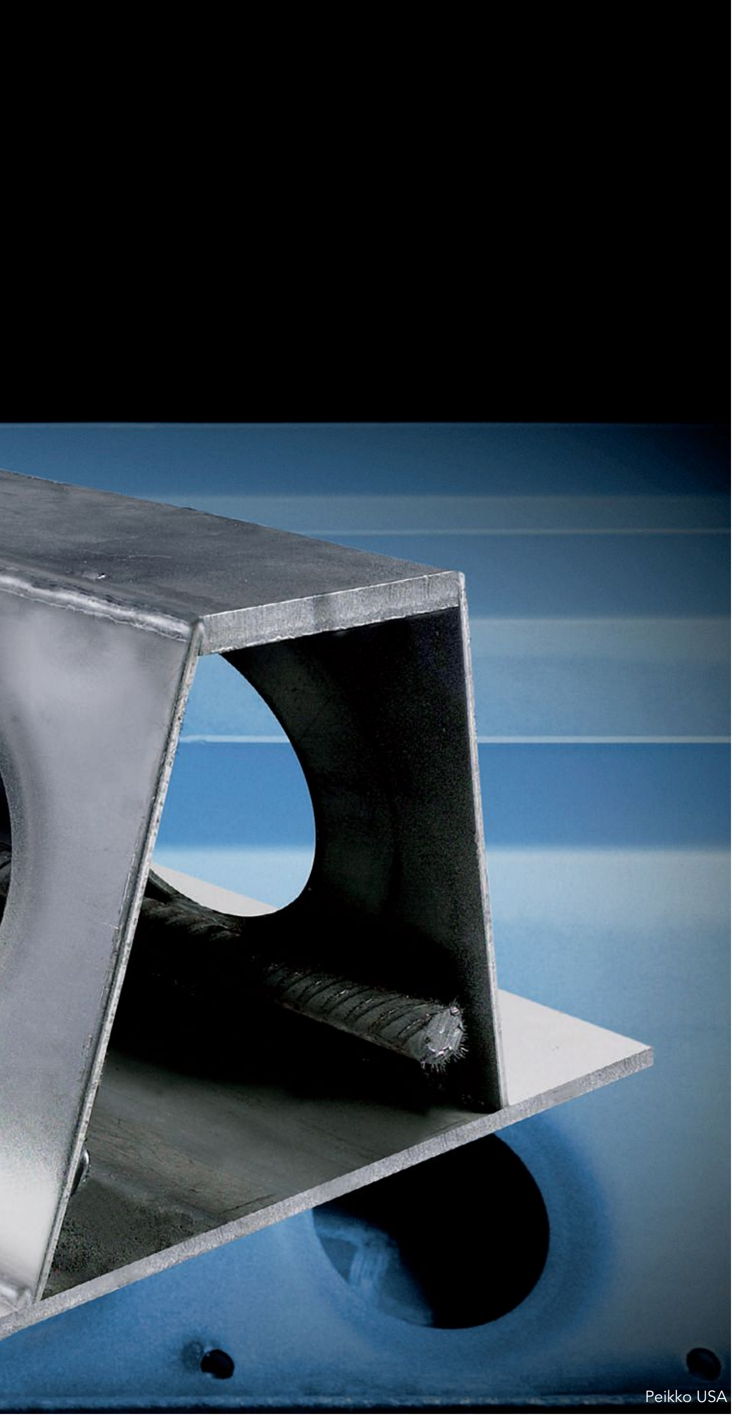
The five-story structure is largely precast concrete construction, but early in the preconstruction meetings the precast supplier, South Beloit, Ill.-based Mid-States Concrete Industries, realized there was a problem with the required van access. Mid-States recommended using the Deltabeam in the areas that required van access as the solution. The solution developed by SEOR Structural Dimension Inc., Brookfield, Wis., and accepted by project architect Engberg Anderson Architects, Milwaukee, was to use Deltabeams to support the hollow-core plank along the vehicle travel. Although it involved just five 11-in.-deep beams, the steel solution kept the project on track and blended seamlessly with the rest of the construction.

The Deltabeam does cost slightly more than a comparable precast beam, according to Mid-States' vice president of preconstruction Jeremy Olivotti. "It is a great system for the right project," he said.

A Thin Floor

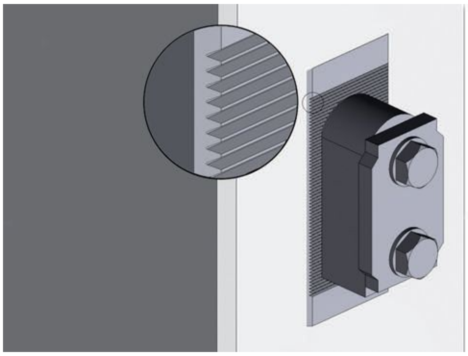
The system combines Deltabeams with precast hollow-core slabs of the same thickness to create a uniform slim floor that can be erected rapidly as well as offer long yet thin spans. It has been used throughout Europe and Canada for a number of years in a variety of structures.

Through the composite action of Deltabeam and precast slabs, the floor assembly can be as thin as 9 in. without a downstanding beam and span as much as 45 ft—all while being lighter than a cast-in-place concrete floor. After casting the grout infill,



Peikko USA

Available in lengths up to 45 ft, Peikko's Deltabeam consists of four steel plates welded together and pre-cambered.



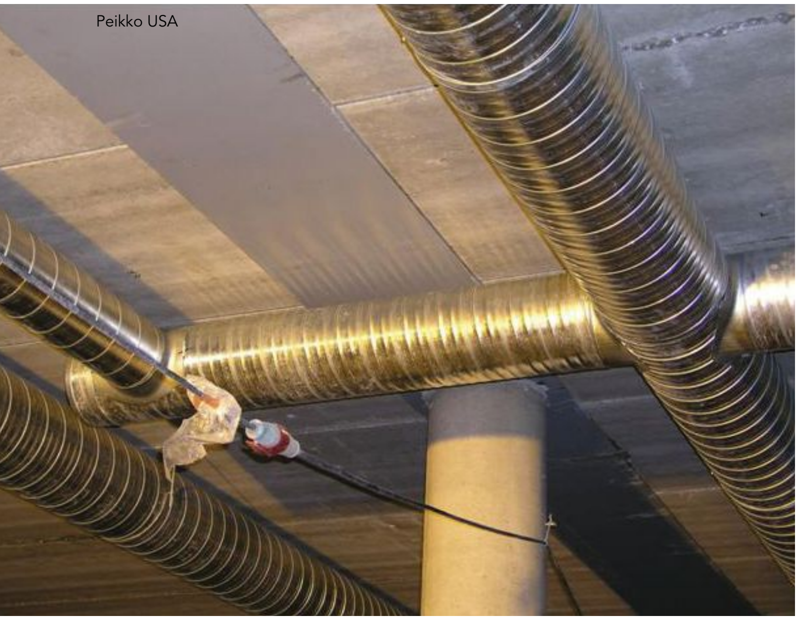
Three photos, above: The key to the Deltabeam's quick erection is the end connection. One piece of the corbel is welded to the steel column in the fabricator's shop. The second piece is bolted to the first and provides vertical adjustment. Grooves on the interfacing surfaces maintain the connection's high capacity. The slotted plate welded to the end of the Deltabeam slips over the corbel and the connection is done.

the Deltabeam becomes nearly invisible, hidden in the floor: only the bottom flange remains visible from the floor below. Having a thin floor means using minimal space on each floor, therefore minimizing the total height of a building or in some cases, having one additional floor for the same building height. Additionally, the integration of building services, especially for large ventilation ducts, becomes simpler and less costly thanks to the uniform, flat ceiling.

Beam Design

The Deltabeam consists of four steel plates—two side plates, one top plate and one bottom plate—of the same length, ranging from about 10 ft to 45 ft, that are welded together and pre-cambered. Rebar spans the full length of the beam. Forged steel studs are welded under the top plate, and end plates are welded at the two ends of the beam. The side plates are perforated every 12 in. with either 3-in. or 6-in. diameter holes.

Mark Johanson is the marketing director of Peikko USA Inc. and an AISC member. He can be contacted at **mark.johanson@peikko.com**.



Peikko USA

▲ The uniform, flat ceiling achieved by using Deltabeams simplifies installation of building services, especially large ventilation ducts.

▼ In their first U.S. installation, using Deltabeams to support the hollow-core floor over the traffic lane solved a headroom problem at the Bay View Housing project in Milwaukee.

Structural Dimension Inc., Brookfield, Wis.

Engberg Anderson Architects, Milwaukee

▲ The Dwell Bay View Housing project being constructed in Milwaukee.

These holes enable the beam to be filled with grout and create the composite action between the steel beam and the concrete. The bottom plate, which projects approximately 5 in. on each side of the beam, serves as support for the hollow core. The top plate has air holes to ensure that the beam is entirely filled with concrete and hook holes for lifting and erection.

The rebar and steel studs reinforce the beam as well as enhance its fire rating. Through physical fire tests done according to ISO 834, the Deltabeam with its exposed bottom flange can withstand fire for up to three hours. Through the composite behavior developed between the steel and concrete, the Deltabeam's bottom flange is no longer load bearing when the concrete in the beam has cured. Physical fire tests are also being done with Underwriters Laboratories (UL) in the spring of 2012.

Last but not least, the steel plates welded at each end of the beam make for very fast erection. No onsite bolting or welding is required to install the Deltabeam. Typically a high shear capacity corbel is welded to the steel column as part of the steel fabrication. That provides the connection point for one of the beams enabling it to be simply supported between two columns. A similar corbel assembly can be embedded for attachment to concrete.

Fast Four-Step Erection

The four-step construction process involving this steel beam can bring erection speed to an average of 8,000 sq. ft per day. Four components are required to make the flat slab assembly: Deltabeams, precast slabs, rebar and minimum onsite grout.

First, Deltabeams are erected in place. Each beam erection can be done in less than a minute, depending on the size of the beam. Next, precast hollow-core slabs are placed on the projecting edges of the Deltabeam's bottom plate. Like the beam placement, this is a simple operation requiring no special trade workers onsite. Because the Deltabeam is a box-shaped beam, it has a high resistance to torsion. It therefore requires minimal shoring near the beam-to-column connections, though shoring can even be avoided altogether through special design of the corbel connection. This also allows different trades to have access to the site earlier, thus reducing the overall time of construction.

The assembly of the slabs also involves placing rebar through the beam and between slabs. Because both Deltabeams and hollow-core slabs are reinforced when produced, the quantity of rebar required to be placed at the job site is small. The only reinforcement needed is placed through the web holes of the Deltabeam and in hollow-core slab keyways every 4 ft.

Finally, the beam and keyways are entirely filled with grout to complete the composite action between steel and concrete. At this point, workers can walk freely on the Deltabeam/hollow-core assembly, making this last operation easy to do and safe. Due to the limited areas where grout is required, this step can be done with minimal heating and hoarding in case of winter conditions. The trapezoid shape of the steel beam leaves enough space between beam edges and precast slab extremities to grout without notching the top of each hollow core end, a practice often mandatory when using other slim floor systems.

If desired, a fourth step consisting of placing a leveling topping can be done to provide a better floor finish. This is typically done after most of the exterior cladding is installed, especially during winter to reduce heating and hoarding costs.

More information is available online at **www.peikousa.com**. MSC

CONFERENCE

Register for NASCC: The Steel Conference Now and Save

With the opening of the 2012 NASCC: The Steel Conference just around the corner, there's no time like the present to complete your registration at **www.aisc.org/nascc**. The conference is scheduled for April 18-20 at the Gaylord Texan Convention Center in Dallas.

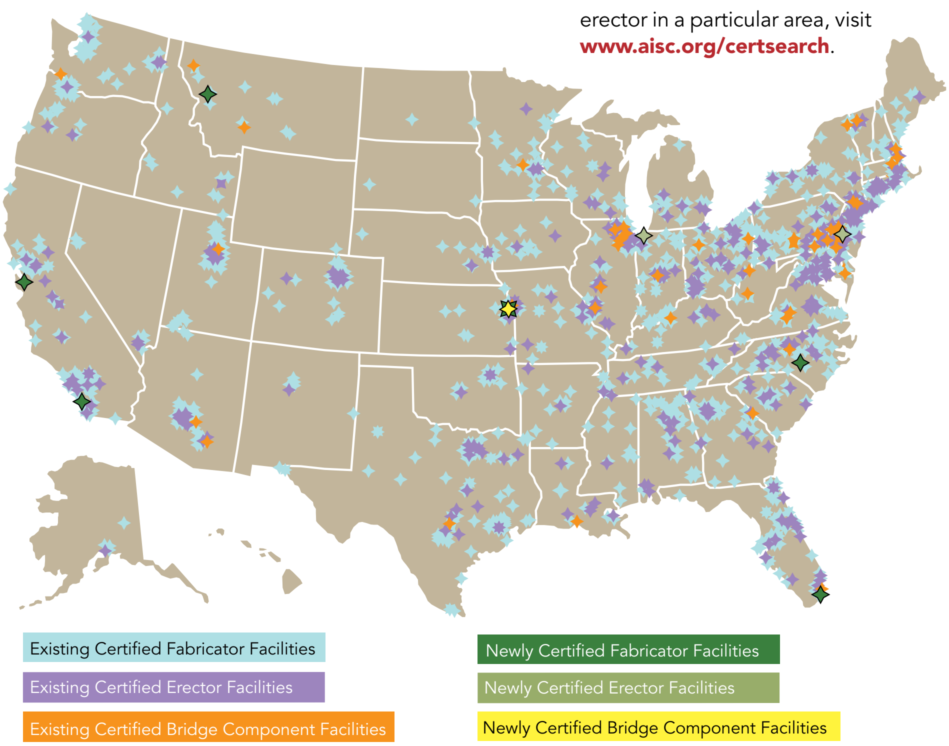
For the first time, this year's Steel Conference, Annual Stability Conference and Sustainable Steel Conference join together with the World Steel Bridge Symposium, the Technology in Steel Construction Conference and the brand new Ruby University, featuring practical sessions on constructability. In addition, AISC member Gerdau is offering a free mill tour for attendees who arrive early (sign-up is required).

For more information about what's in store for you at the conference this year, view AISC's press release at **http://bit.ly/upKRs0**. Also, you can start planning now to get the most out of the technical sessions, networking and product showcase happenings by downloading the Advance Program at **www.aisc.org/nascc**.

Remember, with this year's discount structure the sooner you register the more you save. The price goes up $10 each Sunday at midnight between now and the conference, so don't delay.

Newly Certified Facilities: December 1–31, 2011

To find a certified fabricator or erector in a particular area, visit **www.aisc.org/certsearch**.



Newly Certified Fabricator Facilities

BP Fabrication, Inc., Missoula, Mont.
Campbell Certified, Inc., Oceanside, Calif.
George's Welding Services, Inc., Miami, Fla.
Shawnee Steel & Welding, Inc., Merriam, Kan.
SOS Steel Co., Inc., Santa Clara, Calif.
Steel and Pipe Corporation, Sanford, N.C.

Newly Certified Erector Facilities

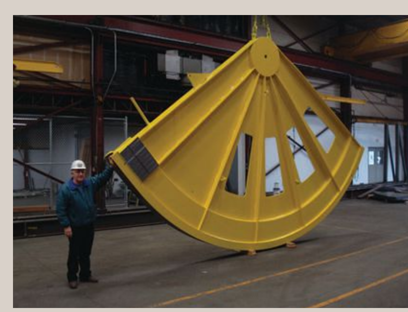
Graycor Industrial Constructors, Portage, Ind.
Northwest Erectors, Inc., Ambler, Pa.

Newly Certified Bridge Component Facilities

Shawnee Steel & Welding, Inc., Merriam, Kan.

People and Firms

- **Andrew W. Herrmann**, P.E., recently was sworn in as the 143rd president of the American Society of Civil Engineers at its annual convention in Memphis, Tenn. A principal with the engineering firm Hardesty & Hanover, LLP, New York City, Herrmann is based in Pittsburgh. He most recently served as chairman of the Committee for the 2009 Report Card for America's Infrastructure. He also was the lead spokesperson for ASCE's recent report, "Failure to Act, The Economic Impact of Current Investment Trends in Surface Transportation Infrastructure," the first-of-its-kind report that applied a price tag to the U.S. surface transportation system if nothing is done to improve it.
- AISC member **Jesse Engineering**, Tacoma, Wash., recently delivered a large-radius rotary draw pipe bender to a customer in British Columbia. Primarily to be used in the manufacture of coated pipe elbows for the oil and gas industry, the bender was furnished with capability for up to 10 in. IPS bends at 20 times diameter. Planned pipe bends include 6 in., 8 in. and 10 in. IPS at 15D and 20D.



- **Design Data** has been named an Autodesk AEC Industry Partner. Its recent release of SDS/2 Connect, an add-in for Autodesk Revit Structure software that automatically designs connections within the Revit environment, provided the foundation for the partnership. The product also was cited as a 2011 *MSC* Hot Product in the August 2011 issue, available at **www.modernsteel.com/backissues**.

IN MEMORY

John A. Martin, Founder of Martin Associates Group

John Alfred (Jack) Martin, S.E., 91, passed away on Saturday, December 3, 2011 in Redondo Beach, Calif. One of the preeminent structural engineers in the western U.S., he was the founder of the Martin Associates Group of engineering companies, including John A. Martin & Associates, Inc. and Martin/Martin, Inc. Martin received AISC's Lifetime Achievement Award in 2004 in recognition of his half century of work blending science and art, technology and creativity, and innovation and safety.

Martin's unique combination of intellect, tenacity and pragmatism brought invaluable lessons to the engineers, architects and contractors he worked with. In a 1998 interview he remarked, "I encourage young engineers to visualize and learn the details of what they are designing, and have a feeling for the size of what they are creating and connecting, before getting into the computer and performing calculations." This philosophy remains today at the core of the Martin organization's work process.

The firm has worked on skyscrapers, condominiums and casinos including the Los Angeles, Anaheim, and Long Beach convention centers, the Staples Center Arena in Los Angeles, and the LAX business district.

A private memorial service was held on New Year's Day (January 1, 2012). He is survived by his eldest son and CEO of John A. Martin & Associates, John A. "Trailer" Martin, Jr. of Redondo Beach, Calif., daughter-in-law Patricia and grandchildren John A. "Trip" Martin, III, Michael Bauer and Margaret "Peggy" Martin; his son Randall Martin and granddaughter Julia Tiffany of Newport Beach, Calif.; and his daughter Jamie Martin and grandsons Jack Otto, Brett West, and Reed West of Denver.

For additional information, visit **www.johnmartin.com**.

NEWS

Engineering Journal Q4 Now Online

The Fourth Quarter 2011 issue of *Engineering Journal* is now available online in digital edition format. View the current issue online at **www.aisc.org/ej**.

Papers in *Engineering Journal* Q4 include:

- "Experimental Investigation of Shear Transfer in Exposed Column Base Connections," by Ivan R. Gomez, Amit M. Kanvinde and Gregory G. Deierlein
- "Repairable Seismic Moment Frames with Bolted WT Connections: Part 1" and "Part 2," by Patrick S. McManus and Jay A. Puckett
- "Design of Steel Columns at Elevated Temperatures Due to Fire: Effects of Rotational Restraints," by Anil Agarwal and Amit H. Varma
- "Current Steel Structures Research No. 28," by Reidar Bjorhovde

Each quarterly current issue of *EJ* is available in digital format and free to the public until the next issue is published.

The complete collection of *Engineering Journal* articles is searchable at **www.aisc.org/ej**. Downloads of current and past articles in PDF format are free to AISC members and ePubs subscribers; just make sure you are logged into the AISC website (**www.aisc.org**) before searching. Nonmembers will be directed to the AISC Bookstore at **www.aisc.org/store** to purchase article downloads.

COMPETITION

Registration Deadline Nears for Student Design Competition

Students are encouraged to register as soon as possible for the 12th annual Steel Design Student Competition sponsored by the American Institute of Steel Construction (AISC) and administered by the Association of Collegiate Schools of Architecture (ACSA). Teams must register by February 15, 2012, to participate in this year's competition, although registration information may be modified until submission of the final project. Entries are due May 24, 2012. There is no fee to enter. Each team must have a faculty sponsor.

Two categories of competition are being offered again this year. Category I challenges architecture students to design a Culinary Arts College in an urban setting. "Steel construction offers students great benefits in this endeavor," the program materials state, "as it is ideal for covering long-spans without sacrificing flexibility and aesthetic lightness, multi-story buildings, quick delivery and assembly in congested urban environments." Category II provides an open design option.

Steel should be used as the primary structural material for entries in either category, with special emphasis placed on innovation in steel design. The structure also should include at least one space that requires long-span steel structure.

There are three possible prizes in each category. Winning students and their faculty sponsors will receive cash prizes totaling $14,000.

Learn more about the ACSA/AISC Steel Design Student Competition on the AISC website at **http://bit.ly/i8lwfk**, where links also are available to the full competition program including all the rules and guidelines.

AWARDS

AISC Honors Industry, Engineers and Educators

AISC awards honor significant projects and individuals who have made a difference in the success of the fabricated structural steel industry. Whether it's for an innovative design, an insightful technical paper, or a lifetime of outstanding service, an AISC award bestows prestige and well-deserved recognition upon its recipient.

Award recipients will be honored at the 2012 NASCC: The Steel Conference at the Gaylord Texan Convention Center in Dallas, April 18-20. For more information on The Steel Conference, visit **www.aisc.org/nascc**.

The Robert P. Stupp Award for Leadership Excellence, the J. Lloyd Kimbrough Award and the Geerhard Haaijer Award for Excellence in Education are AISC's highest honors.

Robert P. Stupp Award for Leadership Excellence

Daniel (Dan) DiMicco, chairman and CEO, Nucor Corporation, is the recipient of the Robert P. Stupp Award for Leadership Excellence. The Stupp Award is given in special recognition to individuals who have provided unparalleled leadership in the steel construction industry.

DiMicco has been a strong advocate of producer support and market breakthrough initiatives. He is an exemplary leader in the steel construction industry and an advocate in promoting domestic manufacturing jobs. As a former member of the AISC Board of Directors, he was instrumental in facilitating cooperation between mills and fabricators and served as an ambassador and an advocate of the fabricated structural steel industry in securing the support of domestic steel producers for AISC. Under his leadership, Nucor has not only championed technological advances in steelmaking but also environmental leadership. DiMicco graduated from Brown University in 1972 with a Bachelor of Science in Engineering, Metallurgy and Materials Science. He graduated with a Master of Science Degree in Metallurgy and Materials Science from the University of Pennsylvania, Philadelphia, in 1975. DiMicco joined Nucor Corporation in 1982 and was elected Chairman of the Board in 2006.

J. Lloyd Kimbrough Award

John M. Kulicki, P.E., Ph.D., chairman and CEO of bridge engineering firm Modjeski and Masters, is the recipient of the J. Lloyd Kimbrough Award for his significant achievements in bridge analysis and design. The Kimbrough award is presented on a selective basis by AISC to honor preeminent engineers and architects who have made an outstanding contribution to the structural steel industry through their design work.

"John's work has advanced the state of the art in every aspect of steel bridge design," said Bill McEleney, director of the National Steel Bridge Alliance (NSBA). "Aside from his signature achievement of leading the AASHTO LRFD bridge code development team, the diversity of steel bridge types he has designed across the country is a clear indication of the broad range of his design acumen. His willingness to share his expertise has benefited the bridge design community today and will continue to inform steel bridge designers of the future." A graduate of Lafayette College and Lehigh University, Kulicki has more than 40 years of experience in bridge analysis and design. He joined Modjeski and Masters in 1974 and has led analysis and design work for numerous bridge projects including suspension, cablestayed, long-span truss and arch bridges, and girder bridges. He supervised the design of such notable structures as the long-span Second Blue Water Bridge between Port Huron, Mich., and Point Edward, Ontario, winning a 1997 ASCE Outstanding Civil Engineering Achievement Award and a Prize Bridge Award by NSBA in 2000.

Lifetime Achievement Award

Tom Ferrell of Ferrell Engineering is nominated for his extensive contributions to the AISC *Specification* and *Manual* as a connection designer. He has generously and selflessly shared the experience, expertise and lessons he and his connection design company have learned to the great benefit of the design community and steel construction industry.

Louis F. Geschwindner's distinguished career of service to the design community tallied more than 40 years and culminated in his continuing service as professor emeritus of architectural engineering at the Pennsylvania State University. As an AISC vice president, Lou was responsible for leading the development of the 2005 AISC *Specification for Structural Steel Buildings* and the 13th Edition AISC *Steel Construction Manual*, as well as all other technical activities of the Institute. Today, he continues to serve on the AISC Committee on Specifications and to chair Task Committee 4, Member Design.

Lifetime Achievement Award (Continued)

James O. Malley of Degenkolb Engineers is being honored for his decades of contribution and leadership on the AISC Committee on Specifications Task Committee on Seismic Design. He was a key figure in helping AISC meet the challenges that followed the Northridge Earthquake, and his technical contributions helped to form the foundation upon which the continued usefulness and success of steel in seismic design is based.

Alexander D. Wilson is the Chair of the Steel Bridge Task Force and very active with NSBA members. He has been influential in the development of bridge material specifications and he was influential in the development of the latest high-performance steel (HPS70W). He has been this generation's key resource for metallurgical information on steel bridges.

Special Achievement Award

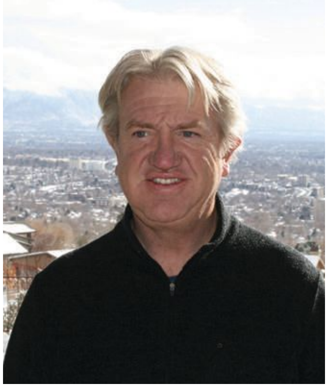
Robert J. Connor, Ph.D., Associate Professor, Purdue University, is being honored for his work on fatigue and fracture of steel bridge structures. Connor rewrote the AREMA and AASHTO fatigue specifications, and his field measurements are the basis for the double cycle counts in the fatigue of orthotropic bridges.

W. Samuel Easterling, Ph.D., Professor, Virginia Tech, is being honored for his research on composite construction, and his contributions to AISC 360, Chapter I, Design of Composite Members.

Special Achievement Award (Continued)

Eric Hines of LeMessurier Consultants is honored for his innovative use of long-span steel truss framing and cast connections in the Wind Technology Testing Center in Charlestown, Mass.

Keith Landwehr is being honored for his leadership in the AISC Certification Committee, his work on the 2010 *Specification*, and for his significant contributions to quality certification programs including helping in the development of Chapter N.



Jim McMinimee of Utah DOT is being honored for his leadership and innovation in steel framing solutions for a self-propelled modular transportation system.

John Parucki is being honored for his leadership and work as National Head Judge for more than 10 years in the Student Steel Bridge Competition.

For more information about the AISC awards programs, visit **www.aisc.org/awards**.

Structural Steel Fabrication companies in CA & AL looking for the following positions:

- Plant Manager
- General Manager
- Pipe Detailer (SolidWorks)
- Structural/Mechanical Engineer/ Draftsman (knowledge of SolidWorks and/or CADD)

KSF INC. Kern Steel Fabrication, Inc.

Must have 7+ yrs experience with knowledge of Steel Fabrication.

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Trina Gray, Kern Steel Fabrications, 627 Williams St., Bakersfield, CA 93305
Fax (661) 638-0872 | e-mail: trina@kernsteel.com | www.kernsteel.com

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION · AISC · FOUNDED 1921 · CERTIFICATION PROGRAMS

Following continued growth in 2011, the American Institute of Steel Construction (AISC) Certification Program is seeking a Certification Engineer to serve as the primary resource for all technical matters pertaining to AISC Certification programs and activities. He or she will develop and communicate technical requirements governing the certified work of the fabricated structural steel industry for integration with AISC specifications, standards and quality management principles.

A primary responsibility is to develop and maintain AISC Certification program administrative and technical requirements. An in-depth knowledge of technical requirements, building and safety codes and regulations related to the structural steel or construction industry is a must as well as practical knowledge and fundamental understanding of construction project engineering principles.

The ideal candidate will have a Bachelor's Degree in Engineering and at least 5 years work experience in a construction related field. A Professional Engineer (PE) License is a plus. This position is located in the AISC Chicago office. The ability to travel approximately 25% of the time and attend industry events is required.

If you are interested in applying for this position, please forward your resume and cover letter, including your desired salary requirements to:

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people to know

THE ENGINEER AS WRITER

Veteran engineer and educator Michel Bruneau has incorporated his understanding of earthquakes and structural engineering into an award-winning novel.

SINCE WHEN can engineers write fiction? That's a difficult question for Michel Bruneau to answer. Bruneau is the recipient of the 2012 AISC T.R. Higgins award, as well as a long-time lecturer and author of numerous technical papers, but the roots of his writing fiction go further back than any of that.

"Writing is something I've always enjoyed and I can't say when I began because it seems like there's never been a start," he said. A self-described child of Quebec's "Quiet Revolution," Bruneau grew up in a culture very much connected to a European outlook on life but with a growing regional identity.

As a result, he said, "all the literature that we studied in classes was not European, but by Quebec authors. And in those days, none of them would make a living writing—all had another primary job—so I never even contemplated the thought of becoming a novelist as a career."

Instead he began to study architecture, but because what he really wanted to do was design bridges and buildings, he soon transferred to civil engineering. Meanwhile, he continued to write.

In 1998 Bruneau published his first book, a collection of short stories written in French and in the spirit of much of the popular literature of that time. "These were very much dark stories with kind of a twist that makes you reflect a bit on things," Bruneau said.

"Writing is a solitary experience," Bruneau said, "and you only harvest the fruits of that labor when you get feedback. Of course, tastes vary and those with classroom experience are in a good position to know that one cannot please everybody, but all the feedback I received so far has been very good. And that is the part that is enjoyable—delivering a story that grabs the reader."

In fact, many of his non-French-speaking friends and colleagues were disappointed that they could not read his book of short stories, which is at least part of the reason he began writing in English.

In 2010, after an 11-year effort often interrupted by his other responsibilities, Bruneau finally published *Shaken Allegiances*. A fictional account of what ensues when a devastating earthquake strikes Montreal Island in the dead of an icy winter, the Kafkaesque tale was well-received by readers and critics alike. The book captured first place for regional fiction and second place overall in the fiction category in the 2010 Next Generation Indie Book Awards, an annual competition that recognizes outstanding independent authors and publishers worldwide.

Although Bruneau has always been comfortable writing in French, having been born in a town that was 99% Francophone, he said that for many years writing in English was more challenging. "You feel like you're writing correctly, but there's always that nagging doubt. So getting a prize was great, and it put that doubt to rest."

To learn more about Bruneau, both as novelist and engineer, visit **www.michelbruneau.com**. If you are at NASCC: The Steel Conference in Dallas April 18-20, be sure to catch his T.R. Higgins Lecture on steel design, which will be in English. MSC

Michel Bruneau, Ph.D., P.Eng., professor in the Department of Civil, Structural and Environmental Engineering at the University at Buffalo, Buffalo, N.Y. and award-winning author of the novel *Shaken Allegiances*. Left: The Quebec Bridge over the St. Lawrence River. Right: The U.B. Structural Engineering and Earthquake Simulation Laboratory (SEESL).