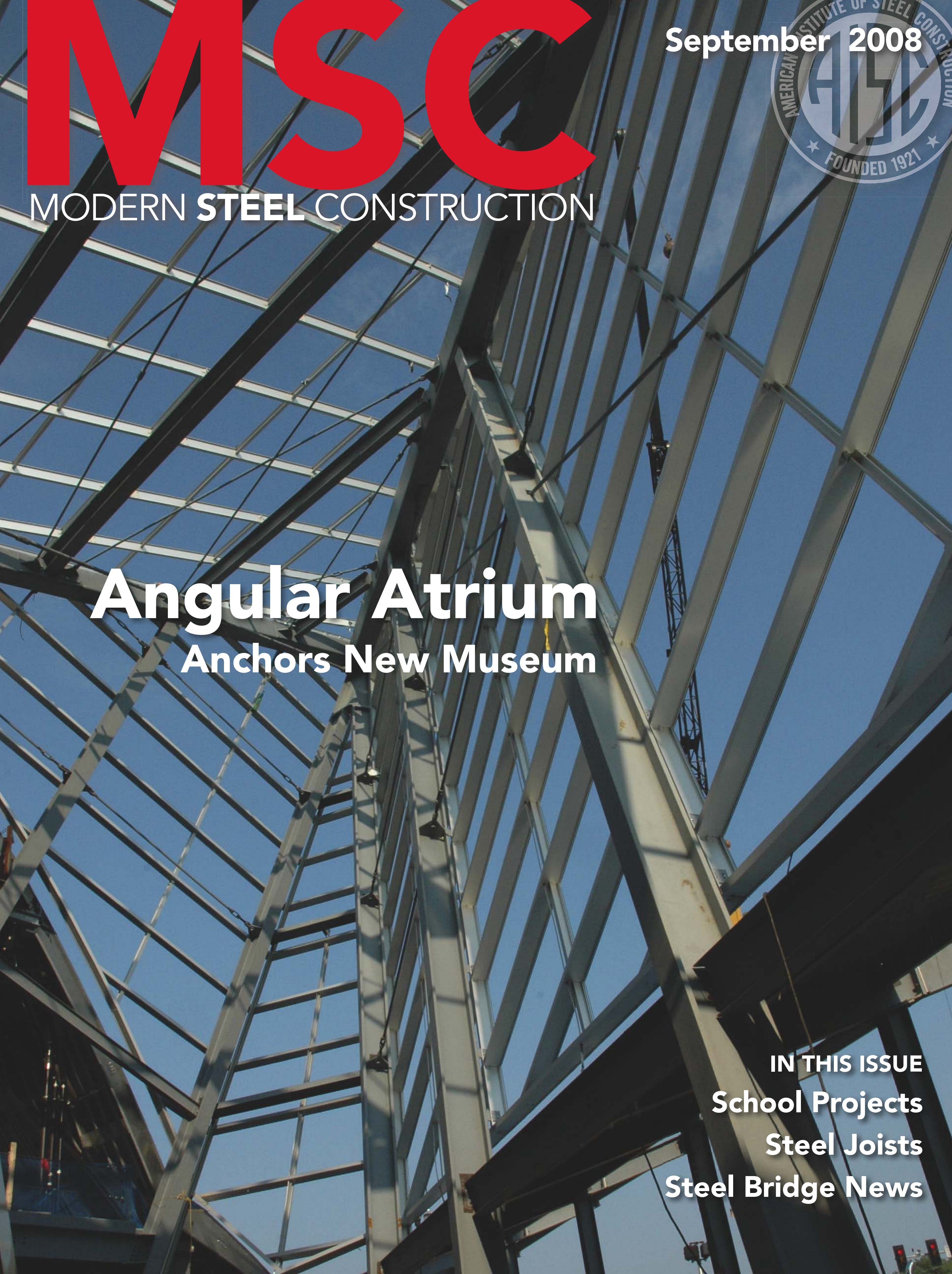
September 2008

# MSC
## MODERN STEEL CONSTRUCTION



# Angular Atrium
## Anchors New Museum

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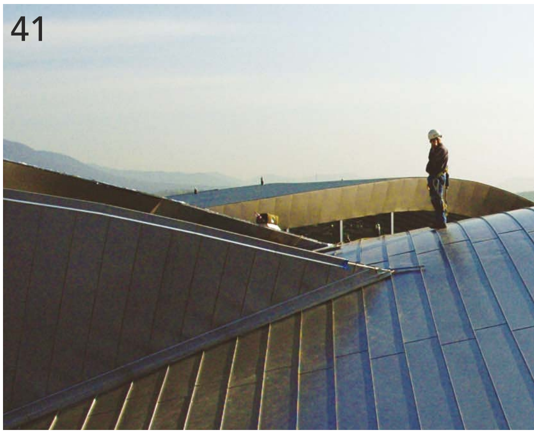



**ON THE COVER:** The Art Museum of Western Virginia in Roanoke. Photo: Randall Stout Architects, Inc.

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# editor's note

**IF YOU'RE UNDER 40, CHANCES ARE YOU DON'T KNOW FROMY ROSENBERG.** Under his quiet guidance, AISC has developed an extensive—and impressive—array of programs for both students and faculty. And while some of the programs pre-date him, his efforts have fostered and grown these important activities.

If you were a civil engineering student during the past two decades, you were probably invited to participate in your school's student steel bridge building competition. And if you did, chances are good that you met Fromy at the competition. This wonderful program pits teams of students against each other in the design, fabrication, and erection of a one-tenth scale bridge. The bridges are designed to be erected in just minutes—and also have minimal deflection when loaded with 2,500 pounds of weights. About 2,000 students participate in this program annually—or nearly 36,000 during Fromy's career!

Probably the most visible presence AISC has on college campuses comes in the form of steel teaching sculptures. These 8-ft-high forms (some are as tall as 15 ft) include most common structural shapes and connections and allow students to see and touch what they learn about in the classroom. The sculptures are donated by local AISC member fabricators and during Fromy's tenure, nearly 150 of these beautiful artworks have been installed.

AISC's most popular program for students, though, is its discounted manual program. Recognizing the importance of every engineering student having a steel manual and also the limited budgets that most students have, Fromy administers a program that works with faculty members and not only discounts the purchase price of the manual but also doesn't charge for shipping (quite a consideration when you consider the weight of the manual!).

One of Fromy's big success stories has been the advent of a program AISC conducts in conjunction with the Association of Collegiate Schools of Architecture. This eight-year-old design competition lets teams of students explore a variety of design issues related to the use of steel in design and construction. The program continues to grow and this year approximately 320 students from 30 universities participated.

Of course, Fromy's efforts aren't all directed just at students. Fromy has also been involved in AISC's Adopt-A-School program, where fabricators, producers, and service centers work closely with a specific school. And he's directed our faculty workshop program and administered our scholarship program. In addition, Fromy has put together an extensive set of teaching aids (which, while targeted to the university setting also have applicability in the professional office). To learn more about all of these programs, simply visit **www.aisc.org** and click on learning opportunities.

Frommy has recently retired, and the entire AISC and steel community will miss him.

**SCOTT MELNICK**
EDITOR

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

## *KL/r* Modified for Single-Angle

**A question and answer on this subject appeared in the January 2008 Steel Interchange (reprinted below). LeRoy Lutz, a member of the AISC Specification Task Committee covering the design of members, was kind enough to provide the following supplementary discussion pertaining to the design of single-angles: Single-Leg Angles per E5.**

This is a response to the comment made with regard to single-angle slenderness in section E5 of the AISC *Specification* in the January 2008 issue of MSC.

a) First, I was unsure what the designer meant when he indicated calculating $KL/r_z$ (I presume that he meant $L/r_z$). One considers the $KL/r$ to be an equivalent $L/r_z$ when designing.
b) The 0.95 and $0.82L/r_z$ limits given in E5 (a) and (b) are in the paragraph that addresses unequal-leg angles with the long leg projecting and does not apply to equal-leg angles.
c) The limits of 0.95 and $0.82L/r_z$ for equal-leg angles would occur when the $KL/r$ is at about the upper limit of 200, so that there would be no need to check those limits for equal-leg angles. For unequal-leg angles with the short leg projecting, those limits would occur well beyond an $L/r$ of 200.

Here is a short summary for stocky and slender equal-leg angles:

1) For an L3×3×¼ equal-leg angle with $L/r_z$ of 30 (and $L/r_x$ of 19), the $KL/r$ (i.e. $L/r_z$) calculated from (a) would be 86.2 and from (b) would be 75.2. The axial load design at these values of slenderness would account for strength reduction based on the load's eccentricity reduced some by the end restraint (as compared to a pinned-end member).
2) For an L3×3×¼ equal-leg angle with $L/r_z$ of 160 (and $L/r_x$ is 101), $KL/r$ (i.e. $L/r_z$) calculated from (a) would be 158 and from (b) would be 146. The axial load design at these values of slenderness account for a slight strength increase due to the end restraint and reduction based on the load's eccentricity (as compared to a pinned-end member).

*LeRoy Lutz*

***Original question/answer from the January 2008 Steel Interchange:***

**For a single-angle compression member, I followed AISC specification section E5 to calculate the modified $KL/r$. I also calculated $KL/r_z$, and it turns out to be greater than $KL/r$ modified. Should I use the larger of the two ($KL/r$ modified, or $KL/r_z$) in section E3?**

If you are in compliance with E5 (including attaching the angle using the longer leg) then you can use the limits on $L/r_z$ that are provided at the ends of the both sections (a) and (b). In the first case the limit is $0.95L/r_z$ and in the second case it is $0.82L/r_z$. In essence, with your condition, you are still designing for $KL/r_z$ but with a $K$ value of less than 1.0 because of the higher end restraints.

*Amanuel Gebremeskel, P.E.*

## Flexural Capacity of Channels

**Why are the maximum strong and weak axis bending stress values for channels limited to $0.6F_y$ and $0.66F_y$ respectively? The weak axis limit seems particularly conservative given that compact, doubly symmetric sections and plate have a $0.75F_y$ limit.**

I am not sure which document you are looking at, but in the 2005 AISC *Specification* (available at **www.aisc.org/2005spec**) channels in strong axis bending have an ASD limit of $0.66F_y$ and $0.9F_y$ for weak axis bending if you approximate a lower bound shape factor and use $S$ to make the comparison to older versions of ASD. The derivation for these is as follows:

For the weak axis, $M_n / \Omega = 0.9F_yZ_y/1.5 = 0.9F_y(1.5S_y)/1.5 = 0.9F_yS_y$. This derivation assumes that $Z_y/S_y = 1.5$, which is reasonable for weak axis bending. For strong axis bending a similar derivation using $Z_x/S_x = 1.1$ results in $M_n / \Omega = 0.66 F_yS_x$. These approximations assume a wide-flange cross-section.

*Amanuel Gebremeskel, P.E.*

## Fillet Weld Strength

**I am familiar with the method of determining the fillet weld strength using the ASD load approach, but I am having difficulty determining this strength when using the LRFD load approach.**

**For a simple ASD fillet weld (load at 90° to the fillet) the magic number is 0.9 kips/in. of weld, which is based upon 0.3*70 ksi electrode per 16th of weld.**

**I noticed in the 13th Edition Manual that the weld strength is increased when the load is at 90° to the fillet. I always thought a weld had the same strength whether a load was in the same direction, along the weld or perpendicular to the weld. The 13th Edition Manual seems to indicate that it is 50% stronger when loaded at 90°.**

The increase for direction of load is covered in Section J2.4(a) of the *Specification*. Where the load is oriented at 90°, this amounts to a 50% increase and applies to both ASD and LRFD load approaches.

The weld value in the old versions of ASD was 0.928 kips per 1⁄16 in. of leg. This comes out to the same Allowable Strength in the ASD approach of the 2005 *Specification*. The Available Strength of Fillet Welded Joints is $0.60F_{EXX}$, regardless of whether the ASD or LRFD load approach is used. (See Table J2.5 of the 2005 *Specification*.)

If the ASD load approach is used, $\Omega = 2.00$, resulting in an Allowable Strength of $0.60F_{EXX}/2.00 = 0.30F_{EXX}$, then this resolves as follows: $(0.30)(70)(0.707)/(16) = 0.928$ kips per 1⁄16 in.

If the LRFD load approach is used, $\phi = 0.75$, resulting in a Design Strength of $(0.60F_{EXX})(0.75) = 0.45F_{EXX}$, then this resolves as follows: $(0.45)(70)(0.707)/(16) = 1.392$ kips per 1⁄16 in.

Note that the LRFD Available Strength is always 1.5 times the ASD Available Strength.

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

# steel interchange

## Anchor Rod Push-Out

**Section 2.9.1 of Design Guide 1 contains the following statement:**

**"When designing anchor rods using setting nuts and washers, it is important to remember these rods are also loaded in compression and their strength should be checked for push-out at the bottom of the footing."**

**How does one go about calculating the push-out of the anchor through the bottom of the footing?**

Another AISC source of information on the subject, Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Buildings*, provides a discussion of anchor-rod push-out. See Section 4.2.7 in that publication.

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

## Fillet Weld for Single-Plate Shear Connection

**My question has to do with single-plate connections to supports. Chapter 10 of the 13th Edition (page 10-101) states "the weld between the single plate and the support should be sized as ⅝ $t_p$, which will develop the strength of the plate." Is this a minimum or maximum limit? Should we design the required weld size needed, and then compare to this value? This question stems from the 9th Edition Commentary on single-plate connections (page 4-53) where it stated the weld size need not exceed 0.75t. Are these (2) requirements discussing the same subject? It seems that the 9th Edition is trying to make sure we have more web thickness than weld, but the 13th Edition Commentary is stating to use an exact amount of weld. Can you shed light on this?**

You are correct that the requirement on page 10-101 in the 13th Edition Manual is intended to develop the plate. This means it is a minimum weld recommendation if the designer wishes to develop the strength of the plate, and is based on weld shear rupture. You do not need to calculate a weld size for load and compare. Rather, if the plate is adequate in shear, the weld size is then selected as ⅝ $t_p$ and will have adequate strength. The idea behind the older ¾ $t$ requirement in the 9th Edition Manual was also the same, but it is an older approach based on weld yield that has been dropped in favor of the new approach.

*Amanuel Gebremeskel, P.E.*

## $L_p$ for Non-Compact Shapes

**I have a question regarding a value in Table 3-2 of the 13th Edition Manual pertaining to the W21×48 (page 3-17). The listed value for $L_p$ is 6.09 ft, whereas when I calculate the value myself (for a 50 ksi beam) I get 5.86 ft. Can you please review this value and let me know if this is an error?**

The number listed in the table for the W21×48 is not an error. A W21×48 is non-compact, and the $L_p$ for non-compact shapes, $L'_p$, is not calculated per Equation (F2-5). Remember that non-compact shapes are not capable of achieving the full plastic moment. Therefore, a point on the moment versus unbraced length curve is used to define the value $M'_p$ for the shape. See page 3-4 in the 13th Edition Manual for discussion, and the Equation upon which the the value of $L'_p$ is based.

*Amanuel Gebremeskel, P.E.*

## ASD or LRFD?

**What is the AISC position on use of LRFD or ASD design? It appears that the equations in the 2005 AISC *Specification* can be utilized in the LRFD or ASD method by multiplying by the $\phi$ factor or dividing by the $\Omega$ factor. Is this correct, because people are telling me one cannot use the ASD method anymore? What about the IBC—do you know if they specifically require the use of LRFD?**

The governing building code typically specifies load combinations. IBC (and ASCE 7) provide load combinations that can be used with either ASD or LRFD design approaches. IBC 2006 also references AISC 360-05 (the 2005 AISC *Specification*), which provides both ASD and LRFD methods of design.

Therefore, the answer to your question is that you can use either method when using the 2005 AISC *Specification*. Since AISC 360 uses the same equations for both methods, the only differences between LRFD and ASD will be due to variations in the load combinations from ASCE 7 or IBC. Just be sure to use the corresponding set of load combinations.

*Amanuel Gebremeskel, P.E.*

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

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

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



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

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# steel quiz

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

This month's Steel Quiz was provided by the Steel Joist Institute. Sharpen your pencils...



1 How can I locate the reaction at the end of an open-web steel joist at the center of a beam?

2 Are standard joists designed to resist any concentrated loads not at a panel point?

3 What special bridging requirements apply when the joist is detailed with bottom-chord bearing, or when a joist has a full-depth cantilevered end condition?

4 Can joists be designed for a moving point load at top and/or bottom chords?

5 True or False: KCS joists are designed so that a single 200-lb concentrated load can be placed between panel points without the need for reinforcing with an additional web.

6 Is a row of uplift bridging required for roof joists designed with 10 psf gross wind uplift and 15 psf dead load?

7 Can open-web steel joist size be controlled by the required fire rating?

8 What is the minimum bearing length when installing a K-Series joist that is detailed to have a 2½-in. R Type top-chord extension, and the connection detail shows the full length of the bearing seat and extension resting on the steel support?

9 What is the minimum bearing seat depth (height) for an LH-Series joist installed at a 1:12 slope?

10 What is the minimum number of rows of bridging required for a 26K8 with a span length of 46 ft-8 in.?

TURN TO PAGE 14 FOR ANSWERS

XTREMELY FLEXIBLE
XTREMELY LEAN
ALL FROM ONE
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Piece Label,
Layout Marks
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and Strip
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and Miter
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# steel quiz

ANSWERS

**1** Provide a note on the drawing such as, "Joist manufacturer to proportion the joist such that the working point is centered on the support." Note that the bearing depth may have to be increased to accomplish this. Conservatively, for every inch you increase the bearing depth, you get an extra inch of clearance to the end diagonal web. For K-Series joists, there is normally only 4 in. of clearance. For LH-Series, there are 6 in. For every additional inch you need, deepen the seats an inch.

**2** No. Any additional loading should be taken into account when specifying the uniform loads. Otherwise, request that the joists be designed for a concentrated load at any panel point, or at any location along the joist top or bottom chord.

**3** A row of diagonal bridging must be located near the support for lateral stability. This bridging must be installed as the joist is erected.

**4** Yes. The joist design will be designed for the worst condition for each member in the joist for the load applied anywhere along the length of the joist.

**5** False. Chord bending due to concentrated loads must be handled with an extra web member shop- or field-applied for K-Series (and KCS*), LH- and DLH-Series. *Note: KCS joists are a subset of K-Series joists.

**6** Yes, using ASCE 7-05 load combinations:

In LRFD, net uplift = $0.9D - 1.6W$ = $0.9(15) - 1.6(10) = -2.5$ psf

In ASD, net uplift = $0.6D - W$ = $0.6(15) - 10 = -1$ psf

**7** Yes. Several Underwriter's Laboratories (UL) designs have a minimum joist size, as well as minimum depth and weight per foot, maximum spacing, minimum bridging size, or minimum element size. In addition, when a UL design requires a reduction in the design stress level of a steel joist, the specifying professional must apply the proper stress level reduction in the selection process.

**8** Five inches. The SJI *Standard Specification for Open Web Steel Joists, K-Series*, Section 5.3(b) states that the joist "shall extend a distance of not less than 2½ in. over the steel supports." This does not consider the joist extension. Because the R Type extended end is fully supported on the steel, it also must be considered when calculating the required bearing length for the installed joist.

**9** Six inches. This is given in the table "Sloped Seat Requirements for Slopes 3/8:12 and Greater, LH- and DLH-Series Steel Joists" found in the Accessories and Details section of the 42nd Edition *SJI Catalog*.

**10** Four rows. Reference SJI *Standard Specifications for Open Web Steel Joists, K-Series*, Table 5.4-1. Note that this joist span length is also in the red shaded area of the SJI Load Tables; therefore, one of these bridging rows nearest mid-span must be bolted cross bridging.

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

Steel Solutions Center

PROJECTS

## Covering the Colts


Walter P Moore

HKS, Inc.

On September, 7 the Indianapolis Colts will play their first game at brand-new Lucas Oil Stadium. Designed by architect HKS, Inc., the stadium seats 63,000 (and can be expanded to 70,000 when it hosts the Super Bowl in 2012). The playing field is 25 ft below street level, allowing fans unobstructed views from their easily accessed seats.

At the Colts' first game in its new home—against the Chicago Bears—fans will be sitting beneath an engineering milestone. The stadium's steel roof, designed by structural engineer Walter P Moore, is the first ever to divide lengthwise into two retractable panels—160 ft long × 600 ft wide and 2.9 million lb each—with each half sliding down the steep, gabled roof of the stadium into the open position. A 960-hp cable drum drive system moves the retractable roof panels up and down the sloped track in 9 to 11 minutes depending on wind conditions. (Structural steel was fabricated by AISC Member Hillsdale Fabricators/Alberici Constructors.)

The project also features a retractable end wall consisting of six glass panels that move to create an 85-ft-tall × 210-ft-wide opening. Each panel rides on a steel rail while the wall opens and closes, and is supported by two hardened steel wheels.

QUALITY NEWS

## Indiana Fabricator Wins Free Audit

Indiana Steel and Engineering Corp., a fabricator in Bedford, Ind., has won Quality Management Company's drawing for a free audit.

Since October 2006, QMC has been administering a voluntary Customer Satisfaction Survey of AISC Certified Fabricators and Erectors upon receipt of their certificate. The objective of the survey is to improve the certification process from invoicing to the audit to issuing the certificate, and companies that complete the survey are automatically entered into the drawing. QMC will draw for another free audit in six months, so keep those surveys coming in!

STEEL SHEET PILING

## Best Practices for Installing Steel Sheet Piling

The North American Steel Sheet Piling Association (NASSPA) earlier this year announced the publication of its *Best Practices Steel Sheet Piling Installation Guide*. This updated and revised manual provides an authoritative guide to the methods of installing steel sheet piling.

The goal is to describe practices that ensure proper steel wall installation, and convey the importance of predicting the "driveability" of sheet piling sections following a thorough evaluation of all ground conditions. The manual presents an inventory of the existing driving systems, from impact hammers to vibratory piling drivers and special systems, and also provides a description of driving methods, ancillary equipment (including guide frames), and all necessary procedures to follow when installing sheet pilings. Finally, some common installation problems are illustrated and several special aspects of driving are briefly outlined.

The guide can be downloaded from the NASSPA web site, **www.nasspa.com**. A hard copy is available upon request; call 866.658.8667.

PROJECTS

# From the Shipyard to the Backyard

BY JAMES FALLS

Steel containers have been a mainstay in the world's shipping industry ever since 1956, when U.S. trucking entrepreneur Malcolm McLean loaded 58 steel containers aboard the tanker Ideal-X and transported them from Newark, N.J. to Houston.

But what happens to these containers when they are "retired?" At the end of its lifespan (typically eight to 12 years), a container is traditionally sold for scrap or, when it's cost-effective, shipped back to its country of origin. But today many retired containers are finding new life as garages or spare bedrooms. Known as intermodal steel building units (ISBU), these repurposed containers are being used to build strong, sustainable, and durable structures.

Some major shipping companies are selling their used containers for approximately $1,200 each, and many containers are being sold over the Internet. A search for "shipping container" returned more than 100 advertisements for shipping containers in the U.S.; most of these are selling for around $4,000 each.

The standard dimensions of a container are 8 ft in width, 8 ft-6 in. or 9 ft-6 in. in height, and lengths of 20 ft, 40 ft, 45 ft, 48 ft, and 53 ft (for a point of reference, a 40-ft container weighs approximately 10,580 lb and can hold approximately 65,000 lb). The containers are made from high-strength weathering steel, water-resistant, and designed to resist harsh oceanic environments, making them more than acceptable for use in building construction.

## Dropping Anchor

Transforming an ISBU into a useable structure is fairly straightforward. Since they are originally designed for efficient transportation, they can easily be relocated to a construction site. After delivery, holes for doors, windows, and other desired openings are cut in the sides. The ISBU is then lifted by crane onto the building foundation and securely welded to the foundation; when properly anchored, it can resist winds of up to 175 miles per hour. Commonly, several ISBUs are stacked on top of each other or placed side-by-side to form a large home or office building. The steel is insulated with a ceramic powder, making it rust-resistant and preventing mildew build-up. Exterior and interior finishes such as drywall, stucco, and wood are then attached to the steel frame as desired. At this point, the transformation from a sea-faring shipping container to a static structure is complete.

## Mixed Use

ISBUs can be used to build a multitude of different structures. Architects use them to build custom homes and trendy bungalows of all sizes and shapes. The U.S. military uses them to set up temporary command centers and training facilities. ISBUs are also very useful as emergency shelters and temporary housing, as they can be delivered quickly and set up with little effort. Press boxes, concession stands, storage facilities, radar stations, and apartment buildings have also employed ISBUs.

In addition, several notable projects around the world have been constructed using these special steel boxes. Architect Peter DeMaria designed a two-story home using eight ISBUs in southern California; the 3,500-sq.-ft home was awarded the 2007 AIA Honor Award for Design Excellence/Innovation. And according to a recent article in *USA Today*, DeMaria plans to offer more of the container homes, at a starting price of $150 per square foot—a bargain considering that traditional homes in the same area typically go for $225 to $250 per square foot.

In Whistler, Canada, home of the 2010 Winter Olympic Games, 294 rooms are being constructed to provide temporary housing for workers, media, and volunteers. Also, the Nomadic Museum, built in 2005 in New York to house a photography exhibit, used 152 shipping containers for its exterior walls. (Interestingly, the museum was dismantled and then rebuilt in Santa Monica, Calif. a year later and Tokyo the following year.) And in Bishkek, Kyrgyzstan, hundreds of containers are double-stacked to create the Dordoy Bazaar, a large wholesale and retail market; an estimated 6,000 to 7,000 containers stretch for more than a kilometer to make up one of the largest commercial centers in the region.

The use of shipping containers to build structures is still in its infancy, but with a little imagination and creativity it is possible to construct almost anything. So the next time you walk by a shipyard or rail yard and see an empty shipping container, don't assume it will be crossing an ocean; it might just turn out to be your neighbor's new kitchen.

*James Falls is a senior civil engineering student at the University of Florida in Gainesville and a summer intern with AISC.*

Photos: Andre Movsesyan/Christian Kienapfel

This 3,500-sq.-ft home in Redondo Beach, Calif., designed by Architect Peter DeMaria, uses eight ISBUs.

ENGINEERING JOURNAL

## Third Quarter 2008 Article Abstracts


The following papers appear in the third quarter 2008 issue of AISC's *Engineering Journal*. EJ is available online (free to AISC members) at **www.aisc.org/epubs**.

### Fracture Modeling of Rectangular Hollow Section Steel Braces

XIANG DING, DOUGLAS FOUTCH, AND SANG-WHAN HAN

Steel braced frames are widely used in all regions of the U.S. including those with high levels of seismicity. Rectangular hollow sections (RHS) are popular because of good section properties and ease of construction. A refined beam model has been developed which is efficient for use in finite element models of buildings. It accounts for local buckling and fracture in the brace and is shown to have good accuracy for nonlinear seismic analyses.

**Topics:** Seismic Design, Hollow Structural Sections, Stability and Bracing

### Design Aid for Triangular Bracket Plates Using AISC Specifications

SHILAK SHAKYA AND SRIRAMULU VINNAKOTA

This paper presents a model to determine the nominal strength of a triangular steel bracket plate using the column strength equations in the AISC *Specification*. Also included are two design tables for such bracket plates for two grades of steel. The nominal strengths obtained by using the relations developed by the authors are compared with the available experimental results and results from other available theoretical approaches. The authors' approach predicts results closer to the experimental results than the other theoretical approaches. Two example problems are worked to illustrate the use of the design tables.

**Topics:** Connections-Simple Shear, Columns and Compression Members, Steel

### A Comparison of Frame Stability Analysis Methods in ANSI/AISC 360-05

CHARLES J. CARTER AND LOUIS F. GESCHWINDNER

Two simple unbraced frames are used to illustrate the application of the following four frame stability analysis methods:

- The Second-Order Analysis Method (ANSI/AISC 360-05, Section C2.2a)
- The First-Order Analysis Method (ANSI/AISC 360-05, Section C2.2b)
- The Direct Analysis Method (ANSI/AISC 360-05, Appendix 7)
- The Simplified Method (13th Ed. *Steel Construction Manual*, page 2-12; AISC *Basic Design Values* cards)

**Topics:** Analysis, Specifications, Stability and Bracing

### Effects of Slab Post-Tensioning on Supporting Steel Beams in Steel-Framed Parking Deck Structures

BHAVNA SHARMA AND KENT A. HARRIES

AISC Design Guide 18, *Steel-Framed Open-Deck Parking Structures*, discusses the use of cast-in-place post tensioned concrete slabs in steel framed parking structures. In Section 3.3.2.1 of Design Guide 18, the authors reflect on the manner in which the post-tensioning force is resisted by and affects the supporting beam: in a non-composite or composite manner. They conclude that the post-tensioning force is carried almost entirely in a composite manner (minus effects of shrinkage and elastic shortening). This conclusion is based on results of unpublished research and is corroborated by earlier design guidance. The objective of this field study is to quantitatively assess the effect that slab post-tensioning forces have on their supporting steel members.

**Topics:** Beams and Flexural Members, Composite Construction

### Current Steel Structures Research

REIDAR BJORHOVDE

This regular feature of the *Engineering Journal* provides information on new and ongoing research around the world. Research projects are summarized on the following topics: structural behavior and strength under seismic loads, including special concentrically braced frames and energy dissipation characteristics of semi-rigid connections; cross-sectional stability of hot-rolled shapes; behavior and strength of steel columns with partial damage of the fire retardant coating; composite beams with precast hollowcore slabs; and plate girder research in Spain, including hybrid and stainless steel girders.

**Topic:** Research

INDUSTRY EVENTS

## May the Best Welder Win

Sparks will fly—literally—at the AWS Skills Competition Weld-Off, an event testing students' welding skills. A component of the SkillsUSA Competition, the Weld-Off is among the highlights of the 2008 FABTECH International and AWS Welding Show, October 6-8 in Las Vegas.

Sponsored by the American Welding Society, it requires welders to demonstrate their skills by completing standard test weldments (plate and pipe), sheet metal projects in aluminum and stainless steel, and a pressure vessel. Welds will be judged by AWS Certified Welding Inspectors for soundness and appearance. Written skills and welding code interpretation also will be considered.

Six finalists, selected from 24 student welders that participated in the SkillsUSA Championships in 2007 and 2008, will compete in the Weld-Off this year and only three will qualify to advance to the SkillsUSA U.S. Open Weld Trials in 2009. The U.S. Open Weld Trials Champion will represent the United States at the WorldSkills Competition in Calgary, Alberta, Canada in 2009. The AWS U.S. Open Weld Trials winner will receive a four-year scholarship, sponsored by the Miller Electric Manufacturing Co., worth $40,000 from the AWS Foundation, a four-year AWS membership, an AWS Certification, and up to $1,000 in AWS publications.

For more on FABTECH International and AWS Welding Show and the AWS Skills Competition Weld-Off, visit **www.fmafabtech.com**.

UNIVERSITY RELATIONS

## Student Bridges Shine in the Sunshine State

The University of Florida's Stephen C. O'Connell Center was full of students this past May 23-24, but the majority of them weren't Gators.

Civil engineering students from around the country, as far away as Hawaii, traveled to the Gainesville campus, as the school played host to the 17th annual AISC/ASCE National Student Steel Bridge Competition. This year's contest ended with the University of California, Berkeley taking first place. Cal's win ended North Dakota State University's bid for a three-peat; that school had won in 2006 and 2007. The University of Florida (UF) and the University of California, Davis took second and third place, respectively.

For the 42 participating universities, the goal of the competition was to design, fabricate, and build the most efficient 21-ft steel bridge that could support a vertical load of 2,500 lbs. Each bridge was ranked in six categories: construction speed, lightness, aesthetics, stiffness, economy, and efficiency. Teams spent the entire year and countless hours preparing for the competition.

### Beyond the Bridge

For Florida students, however, designing an award-winning bridge was only part of the goal. As the host school, the larger feat was figuring out how 42 bridges were going to be assembled and tested in only eight hours.

Nearly 650 students, professors, and professionals attended the competition expecting the competition to run smoothly and efficiently. What many of them didn't realize is that well before the rules were released in August of 2007—and before many students knew they would even be on a steel bridge team—UF students were already preparing for the 2008 competition. In fact, Dr. Tom Sputo, UF ASCE Faculty Advisor, began the long process in 2006 when he requested that UF be considered as the host of the 2008 NSSBC.

A small committee of dedicated students was assembled to plan, coordinate, and execute the competition. The students, with already busy schedules—juggling homework and oftentimes a job, as well as other responsibilities—added another demanding item to their plate, spending countless hours making hotel arrangements, setting up contracts with vendors, contacting professionals to be judges, and coordinating with the qualifying universities to make sure they were ready for the competition. Many of the venues, including the O'Connell Center and the Reitz Union Grand Ballroom, were reserved two years in advance—even before the 2007 NSSBC, hosted by California State University, Northridge, took place.

Student Director of the competition and AISC summer intern James Falls described his experience: "You have to be dedicated and proud to host the best student engineering competition in the country. It was a tremendous amount of work but extremely rewarding when two years of hard work all comes together." Without question, he explained, the students in charge from year to year are the unsung heroes that make the competition possible.

Fromy Rosenberg, AISC's Director of University Relations, commented, "This is the premier competition for student engineers. It brings together everything students have learned in the classroom. Participating students practice basic steel

Photos: Bob Phelan/Missouri University of Science and Technology

design and fabrication and project scheduling and management, and gain hands-on appreciation for the strength and versatility of structural steel."

In order to reach the national competition, student teams nationwide competed in 18 regional competitions. The winners in those competitions were invited to compete in the national event. This year a total of 182 universities competed in the regional competitions.

The 2009 NSSBC competition will be hosted by the University of Nevada, Las Vegas, May 22-23, 2009—and yes, UNLV has already been preparing for months.

### 2008 NSSBC Winners

**Overall**

**University of California, Berkeley**
**University of Florida**
**University of California, Davis**

**Construction Speed**

SUNY College of Technology at Canton
University of California, Berkeley
University of Wyoming

**Lightness**

University of California, Berkeley
Virginia Polytechnic Institute
University of Florida

**Aesthetics**

Iowa State University
South Dakota School of Mines
University of Wisconsin, Madison

**Stiffness**

University of Missouri, Kansas City
University of Wisconsin, Madison
Seattle University

**Economy**

SUNY College of Technology at Canton
University of California, Berkeley
University of Florida

**Efficiency**

University of California, Berkeley
University of California, Davis
University of Florida

Full results of the overall competition and each category are posted on the official 2008 NSSBC web site, **www.2008steelbridge.com.**

# Artistic Achievement

BY MARIO ELCID, P.E., AND BOB VARGA, AIA, LEED AP

**The arts campus at Western Michigan University finds the "missing link" with a dynamic new building that complements and enhances its neighbors.**

SOMETIMES, IT TAKES JUST THE RIGHT PIECE OF FURNITURE—OR PERHAPS THE RIGHT RUG—TO "TIE THE ROOM TOGETHER."

Entire buildings can also play this role—on a grander scale, of course—as illustrated on the Western Michigan University campus in Kalamazoo.

The Richmond Center for Visual Arts (RCVA) completes the campus' Miller Plaza Quad and adds a new, striking element to WMU's School of Visual Arts. RCVA physically connects the Miller Parking Deck to the Dalton Center (which houses the music and dance departments) and provides a link to the south wing of Kohrman Hall, the future location for the School of Art. Conceptually, the completed Miller Plaza Quad imbues these connections, which will be used by students and visitors alike, with a broad visual art experience. And RCVA becomes a physical connector with the equally important function of allowing visitors to view and experience visual art at various scales and locations.

## Designer Needs

The building uses form to express and delineate functions that the public at large will attend, such as student exhibits, main exhibits, and lectures. Although RCVA is a relatively small building, it had to meet rather high standards. One of the design challenges was to provide a high-quality and well-designed space on a budget considered tight for buildings achieving American Association of Museums (AAM) standards. To meet the budget, the design visually expresses typically concealed construction materials such as the structural steel system, which forms an integral part of the design and helps reveal the notion of "construction" to the art students inhabiting the space. All the structural elements become architectural finishes and work together to serve a specific visual purpose.

The exhibit areas are wrapped in a dynamic bent copper plane forming both the roof and south wall, allowing light to bleed in via north-facing clerestory glazing. The south exterior tilted wall in the exhibit hall seamlessly and gradually curves and transforms into a roof structure. Along that wall two rows of columns are placed to provide two structural functions and convey one visual intent. The outer sloped column line, with thinner columns spaced at 6 ft-8 in., supports the exterior copper wall, while the inner column line has larger columns and girders that are spaced at 20 ft on center and serve as roof support and are part of the moment frame lateral system. Horizontal angles are provided in the hidden cavity between the outer and inner columns to appropriately transfer the wind loads from the wall into the moment frames.

We guided the structural design to achieve a specific contrast between the thin planar, more rhythmic character of the exterior wall and the heavier inner columns and girders. This approach turned out to be very beneficial from an economics standpoint, as the larger columns had to be designed to resist the wind loads as well as the seismic loads in the north-south direction due to the unique geometry of the building.

## Not-So-Basic Structural Design

RAM Steel was used to design the basic gravity framing elements of the building. Its ability to take into consideration the size restrictions of limiting physical factors and visual design intent also proved useful. For example, in the east-west direction the column spacing is kept consistent at 20 ft on center for the majority of the building. In the north-south direction the columns' spacing varies and ranges between 20 ft and 40 ft, which creates several framing scenarios where each bay has to be checked individually to meet the vibration criteria. W-Shapes are used to support the first floor slab above the partial basement, and the

Hedrich Blessing Photography

Laszlo Regos Photography

The south exterior tilted wall in the exhibit hall seamlessly and gradually curves and transforms into a roof structure.

second floor. Lightweight concrete was used for the second floor composite slab, while normal-weight concrete was used for the first floor composite slab to match the color and appearance of the slab-on-grade concrete around that location.

The complex geometry of the building's large second floor openings into the gallery area, sloped lecture halls, double-story entry space, and a tilted exterior wall and roof prompted a careful approach to designing the lateral system. RISA 2D and RISA 3D models were created to calculate the frames' stiffness, while a few specially designed spreadsheets were generated to ensure an appropriate and efficient distribution of the lateral forces while minimizing the effect of torsion. The lateral resisting system consists of moment frames in the north-east direction and vertical braces in the east-west direction. Horizontal braces were added within the floors, around openings, to help transfer the lateral seismic loads to the appropriate braces. Transfer vertical kickers were strategically placed between the exterior tilted wall and the moment frame columns to transfer wind loads from the building's skin to the main lateral force resisting system. The seemingly simplistic and partially exposed lateral system appears minimal and fits well with the overall design expression, but it conceals many more complex components within the architectural finishes.

## Special Features

In addition to the basic structural steel members, the steel used in this building is heavily influenced by architectural requirements. Steel pieces are used to construct glass and steel display cases that showcase specially displayed pieces of art by the main donors. These boxes give the illusion that they are part of the building's main structural system, but in reality they complement the design and are purely architectural features.

WMU desired a very transparent front face for the building in order to draw the attention of students passing by on the plaza to what was happening inside the building. To that end, a very specific rhythm of expressed and concealed structure was set up to create larger "windows" into the building. For example, one of the main building columns on the east (front) building face is wrapped in an asymmetric metal panel skin that projected beyond the glass curtain wall. The lower part of the metal enclosure was widened to emphasize the asymmetry of the front entrance. Other columns on the same column line were left visibly exposed behind the glass to enhance the notion of "skin." In addition, the columns provide a visual frame for a large projection screen that descends from the ceiling into the lobby and can be viewed from outside the building.

The larger of RCVA's two lecture halls, located on the second floor, has a curved exterior that cantilevers outside of the first-floor borders. The open-space, cantilevered structure and curved exterior required for both lecture halls combined to provide additional challenges in keeping the steel framing as shallow as possible, as the allowable number of columns to be used in this space was limited. Several framing schemes were developed using RAM Steel, paying careful attention to the structural depth variations due to subtle framing changes. Continuous girders run over columns and are spliced at the zero moment transition point, helping to keep a shallow framing in the space, while filler beams are added within each bay helping to meet the required vibration criteria.

The new bridges that connect the building to the adjacent structures are another indicator of the level of detail and collaboration that makes the structural steel enhance the appearance of the project, and proves that design considerations were not limited to the interior of the building. For example, the bridge on the south side of the building connecting RCVA to the parking garage is supported by extremely narrow exposed galvanized wide-flange columns that are offset by deep side girders to draw the viewer's attention and focus to the bridge.

RCVA is a great example of what happens when art, design and structure come together to form an aesthetically appealing and extremely functional space. And steel plays a leading role in punctuating this design purpose, creating a bold, modern addition to a campus area that focuses on the arts. MSC

*Mario Elcid is a structural engineer and a project manager, and Bob Varga is an architect and designer, both with SmithGroup.*

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# A New Link to Learning

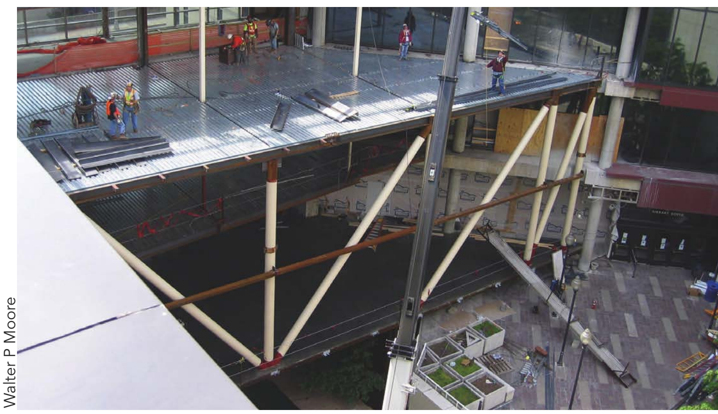
BY BRENT BANDY, P.E., LEED AP

**An expanded bridge on Georgia State University's campus brings two library buildings together as one.**

**WHEN I FIRST WALKED INTO THE GEORGIA STATE UNIVERSITY LIBRARY** at the start of this project, I was transported back to my college days at nearby Georgia Tech. The smell of musty old books, the long rows of shelves, and the dim lighting evoked "knowledge" to me.

For today's students, however, natural lighting, the hum of a hard drive, and the smell of lattes are what opens their minds for learning. Georgia State, in downtown Atlanta, understood this and had a vision to make its library more inviting and up-to-date, with goals to:

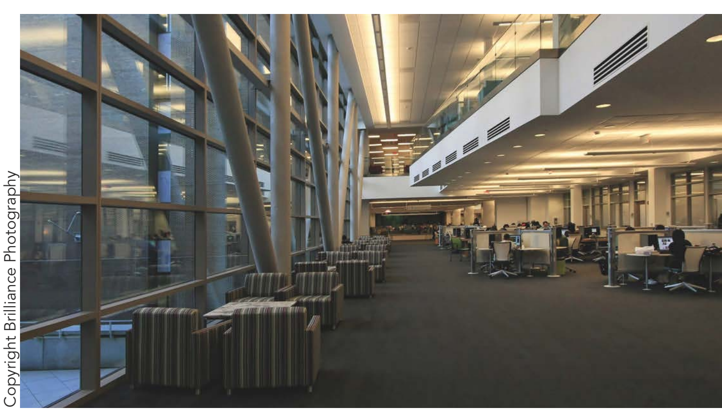
- Update finishes, furniture, and building systems
- Increase reader seats and collaboration spaces
- Expand collection space by adding compact shelving
- Expand the narrow three-story steel pedestrian bridge spanning Decatur Street and link the library's north and south buildings (from the 1960s and 1980s, respectively), making them act and feel like a single building



Walter P Moore

**Above:** Bridge erection was performed over a street, without shoring.

**Center:** The updated bridge included a vertical expansion of one floor, plus a horizontal expansion of 35 ft at the two lower bridge levels and 19 ft at the two upper levels.



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## BIM from the Start

Both the architectural and structural teams were committed to using the Revit platform on the $17.5 million project, both for collaboration and production of documents. Walter P Moore was still in the early stages of implementing the new software when this project started. Despite the steep learning curve, we quickly saw the benefits of three-dimensional collaboration and coordination.

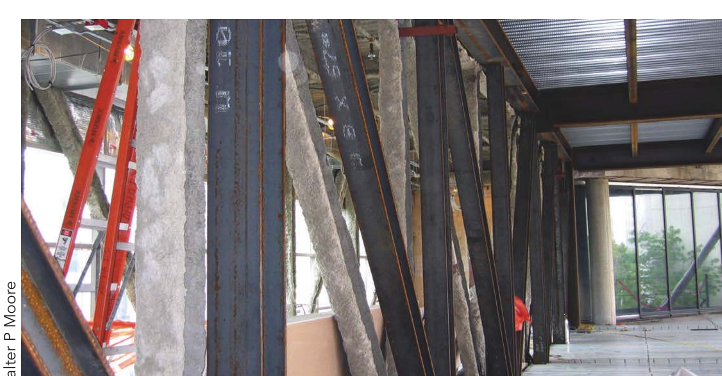
Modeling the existing concrete buildings and steel pedestrian bridge helped us to visualize and coordinate between old and new. Multiple schemes for the new link were explored for aesthetics, constructability, and cost. Three-dimensional views of the design helped the owner more easily understand the options. The construction manager and subcontractors used these same views to better comprehend and more accurately price the options, providing a clear direction at the end of schematic design that proved itself throughout the remainder of design and construction.

The chosen scheme for the new link was a downward vertical expansion of one floor, resulting in a four-story bridge, plus a horizontal expansion of 35 ft at the two lower bridge levels and 19 ft at the two upper levels. This resulted in the addition of about 11,000 sq. ft. The second level was supported at each side by trusses, while the middle portion of the floor was supported by a truss at the level above. A truss at the third level was already in place, so a similar truss was set alongside the existing to limit the impact on usable space. The new floors at levels five and six were supported by transfer girders at level four, which were in turn supported by the new trusses. This scheme maximized the usable open space between library levels two and four and optimized the structural load path to provide an efficient overall design.

## Link Analysis

The original 15-ft-wide pedestrian bridge consisted of one-story trusses on each side, spanning about 75 ft and topped with two more stories of steel beam and column framing. The south end was attached to the south library structure, while the north was supported to the ground with a braced frame system on auger-cast pile foundations separate from the library's original north structure.

Our goal from the start was to avoid costly modifications to the existing walkway members and connections. We studied various structural



Walter P Moore

**Above:** A new mid-truss "sistered" alongside an existing truss.

design concepts that minimized the impact on the existing trusses. The final design entailed three trusses:

- The west truss is two stories high and spans between two new steel columns that aren't attached to the existing structure.
- The new east truss sat directly below the existing east truss to pick up a portion of the new second floor. This new east truss was designed to have stiffness compatible with that of the existing east truss so that new floor live loads would cause no net increase in loading on the existing truss.
- Similarly, the new "mid-truss," adjacent to the existing west truss, was designed for deflection compatibility with the existing west truss.

A staged analysis of the system was performed to ensure that the different phases of construction and various loading combinations did not overload the existing trusses and supporting structure. The mid-truss was set alongside the existing west truss and stitched together to ensure that there were no vertical stresses in the floors at the construction joint between the old and new slabs. The trusses were not stitched until after all the other framing and concrete slabs had been cast, in order that the only load to be shared between trusses would be the live load.

The original geotechnical report classified the downtown site as Site Class E. The team requested a site specific hazard study as well as shear wave velocity testing in order to more accurately assess the site class, which paid off by reducing the Site Class to D and the Seismic Design Category to B. The decreased detailing requirements and seismic forces helped reduce the required upgrades to the existing lateral system. The link expansion had very little impact on the eight-story concrete frame to the south, but the combination moment frame and braced frame at the north end was upgraded for the link expansion loads.

## Constructability Challenges

Key to the project's constructability was the planning of the sequence of steel erection and the need to avoid shoring. The sequencing plan on the structural drawings specified that the trusses should be erected first, and the second and fourth floors should be erected next, which provided a working platform for the rest of the work over Decatur Street. While it would have been possible to keep the street open during construction, the contractor closed it for much-needed layout and staging space. Only after the remaining steel framing and concrete deck was placed could the connections between the new and existing structures be made. After all the structures were tied together, the cladding and finishes were attached.

Another constructability challenge was the deep foundations that were to be installed in the basement of the existing north library. The north end of the new link required new columns and foundations. The basement working clearance was only about 8 ft and required low-headroom equipment to install foundations. Auger-cast piles were chosen for this application since most of the foundations on the project were a standard installation. The steel framing at the lowest level was configured to take advantage of some significant overstrength in existing foundations and reduce the number of low-overhead foundations by about two-thirds.

### Exposing the Structure

The design team initially leaned toward minimizing visibility of the structural systems, but the trusses were eventually turned into a signature element of the building. While the link's two-story open space over the road is dramatic in itself, the exposed pipe truss accentuates the height of the space. Once we had determined the diameter of the two-story truss web members, the architect did not want to wrap them in cementitious fireproofing and drywall, because they would become too bulky and obscure the views from both inside and outside. Also, because the truss web members connected right at the floor levels, pipe-to-pipe connections were made to eliminate gussets, which further enhanced the overall appeal of the system and minimized visual obstructions through the glass curtain wall.

Due to a very efficient design of the steel structure—resulting in a steel weight of 120 tons—the budget was able to accommodate the use of intumescent fireproofing on the web members of the exposed two-story truss. The wall thicknesses of some pipe member sizes were increased to reduce the required thickness of coating for the fire rating. The main shafts came to the site primed and painted with only the ends primed. The final coating was kept from the joints to keep the welding heat from damaging the intumescent paint.

### Successful Transformation

Even with the exposed trusses, sleek connections, intumescent paint, difficult foundations, and other challenging existing conditions, the bridge project came in on time and under budget. The project team received an AIA award for collaboration using BIM technology. Both the owner and university community are thrilled with the "brand new" library. But most importantly, following the "scent of knowledge," students flock to the library every day and fill all the reader seats by early morning. MSC

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# Moving In

BY NAT OPPENHEIMER, YEGAL SHAMASH, AND BEN ROSENBERG

Polshek Partnership Architects/ Crystal Computer Graphics Ltd.

**Dickinson School of Law, formerly an independent entity, will soon have its own building on the Penn State campus.**

A DECADE AGO, TWO PROMINENT PENNSYLVANIA INSTITUTIONS OF HIGHER LEARNING— the Dickinson School of Law (DSL) in Carlisle and Penn State University in State College—joined forces in a mutually beneficial relationship. Penn State got a law school (it didn't have one before), and DSL was able to take advantage of Penn State's resources as a preeminent research institution.

But there was one problem: The two schools were almost two hours apart. From the beginning, the two schools shared a vision to create a flagship building for DSL on Penn State's main campus. Almost 10 years after the initial merger, this vision will become reality, as DSL's new Lewis Katz Building will open in State College this December. (A new sister building, also named for Lewis Katz, is scheduled to open next year on the Carlisle campus).

Structurally, the sinewy new building in State College is essentially two long halves fit and joined together along the length. The north half is brick-clad with a flat roof and connects to the campus' mechanical and transportation core. The south half is clad with a curtain wall and stone and has a sloping roof that soars over 60 ft into the air at its eastern end. Containing the library, auditorium, courtroom, and café, the south half is the visual centerpiece of the building.

### Staying Light

During schematic design, we decided to use steel framing with composite concrete on metal deck. Initially, the foundation system was designed as piles, due to State College's typical ground conditions, and the structure needed to be as light as possible in order to minimize the number of piles. In addition, the client wanted an efficient system that could meet the demands of the plan irregularities that made the design so unique. After looking at alternate systems, we chose a 4½-in. light concrete slab on composite metal deck system. This system provided the appropriate fire rating and was the lightest system that was flexible enough to address the challenge of the proposed layout. In this case, repetitive or prefabricated systems did not gain the advantage due to the non-repetitive layout.

During the design development phase we discovered that the local building soil conditions were suitable for spread footings; indeed, rock was found not far below the surface. We continued to use steel framing, as spans as long as 50 ft were necessary and, as noted above, the normal cost-effectiveness of repetitive lightweight systems did not have an advantage here.

While very lightweight, the slab and deck system challenged us in two ways: vibration and noise. After sizing members for stress, deflection, and architectural constraints, we conducted a comprehensive vibration analysis on the structure and determined that several beams and girders needed to be increased in size due to vibration limitations. There was also concern about noise from mechanical rooms in the basement; three large classrooms with auditorium-style seating sit above the mechanical rooms. On the advice of the acoustical consultant, we chose a 7½-in. normal weight composite deck system at all bays above the mechanical room in order to minimize noise.

### Keeping Up with the Architecture

As the building is visually striking, the exposed steel needed to be designed to meet this aesthetic. Round hollow structural sections (HSS) comprise a series of full-height columns along the south side of the building. Because all of these round HSS columns needed to be the same nominal diameter, we used ASTM A500 Grade C with a yield stress of 46 ksi instead of the more conventional Grade B, 42 ksi steel.

The most intriguing design challenge of the building was the 30-ft-high, two-story sloping ramp that carried a curtain wall and ran along the south face of the building. The view from the south side of the building, which faces the rest of the campus, is dominated by this curtain wall, and it was important to minimize the structure so as not to impede the view either from outside or from within. The ramp, which connects the two floors of the building's library, is cantilevered over the south entrances to the building. We designed a system to hang the ramp from the roof framing using a series of outrigger plates and hanger rods with clevises. Because the ramp support framing was exposed, plates allowed us to maintain the desired visual appearance. The 1-in.-diameter hanger rods, spaced at 10 ft on center, are cleanly connected with stainless steel pins to steel plates at the ramp floor structure and with clevises at the top. To carry the wind loads on the exterior curtain wall, a series of strong-back plates, also spaced at 10 ft on center, march up the ramp at a consistent offset from the hanger rods. Each strong-back consists of two vertical plates spanning the height of the curtain wall, separated by a small gap with spacer plates. The hanger rods carry gravity loads, the strong-backs carry lateral wind loads, and both work together to provide a clean visual system.

### Laterally Speaking

The building's lateral system is a combination of moment frame and braced bays. A multi-story interior skylight that runs along the boundary between the north and south halves of the building creates an open corridor with a desire for little visible structure within it. We used a series of connector HSS16x8 tubes to carry lateral loads between the two halves of the building to meet both structural and aesthetic requirements.

### Framing Crisis Averted

Due to a necessary architectural design change during construction, we needed to develop a solution quickly when we found that one of our double-angle braces was blocking a relocated door to a café support room. Relocating this primary lateral frame was not an option. We looked at various solutions and eventually designed a combination moment frame and braced bay that essentially framed the door. The moment frame between the head of the door and the floor above is a series of W12x72s. There is also a miniature brace bay to the side of the door, safely enclosed within a wall. We were able to safely carry the required lateral loads without disturbing the architectural design and carry out the redesign within a few days during standard shop drawing review.

### Curbing Problems Early On

All invloved parties agree that for a project of this size, the steel went up in a relatively uninterrupted pattern and with very few issues arising during erection. The detailer was able to build a very accurate BIM model of the entire frame at the beginning of the shop drawing phase. Although the creation of this model generated a significant number of RFIs, all parties understood that these early RFIs would largely eliminate more urgent ones later on in construction, and communication flowed smoothly between all parties.

The fabrication shop was methodical in forwarding erection drawings and shop standards. Once these were approved, a rapid but reasonable schedule was established and held by all parties throughout significant completion of shop drawings, whereby the detailer forwarded a standard number of sheets to the general contractor once a week, on Thursdays, and these were forwarded to RSA by Monday morning. Knowing that we would be receiving a package each Monday morning allowed us to turn the package around by the Thursday of the same week, back to the architect, with little disruption to the schedule.

By establishing and maintaining a constant flow of information, all parties were able to collaborate successfully and help each other get an otherwise complex steel project erected within schedule and budget. MSC

*Nat Oppenheimer is a principal, Yegal Shamash is an associate, and Ben Rosenberg is a project engineer, all with Robert Silman Associates.*

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Gilbane Building Company, Providence, R.I.


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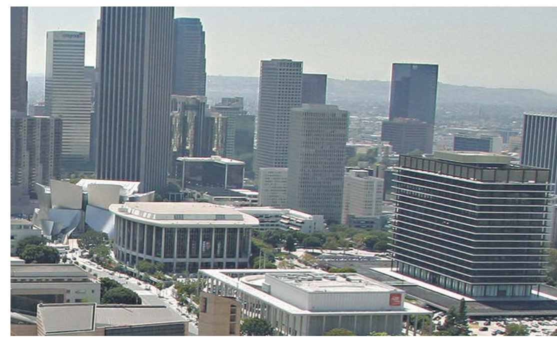
# Facilitating Creativity

Eye-catching architecture and innovative structural design are all but requirements when building a school geared toward the arts.



BY TERRY TSANG, S.E., AND GARY GIDCUMB, R.A.

**THE ARTS ARE THRIVING IN LOS ANGELES**—and not just in Hollywood and the area's mega-studios. Central L.A., with the Walt Disney Concert Hall, Museum of Contemporary Art, and sprawling Music Center complex, has become a center for arts and culture in the area. And next year, the High School for the Visual and Performing Arts will bring an educational component to this nexus of creativity.

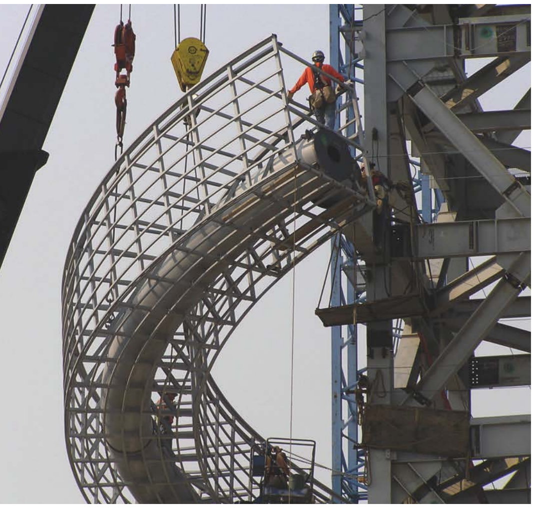
As part of the Los Angeles Unified School District's (LAUSD) 12-year, $20 billion construction program, the high school stands at the north end of the Grand Avenue Cultural Corridor, within easy walking distance of the above institutions. The state-of-the-art campus, built to accommodate 1,800 students, will merge sustainability, security, and a dedication to the arts in a new cultural icon.

## Elements of Design

To express the LAUSD's educational visions, designers created three sculptural expressions of form in the project: a theater tower, a main performance lobby, and a library.

The tower, visible from surrounding buildings and a neighboring freeway, cuts against the downtown landscape and proclaims the school's identity. Elevated above the theater, the tower is part of the theater building's two integrated structural systems. The main building is a concrete shear wall system with steel gravity framing, with the steel tower projected from the main roof via steel braces.

The primary lateral resisting system uses concrete shear walls resting on continuous footings or concrete grade beams, and steps down with various grade changes along the building. While the engineers worked with the architect to incorporate concrete as a design feature, structural steel was chosen for the main building's primary and secondary framing systems. The steel framing system accommodates the unique functional and patial requirements of the theater, accounting for numerous floor elevations and irregularly-shaped platforms between the orchestra pit and the roof above the stage. The main seating area is formed by a series of bent steel composite girders ranging from W10 at shorter spans to W24 at the center of the space for increased headroom. These



Photos: HMC Architects

Steel being erected for the theater tower.

members slope and fan out to support a metal deck with concrete topping.

The "Grid-Iron" platform was constructed with a combination of steel channel-shaped beams and steel grating to accommodate the increased live load. The 60-ft-tall rigging wall, which supports the heavy arbors, was constructed with HSS members, forming a grid system (vertical and horizontal) to resist induced seismic force. Framing for acoustic reflectors and clouds employed round steel pipes, channels, tubes, and angles.

### The Tower

The elevated tower is an unoccupied space with an enlarged box (referred to as the tower head) located 150 ft above the lowest building grade. Early in the process, the design team decided on a structural steel braced frame system as the basic lateral resisting method. The major structural objective was to keep the tower as light as possible, thus minimizing the unnecessary burden on and demand for the main building's lateral resisting elements. By virtue of the vertical massing, a result of an unoccupied tower head with a lighter mass, and the stiffness provided by the braced members and frames, the design minimized an extra lateral load at the top level, creating a "whiplash effect."

The tower structure consisted of W14 diagonal steel braces, columns, and beams. All steel members are connected concentrically at their work point with weld and gusset plates. The tower's columns are extended to the foundation 80 ft below and encased in concrete, working with the main building's concrete shear wall and acting as composite columns. This design resulted in an increased slenderness ratio and enhanced compression-and-tension capacity. The positive load path for the tower's induced lateral load is to the shear wall and foundation below.

### The Spiral

Hovering around the tower, a second architectural feature, called the "Spiral," projects from the main structural system high above the adjacent freeway crossing through downtown L.A. As with the tower, the major structural objective was to keep the design as light as possible. The spiral assembly consists of fabricated steel pipes, each 3 ft in diameter, with a series of transverse stiffener plates inside each pipe. The spiral's exterior cover is an 8-ft square section built with structural steel tubes and angles and cladded with architectural metal panels. The pipe acts as a spine, providing the necessary support for the assembly. The pipe supports a series of steel outrigger beams cantilevered from the tower's structure and reinforced with added steel side plates on each side. These plates improve the weak axis bending and torsion resistance of the W27 outrigger beam for the induced lateral force seismic or wind, which can originate from any direction.

### The Lobby

During the preliminary design phase of the lobby, several "stick models" were used to illustrate the architectural form, and a collaboration of aesthetics, function, and structural performance was achieved.

The lobby structure makes use of 10-in. square hollow structural section (HSS) members as the primary structural system, the skeleton of the form. HSS members are welded together to form a moment resisting connection. Secondary HSS members support the lobby's exterior wall and architectural metal panel glazing system.

Continuity steel plates, added to all joints of the HSS members and acting as a rigid box, ensure a direct and strong load path for the induced joint moment developed at the intersection of the HSS members. An upper-level horizontal ring truss system, referred to as the "donut," was constructed at the same level as the main building's primary roof. The donut level consists of HSS members to provide diaphragm action and positive load transfer for the lobby's framing above the primary roof. All HSS columns and braces are embedded into the foundation concrete. Additionally, the insides of the lower portion of these HSS members are filled solid with non-shrink and non-metallic grout to increase their torsion resistance at the critical section.

The library's base is approximately 91 ft in diameter, with a total of 16 steel columns sloping to a top ring at varied angles.

## Analyzing the Theater

The theater consists of three components: the main building, the tower, and the lobby. The engineering team developed a SAP 2000 3D finite element computer program to simulate the structural behavior of all three components to capture all possible structural demand for them. Three major computer runs included one for the lobby as a stand-alone structure, one for the tower with the main building only, and a third with the three components combined. Through this effort, the engineering team captured the entire loading envelope of the structural design.

## Library Research and Design

Designers originally considered the form of the library as an asymmetrical, cone-shaped concrete structure. In design development, however, a more economical structural steel system emerged that would set the central campus quadrant apart from traditionally designed schools. The team used a 3D ETAB model to analyze the structure.

The primary lateral resisting system for the library is the steel special concentric braced frame (SCBF) on composite steel and concrete grade beams. The library's base is approximately 91 ft in diameter, with a total of 16 steel columns sloping to a top ring at varied angles. The top compression ring is constructed with HSS members. The bottom tension ring is constructed with steel beams embedded inside the 4-ft, 6-in.-deep concrete grade beam. Round HSS was used for the bracing members of the SCBF, along with W14 members at the frame columns and beams. The complex interior layout of the library challenged the engineering team to provide secondary framing for the support of the compound, and interior walls and soffits were constructed with HSS members and angles with infill metal stud framing.

### Testing and Inspection

Welding is the most critical and vital element for interconnecting the structural steel members in the lobby and tower structures. In a process with no margin for error, a new technique was used in the lobby to check the joint welding. With the understanding of the client and technical assistance from Smith Emery (the material testing lab representing LAUSD), joint welds were inspected by phased array ultrasonic inspection in addition to conventional ultrasonic techniques. This recommendation from the testing lab is a more user-friendly technique than conventional ultrasonic testing (UT), simple in principle but complex in practice. Increased restrictions on weld testing requirements were also needed with the steel in the tower. Inspection and testing was based on Table 6.3 of the America Welding Standard (AWS) - UT Acceptance-Rejection Criteria (cyclically loaded non-tubular connections) rather than the industry standard AWS D1 Table 6.2 (statically loaded non-tubular connections).

As with any complex project, solutions depended upon the input from various sources and open communication within the team. The early decisions of the engineering group facilitated the construction process and laid the groundwork for a successful, structurally innovative facility. The High School for the Visual and Performing Arts is on schedule to be completed next year, less than 30 months after the initial groundbreaking. MSC

*Terry Tsang is a project manager and vice president with TMAD Taylor and Gaines, and Gary Gidcumb is an architect and associate principal with HMC Architects.*

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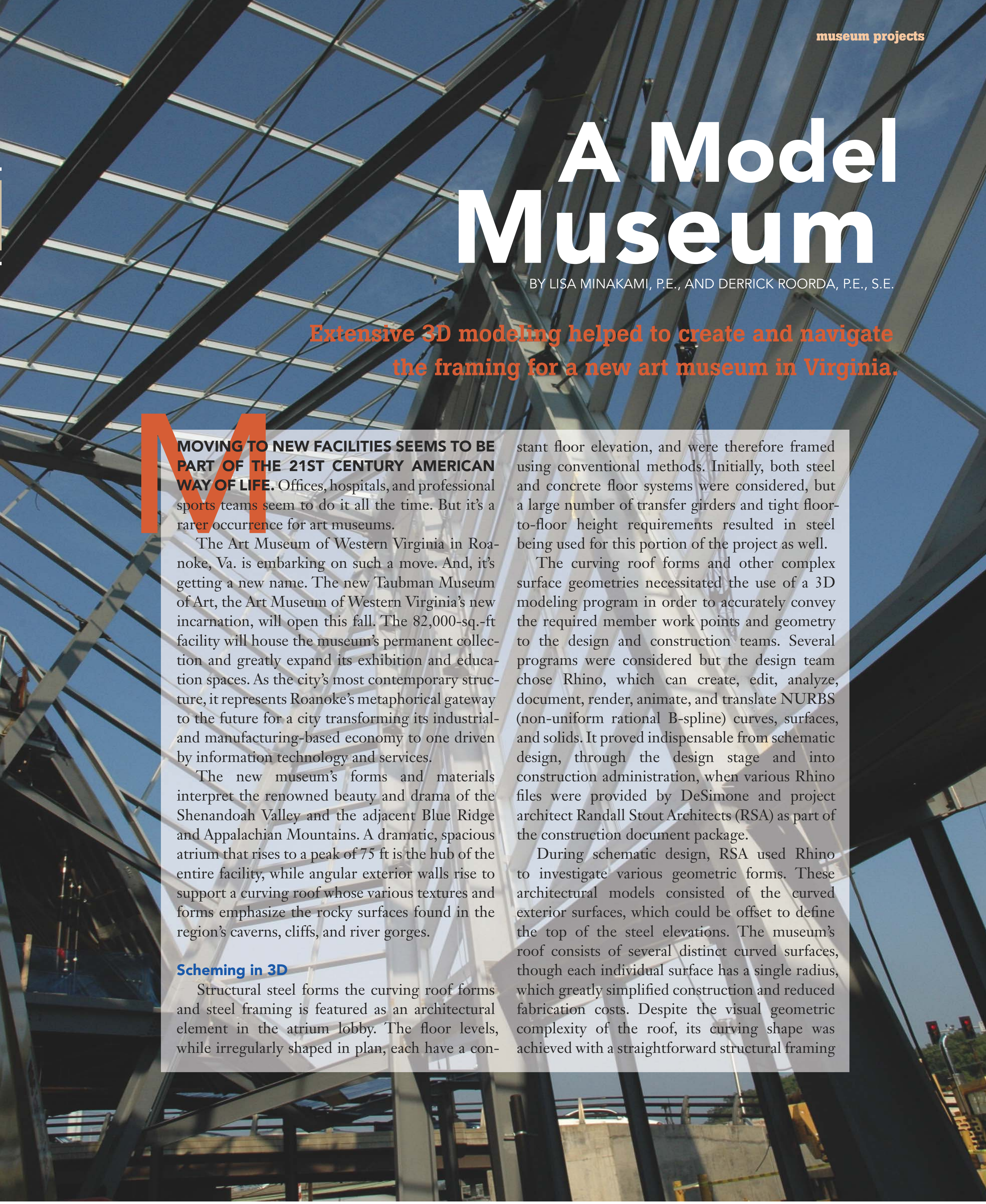


# A Model Museum

BY LISA MINAKAMI, P.E., AND DERRICK ROORDA, P.E., S.E.

**Extensive 3D modeling helped to create and navigate the framing for a new art museum in Virginia.**

**MOVING TO NEW FACILITIES SEEMS TO BE PART OF THE 21ST CENTURY AMERICAN WAY OF LIFE.** Offices, hospitals, and professional sports teams seem to do it all the time. But it's a rarer occurrence for art museums.

The Art Museum of Western Virginia in Roanoke, Va. is embarking on such a move. And, it's getting a new name. The new Taubman Museum of Art, the Art Museum of Western Virginia's new incarnation, will open this fall. The 82,000-sq.-ft facility will house the museum's permanent collection and greatly expand its exhibition and education spaces. As the city's most contemporary structure, it represents Roanoke's metaphorical gateway to the future for a city transforming its industrial- and manufacturing-based economy to one driven by information technology and services.

The new museum's forms and materials interpret the renowned beauty and drama of the Shenandoah Valley and the adjacent Blue Ridge and Appalachian Mountains. A dramatic, spacious atrium that rises to a peak of 75 ft is the hub of the entire facility, while angular exterior walls rise to support a curving roof whose various textures and forms emphasize the rocky surfaces found in the region's caverns, cliffs, and river gorges.

## Scheming in 3D

Structural steel forms the curving roof forms and steel framing is featured as an architectural element in the atrium lobby. The floor levels, while irregularly shaped in plan, each have a constant floor elevation, and were therefore framed using conventional methods. Initially, both steel and concrete floor systems were considered, but a large number of transfer girders and tight floor-to-floor height requirements resulted in steel being used for this portion of the project as well.

The curving roof forms and other complex surface geometries necessitated the use of a 3D modeling program in order to accurately convey the required member work points and geometry to the design and construction teams. Several programs were considered but the design team chose Rhino, which can create, edit, analyze, document, render, animate, and translate NURBS (non-uniform rational B-spline) curves, surfaces, and solids. It proved indispensable from schematic design, through the design stage and into construction administration, when various Rhino files were provided by DeSimone and project architect Randall Stout Architects (RSA) as part of the construction document package.

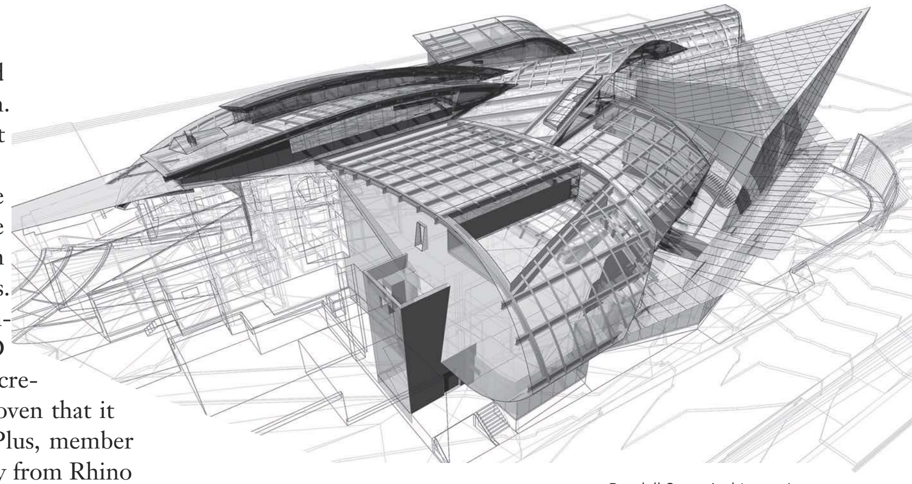
During schematic design, RSA used Rhino to investigate various geometric forms. These architectural models consisted of the curved exterior surfaces, which could be offset to define the top of the steel elevations. The museum's roof consists of several distinct curved surfaces, though each individual surface has a single radius, which greatly simplified construction and reduced fabrication costs. Despite the visual geometric complexity of the roof, its curving shape was achieved with a straightforward structural framing

Randall Stout Architects, Inc.

The Art Museum of Western Virginia's lobby atrium features nine architecturally exposed W14 structural steel columns that each slope 23° from vertical.

system; curved WT4×17 joists spaced at 6 ft supported a metal roof deck that curved in its weak direction. The WT4×17s in turn were supported by straight beams that framed into kinked girders.

Finding column locations that worked with the roof geometry, and that did not interfere with the gallery space below, was a challenge that resulted in numerous column transfers and cantilevered beams. To keep track of the multiple load path transitions running vertically down the building, a single RISA-3D model with both gravity and lateral members was created. Past project experience with RISA-3D had proven that it could accommodate unusual building geometries. Plus, member locations and orientations could be imported directly from Rhino using .dxf files. The composite floor slabs, in turn, were designed using RISAFloor, which seamlessly integrates with RISA-3D.


Randall Stout Architects, Inc.

## Share and Share Alike

During the detailing stage, the structural Rhino model was provided to the steel detailer, Superior Steel, who imported the steel members into the detailing program SDS/2. Due to the unique challenges posed by the building's geometry, the design and construction teams decided to submit and review "shop models" prior to the creation of shop drawings. After the steel detailer completed detailing a portion of the structure in SDS/2, the model was then converted back into a Rhino model and provided to the design team for review. The shop model review was primarily to review member geometries and to identify conflicts with architectural surfaces and building systems; verification of member sizes and connections were done later in the shop drawing phase. Both DeSimone and RSA formally commented on the shop models using Rhino's "dots" and "notes" features, in which members were "tagged" with text dots, directing the detailer to the appropriate comment in the Notes section. If geometry revisions were required by the design team, the affected structural members were remodeled, placed on a separate layer, and highlighted in red—the shop model version of the cloud and delta procedure used for 2D drawings , in which drawing changes are clouded and marked with a revision number.

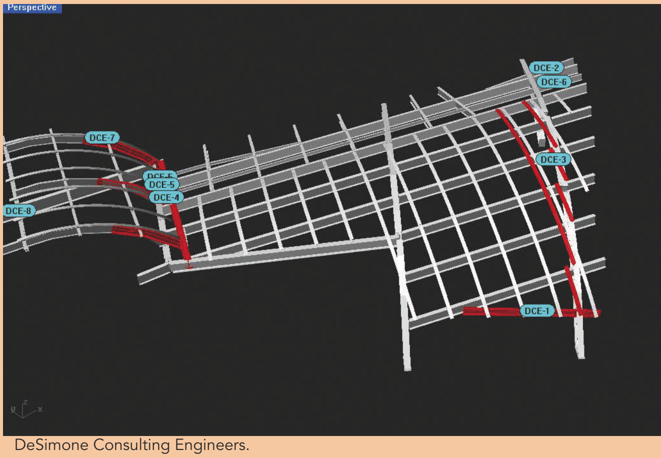
Shop models were also reviewed by the glazing and cladding subcontractors, whose secondary support members attached to the structural steel. During each shop model review period, the design and construction teams, which were scattered across the country, conducted web conferences using Webex, a technology that allows participants to share and view each others' desktop applications. Webex enabled the architect, structural engineer, general contractor, steel detailer, and glazing and cladding subcontractors to simultaneously view a single shop


Randall Stout Architects, Inc.

The museum's exterior is intended to echo the surrounding area's geological features.

### FROM 3D TO 2D

To fully convey the building geometry to the construction team, both RSA and DeSimone provided Rhino models in addition to conventional 2D drawings. RSA's Rhino models contained information such as the exterior surface geometries, mullion patterns, and top-of-concrete elevations. DeSimone's Rhino model contained solid and wireframe modeling of the roof and atrium steel members. The 2D structural drawings contained framing plans, column schedules, sections, and details as would be found in a typical construction drawing set. Only by also providing a Rhino model could geometries such as the curve of the WTs and the kink of the roof girders, be defined. Besides being a powerful visual tool, Rhino also proved useful in creating 2D axionometric views of the more complex geometric forms. (Using the "Make2D" command, Rhino can flatten a model's current view into a series of lines, which can then be imported into AutoCAD to create elevations and details.)



DeSimone Consulting Engineers.

model. In this manner, conflicts were identified and resolved quickly by all affected parties.

### The Atrium, "Knuckle," and Theater Cantilever

The lobby atrium posed a unique structural challenge for DeSimone. It features nine architecturally exposed W14 structural steel columns that each slope 23° from vertical. To stabilize the atrium columns and essentially keep the atrium from tipping over, 2-in.- and 3½-in.-diameter pretensioned rods pull the peak of the atrium and tie it down to the rest of the structure. A continuous ring of HSS22×20 beams further stabilize the atrium columns and support the glazing system, which hangs from the ring using a series of pre-stressed cables that match the slope of the atrium columns. DeSimone performed a non-linear analysis of the roof diaphragm, which consists of 2-in.-diameter pretensioned cross-rods that span between the HSS22×20 beams. Pretensioned rods also span between the atrium columns to provide weak-axis lateral support.

Just northeast of the lobby atrium is a portion of the structure referred to as the "knuckle" in which a conical surface spans vertically between the second and third floor levels. This conical surface, along with several other unusually shaped exterior surfaces, uses ZEPPS, an engineered panel system created by A. Zahner Company, the cladding subcontractor. ZEPPS panels can take on any shape, curve, or bend and are prefabricated in the shop, eliminating the labor-intensive field fabrication associated with build-in-place systems. At the knuckle, the use of ZEPPS meant that a single conical panel needed to be attached to the primary steel structure, which greatly simplified construction and resulted in a better quality building envelope.

Another striking feature of the museum is the "theater cantilever" at the southeast corner of the building. As its name implies, the theater cantilever is supported at the second floor level by a series of W33 beams that cantilever dramatically over the exterior sidewalk. Transfer columns supporting the third floor framing sit on the tips of these cantilevered beams, while HSS "arms" cantilever from the transfer columns to support the curved surfaces that wrap the theater cantilever. Similarly, complex load paths occurred throughout the building structure, requiring careful analysis and evaluation of expected construction sequences and vertical displacements. MSC

*Lisa Minakami is a senior project engineer and Derrick Roorda is a senior associate principal at DeSimone Consulting Engineers in San Francisco.*

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**General Contractor**
Balfour Beatty Construction, Fairfax, Va.

# Stability Analysis: It's not as Hard as You Think

BY CHRISTOPHER M. HEWITT, S.E.

The Direct Analysis Method is a good choice for stability design—and with a little guidance, it can be a relatively simple process.

**STABILITY IS FUNDAMENTAL TO DESIGN, YET IT CAN BE CHALLENGING TO** understand, as many of the current provisions are new. The AISC *Specification* allows designers to use any method of stability analysis that considers each of the following:

- Second-order effects
- Flexural, shear, and axial deformations
- Component and connection deformations
- Member stiffness reduction due to residual stresses
- Geometric imperfections

Each of these effects is considered in all three stability design methods presented in the AISC *Specification* (the Effective Length Method in Section C2.2a, the First-Order Analysis Method in Section C2.2b, and the Direct Analysis Method in Appendix 7). The Direct Analysis Method is discussed here, as it offers an advantage: It eliminates the need to calculate *K*-factors in design. The *K*-factor, a long-standing feature of structural frame design, is well accepted as a means to implicitly capture many of these effects, in spite of its many limitations and underlying assumptions that are rarely satisfied in real structures.

The Effective Length Method is still permitted with minor changes in the AISC *Specification* and remains based in the use of *K*-factors in design. But after evaluating the Direct Analysis Method, you may see that it allows for a more transparent, intuitive, and often easier approach to design for stability.

## Buckling

The most fundamental theoretical formulation in stability design and buckling of columns is the Euler formula, which defines the elastic axial buckling strength of a member as:

$$P = \frac{\pi^2 EI}{L^2}$$

The theory behind this strength equation assumes that the column is perfectly plumb and straight, behaves elastically, and has pinned ends that are restrained against lateral movement. These assumptions all are commonly violated in real structures. In practice, columns lean due to fabrication and erection tolerances and are out-of-straight between braced points due to mill and fabrication tolerances. Residual stresses are present that cause inelastic behavior. Further, as a structure is loaded, deformations and drift also occur, adding second-order forces and moments. Not only must these effects be accounted for in design, but the fact that column buckling always involves both an axial force and bending effects also needs to be recognized.

## Second-Order Effects

When flexure is introduced into an axially loaded member from the axial force acting through the sidesway of a frame and curvature of a member, this is referred to as a second-order effect. The analysis of the structure must be modified to capture the impact of these effects, as they will not be realized in a first-order design model of initially plumb frames and straight members. The primary effects to be considered are $P$-$\Delta$ moments, associated with the sidesway of the structure, and $P$-$\delta$ moments, associated with the curvature of each individual member as it deflects and deforms.

The moments generated by these effects can be captured in the analysis in several ways, and any method that a designer chooses to analyze a structure that captures each of the possible effects is acceptable. Therefore, it is equally permissible to analyze a structure by a direct, rigorous second-order analysis, or to use an approximate method of second-order analysis, such as the one presented in *Specification* Section C2.1b.

In the former case, the frame and member deformations are tracked directly within the analysis software as a part of the analysis. In the latter case, a first-order analysis is made and the resulting forces and moments are amplified using the variables $B_1$ and $B_2$ from the AISC *Specification*. $B_1$ captures the amplification of forces and moments due to the curvature or out-of-straightness of the member ($P$-$\delta$), and $B_2$ captures the amplification of forces and moments from the drift of the frame ($P$-$\Delta$). These effects are illustrated for a single column in Figure 1.

*Christopher Hewitt is a project engineer with Forefront Structural Engineers in Chicago. He is a former senior engineer with AISC.*

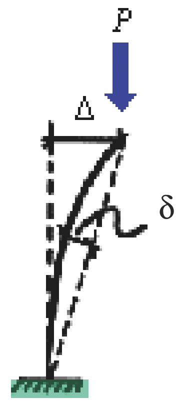


Figure 1. Basic model describing $P$-$\Delta$ and $P$-$\delta$ effects for a single fixed-base column.

### Deformation of the Structure

Engineers are generally familiar with methods of calculating the deflections of members under load. While the structure can be analyzed conventionally for the deflections of individual members, it is important to be sure that these deflections are captured in a second-order analysis of the frame. As stated previously, it is equally permissible to analyze a structure by a direct, rigorous second-order analysis, or to use an approximate method of second-order analysis, such as the one presented in *Specification* Section C2.1b. It also is important to consider the effect of connection and panel-zone deformations in the analysis.

### Residual Stresses

Residual stresses are introduced into structural shapes as a result of the production process. Residual stresses include stresses due to temperature, as some elements of the hot rolled cross-section will cool faster than others, and also due to the effects of straightening that must be done to meet ASTM A6 tolerances. Areas with residual stress will yield prior to the overall yielding of the section, causing the column to lose some of its stiffness before reaching its theoretical buckling strength. The effects of residual stresses on member strength are accounted for in the column equations. However, the loss of stiffness due to residual stresses also will increase the frame and member deformations. This is accounted for in the Direct Analysis Method by using a reduced stiffness for all members in the analysis: multiplying the axial stiffness, $EA$, of all members by 0.8 and multiplying the flexural stiffness, $EI$, of all members by $0.8\tau_b$, where $\tau_b$ is the column stiffness reduction factor.

### Geometric Imperfections

Geometric imperfections are inherent in all structures, and limits on these are found in the AISC *Code of Standard Practice* (plumbness of frames) and the ASTM standards for structural shapes (straightness of members). Frame out-of-plumbness is modeled directly in the Direct Analysis Method using notional loads acting laterally at each floor (alternatively, this can be done by direct modeling of the out-of-plumb frame geometry, if it is known).

A notional load is an equivalent lateral load of appropriate magnitude such that it will generate a story shear in the structural model equivalent to the effect of the axial loads in a story acting in the deformed position, as illustrated in Figure 2. According to the AISC *Code of Standard Practice*, the permissible tolerance on out-of-plumbness of any individual column is no larger than $L/500$, and the notional load is specified to generate a story shear corresponding to this amount of out-of-plumbness. A horizontal notional load of 0.002 times the story gravity load in the horizontal direction is applied, with the 0.002 coefficient being equal to 1/500—the erection tolerance permitted by the *Code of Standard Practice*.

### Leaning Columns

In any stability analysis, it is necessary to capture the destabilizing effects of columns that rely on the lateral frame for stability but are not a part of the lateral frame. These columns with pinned ends are commonly referred to as "leaning columns." When modeling the frame, leaning-column effects can be captured either by developing a complete 3D model of the frame or by assigning a single equivalent leaning column carrying the summation of all of the gravity loads on all of the leaning columns in the structure, as a pin-connected part of a 2D frame. An example of how this might be modeled in a 2D analysis is shown in Figure 3.

### Step-by-Step Analysis

Now that you know the basics, here is a simple step-by-step process to guide you as you use the Direct Analysis Method:

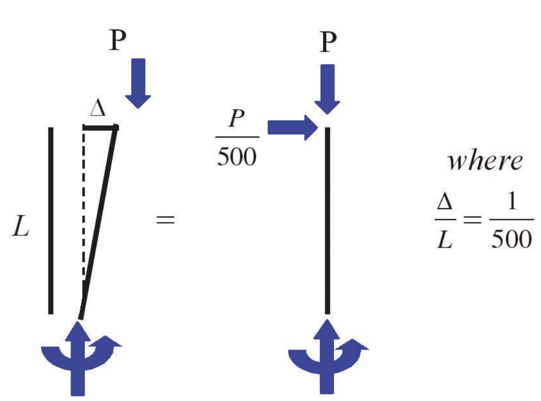


Figure 2. Equivalent loading using notional loads to represent the effect of geometric imperfections on a column.

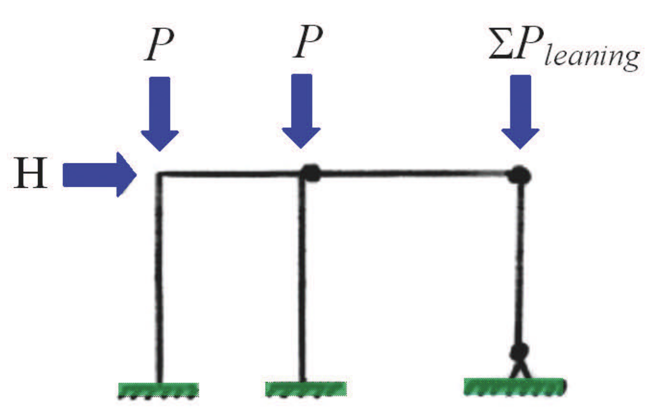


Figure 3. 2D frame model that captures leaning column effects.

1. Create a model of the lateral frame being analyzed, including the leaning columns.
2. Reduce the stiffness of lateral framing members in your model.
   a) **For a braced frame** modeled with pinned connections, this can be done by modifying the modulus of elasticity in your model to 0.8 times $E$ (23,200 ksi).
   b) **For a moment frame** (or a braced frame modeled with rigidly connected members), this can be done by modifying the modulus of elasticity in your model to $0.8\tau_b E$. As an alternative to applying $\tau_b$, add a notional load of 0.001 times the gravity load to the notional load in step 3.*
3. Apply notional loads to all load combinations equal to 0.002 times the gravity load† on each story at the story level.**
4. Conduct a second-order analysis of the structure under applied loads, either by the Amplified First-Order Analysis approach ($B_1/B_2$) or by completing a direct, rigorous second-order analysis of the structure.
5. Reset your modulus of elasticity to $E$ = 29,000 ksi and design the members using the equations in the AISC *Specification* to resist the forces that you have just determined, with $K = 1$.
6. Check drift limits for wind and seismic design.
7. Pat yourself on the back for doing a great job on your analysis! MSC

*This topic is discussed in greater detail in a presentation by R. Shankar Nair, S.E., Ph.D.—the 2007 AISC T.R. Higgins Lecture—which can be viewed for free at www.aisc.org/boxedlunch.*

**Footnotes**

* $\tau_b$ may be taken as 1 if $\alpha P_r < P_y$. This may, however, require iteration in the analysis if the member sizes change and the inequality reverses. Moment frames will often satisfy this inequality, allowing $\tau_b$ to be taken as 1.

† The term "gravity load" refers to either the LRFD gravity load combination or 1.6 times the ASD gravity load combination on each story.

** Per the AISC *Specification*, notional loads only need to be added to load combinations in which the notional load is larger than the lateral load on the frame. Thus, notional loads usually can be ignored in all but the gravity-only load combinations. However, if a designer wishes to simplify the design process, it is always conservative to include the notional loads.

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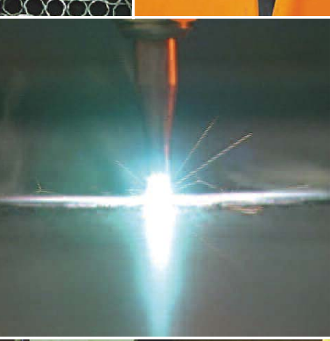

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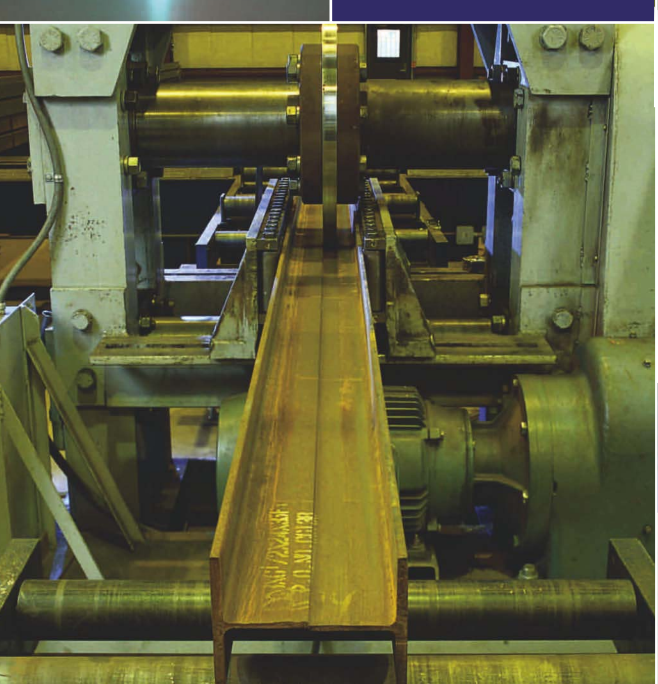

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# The Quality Diet: Building A Healthy Business

BY TIMOTHY J. FOLKERTS

**Implementing a corporate quality system is much like dieting—and there's a lot more to both of these processes than you might think.**

**LET'S FACE IT: QUALITY IS NOT ALWAYS AN EASY SELL.** Explaining the principles and techniques to people outside the field can be challenging. Convincing management to invest in the effort can require considerable perseverance. Sacrificing the hard cash you'll get from today's shipment for future good will is the sort of delayed gratification businesses aren't always ready to accept. At every turn there is pressure to cut corners, hide problems, and just go along.

When facing such challenges, it is helpful to remember one simple analogy: Succeeding with quality improvement is like succeeding with a diet.

## Quality Pros/Corporate Dietitians

Quality professionals could be called corporate dietitians—there to guide businesses to healthier lifestyles.

A poor diet leaves a person vulnerable. Carrying around extra weight makes everything just a little more difficult and time consuming. Chronic problems like diabetes are aggravated by obesity. Hidden problems like high cholesterol caused by too much saturated fat or high blood pressure caused by too much dietary sodium can lead to sudden, life-threatening heart attacks. Of course, better nutrition isn't a cure-all for these problems but it certainly can limit the risks.

Similarly, poor quality leaves a company vulnerable. Inefficient procedures and management make everything just a little more difficult and time consuming. Chronic cost overruns due to poor quality of incoming materials and supplies sap competitiveness. Hidden problems in the quality of outgoing products can lead to sudden, bankrupting product recalls or lawsuits. Better quality isn't a cure-all for these problems but it certainly limits the risks.

With people, survival of the fittest has been mitigated somewhat. We have tamed the world around us and eliminated many natural hazards. We have family and friends to support us when we are sick or weak.

The corporate world is not so forgiving. When times are good, it is possible for a poorly performing company to survive. When times are bad, the wolves will descend, culling many of the weak and inefficient.

## Potential Gains

Many people don't realize that the primary purpose of a diet should not be to lose weight. Instead, the primary purpose should be to improve health. Weight loss is just a pleasant, visible side effect. In fact, the original motivation for the diet could be something besides weight loss; it could be reducing sodium, fat, and cholesterol, for example. Concentrating simply on one facet of the diet can lead to poor nutrition—insufficient vitamins, minerals, or protein for proper health. Taken to an extreme, concentrating exclusively on weight loss can lead to anorexia—a life-threatening condition.

For a corporation, the primary goal of a quality improvement initiative is improved corporate health. A single-minded effort aimed at cutting costs won't do. Neither will efforts aimed solely at increasing quarterly profits, eliminating defects, or pushing more products out the door.

While costs, quarterly profits, defects, and production all are important, pursuing any one too aggressively will only lead to problems. Excessive cost-cutting will leave a company anemic. Relentless pursuit of short-term profits often simply delays problems, as Enron learned. Reducing defects significantly is often possible, but making the products perfect becomes cost-prohibitive. Raising production without improving or at least maintaining quality can lead to loss of customer loyalty and sales.

Quality is not just about reducing fat but also about improving the true viability of a company.

## Choosing an Approach

The potential approaches to dieting and weight loss are almost limitless: low fat, low carb, vegetarian, replacing meals with diet shakes, skipping dessert, eating lots of grapefruit, using diet pills advertised on late-night television, proprietary programs like Jenny Craig and Weight Watchers, fasting, gastric surgery, and liposuction.

Many approaches can be quite effective at both weight loss and improved health, some are mildly helpful, but a few are actually dangerous. You can go it alone or join a group. You can sign up for a brand name plan or create your own. You can mix and match several different approaches. You can get advice from an expert or buy a book or simply jump on the latest bandwagon.

Ideally, a person interested in improved health should work with a dietitian or nutritionist to develop the right plan. The expert can ensure the diet will help achieve desired goals while still providing the balanced nutrition required to maintain health.

Similarly, a company interested in quality improvement ideally should use quality professionals to develop the plan that is right for that company, because quality improvement choices are

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

as varied as diets: total quality management, plan-do-check or study-act, statistical process control, design of experiments, acceptance sampling, multi-vari plots, ISO 9001, TS 16949, lean, Six Sigma, lean Six Sigma, Dorian Shanin's Red X, W. Edwards Deming's 14 points, Joseph M. Juran's quality trilogy, and Philip Crosby's zero defects, to name several.

The right method, or combination of methods, will improve the viability of an organization by improving products and processes. The wrong approach could actually weaken an organization by diverting resources to the wrong problems.

### Implementing the Plan

Even after a plan of attack has been chosen, success is still a long way off, and that success cannot be achieved by the dietitian. The dietitian can educate the client about nutrition issues. The dietitian can encourage the client to develop a support network. The dietitian can provide forms to track progress. The dietitian can applaud success and watch for backsliding. The dietitian can set up weekly or monthly meetings to provide in-person feedback.

Ultimately, though, no matter how well-crafted the plan, it won't succeed unless the client acts on the plan. The client needs to be committed to improved health.

Quality professionals face the same sort of challenge. Corporate leaders might say they want improvement but not carry through with appropriate action. It is important to educate and encourage. It is helpful to get suppliers and customers on board. It is valuable to track progress—anything to provide motivation and keep focused on the ultimate goals. The quality professional provides his or her support, but without leadership buy-in to quality improvement, it can't and won't happen.

### Potential Pitfalls

Quality improvement and dieting can encounter similar pitfalls. Once you recognize the following five potential pitfalls, you improve your chances of overcoming them, allowing you to lead your organization to quality improvement and business health:

1. Lack of tact
2. Not understanding the system
3. Focusing on the short term
4. Yo-yo quality
5. False economy

**Lack of tact:** People don't like to be told they are fat, and bosses don't like to be told they are running a poor-quality operation.

With a receptive boss, a direct approach might work best but make sure you have a plan before you start pointing out problems. Even though a wise leader will acknowledge the truth and recognize that something must be done, he or she must be approached tactfully. Issues with suppliers or customers can often provide an opening to broach the subject.

At other times a stealth quality approach might be better. This is the equivalent of not telling your overweight spouse that you are now buying low-fat ground beef and diet desserts. After gains have been realized in small ways, the boss might be more receptive to learning about bigger improvement opportunities.

**Not understanding the system:** Weight Watchers uses a point system to monitor food intake. For example, one large apple is two points. If you aren't careful, it is easy to miscount points. If you make the mistake of counting one apple as one point several times, the diet will fail.

Similarly, small misunderstandings can lead to the failure of quality improvement initiatives. Whether it is miscalculation, miscommunication, miscalibration, or any number of other mistakes, quality plans have the potential to misfire if key ideas about the system are not understood.

**Focusing on the short term:** It is not uncommon to lose an unusually large amount of weight during the first week of a diet. This can set up unrealistic expectations and lead to later discouragement. It is also quite possible to stick faithfully to a plan, but see different amounts of weight loss each week. The dieter might be tempted to restrict calorie intake even more following a week of poor performance or to ease up after a week of exceptional performance.

**Yo-yo quality:** Many managers don't seem to have time to do things right the first time but they always seem to have time to fix things later. This is the equivalent of yo-yo dieting. You overeat and exercise too little for a couple months, and then you discover you've put on 20 pounds. For the next two months, you have to work three times as hard to lose those pounds and get back in shape. Maintaining steady progress all along is much easier overall than reacting to fix self-created problems.

**False economy:** Skipping lunch but then eating three candy bars in the afternoon because you are starving is not effective for dieting. Bragging about all the lunches you've skipped won't change the fact that you are still gaining weight.

Similarly, cutting costs in one division but passing along equal or greater costs to other divisions is not a viable business plan. The manager or vice president of the cost-cutting division might be able to brag about improvements but the improvements don't prevent the business from losing more money than before.

## Lasting Success

Quality can't be temporary. Quality can't be just a slogan. Quality can't be the job of just one person or one department. Quality can't be subordinate to today's production quotas.

Ultimately, success means a fundamental change in corporate lifestyle. As long as the mentality is, "We just need to do this until we reach our goals," or, "We just need to do this until the boss gets a new pet project," then failure is never far away.

So, the next time you have one minute to explain what quality professionals do, tell them you are the corporate dietitian. You have the plans to make the company healthier by eliminating poor habits that sap profits. You champion improvements that make everyone look better. You study and educate and encourage and implement to make those improvements a reality. And you would be happy to share that knowledge with them anytime they want to know more. MSC

*Timothy J. Folkerts is a physics instructor at Barton County Community College in Great Bend, Kan. He earned a doctorate in physics from the University of California at Davis. Folkerts is a member of ASQ and a certified quality engineer and reliability engineer.*

*A longer version of this article first appeared in the May 2007 issue of* Quality Progress. *This excerpt is reprinted with permission.*


# HOW SOLID STEEL HELPED STRENGTHEN OKLAHOMA PIGSKIN.

*Because Oklahoma weather is so volatile, the OU Sooners desperately needed an indoor practice facility. But the construction site was so small, there was no room to store the mammoth bowstring and long-span joists needed to support the building's roof. So when the Nucor Vulcraft Group was able to deliver the steel in precisely timed waves, we were able to help finish the project ahead of time. And give the Sooners the ability to practice through rain, sleet, snow and dark of night. 24/7 if need be. Sorry about that, guys.*

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# A FAIR AND BALANCED OFFER

BY TIM R. JOHNSON

**Framing a job offer properly can be the difference between landing a top-notch candidate and scaring him away.**

**WHEN PURSUING GAINFULLY EMPLOYED CANDIDATES—** those top-talent hires that mean so much to the success of a firm—framing the best possible offer is critical. Approaching those best-of-the-best with new employment opportunities virtually always requires both promotional possibilities and a compensatory increase.

Recruitment that includes relocation almost always requires a salary increase of 15% or more, I've found—especially in the current real estate market—and usually comes with a whole grab-bag of surprise challenges. That means the most viable hires needed for company growth and success are just next door, on the staff of local competitors. Attracting talent from nearby competition, however, is no simple task itself; it requires a careful approach. Perhaps most important is framing the best possible offer. Here's an example of a common mistake that is easily overlooked, and just as easily overcome when handled appropriately.

In pursuing a top-notch candidate, a client of mine recently faced the challenge of how to frame the best offer. (To protect the innocent—and myself—let's call the candidate Joe and the company Tobinelli Associates.) Joe was interested in working for Tobinelli Associates upon the first introduction, and viewed the position as a highly advantageous next step in his career. After several meetings, both Joe and Tobinelli Associates felt that there was an excellent connection on both cultural and personal levels. Everything seemed to align well in everyone's favor. Then, the offer went out. And Joe turned it down.

Joe rejected the position because of money, even though he was well within the targeted salary range and was not looking for an unreasonable increase. Tobinelli could have easily overcome this negative reaction had they managed the offer stage more effectively.

Early on in the recruitment process, Joe made it very clear that his principal interests were a fair salary boost and opportunity for increased responsibilities in a new position. Both interests weighed in equally. Tobinelli wasn't sure they could deliver completely on the salary side, and were looking to offer about an 8% increase. To make up for what they felt was a shortcoming, Tobinelli framed the position as a high management post with several staff reports and significant business development responsibilities. The intention was to make the job sound "big"—in fact, "bigger" than it truly was—so as to inflate the growth opportunity in place of a much higher salary.

After listening to this new pitch, Joe actually asked for *more* money than he originally wanted, because Tobinelli made the job seem like it entailed significantly more responsibility than Joe had initially anticipated. Tobinelli countered with a revised offer—more than they first intended to pay—but it still wasn't enough, because they had already cast the die for the position, and it was too late to amend the job description. Joe decided to stay with his current employer, in a position with fewer responsibilities, and at a lower salary than he could have accepted.

What could Tobinelli Associates have done differently? First, never oversell responsibility to balance out what may be a lack in immediate monetary compensation, because that will have an adverse effect and cause candidates to want more money. Instead, sell *opportunity*. Explain that the position pays a certain figure on day one, but enumerate the opportunities for career growth (and with that, compensation growth) months and years down the line.

There's more to it than money, of course. For example, Joe mentioned during the interview that he desired to pursue an MBA. Tobinelli could have focused on the fact that they offer an excellent tuition reimbursement plan, and could have also explained to Joe the additional opportunities that would have been available to him once he earned that degree. Tobinelli should have focused on strong short- and long-term selling points, rather than trying to play a balancing act with salary and responsibilities alone.

The bottom line is that, regardless of what many say, salary is going to be a primary issue when hiring managerial staff members. But the salary needn't always be delivered on the first day if a firm has benefits and culture that are attractive to prospective candidates. Hiring firms always need to listen to cues from candidates about non-salary desires and focus on ways to help a candidate parlay into greater responsibilities and higher salaries in the future. Frame offers carefully. With responsibilities on one side and salary on the other, don't overcompensate for one to make up for a lack of the other. The key to a properly presented offer is balance, and when a firm looks at all angles, there are certainly more than two sides that hold weight. MSC

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# SUPPORTING ROLE

## Erecting steel joists without proper bridging can cause it all to come crashing down.

BY PERRY S. GREEN, PH.D., AND TIM HOLTERMANN, P.E.

**ONE OF THE MOST IMPORTANT ASPECTS OF SAFE STEEL JOIST ERECTION IS PROPER BRIDGING.** A joist may be unstable as soon as the hoisting cable is released, even if other applied construction loads aren't present. As such, three types of joist bridging play three different roles in creating stability: erection, construction, and permanent.

Erection bridging is required if the joist is not stable under the combination of its own weight plus the weight of the worker releasing the hoisting cable at the joist mid-point. Erection bridging is defined in the OSHA Federal Safety Standards, 29 CFR 1926.757, as "the bolted diagonal bridging that is required to be installed prior to releasing the hoisting cables from the steel joists." However, not all joists require erection bridging when being set into position on a structure.

The second bridging type—construction bridging—provides adequate lateral support for a joist and any construction loads on top of the joist while a permanent means of top chord lateral support, normally metal decking, is installed. Without proper lateral support, loads during construction can cause the joist to distort, roll over, or shift from its intended position, resulting in the ironworkers, joists, or construction loads—or all three—falling and causing injury and damage. Proper construction bridging can prevent such mishaps.

Finally, permanent bridging serves to permanently brace the top and bottom chords of the joist; it remains as part of the structural system for the life of the structure. In cases where the metal decking creates a sufficient diaphragm, permanent bridging isn't necessary at the top chord after construction is complete. One exception, however, is when a standing-seam metal roof is attached directly to a joist top chord. The Steel Joist Institute (SJI) has determined that this type of roofing system does not provide the required diaphragm strength, and permanent bridging is necessary for the joists to function properly.

For the joist bottom chord, permanent bridging is required. Since the bottom chord serves to laterally brace the joist web members—when modeled as pinned ends—it too must be braced laterally, even when it is in tension. Also, when joists are used in roof construction, they are frequently subject to a net uplift force, creating compression in the bottom chord. In this scenario, permanent bridging in the bottom chord will provide the necessary buckling resistance.

Let's take a closer look at all three types of steel joist bridging.

### Erection Bridging

The SJI K-Series and LH- / DLH-Series Load Tables in the 42nd Edition *Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders* show the total safe uniformly distributed loads for standard products at various spans. As the span increases for a particular joist designation, the uniformly distributed load-carrying capacity decreases. The Load Tables also indicate when the span becomes too great for a particular joist designation to be erected without erection bridging.

Table A, Erection Bridging for Short-Span Joists, in the OSHA *Federal Safety Standards* gives the minimum span for each short-span joist designation (e.g., 26K8) and indicates when erection bridging must be installed. If Table A indicates that erection bridging is not mandatory (NM), the joists can be spaced out, attached, and then bridged in accordance with Section 6 of the SJI *Standard Specifications for Open Web Steel Joists, K-Series*.

The required bolted diagonal erection bridging for K-Series joists must be installed as the row of bridging nearest the mid-span of the joist. The erection bridging must also be anchored to prevent lateral movement of the joist prior to the hoisting cables being released. This can be accomplished by securing the bridging to a fixed object such as a concrete or masonry wall, steel beam, or other stable portion of the structure. OSHA refers to this anchorage point as a "bridging terminus point."

Table B, Erection Bridging for Long-Span Joists, in the OSHA Standards gives the minimum span for each long-span joist designation (e.g., 32LH06) and indicates when erection bridging must be installed. If Table B indicates erection bridging as NM, the joists can be spaced out, attached, and then bridged in accordance with Section 105 of the SJI *Standard Specifications for Longspan Joists, LH-Series*.

The required bolted diagonal erection bridging for any LH-Series joist depends on its length. Where the span of the steel joist is less than 60 ft, the bolted diagonal erection bridging must be installed as the row of bridging nearest the mid-span of the joist. Where the span of the steel joist is between 60 ft and 100 ft, the required bolted diagonal erection bridging must be installed as the two rows of bridging nearest the third points of the joist. Where the span of the steel joist is 100 ft to 144 ft, all rows of bridging are considered erection bridging and must be completely installed. As stated in Section 105 of the LH-Series Specification, the bridging row(s) must be anchored to prevent lateral movement of the joist.

Since all erection bridging will in turn become construction bridging, the more stringent construction bridging criteria are used to establish the erection bridging forces, sizes, and connections.

### Bridging for Construction Loads

After any required erection bridging is installed and the hoisting cables have been released, additional bridging rows required for construction bridging need to be installed before the application of additional construction loads. Under no circumstances should construction loads of any description to be placed on unbridged joists. As previously described, many joists are laterally unstable until the joists are properly bridged and the bridging and joists are properly anchored. The joists should be completely bridged immediately after final placement and end attachment is completed in accordance with OSHA and SJI requirements.

Construction loads are defined in the OSHA *Federal Safety Standards* as "any load other than the employee(s), the joists, and the bridging bundles." These loads include the weight of metal deck bundles and individual sheets being placed, the weight of multiple erectors placing the deck, and equipment loads such as welding machines and leads, hand tools, bridging for adjacent bays, etc. The OSHA Standards strictly prohibit placing construction loads on unbridged joists and give the proper procedure for landing bridging bundles on unbridged joists; see 29 CFR 1926.757 (e) (1), (2), and (3). It is critical that construction loads on any one joist be minimized, and it is advisable that the loads be placed as close as possible to the ends of the joist. There is an exception for the placement of a bundle of decking after the installation of at least one but not all bridging rows if certain stringent conditions are met. Any erector who allows construction loads to be placed on unbridged joists is in direct violation of the OSHA standards, as well as Section 6 and Section 105 of SJI's K-Series and LH- / DLH-Series Specifications.

SJI *Standard Specifications for Open Web Steel Joists, K-Series*, Section 5.4 requires that each bridging connection resist a nominal (unfactored) horizontal force of not less than 700 lb. The spacing of the bridging rows is determined by the radius of gyration of the top chord about its vertical axis and shall not be less than $\ell/145$. Also, to meet this criteria the quantity of top chord bridging rows shall not be less than the number shown in Table 5.4-1. The number of rows of bottom chord bridging shall not be less than the number of top chord rows; the top and bottom chord bridging rows may be spaced independently.

SJI *Standard Specifications for Longspan Steel Joists, LH-Series and Deep Longspan Steel Joists, DLH-Series*, Section 104 requires that each bridging connection to the joist must be able to resist a horizontal force not less than that specified in Table 104.5-1. Where two attachment points to a joist are utilized, each attachment must be able to resist one-half of the bridging force given in the table. The spacing of the bridging rows shall be determined by the radius of gyration of the top chord about its vertical axis and shall not be less than $\ell/170$, and to meet this criteria the maximum spacing of lines of top chord bridging shall not exceed the values in Table 104.5-1. The number of rows of bottom chord bridging shall not be less than the number of top chord rows; the top and bottom chord bridging rows may be spaced independently.

The bracing force that a joist imparts on the bridging is based on three assumptions: 1) an initial out-of-straightness in the chord of $\ell/920$; 2) a resultant total nominal bracing force of $0.0044P$ (in other words, the horizontal bridging rows must be continuous and each joist must be braced from both sides; therefore, the total bracing force is divided by two and rounded up to $0.0025P$, where $P$ represents the chord axial force); and 3) there is an assumed construction stress in the top chord due to the chord axial force $P$. For K-Series joists, the bridging criteria are based on a top chord axial construction ultimate stress ($F_{cr}$) of 17 ksi. Due to the continuity of the top chord on either side of the bridging attachments, a $K$-factor of 0.9 is used in calculating the top chord slenderness ratio. Hence, for an ultimate Euler stress of 17 ksi and a $K$-factor of 0.9, the permissible slenderness ratio $l/r_y$ is limited to 145 for K-Series as given above. LH- and DLH-Series joists are similar, except that the assumed construction stress is taken as 12 ksi, and the resultant slenderness limit, $l/r_y$, is 170.

Therefore, the nominal compressive force that accumulates in a horizontal bridging row is:

$$P_{br} = 0.0025\, n\, A_t F_{construction}$$

where

$F_{construction}$ = 17 ksi for K-Series joists, and 12 ksi for LH- and DLH-Series joists, as noted above

$A_t$ = is the top chord area

$n$ = the number of joists

For horizontal bridging, $n$ is taken as eight joist spaces because the construction loads tend to be localized, rather than spread uniformly over an entire bay. Also, the probability is low that all joists in a bay would exhibit the maximum out-of-straightness all in the same direction at any given time. For horizontal bridging, the bracing force $P_{br}$ must be taken in compression. Diagonal bridging creates a load path whereby the forces are resolved at each braced joist space and do not accumulate. However, recall that the bracing force was divided by two on the presumption of continuous bridging on each side of the joist chord. Since continuity is not required of diagonal bridging rows, and to allow the diagonal bridging force to be considered only in tension, $n$ is taken as two for diagonal bridging.

The tables provided in the SJI specifications for bridging sizes are based upon the force $P_{br}$ for the typical top chord areas for a given designation. Table 2.6-1a of the SJI *Code of Standard Practice* gives the maximum joist spacing for certain horizontal bridging sizes based on joist chord section numbers for K-Series joists and using $K$ = 0.9 for the bridging design; Table 2.6-1b provides the same information for LH- and DLH-Series joists. Diagonal bridging is only subject to tension forces, and so the bridging size is governed by a slenderness limit (between connections) of 200, rather than by strength.

Recently, SJI has begun to investigate the difference between the assumed K-Series construction stress of 17 ksi verses 12 ksi for LH- and DLH-Series joists. Preliminary findings indicate that the construction stress has very little to do with the chosen joist series but is heavily influenced by both the span and depth of a joist. For a given span length and joist spacing, the construction load arguably will be the same regardless of joist depth, while the top chord construction stress will clearly be less for a deeper joist. On this basis, equations were developed for the newest SJI *Standard Specification for Composite Steel Joists, CJ-Series*, in which $F_{construction}$ and the top chord slenderness limit varies depending on the depth and span length as follows:

$$F_{construction} = \frac{\pi^2 E}{\left(\dfrac{0.9\ell_{br}}{r_y}\right)^2} \geq 12.2 \text{ ksi}$$

$$\ell_{br} = \left(100 + 0.67 d_j + 40 \frac{d_j}{L}\right) r_y$$

but not greater than, $\ell_{br} = 170\, r_y$

It is anticipated that a similar methodology may be adopted for the K- and LH-/DLH-Series joists in the future. However, they would not exactly match the CJ-Series equations, because the CJ-Series allows for an ultimate top chord construction stress in excess of 17 ksi. This is due to the fact that composite joist top chords are inherently smaller than comparable non-composite top chord sizes, and the maximum span to depth ratio is greater for composite than non-composite joists.

In certain design applications that use bottom bearing or square-ended joists, the product is designed to bear on the bottom chord. This produces a "top-heavy" condition. Therefore, their ends must be restrained laterally in accordance with SJI *Standard Specifications*, *K-Series*, Section 5.4(d) or LH- and DLH-Series Section 104.5(f). This is accomplished by means of an additional row of diagonal bridging placed at or near the bearing support ends of the joists as they are being erected. Where a bottom bearing joist is extended beyond its support to form a cantilevered end, a row of diagonal bridging near the support should first be installed. In addition, the structural drawings may indicate a row of diagonal bridging in the cantilevered portion to provide lateral stability. If the joists have bottom chords extended over and connected to a column, beam, wall, or other structure, the connection should be made in accordance with the structural drawings and/or instructions from the engineer of record.

### Bridging for Permanent Loads

Top chord bridging serves a role as permanent bridging in the absence of a deck diaphragm, as is the case with a standing-seam metal roof applied directly to the joist top chords. Sections 5.8(g) and 104.9(g) of the SJI *Standard Specifications* for K-Series and LH-/DLH-Series joists, respectively, provide the requirements for the horizontal bridging design. The compressive force equation is:

$$P_{br} = 0.0025nP$$

This equation is similar to the equation given above for construction bridging, but here, $n$ is not limited to eight, and is equal to the total number of joists between end anchors. $P$ represents the actual top chord design force, rather than the chord area times an arbitrary construction stress.

Bottom chord bridging is always permanent bridging, and either limits slenderness for bottom chords in tension or braces the bottom chord laterally for compression forces, such as those present in a net uplift loading case. Traditionally, and for simplicity in the field, the bottom chord bridging size is equal to the size as determined for the top chord.

When uplift forces are a design consideration, a row of bottom chord bridging is required near each end of short-span joists in accordance with the SJI *Standard Specifications*, *K-Series*, Sections 5.6 and 5.11 and long-span joists in accordance with the SJI *Standard Specifications*, *LH-/DLH-Series*, Sections 104.7 and 104.12.

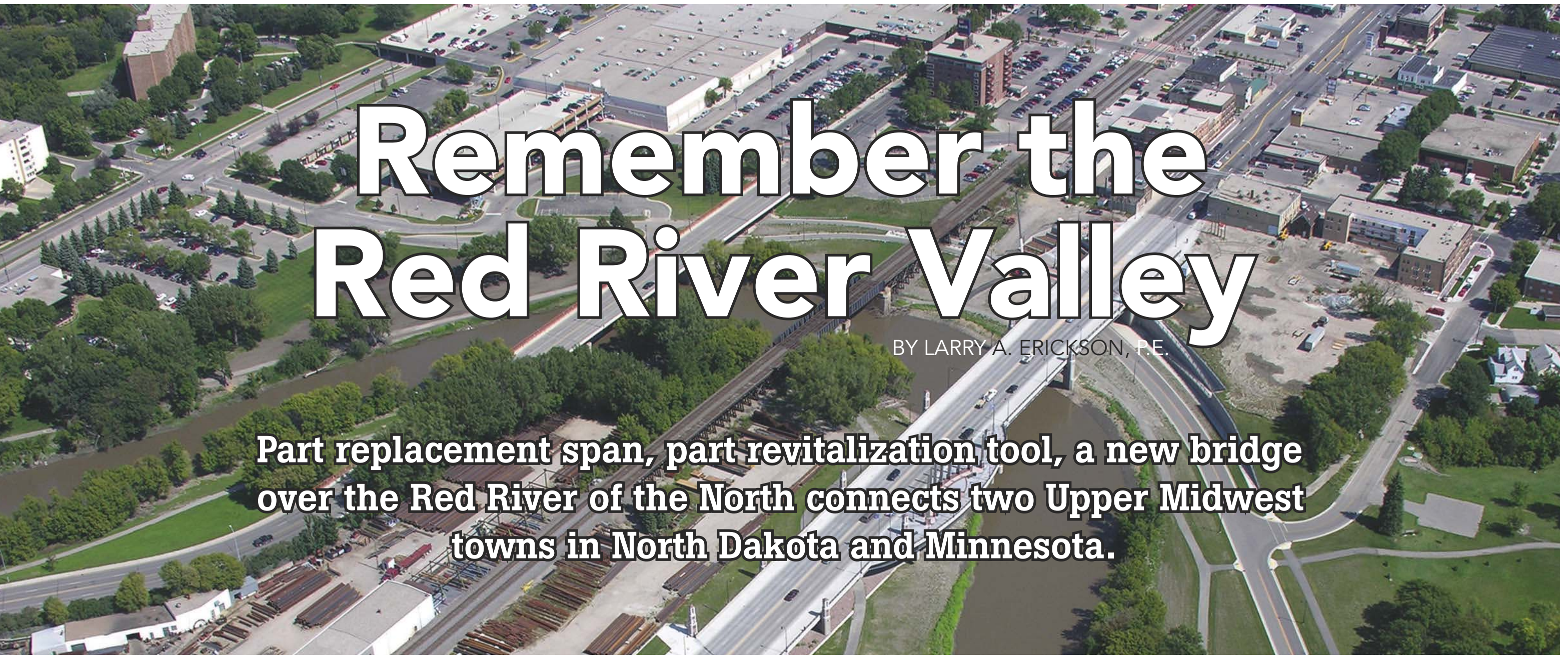
### What's Next?

Future research on joist bridging requirements is likely to include a review of the construction stress levels, a unification of the K-Series and LH-/DLH-Series bridging requirements, and a better understanding of the accumulation of bridging forces when a net uplift loading condition is present. MSC

*Perry S. Green is technical director of the Steel Joist Institute, and Tim Holtermann is chairman of SJI's Engineering Practice Committee and corporate engineering manager of Canam Steel Corp.*

*The SJI* 42nd Edition Catalog *containing the above referenced* Standard Specifications *and the* Code of Standard Practice *can be found at www.steeljoist.org*.

Steel Bridge NEWS
NATIONAL STEEL BRIDGE ALLIANCE
SEPTEMBER 2008



# Remember the Red River Valley

BY LARRY A. ERICKSON, P.E.

**Part replacement span, part revitalization tool, a new bridge over the Red River of the North connects two Upper Midwest towns in North Dakota and Minnesota.**

**WHEN MOST PEOPLE HEAR "RED RIVER," THEY LIKELY THINK OF THE BORDER BETWEEN TEXAS AND OKLAHOMA.** But there's a second Red River—also serving as a border between two states—further north.

A major crossing of this "other" Red River was recently updated: the Main Avenue Bridge (Trunk Highway 10), which crosses the river between Moorhead, Minn. and Fargo, N.D. A combination of geometric and structural deficiencies, along with flooding issues, led the Minnesota and North Dakota Departments of Transportation to replace the existing bridge, which was constructed in 1936. While the bridge replacement was the project's initial focus, the new structure also became an integral component to the area's riverfront and Moorhead's downtown revitalization efforts.

The result is a state-of-the-art, environmentally durable vehicular structure that is able to handle growing traffic volumes and addresses improvements to substructure movement. Developing a steel girder system was key to accommodating the possibility of "adding on" aesthetic enhancements that reflected a sense of connectivity between the two neighboring cities.

With five spans, the new 800-ft-long steel girder bridge incorporates several architectural enhancements, creating a recognizable regional landmark that melds engineering, art, and design. The project consisted of replacing the original bridge, reconstruction of roadway approaches in both cities, and reconstruction of Third Street in Moorhead as a riverfront parkway. The bridge also features a unique mid-span plaza, bridge-head plazas on either end of

A pedestrian promenade on the mid-span plaza provides views of the Red River and riverfront parks.

The curved mid-span plaza girders sweep over the Red River, bearing only at the pier and framing into the bridge girders.

the structure, and grand stairs that provide a pedestrian connection to the riverfront.

## Unstable Conditions

The selection of steel for the superstructure ultimately turned out to be the only possible solution, given the desire to provide an arched mid-span plaza—not to mention the need to hold up to the soil conditions in the Red River Valley. The Fargo-Moorhead area lies in a region of poor soil/construction conditions and is prone to geological instability of the riverbanks. In particular, the channel walls of the Red River and its tributaries are prone to slope failure and soil movement towards the river.

If the piers were to move slightly, steel would be easier to jack and reset the bearings. To offset potential movement of the piers, the bridge designers used larger, high-capacity 16-in. steel sheet piles. These were used at a higher pile spacing than usual to allow for soil movement between the piles. The bridge designer implemented unique design considerations for movement of the abutment and piers, which is not unusual for structures in this area.

Another precaution taken to address soil movement was to design the bearing assemblies with a PTFE bearing system, the first of its kind used in the two bordering states. The stainless steel-plated sole plate was oversized in the longitudinal direction, allowing for bearing adjustment of the Teflon bearing plate in the event of slope movement.

## A Pleasing Plaza

The 50-ft x 250-ft mid-span public plaza was a distinctive attribute to the river crossing. A steel girder system was selected, as it provided the ability to fabricate main girder members to fit the curvilinear outside shape of the plaza. The use of steel allowed the necessary flexibility in the shape, size, and visual appearance of the plaza. The plaza was designed as a curved shape so the girders would sweep over the Red River, bearing only at the piers and efficiently supporting the structure.

Another design feature of the public plaza was the use of stainless steel spires to create a series of vertical elements that augment the design of the public plaza and house vertical fiber-optic lighting. The aesthetic lighting supplements the roadway lighting and continually changes color to tie into seasonal or other themes, such as displaying a stylized "aurora borealis."

The bridge has received numerous awards for its engineering complexity and thoughtful application of artistic and interpretive amenities. It is an iconic gateway for both cities and offers an enhanced urban environment and usage experience for travelers, while also serving as a gathering place for civic celebrations and community enjoyment. MSC

*Larry Erickson of SRF Consulting Group was the lead bridge engineer for this project and has been involved in the design of more than 280 bridges. He has managed several major river crossings during his 30-year career.*

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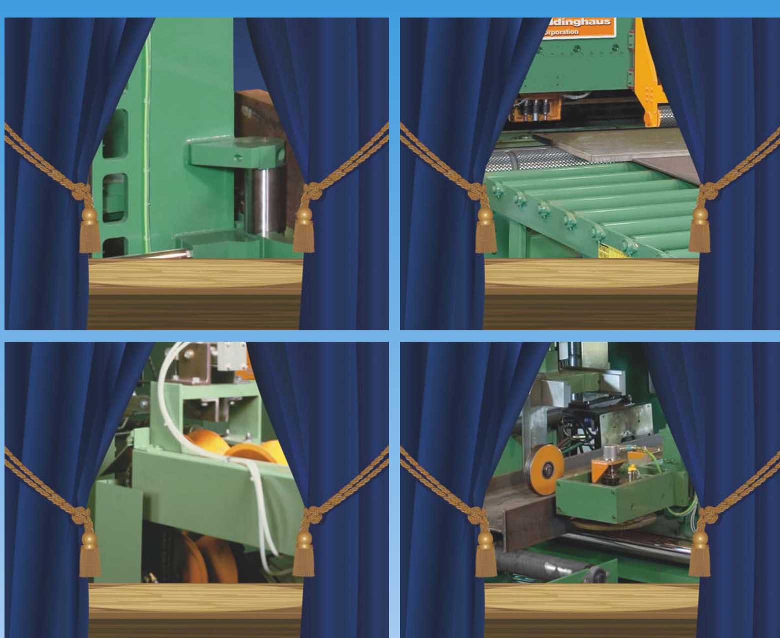

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In general, these products have been introduced within the past six months. If you're looking for a specific product, visit MSC's online product directory at **www.modernsteel.com/products**. You can browse by product category or search on any term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

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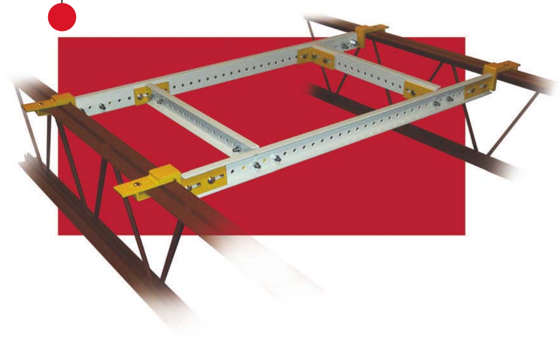

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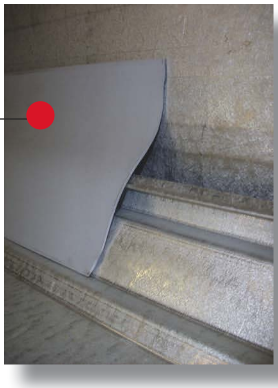
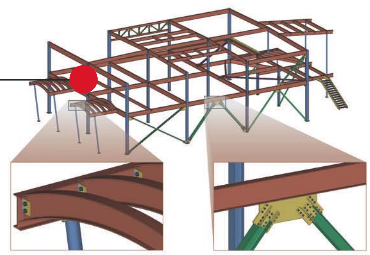


All products submitted are considered for publication, and we encourage submittals related to all segments of the steel industry: engineering, detailing, fabrication, and erection. Submit product information via e-mail to Geoff Weisenberger (weisenberger@modernsteel.com). To be included in MSC's online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

## Streamlining the Detailing Process

Design Data's SDS/2 steel detailing software features built-in intelligence to automatically design connections using a 3D model with a multitude of options for beams, columns, bracing, and joists. With the release of SDS/2 Version 7.1, SDS/2 users will notice some exciting new features that will help streamline processes and increase functionality. The software now allows users to add curved members with connection design and add clevises and turnbuckles into the 3D model. Members can now be grouped and detailed together with the bill of material, and it is also easier to copy and paste member information, such as connections and specifications, from member to member. This new version also features additional reports, including Status Report by Proximity, which will help the user to determine which members may be affected when a member is revised. In addition, SDS/2 will now design welded connections for vertical braces and will support sloping columns.

**For more information, visit www.sds2.com or call 800.443.0782.**



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*continued from page 70*

avoid someone who is standing over you. Somehow, in an exclusively engineering world, I felt sheltered from the mundane and miscellaneous world of architectural detailing. In an A/E firm, the lines between the architect's and engineer's responsibility become skewed and fuzzy. I have taken on more design responsibility in my work with A/E firms. I have been asked to look at curtain wall details, interior partition wall details, and ceiling details. I have even detailed and sized framing for bathroom vanities. This partially comes from an industry-wide misunderstanding that all elements having any sort of gravity load are a structural responsibility. We all know that everything thousands of feet above the earth's surface has gravity loads, but I am not designing everything thousands of feet above the earth's surface. (I can't help thinking that if I weren't sitting within just a few feet the architect, somehow they would find a way to frame the architectural details.)

Additionally, structural engineers in an A/E firm take on more responsibility because, unlike engineering companies, there are no contracts outlining the project role and responsibility of the structural engineer. Engineering companies can ask for more money to design elements that were not initially called out in their contract. For example, if you're dealing with non-structural elements–as in the case of the bathroom vanity—the architects would figure out how to address the issues without structural input, or they would renegotiate the contract. Either way, this process keeps elements that are not part of the building frame design—and are traditionally allocated to architects—from landing in the structural engineer's lap.

Having said all this, I must point out that it is perhaps a generalization based on my personal experiences. I recognize that variances exist among individual engineering and A/E firms. In the end, the management and the culture of the firm you are working with will strongly dictate your experience. A willing and supportive management goes a long ways and is definitely not a disadvantage. A culture that appreciates and recognizes what you have to offer as a structural engineer is always a benefit. MSC

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# NOTES FROM AN E IN AN A/E WORLD

BY ASMA MOMIN

## Being an engineer at an A/E firm involves lots of challenges—but also plenty of opportunities that might not be available at an engineering firm.

**WHEN MANY OF MY ENGINEERING FRIENDS** ask me what I like about working for an A/E firm, the first thing that comes to my mind is that I prefer to work with architects rather than engineers. I am only half joking. What I am getting at is this: I prefer having the architect as a *colleague* rather than a *client*. I have a lot more influence on projects with regards to schedules, among other things.

I remember my first job at a structural engineering firm. I was working on multiple projects with multiple architectural firms, and they all had short deadlines. Of course, everyone wanted everything yesterday, and when the architect is your client you do not have the luxury of telling them their project is not on the top of your list but you'll get to it eventually. You cannot negotiate schedules by stating the obvious: that you are working on another project, for another architect, at another firm. I remember the principal at my engineering firm telling me that you can't say "no" to the architect's schedule, nor can you even allude to working on any other projects.

What do you do, then, when the architects have impossible schedules, and you have to meet impossible deadlines? There's not much you can do. You are working on a tight deadline and waiting for the architect to send you their background. And as soon as they send it, they change it. Honestly, when it comes to sharing information and files, I do not even remember how I managed to coordinate my projects at an engineering firm. I must confess that, in addition to coffee, the ability to readily access the updated architectural/mechanical files is a necessity for me.

*Asma Momin is a structural engineer with A/E firm Page Southerland Page, LLP in Dallas. She can be reached at amomin@pspaec.com.*

Along the same lines, I like the opportunity to be able to walk across the office to discuss plans, sections, and details in person. I get more answers that way. It is hard to avoid a person standing over you. I don't have to drive across town for coordination meetings, I don't have to coordinate over the phone and guess what they are looking at, and I don't have to play phone tag. I save a lot of time not running after the information. This goes not only for architectural coordination but also for mechanical and other disciplines, which brings me to my next point.

Someone told me that diversity brings an opportunity to grow and learn. This is definitely the case at an A/E firm. Through conversations and discussions with other disciplines, I am more exposed to a global building perspective that is very different than the microscopic world of structural engineering. At an engineering company, it is easy to get sucked into the idea that a building begins and ends with structural engineering. The projects get into the books when the structure is ready to be designed and off the books when the structure is built. Everyone is talking, eating, and walking *structure* all the time. But in an A/E firm, you are constantly engaged in solving interdisciplinary problems, and therefore you become more aware of how your structure affects other disciplines. In A/E firms, I have been involved with the construction process at more levels than at straight-up engineering firms. It has been an enlightening experience that helps you become a better engineer.

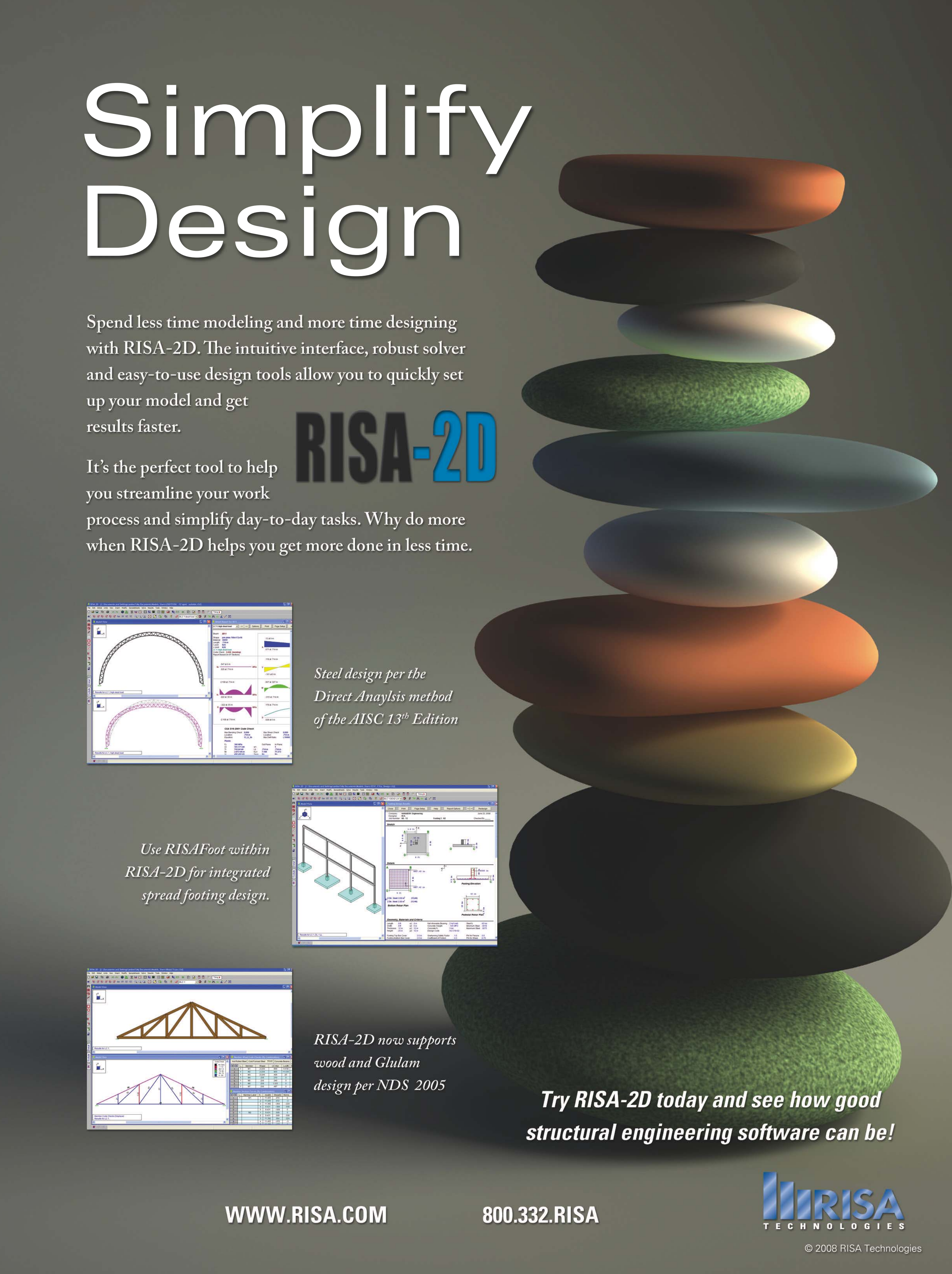
At this point, you must be wondering what, if anything, I miss about working for an engineering firm. At times, I do miss certain things. Many of the A/E firms I have come across do not have large structural departments, and structural engineering is not their core business. Many people would argue that this correlates to job security. I must confess that I have heard of a few A/E firms laying off engineering departments during rough periods. However, I am not too concerned with that. It is in the best interest of an architectural firm to have an in-house engineering department. Not only is it a selling point to the clients, but it also keeps profits that would otherwise go to another company, in-house.

The abundance of structural resources available at an engineering company far exceed that of an A/E firm in terms of people, books, seminars, and technology. The structural departments of A/E firms I have worked with housed, at most, six engineers. I miss the abundance of structural minds and the plurality of structural opinions that you find in a structural engineering firm. There was always someone there with whom I could discuss project-related issues and concerns. Partially because it is the core business, there is a bigger appreciation for engineers and their contributions at an engineering firm.

There is also something to be said about being too available to the architects. As I mentioned, it is hard to

*continued on page 65*

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