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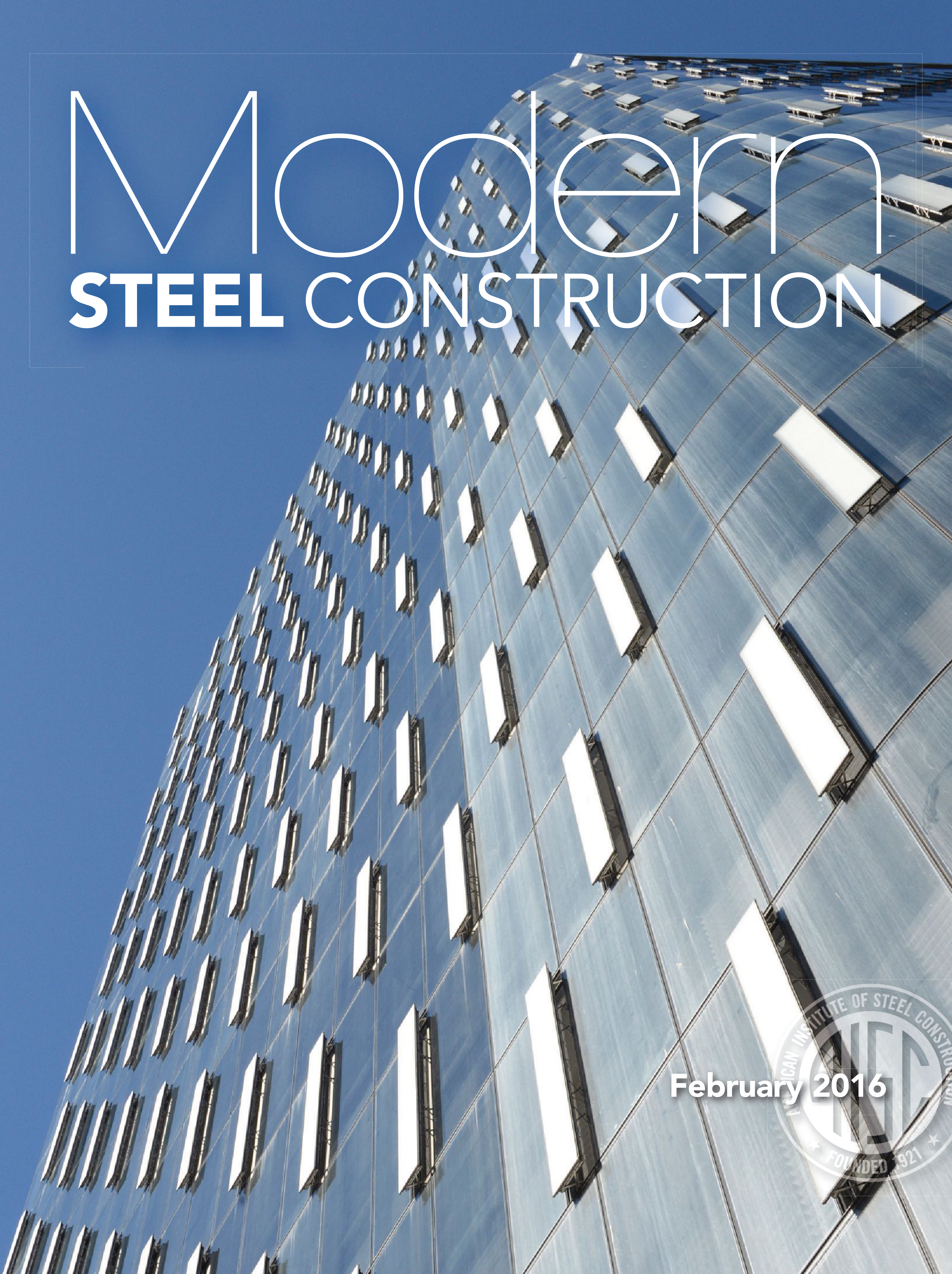

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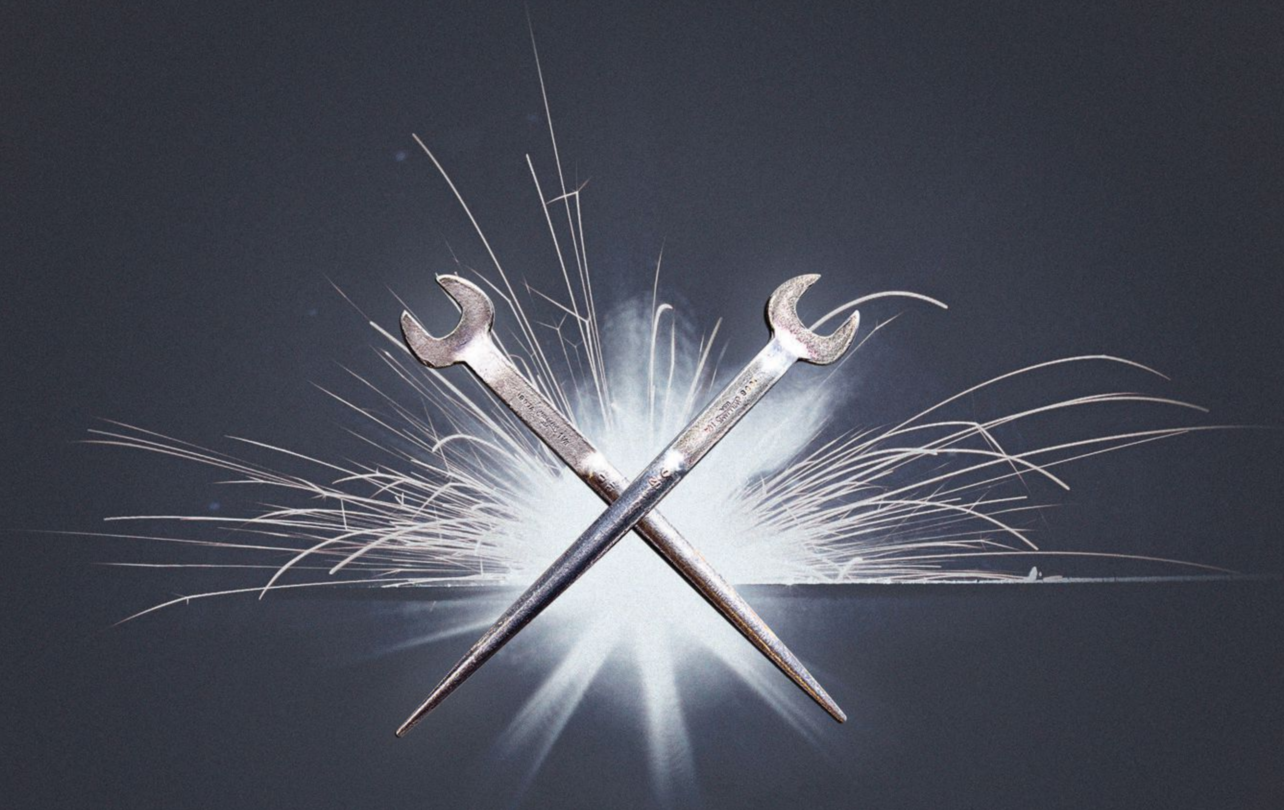

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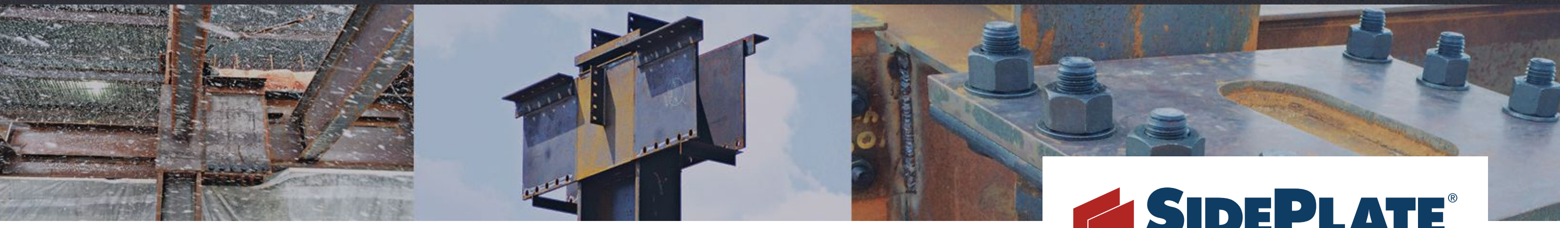

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ON THE COVER: PNC Tower in Pittsburgh has lofty goals, aiming to become the world's greenest office building, Page 26. (Photo: The PNC Financial Services Group, Inc.)

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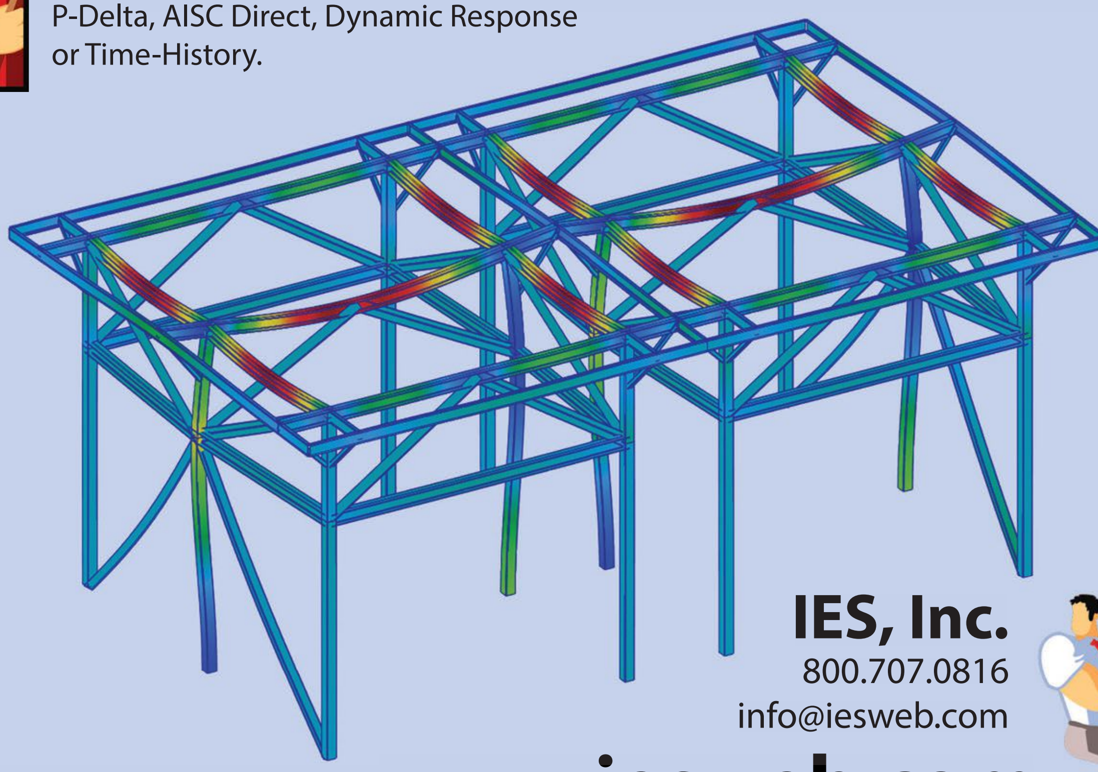

editor's note

WHEN I WAS BORN, MY DAD WORKED FOR A SMALL MISCELLANEOUS STEEL FABRICATOR IN THE WILLIAMSBURG SECTION OF BROOKLYN. Not too long after, he and my mom started their own firm, manufacturing and installing steel door frames (along with the doors and hardware).

It was a fascinating business for a child to see, not in the least because the warehouse full of frames provided a fertile playground for games of tag or hide-and-go-seek. But beyond a ready source of materials with which to build elaborate clubhouses, my parents' work provided me with some interesting business lessons.

First and foremost, my dad's success was dependent on his ability to provide something his competitors didn't. I don't think his frames were better than anyone else's, and his doors and hardware were purchased from the same suppliers his competitors used. What made his business special was my dad. He didn't just bid on projects. Instead, he got in before his competitors and provided a free specification service. He would take the blueprints and create the hardware schedule from which the project would be bid—by both his company and his competitors. And since he knew the requirements better than anyone else, he was often able to provide the most competitive bid.

My dad also knew the success of his business was based on relationships. It wasn't unusual to see Jack (one of Long Island's largest office park developers) at the house. Or to visit my dad's best friend, Mickey, who also was a vice president at the firm from whom my dad bought most of his door knobs, locks and hinges (even to this day, Mickey's youngest son is my best friend).

Many of these types of relationships are formed and fostered at industry meetings. Sometimes they're huge annual events such as NASCC: The Steel Conference (**www.aisc.org/nascc**); sometimes they're smaller specialty events such as the upcoming AISC Town Hall Meeting (**www.aisc.org/townhallmeeting**). And there are plenty of other opportunities to develop new relationships and reinforce existing ones as well: Join your local SEA. Participate in your regional fabricator association. Attend conferences and conventions.

But when you participate in these groups, don't just think of them as furthering your professional ambitions. The connections you make aren't just for work; they're also personal connections that help enrich your life. When you see me at the Steel Conference, don't just think about *Modern Steel Construction*; also think about my son's piano recital and your daughter's hockey game. These are the conversations that will be remembered and the connections that will make a difference in your life.

Scott Melnick

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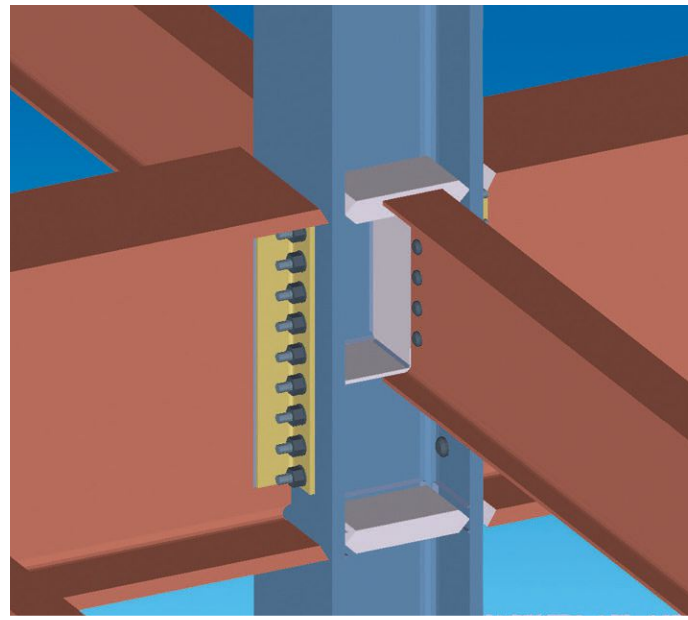

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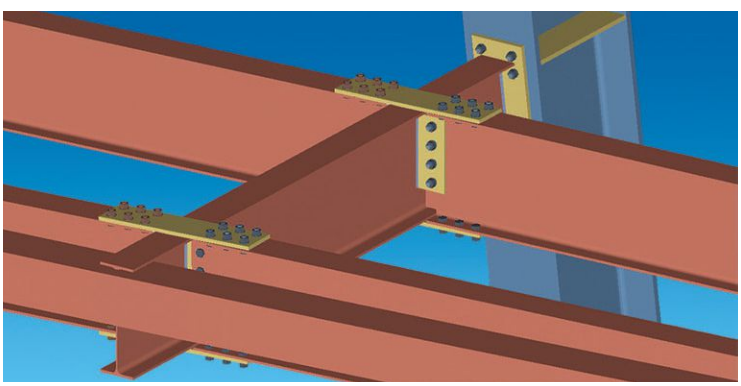

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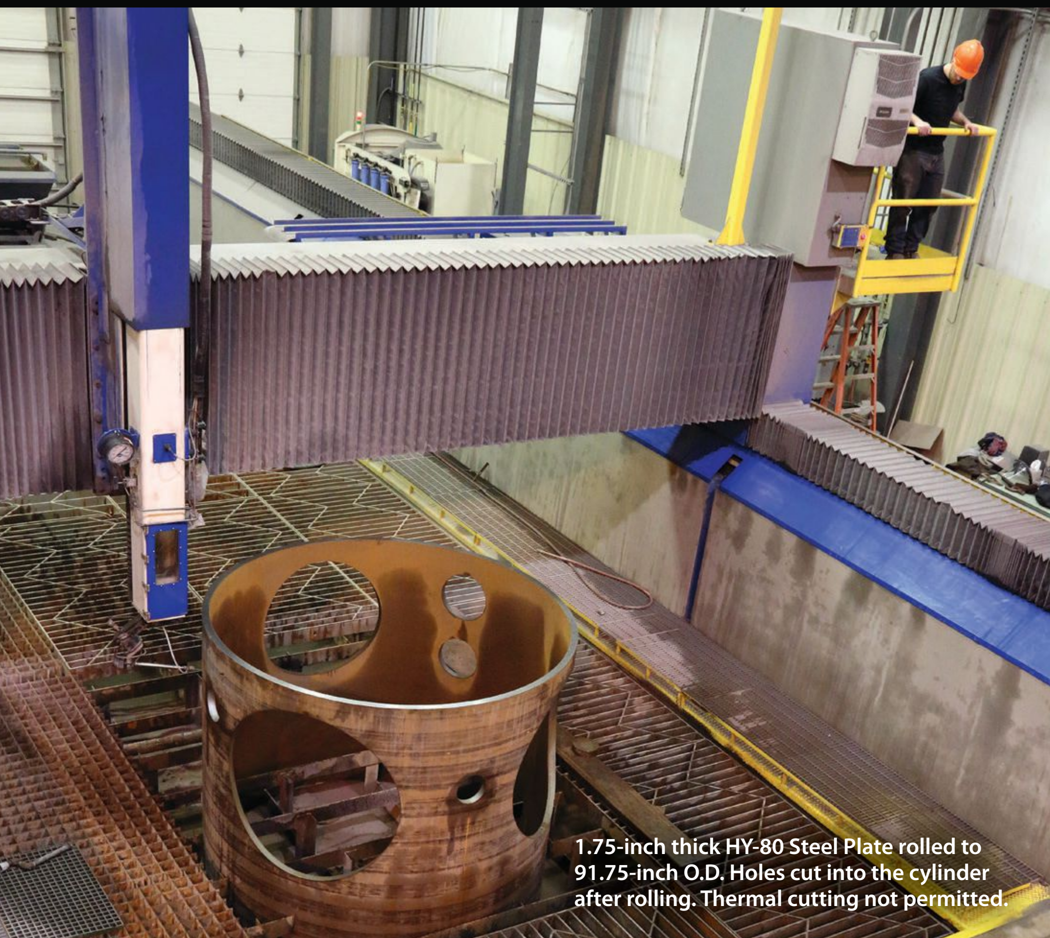

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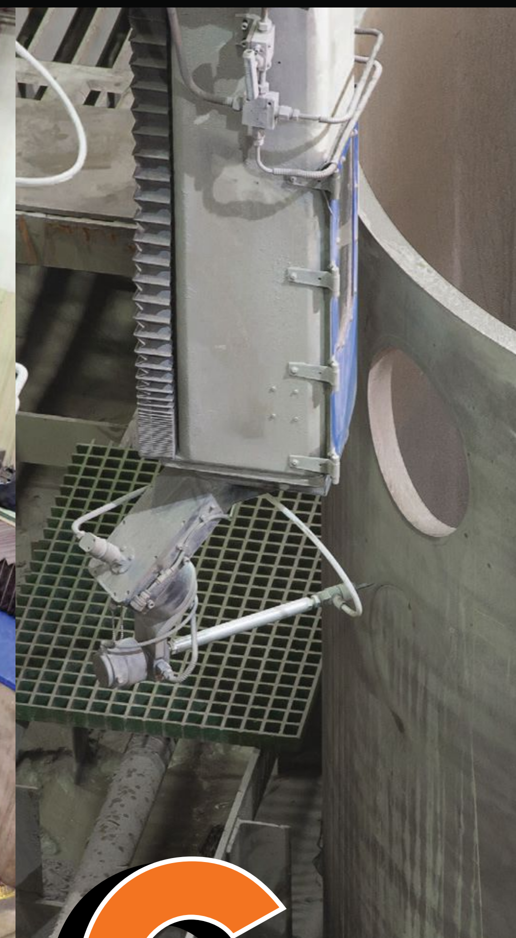

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If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

Eccentricity on Columns

Are there any formal recommendations concerning the inclusion of eccentric moment in a steel column due to the physical distance between the beam end and column centerline?

The decision to account for the eccentricity or neglect it is one that you have to make based on your engineering judgment. Ioannides ("Minimum Eccentricity for Simple Columns," *ASCE Structures Congress Proceedings*, Volume 1, 1995) suggests that even for a column that is loaded on one side only, the restraint a connection provides to the column will help mitigate the eccentric effects in normal framing configurations.

There are some common parallels in design where we neglect eccentricity. The AISC *Steel Construction Manual* states that for standard or short-slotted holes, eccentricity on the beam side of double angle connections may be neglected for gages (distance from the face of the support to the centerline of a single vertical bolt row, shown as dimension a in Figure 10-4[a] in the *Manual*) not exceeding 3 in. While you are permitted to neglect this eccentricity, there will still be a resulting moment that will exist somewhere in the system. Some of the moment will go to the column and some to the beam, based on the stiffness of the elements. The reasons we neglect the eccentricity are largely historical: The basics of bolted joint design evolved before analysis capabilities had progressed sufficiently to account for it. However, there are some technical justifications. First, any assumption about where the moment will exist will be wrong, since the system will distribute the moment throughout the system based on stiffness. The usual eccentricity is relatively small, and its effects become arguably negligible when distributed within the system—even if the effects might be significant when assumed to be concentrated at an individual element. There also are other influences like the fact that the bolt strengths provided in the AISC *Specification* (available for free at **www.aisc.org/2010spec**) have been reduced to account for uneven loadings that occur in primarily end-loaded connections. These reductions also help to account for some eccentricity without explicit consideration of the moment by the designer.

A final thought: Check the settings in your software to see if your columns are already being designed for eccentricity automatically. Some software programs account for an assumed amount of connection eccentricity as a default when sizing the columns.

Carlo Lini

Minimum Weld Sizes

We have a project where a 1-in.-thick angle is welded to a 1-in.-thick plate. A ¼-in. weld has sufficient strength, but a $\frac{5}{16}$-in. fillet weld was specified to meet AISC *Specification* Table J2.4's minimum requirements. A ¼-in. weld was completed in the field. Is it possible to come back and augment the existing weld, or does the weld need to be removed?

Adding additional weld would not address the issue, which is related to having a high enough heat input to prevent cracking. However, you may not need to repair the weld either. Duane Miller addressed this issue at the 2013 NASCC: The Steel Conference in his presentation "Welding Questions Answered" (view session N78b at at **www.aisc.org/2013nascconline**). Fast-forward to the 19:00 minute mark, and you'll see a progression of three options: one based upon low-hydrogen process solutions, one based upon evaluation of heat input, carbon equivalent and cooling rate and one based upon removal and replacement. I believe the information from this presentation should help you address this issue.

Carlo Lini

Slenderness Limits on Columns

Does Equation E3-3 in the AISC *Specification* apply even when *KL/r* is greater than 200? Do the provisions of Section E5 for single angles apply when *KL/r* is greater than 200?

According to Equation E3-3, the nominal critical stress is $F_{cr} = 0.877F_e$. F_e is the theoretically derived elastic buckling stress according to Equation E3-4. It was originally derived by Euler in a slightly different form. The coefficient 0.877 is an empirical reduction factor that is based on a statistical analysis of the geometric imperfections. The User Note in Section E2 recommends that "the effective slenderness ratio *KL/r* preferably should not exceed 200." This is a recommendation, not a requirement. Equation E3-3 is valid for $KL/r > 200$. The Commentary discusses this further.

Because the equations in Sections E5(a) and E5(b) were developed empirically, the stated limits of $KL/r \le 200$ must be met. If the effective *KL/r* is greater than 200, concentrically loaded angles can be designed according to Section E3, E4, or E7, as appropriate. If the angle is loaded eccentrically by connecting one leg to a gusset plate, the eccentricity can be addressed using the equations in Chapter H. AISC *Manual* Table 4-12 was developed using Section E3, E4, E7 and Chapters F and H.

Bo Dowswell, P.E., Ph.D.

Minimum Loads for Splices

We designed a beam to support a floor. The contractor has asked to put a bolted splice in the beam to simplify erection. We provided a shear and moment from the design loads at the splice. Since the loads at this location are small, the splice they have designed seems too light. Is there a minimum splice required, such as to design the moment splice to 75% beam capacity, regardless of the actual loads?

No, there are no minimum criteria in AISC standards for beam splices. Generally, the AISC *Specification* provides requirements relative to design and detailing based on the forces determined by the engineer. As such, the *Specification* typically does not provide minimums. Section J6 simply states that "splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice."

That said, engineering judgment must be exercised. The splice must provide both sufficient strength and stiffness.

Susan Burmeister, P.E.

Delegated Connection Design

When performing connection design that has been delegated by the engineer of record (EOR), we sometimes receive contract documents that do not appear to comply with the building code. We seem to have two choices: We could strictly follow the requirements as shown in the contract documents, or we could redesign the connections to meet the building code. Doing the latter may be stepping on the EOR's toes and would be detrimental to our client, the fabricator, as the details would likely be more expensive than those shown in the contract documents. Can you provide any advice?

There are really two issues: your responsibility as a licensed engineer and your responsibility to your client as it relates to your and their contractual obligations.

As a connection design engineer, you must satisfy the intent of the engineer of record as it is conveyed in the contract documents. The EOR ultimately has responsibility over the project. Compliance and interpretation of the building code—including the building code compliance of the connection design criteria specified in the contract documents—is within the EOR's scope. Unless I have strong reasons to believe the EOR is doing something unsafe, I ultimately would leave these decisions to them.

This is not to say that I would remain silent. If I saw something that appears to be unsafe or conflicts with my understanding of the design intent, I would question it. This is consistent with Section 3.3 of the AISC *Code of Standard Practice*, which requires that the fabricator promptly notify the owner's designated representative for construction (usually the general contractor) of discrepancies. Note the fabricator and delegated connection engineer need not review the documents for discrepancies, but must notify the owner's representative of discrepancies that have been recognized. As described in Section 3.3, it is the owner's designated representative for design (the EOR) who resolves the discrepancy.

The *Code* also provides explicit requirements related to delegated connection design for option 3 of Section 3.1.2. It should be noted that the connection design engineer is required to provide substantiating connection information, which the EOR then reviews for conformance with the contract documents. These requirements define a process intended to ensure both engineers are on the same page. This reflects the relationship described above, where the connection design engineer strives to satisfy the EOR's intent. The contract documents communicate the EOR's intent to the contractors. If the EOR's intent changes or the original contract documents prove insufficient to properly convey the intent, then the contract documents must be revised. The *Code* addresses this situation as well.

Section 9.3 addresses revisions to the contract documents and indicates that contract price and schedule shall be adjusted in accordance with Sections 9.4 and 9.5 when the contract is revised.

As the connection design engineer, you cannot unilaterally change the contract by introducing requirements. If you feel discrepancies exist, you must notify the owner's designated representatives. If your arguments are persuasive, then the owner's designated representatives will revise the contract to address your concerns. This revision will prompt the parties, including your client, the fabricator, to assess the impact of the change. The fabricator should not absorb the costs associated with requirements that were not clearly shown in the contract bid package.

Larry S. Muir, 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**.

Larry Muir is director of technical assistance and Carlo Lini is a staff engineer—technical assistance, both with AISC. Bo Dowswell and Susan Burmeister are consultants to AISC.

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

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

This month's Steel Quiz takes a look at the design of composite members as covered in Chapter I of the AISC *Specification*.

1 True or False: Designers are permitted in the AISC *Specification* to use allowable strength design (ASD) when designing a composite member.

2 For the axially loaded encased composite member shown in Figure 1, the area of the steel core, A_s, must comprise at least _____ of the total composite cross section.
a. 1% **b.** 3.5% **c.** 10% **d.** 12.5%

3 For the axially loaded encased composite members shown in Figure 1, the minimum reinforcement ratio for continuous longitudinal reinforcing, A_{sr}/A_g must be _____
a. 0.001
b. 0.004
c. 0.010
d. 0.018

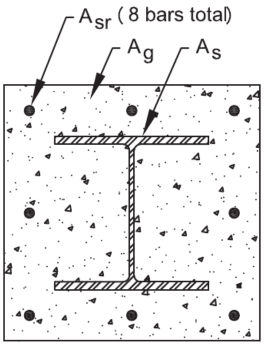

Figure 1

4 True or False: The required minimum longitudinal reinforcing ratio for encased composite members, as highlighted in Question 3, comes from ACI 318.

5 Per the AISC *Specification*, the nominal strength of composite sections can be determined via a **plastic stress distribution method** or the **strain compatibility method**. Figure 2 is representative of the _____ method and Figure 3 is representative of the _____ method.

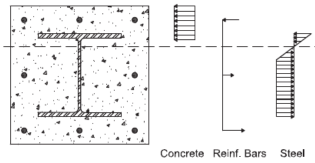

Figure 2

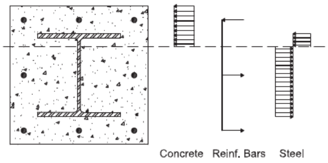

Figure 3

6 True or False: Local buckling effects need not be considered for encased composite members.

TURN TO PAGE 14 FOR ANSWERS

1 **True.** The design basis for ACI 318 is strength design. Designers using ASD for steel design must be conscious of the different load factors between the two specifications.

2 **a.** 1%. In the *AISC Specification*, the use of composite compression members is applicable to a minimum steel ratio (area of steel shape divided by the gross area of the member) equal to or greater than 1%.

3 **b.** 0.004. A minimum amount of longitudinal reinforcing steel is prescribed to ensure that unreinforced concrete encasements are not designed with the provisions in the AISC *Specification*.

4 **False.** The limitation of $0.01A_g$ in ACI 318 for the minimum longitudinal reinforcing ratio of reinforced concrete compression members is based upon the phenomena of stress transfer under service load levels from the concrete to the reinforcement due to creep and shrinkage. It is also intended for resisting incidental loading not captured in the analysis. The inclusion of an encased structural steel section meeting the requirements of Section I2.1a aids in mitigating this effect and consequently allows a reduction in minimum longitudinal reinforcing requirements.

5 Figure 2 is representative of the strain compatibility method, and Figure 3 is representative of the plastic stress distribution method. Both methods are permitted, but the strain compatibility method should be used to determine nominal strength for irregular sections.

6 **True.** In contrast, local buckling effects do need to be considered for filled composite members as defined in the AISC *Specification*.

Steel Solutions Center

Everyone 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**.

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steelwise

PROPER APPLICATION OF STEEL BEAM LOAD RESTRICTION FACTORS TO UL DESIGNS

UL, AISC and AISI update the applicability of load restrictions for UL Designs for steel beams.

BY CHARLES J. CARTER, S.E., P.E., PH.D.,
FARID ALFAWAKHIRI, P.ENG., PH.D.,
ROBERT M. BERHINIG, P.E., AND LUKE C. WOODS

AS AN UPDATE to the "UL Design Considerations" article (October 2015, available at **www.modernsteel.com**), following is the latest from Underwriters Laboratories. This and subsequent information will continue to be available at **www.aisc.org/ULclarity**.

Underwriters Laboratories (UL), the American Iron and Steel Institute (AISI) and the American Institute of Steel Construction (AISC) have been collaborating to provide answers and solutions to questions that have been raised about the need for load restriction factors with UL Designs. We have identified a number of clarifications and updates that will be made in UL Guide BXUV, as well as in UL Designs themselves. We jointly offer the following summary so that the information is known and can be used now, while UL updates their documents.

Fire-Rated Design Bulletin

The information in this article was extracted from a UL bulletin to its members, jointly drafted by UL, the American Iron and Steel Institute (AISI) and AISC, and is meant to inform the industry of updates to UL fire-rated designs that specify a "Restricted Load Condition." It is being presented as the result of a series of tests sponsored by AISI to investigate this subject matter. Over the coming months, the UL directory will be updated to reflect these new findings both in the general information of BXUV as well as in the specific fire rated designs. Proposed updates are also included.

Recent testing conducted by UL for AISI and AISC provides for the following conclusions related to application of load restriction factors to UL Designs for steel beams in US practice:

1. Load restriction factors for steel beams need not be applied to any UL Design that is based upon strength calculated using the 2005 or 2010 AISC *Specification*. Table 1 below shows the UL (and ULC) Designs that meet this condition.

Table 1. Unrestricted UL and ULC Designs

	For W-Shape Beams	For Specialty Beam Products
UL Designs	G592, D798, D799, D982, D985, D988, E701, E702, N743, N852, N860, S750, S751, and S812	N858, N904, N905, and N906
ULC Designs	D501, F906, F912, and N815	O710, N900, N901, and N902

View these and other UL Designs at **www.ul.com/firewizard**.




Charles J. Carter is a vice president and chief structural engineer with AISC, **Farid Alfawakhiri** is a senior engineer, Construction Codes and Standards, with the American Iron and Steel Institute, **Robert M. Berhinig** is a consultant with Berhinig Services, LLC, and **Luke C. Woods** is a principal engineer - Fire Resistance and Containment, Building and Life Safety Technologies, with Underwriters Laboratories, LLC.

2. Load restriction factors for steel beams need not be applied to any other UL Design if an unrestrained beam rating is used. Unrestrained beam ratings are determined using a limiting temperature criterion of 1,100 °F and a load maintenance criterion. The testing of steel beams at varying load levels has shown that the time it takes to reach this limiting temperature is not a function of the magnitude of the applied load.

3. Load restriction factors for steel beams need not be applied to any other UL Design if a 1-hour restrained beam rating is used. A 1-hour restrained beam rating is based upon the same criteria used for a 1-hour unrestrained beam rating. Therefore, as stated in item 2 above, the rating is not a function of the magnitude of the applied load.

4. When using a UL Design for which none of the foregoing conditions applies, a load restriction factor of 0.9 is applicable for both composite design and non-composite design in U.S. practice. UL, AISI and AISC have determined that the load restriction factors specified for use with Canadian design codes are not appropriate for use in the US. In the US marketplace, a smaller load reduction of 10% is appropriate for UL Designs based upon 1989 or earlier AISC ASD *Specification* requirements.

Stated more directly, load restriction is only applicable to 1.5-, 2-, 3- and 4-hour restrained beam ratings in UL Designs that were loaded based upon 1989 or earlier AISC ASD *Specification* requirements. In these cases, a 10% load reduction (0.9 load restriction factor) shall be used.

Moving forward, UL, AISI and AISC understand the need for practical and useful solutions to make fire protection selection and design easier for all. Accordingly, we are now collaborating to develop an approach wherein the fire protection thickness can be adjusted to account for conditions that differ from those used in the testing for a given design. We expect that this approach will be preferable in the marketplace and intend that it will replace the load restriction approach when available.

Loading of Test Specimens

Following are proposed updates to ANSI/UL 263–*Standard for Fire Tests of Building Construction and Materials*:

ANSI/UL 263 requires the load applied to test samples to be based upon the limiting conditions of design as determined by nationally recognized structural design criteria. For some applications, the nationally recognized design criteria may be based upon the Allowable Stress Design (ASD) Method or the Load and Resistance Factor Design (LRFD) Method. For applications where these two design methods are available, the load applied to the test sample was determined in accordance with the Allowable Stress Design Method unless the rated

assembly specifically references the Load and Resistance Factor Design. Also, unless otherwise stated, the load capacity of steel beams assumes the beams are fabricated from A36 steel.

ANSI/UL 263 permits samples to be tested with the applied load being less than the maximum allowable load as determined by the limiting conditions of nationally recognized structural design criteria. The ratings for assemblies determined from tests where the applied load was less than allowed by the nationally recognized structural design criteria are identified as "Restricted Load Condition." The percent of the maximum load, the percent of the maximum stress and the nationally recognized design criteria is identified in the text describing the structural element of rated assemblies with a restricted load condition. An example of the text used in an assembly with a restricted load condition and steel joist loaded to 80% of the maximum allowable is:

The design load for the structural member described in this design should not: (1) exceed 80% of the maximum allowable load specified in "Catalog of Standard Specifications and Load Tables for Steel Joists and Steel Girders," published by the Steel Joist Institute, or (2) develop a tensile stress greater than 24 ksi, which is 80% of the maximum allowable tensile stress of 30 ksi. (Note: The maximum allowable total load develops a tensile stress of approximately 30 ksi.)

Some restricted load conditions have resulted from changes in product availability. An example is the substitution of K-Series joists for other series joists as described under Section III, FLOOR-CEILINGS AND ROOF-CEILINGS, Item 7, Steel Joists.

Assemblies tested with less than the maximum allowable load that would result from loading calculated using the Limit States Design Method in Canada or post-2005 AISC *Specification* criteria in the United States are identified as "Restricted Load Condition." The Percent Load Reduction and corresponding Load Restricted Factor for typical assemblies noted in Table 2 are based upon loading calculated in accordance with pre-2005 AISC ASD *Specification* criteria as compared to loading calculated in accordance with 2005 and later AISC *Specification* criteria in the United States.

The calculations were performed for assemblies representing spans and member sizes of typical fire-test assemblies. The loads were calculated assuming a span of 13 ft for floors and roofs and 10 ft for walls. Calculations for wide flanged steel beams assume a live to dead load ratio of 3:1.

A load restriction need not be applied for an unrestrained condition of any hourly rating nor applied for a restrained condition with a hourly rating of one hour or less.

Some fire-resistive designs are specified with a Restricted Load Condition. When using fire-resistive designs with a Restricted Load Condition, the factored resistance of the structural members or components should be reduced by multiplying the factored resistance by the Load Restricted Factor specified in the individual fire-resistive designs.

Table 2

Type of Assembly	Percent Load Reduction (LRFD-ASD) / LRFD	Load Restricted Factor
W8×28 – AISC (W200×42 – CISC) noncomposite steel beam	10%	0.9
W8×28 – AISC (W200×42 – CISC) composite steel beam	10%	0.9
Floor/Roof supported by open-web steel joists	4%	0.96
Floor supported by cold-formed steel channels	0%	none
Floor supported by 2 × 10 in. (38 × 235 mm) wood joists	35%	0.65
Wall supported by 2 × 4 in. (38 × 89 mm) wood studs	18%	0.82
Wall supported by cold-formed steel studs	0%	none
Steel columns	*	*

The ratings for floors supported by cold-formed steel channels and walls supported by coldformed steel studs do not have a Load Restriction Factor as the associated loads in Canada and the U.S. are based on the same standard: CSA S136, "North American Specification for the Design of Cold-Formed Steel Structural Members," and "North American Specification and Commentary for the Design of Cold-Formed Steel Structural Members."

*Unless otherwise specified in the individual designs, columns do not have a Load Restriction Factor, as those ratings are based on temperature limitations in accordance with ANSI/UL 263.

The Load Restricted Factor should be applied to the factored resistance of all structural members or components, including, but not limited to, factored moment resistance (M_r), factored shear resistance (V_r), factored tensile resistance (T_r) and factored compressive resistance (C_r).

The engineer of record should be consulted whenever fire-resistive assemblies with Load Restricted Factors are selected. The indicated load reductions are based upon factored load effects that are governed by the reduced factored resistance of the structural elements. The selection of structural elements is, at times, based upon service limits, such as deflection and vibration. These factors and others, such as the change in material strength properties as a function of temperature, should be considered when selecting fire-resistive assemblies with Load Restricted ratings.

Unless stated in a design, it is recommended the Load Restricted Factors in Table 2 be used.

Assemblies developed from tests where the load applied on the sample was based upon calculations in accordance with the Load and Resistance Factor Design are identified in the individual certifications. These assemblies shall not be considered "Load Restricted." ■


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How much is ineffective communication costing your company?

COMMUNICATION SMOG

BY MARK A. VICKERS

WHAT WOULD YOU DO if a pollutant in your office was killing $5,000 worth of profit this year?

What would you do if every single employee spewed forth that much pollution each year?

This pollutant, called "Ineffective Communication," can affect every employee and cause a proverbial smog in your organization that prevents clear and concise communication while killing profits.

While most executives believe that communication is important, *very* few have ever tried to quantify their losses. Communication smog causes an average loss of 40 minutes of productive time for every employee, every day of every year—and that is just Category 1 Smog. As the pollutant builds and thickens, it can cause your organization significant damage.

As ineffective communication pollution continues to be added to your environment, the smog thickens, covering more of your organization.

Category 1

If your company is like most, you operate in a continual state of Category 1 communication smog, and the typical employee will lose 167 hours of productive time per year.

At the 2015 U.S. average for salary, benefits and tax levels, that equates to $5,200 per employee per year. Most people don't realize the amount of time lost daily due to issues like:

- Seeking clarification
- Asking a question multiple times
- Resolving customer or employee conflicts
- Never-ending email threads
- Crisis management due to missed deadlines
- Rework

The reason companies often don't recognize the importance of these issues is that with Category 1 smog, the impact of each issue is too small to be noticed as a financial impact. While this pollutant could be considered insignificant at the individual issue level, over the course of a year, a company with 20 employees is likely to lose over $100,000 of productive time. At this level all it takes is a good breeze of education and coaching to clear the air, regain that productive time and prevent further pollution.

Category 2

As ineffective communication pollution continues to be added to your environment, the smog thickens, covering more of your organization. The impact increases and now becomes visible on your financial statements.

In addition to all of the Category 1 impacts, you start experiencing:

- Lost sales and customers
- Increased marketing, customer acquisition and customer service costs
- Increased staff turnover, hiring and training costs
- Decreased operational continuity

In Category 2 Communication Smog, the following symptoms appear regularly:

- Sarcastic and negative comments emitted towards customers, employees and management
- Employees resistant to raising issues in any forum
- All levels of staff operate in a CYA (cover your assets) mode
- Lack of faith in the team and the organization

When you have Category 1 Smog, a good breeze of education and coaching will clear the air—but when you reach Category 2, you are going to need gale force winds. At this level of dysfunction, the cost, time, resources, and organization discomfort required to correct the issue is exponentially greater than Category 1.

Category 3

Left unattended, the smog will continue to build and become so thick that serious financial impacts will be felt.

Mark A. Vickers, a Certified Professional Coach and Certified World Class Speaking Coach, is a communications consultant focused on helping individuals and companies improve performance through improved communication and speaking skills. He is also the creator of the Communications Challenge, an objective way to measure communication effectiveness. For more information about Mark and his programs, please visit **http://speakingisselling.com**.

When communication problems are allowed to evolve to Category 3, the smog escalates issues and creates a high financial impact. You will observe:

- Prevalent sarcasm and negativity
- Inappropriate comments about customers and management
- Conflicting objectives within management
- A complete breakdown of trust and communication
- High employee turnover
- Legal costs skyrocketing due to escalating issues
- Large-scale customer defections
- Loss of reputation

Picture the air in a post-apocalyptic sci-fi movie; that grey, murky sky, with people scurrying around in the shadows. That is the environment in your company when you reach Category 3 smog. It will require hurricane force winds to clear the air. Success will be difficult without significant management changes, and a wholesale cultural change.

Don't think it could happen to you? Ineffective communication is a slippery slope that, left unattended, can grow silently until one day you are losing valued customers.

When surveying business owners and executives, concern is warranted as they report that:

- Communication skills are a critical part of their long term success
- Virtually none measure the impact of ineffective communication
- Very few have a comprehensive plan to develop this critical skill

How can something that has this big of an impact on business, get so little attention?

In order for any plan to be successful, all levels of management must agree that communication is important and that everyone has room to improve.

Clearing the AIR

Whether you are like most companies and facing a Category 1 communication smog or a larger threat to your business, there comes a time where your success will require you to get rid of the smog and clear the air, allowing clean and concise communication to work its magic.

To clear the AIR, simply remember: **A**cknowledgement, **I**dentification and **R**emediation.

Acknowledgement. The first and often hardest step in clearing the air is the acknowledgement of the problem, and that it impacts *everyone* in the organization. In order for any plan to be successful, all levels of management must agree that communication is important and that everyone has room to improve. Once there is true acknowledgement that communication needs to be addressed, you can move to the next step.

Identification. In order to maximize the results from any remediation plan, it is critical that the highest impact communication problems be addressed first. To properly assign priority, an assessment should be done that evaluates communication based on:

- Job title/position
- Duration of service with the company
- General categories of communication
- Common communication issues

Based on the assessment, you can now begin developing a remediation plan that will get you maximum gain quickly, and start clearing the air.

Remediation. Your remediation plan should focus on the processes and skills required to create an environment for clear and concise communications. By launching initial elements of the remediation plan on high target areas, you are able to quickly gain the momentum and create a ripple effect that will be required to flush the pollution and the smog from your environment. A common sequence of learning priorities is:

- Establishing a habit of having a clear intention when communicating
- Maintaining a focus on the needs of the other to ensure clear communication
- Developing and delivering messages with clarity
- Enhancing your message with improved presentation skills
- Learning how to formulate and ask powerful questions

Take an intentional approach to communication skills throughout your organization, and you will create a clean air environment where communication flows freely and effectively.

You have accounting professionals making sure your money and finances are taken care of and legal professionals keeping you out of trouble. Why not also have a professional on your team making sure you are not falling victim to communication smog? ■


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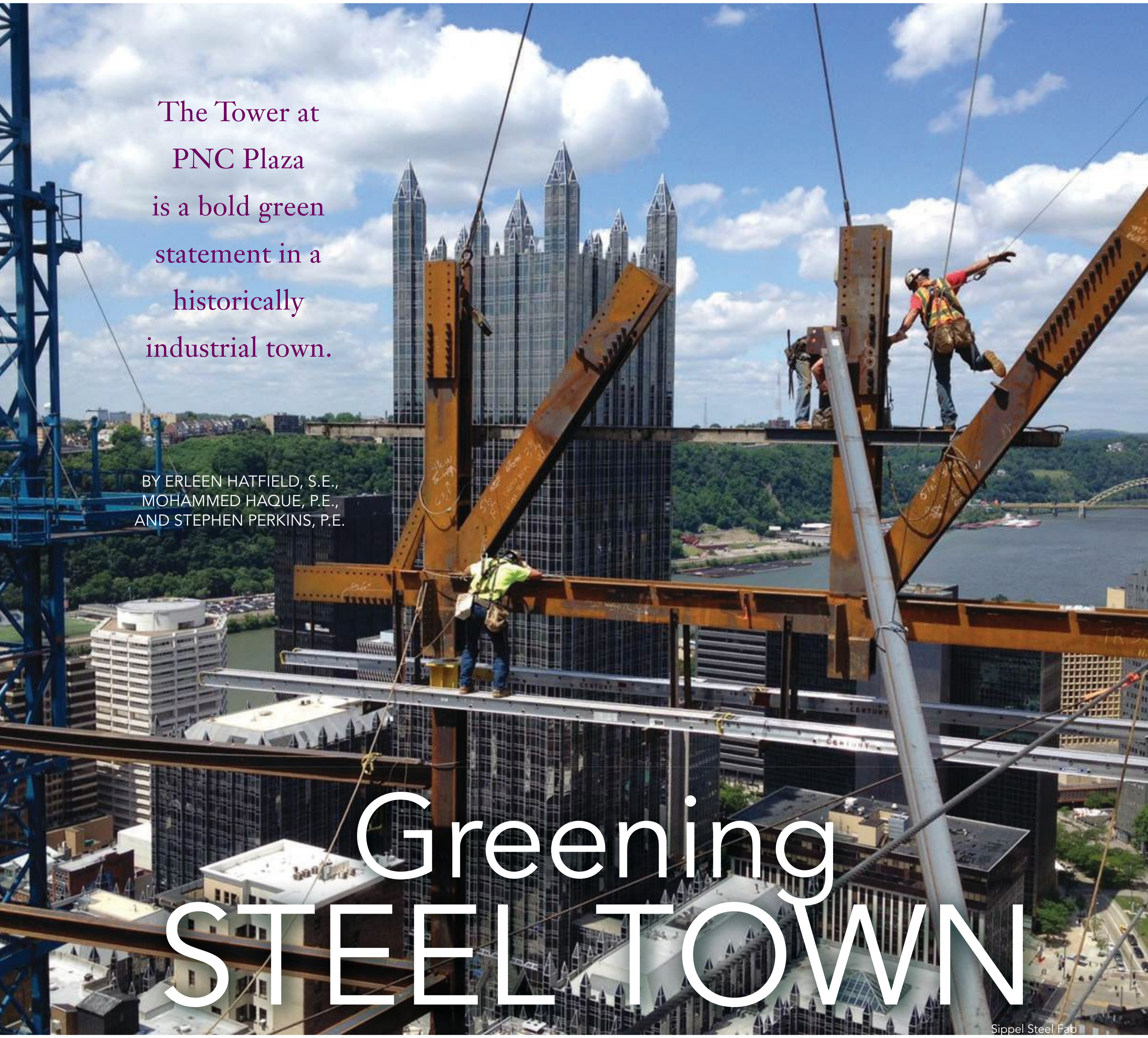


Sippel Steel Fab

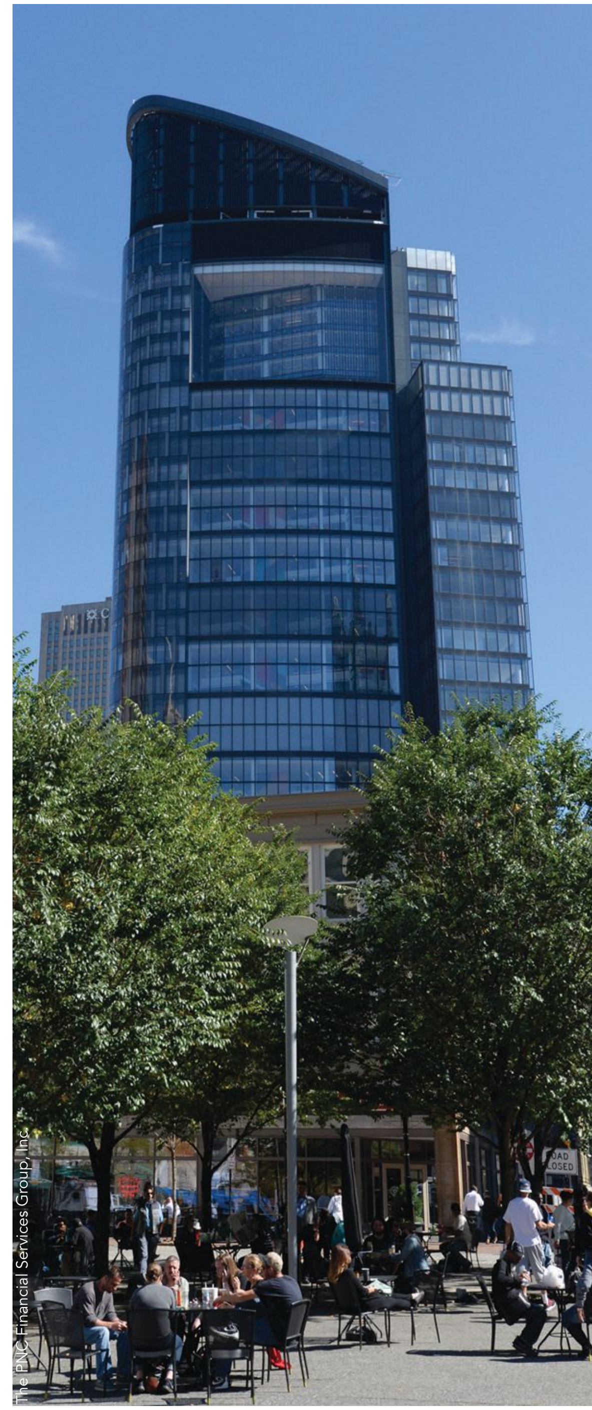
Greening STEEL TOWN

The Tower at PNC Plaza is a bold green statement in a historically industrial town.

BY ERLEEN HATFIELD, S.E., MOHAMMED HAQUE, P.E., AND STEPHEN PERKINS, P.E.

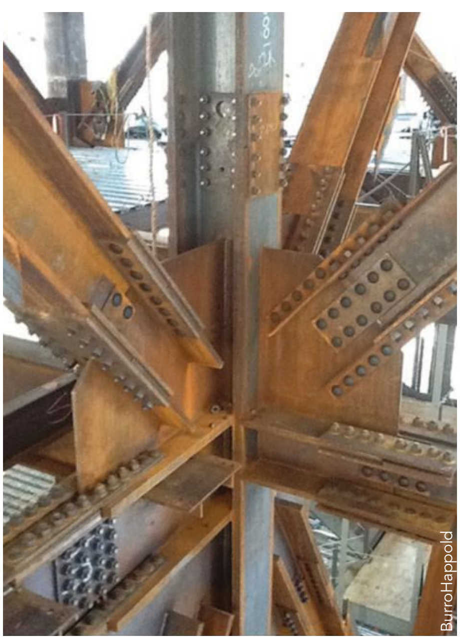
Erleen Hatfield is a partner and the leader of structural engineering in the U.S., **Mohammed Haque** is an associate principal and the project leader for The Tower at PNC Plaza and **Stephen Perkins** is a senior structural engineer and designed multiple aspects of the project. All three are at BuroHappold's New York office.

Sippel Steel Fab



The PNC Financial Services Group, Inc.

◄ ▲ ▼ ► The new 33-story Tower at PNC Plaza sits on a five-story podium and houses more than 800,000 sq. ft of commercial office, auditorium, lobby, cafeteria, parking and amenity spaces. The superstructure is entirely steel-framed, using composite steel beams with a steel braced lateral system.



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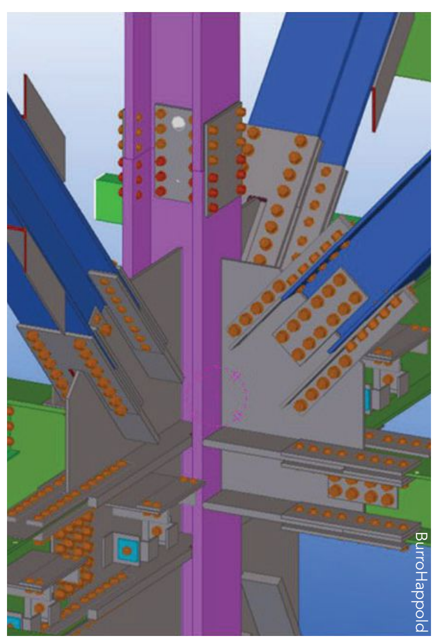

BurroHappold

IN ENVISIONING ITS NEW HEADQUARTERS, PNC Bank wanted to set a new standard in sustainable commercial design.

To achieve these lofty aspirations, a holistic design approach was essential. Detailed coordination with architecture and mechanical systems yielded an innovative structure that earned LEED Platinum certification. The new 33-story Tower at PNC Plaza sits on a five-story podium and features nearly 800,000 sq. ft of commercial office space, as well as an auditorium, a lobby, a cafeteria, parking and other amenities. The superstructure is entirely steel-framed, using composite steel beams with a steel braced lateral system. In all, the project uses 8,000 tons of steel and 200,000 bolts.

A Tower that Breathes

One of the boldest sustainable features of PNC Tower is natural ventilation, which allows it to "breathe" passively without mechanical assistance. To achieve this, the design team incorporated two vertical open-air shafts within the central core that extend through the height of the building and terminate at the underside of a 30° sloping roof. Coordination of an efficient two-story, X-braced lateral force-resisting system was crucial to achieving airflow inside the open-air shaft with minimal turbulence.

At the tower's top is a striking sloped roof clad in concrete with a glass skylight that creates a solar heat sink that naturally draws air upward through the shaft. In an effort to maximize

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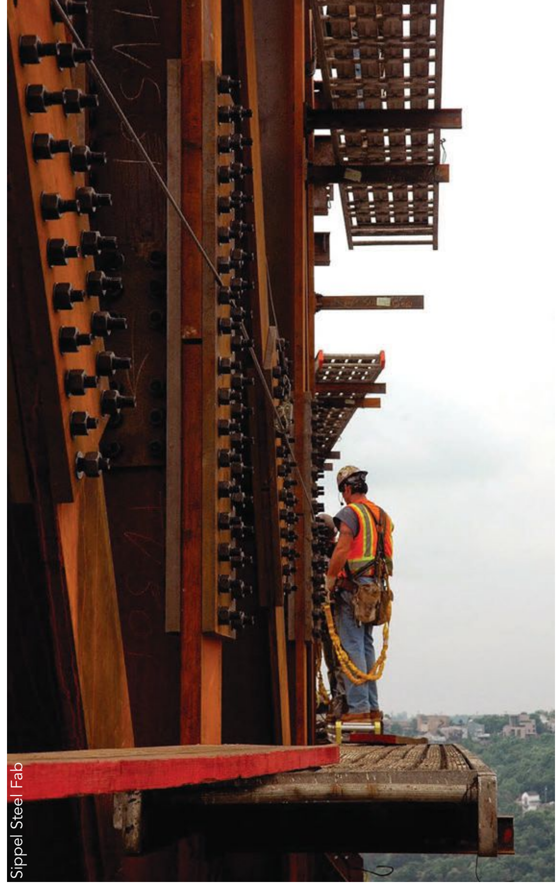

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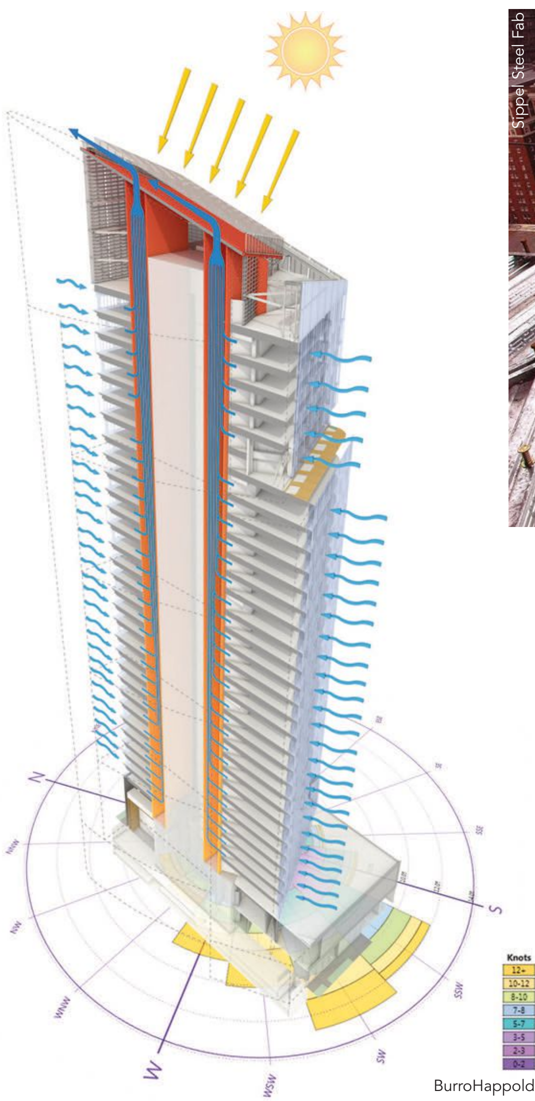
In all, the project uses 8,000 tons of steel and 200,000 bolts.

A diagram of the tower's natural ventilation system.



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The building's owners have hopes that it will be the greenest office tower in the world; it has already earned LEED Platinum certification.

The PNC Financial Services Group, Inc.

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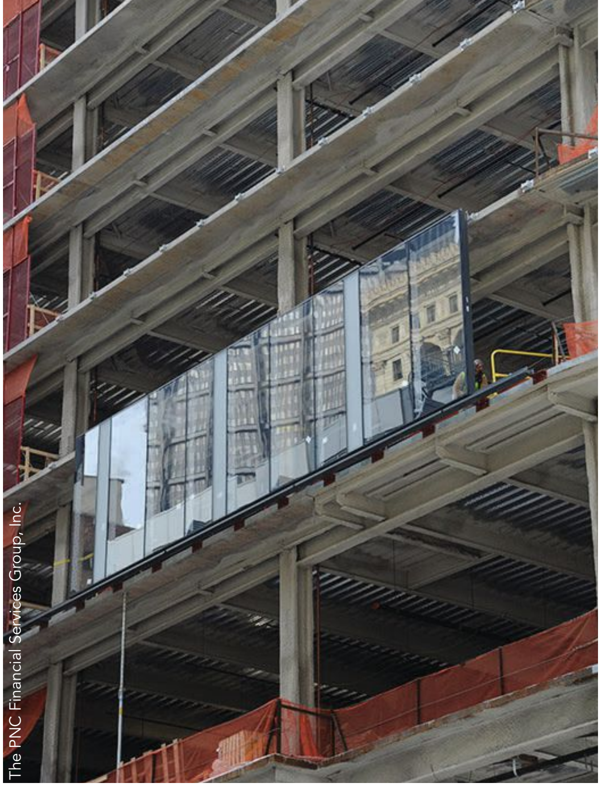

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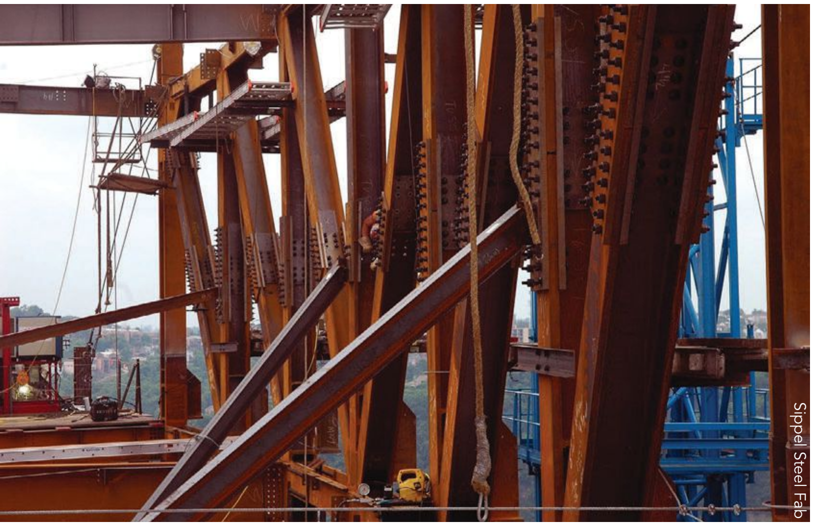
The tower's structural slab edge cantilevers 4 ft, 6 in. beyond the spandrel line to support a double-skin façade.

The two skins of the façade are essential to the natural ventilation strategy while also creating a cavity that was uninterrupted by columns.



Sippel Steel Fab

The tower's steel framing was required to be inboard of the thermal break location to prevent thermal bridging from the outside.

A 120-ft-tall cable-net façade clads the atrium space.

The PNC Financial Services Group, Inc.

solar heat gain, the roof grid was offset from the tower grid so the skylight would face the southern sun at an optimum angle. This offset roof grid and the weight of the sloping concrete created a complex engineering problem. The final design creatively used steel framing to achieve long cantilevered spans while providing a structural system that cleverly coordinated the precast panel module, skylight support frame, curved upper catwalk and offset tower grid below.

Living on the Edge

The building's structural slab edge design is unlike any other commercial tower in the U.S. Not only does it cantilever 4 ft, 6 in. beyond the spandrel line to support a double-skin façade, it does so while incorporating a thermal break component embedded within the thin depth of the slab. The two skins of the façade are essential to the natural ventilation strategy while also creating a cavity that was uninterrupted by columns. The thermal break unit consists of several components including insulation, fire protection board, pressure bearings and stainless steel rebar that, when combined together, form a lightweight premanufactured assembly that prevents unwanted thermal transfer between the cavity and the interior space. This helps to reduce energy load on the HVAC system and create a more comfortable working environment.

The design team actively coordinated early in the design phase with the thermal break and façade manufacturers to verify the acceptable location of the thermal break in relation to the façade anchors as well as the structural loads it was to be designed for. Short W10 outriggers were aligned with the infill steel beams inboard of the spandrel line and moment connected to create an adequate back span, which controlled deflection at the slab edge. Shear studs were welded to the W10 top flange to develop composite action between the cast-in-place concrete and structural steel. The steel framing was required to be inboard of the thermal break location to prevent thermal bridging from the outside.

BuroHappold Engineering detailed the steel rebar on both sides of the thermal break in order to achieve proper compatibility between the cantilever concrete slab and composite steel deck floor system. Once construction documents were issued, BuroHappold worked with the general contractor to mock up a typical slab edge condition in order to verify installation sequence and location of façade cast-in anchors, thermal break and steel reinforcement. These mock ups proved beneficial as they enabled the construction of this complex slab edge to keep pace with the steel erection and reduce any negative impact to the construction schedule. The end result is a breathtaking space at every floor that plays a crucial role in the sustainable strategy of the tower.

Inspiring View

Groundbreaking sustainable features aside, the

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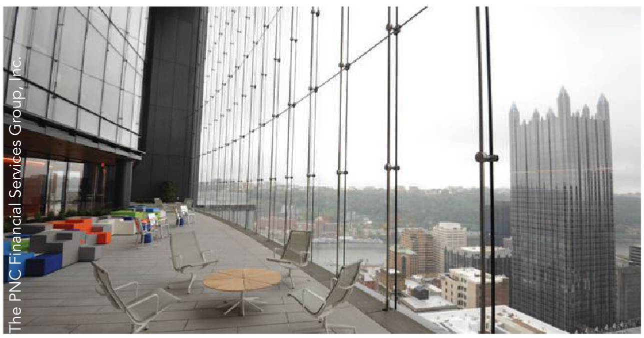
A segmented steel truss supports the cable-net façade.

A diagram of the cantilever slab edge.

The truss supporting the cable-net truss handles 45 tons of cable force at 5 ft intervals along its length.



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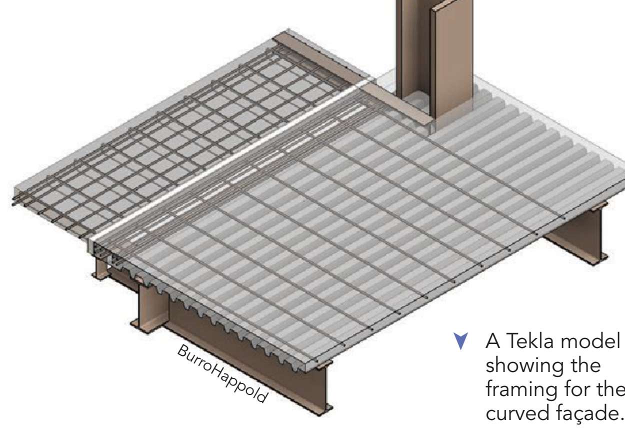

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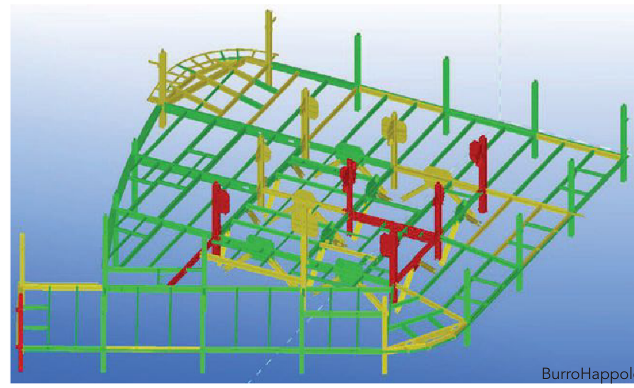
A Tekla model showing the framing for the curved façade.



BurroHappold

PNC Tower works to elevate the typical high-rise workplace environment by creating two-story "neighborhood" spaces on alternating floors. The signature neighborhood space is on the 28th floor, where a five-story atrium overlooks downtown Pittsburgh.

A segmented steel truss spanning 105 ft supports the 120-ft-tall cable-net façade that clads the atrium space. The truss is 23 ft, 6 in. tall with a total weight of 59 tons in six shipping pieces, three upper and three lower. The top and bottom chords are horizontal but curved in plan to match the building exterior. The bottom chord of the truss, at level 33, was temporarily shored from level 28 until fully assembled. The use of 3D modeling, CNC processing and shop preassembly yielded excellent fit-up at erection (the most challenging aspect was the 100% CJP welding of the truss node points).

Consisting of W14 members, the truss supports 45 tons of cable force at 5-ft intervals along its length and engages the floor system at the top and bottom chords to resist substantial torsion induced by the 2-ft eccentricity of the cable forces. This ingenious truss system creates a column-free space with unobstructed views of Pittsburgh's famous three rivers, making it an ideal place for occupants to gather, collaborate and enjoy.

Alternate Delivery Method Saves Time and Money

To expedite the compressed schedule, BuroHappold provided a Tekla 3D steel mill order model to the contractor, PJ Dick, for use in bidding, which allowed for a higher degree of accuracy in steel estimates and gave the client a faster and more competitive bidding process. This mill order model helped to eliminate duplicative modeling and allowed steel fabricator Sippel Steel Fab and the detailers to begin detailing steel earlier, thus saving time.

Unprecedented for a building of this size and type, steel shop drawings were submitted to PNC's team in a 3D Tekla model as the official medium for review as required per the structural steel specifications. This virtual 3D steel shop drawing process facilitated the review of the steel submittals and was especially helpful for review of the complex steel nodes in the lateral system where beams, columns and braces come together in PNC Tower's core and roof, and allowed portions of the model to be reviewed quickly instead of reviewing thousands of 2D PDF files. Tekla's capabilities were leveraged to create customizable reports so the reviewers could easily extract large quantities of information and export into spreadsheets for clearer and faster interpretation of steel assemblies within a given submittal. This 3D in-model-review approach saved significant time and yielded greater accuracy in the review of steel shop drawings. PJ Dick also used the 3D steel fabrication models, with connections included, to assist in a more accurate and rigorous BIM coordination with the architectural and MEP systems, which helped solve coordination issues in the office instead of the field—again, saving significant time.

The Tower at PNC Plaza officially opened its doors this past October and is a shining testament to how a holistic and innovative design approach can advance building sustainability while also reinventing the future of the modern workplace environment.

Owner
PNC Bank

General Contractor
PJ Dick

Architect
Gensler

Structural Engineer
BuroHappold Engineering

Steel Team

Fabricator
Sippel Steel Fab

Detailers
Axis Steel Detailing, Inc.
International Design Services, Inc.
Steel Structures Detailing, Inc.

Ductility in MODERATION

BY ERIC M. HINES, P.E., PH.D., AND LARRY A. FAHNESTOCK, P.E., PH.D.

Design considerations for low- and moderate-seismic regions.

NEES@Lehigh

Eric Hines (ehines@lemessurier.com) is a principal with LeMessurier and a Professor of Practice at Tufts University in Boston, and **Larry Fahnestock** (fhnstck@illinois.edu) is an associate professor and CEE Excellence Faculty Fellow with the Department of Civil and Environmental Engineering at the University of Illinois at Urbana-Champaign.

SEISMIC DESIGN OF STEEL STRUCTURES in the U.S. emphasizes the development of ductile elements designed to experience inelastic behavior during a seismic event.

The system containing these elements must be "capacity designed" with enough strength in the non-yielding elements to ensure that the yielding elements can sustain significant inelastic deformation. Thus, the expense of achieving high-seismic performance resides both within the detailing of the ductile elements themselves and the strengthening and detailing of the surrounding system to remain elastic.

A large share of the seismic research in the U.S. has focused on developing the detailing needed to achieve ductility. For decades, the cost of this detailing has been perceived to be less than the cost of designing a stronger system both in high-seismic regions as well as moderate-seismic regions. But as seismic detailing requirements have grown more sophisticated and stringent since the 1990s, engineers in areas of moderate seismicity have observed that ductile detailing of elements within a capacity-designed system can be prohibitively expensive. Consequently, the use of the R = 3 provision for steel struc-

▲ Dudley Square Police Station in Boston.

▲ OCBF brace buckling testing.

◄ Ordinary concentrically braced frame (OCBF) testing at Lehigh University.

▼ *R* = 3 brace buckling testing.

▼ *R* = 3 chevron braced frame testing.

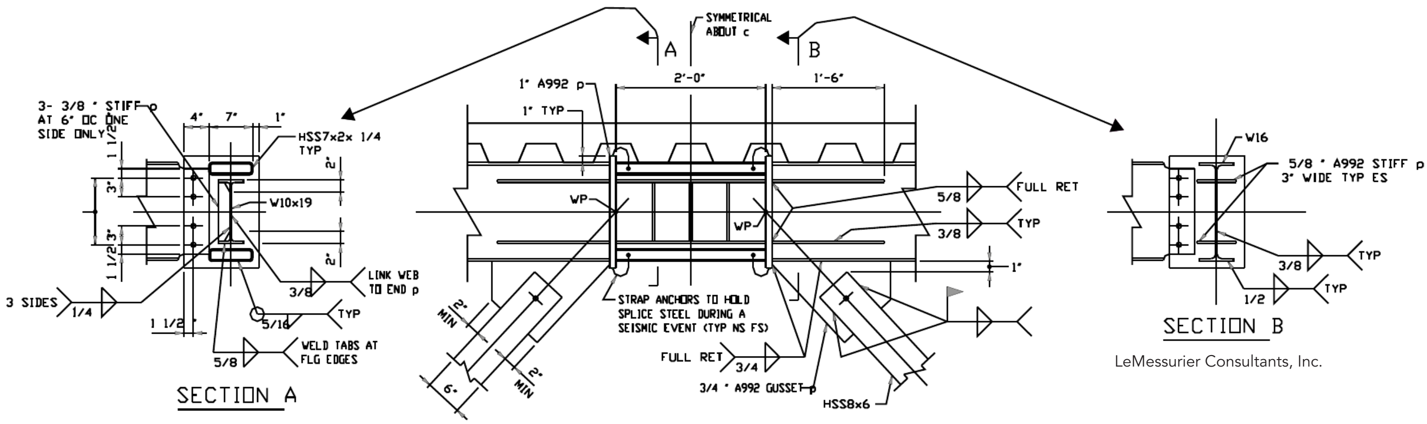
tures, which allows for seismic force reduction without ductile detailing, has expanded significantly. Within the last 10 years it has become clear that if engineers practicing in moderate-seismic regions wish to employ a consistent seismic design philosophy, a new approach to seismic research is required.

Whereas past seismic research focused on developing details to achieve a given level of ductility has placed cost in a secondary role, this new research must place priority on reducing cost while recognizing that significant performance may be achieved with moderate ductility levels. A deeper understanding of moderate-ductility systems will allow for the development of a new seismic design philosophy based on system reserve capacity.

Robust Flexibility

In moderate-seismic regions, the concept of reserve capacity can complement the concept of ductility in a manner that offers flexibility for structural designers to develop robust systems for complex structures. Low-ductility steel concentrically braced frame (CBF) structures comprise a significant portion of the national building stock, yet their inelastic seismic response is not well understood. While these structures have brittle brace elements and connections, they can achieve system ductility through contributions from the braced frame gusset plate connections and the gravity framing. The resulting "reserve" moment frame system can prevent sidesway collapse even when the primary lateral force resisting system (LFRS) is significantly damaged. In this context, ductility is viewed not as deformation capacity while maintaining full lateral strength, but rather deformation capacity while maintaining a reduced level of lateral strength, provided by the reserve system after degradation of the primary CBF.

Fundamentally, a reserve system is more flexible than a primary LFRS, hence reserve capacity activates after significant damage to the LFRS. This stiffness incompatibility between reserve system and primary LFRS differentiates reserve capacity conceptually from redundancy provided by extra LFRS elements. Although reserve capacity is not currently quantified in design, the *R* = 3 provision for steel structures in low- and moderate-seismic regions implicitly relies on reserve capacity for collapse prevention, even though the nature of this reserve capacity is not well-understood and can vary widely. Research



A shear link detail for Dudley Square Police Station.

to date has not thoroughly studied reserve capacity, so relying on it without proper understanding in moderate-seismic regions jeopardizes safety. Thus, there is an essential need for clarity and consistency in considering reserve capacity for seismic design and assessment in moderate-seismic regions.

The philosophy of system reserve capacity opens new possibilities for designing structures in moderate-seismic regions, with potential influence on assessing and retrofitting structures in high-seismic regions. This philosophy, which prioritizes cost reduction over achieving optimum levels of ductility, may also impact design in developing countries where ductile seismic details are not affordable or achievable within common practice. Reserve capacity ought to be seen as complementary to ductility. The level to which each philosophical approach is used on a given project ought to be determined by the structural designer in a manner that best suits the project.

Next, we'll discuss recent testing of low- and moderate-ductility systems, then focus on a recent police station project in Boston where the philosophies of ductility and reserve capacity were combined to achieve a high-performing, economical design for a complex system.

Full-Scale Testing

During the summer and fall of 2014, we tested two full-scale CBFs at Lehigh University's Advanced Technology for Large Structural Systems (ATLSS) laboratory in the NEES@Lehigh facility. This work was led by the University of Illinois at Urbana-Champaign (UIUC) and Tufts University as part of a Network for Earthquake Engineering Simulation Research (NEESR) project funded by the National Science Foundation (NSF). UIUC and Tufts are collaborating with a team from the École Polytechnique Montreal (EPM) on this work as part of an international program to investigate reserve capacity in low-ductility CBFs [Fahnestock et al. 2014]. This program also includes testing and analysis of top and seat angle connections for enhanced beam-to-column moment capacity [Nelson et al. 2014], testing of tubular brace reengagement with gusset plates after connection fracture [Davaran et al. 2014], and collapse analysis of 3-, 6- and 9-story low-ductility braced frame buildings with varying levels of reserve capacity [Hines et al. 2009, Sizemore et al. 2014]. The NEES@Lehigh tests included a two-story, R = 3, chevron brace configuration (Fig. 1) and a two-story, OCBF, split-X brace configuration.

These tests were designed to explore post-elastic behavior in low-ductility braced frames with a particular emphasis on brittle damage mechanisms. The full report on these tests can be found in Bradley [2015] and will soon be submitted for journal publication. These tests allow direct comparisons between detailing requirements for OCBF and R = 3 frames and well as direct comparisons between split-X and chevron configurations.

In general, the tests showed positive results for the buckling of OCBF braces designed to meet the moderately ductile member b/t requirements and connected for amplified seismic forces. For the OCBF, the upper story braces were HSS6×6×⅜ (b/t = 14.2), and the lower story braces were HSS6×6×½ (b/t = 9.90). Throughout the entire test, these braces did not experience local buckling, and they developed stable distributed plastic hinge behavior. The OCBF system survived as a moderately ductile system up to 1.5% story drift and an overstrength of approximately 3. Shortly beyond the 1.5% drift level, two fractures at the central split-X connection occurred in close succession and created a very weak two-story mechanism, leaving almost no reserve capacity. For the R = 3 system, the upper story braces were HSS8×8×5⁄16 (b/t = 24.5), and the lower story braces were HSS8×8×⅜ (b/t = 19.9). The R = 3 upper story braces buckled suddenly, with significant local buckling (Fig. 3), at a story drift of 0.3%, transitioning the system directly from an elastic behavior to a robust reserve capacity driven by frame action in the test unit.

This test program investigated two fundamental parameters that influence seismic response: system type and system configuration. System type is defined by the level of force reduction (R value) and the detailing and capacity design requirements. In this case, the system type distinction is primarily related to OCBF vs. R = 3 detailing. System configuration is defined by global frame geometry, in this case a split-X configuration versus a chevron configuration. These two tests clearly demonstrated the superiority of OCBF detailing over R = 3 detailing for achieving a moderate level of ductility through brace buckling and yielding. They also demonstrated the vulnerability of a split-X system to collapse if such a system were to form a two-story mechanism, as it did during testing. While the R = 3 system demonstrated poor ductile performance with respect to the bracing, the systems' tendency to form a single story mechanism deriving strength both from column continuity and possible "long-link EBF" mechanisms demonstrated the effectiveness of reserve capacity in maintaining system stability.

It is important to consider that the essential question of reserve capacity in these two systems hinged on whether they

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Throughout Dudley Square Police Station, uniformly sized HSS column sections were designed to be exposed to view.

formed a one-story or two-story mechanism in the post-fracture range of behavior. While the potential vulnerability of a split-X system is clear from the test, it is not possible to conclude from a single chevron test that chevron configurations can be expected, in general, to form only single story mechanisms. Designers wishing to rely on such single story mechanism behavior ought to consider exercising a form of capacity design where the story likely to form a mechanism can be identified. Having identified such a story, a designer ought to maintain an awareness of how column orientation, framing configurations, structural discontinuities and column splices would affect the behavior of the identified reserve system. If there is any chance for the formation of a brittle multi-story mechanism, it is critical for the designer to consider beam column connections that can provide adequate moment resisting capacity to form a reserve system. Such connections may be achieved economically with the use of top and/or seat angles, gusset plates and slab reinforcement as discussed by Stoakes and Fahnestock [2011] and Nelson et al. [2014].

Both of these test units represented a level of detailing consistent with the economic constraints faced by designers in moderate-seismic regions. The economy in these frames is achieved both in the compromise of ductile detailing and in the lack of a rigorous capacity design process. For collapse resistance at higher drifts, both systems would rely heavily on their reserve capacity provided by frame action and other possible post-buckling, post-fracture mechanisms. Such behavior may be considered acceptable for a large share of the building stock in moderate-seismic regions, but for essential structures, or for buildings where a higher performance objective is desired, it is reasonable to ask whether ductility can be achieved at a lower cost if the relationship between wind and seismic loads is considered carefully. Such was the case for the project we'll discuss next, where capacity design requirements were reduced by designing weak shear links to act as structural fuses and where reserve capacity concepts were invoked in locations where the architectural program complicated the direct use of ductile detailing.

Designing for Dudley Square

The new Area B-2 Police Station in Boston's Dudley Square neighborhood, designed by architect Leers Weinzapfel Associates and structural engineer LeMessurier Consultants, Inc., provided an opportunity for designers to consider the role of an essential facility as contemporary community building. For the steel structure, this meant framing a glass lobby, a perimeter clerestory and a cantilevered roof to emerge cleanly from a tight-fitting limestone ashlar façade bearing directly on the foundation with no horizontal relieving joints. Throughout the building, consistent HSS column sections were designed to be exposed to view. Architectural considerations related to the dense program within the building, the carefully crafted façade and the desire for future flexibility—combined with the structural imperative of maintaining operation after a hazardous earthquake event—led to the choice of developing the steel framing system independently of the masonry façade and partition system.

Although allowed by code for such a facility, R = 3 CBFs and OCBFs were not attractive candidates for the lateral system due to their inherently brittle behavior and unproven seismic performance. During design development, lack of opportunities for consistent bays of lateral framing in the long direction led to consideration of moment resisting frames (MRFs). However, building stiffness requirements, difficulties with detailing MRFs to perform in a ductile manner with the building's HSS

A sample shear link at Dudley Square.

The 8-in.-square HSS columns provided adequate capacity in most cases under load combinations.

columns and expenses related to the number of required moment connections spurred the design team to evaluate bracing in both directions. In order to provide the building with significant ductility capacity and stiffness, the design team looked into the possibility of using eccentrically braced frames (EBFs), more typically used in high-seismic regions.

For buildings in moderate-seismic regions that are expected to experience wind loads greater or equal to seismic loads, EBFs are commonly thought to be too expensive owing to their capacity design requirements for braces, beams and columns and field-welded erection details. Rethinking the typical EBF link details, however, allowed the designers to reduce capacity design requirements and maintain erection details consistent with a typical CBF. The result was special shop-fabricated link beams with W10×19 shear links that were proportioned to meet the elastic design requirements for the building, which were controlled by wind loads in the short direction and seismic loads in the long direction. Shear links ranged in length from 2 ft to 4 ft in accordance with architectural requirements and column capacity design limits. Horizontal HSS stiffeners on either side of each link provided both stiffness on the weak axis during erection and a surface for attaching the composite floor deck without disturbing the link itself. These stiffeners were designed as sacrificial elements, and their incidental strength was considered in the capacity design of the system. The link beams required no special measures for erection and allowed the use of relatively light braces with no special slenderness requirements.

In the absence of specific provisions for design of weak shear links in such an application, links were selected to resemble as closely as possible those tested by Okazaki and Engelhardt [2007]. These links were constructed from A992 steel in contrast to the links tested in the 1980s that were constructed from A36 steel. The test units themselves were wide-flange sections welded to end plates and then bolted to the test setup. The Dudley Square link details were designed to imitate the details of the actual test setup as closely as possible. The 33,000-sq.-ft, three-story structure was designed in conformance with the *Massachusetts State Building Code*. Per this code, it was assessed to have a fundamental period of T = 0.51 sec. Considering the number of CMU partition walls in the structure, a decision was made not to amplify the building period beyond its base value.

The seismic weight of the structure was calculated to be W = 4200 k. For Site Class C in Boston, and a seismic importance factor of 1.5, the LRFD seismic base shear was calculated to be V_E = 135 k. For exposure B and a wind importance factor of 1.15, the LRFD wind base shear was calculated to be V_W = 278 kips in the short direction and 130 kips in the long direction. Hence, while the lateral system design was controlled by seismic forces in the long direction, in the short direction the building's effective R-factor can be calculated as:

$$R_{eff} = R\left(\frac{1.0V_E}{1.6V_W}\right) = 7\left(\frac{135k}{278k}\right) = 3.4$$

This implies an additional level of safety due to the inherent strength of the EBF. Limits on possible bracing locations reduced the bracing in the building's long direction to one bay on the third floor and one bay on the second floor. Since the building's long direction was controlled by seismic forces, two moment frame bays were also designed for this direction. These moment frames plus the continuity of all of the buildings' columns were intentionally allowed to be more flexible than the EBF system in order to provide building redundancy in the form of reserve capacity, should the EBF bays become compromised.

The 8-in.-square HSS columns provided adequate capacity in most cases under load combinations, including link overstrength and gravity loads. However, four columns in the center of the structure, supporting 40-ft spans plus several bays of bracing, were designed as built-up sections from ¾-in. plate. Welds were completed on the column faces according to AESS standards, ground smooth and left exposed to view.

In order to match as closely as possible the test configuration from Okazaki and Engelhardt, W10×19 links were connected to end plates by two-sided fillet welds that are one-and-one-half times the size of the flange or web. Weld tabs were provided at the flange edges to "avoid introducing undercuts or weld defects at these edges" [Okazaki and Engelhardt 2007, p. 761]. End plates forming the transition between the W10 shear link and the W16 beam were specified as 1 in. thick. System performance was found to be excellent for a range of link sizes studies in the context of non-linear time history analyses [Hines and Jacob 2010, Jacob 2010].

The economical construction of this facility demonstrates that an EBF can be designed for a moderate-seismic region and still be economically competitive with more conventional braced frame systems. Expenses incurred via capacity design requirements can be mitigated by selecting the smallest possible links to withstand wind forces, and these link-beam assemblies can be fabricated as a single element in the shop. In the field,

Dudley Square demonstrated that an economically competitive EBF can be designed for a moderate-seismic region.

these built-up link beams and the braces can be erected in a manner similar to a typical CBF with no special detailing requirements. The extra fabrication effort required for the built-up link beams seems to be well worth the reliable safety benefits of providing a robust seismic force resisting system.

From a design point of view, the length of the link is closely related to inelastic drift requirements. The 2-ft links in this design were considered by the AISC *Seismic Provisions* to have an available link rotation angle of 0.08 radians, whereas the 4-ft links were considered to have only a 0.02 radian link rotation angle. Further testing of continuous link beams with longer links and flange yielding outside the link region could help to create more latitude for designers in moderate-seismic regions, where drift demands are expected to be significantly lower than in high-seismic regions. Results reported by Engelhardt and Popov [1992] for beams outside of links that were overloaded axially ($\pm 0.7P_y$) and in bending, intentionally to violate capacity design principles, still allowed links to achieve approximately 0.02 radians of plastic rotation. Since the test setup for this study did not include a slab, the links were framed into columns on one end, and the tests were designed to illustrate poor performance with flexible braces that allowed most of the moment to be taken by the beam. What was considered poor performance for high-seismic regions may yet imply superior performance when compared to low-ductility, low-reserve capacity CBF designs in moderate-seismic regions.

Looking Forward

In high-seismic regions, structural designers associate an expected level of seismic performance with the buildings that they design. Although the urgency associated with seismic design in moderate-seismic regions is understandably reduced, designers should still envision intended inelastic response targeted for acceptable performance when proportioning building systems [Hines and Fahnestock 2010].

In moderate-seismic regions, there is an increased tolerance of damage due to a large earthquake, so collapse prevention is the dominant performance objective. Ductility and reserve capacity are both viable approaches to achieving collapse prevention in moderate-seismic regions—and in both approaches, the relative strengths and deformation capacities of the system elements are critical considerations. Current research is developing a framework for employing reserve capacity in moderate-seismic design. ■

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TEMPERING Tremors

BY MARK SARKISIAN, S.E., P.E.,
PETER LEE, S.E., P.E.,
LINDSAY HU, P.E., AND
ALVIN TSUI, S.E., P.E.

San Bernardino's new courthouse features a comprehensive seismic design that minimizes life-cycle costs.

SAN BERNARDINO JUSTICE CENTER (SBJC) is the tallest building in San Bernardino County, Calif., and one of the tallest seismically base-isolated buildings in the United States.

Consisting of two building elements—an 11-story courtroom tower and an interconnected, three-story podium—the new 383,745-sq.-ft building contains 35 courtrooms and improves the efficiency of the courts by consolidating functions that had previously been spread throughout the county across 12 different buildings, many of them vulnerable to earthquakes.

In close proximity to known active faults including the San Jacinto, San Andreas and the Cucamonga Faults, the SBJC is located in one of the most active seismic regions in the U.S. It is designed in conformance with the 2006 *Trial Court Facility Standards of the Judicial Council of California* (JCC) to achieve "enhanced" seismic performance objectives to limit damage and loss of operations under expected moderate to major earthquake events. Analysis and design of the steel superstructure gravity and lateral systems was completed in compliance with provisions of the 2007 *California Building Code* (CBC) and ASCE 7-05 requirements including design review panel oversight.

Superstructure Framing

The gravity steel-framed structure consists typically of a 3¼-in. lightweight concrete fill over 3-in. 20-gauge metal deck. Composite steel floor framing at the mechanical level, roof penthouse, Level 1 and below grade levels consists of 4½-in. normal weight concrete fill over 3-in. 18-gauge metal deck. The standard floor plan, with an open, column-free layout and story height of 16 ft, accommodates the courtrooms with clear ceiling heights of 12 ft.

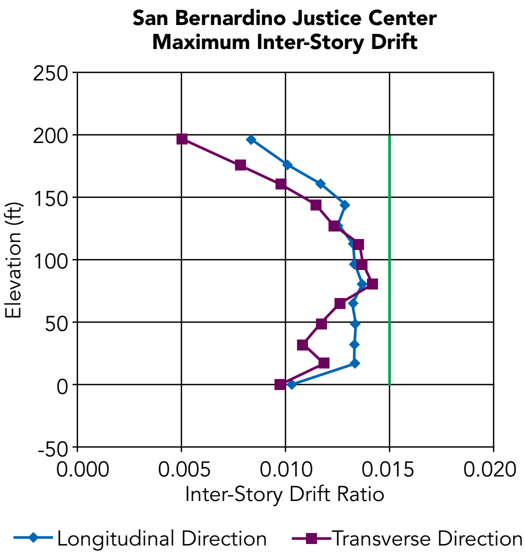
The lateral force resisting superstructure system consists of steel special moment frames (SMF) on essentially all frame grid lines in each direction and at each level, with 184 distributed supplementary viscous damping devices (VDD), made by Taylor Devices, Inc., with extender brace elements. The superstructure frame is supported on an energy-dissipating seismic isolation system above the lowest mat foundation level. The steel SMFs consist typically of reduced-beam section (RBS) W24 beam/girder two-way moment connections to 18-in. to 24-in. square built-up box columns and W24 cruciform column elements in conformance with the prequalification connection

The new San Bernardino Justice Center is one of the tallest seismically base-isolated buildings in the U.S.

Consisting of two building elements—an 11-story courtroom tower and an interconnected, three-story podium—the new building contains 35 courtrooms and consolidates functions that had previously been spread throughout the county across 12 different buildings.

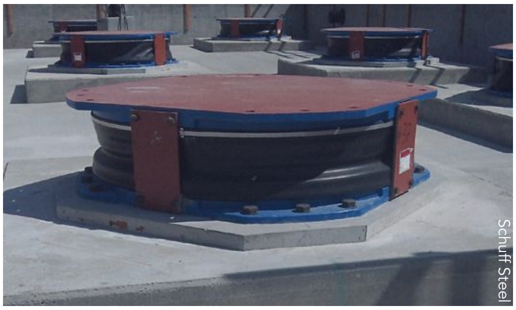


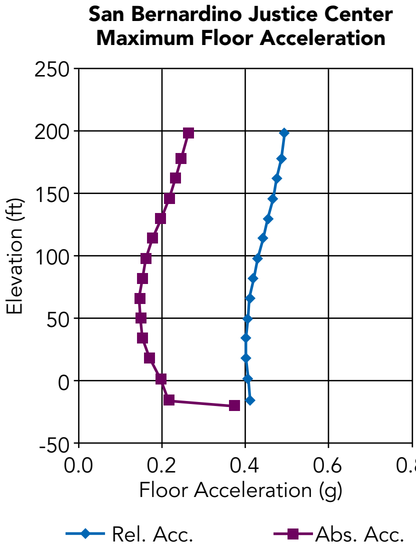

Mark Sarkisian is a partner, **Peter Lee** is associate director, **Lindsay Hu** is an associate and **Alvin Tsui** is an associate, all with Skidmore, Owings and Merrill, LLP, in San Francisco.



▲ The gravity steel-framed structure consists typically of a 3¼-in. lightweight concrete fill over 3-in. 20-gauge metal deck.

◄ ▲ ▼ The superstructure frame is supported on an energy-dissipating seismic isolation system above the lowest mat foundation level. This system in made up of 69 triple concave-friction pendulum (TC-FP) bearings.





criteria of AISC 341-05 and AISC 358-05. The VDD brace elements have a 440-kip design force with a +/- 5-in. stroke capacity. VDDs control seismic drift demands while the SMFs minimize uplift on the base seismic isolation system.

Seismic Isolation System

The seismic isolation bearing system, manufactured by Earthquake Protection Systems (EPS), consists of 69 triple concave-friction pendulum (TC-FP) bearings located above the base mat foundation. The TC-FP isolation bearing system will accommodate up to 42 in. of lateral movement at the seismic isolation plane and perimeter moat walls. The bearings transfer gravity and lateral forces to the supporting foundation subgrade via reinforced concrete pedestals and mat foundation slab system.

The isolation plane was carefully selected at below the lowest occupied level and at perimeter building perimeter conditions to minimize the number of building utilities and service elements required to be detailed to accommodate movements across the plane. Locating the isolation plane above the mat foundation required approximately 55,000 sq. ft of steel-framed space at the lowest occupied building level.

Analysis modeling of each seismic isolation bearing was developed on the basis of assumed mechanical properties of the TC-FP bearings, verified by both prototype and production bearing testing. Architect and structural engineer SOM collaborated with EPS during the design phases in evaluating five different bearing geometries and properties, each with two or more bearing types, in determining the most appropriate and optimal isolation system properties for the isolated structure in response to the site-specific ground motion criteria.

The effective radii (L) of the outer and inner concave surfaces are 300 in. and 72 in., respectively, and the friction coefficients of breakaway, target, lower bound (LB) and upper bound (UB) are 5.0%, 9.0%, 7.5% and 10.5%, respectively. The upper and lower sliding surfaces have equal friction coefficients, and UB and LB properties include adjustments for potential aging and contamination effects. Modeling the TC-FP was based on the equivalent bilinear model to represent the breakaway friction coefficients of inner and outer sliding surfaces. The structural analyses included the bounded analysis of the UB and LB friction coefficients to determine maximum force and displacement demands. EPS confirmed analysis parameters based on the results of a prototype bearing testing program conducted during the construction document phase under direction of SOM and the design review panel.

Damping Devices

The VDDs have a nonlinear velocity exponent of 0.5, a damping constant of 150 kip-sec/in., a maximum stroke capacity of ±5 in. and a maximum design force of 440 kips. The structural analyses included the bounded analysis of ±15%, using the upper and lower bound damper properties of a damping constant. The VDD elements were modeled using nonlinear damper elements in ETABS, and the damping devices were designed to have displacement and design force demands for maximum considered earthquake (MCE) level ground motions. The connections and extenders transferring the damper forces to the structural members were designed to develop maximum forces in the damping devices. Since the damper forces are substantially out-of-phase with elastic forces, the dampers do not significantly increase loading on the structural members while providing significant levels of damping in response to earthquake loads.

Using seismic isolation and VDDs significantly reduces base shears, overturning moments, story drifts and floor accelerations. The seismic isolation system effectively reduces the superstructure floor accelerations by increasing the period and controls base displacements with increased damping and energy dissipation. The supplemental VDDs absorb a significant amount of seismic energy, further reducing the story drift demands in the superstructure. The vertical displacement of the isolation bearings was reduced significantly below the maximum allowable story drift of 0.015. The story drift ratios were estimated from the analysis results from the upper bound and lower bound damper models with upper bound and lower bound seismic isolation system properties.

Based on the series of nonlinear response-history analyses, the maximum average drift ratios are about 1.4% under the design earthquake (DE) and about 1.7% under maximum (MCE) ground motions. The maximum story drift and floor acceleration demands in the seismically isolated building are significantly smaller than that in a fixed-based building due to the decoupling effects of the seismic isolation system. Significant enhanced seismic performance is achieved using both the VDD and isolated systems to reduce peak floor acceleration and drift demands to minimize damage on sensitive non-structural elements and components.

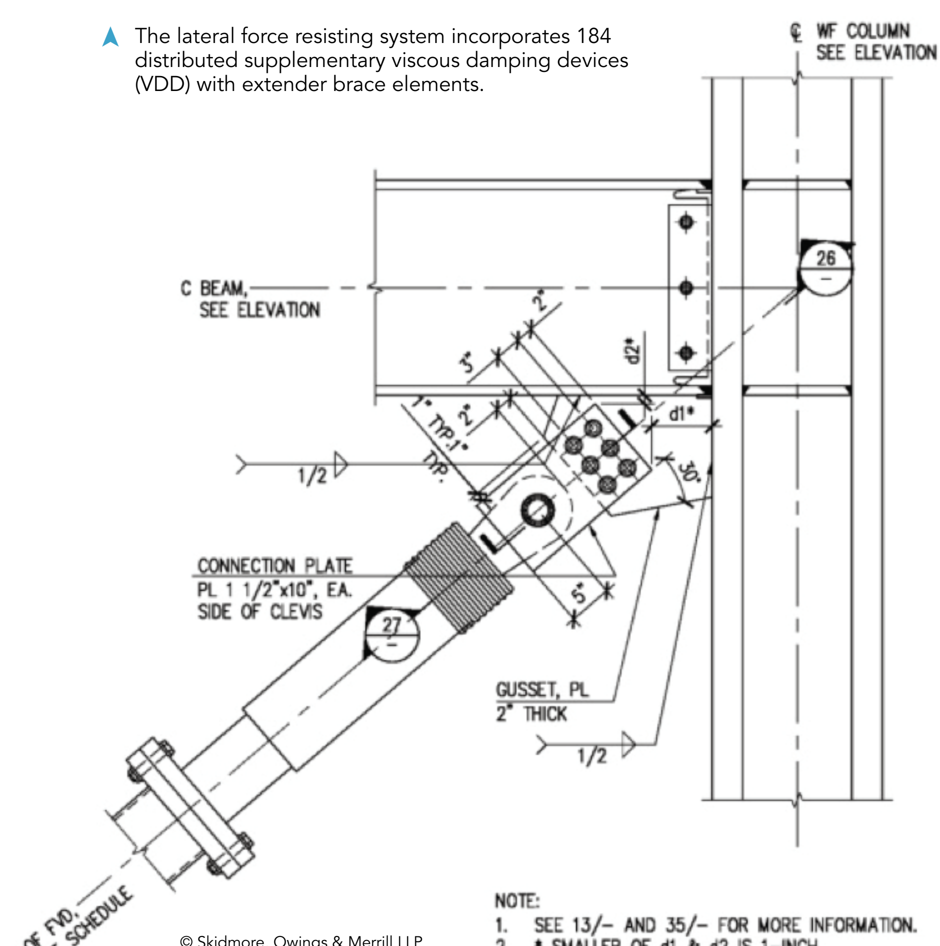
The lateral force resisting superstructure system consists of steel special moment frames (SMFs) on essentially all frame grid lines in each direction and at each level.

The lateral force resisting system incorporates 184 distributed supplementary viscous damping devices (VDD) with extender brace elements.

Seismic Risk and LCA

Consistent with an "enhanced" seismic performance objective for the SBJC, this is the first project for which the JCC embraced life-cycle analysis (LCA) to consider the impact of alternate structural systems on the long-term seismic performance and the relative return on investment in a region of high seismicity. In collaboration with Certus Consulting, Inc., SOM conducted a seismic risk assessment and LCA based on a 25-year return period during design development to inform client decision making. The evaluation of alternate conventional fixed-based (non-isolated) options showed an 18.5% return on investment for the seismically isolated superstructure that included estimated mean annual losses from damage to structural, nonstructural (drift and acceleration sensitive) and building contents, as well as loss of use and business interruption impacts.

Working with the California Strong Motion Instrumentation Program (CSMIP), SOM developed a strong motion seismic instrumentation system as part of the Justice Center's new construction, which provides for the recording of earthquake motion data within, below and adjacent to the building during earthquake induced events. Basic elements of the instrumentation layout included 36 sensors (32 accelerometers and four relative displacement sensors) within the building as well as three ground station (free field) sensors at the southeast corner of the building site. All accelerometers are interconnected to centrally located computer controlled digital recording equipment for common start, timing and synchronization. In coordination with the general contractor and subcontractors, CSMIP provided support and assisted in the installation of the SBJC's seismic instrumentation system while providing long-term maintenance and ongoing monitoring during future seismic events as part of California's statewide network. Originally targeting LEED Silver certification from the U.S. Green Building Council, the courthouse achieved LEED



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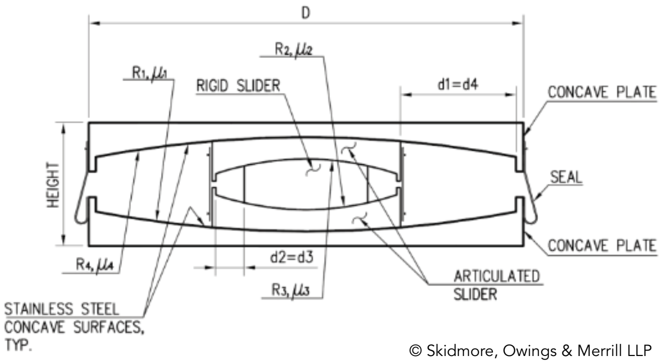
A typical damper device connection detail.

Triple Concave Friction Pendulum (TP-FP) Isolator Geometry



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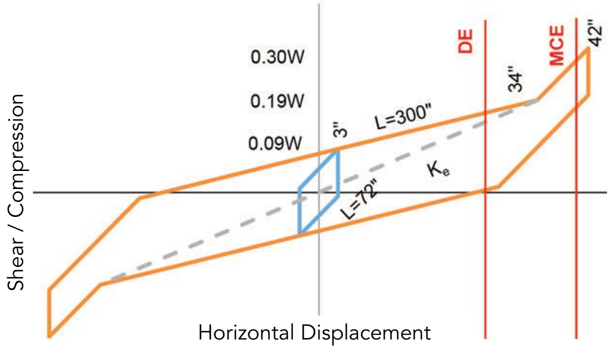
TC-FP Isolator Force-Displacement Properties



A steel assembly atop one of the seismic isolation devices.

Gold at no added cost, in part thanks to its comprehensive consideration of earthquake resilience. ■

Owner
Judicial Council of California, State of California

Construction Manager/General Contractor
Rudolph and Sletten

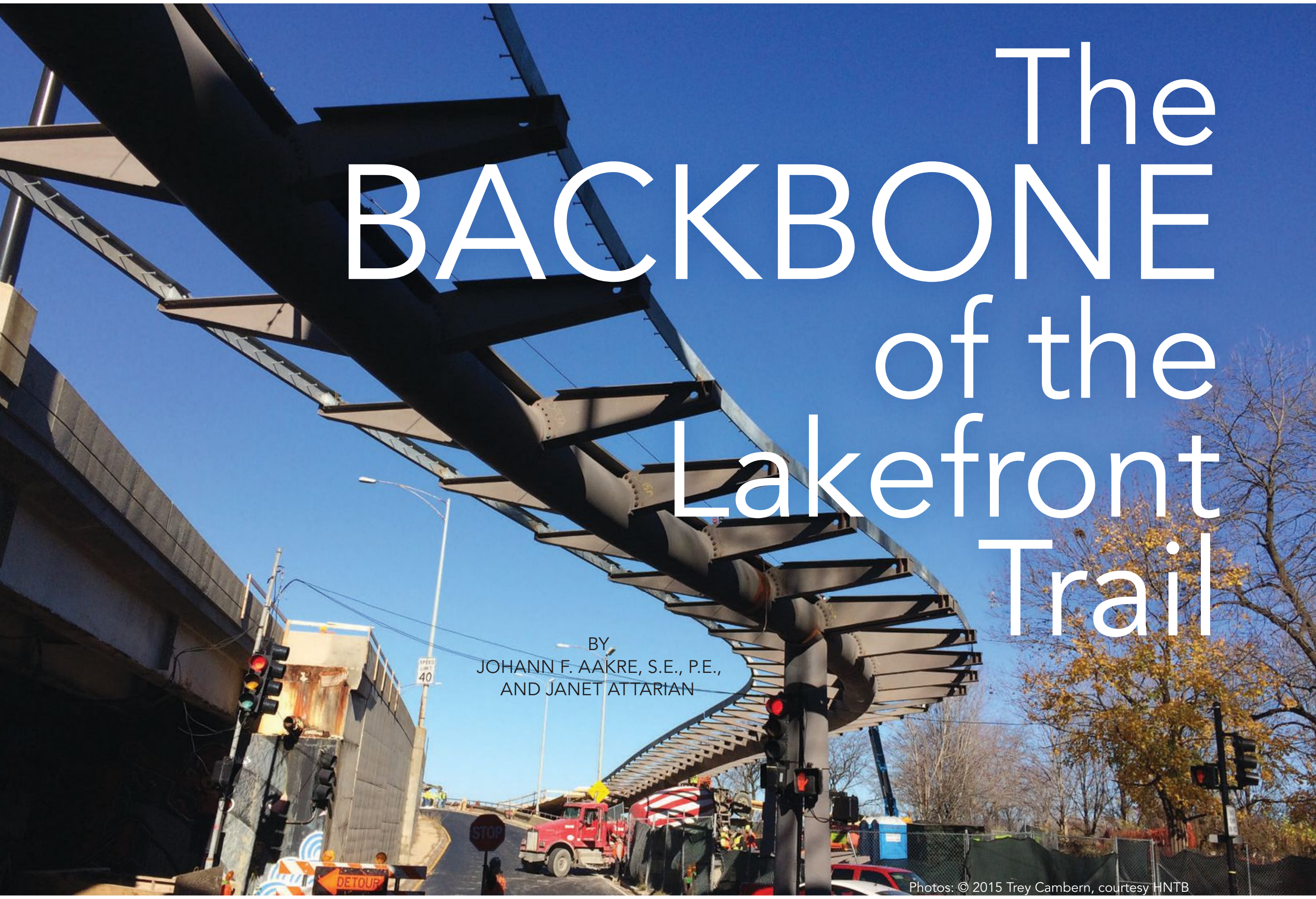
Architect and Structural Engineer
Skidmore, Owings and Merrill LLP

Steel Team

Fabricator and Erector
Schuff Steel

Detailer
Steel Systems Engineering, Inc.

A new elevated pathway provides safe passage for pedestrians and cyclists near one of the Windy City's premier tourist destinations.



The BACKBONE of the Lakefront Trail

BY JOHANN F. AAKRE, S.E., P.E., AND JANET ATTARIAN

Photos: © 2015 Trey Cambern, courtesy HNTB

CHICAGO'S LAKEFRONT TRAIL is a popular cycling, walking and running venue for residents and tourists alike.

The 1,750-ft-long, $60 million "Navy Pier Flyover" now under construction as part of Mayor Rahm Emanuel's "Building a New Chicago" program will eventually relieve congestion along the most heavily used section of the 18.5-mile trail.

Site constraints and limited clearances created by a parking garage, a residential high-rise, local streets, highway access ramps and several parks, a serpentine alignment was essentially the only option for the grade-separated pathway. Aesthetics were equally as important as functionality because the flyover will be visible from every angle in a highly visible part of the city. A dramatic steel spine-rib superstructure support system became the essential component that helped the team achieve objectives of both form and function.

Construction is being executed in three phases: The first, northern phase is between Jane Addams Park and Ogden Slip; the second stretches from this slip to the Chicago River; and the third phase will span the river by modifying the path on the Lake Shore Drive bridge. Construction of the supporting steel superstructure for phase 1 is now finished as the $27.9 million first segment nears completion.

Eliminating a Bottleneck

With 60,000 users traversing this segment of the Lakefront Trail during peak use, in the late 1990s the Chicago Department of Transportation (CDOT) began looking at ways to alleviate the bottleneck created when trail users crossed paths with pedestrians on city streets, local vehicle traffic and highway entrance ramps. All of this congestion, combined with missing or deteriorated pavement markings, lack of way-finding signage and poor trail surface conditions were contributing to frequent accidents.

In 2003, HNTB began working with CDOT and Muller and Muller Architects to develop a bridge configuration that was functional, aesthetically pleasing and contextually appropriate. The overriding objective for the new path was to separate shared crossing points between pedestrians and vehicles at Illinois Street and Grand Avenue, the heavily congested (not to mention only) entry and exit streets to the Navy Pier facilities.

Initially, finding a viable path for the structure was the most formidable challenge. In one area, the partial removal of the east shoulder of upper Lake Shore Drive was deemed necessary to squeeze the elevated path between it and Lake Point Tower, an iconic residential high-rise. Given the proximity of the path to the 70-story condominium tower—there will be only 9 in. separating them—concessions were made with residents to erect a mesh screen wall between the structures for added security.

In another section, the path alignment had to be shifted across the currently undeveloped DuSable Park, designated as a Superfund site by the U.S. Environmental Protection Agency because of radioactive thorium nitrate present in fill material placed in the early 1900s. To minimize the need for soil remediation at flyover support columns, HNTB used minimally invasive steel H-pile foundations.

A Flexible Spine

Tasked with designing a serpentine structure that would be practical, structurally sound and visually appealing, HNTB incorporated the use of an easily manipulated, central steel spine. The longitudinal spine-rib support system accommodates the complex bridge's need to curve both horizontally and vertically and provides the desired aesthetics.

The bridge's central spine is fabricated from 30-in.-diameter steel pipe, either 1¼ in. or 1¾ in. thick depending upon span and strength requirements. A T-shaped web and flange are welded on top of the pipe to provide added strength and a surface for shear studs, which enable composite action between the steel and the path's 6-in.-thick, 17-ft, 10-in.-wide concrete deck. For Phases 1 and 2, nearly 350 tons of API 2B steel pipe will be used in the spine.

The size of the pipe combined with the serpentine alignment did come with a challenge. Since the project is federally funded, the steel needed to meet Buy America provisions and most pipe of this size and thickness is produced overseas. In discussions with fabricators and the steel industry, it was determined that the pipe could be produced domestically in accordance with API 2B specifications, which is for pipe manufactured from plate that is rolled into cans and then longitudinally welded.

Rib elements, fabricated from steel plates, are connected to each side of the steel spine on 8-ft centers, tapering in

The 1,750-ft-long, $60 million "Navy Pier Flyover," now under construction as part of Mayor Rahm Emanuel's "Building a New Chicago" program, will eventually relieve congestion along the most heavily used section of the 18.5-mile Lakefront Trail.

Johann F. Aakre is a project manager with HNTB Corporation and **Janet Attarian** is Livable Streets director with the Chicago Department of Transportation.

depth from approximately 2 ft, 2 in. at the central spine to less than 5 in. at the outer deck edge to create a sleek, graceful appearance. A longitudinal steel channel, running parallel to the steel pipe spine, is bolted to the ends of the steel ribs to facilitate construction of the deck and to support the path railing.

The design uses the frame action created by the spine-column rigid connection. This not only controls in-plane bending, but also resists out of plan bending and torsion in the spine. As such, the analysis considered the column supports of the structure, rather than treating the superstructure as a continuous beam supported atop various piers. Thermal range was also factored into the design, requiring the development of unique expansion bearing and column connections. In one location, where the spine is supported on Lake Shore Drive itself, a dapped connection was inserted into the spine so that its thermal displacements would act in conjunction with the Lake Shore Drive Bridge. At this location and at the expansion piers, the spine is supported by low-profile disc bearings, serving to maximize the amount of steel pipe available to carry the forces at the bridge supports.

Connections between the steel elements of this unique structure also came with a set of challenges. The connections needed to fit within the aesthetic constraints of the project and custom details were developed. Where possible, AISC design guidelines for HSS connections and CIDECT publications were used as resources. In some cases, however, refined 3D finite element models were relied upon where the details did not fit entirely within the context of the code.

Steel Substructure and Foundation

The steel spine is supported by steel columns created from 1¼-in. steel plate bent into a 30-in. by 22-in. elliptical shape along the main alignment and by cantilevered concrete abutments at its ends. At three locations, the path is supported directly from the existing bents of Lake Shore Drive. The elliptical column extends 2 ft below the bottom of the spine before separating into two half-elliptical column legs that splay apart to create a wider base for additional stability. This arrangement was devised to provide an aesthetically continuous and pleasing transition between the bridge elements.

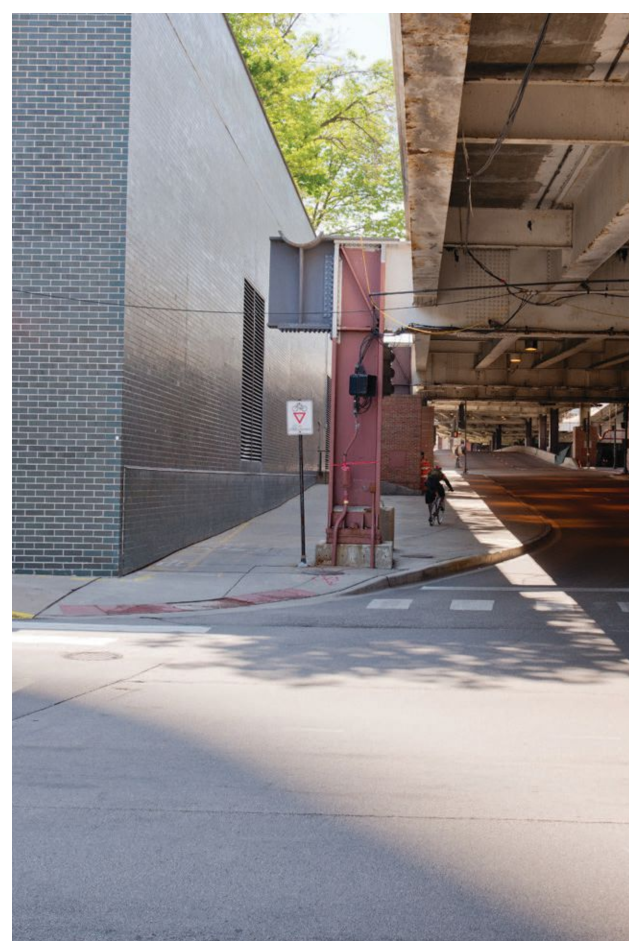
The foundation system is primarily comprised of steel piles driven to 50-ton capacity and embedded into a 3-ft, 3-inch-thick rein-

▲▼ The flyover spine is supported by steel columns created from 1¼-in. steel plate bent into a 30-in. by 22-in. elliptical shape along the main alignment and by cantilevered concrete abutments at its ends. At three locations, the path is supported directly from the existing bents of Lake Shore Drive.

▲▼ The bridge's central spine is fabricated from 30-in.-diameter steel pipe, either 1¼ in. or 1¾ in. thick depending upon span and strength requirements.

▲ A full-scale mockup of the spine, deck and railings was constructed on-site to refine the details.

▼ The path will come as close as 9 in. to an adjacent 70-story residential high-rise.



Steel plate rib elements are connected to each side of the steel spine on 8-ft centers, tapering in depth from approximately 2 ft, 2 in. at the central spine to less than 5 in. at the outer deck edge.

The design uses the frame action created by the spine-column rigid connection. This not only controls in-plane bending, but also resists out of plan bending and torsion in the spine.

forced concrete cap. Steel piling was selected as the preferred foundation type since it minimized the amount of excavation and spoils, especially necessary when working at the Superfund site.

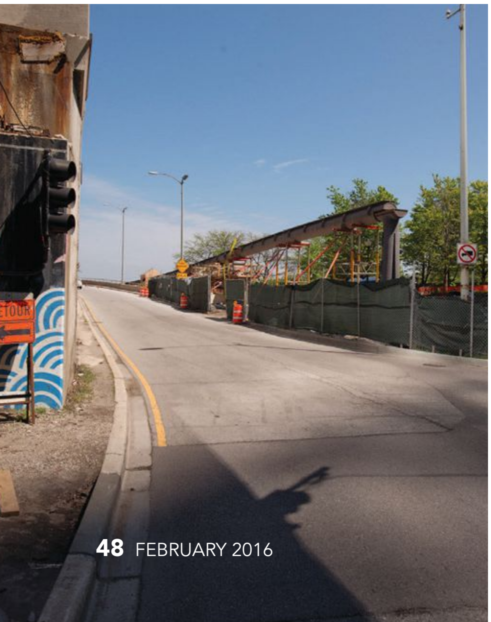
Attention to Detail

All details were highly scrutinized to ensure that the desired function and look would be achieved. Architectural cable steel railings and panels and custom steel deck nosings were incorporated into the design. In addition, the paint system is not the typical zinc/epoxy/urethane system used on highway bridges, but rather a three-coat system comprised of a primer, an intermediate coat and a fluoropolymer finish coat. Typically used on building applications, this system is highly durable and provides enhanced color and finish retention.

Highlighting these details is a comprehensive LED lighting system that will illuminate the ribs, spine and columns from below and will shine down from the cable railing posts above. Most conduits for lighting power supply are embedded in the concrete curb or are enclosed within the steel elliptical column sections. Drainage downspouts were custom detailed to follow the path of the ribs and columns to integrate with the structure. A ⅜-in.-thick stainless steel curb cover plate extends along the deck edges to provide a uniform shape and enhanced appearance. To ensure that all details were worked out during construction, the contractor created a full-scale mock-up before starting full production.

When all three phases are complete in 2018, the elevated pathway will significantly improve this key segment of the Lakefront Trail, safely guiding pedestrians and cyclists through one of the most heavily trafficked regions of the city. By maneuvering the path through a 3D obstacle course of existing structures, the pathway's steel spine system has proven itself to be up to the challenge. ■

The paint system is not the typical zinc/epoxy/urethane system used on highway bridges, but rather a three-coat system comprised of a primer, an intermediate coat and a fluoropolymer finish coat. Typically used on building applications, this system is highly durable and provides enhanced color and finish retention.



➤ When all three phases are complete in 2018, the elevated pathway will significantly improve a key segment of the Lakefront Trail.

Owners
Chicago Department of Transportation (primary), Chicago Park District, Illinois Department of Transportation

General Contractor
F. H. Paschen

Construction Manager
T.Y. Lin International Group

Architect
Muller and Muller Architects

Structural Engineer
HNTB Corporation

Erection Engineer
Bowman, Barrett and Associates Inc.

Steel Team

Steel Fabricator
Hillsdale Fabricators, an Alberici Enterprise (AISC Certified Fabricator)

Bender-Roller
Bendco (AISC Associate)

EXTREME Makeover

BY RON KLEMENCIC, S.E., P.E.

Nu Skin Enterprises' Utah campus gets a fresh new look.

THE NU SKIN INNOVATION CENTER brings a new face to Nu Skin Enterprises.

The anti-aging product company's new Provo, Utah, headquarters houses research laboratories, conference spaces, two cafés, a retail storefront, a fitness center, three floors of executive offices and a data center in a series of elegant, light-filled spaces.

A Structural Triptych

The 170,000-sq.-ft, $74 million Innovation Center is comprised of three primary elements: a more modest three-story building to the north that responds to the scale of Provo's historic Center Street; a six-story steel-framed building to the south; and a four-story steel atrium linking the new buildings to each other and to the existing Nu Skin office tower. The atrium is the heart of the new campus, acting as a glazed spine and entry hall designed to host thousands of people from around the world and to accommodate multiple activities and events concurrently.

While the framing for the north building is concrete, the south building and atrium use structural steel framing. The typical framing for the south building is comprised of structural steel columns supporting composite steel beams and composite floor slabs with 3 in. of normal-weight concrete over 3-in. steel deck. For the south building, W14×90 to W14×342 columns were used; HSS20×8 and HSS18×6 columns were used for the atrium. The most common beam sizes were W18, W21 and W24, and W24, W30 and W33 were the most common girder sizes; the largest were W30×235, W33×118 and W36×210.

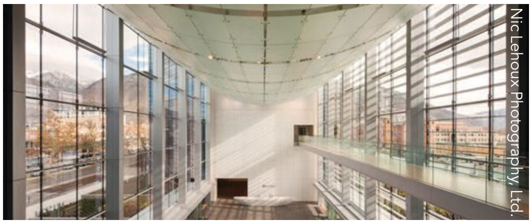
To eliminate columns in a large meeting room at the first floor, six tower columns are transferred at the third floor. These columns are supported by two 67-ft-long built-up steel plate girders spanning in the north-south direction and two 85-ft-long story-deep trusses spanning in the east-west direction.

Crowning the south building is an airfoil-shaped mechanical penthouse, a nod to the barrel-vaulted forms of the original Nu Skin tower (which used the same fabricator as this project, Tech-Steel). This sharply curved, steel-framed element is one of the exterior highlights of the project: the radii of the roof beams vary from 109 ft, 8 in. all the way down to 3 ft, 4 in. W18×65 and HSS10.000×0.500 are used at the braced frames at each end of the barrel vaults, and the typical curved beams are W18×40 and are spaced at 11 ft on center.

The exterior of the building is composed of sleek, transparent volumes anchored by crisp, aluminum-clad core spaces. Delicate sunshades along the south elevation of the entry hall and offices shade the interior spaces from direct sunlight while framing

The 170,000-sq.-ft Innovation Center is comprised of three primary elements: a three-story building to the north that responds to the scale of Provo's historic Center Street; a six-story steel-framed building to the south; and a four-story steel atrium linking the new buildings to each other and to the existing Nu Skin office tower.

Two steel-framed bridges span across the atrium, which connects the two new buildings, in the north-south direction.



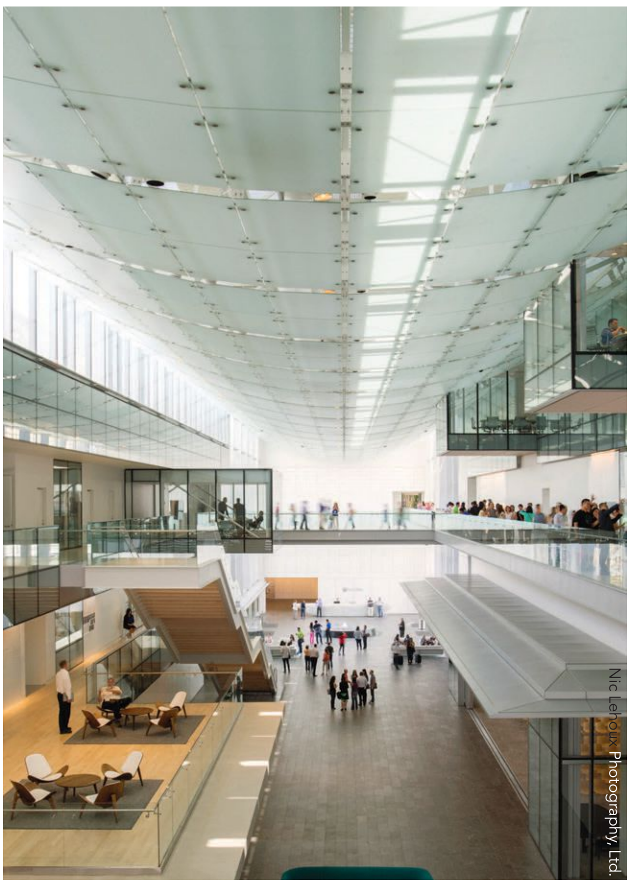
Ron Klemencic, S.E., P.E., (**rklemencic@mka.com**) is Magnusson Klemencic Associates' chairman and CEO.

views of the nearby Wasatch Mountains. Slender HSS columns (6 in. in diameter and 18 ft tall) support a canopy on the south elevation that extends the interior spaces 28 ft into the landscape, while providing shade and protection during inclement weather.

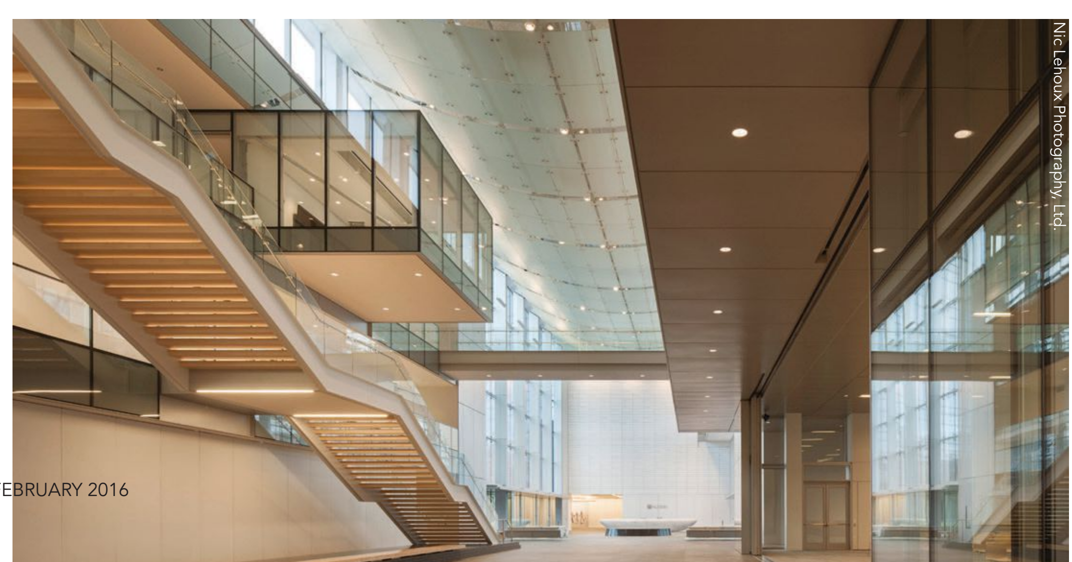
Glass and Steel Heart

The atrium is the heart of the Nu Skin campus. At the entrance, a granite fountain and sculpted marble reception desk greet visitors to the space. Further west, telescoping glass walls open to a 500-seat meeting room and offer views of a new garden and the campus to the south. Across from the meeting room, a monumental staircase—a built-up plate assembly—draws visitors and staff up to the data center, laboratory and office levels that are connected by circulation paths bridging across the space. The two second-floor plate girders for the atrium, (6 ft, 6 in. deep) are suspended from story-high trusses above, and four temporary columns were used to support the girders while the trusses were field-assembled.

There are two steel-framed bridges that span across the atrium in the north-south direction, one at the east end (which spans 41 ft) and one at the west (47 ft long); both are located at the third level. The bridge girders are W24×68 beams rigidly connected to the south (steel) building and resting on slide bearings on the north building. The slab of the bridges consists of 2.5 in. of concrete on 2-in. steel deck supported on W8×10 purlins that span between the bridge girders.



▲▼ Above the atrium, glass conference rooms cantilever into the space, and a gently curving ceiling of translucent glass is suspended below steel trusses supporting the skylight roof, mitigating the intense Utah sunlight and softening the interior space. There are three conference rooms—one at the third level that cantilevers 16 ft and two at the fourth level that cantilever 11 ft.

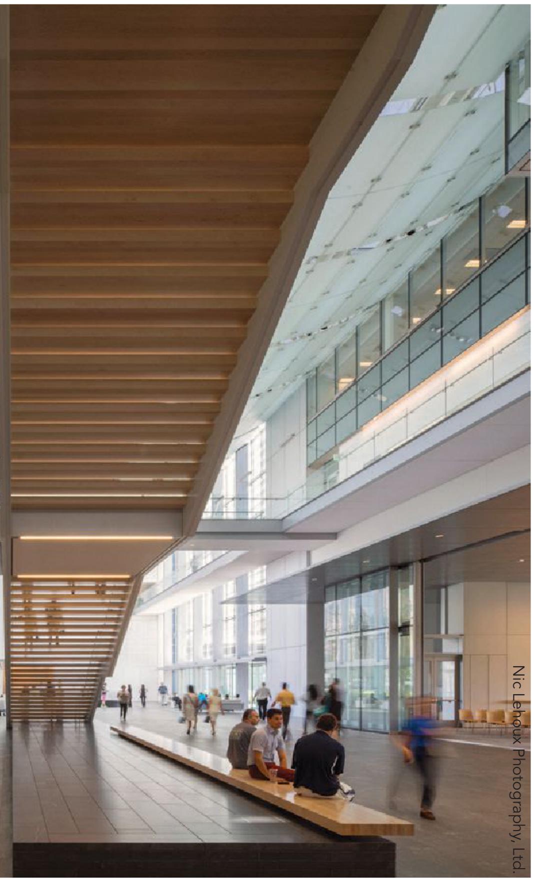


Above the atrium, glass conference rooms cantilever into the space, and a gently curving ceiling of translucent glass is suspended below steel trusses supporting the skylight roof, mitigating the intense Utah sunlight and softening the interior space. The chords of the steel trusses supporting the curved glass ceiling are composed of two C4×5.4s back to back; the members forming the web are HSS3×1½×¼. Purlins spanning between trusses are HSS4×3×¼. There are three conference rooms—one at the third level that cantilevers 16 ft and two at the fourth level that cantilever 11 ft.

The glass roof is supported by steel girders that span between the north and south buildings, along with intermediate steel beams and tension bracing, and the translucent glass ceiling is hung from trusses, which are in turn suspended from the roof girders. The 10-ft, 6-in.-wide feature stair rises 29 ft between levels 1 and 3 and runs 93 ft continuously along the atrium. The stair stringers and treads are both supported by steel channels, and the bridges spanning the atrium are supported by steel beams.

The conference rooms cantilevering into the atrium are supported by shallow beams (W10×12), post and tension rods, and the four-story-high glass walls are supported by primary and secondary structural steel.

Seismic design is a primary consideration in Provo (Provo is a high-seismic zone and the structure considered Seismic Design Category D and the soil is Site Class D) so the atrium roof

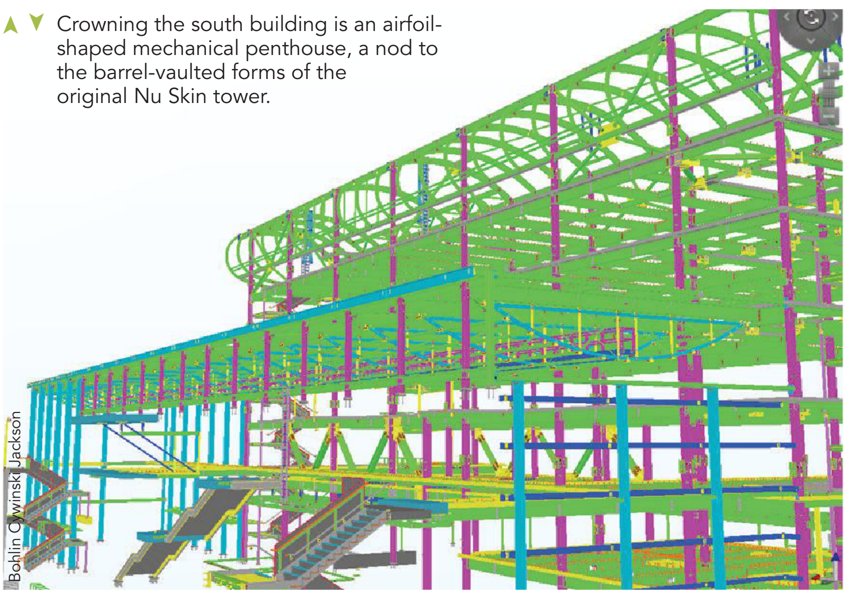


Nic Lehoux Photography, Ltd.

▲▼ The 10-ft, 6-in.-wide feature stair rises 29 ft between levels 1 and 3 and runs 93 ft continuously along the atrium.

Bohlin Cywinski Jackson

▲▼ Crowning the south building is an airfoil-shaped mechanical penthouse, a nod to the barrel-vaulted forms of the original Nu Skin tower.



▼ The steel-framed mechanical penthouse uses roof beams whose radii vary from 109 ft, 8 in. all the way down to 3 ft, 4 in.

▲ The translucent glass ceiling is hung from steel trusses, which are in turn suspended from the roof girders.

and bridges are seismically separated from the north building with an expansion joint. Therefore, the North and South Buildings are seismically separated with an expansion joint at the north side of the atrium. Lateral forces for the both buildings are resisted by shear walls, which minimize relative movement between the buildings during seismic events. Columns from the atrium roof rest on Teflon coated plates on the top of the concrete structure allowing them to move independently. The north-south spanning bridges use a similar detail where they meet the concrete structure of the north building. ■

Owner
Nu Skin Enterprises

General Contractor
Okland Construction

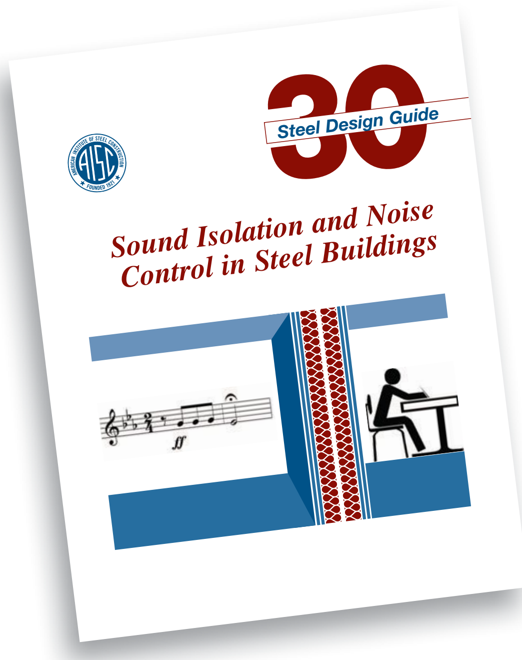
Architect
Bohlin Cywinski Jackson

Structural Engineer
Magnusson Klemencic Associates

Steel Team

Fabricator
Tech-Steel (AISC Certified Fabricator)

Detailer
Axis Steel Detailing, Inc. (AISC Detailer)



SOUND Judgment

BY BENJAMIN MARKHAM

A brief summary of AISC's new Design Guide 30: *Sound Isolation and Noise Control in Steel Buildings*.

SOUND MATTERS.

Research by AISC suggests that a key factor that influences what structural system to specify is acoustics. Regardless of which structural system is selected, good acoustical performance is a matter of good design. When it comes to acoustical design considerations in steel-framed projects, the new AISC *Design Guide 30: Sound Isolation and Noise Control in Steel Buildings* provides detailed guidance on how to optimize a building's design. Following is a brief overview of the guide.

Acoustical Roadmap

After a brief introductory chapter, the Design Guide begins in **Chapter 2** with a roadmap for addressing sound isolation and noise control. The process begins with a clear understanding of the acoustical goals in each space and of the potential noise sources that might disrupt those spaces. Once the acoustical goals and noise sources are known, the transmission paths are identified. This is where building design comes into play. The detailed design of floor/ceiling assemblies, walls, building envelope constructions and noise control design for mechanical systems are developed directly from the building's specific sound isolation and noise control criteria.

Benjamin Markham is director, Architectural Acoustics, with ACENTECH.

Before delving into specifics, the guide continues in **Chapter 3** with an introduction into the basics of sound in buildings. Terms like *sound pressure level* and *frequency* are defined, metrics like *NC ratings* and *reverberation time* are introduced and the basic tenets of human perception of sound in buildings are outlined. One of the most important themes of building acoustics (and perhaps the most commonly misunderstood) is discussed in some detail: *Absorbing* sound and *blocking* sound are not the same!

The balance of Design Guide 30 is organized according to the roadmap outlined in Chapter 2.

Chapters 4 and 5 outline the typical acoustical goals and criteria for projects. Chapter 4 focuses on room noise criteria, with references to key standards and other industry references throughout. It introduces the concept that privacy is related not just to the loudness of an intrusive sound, but rather the relationship between the intrusive sound level and the level of the prevailing ambient background noise. Chapter 5 introduces sound isolation criteria, such as those found in many state building codes for multifamily housing. Standard reference criteria for sound isolation in other building types are outlined as well, including offices, schools, healthcare facilities and courthouses.

Chapter 6 provides guidelines for estimating noise levels of common noise sources. Interior noises (like speech), exterior noises (like traffic), impact sound sources (like footfalls) and mechanical system noise (like chillers and boilers) are all discussed. A simple method for calculating the combined contribution of multiple sources is outlined and illustrated by an example calculation of noise from mechanical equipment in a room.

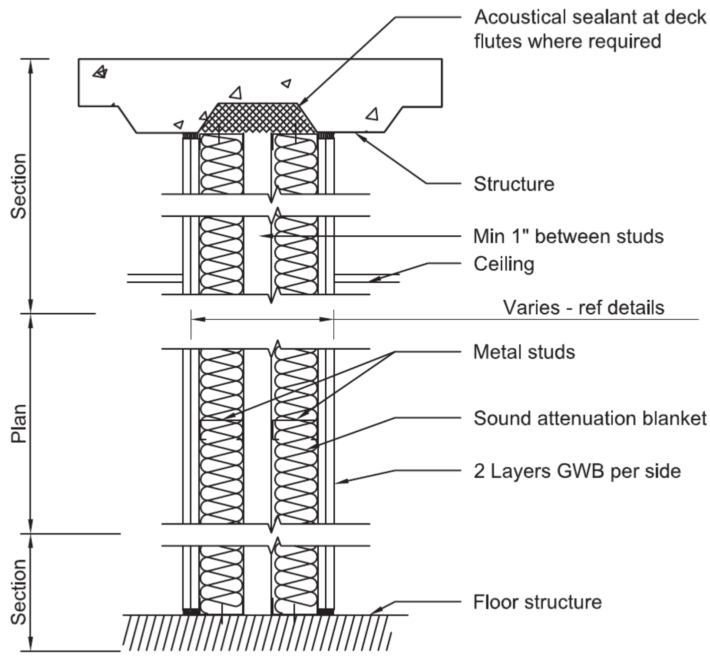
Chapters 7 and 8 get into the meat of the Design Guide and serve as a useful reference for designers working to detail specific sound-isolating assemblies in steel buildings. **Chapter 7** introduces basic concepts of airborne sound isolation. Sound transmission through simple and multi-component assemblies is introduced, and the effects of sound absorption in the receiving room on perceived sound isolation are described in detail. Section 7.2 goes on to introduce Sound Transmission Class (STC), the most common metric for sound isolation in the United States, as well as other relevant sound isolation metrics. Section 7.3 introduces basics of sound isolation improvement: mass, stiffness, damping, separation or decoupling, leaks and gaps and other building properties that affect sound isolation between spaces.

Chapter 8 covers building assemblies, providing information about sound transmission for wall assemblies, floor/ceiling assemblies, facades and roof structures. Section 8.3 continues with a discussion of impact sound transmission (primarily caused by footfall), and Section 8.4 details a range of floor/ceiling assembly designs found in steel buildings and developed to isolate increasing levels of impact sound. The chapter concludes with a brief discussion in Section 8.5 of "acoustical deck"—steel deck with a perforated bottom layer designed to incorporate a sound-absorbing finish into the structural deck system.

Finally, **Chapter 9** discusses mechanical noise and vibration control; indoor and outdoor mechanical noise criteria are introduced, and noise and vibration control methods for rooftop equipment and mechanical equipment rooms are outlined.

Whether for an office building, a residential tower, a school or a courthouse, steel buildings can provide excellent sound isolation and noise control for—and between—their occupants. This new resource for assisting in developing building details with good acoustics is now available now. AISC members can download it for free at **www.aisc.org/dg**. ■

Figure 1. An example partition detail for a double-stud demising wall.



Provide a continuous bead of acoustical sealant around ceiling and floor perimeters of partition.

A new prequalified connection for seismic moment frames features an energy-dissipating fuse.

SPECIALIZED Seismic Solution

BY STEVEN E. PRYOR, S.E., P.E.

A NEW TYPE of partial-strength steel moment resisting connection has been approved for inclusion in the upcoming ANSI/AISC 358-16 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*.

Developed by Simpson Strong-Tie specifically for seismic environments, it incorporates the company's patented Strong Frame Yield-Link structural fuse and is the first partial-strength steel moment frame connection to be qualified for use in special moment frame (SMF) systems. Merging a number of different technologies, the new field-bolted moment connection focuses seismic energy dissipation into bolt-on/bolt-off replaceable structural fuses facilitating resilience and rapid recovery after a seismic event. In doing so, the structural integrity of the beams and columns is maintained even in severe earthquakes.

Based on Capacity

At its core, the connection relies on capacity-based design. Unlike current SMF connection designs, the beam-to-column connection is designed to be the yielding hinge as opposed to the beam itself. Once the necessary connection strength and stiffness is determined, the beam, column and column panel zone are all designed to develop the full inelastic strength of the connection, while themselves experiencing little, if any, inelastic demand. Given that the beam is stronger than the connection, the connection becomes classified as a partially restrained connection.

Under factored LRFD load combinations (not including load combinations that consider overstrength) the connection is required to remain elastic. Like any other SMF connection, code drift limits and base shear strength requirements still apply. This means that while the connection itself may be classified as partially restrained, a building using these connections will have the same minimum lateral strength and stiffness as required for other SMF connections. Full design and detailing requirements can be found in ANSI/AISC 358-16, Chapter 12.

The Yield-Link uses a T-section bolted to both the column flange and the beam flange at both beam flanges, and it acts in both tension and compression. The stem of the T contains a reduced section to control axial stiffness, yield strength and tensile strength. A buckling restraint plate is placed over the yielding section to prevent buckling when in compression. The buckling restraint plate is bolted to the beam flange on either side of the reduced area, with the bolts passing through a spacer plate with the same thickness as the stem of the T-section. Beam shear is transferred to the column via a single-plate shear connection, which uses a combination of holes and slots to avoid moment transfer through the shear plate.

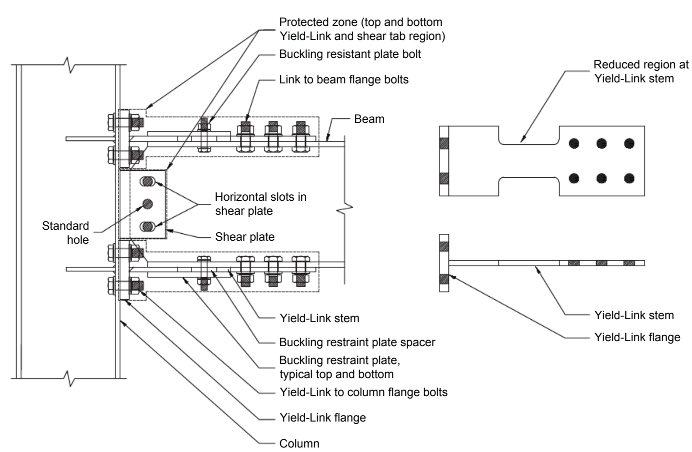
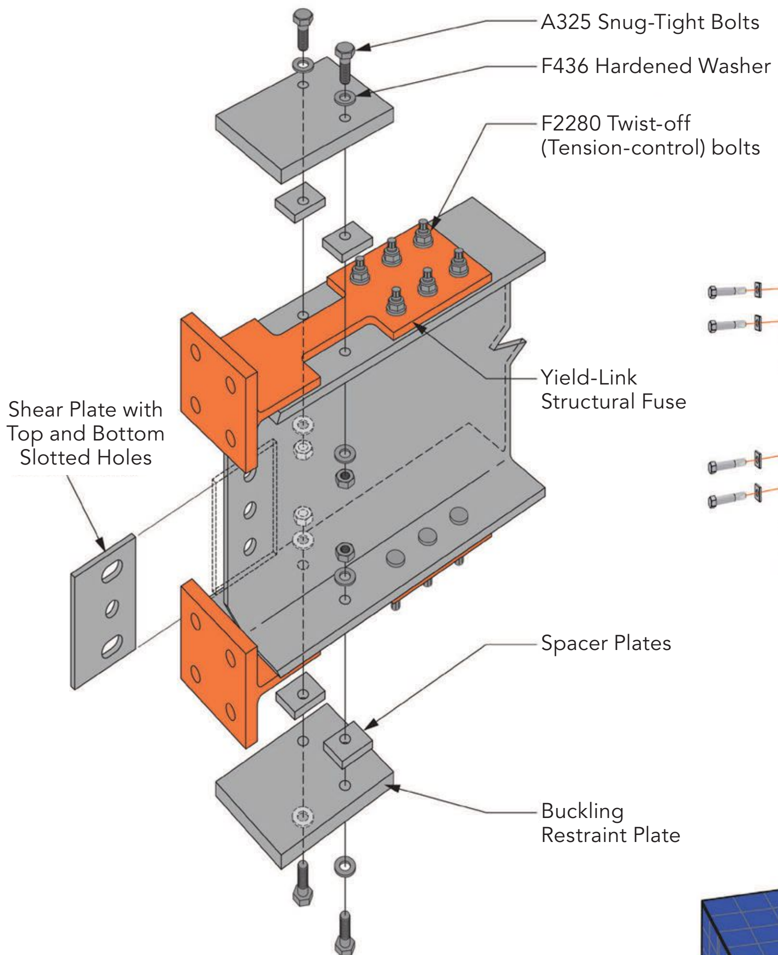
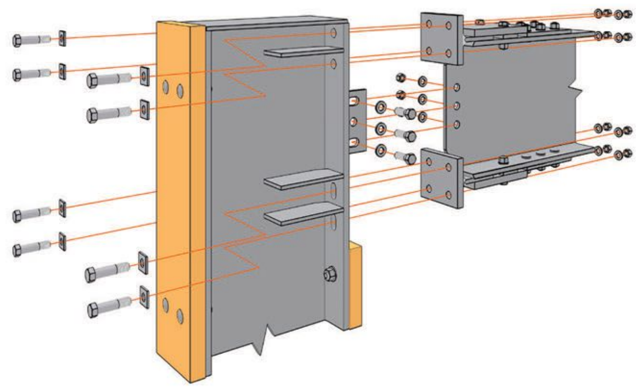


Figure 12.1 of 358-16. Simpson Strong-Tie Strong Frame moment connection.

Beam Bracing

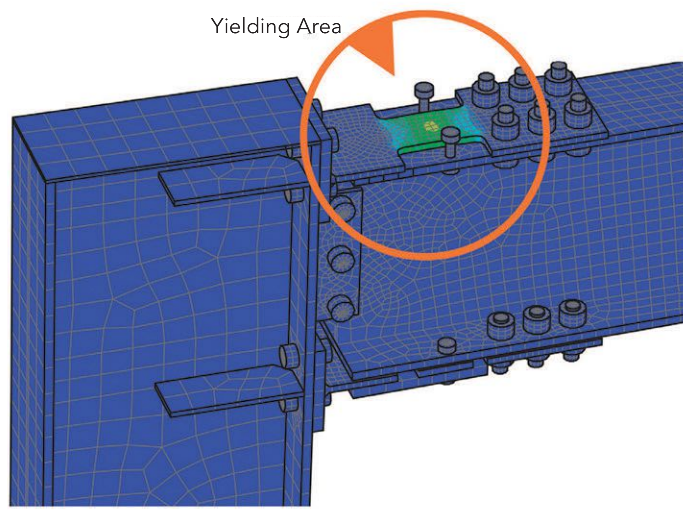
A primary reason for developing this new technology was to provide an SMF solution in situations where the surrounding structure (such as a wood structure) prevents the development of the required strength and stiffness in the beam bracing system required to prevent lateral-torsional bucking of the beam during seismic events. Because seismic inelastic demand is concentrated in the connection elements rather than the beam cross section, the beam remains essentially elastic and can be designed according to the AISC *Specification*. If one chooses, the beam can be selected so that for the span and loads under consideration it is adequate even without flange bracing. Addition-

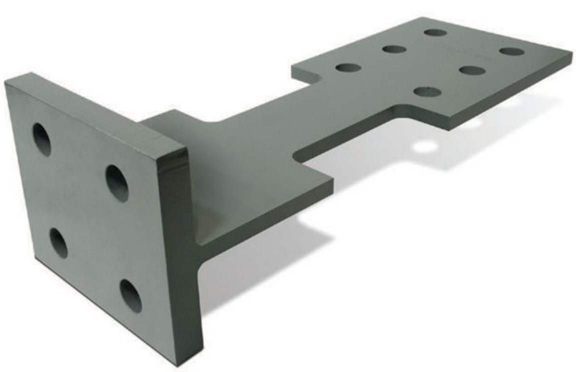




A 3D connection detail.

An installation diagram.



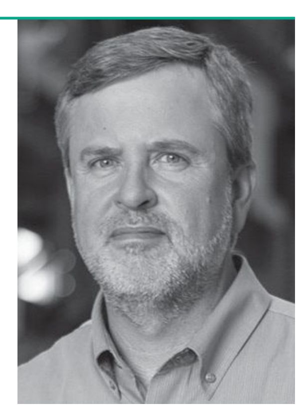


The highlighted green section illustrates the yield-link area on the Strong Frame special moment frame connection.

The yield-link itself.

ally, with the exception of a minimum flange thickness of 0.40 in. (10 mm) and the requirement that the flange width-to-thickness ratio not exceed λ_r, beam and flange width-to-thickness requirements are required to comply with the AISC *Specification* rather than the AISC *Seismic Provisions*. This is proving to be very useful for seismic retrofits in wood structures that want to use a high-performance SMF solution but otherwise cannot meet the requirements for both strength and stiffness in the flange bracing system, while the field-bolted nature of the construction avoids the potential fire hazards associated with field welding in an existing wood structure. Another area of potential use would be frame systems that would otherwise have to be designed as ordinary moment frames, such as those used in the metal building industry.

Steven E. Pryor
(**spryor@strongtie.com**)
is Simpson Strong-Tie's international director of building systems.



➤ The Strong Frame system.

Like other connections, there are limits to what has been prequalified:

- ➤ Yield-Link stem thickness of 0.50 in. (12.5 mm)
- ➤ Maximum width of the reduced yielding portion of the stem of 3.50 in. (89 mm)
- ➤ Buckling restraint plate thickness of 0.875 in. (22 mm)
- ➤ Snug-tight bolts are permitted at the Yield-Link-to-column and shear plate connection, and are required at the buckling restraint plate connection
- ➤ Fully pretensioned bolts are required at the Yield-Link-to-beam connection (no paint in the faying surface, but no special surface preparation)
- ➤ Maximum beam depth consistent with W16 profiles
- ➤ Maximum column depth consistent with W18 profiles
- ➤ Strong-axis connections only

Cyclic testing has been performed, which has successfully demonstrated that the connection meets or exceeds the requirements for SMF connections as required by the AISC *Seismic Provisions*. Additionally, full-scale shake table testing beyond the requirements of the *Seismic Provisions* has also demonstrated the outstanding performance of frames using the connection in conjunction with beams lacking flange bracing.

The connection is well suited to both new and retrofit applications, and with the ever-increasing awareness of the role that structural resiliency plays in our communities, the connection is ready to do its part in a new and unique way. ■

AISC 358-16 will be available to the public later this year. You can find out more about the Simpson Strong-Tie special moment frame at ***www.strongtie.com/smf****.*

Prequalified Connections

Below are some FAQs about how prequalified connections are addressed in AISC 358.

What is a prequalified moment connection?

Prequalified moment connections are structural steel moment connection configurations and details that have been reviewed by AISC's Connection Prequalification Review Panel (CPRP) and incorporated into the AISC 358 standard. The criteria for prequalification are spelled out in AISC's seismic provisions, AISC 341. In short, AISC 341 contains performance and testing requirements that have been shown to produce robust moment connections, and AISC 358 details connections that meet those criteria. Both standards are free downloads at **www.aisc.org/specifications**.

What's involved in prequalification?

Chapter K of AISC 341 describes parameters for test specimens as well as the testing requirements that all connections must satisfy. Test specimens must essentially be full scale and cover a range of shapes and sizes for which the connection is proposed. The test results as well as proposed design provisions are provided to the Connection Prequalification Review Panel for review and approval. The CPRP does not charge any fees for reviewing connections.

Full-scale testing is expensive, as you can imagine, and CPRP does not fund or perform testing itself. Various industry initiatives have funded tests for several non-proprietary (not patented) moment connections, and the developers of several proprietary (patented) moment connections have funded and performed their own tests for their respective connections.

Why does prequalification exist?

Before the 1994 Northridge (California) earthquake, most seismic moment connections were prescriptively designed welded flange connections with bolted single-plate web connections. After the quake, more damaged connections were observed than what was expected, which led to a program of moment connection testing funded by federal agencies, state associations and various trade groups, including AISC. The result of those tests was a series of seismic moment connection guidelines published by FEMA (the Federal Emergency Management Agency).

To integrate all of this knowledge into contemporary structural steel design—and to create a logical path for extending the industry's knowledge going forward—AISC adopted (and adapted) the FEMA guidelines into the AISC 358 standard in 2005. Over the past 10 years, AISC 358 has expanded as more structural steel moment connections for seismic applications have been studied and codified.

What if a moment connection configuration doesn't fit the parameters of a prequalified connection?

Each prequalified connection in AISC 358 contains the parameters for which the connection is considered prequalified. Connections that don't fall within the criteria for an existing prequalified connection would have to be tested according to the requirements of Section K2 in AISC 341. In practical terms, this means that some common sloped and skewed configurations (for example, sloping roof girders) do not fit the prequalification criteria, and engineering judgment is required.

news

CERTIFICATION

AISC to Issue New Certification Program Requirements

AISC is currently in the process of reviewing and addressing public comments that were submitted for its new Hydraulic Steel Structures Certification Program and updated Building Fabricator Certification Program. Both sets of program requirements will be published on March 31, 2016. On May 2, the Hydraulic Steel Structures program will begin, and the revisions to the Building Fabricator program will take effect for new applicants. Existing participants in the Building Fabricator program will transition to the new requirements over a two-year period.

The Hydraulic Steel Structures Program is the newest addition to AISC's current set of certification programs, which include building fabrication, bridge fabrication, component manufacturing and erection. AISC developed the new program and requirements in response to the needs of the United States Army Corps of Engineers.

"The use of a governing requirement document began with our completed and successful conversion of the bridge certification program from a checklist to the bridge requirements, which was followed by the conversion of our erector certification program," said Jacques Cattan, AISC vice president responsible for certification. "Our goal is to provide consistency, clarity and transparency to our program requirements and processes."

AISC also developed new requirements for its Building Fabricator Program. These requirements will reference the *Standard for Steel Building Structures–2006* (AISC 201-06). The standard describes the essential elements of the quality management system for structural steel building fabrication.

If you have questions regarding these programs, please contact AISC Certification at **certification@aisc.org** or 312.670.7520.

NASCC

World Steel Bridge Symposium Highlights Hot Bridge Topics at NASCC

The 2016 World Steel Bridge Symposium (WSBS) will bring 20 specialized sessions to NASCC: The Steel Conference, April 13–15 in Orlando. Sessions include such hot topics as accelerated bridge construction, corrosion protection solutions and advanced analysis techniques for design and erection.

The Symposium, held every other year in conjunction with The Steel Conference, convenes bridge design engineers, construction professionals, academicians, transportation officials, fabricators, erectors and constructors to discuss and learn state-of-the-art practices for enhancing steel bridge design, fabrication and construction techniques. All WSBS sessions are included in Steel Conference registration.

The Steel Conference offers dynamic, expert-led sessions that focus on structural engineering, steel fabrication, erection and detailing. Unlike other conferences that issue a general call for papers, The Steel Conference carefully selects topics of interest and then seeks out the top experts and presenters. Some of the presenters are very well known, while others may not be household names but still bring a distinct expertise to the program. Speakers range from AISC's Charlie Carter on "What's New with the 2016 Code of Standard Practice" to Cives Steel's Patrick Fortney on "Design of Stability Connections for Beams Used in Steel Seismic Frames."

The conference also offers an extensive trade show (featuring products ranging from structural software to machinery for cutting steel beams). It's a once-a-year opportunity to learn the latest trends, see the most innovative products and network with your peers and clients. And one low registration fee gains you admittance to technical sessions, the keynote address, the T.R. Higgins Lecture and the exhibition hall.

For more information and to register for The Steel Conference/WSBS, go to **www.aisc.org/nascc**.

People and Firms

- **SidePlate**, an AISC Member based in Mission Viejo, Calif., recently announced that three full-scale seismic tests have been successfully carried out by the University of San Diego on the company's Bolted connection, a field-bolted moment frame that debuted in 2013. With the success of these new tests, the connection meets the Special Moment Frame (SMF) requirements in the ANSI/AISC 341-10 specification, *Seismic Provisions for Structural Steel Buildings*. Two more tests are planned in the near future.

  "Our 20+ years of high-seismic research has guided us through the development of the SidePlate Bolted connection, which now allows contractors to quickly erect structural steel on projects in any design criteria," said Henry Gallart, SidePlate's president.

  The connection requires no field welding. It has already proven to be a success in low-seismic areas, where contractors and erectors have commented that they are saving time in the field because the connection can be quickly assembled. Using the positive results of these recent tests, project teams in high-seismic areas can now also use the connection and experience the same benefits.

  All of SidePlate's designs are implemented with the help of structural engineers who provide assistance and customer service at no charge to the design team. Visit **www.sideplate.com** to learn more.
- **Gary Jaster, P.E.**, founding partner and principal of structural and civil engineering firm JQ, announced today that **Stephen H. Lucy, P.E.** has been named the firm's CEO. Jaster will continue to serve in an advisory capacity during the leadership transition period.

BOARD OF DIRECTORS

AISC Elects New Board Chair, Vice Chair and Directors

AISC is pleased to announce the election of James G. Thompson, president of Palmer Steel Supplies, Inc., McAllen, Texas, as the new chair of its board of directors, and David Zalesne, president of Owen Steel Company, Inc., Columbia, S.C., as the new vice chair. Both were elected to two-year terms during AISC's annual meeting in Chicago late last year. In addition, the board welcomes two new directors: Hollie Noveletsky, CEO of Novel Iron Works, Inc., Greenland, N.H., and Robert A. Simon, vice president of structural products at Steel Dynamics, Inc., Columbia City, Ind.

Thompson previously served as board vice chair and succeeds Jeffrey E. Dave, P.E., president and CEO of Dave Steel Company, Inc., Asheville, N.C., as chair. Dave, who will continue to serve on the board, commented, "My two years as AISC chair have been very rewarding, and I'm very excited about the future of AISC. The board and our membership have elected two highly qualified people to lead our industry. Jim Thompson's work as vice chair, along with his industry experience has readied him for the responsibility. And our new vice chair, David Zalesne, brings the demonstrated experience, commitment, knowledge and leadership to carry on the past two years' success and build on them with new vision. In addition to his general industry knowledge, David has been and will continue to be a strong advocate for our industry engagement with important issues in Washington D.C."

Thompson brings to the position more than 40 years of experience in steel fabrication and erection. His expertise includes sales, estimating, production management, operations management and administration management. He grew up in numerous locations in the U.S. and Europe and graduated from Texas Christian University (TCU) in Fort Worth with a bachelor's degree in mathematics. While at TCU he was enrolled in the Reserve Officers' Training Corps (ROTC) and at graduation was commissioned a 2nd lieutenant in the U.S. Air Force. After serving five years as a pilot, he left the service in 1974 and immediately went to work at Palmer Steel Supplies. He began as a steel detailer and ascended to general manager after several years in management training. In 1984 he became president of the company and obtained ownership several years later. He has also served the McAllen community as a director on several local boards. In 2007 he joined the AISC board of directors and has served on various committees.

Zalesne has served as an AISC board member for 10 years and is also an appointed member of the Industry Trade Advisory Committee for the steel industry. Prior to becoming president of Owen Steel Company in 2004, he practiced law as a partner in the Litigation Department of Klehr, Harrison in Philadelphia, and also served as an assistant U.S. attorney in the eastern district of Pennsylvania. He has also served on the boards of several local business and community organizations. He received a bachelor's degree in international relations and finance from the University of Pennsylvania, and his law degree from Emory University School of Law.

Noveletsky and Simon will immediately begin serving on the board, assisting with the AISC's planning and leadership in the steel construction industry. "Two industry leaders have been elected to the board of directors," added Dave. "Hollie Noveletsky brings great overall knowledge of the steel fabricating industry and community leadership, and the election of Robert Simon continues a commitment by Steel Dynamics to help shape the future of our industry."

Noveletsky started working at Novel Iron Works while enrolled at Lawrence University, where she began studying engineering before switching her path of study to nursing. She worked summers in Novel's computer department and then in the estimating department while in graduate school at Boston University. She also holds a post-master's certificate in psychiatric nursing from MGH Institute of Health Professions, and a doctorate of philosophy in nursing research from Boston College. In 1999 she stepped into the company full-time after her father, who was president of Novel, passed away. She also joined the Steel Fabricators of New England (SFNE) and eventually became president, where she spearheaded a change of the organizational structure. Additionally, she maintains her own geriatric psychiatry practice and also volunteers with various natural disaster relief efforts.

As vice president for Steel Dynamics, Simon is responsible for the structural and rail division in Columbia City, Ind., as well as for Steel of West Virginia in Huntington. He joined Steel Dynamics in 2013 following a 20-year career with EVRAZ North America. He is a former chairman of the Steel Manufacturers Association and has also served on numerous boards for the communities he's been part of, most recently the Fort Wayne Philharmonic board of directors. He received his bachelor's degree in industrial engineering from West Virginia University.

Thompson

Zalesne

Noveletsky

Simon

QMC *Contract Auditor*

Quality Management Company, LLC is seeking contractors to conduct audits for the AISC Certified Fabricator and AISC Certified Erector Programs. Contractors must have knowledge of quality management practices as well as knowledge of audit principles, practices and techniques and knowledge of the steel construction industry. If you are interested, please submit your statement of interest **contractor@qmconline.org**.

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President

American Institute of Steel Construction

With the upcoming retirement of our President, the American Institute of Steel Construction is seeking a dynamic leader to work with members, Board of Directors and staff to create and advance a strategic plan to accomplish the Institute's mission of increasing steel's share of the construction market. The President has responsibility for all AISC departments and activities and must have a track record of success in managing a complex and expanding organization while maintaining its financial stability and revenue growth. The position involves substantial travel to meet with members, attend meetings and conferences, make presentations and serve as the lead spokesperson for AISC and the fabricated structural steel industry. The President must be able to continue fostering a culture that motivates high performance across the organization and drives our high-energy, aggressive, technical and marketing teams. Technical and financial expertise required. Familiarity with AISC, fabrication and steel industry knowledge are a must—but equally critical is a vision for the future of the Institute. Civil/structural engineering degree and/or MBA strongly preferred.

For consideration, e-mail your resume to **aiscpres@gmail.com**.

Vice President of Finance & Administration

The American Institute of Steel Construction (AISC) is looking for an experienced finance executive to join our Senior Management team and participate in the development of the strategic plans supporting our mission and goals. The Vice President of Finance and Administration reports to the President of AISC, and acts as lead spokesperson to the AISC Board of Directors for activities related to finance, business administration, and information systems.

This role provides participative leadership, financial management, strategic management, and direct hands-on help for finance, accounting, information systems, facilities and risk management activities in support of AISC's operations.

To qualify, you must have a Bachelor's degree in Finance, Accounting or Business Administration, MBA and/or CPA strongly preferred. Minimum 10 years of experience in a senior management role with responsibility for finance, accounting, facilities administration, and information systems.

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LINCOLN LINK

Louisville-Southern Indiana Ohio River Bridges Project

INTERSTATE 65 has a new route for crossing the Ohio River.

The new Abraham Lincoln Bridge opened to traffic in December. Designed by engineer Buckland and Taylor and fabricated by Prospect Steel (AISC/NSBA Member/Certified fabricator), with Walsh Construction acting as the general contractor, the 2,100-ft-long, 100-ft-wide cable-stayed bridge connects Louisville, Ky., and Jeffersonville, Ind., across the Ohio River and carries three lanes of traffic.

The cable-stayed portion of the new bridge uses 6,000 tons of structural steel in all. About 800 ironworkers worked on the crossing and assembled more than 650 pieces of steel, which required about 76,000 bolts. Horizontal steel plate girders form the foundation of the bridge deck. The edge girders were approximately 45 ft long and 6 ft deep, and the floor beams were approximately 98 ft long with a center depth of 5 ft, 10 in. There were a total of 94 heavy plate girders, weighing a combined 2,092 tons. Additionally, the steel cables that extend from the three towers to the deck of the bridge required 1.4 million ft of steel strand, enough to stretch from Louisville to Chicago.

Crews set the first piece of structural steel for the new bridge in October 2014. The last piece of steel was set one year later. ■

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