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January 2016

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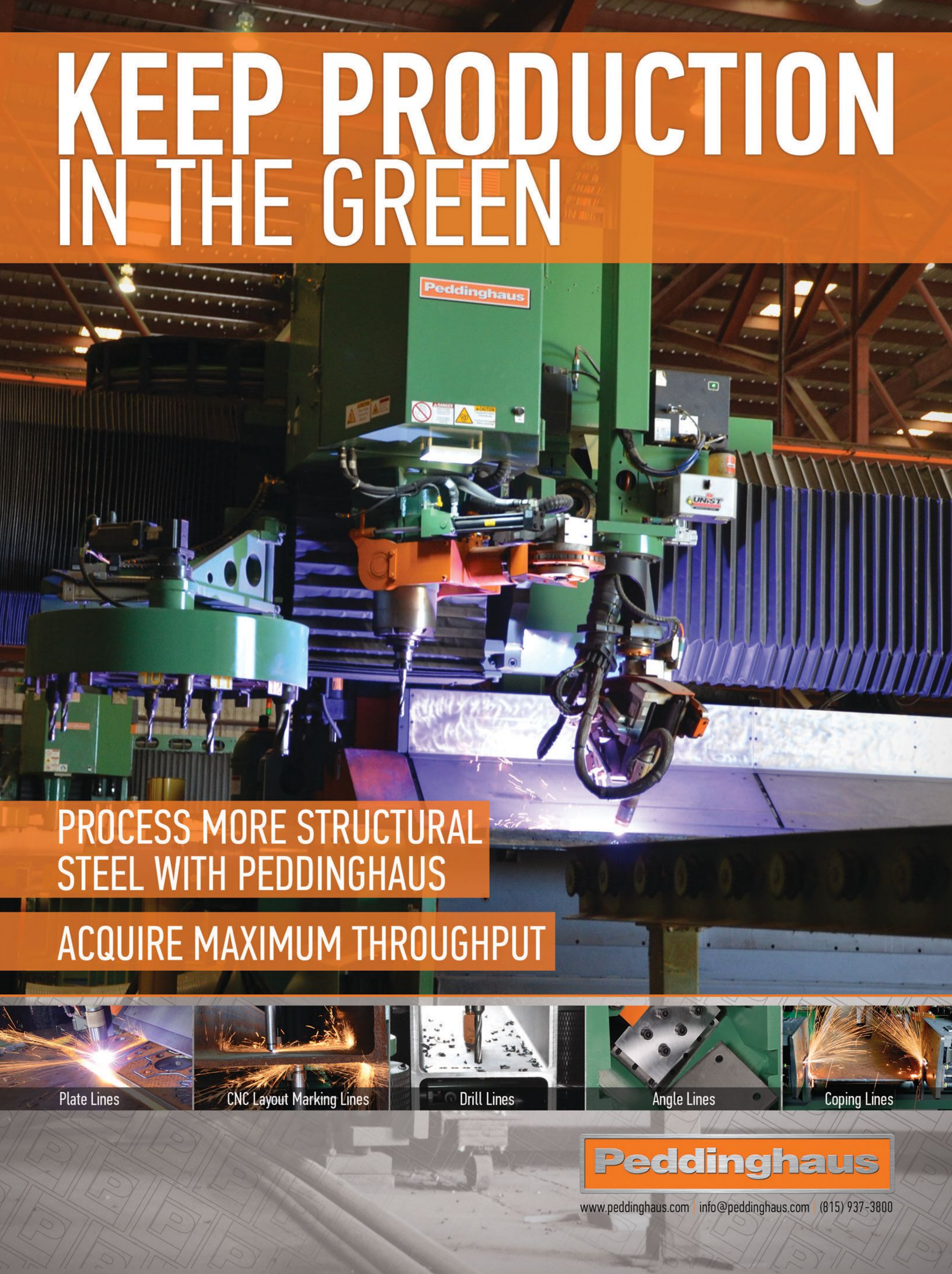
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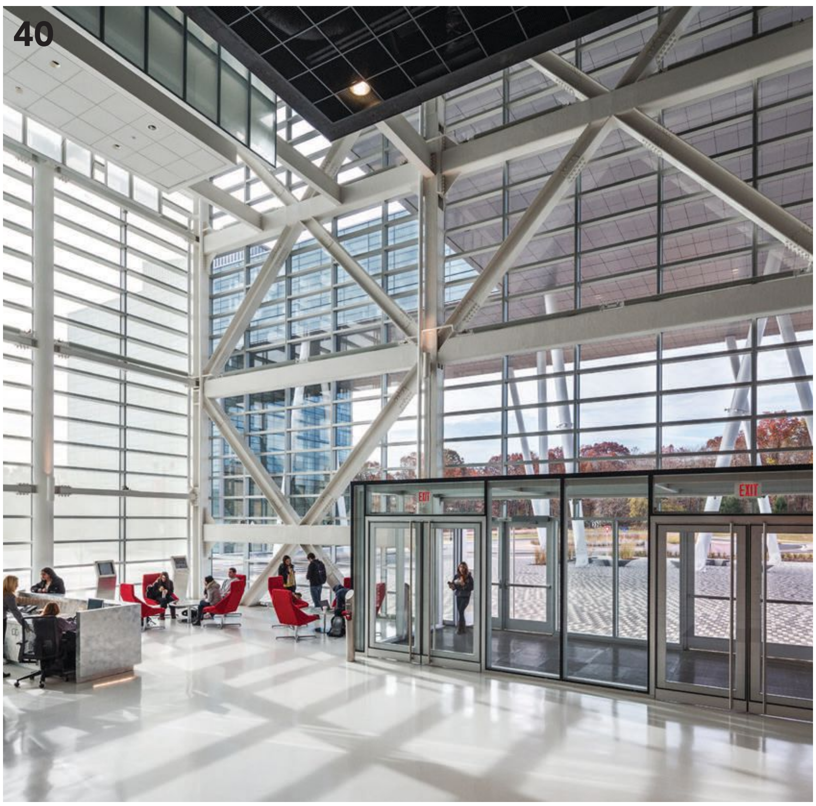

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ON THE COVER: X marks the spot at Dickinson College's new sports facility, page 24. (Photo: CannonDesign/Scott Frances)

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editor's note

A FUN FACEBOOK FEATURE IS "ON THIS DAY," WHICH SHOWS YOU AND YOUR FRIENDS SOME OF YOUR MEMORIES (SUCH AS PICTURES AND POSTS) FROM THE PAST. Most of the memories are just a year or two old, but occasionally one will pop up from five or even seven years ago.

Not surprisingly, many of my memories are pictures of my children. And I'm often surprised when I see these old pictures by how little my kids were and how grown up they are now. For me, the changes have been so gradual that I don't always notice them until I see these old pictures.

I was reminded recently that it's not just people who change over time. Renae Gurthet, who has been in charge of the exhibit hall at NASCC: The Steel Conference for the past decade, recently sent me a note about how much the conference had grown in that time frame. While to me the change was a gradual evolution, the numbers tell a different story. In the past decade attendance and exhibitors have more than doubled. Last year we had more than 4,500 visitors (mostly structural engineers and steel fabricators but also erectors, detailers, educators, students, exhibitors and contractors).

The Steel Conference has also changed in tone. If you go back a quarter-century or more, it was a sleepy event with a strong academic tilt. Now, it's a vibrant three days where you can hear about "Elasto-Plastic Stress States and Reduced Flexural Stiffness of Steel Beam Columns" but also learn about "Proposals that Win," "Advances in Welding Automation" and "Advances in Steel Connection Analysis."

You can participate in seminars with up-and-coming professors at leading universities and with some of the best-known structural engineers in the world. The conference is all about learning and networking. It's a great place to meet your colleagues, your customers, your clients and future friends (amazingly, I even know one since-married couple who first met at a Steel Conference).

You'll learn about the latest design information and you'll see the latest advances in both fabrication equipment and structural design software.

The Steel Conference, which will be held in Orlando, April 13–15, offers more than 120 fabulous sessions, including the World Steel Bridge Symposium. And there are more than 200 amazing exhibitors. You can see steel plate cut as fast as butter or try your hand at a welding simulator. Whether you're looking for software or bolts, if it's related to structural steel design and construction, it's at the Steel Conference.

If you've never attended a Steel Conference, ask around. It's a stimulating three days that will teach you and invigorate you. Our goal is for every attendee to come away with ideas they can put into practice immediately.

Visit **www.aisc.org/nascc** to see the full program. And make sure you register early. When registration opens on January 4, AISC members can register for just $340; the fee increases $10 every week.

I hope to see you in Orlando!

SCOTT MELNICK
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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

Extended Gages

Part 10 of the AISC *Manual* contains a discussion entitled "Eccentric Effect of Extended Gages." The discussion seems to indicate that the engineer can assume the moment due to the eccentricity exists either at the bolt group or at the weld. I have two questions.

1. **If the moment is assumed at the bolt group, can the supporting member and the welds be designed to resist only the vertical shear?**
2. **If the moment is assumed at the weld, then the supporting girder must be designed to resist torsion. An open section, like a wide-flange girder, is torsionally quite weak, and designing it to resist torsions will result in a heavy, uneconomical section. Isn't it prudent to always design the connection, rather the support, to resist the moment?**

Though the second question specifically mentions a beam/girder as the support, the answers below consider conditions with both supporting beams and columns.

1. Yes, but ductility must also be considered. The design procedures for single-plate shear connections assume the model described in the *Manual* discussion: eccentricity on the bolts and shear on the weld. However, the procedure then ensures that the bolts and welds are stronger than the plate to accommodate end rotations. Section B3.6a of the *Specification* states: "A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure." The design procedures for single-plate shear connections implicitly target an end rotation of about 0.03 radians and have been shown to accommodate such rotations through physical tests. An end rotation of 0.03 radians is relatively large and probably unrealistically high for a serviceable beam. You do not necessarily have to satisfy the design procedures shown in the *Manual*, but you must consider ductility and the effects of end rotation.
2. No, not always. The argument makes sense for connections to open-section beams that have framing to one side only, but may not make sense for other girder conditions (or connection to columns, where it is a moment and not a torsion).

When framing to a column flange, it is probably more economical to upsize the column to account for the eccentricity than it would be to account for the eccentricity in the connection. This is especially true for vertical brace connections. The additional cost of material in the column will usually be less expensive than the alternative of more and larger bolts and larger welds, since labor costs are typically higher than material costs. However, in my experience, it is rare for the moment to be resisted by the support, even when it is clear that doing so will result in a more economical design. There are a number of reasons for this:

- Columns other than those in moment frames have historically been designed as axial members, and engineers are sometimes reluctant to break with tradition.
- The design models are simpler if the column is assumed to resist only axial loads. However, this reason may be fading, as engineers routinely use computer analysis and design programs. Many of these programs can (and may, by default) include some eccentricity in the columns. In such cases, the moment is often accounted for twice.
- Owners, architects and general contractors tend to judge an engineer's performance based on indirect and inaccurate measures like weight per square foot. In effect, the individuals charged most directly with maintaining the economy of the project sometimes incentivize practices that increase the overall structural costs. Decades of efforts on the part of AISC and others have failed to change this.

For a girder support, an open section is much less efficient in torsion, so the weight penalty would be greater. Another consideration is that the connection would somehow have to transfer the moment into the girder. Since the web of the beam is also inefficient in weak-axis flexure, it might be beneficial to attach the connection to the flanges of the girder. This would involve additional labor and materials.

When connecting to a column web, the gage may be extended to get the bolted connection beyond the flanges to make erection more efficient. It might seem that designing the column for the resulting moment is inherently more efficient than adding bolts. But again, the moment must somehow get through the inherently inefficient web to the flanges of the column. Like at the girder, this will typically involve additional labor and materials.

A further consideration is that the additional labor and materials related to transferring the moment from the web of the support to the flanges will be performed in the shop, whereas additional bolts will be installed in the field.

So in my opinion:

- If you are framing to a column flange, in most cases it is probably more efficient to upsize the column than to resist the eccentricity at the bolts.

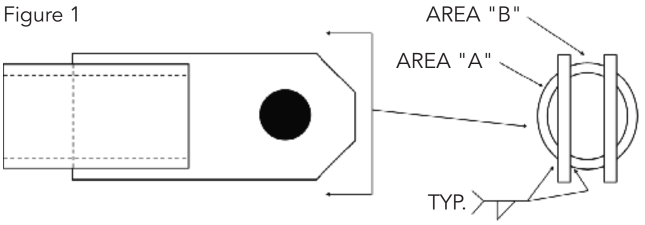
- If you are framing to a column web, in most cases I suspect it still may be more efficient to upsize the column than to resist the eccentricity at the bolts. However, a little more consideration is probably warranted and it is likely to be much more conditional.
- If you are framing to a girder, I suspect it is more efficient to resist the eccentricity at the bolts than to upsize the girder.
- Note that ductility needs to be considered regardless of the model assumed.

Larry S. Muir, P.E.

Double-Slotted HSS Connections

Two plates are slotted into and welded to a round HSS tension member, as shown in Figure 1. Since I am connecting more of the section than is assumed in Case 5 of Table D3.1 in the AISC *Specification*, I assume it would be conservative to check tensile rupture using an effective area based on that case. Is this correct?

Figure 1



Probably not.

This condition falls outside the scope of Table D3.1; attaching more of the section does not mean that more of the section is effective—i.e., Case 5 is not conservative. However, you are not merely seeking to satisfy an equation, or a set of equations, in a vacuum, but rather are trying to design a real structure to resist actual loads.

Case 5 is pretty simple: Each pair of welds delivers half of the load to half of the section. Your configuration is more complex and deserves more consideration. If you simply applied the checks one would typically apply to a connection similar to Case 5, you might end up using inconsistent models to design the various elements in the connection. This will be illustrated for the condition under consideration.

The following assumptions are made to simplify the discussion:

1. The length of the welds is assumed to be greater than 1.3 times the diameter of the HSS.
2. The full net area is effective. This is consistent with your assertion that Case 5 of Table D3.1 can be conservatively applied.
3. The required strength is assumed equal to the available net section strength of the HSS.
4. The net area is assumed to be 4.75 sq. in. and the tensile strength of the HSS is 58 ksi. This is done simply to so that we can work with numbers instead of variables or percentages.

From this, the available net section strength is (0.75)(58 ksi)(4.75 in.^{2}) = 207 kips. A typical approach would be to assume each of the eight fillet welds transfers one-eighth of the load, which is 25.9 kips.

Load is delivered by two welds to AREA "A". Therefore AREA "A" must resist one-quarter of the load, which is 51.8 kips. It can be seen that AREA "A" is roughly one-third of the net area, 4.75/3 = 1.58 in.^{2} The rupture strength of this section is (0.75)(58 ksi)(1.58 in.^{2}) = 68.7 kips, which is greater than the 51.8 kips assumed to be delivered by the welds.

However, an analysis of AREA "B" turns out differently. The load is also delivered by two welds to AREA "B". Therefore AREA "B" must also resist one-quarter of the load, 51.8 kips. It can be seen that AREA "B" is roughly one-sixth of the net area, 4.75/6 = 0.792 in.^{2}. The rupture strength of this section is (0.75)(58 ksi)(0.792 in.^{2}) = 34.5 kips, which is less than the 51.8 kips assumed to be delivered by the welds.

The assumption that the HSS is uniformly loaded is inconsistent with the assumption that welds are uniformly loaded. A uniform distribution among the welds optimizes the welds. A uniform distribution within the HSS optimizes the HSS. Both cannot be optimized simultaneously with the given geometry.

There are probably a number of ways to analyze this condition, but ultimately you must choose a model and then check each element based on the loads from the chosen model.

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 at AISC.

steel quiz

This month's Steel Quiz takes a look at the recommended design procedure for column base plates subjected to axial compression, as covered in Part 14 of the AISC 14th Edition *Steel Construction Manual*.

1 Consider the base plate configuration shown in Figure 1. Match the following four expressions based on Equation 14-7 of the AISC 14th Edition *Steel Construction Manual* with the correct limit states shown in Figure 2 through 5.

$t_{min} = m\sqrt{\dfrac{2P_u}{0.9F_yBN}}$ checks the flexural strength of the shaded portion of the base plate shown in Figure ______.

$t_{min} = n\sqrt{\dfrac{2P_u}{0.9F_yBN}}$ checks the flexural strength of the shaded portion of the base plate shown in Figure ______.

$t_{min} = \lambda n'\sqrt{\dfrac{2P_u}{0.9F_yBN}}$, where $\lambda = 1$, checks the flexural strength of the shaded portion of the base plate shown in Figure _____.

$t_{min} = \lambda n'\sqrt{\dfrac{2P_u}{0.9F_yBN}}$, where $\lambda < 1$, checks the flexural strength of the shaded portion of the base plate shown in Figure _____.

Figure 1

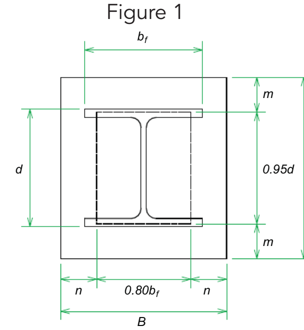


Figure 2

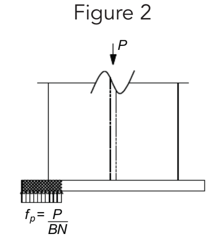


Figure 3

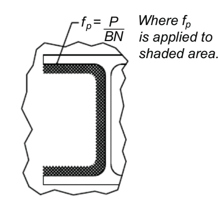


Figure 4

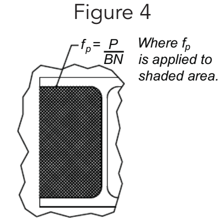
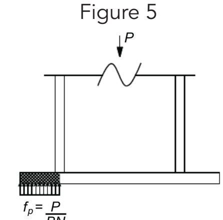


Figure 5



2 True or False: λ used in Equation 14-7 in the AISC 14th Edition *Steel Construction Manual* can always be taken conservatively as 1.

3 True or False: The most economical base plate thickness usually occurs when *m* and *n*, shown in Figure 1, are equal.

4 Derive the formula shown in Equation 14-7a (LRFD) provided in the AISC 14th Edition *Steel Construction Manual* (also shown below).

$$t_{min} = l\sqrt{\frac{2P_u}{0.9F_yBN}} \qquad (14\text{-}7a)$$

TURN TO PAGE 14 FOR ANSWERS

steel quiz

ANSWERS

1 Figure 5, Figure 2, Figure 4, Figure 3.

2 True. λ helps identify whether a column base plate is lightly loaded or not. If so, the base plate material within the hatched line area shown in Figure 1 can be checked as shown in Figure 3. If $\lambda = 1$, then this same area is checked using the approach shown in Figure 4. Conservatively setting $\lambda = 1$ means that you are always checking the base plate, even those that would qualify as lightly loaded, using the approach shown in Figure 4, which would yield a conservative base plate thickness relative to the thickness obtained using the approach shown in Figure 3 for lightly loaded columns.

3 True. Setting *m* and *n* equal to one another will generate the same flexural demand on the base plate for the cases shown in Figures 2 and 5.

4 The equation for a cantilevered beam that is uniformly loaded, per unit width, is equal to:

$$M_u = f_p \frac{l^2}{2} = \frac{P_u}{BN}\frac{l^2}{2}$$

The available flexural strength of a plate, per unit width, based on the yielding limit state (see Section F11 in the AISC *Specification*) is:

$$\phi M_n = \phi F_y Z = 0.9 F_y \frac{t^2}{4}$$

Set $M_u = \phi M_n$ and solve for t. The resulting value is t_{min}.

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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SPECIFY WITH CARE

When—and when not—to specify slip-critical connections.

BY CLARE TERPSTRA
AND LEIGH ARBER, S.E., P.E.

LET'S FACE IT. Slip-critical connections are specified too often.

It is important to understand how slip-critical connections behave and when they are actually required in order to avoid unnecessary fabrication and erection costs.

Transferring Shear

What is a slip-critical connection? It's one that transmits shear via friction between the faying surfaces. This is in contrast with bearing-type connections, in which bolt shear and bearing are responsible for transferring shear force (see Figure 1). Slip-critical connections are required to have a minimum amount of tension in the bolt, called "pretension," which creates a normal force between the connected elements. This normal force results in friction between the two surfaces in contact, which is utilized to resist shearing forces. As with any frictional force, the amount of force that can be transmitted is primarily a function of the amount of pretension and the slip surface. This is reflected in the available strength equation for slip-critical connections, from AISC *Specification* Section J3.8:

$$R_n = \mu D_u h_f T_b n_s$$

The essence of the equation is a coefficient of friction, μ, multiplied by a normal force equal to the bolt pretension. AISC *Specification* Section J3.8 gives further information on this equation. AISC Design Guide 17: *High Strength Bolts – A Primer for Engineers* by Geoffrey Kulak is a good reference on bolted connections.

Slip-critical connection required	Slip-critical connection NOT required
➤ Joints that use oversized holes ➤ Joints that are subject to fatigue load with reversal of the loading direction ➤ Joints that use slotted holes with the applied load parallel to the direction of the slot ➤ Joints in which slip at the faying surfaces would be detrimental to the performance of the structure (e.g., sensitive machinery) ➤ Joints that use partial-length cover plates per AISC *Specification* Section F13.3	➤ Just because bolts are pretensioned ➤ Just because tension-control bolts are used ➤ Just because slotted holes are used ➤ Just because bolts are part of the seismic force resisting system ➤ The bolted web connection of a direct welded moment connection ➤ Vertical and horizontal bracing connections ➤ Joints subject to fatigue without reversal of the loading direction ➤ When bolts see only tension and not shear ➤ Other cases not specifically listed by RCSC and AISC

Because slip can occur when designing a slip-critical connection, the strength of the connection in bearing must also be checked. The bearing strength does not typically control but still must be checked to ensure that the connection is adequate if the bolts were to slip into bearing.

When should a slip-critical connection be specified?

The use of slip-critical connections should be carefully considered. It is estimated that a slip-critical connection costs about three times as much as a snug-tightened, bearing-type connection. The factors that increase cost include lower strength per bolt, surface preparation requirements to achieve the required slip coefficient, and more extensive bolt installation and inspection requirements.

The most common reason slip-critical connections are required is to limit the structural deformations possible when using oversized holes. The adjacent table outlines this case and others when slip-critical connections are required. It includes requirements from Section 4.3 of the RCSC *Specification for Structural Joints using High-Strength Bolts* as well as requirements in the AISC *Specification*.

Strength

As has already been stated, slip-critical connections resist shear through friction at the faying surfaces. Calculating a strength per bolt can be misleading from a theoretical standpoint, but it is convenient both in practical design and when making comparisons between slip-critical and bearing connections.

Table 7-1 of the AISC *Manual* provides the per bolt strength for bearing joints. The strength for a $\frac{7}{8}$-in.-diameter A325 X-type bolt is 30.7 kips (LRFD). Table 7-3 provides the per bolt strength for slip-critical joints. The strength for a $\frac{7}{8}$-in.-diam-

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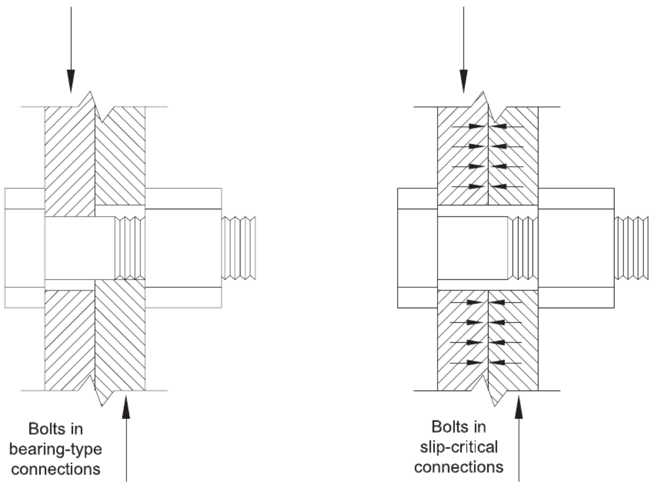

eter A325 with oversize holes and a Class A faying surface is 11.2 kips (LRFD). Depending on which connection limit state governs, these values show that a slip-critical connection may require more than twice as many bolts as a bearing-type connection to resist the same force. This will generally mean that the slip-critical connection will be larger, requiring more material, but the material cost is a secondary concern. The labor costs are the primary concern, and costs become nonlinearly more expensive in cases where more bolts don't fit in the member. As an example, a W21 that works with four bolts in bearing but needs five for slip resistance and must have an extension welded to the bottom flange and flanges coped away to fit the extra bolt. The larger joints can also mean more potential for interfering with other elements, such as mechanical and architectural components.

Combined Shear and Tension

Applied tension in a slip-critical joint is handled differently than in a bearing-type joint, as discussed in the *Engineering Journal* article "Prying Action for Slip-Critical Connections with Bolt Tension and Shear Interaction" (third quarter 2012, available at **www.aisc.org/ej**). In a bolted joint with no pretension, the entire applied tension is transferred to the bolts immediately. In a bolted joint with pretension, some of the applied tension will overcome the pretension in the bolt. Because the compression between the faying surfaces is reduced, the friction force that resists shear is also reduced. For this reason, the shear strength of bolts in slip-critical connections is reduced when there is tension present, per the procedure in AISC *Specification* Section J3.9. This is opposite from bearing-type connections, where the bolt tensile strength is reduced due to the effect of shear forces. This means that in joints that see tension as well as shear, slip-critical connections become even more uneconomical.

Faying Surface Preparation

In order to guarantee adequate friction, the surfaces between the plies of a slip-critical connection are specially prepared, which significantly increases the fabrication cost. Class A surfaces require relatively little prep and therefore have much more variability than Class B surfaces. Class B surfaces must be blast cleaned or blast cleaned *and* coated with an SC-qualified paint. This provides a more predictable surface with a higher slip resistance, but at an additional cost. Routine (non-qualified) paint systems can be used if the faying surfaces are masked, but this adds cost and also there may be a need to clean overspray from faying surfaces.

The requirements for Class A surfaces are less demanding, and can be satisfied by clean mill scale. There also are blast cleaned surfaces with Class A coating that will have similar cost implications to Class B coated surfaces and Class A roughened hot-dip galvanized surfaces. Galvanized faying surfaces must be roughened with hand wire brushing, because power wire brushing tends to polish the surface. Hand-wire brushing is time consuming, but it is also a logistical problem. The beam to be galvanized already has left the fabricator's shop, so the fabricator cannot do it. Typically the erector is responsible, but this involves additional cost in the erection contract.

Pretensioning

Bolts in slip-critical connections require a specified amount of pretension, as given in AISC *Specification* Table J3.1. There are several methods that can be used to apply adequate bolt pretension, but the associated installation requirements for all methods add cost relative to a typical bearing-type connection.

All pretensioning methods begin with bringing the bolts into the snug-tightened condition; the plies are drawn into firm contact to meet the requirements in the RCSC *Specification*. Thereafter, one of the four pretensioning methods provided by RCSC is used:

- Tension-control bolts
- Direct-tension indicator washers
- Turn-of-nut tightening
- Calibrated wrench tightening

All of these methods start with the snug-tightened condition, and so it is easy to see why these extra steps add cost. When the joint could be snug-tightened and not slip-critical (or pretensioned, for that matter), that cost is an unnecessary addition. To say nothing of costs to settle disagreements that might result about the inspection results!

Slip-critical connections involve a lot more work in both fabrication and erection. When specifying a slip-critical joint, additional costs must be considered for surface preparation, pretensioning, preinstallation verification and additional inspection requirements. This is in addition to the heightened cost of the connections due to additional bolts and connection material compared to a bearing-type connection. Use slip-critical connections when they are necessary in certain situations. Otherwise, bearing-type connections should be used. ■

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economics

HOW LONG WILL THE GOOD TIMES LAST?

The construction market in 2016 and beyond.

BY JOHN CROSS, P.E.

GOOD TIMES FOR CONSTRUCTION? That may be a bit of an overstatement.

For the nonresidential building construction market, the impacts of the Great Recession of 2008 through 2010 are still being felt. Nonresidential construction has not returned to prerecession levels; in fact, nonresidential construction levels have only grown by 250 million sq. ft from the dark days of 2010 and are still 737 million sq. ft below the peak level of 2007. The fact is that we have only regained 25% of the nonresidential market that was lost in the recession.

The recovery that has occurred in the building construction market has been in the residential sector and unlike prior recoveries, this recovery has been centered in multifamily buildings, not single-family homes. Single-family residential starts dipped from a high of 1.6 million units in 2005 to a low of 413,000 units in 2010. The market has recovered to 750,000 units in 2015. Similar to nonresidential construction, single-family home construction has only recovered 28% of the lost volume. But the same is not true of the apartment and condominium market, where 72% of the market lost during the recession has been recovered. This is particularly the case in multistory apartment and condominium projects greater than four stories in height, where activity has surpassed prerecession levels by 9%. This demographic shift is the result of a variety of factors including a generational desire to live in an urban environment, a reticence to invest in single-family homes, high college debt levels resulting in a lack of funds for down payments and a preference for rapid relocation.

So what does all of this mean for construction activity in 2016? The answer to that question is a balancing act between a series of competing influences that can best be addressed as a series of questions…

If nonresidential construction has not yet bounced back from the recession, does that mean that we are poised for a major increase in construction activity in 2016?

No, nothing in the data suggests robust growth in 2016. The AISC Business Barometer indicates a modest level of pessimism among structural steel fabricators, the AIA Architect's Billing Index is vacillating around neutral growth and the Dodge Momentum Index is taking one step back for every two steps forward.

Is there pent-up demand for nonresidential construction?

No, during previous construction cycles, growth in single-family housing generated demand for new schools, strip malls, office buildings and hospitals. This cycle is characterized by multifamily apartments and condos primarily being constructed in urban centers where nonresidential infrastructure already exists. While this activity may spur renovation of existing nonresidential buildings, it is not the same driver for new construction as greenfield single-family residential construction. In addition, many of these multistory residential projects are actually limited mixed use projects that include some retail and office space.

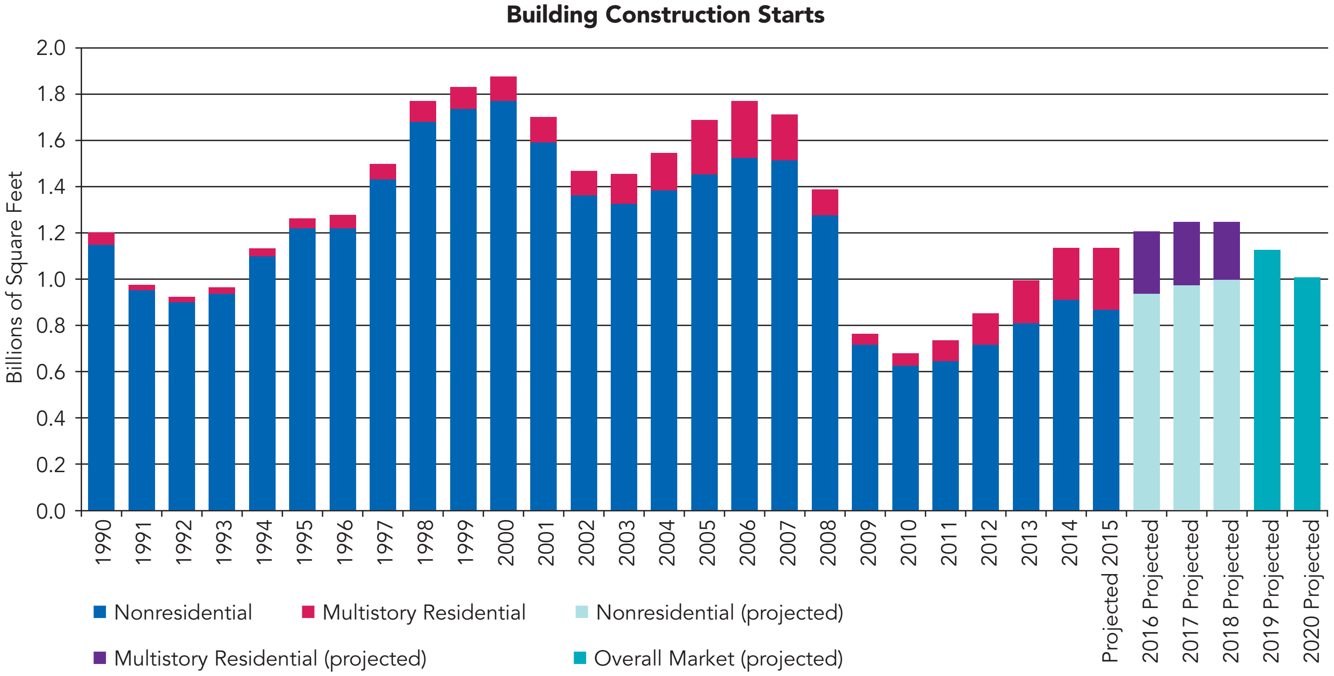
Is the current trend away from suburban single-family homes to urban apartments and condos a short-term phenomenon or a long-term reality?

It is unclear what the housing market will look like in ten years, but for this construction cycle the demand will continue to be for urban living rather than suburban single-family homes.

Is the overall economy pointing to a decrease or increase in construction activity?

The signals are mixed. Third quarter GDP growth was originally reported to be just 1.5%, which sent chills up and down the spine of every economist. Subsequently, the Bureau of Economic Analysis revised the third quarter GDP estimate to a much more comfortable value of 2.1%. At the same time, predictions for 2016 indicate GDP growth of 3.1%, a level that would support robust construction growth. At the same time, the government is reporting a 5% unemployment rate, which should indicate a growing workforce driving economic

John Cross (cross@aisc.org) is an AISC vice president.



and construction growth. However, the optimism of a low unemployment rate is offset by a decreasing labor utilization rate, indicating that a smaller percentage of eligible workers is employed or actively seeking employment. In addition, a low unemployment rate is typically accompanied by a spike in average earnings, which has not occurred in this cycle.

Is new construction economically viable?

Yes, prices for existing homes and commercial properties in central business districts are continuing to rebound from recessionary lows and are approaching parity with new construction.

Is financing available to support expanded construction activity?

Yes, there has been a gradual loosening of lending standards over the past two years, and interest rates are at a historic low. While some hesitation exists in financial markets regarding the impact of the Fed raising interest rates, it is doubtful that rate will rise by more than 1% over the next year, still leaving rates significantly below the traditional long-term level.

Are businesses poised to invest in capital expenditures such as buildings?

No, even though consumer spending is rising, capital expenditures by companies have declined by nearly 4% during the first 10 months of 2015. While some of the pullback in capital expenditures is the direct result of low oil prices impacting the energy industry and a strong dollar limiting exports, the trend is evident across the broader economy.

Is the construction sector poised for a cyclical downturn?

Historically, five to six years of growth follows the bottom of a construction cycle. The current construction cycle started in 2010, indicating that a peak should be experienced in 2015 or 2016, followed by a cyclical downturn. But this cycle has not been typical in that it began from a lower bottom and it has not experienced any periods of rapid growth. This would seem to indicate that the current cycle will be more extended than prior cycles.

So what are construction levels expected to be in 2016 and beyond?

It is anticipated that nonresidential building construction activity will grow by 6% to 8% in 2016. During this same period, multistory residential activity will grow by an additional 5% to 6%, resulting in an overall growth in the nonresidential plus multistory residential sector of 6% to 7%. While the mix between nonresidential and multistory residential construction will change in 2017, the overall level of growth will be similar, with construction activity reaching 1.2 billion sq. ft. The peak reached in 2017 will be maintained in 2018, followed by a gradual reduction in construction volume over the next two to three years. This compares to 1.7 billion sq. ft at the last peak in 2007 and 680 million sq. ft at the bottom of the recession in 2010.

Where will this growth be most apparent?

From a project type perspective, growth will occur in office market (+17%, mostly in central business districts), arena and stadiums (+15%), warehouses (+9%) and schools (+9%). It should be noted that the growth in office construction, while at 17%, is still significantly below pre-recession levels. Geographically, the greatest regions for growth continue to be in the southern states.

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A series of steel trees welcomes visitors to Dickinson College's new squash-centered athletic facility.

SQUASHING the Competition

BY STEPHANIE J. HAUTZINGER, S.E.

Photos: CannonDesign/Scott Frances

DICKINSON COLLEGE in Carlisle, Pa., has a long and successful history in sports—specifically squash.

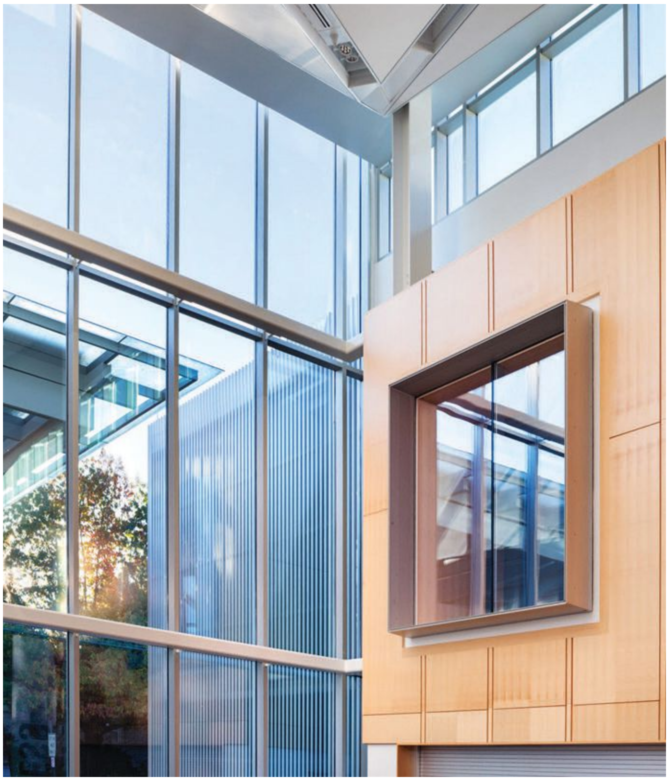
And the Dickinson College Red Devils have consistently excelled in athletics despite facilities that have not always been on par with the quality of and commitment to their athletic programs. The Kline Fitness Center Addition is the first component of a campus athletics master plan, focused on elevating sports facilities at the college. Completed in the fall of 2014, the project is a 30,000-sq.-ft, state-of-the-art fitness facility that includes five international-sized squash courts to support a new varsity men's and women's squash program.

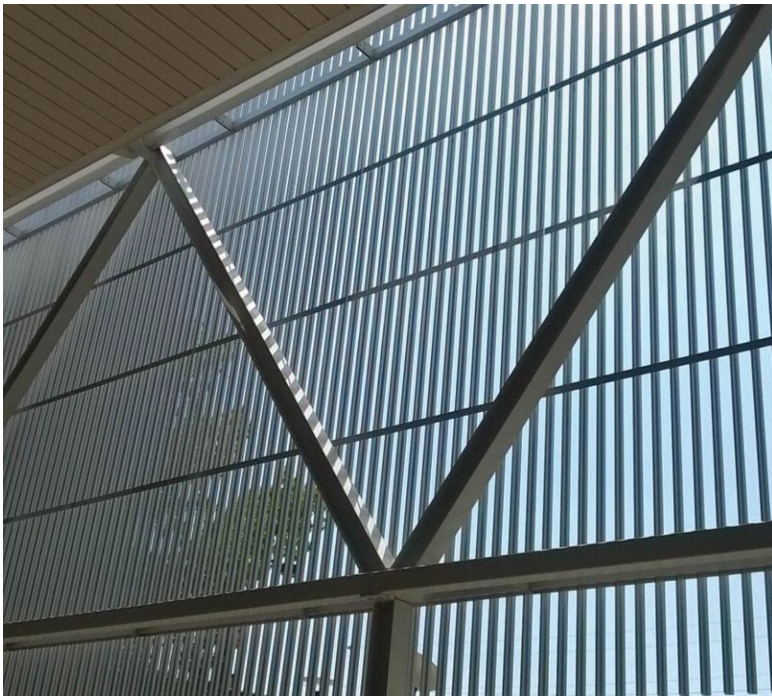
Stephanie J. Hautzinger (**shautzinger@cannondesign.com**) is a senior associate and structural engineer at the Chicago office of CannonDesign.

The existing 1980s structure has a unique and active roof form created by a series of hyperbolic paraboloid-shaped timber-framed roofs. While the existing structural system was an efficient means of enclosing the large spaces within, the interior of the Kline Center is dark and disconnected from the exterior. Per client and architectural goals, the addition was designed to counteract that dark and disconnected quality by bringing in natural light and views.

From the beginning, there was a commitment to express the structural solution as the primary architectural feature while complementing the exposed structure of the existing Kline Center. Much consideration was given to the appropriate means to harmoniously add on to the strong roof form of the existing building. A steel frame was the clear choice to provide a sculptural and aesthetically beautiful frame while allowing maximum transparency of the exterior wall, thus creating a light-filled and open interior space. No material was more appropriate or cost-effective than structural steel in creating this dramatic architectural expression, as well as in meeting the

◄ ▲ ▼ ► The independent canopy structure is comprised of four HSS8×8 tree columns, rotated at 45° in plan, with four W8×28 wide-flange branches springing from each column face and rising to support framing for the glass roof plane above. Select exposed structural steel members close to view were designated as AESS.





► The project is a 30,000-sq.-ft, state-of-the-art fitness facility that includes five international-sized squash courts to support a new varsity men's and women's squash program.

tight construction schedule. The structurally significant aspects of the project include the fitness center roof, the porch and sunshade and the entry canopy and concourse. Again, exposed structural steel framing is the primary architectural feature at each of these key components.

Building Elements

The main fitness space is a two-story volume with a partial mezzanine. The roof structure is comprised of exposed wide-flange beams in an X configuration, spanning 43 ft and supporting long-span 3½-in.-deep acoustical deck. The X-shaped layout of the beams references the crisscrossing form of the glulam members in the existing Kline Center. From the interior of the building, the system achieves the architectural goal of creating a conceptual link to the roof structure of the existing Kline Center.

The porch is an exterior athletic space, its roof supported by slender Y-shaped columns comprised of W6×25 shapes. The Y-columns are 26 ft tall and approximately 23 ft wide. Angles of the Y's subtly mirror the angles of the existing Kline Center roof structure.

The Y-columns continue around the building; beyond the porch, however, they become a second layer of framing, supporting only an exterior sunshade, with the primary building columns located behind the exterior wall. The sunshade is comprised of 1¾-in.-diameter vertical aluminum tubes supported off of the Y's and serves to filter daylight through the expansive glass walls.

The Kline Center forms the western terminus of a major campus pedestrian route called Dickinson Walk. The architectural intent of the addition was to continue the tree-lined walk metaphorically with structural steel "trees" supporting the exterior glass canopy as well as marching along the interior sky-lit concourse.

The independent canopy structure is comprised of four elegantly detailed HSS8×8 tree columns, rotated at 45° in plan, with four W8×28 wide-flange branches springing from each column face and rising to support framing for the glass roof plane above. Select exposed structural steel members close to view were designated as AESS.

Design-Assist Process

A design-assist approach was chosen to enhance architectural design through thoughtful detailing, ensure ease of construction, reduce cost and save time in the construction schedule through early preparation of shop fabrication drawings.

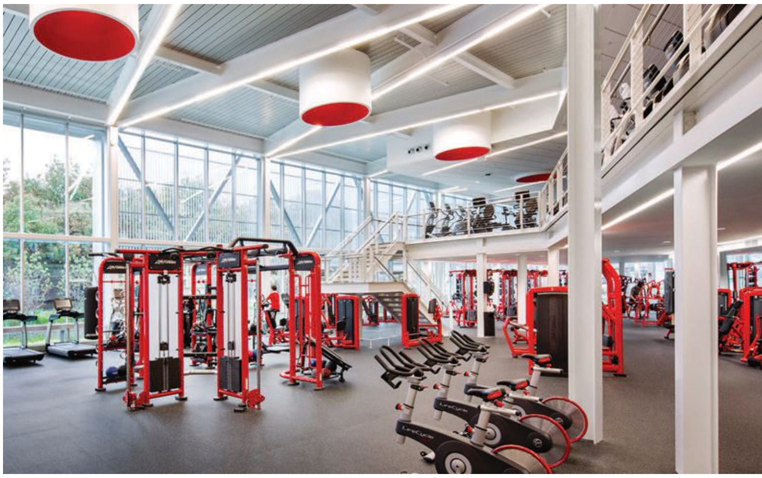
From our design perspective, there was some added time to provide information to the detailer early in the process; a detail-

◄ ▲▼ The structurally significant aspects of the project include the fitness center roof, the porch and sunshade and the entry canopy and concourse. The exposed structural steel framing is the primary architectural feature at each of these key components.

▲ The main fitness space is a two-story volume with a partial mezzanine.

▼ The entrance canopy. The existing Kline Center is visible in the background to the right.

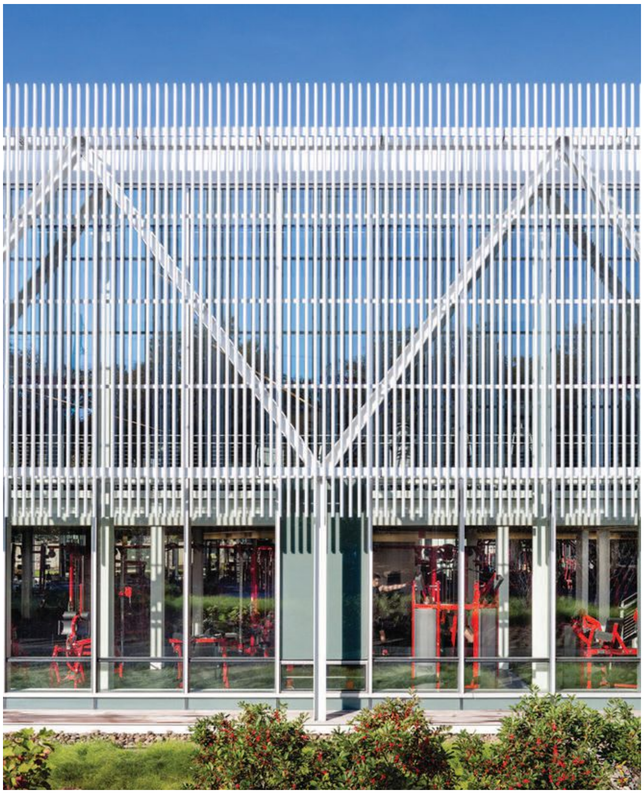
▲▼ Structural steel is exposed prominently throughout the facility, both inside and out.



▲ Framing for the roof.

▼ Natural daylighting is prominent throughout the building.

▼ Y-columns supporting the sunshade.



ing model was started off of CannonDesign's 50% construction document set. The 100% construction document/issue for bid package included a complete set of structural steel shop drawings. A "soft approval" of the shop drawings was completed by CannonDesign during the four-week bidding phase of the project. This sped up the traditional process in that all steel shop drawings were complete at the issue-for-bid date, and tighter bids were received due to full steel detailing being complete.

A number of cost-saving ideas were incorporated into the design, including making the main fitness roof flat instead of sloped to keep the typical skewed roof beam connection details identical, increasing column sizes at some skewed connections to simplify detailing and careful consideration of detailing of the Y-column joints and canopy tree column joints for ease of shop fabrication and field assembly (while bearing shipping limitations in mind).

The Kline Fitness Center addition begins the transformation of the existing Kline Center into a modern athletic complex. It puts physical fitness on display with its graceful exposed structure and expansive glazed walls and draws visitors in through its metaphoric continuation of Dickinson Walk.

The sunshade is comprised of 1¾-in.-diameter vertical aluminum tubes supported off of the Y-columns and serves to filter daylight through the expansive glass walls.

Steel trees support the exterior canopy.

This project exceeds the college's needs by providing a state-of-the-art athletics facility that addresses its continued commitment to fitness and participation in competitive sports, and it does so with a structurally expressive, signature piece of architecture that the college is very proud of and that has become a prominent and celebrated building on campus. ■

Owner
Dickinson College

Architect
CannonDesign

Structural Engineer
CannonDesign

General Contractor
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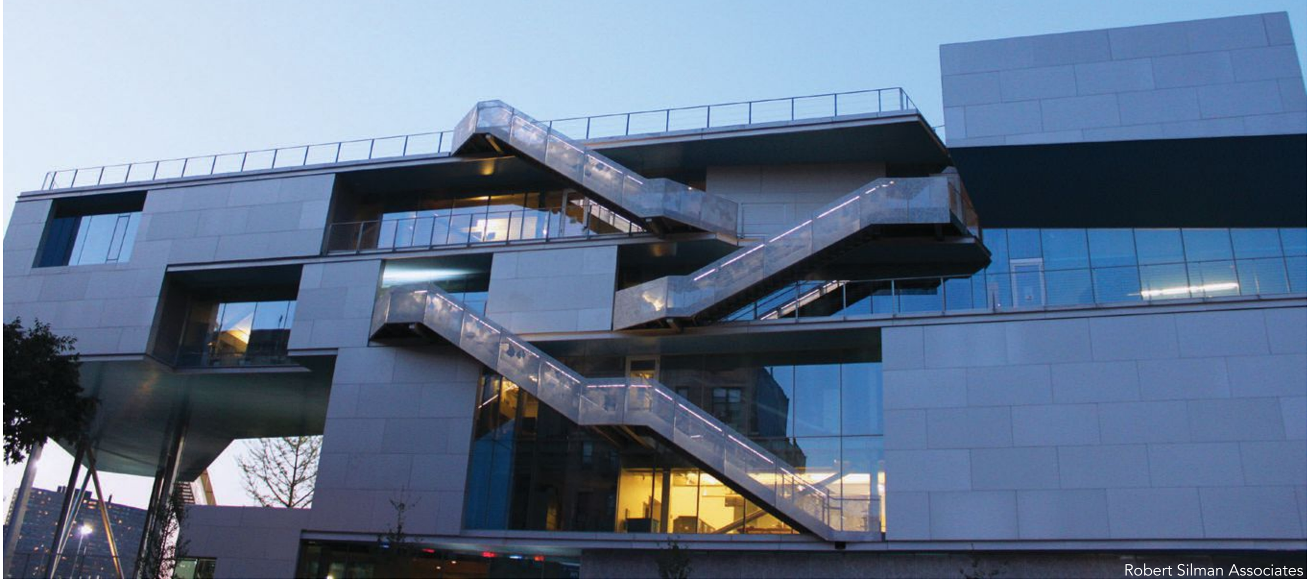
Sporting a NEW LOOK

BY JUSTIN DEN HERDER, P.E.

Exposed steel framing flexes its muscle at a new university sports facility.


Robert Silman Associates

The five-story, 48,000-sq.-ft Campbell Sports Center uses 325 tons of structural steel in all to achieve shallow floor assemblies and long spans.

On the interior, many of the steel beams are upset—the plank bears on top of the beam's bottom flanges—to maximize floor-to-floor heights.


Robert Silman Associates


Steven Holl Architects

CAMPBELL SPORTS CENTER is the new face of athletics at Columbia University in Manhattan.

The five-story, 48,000-sq.-ft building, designed by Steven Holl Architects, is the primary athletics facility for all of the school's outdoor sports programs. A combination of strength and conditioning facilities and student-athlete spaces coexist in the state-of-the-art building, serving the minds and bodies of the school's student athletes.

The conceptual design for the building is taken from the phrase "points on the ground, lines in space," which references the diagrams used by sports teams to strategize their plays. In this case, the points are slender, tilted columns where they meet the sloping grade, and the lines are its bracing, terraces and external stairs.

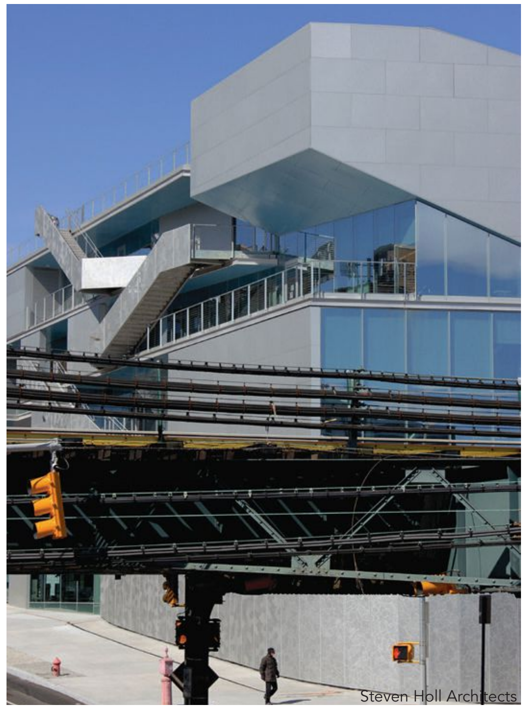
Given the relatively modest budget ($30 million), challenging site topography and a complex geometrical floor plan, structural engineer Robert Silman Associates saw this project as an opportunity to use typical construction materials in creative and ingenious ways. The building is steel-framed, which was dictated by the architectural and programmatic requirements, chiefly the desire for shallow floor assemblies and the need for long spans over vast spaces. It uses 325 tons of structural steel in all.

The building consists of a primary body, which accommodates the strength and conditioning room, offices and a hospitality suite. At the fourth floor, an arm juts out from this main part of the building to the west to form a portal-type structure that's supported at its tip by a combination of the slender, sloping columns (the points on the ground). At the east end of the building, an auditorium constructed of two-story, full-height trusses slopes upward and cantilevers from the fourth-floor framing level, supported on two custom-fabricated plate girders. Another massive 22.5-ton, 54-in.-deep plate girder spans 60 ft over the double-height fitness room. Its presence adds brawn to the space and complements the clanking of the steel weights below.

The superstructure is predominantly made up of 12-in.-deep hollow-core precast plank elements that span up to 40 ft across a series of east-west steel frames. At the building's perimeter, the steel was lowered to allow the planks to span over top of the spandrel beams and cantilever up to 8 ft to form the exterior terraces. On the interior, many of the steel beams are upset—the plank bears on top of the beam's bottom flanges—to maximize floor-to-floor heights. Careful consideration was given to construction sequencing and erection during the layout of these upset members.

Justin Den Herder (**denherder@silman.com**) is a senior project engineer with Robert Silman Associates.

Robert Silman Associates



Steven Holl Architects

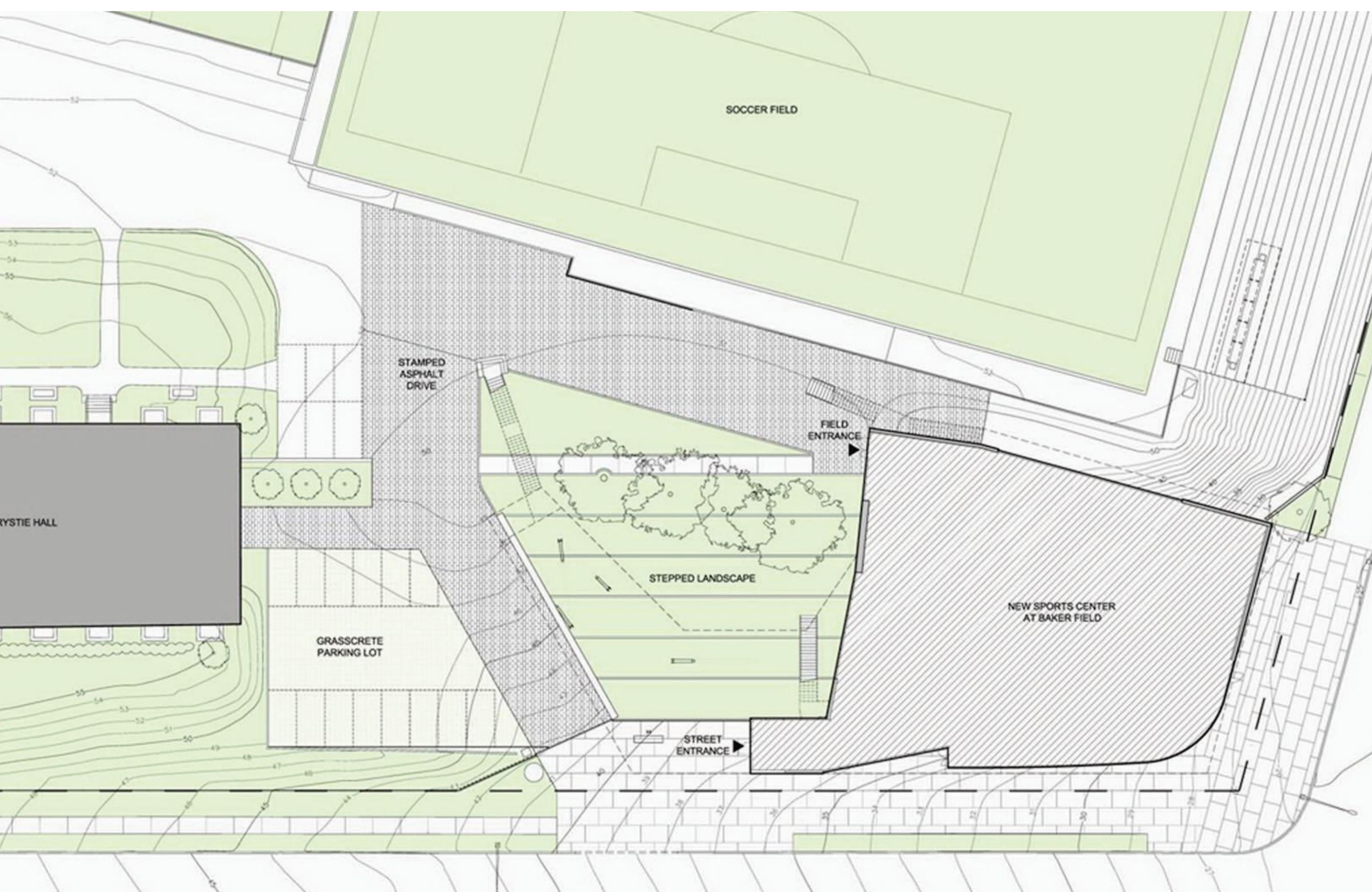
Robert Silman Associates

▲ Exterior support.

▼ A site plan of the project.

▲ The building is adjacent to elevated New York City Subway tracks.

▲ One of the building's metal staircases.



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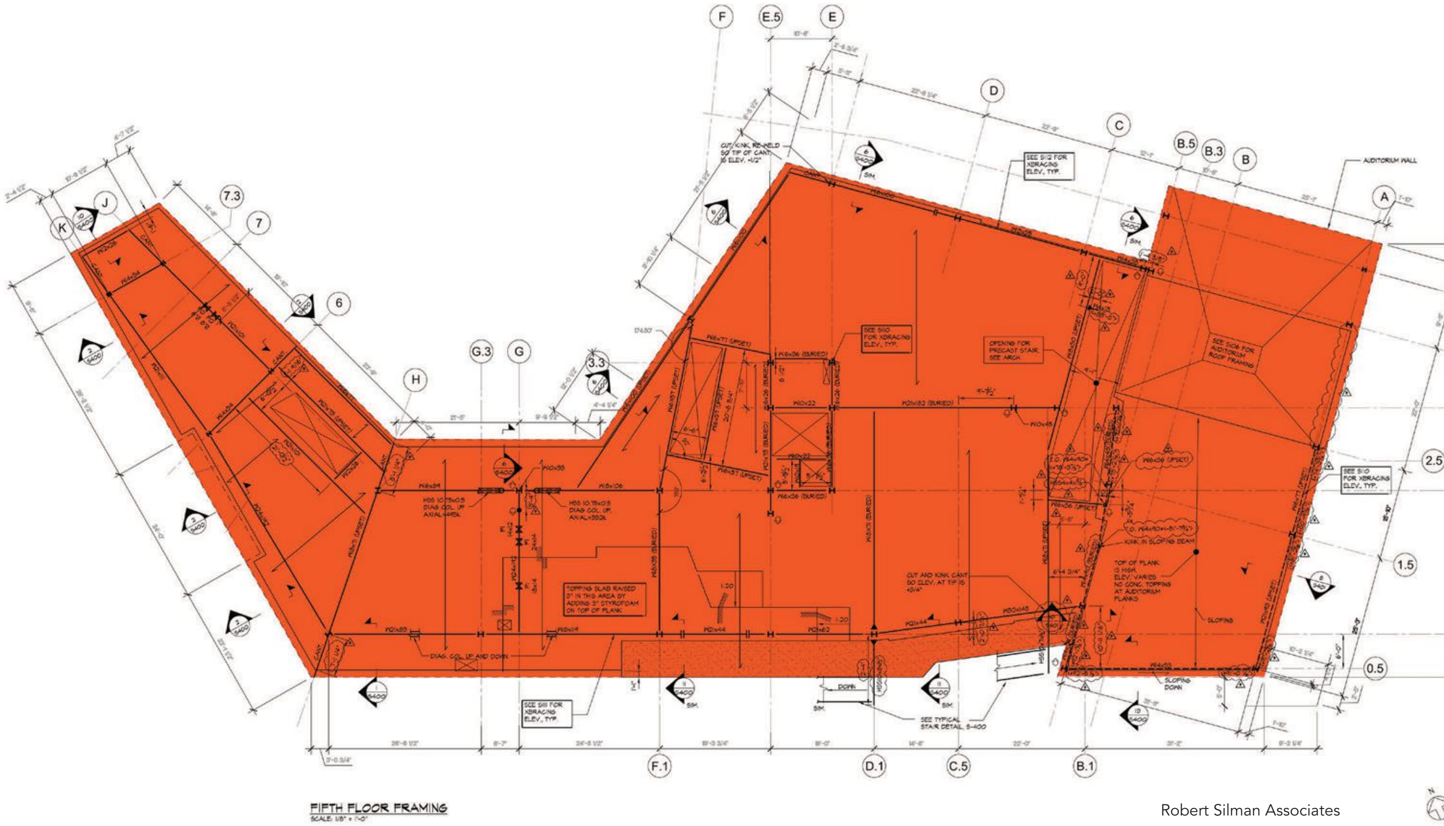

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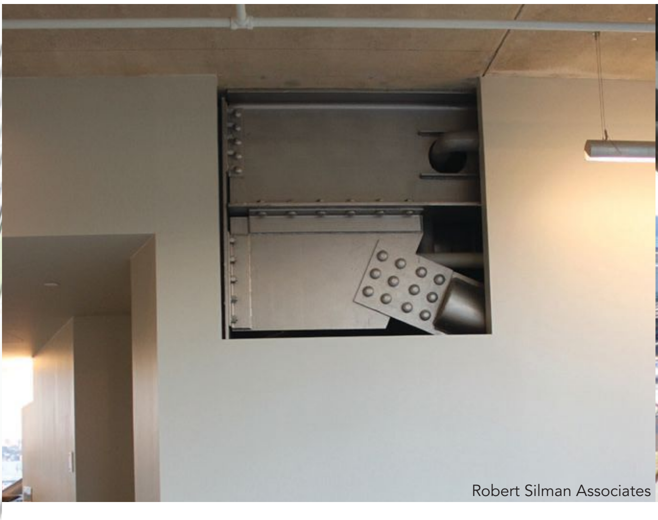
▲ The framing plan for the fifth floor.

▼ The lateral system is a combination of HSS braced frames and moment frames in the east-west direction and HSS braced frames in the north-south direction.



Robert Silman Associates

Robert Silman Associates

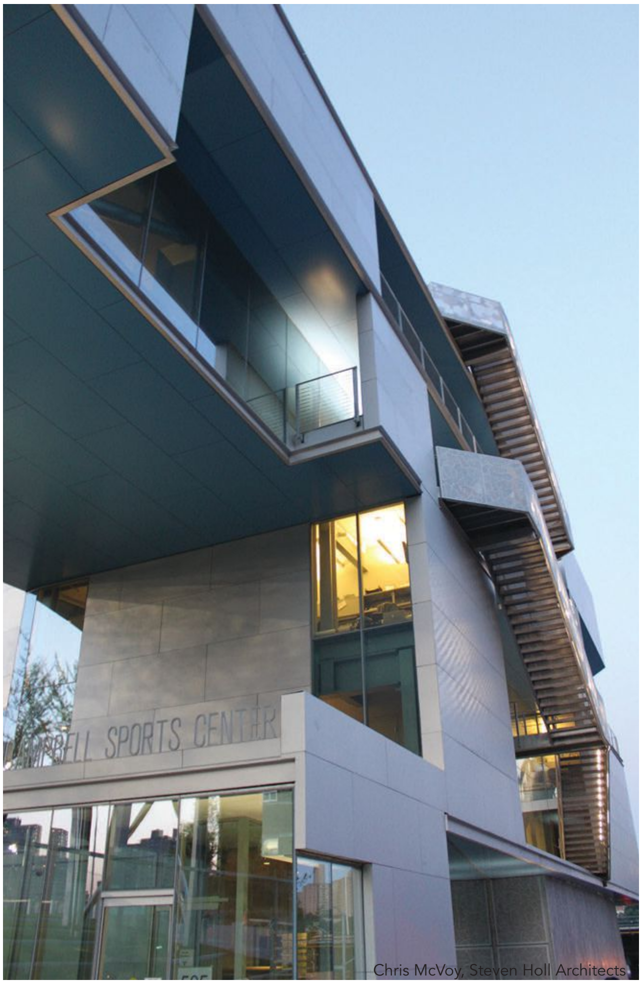
An Array of Vertical Elements

The columns and braced frames were aspects that the design team deliberated extensively, as the design intent was to expose a majority of the framing and integrate it within the language of the building. The columns beneath the tip of the building's elevated arm were studied intensely in a variety of schemes that combined sloping, skewed columns with straight, slender ones. Of the six configurations, the design team chose an elegant and efficient option that orients the columns to run in concert with the natural inclinations of the load paths of the framing above.

The lightness of the arm was made possible by designing a 65-ft-long truss that forms a bridge connecting the arm back to the body of the building, thus maintaining a column-free area below most of the arm.

The lateral system is a combination of HSS braced frames and moment frames in the east-west direction and HSS braced frames in the north-south direction. These bracing elements are exposed, aesthetic features of the building, fulfilling the lines in space motif. Because the braces were always intended to be exposed, their gusset plate profiles were another aspect of the design that was rigorously worked out with the architect. The geometric profiles of the gusset plates were streamlined, thereby minimizing the amount of steel, alleviating conflicts with continuous plank pieces and permitting them to serve as featured architectural elements.

The close proximity of the elevated subway tracks placed restrictions on the potential locations for crane placement, which influenced the erection sequencing of the framing.



The angular nature of the building is apparent from virtually any point of view.

Intricate Coordination

The close proximity of the elevated subway tracks placed restrictions on the potential locations for crane placement, which in turn influenced the erection sequencing of the framing. As a result, it became necessary to erect the steel at the arm portion of the building first. Silman worked closely with the crane engineer and the shoring engineer to locate temporary steel that would allow for the arm to be self-supporting. Because of the nature of the sloping columns supporting this portion of the building, the overall stability of the arm relied on the mass of the main body of the building.

Since the building's services were to be exposed in much of the structure, the layout of MEP services was critical. This resulted in numerous web penetrations through the beams, which required a concerted, coordinated effort from the design team to determine duct routing in an efficient and effective layout. In locations where penetrations were not feasible, steel was upset into the planking to avoid web penetrations altogether. Networks of conduits run through the 3-in. topping slab above the precast plank. Where these conduits conflicted with top flanges of the steel framing, a plate-reinforcing detail was developed.

By producing framing elevations that provided the anticipated dead, superimposed dead and live load deflections of the spandrel beams, it allowed the contractor to build these tolerances into the design and installation of the curtain wall elements. Further breakdowns of the dead load deflections were calculated to mimic the exact site conditions at any given instance throughout the curtain wall installation.

The design of the building proved that with careful consideration of construction sequencing, wonderful moments of architecture can be achieved—in a budget-conscious manner—with the simple materials that are used every day. Rigorous global analysis coupled with an in-depth study of each individual gusset plate and moment connection accounted for the design of the steel-framed skeleton, whose size and scale blends seamlessly with the architecture. ■

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Steven Holl Architects

Structural Engineer
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13	Tee Stem Up	MEAN DIA.	22" x $142\frac{1}{2}$# Tee
14	Angle Heel In	INSIDE DIA.	8" x 8" x 1" Angle
15	Angle Heel Out	OUTSIDE DIA.	8" x 8" x 1" Angle
16	Angle Heel Up	MEAN DIA.	8" x 8"x1" Angle
17	Square Tube	INSIDE DIA.	24" x $\frac{1}{2}$" Tube
18	Rectangular Tube The Easy Way (Y-Y Axis)	INSIDE DIA.	20" x 12" x $\frac{5}{8}$" Tube
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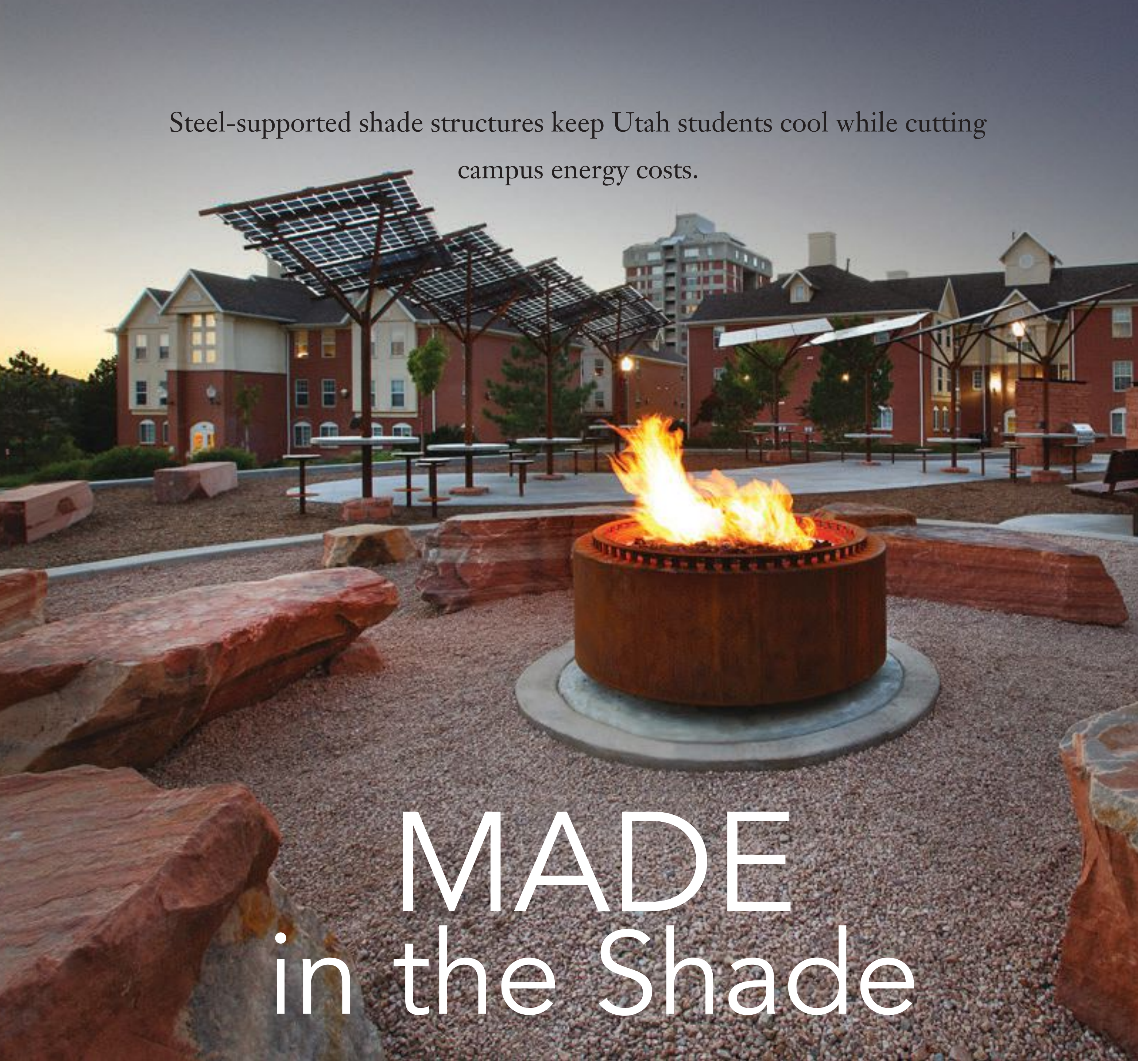


BY TRACY STOCKING AND NATHAN MURRAY

THE UNIVERSITY OF UTAH'S Housing and Residential Education department has upped the green ante with its outdoor public spaces.

The Student Solar Plaza at the university's Shoreline Ridge Residential Community features a series of steel-supported bifacial solar panels that harvest solar power from both the top and bottom surfaces. Originally constructed as athlete housing during the 2002 Winter Olympics, the Shoreline Student apartments were repurposed for student housing.

Located on the university's campus, which sits at the eastern edge of Salt Lake City and at the base of the Wasatch Mountains, the plaza is composed of eight weathering steel "trees"—steel poles and "branches"—oriented in an ellipse and topped with three translucent bifacial photovoltaic (PV) panels; concrete tabletops are located at each tree, with the panels providing shade to students while simultaneously gathering sunlight. A kitchen tower with two gas grills and a weathering steel-clad fire pit surrounded by monolithic stone seating also highlight the communal space.

Inherently Sustainable

The project followed an underlying design premise of using basic, natural building materials that are inherently sustainable. This allowed TSA Architects to establish a palette of native Utah sandstone, concrete, Ipe wood and weathering steel. Each of these materials are deployed in their most basic natural state, eliminating the need for paint and ongoing maintenance while connecting the project to the surrounding environment.

Jim Fairchild, Fairchild-creative.com

Jim Fairchild, Fairchild-creative.com

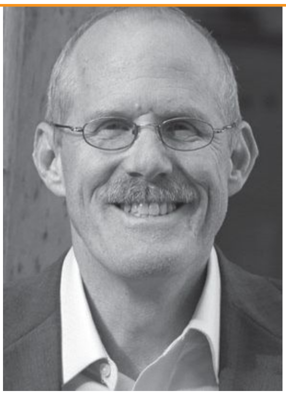
◄ ▲ ▼ The Shoreline Student apartments have been repurposed to student housing from their original use as housing for the 2002 Winter Olympics. The plaza features eight steel-supported bifacial solar panels that harvest solar power from both the top and bottom surfaces.

TSA Architects

The use of exposed weathering steel HSS (4-in.-diameter) enabled TSA and structural engineer BHB Engineering to create an abstraction of a tree. Not only do the trees serve as structural support for the tables and canopies, but they also act as conduit for the electrical wiring connected to the solar panels. The natural oxidized patina on the steel deepens after exposure to weather and remains maintenance-free and resists corrosive weather effects.

Incorporating the solar element into the structure, which was not originally part of the plan, allowed the trees to take on a more active role. In the original plan,

Tracy Stocking (**tracy@tsa-usa.com**) is president and **Nathan Murray** (**nmurray@tsa-usa.com**) is a design principal, both with TSA Architects.



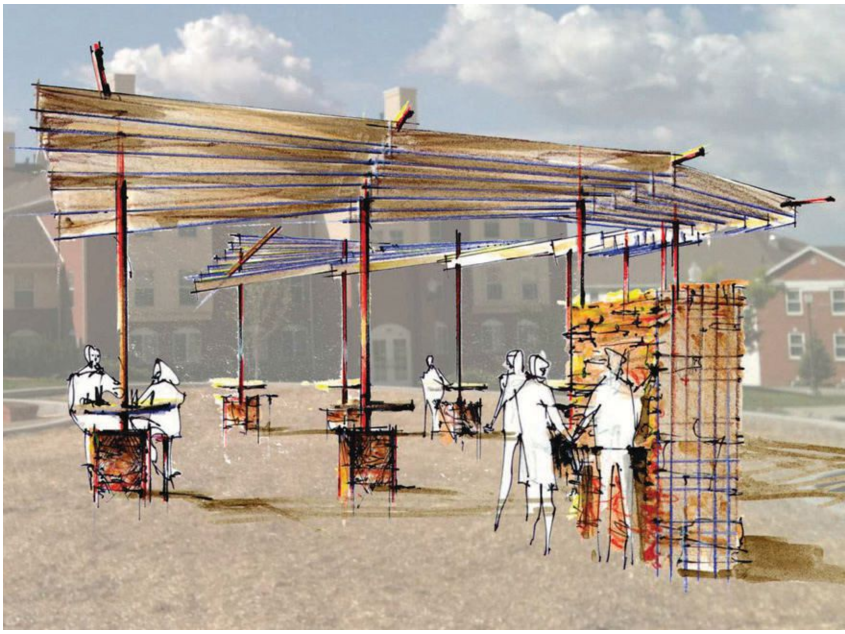
➤ ▲ ▼ The panels are capable of generating about 9,000 kW of power each year. Gathered energy is available in outlets at the base of the structures, and unused energy is redirected back to the power grid.

the general structure was the same, but the deflection limits were more stringent with the addition of the solar panels, and the purlin layout was modified to accommodate panel connection locations.

Energy gathered by the panels is available in outlets at the base of the structures for students to plug in laptops or equipment for outdoor cooking. Energy not used at the site is redirected back to the power grid and helps reduce the school's overall power bill. The project is capable of generating about 9,000 kW of power each year, promoting sustainability and demonstrating an immediately accessible use of solar power.

One of the biggest challenges the team anticipated was maintaining tight tolerances when building the trees, which were welded together on-site. As such, all of the trees were designed to be identical. However, they are oriented in an intentionally random arrangement to create a sense of natural variation. The entire plaza was 3D modeled using Revit, which facilitated precision between the steel support structures in maximizing solar orientation and shading from the canopies.

Another challenge was coordinating the attachment of the PV panels to the steel frames, and TSA shared the Revit model directly with the fabricator to ensure a higher level of accuracy for replication. The attachments were made by installing ¼-in.-



▲▼➤ The entire plaza was 3D modeled using Revit, which facilitated precision between the steel support structures in maximizing solar orientation and shading from the canopies.

thick tabs on the purlins at the locations dictated by the PV panel supplier, and then the panels were bolted to the tabs.

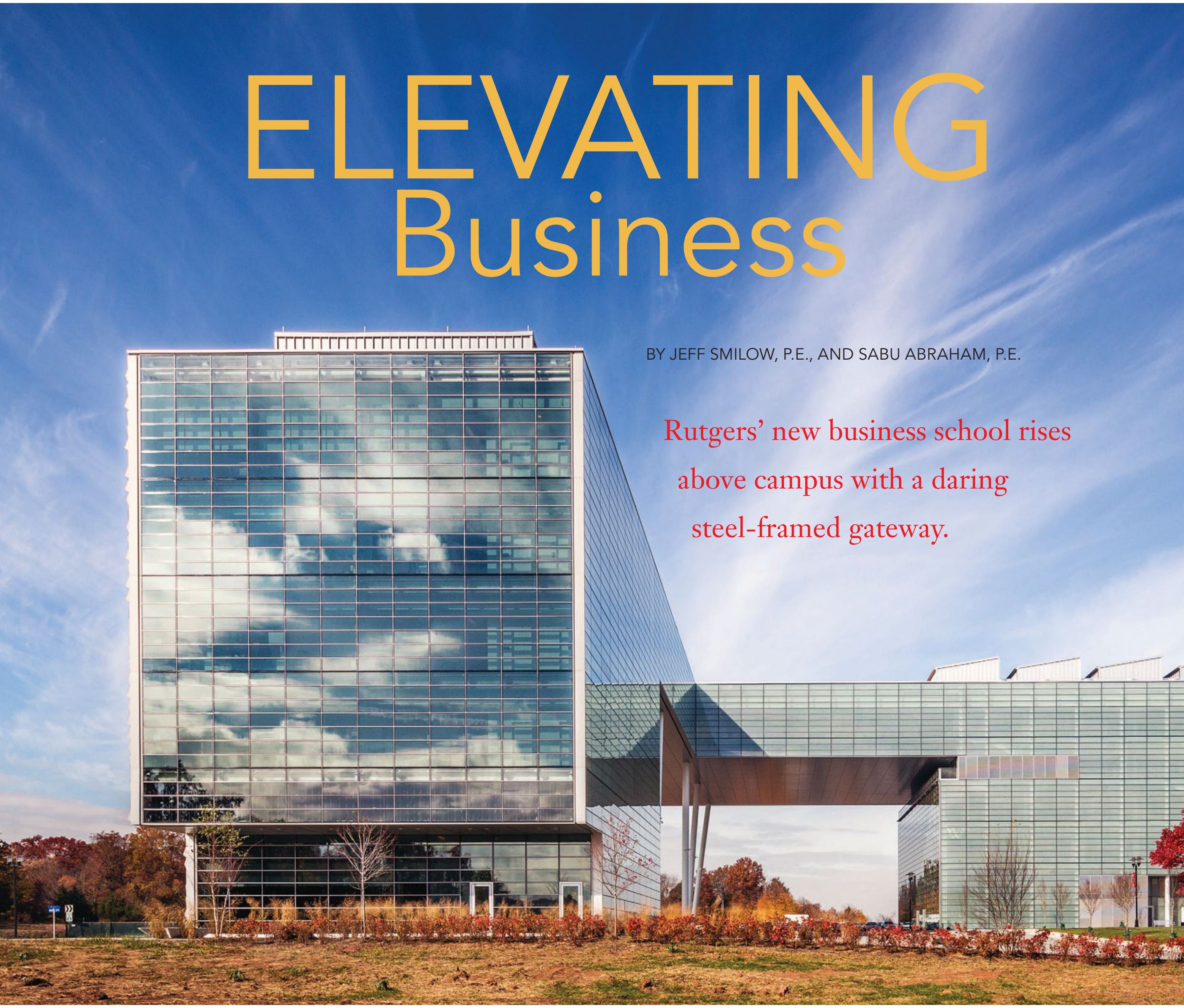
Completed in the fall of 2014, the plaza illustrates how to integrate bifacial photovoltaic (PV) solar panels with structural steel in an aesthetically pleasing way while simultaneously achieving increased technological awareness and demonstrating innovative use of existing and emerging technologies. The result is a harmonious fusion of green technology and materials. ■

Owner
University of Utah, Housing and Residential Education

General Contractor
Terra Engineering and Construction

Architect
TSA Architects

Structural Engineer
BHB Engineering



ELEVATING Business

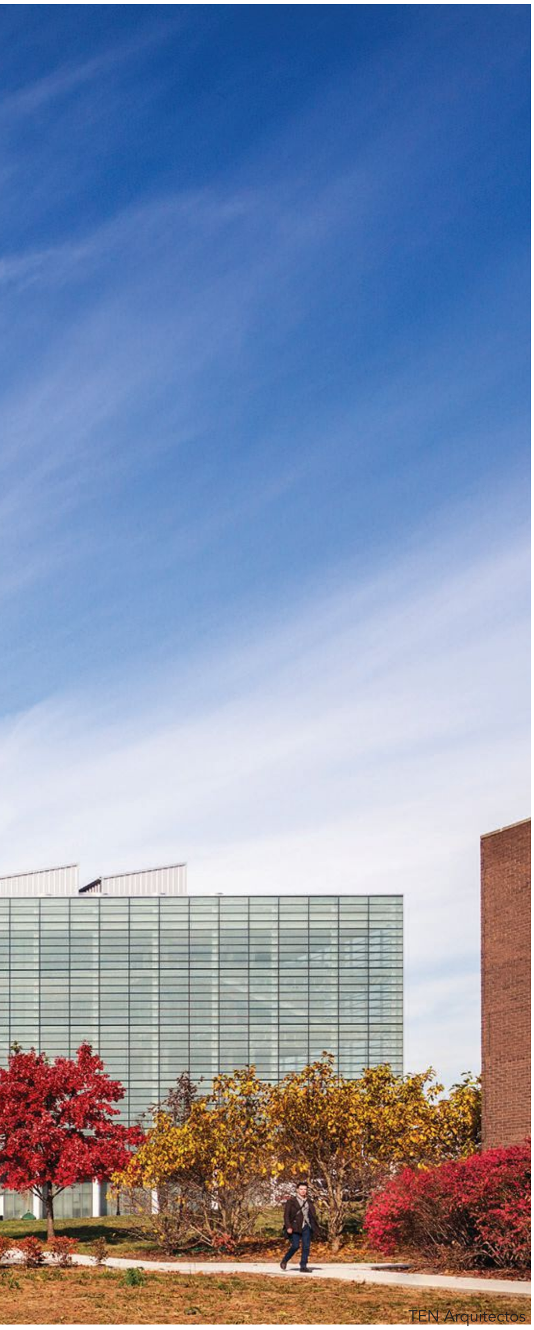
BY JEFF SMILOW, P.E., AND SABU ABRAHAM, P.E.

Rutgers' new business school rises above campus with a daring steel-framed gateway.

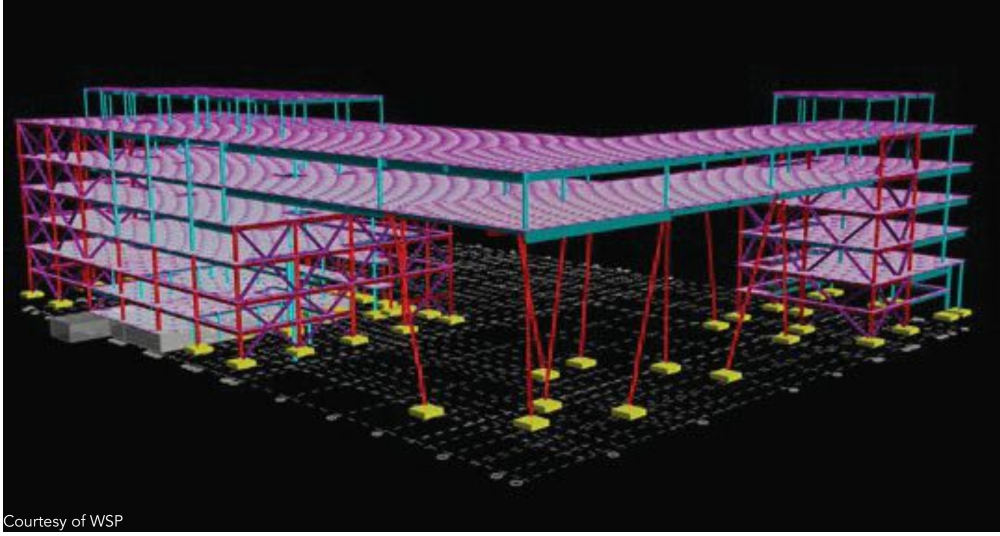
Jeff Smilow (jeffrey.smilow@wspgroup.com) is executive vice president and USA director of building structures and **Sabu Abraham** (sabu.abraham@wspgroup.com) is a senior associate, both with WSP USA Corp.

RUTGERS UNIVERSITY is relying on its new business school for more than simply educational purposes.

The new 150,000-sq.-ft building also serves as the gateway to the school's new Livingston Campus in Piscataway, N.J. The L-shaped form of the building appears to float 60 ft above Rockefeller Road, and most campus traffic passes under and through the building. The design keeps within the goals of the master plan of creating a high-density academic development complete with urban facilities, shared amenities and a walkable campus. In addition, it reflects the ongoing shift in higher education that supports collaboration and moves away from a focus on simply classroom-oriented organization, which in turn prepares students for contemporary business models where cross-collaborative ideas, cultures and concepts help create market innovation.



TEN Arquitectos



Courtesy of WSP

▲ A 3D model of the building.

◄ The 150,000-sq.-ft building serves as the gateway to the school's new Livingston Campus in Piscataway, N.J.

▼ Twelve 65-ft-long, 36-in.-diameter round sloping columns support the "floating" L-shaped building form above.

TEN Arquitectos

Not-so-Slippery Slope

The building is organized into three programmatic "bands": classrooms, offices and public spaces. Architect TEN Arquitectos connected the bands vertically with an atrium and horizontally with communal spaces of different sizes. Structurally, there are twelve 65-ft-long, 36-in.-diameter round sloping columns that support the L-shaped building form above. These exposed exterior columns feature an intumescent coating for fire resistance. In order to achieve the necessary strength, they were filled with self-consolidating concrete after the steel was erected but before the fifth-floor slab above was poured; the concrete was poured through two 6-in.-diameter holes in the cap plates of the columns.

Tapered details were used at column bases. Instead of creating the tapered ends with castings, the fabricator made each one by scribing out a piece of the pipe section and welding a plate into the scribed area. The plate was then welded to the tip of the column to receive the 5-in. pin, a more economical solution than using custom casting due to the particularly large tube diameter required.

The floating L-shaped feature connects the two sections of the building at the fifth floor and includes a 92-ft, column-free span. To achieve this long span, 60-in.-deep built-up plate girders were used are supported by the sloping columns at one end and "regular" building columns at the other. The plate girders were subcontracted to a specialty highway girder manufacturer and delivered full-length directly to the site.

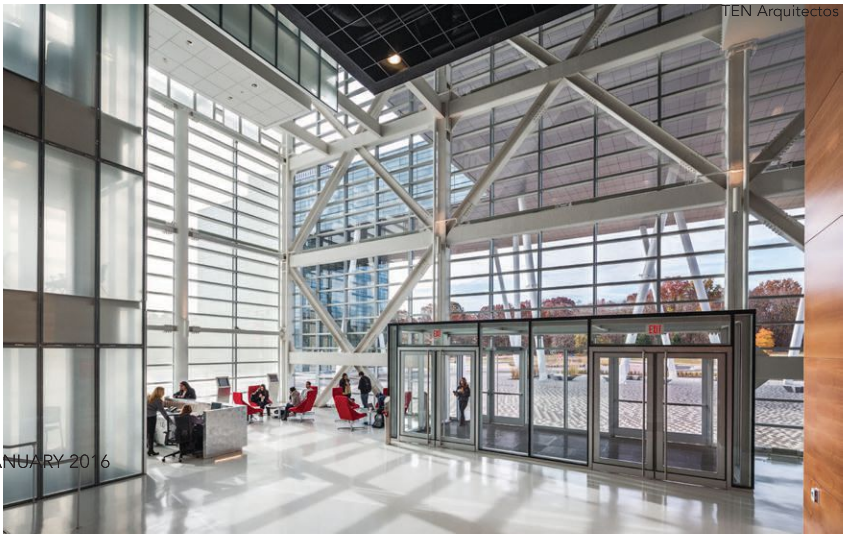
As one can imagine, erecting sloping columns supporting 92-ft-long girders posed a significant erection challenges. The steel erector answered this with optimized sequencing to hold up the sloping columns while connecting beams and girders,

Courtesy of WSP

The columns were filled with self-consolidating concrete after the steel was erected but before the fifth-floor slab above was placed.

The design team created a finite-element model to study human-induced vibration for the floating portion of the building.

Courtesy of WSP



TEN Arquitectos

Exposed bracing inside the building became an architectural feature.

and sufficient columns, beams and girders had to be connected together before a section became stable.

Addressing Vibration

In order to assure there would not be a vibration issue with the floating portion of the building, the design team at WSP Structures created a finite-element model to study human-induced vibration for this area as well as performed a time history analysis following the AISC Design Guide 11 *Floor Vibrations Due To Human Activity* recommendations. Based on the analysis, it was determined that the human-induced vibrations would be considerably less than the acceptable vibration levels defined by the ISO chart in Chapter 2 of the guide.

Dr. Thomas Murray, the guide's author, visited the site before the building fit-out and façade erection to study the vibrations on the floating L-shape. With an accelerometer placed on the floor and attached to a smart phone, a volunteer walked the floor at different frequencies with a metronome in hand. Based on the data collected, the floor performed exactly as predicted by the time history analysis.

Enhanced Exposure

Structural steel members also created other architectural features within the building. Exposed bracing was one example, and ensuring that the lateral forces induced from wind and seismic events could get to these bracing systems turned out to be a challenge as well. Because of the open nature of the building, numerous openings in the floor diaphragms were required. These openings, in conjuncture with the L-shaped building mass connecting the two parts of the building, required the design team to carefully follow the load paths of the wind- and seismic-induced loads into the bracing systems.

In the end, the architect's vision created numerous structural challenges for

➤ The building is organized into three programmatic "bands": classrooms, offices and public spaces.

Allen Thompson courtesy of WSP

the design team, but those challenges were overcome using the same collaborative, problem-solving techniques that the new business school promotes. ■

Owner
Rutgers University

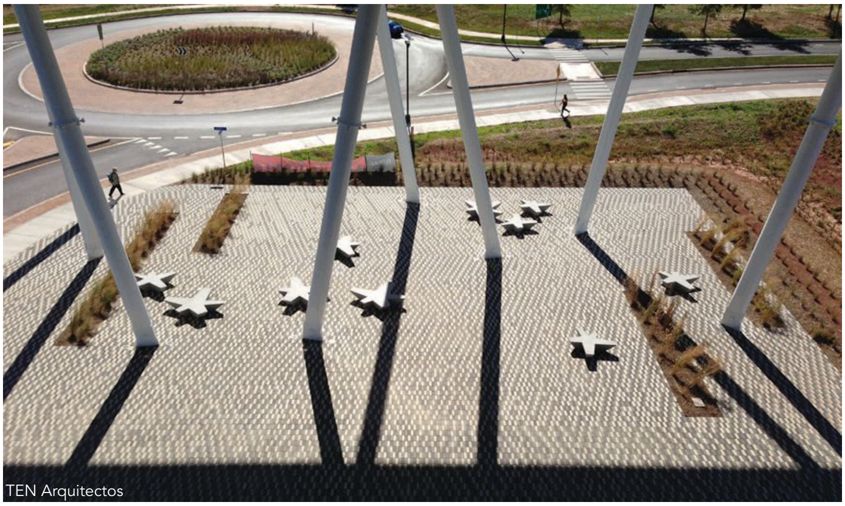
General Contractor
Century 21 Construction

Construction Manager
Structure Tone

Architect
TEN Arquitectos

Structural Engineer
WSP USA Corp.

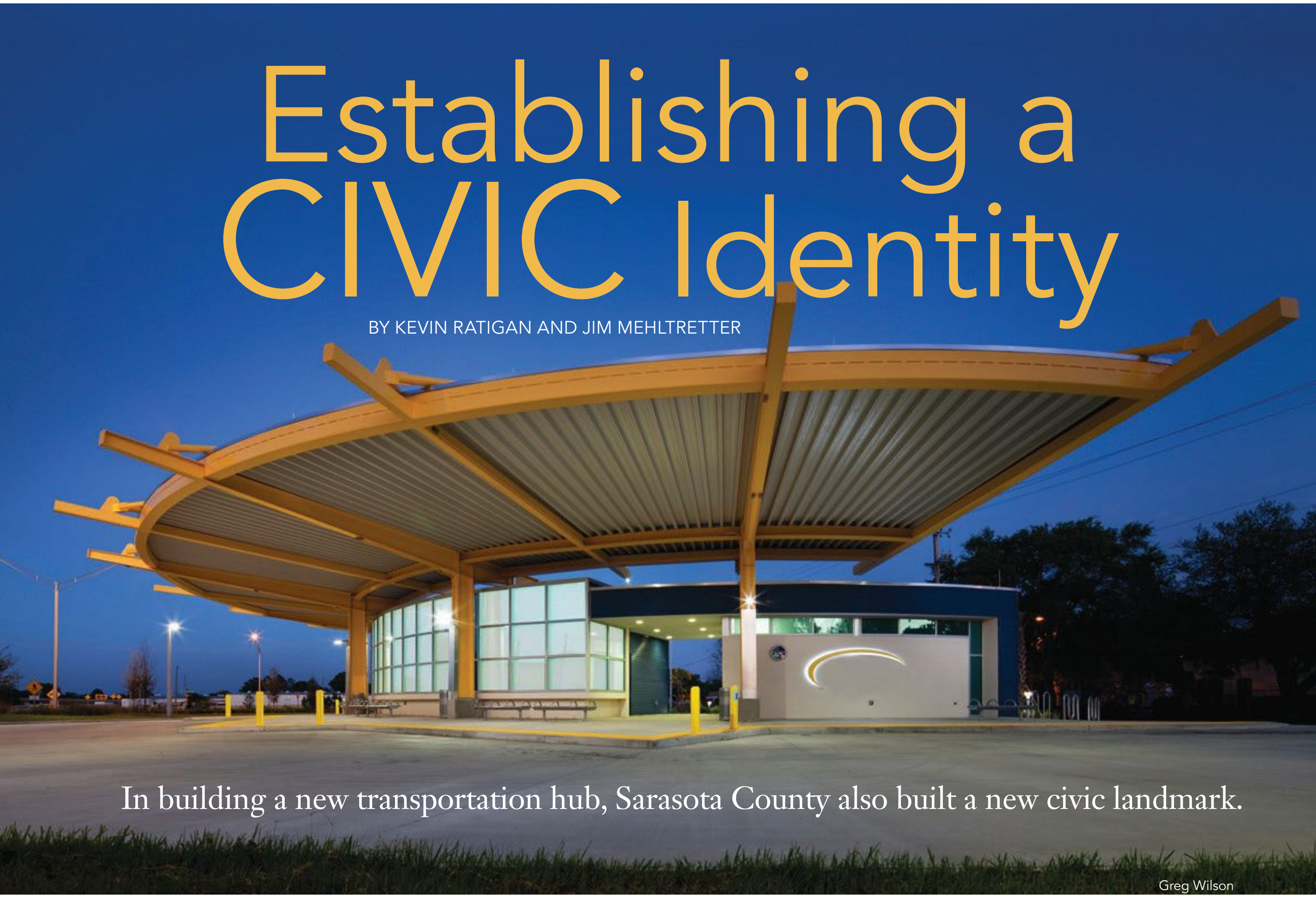
➤ The exposed exterior columns are coated with white intumescent paint.



TEN Arquitectos

Establishing a CIVIC Identity

BY KEVIN RATIGAN AND JIM MEHLTRETTER



In building a new transportation hub, Sarasota County also built a new civic landmark.

Greg Wilson

SARASOTA COUNTY, FLA., had long felt the need to consolidate its services into one area.

And so it did, opening a county facilities campus outside the downtown core of the city of Sarasota. The master plan is centered around an "eco-campus" concept where all vehicular traffic is pushed to the outside and the internal campus is focused on reclaiming wetlands and natural vegetation. A prominent feature of the campus is the Bus Transfer Station. Loading and unloading of riders at the station is served by a long, arcing steel canopy that also acts as a gateway to the county campus.

Structural steel was selected for the project because no other material could achieve the form and structural spans required and satisfy the architect's design requirements while preserving the owner's budget. Steel also made it possible to economically support the various load requirements, including Florida's potential for hurricane winds exceeding 150 mph. The new canopy flies over a conventionally constructed steel-framed building that houses restrooms, ticketing offices and a drivers' lounge. The lounge is situated to allow for clear views of all bus staging areas, using clear and translucent glazing to achieve both privacy and visibility from inside. While the building and canopy are separate, the building serves as a counterweight for the canopy's foundation.

Structural Expression

The site selection and specific configuration of the structure was born out of numerous studies analyzing major route patterns, circulation paths and population growth patterns. The primary goal was to create accessible public transportation hubs and multimodal connectivity to both help the area grow and divert vehicular traffic impacts. The project is part of Sarasota's effort to breathe new life into its public transportation system by promoting brand identity and ridership through functional structural expression, use of color and open space. The colors for the station were selected from the transit authority's logo to reinforce the county's brand identity, and the more vibrant yellow perfectly expresses the radial form of the dynamic steel structure.

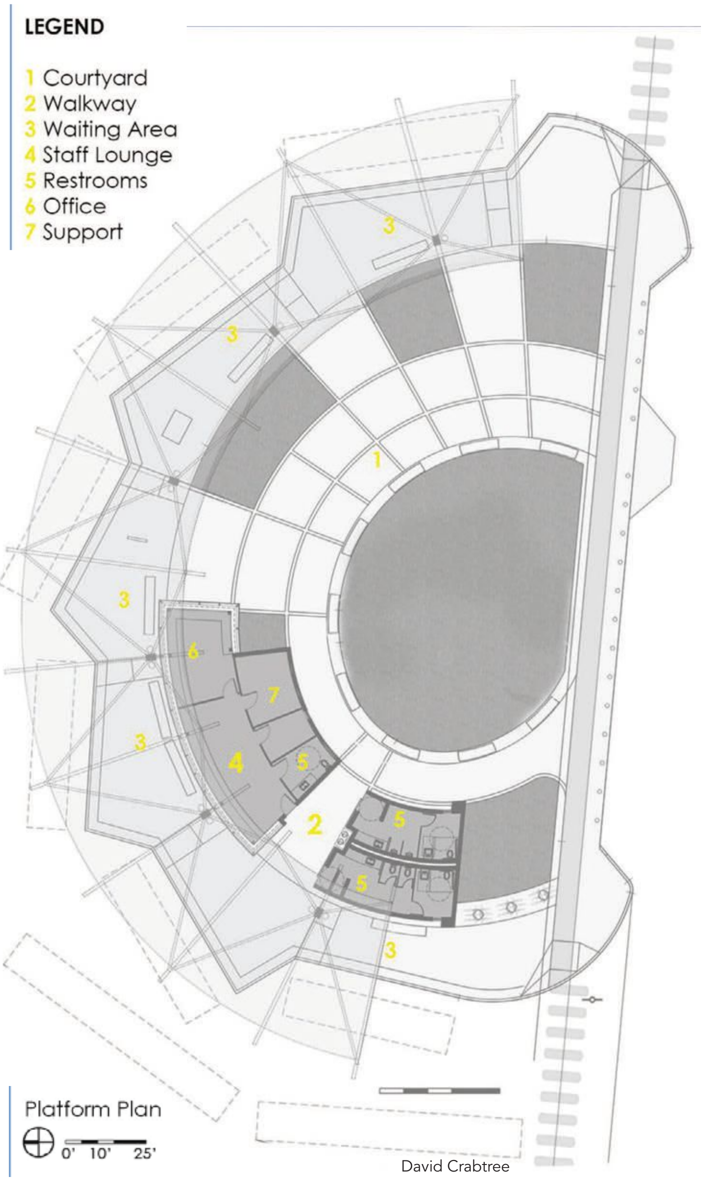
The new facility's layout provides easy bus access, limits crossing points of buses and people and creates a large civic plaza to the south. The design for the canopy demanded a structural solution that could facilitate long spans while minimizing the number of column intrusions, thus creating an open platform with clear views for both pedestrians and bus drivers. The canopy extends 280 ft organized along a structural spine supported by six mast columns and hollow structural section (HSS) supports that cantilever 40 ft out over the driving area.

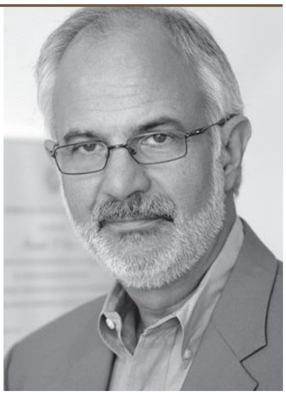
The six mast columns are hammerhead assemblies with 30-in. wide-flange columns fastened to the foundations us-

The new facility's long radial form provides easy bus access, limits crossing points of buses and people and creates a large civic plaza to the south.

A bird's-eye view plan of the transit hub.

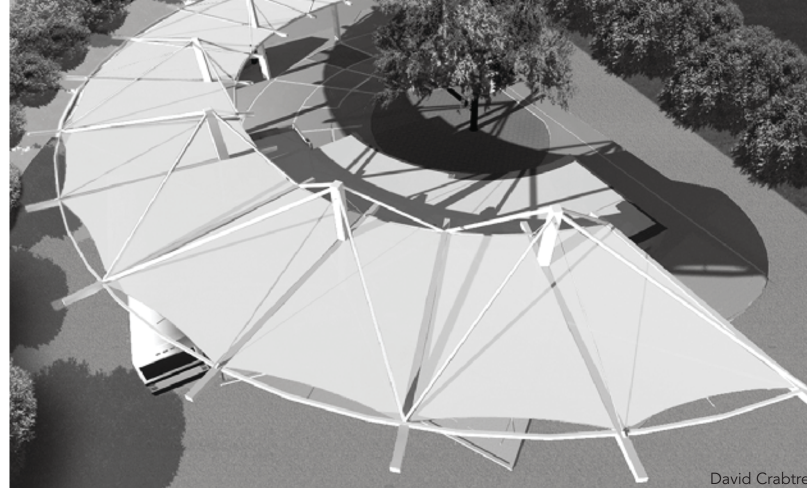
Yellow painted HSS cantilevers 40 ft out over the driving area.





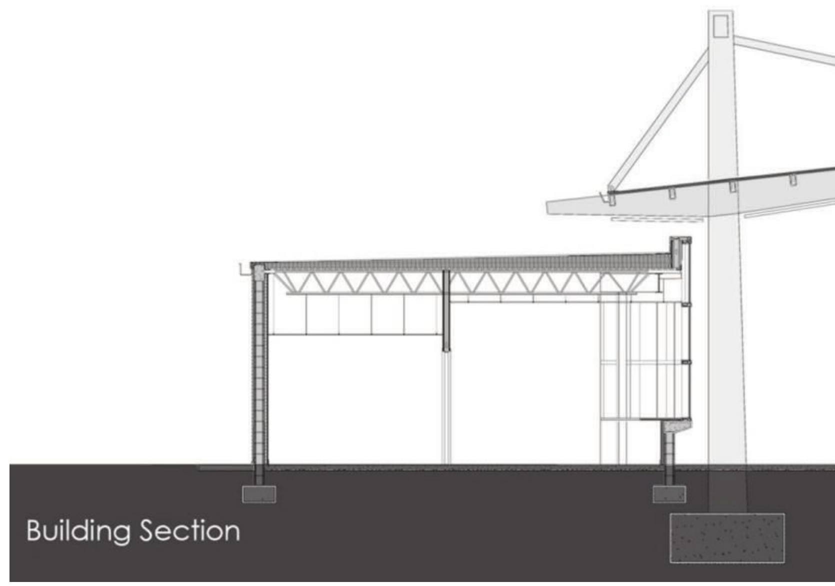
Kevin Ratigan (kevinr@adgusa.org) is a senior vice president with Architects Design Group and **Jim Mehltretter** (jim.mehltretter@mcengineers.com) is a senior principal with Master Consulting Engineers.

➤ The canopy roof was ultimately designed as a corrugated metal canopy with a radial ribbing pattern that compliments the overall expression of the structure at a total weight of 130 tons.



David Crabtree

➤ A cross section of the facility.



ing 24 1½-in.-diameter A307 bolts. The hammerhead beams (W30×99) were tapered by removing the bottom flange, then the web was cut and the bottom flange was shop welded to the web, creating the shape designed by the architect. The bracing members are 6-in.-diameter extra-strong HSS slotted and shop welded to steel knife plates. The top of the hammerhead columns incorporate knife plates designed to accept the stabilization bracing installed at the site. In addition, 10-in.-deep wide-flange beams were rolled to 60-ft and 75-ft radii to create the gentle curvature; the HSS was also rolled to the same radii. Pipes were selected as the bracing members because they were capable of resisting tension forces due to gravity loads and compression forces during Florida's hurricane wind loads.

From Fabric to Metal

Originally conceived as a lightweight tensioned fabric assembly intended to reduce the structural mass and allow for diffuse natural light, the canopy roof was ultimately designed as a corrugated metal canopy with a radial ribbing pattern that compliments the overall expression of the structure at a total weight of 130 tons. The result is a more durable structure that casts a dramatic shadow for those riders escaping the intense Florida sunlight. The 7½-in.-wide long-span roof deck is supported by the rolled steel framing and was selected because it could be painted to the architect's specifications and provide a finished surface on the underside without allowing roofing screws to be visible from below. Research into the dynamics of color application and human perception concluded that the primary yellow results in a slender and elegant profile, while providing immediate identification from vehicles on the adjacent road traveling in excess of 40 mph. ■

Owner
Sarasota Country Transit Authority

General Contractor
Halfacre Construction Company

Architect
Architects Design Group

Structural Engineer
Master Consulting Engineers

Steel Fabricator, Erector and Detailer
All Steel Consultants, Inc.

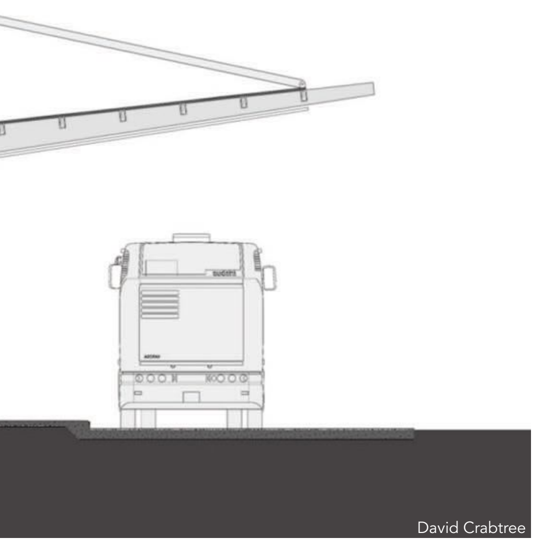
➤ The six mast columns are hammerhead assemblies with 30-in. wide-flange columns fastened to the foundations using 24 1½-in.-diameter A307 bolts.


Greg Wilson


Greg Wilson

▲ Steel made it possible to economically support the various load requirements, including Florida's potential for hurricane winds exceeding 150 mph.

▲ The colors for the station were selected from the transit authority's logo to reinforce the county's brand identity.


David Crabtree


Dan Ionescu

NUCLEAR Design Development

BY SAAHASTARANSHU R. BHARDWAJ, AMIT H. VARMA AND TAHA AL-SHAWAF

A new spec provides guidance for using steel-plate composite walls in nuclear facilities.

IN RECENT YEARS, there has been a move to explore modularized construction methods for nuclear power plants to improve overall cost and schedule.

However, the lack of a U.S.-based design code for steel-plate composite (SC) wall construction was a major impediment to the adoption of these types of assemblies in the U.S. But in 2006, AISC formed an ad hoc subcommittee under the Task Committee on Nuclear Facilities to look at current research and initiate the development of design criteria for SC walls. Resulting from this work, the design provisions for SC walls in safety related-nuclear facilities are included in the recently released Supplement No. 1 to *Specification for Safety-Related Steel Structures for Nuclear Facilities* (ANSI/AISC N690-12). For ease of use, the Supplement has been incorporated into the ANSI/AISC N690-12 document (a free download at **www.aisc.org/epubs**).

Modular SC

So what is modular SC construction? SC walls involve concrete walls reinforced with steel faceplates that are anchored to concrete using steel anchors. The faceplates are connected to each other using tie bars and concrete is poured in between the walls. Modular SC construction reduces the project schedule and labor requirements significantly compared to typical reinforced concrete (RC) walls. Faceplates eliminate the requirement of external formwork and reduce congestion in comparison to RC walls by acting as equivalent reinforcement (no massive reinforcing cages). A typical SC wall section is shown in Figure 1.

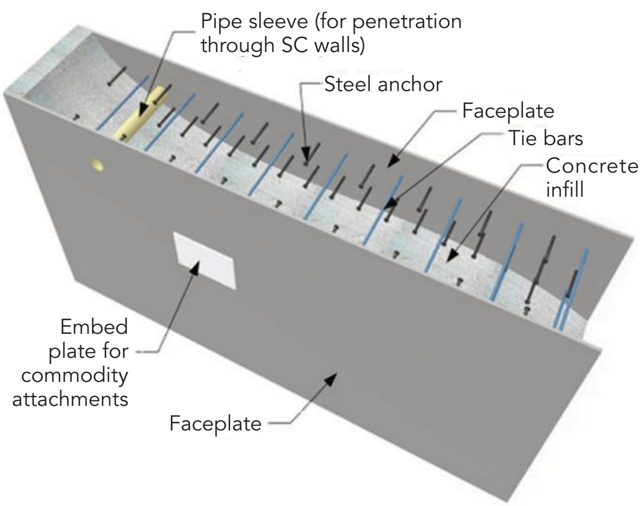


Figure 1. Typical SC wall section.

Saahastaranshu R. Bhardwaj (**sbhardwa@purdue.edu**) is a Ph.D. candidate and **Amit H. Varma** (**ahvarma@purdue.edu**) is a professor, both at the School of Civil Engineering at Purdue University. **Taha Al-Shawaf** (**taha.alshawaf@areva.com**) is a technical consultant with AREVA, Inc., in Naperville, Ill.

Appendix N9 is applicable to the design of SC walls and SC wall connections and anchorages. The experimental database that forms the basis of the provisions is discussed in the commentary to Appendix N9. The appendix is limited to SC walls with two faceplates on exterior surfaces and no additional reinforcing bars. The general requirements of the appendix specify the conditions necessary for applicability of the provisions. Section detailing requirements of the appendix address SC-specific limit states of local buckling, interfacial shear failure, and section delamination.

Organization of Appendix N9

Appendix N9 is organized into four major sections. These sections are further organized into subsections. The sections and subsections of the Appendix are listed as follows:

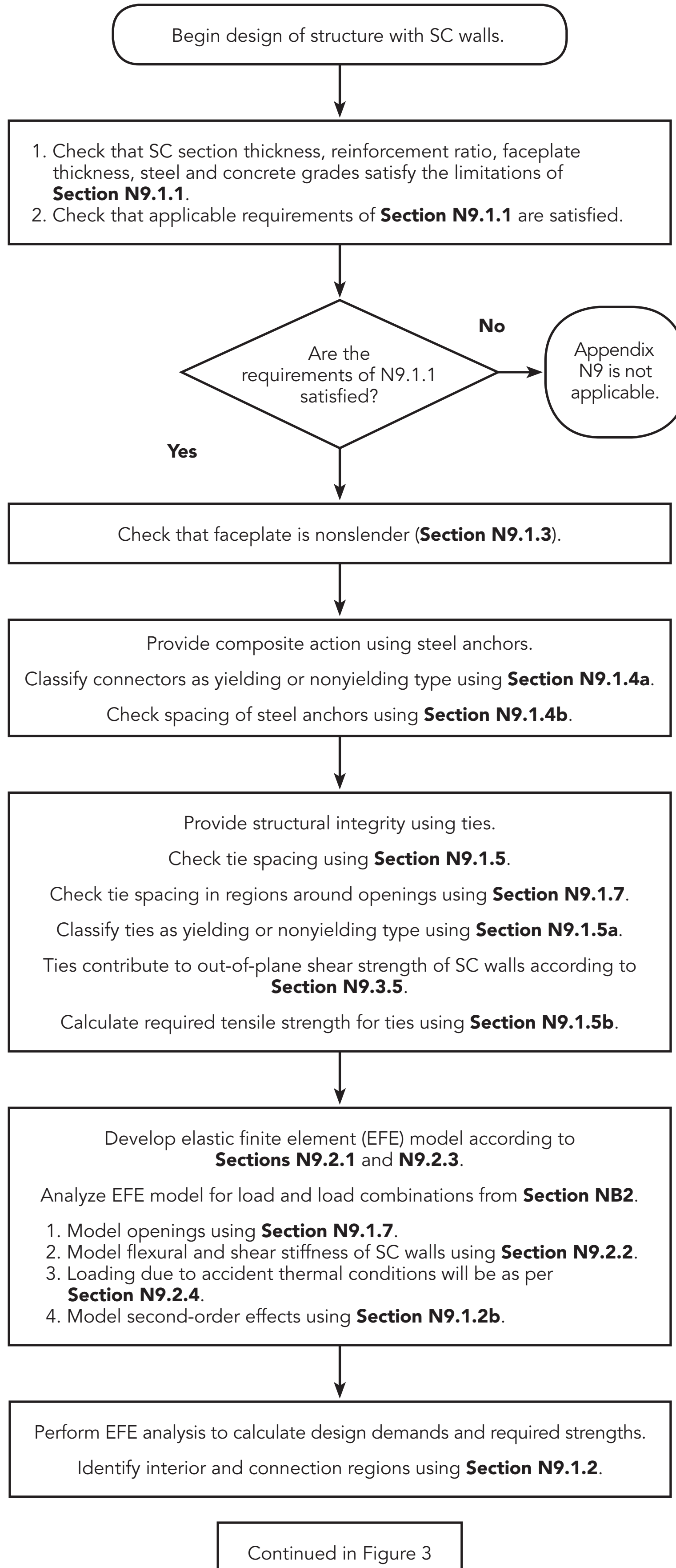
- N9.1 Design Requirements
  - N9.1.1 General Provisions
  - N9.1.2 Design Basis
  - N9.1.3 Faceplate Slenderness Requirement
  - N9.1.4 Requirements for Composite Action
  - N9.1.5 Tie Requirements
  - N9.1.6 Design for Impactive and Impulsive loads
  - N9.1.7 Design and Detailing Around Openings
- N9.2 Analysis Requirements
  - N9.2.1 General Provisions
  - N9.2.2 Effective Stiffness for Analysis
  - N9.2.3 Geometric and Material Properties for Finite Element Analysis
  - N9.2.4 Analyses Involving Accident Thermal Conditions
  - N9.2.5 Determination of Required Strengths
- N9.3 Design of SC Walls
  - N9.3.1 Uniaxial Tensile Strength
  - N9.3.2 Compressive Strength
  - N9.3.3 Out-of-Plane Flexural Strength
  - N9.3.4 In-Plane Shear Strength
  - N9.3.5 Out-of-Plane Shear Strength
  - N9.3.6 Strength Under Combined Forces
  - N9.3.7 Strength of Composite Linear Members in Combination with SC walls
- N9.4 Design of SC Wall Connections
  - N9.4.1 General Provisions
  - N9.4.2 Required Strength
  - N9.4.3 Available Strength

Additions to ANSI/AISC N690-12

In order to incorporate Appendix N9 into ANSI/AISC N690-12, a few additions or updates were made to the existing text of ANSI/AISC N690-12. These modifications include the following:

- American Concrete Institute (ACI), American Society of Mechanical Engineers (ASME) and ASTM International (ASTM) specifications cited in Appendix N9, and not already cited in ANSI/AISC N690-12, have been added to Section NA2.

Figure 2. Flowchart to facilitate use of Appendix N9.



- ASTM materials for plate cited in Appendix N9, and not already cited in ANSI/AISC N690-12, have been added to Section NA3.
- Section NB2 contains the updated load combinations to consider fluid and soil loads. Load factors for some loads have also been updated based on the U.S. Nuclear Regulatory Commission's Regulatory Guide 1.142.
- A reference to Appendix N9 has been added in Section NB3 for design of SC walls for impactive and impulsive loads.
- Provision for welding of SC wall elements to ASME Class MC components have been added to Section NM2. Dimensional tolerances for SC walls during fabrication, fit up, erection of modules, before concrete placement, and after concrete curing have been provided in Section NM2.
- Inspection requirements for SC walls before and after concrete placement and for welding of faceplates have been provided in Section NN6.

Designing SC Walls Using N9

In order to facilitate the use of Appendix N9, a flowchart has been provided in the commentary to Appendix N9. The flowchart has been reproduced in Figure 2 and Figure 3, and discussed briefly below.

In order to design an SC wall structure using Appendix N9, the designer needs to first ensure that the SC wall parameters comply with the requirements of Section N9.1.1. Once these requirements are met, faceplate slenderness requirements of Section N9.1.3 are checked and the steel anchor and tie bar detailing requirements of Sections N9.1.4 and N9.1.5 are then checked. For determining the demands for the SC wall, analysis is performed based on provisions of Section N9.2. The required strengths are compared with available strengths determined per the provisions of Section N9.3. SC wall connections are designed per Section N9.4 using the impactive and impulsive loads per Section N9.1.6. The detailing and fabrication tolerances for SC walls are specified as per Section N9.1.7 and Chapter NM. The quality assurance and quality control of the constructed SC wall is in accordance with Chapter NN.

The development and publication of this new supplement to ANSI/AISC N690-12 provides the first U.S. standard for design and construction of SC wall structures. This will facilitate the use of modular composite construction in nuclear facilities. ■

Supplement No. 1 to Specification for Safety-Related Steel Structures for Nuclear Facilities *is now available for free at **www.aisc.org/epubs**. A session on this topic will be presented at the 2016 NASCC: The Steel Conference, which takes place April 13–15 in Orlando (**www.aisc.org/nascc**).*

Continued from Figure 2

Design Process for SC Walls: Required strengths ≤ Available strengths

1. Calculate required strengths for each demand type using **Section N9.2.5**.
2. Calculate available strengths for each demand type using **Section N9.3**. The sub-sections are:
   (a) Available uniaxial tensile strength using **Section N9.3.1**.
   (b) Available compressive strength using **Section N9.3.2**.
   (c) Available out-of-plane flexural strength using **Section N9.3.3**.
   (d) Available in-plane shear strength using **Section N9.3.4**.
   (e) Available out-of-plane shear strength using **Section N9.3.5**.
   (f) Check available strength for combined forces using **Section N9.3.6**.
      (i) Combined out-of-plane shear demands using **Section N9.3.6a**.
      (ii) Combined in-plane membrane forces and out-of-plane moments using **Section N9.3.6b**.

↓

Design Process for SC Wall Connections

1. Select connection design philosophy and design force transfer mechanisms for connections as per **Section N9.4.1**.
2. Calculate connection required strength in accordance with **Section N9.4.2**.
3. Calculate connection available strength using **Section N9.4.3**.
4. Check connection required strength ≤ connection available strength.

↓

Check SC wall design for impactive and impulsive loads in accordance with **Section N9.1.6**.

↓

Fabrication, Erection and Construction Requirements

1. Specify detailing for regions around openings using **Section N9.1.7**.
2. Specify dimensional tolerances for fabrication of SC wall panels, submodules and modules using **Chapter NM**.

↓

Specify quality assurance/quality control requirements for SC walls in accordance with **Chapter NN**.

↓

End design of structure with SC walls.

Figure 3. Flowchart to facilitate use of Appendix N9 (continued).

General Note:
The elastic finite element model should be made using any system of consistent units. The design demands and required strengths are calculated by performing an elastic finite element analysis. However, before using the equations in this Appendix, the units of the calculated design demands and required strengths should be made consistent with the corresponding units in the Appendix equations. For example, the units for design demands and other material parameters used in the equations of this Appendix are as follows:
(a) The required and available out-of-plane moment strengths are in kip-in./ft (N-mm/m).
(b) The required and available membrane in-plane force strengths, and out-of-plane shear force strengths are in kip/ft (N/m).
(c) The modulus of elasticity for steel and concrete are in ksi (MPa).

A look at the basics of curving steel—a process that is anything but basic.

There's More than One Way to BEND A BEAM

BY GEOFF WEISENBERGER

Photos are from Paramount Roll and Forming, Inc.'s facility in Santa Fe Springs, Calif.

▲ Various examples of curved steel, ready for fabrication.

Geoff Weisenberger (**weisenberger@aisc.org**) is *Modern Steel Construction*'s senior editor.

WHY FRAME A BUILDING with straight structural steel when you can use curved steel instead?

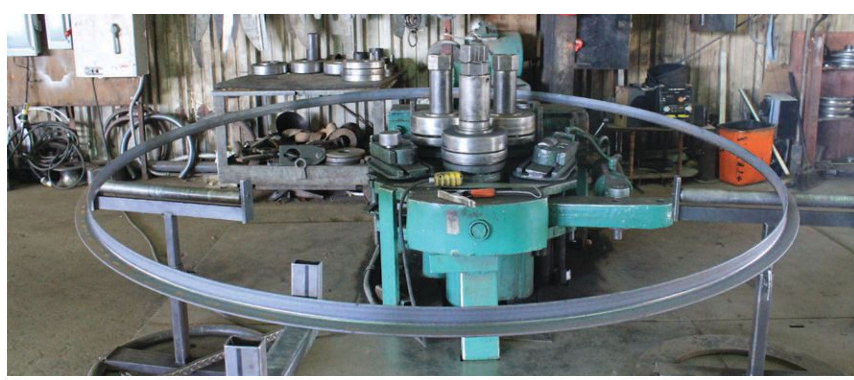
Obviously, I say that in jest. But for those applications where a certain amount of curved steel is desired, there are a few things to know. First of all, not all curved steel is created equal—or at least not curved in the same exact way. While different bender-rollers might use similar equipment, it's a matter of how they use that equipment that sets them apart.

"Much of rolling and bending is an art and not a science—one company's ability to use a certain machine in a certain method," says Barry Feldman, president of bender-roller Kottler Metal Products. "No matter how good the machine is, you need a skilled operator in order to achieve a quality bend. You can't just put the steel in and push a button and expect it to come out perfectly. On many of these jobs, especially for architectural applications, you may have 100 pieces, but they may have different radii and different degrees."

➤ Bending rectangular HSS using the heat induction method.

▲ Curved steel plate, with the welds visible.

▼ Curving steel angle via the roll/pyramid bending method.



Means and Methods

Secondly, there a multiple methods used to bend steel. Two of the most common types of bending are rotary-draw/compression bending and roll bending or pyramid bending. The latter gets its name from the fact that the rolling machine has three adjustable rolls in a triangle or pyramid configuration. Tighter roll spacing results in a tighter radius. Here's how it works: The beam is placed in the rolling machine and the operator adjusts the three rolls to the proper spacing before starting the bending process. The operator slowly begins rolling, and he or she frequently checks the beam for distortion of the web and flanges in these early passes. Several additional passes are carried out, with the operator measuring the overall radius after each pass to check the beam's progress.

Rotary-draw/compression bending is a different animal and is mainly used for complicated bends in the machine and parts industry. In this process, the structural member is bent by rotating it around a die. The member is clamped into a form and then is drawn through the machine until the bend is formed, producing a very tight radii.

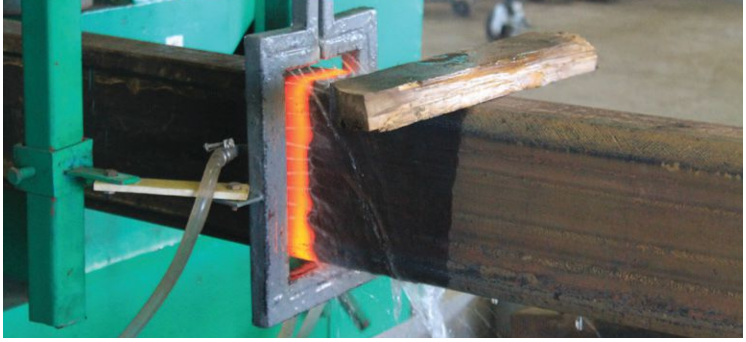
While both of these methods curve steel in the cold condition, another method—heat induction—calls for the steel to be heated while it is being curved.

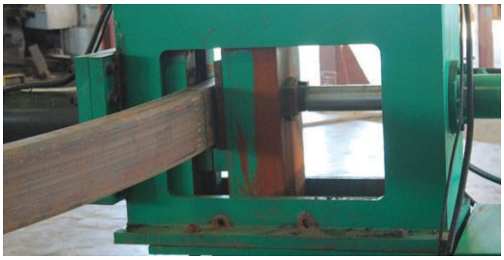
"Heat induction bending relaxes the steel during the bending process, which helps achieve tight radii as well as minimizes distortions," explains Ken Moscrip, president of Paramount Roll and Forming. In this process, an electric heating coil is placed around the bend point. Once the steel reaches a certain temperature range, pressure is applied to the front end of the member in order to bend it to the proper radius. When bending is completed, the steel is quenched via water spray.

Two other methods—incremental/camber bending and rotoform bending—can also be used to curve steel. The first

➤ HSS passing through an induction coil.

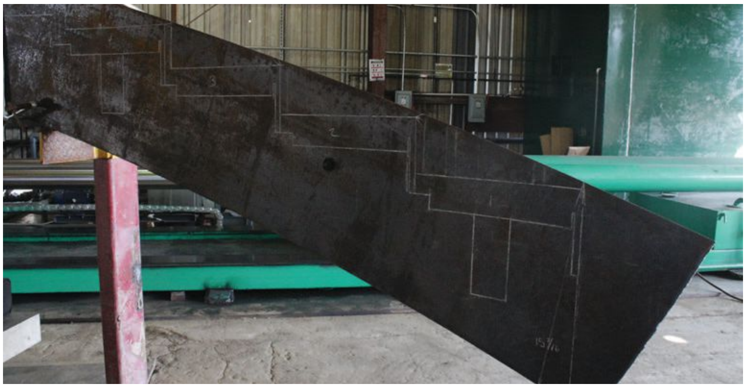
➤ Specialized equipment further down the member from the coil slowly applies pressure to the steel to bend it into shape.





AISC Associate Member Bender-Rollers
Albina Company, Inc.
Bendco, Inc.
Bowers Fabrication
BendTec, Inc.
Chicago Metal Rolled Products Company (two locations)
Greiner Industries, Inc.
Hodgson Custom Rolling, Inc.
Hornsby Steel
Kottler Metal Products
Kubes Steel, Inc.
Max Weiss Company
Metals USA (four locations)
Paramount Roll and Forming
Shaped Steel, Inc.
SIMS Steel, Inc.
Whitefab

Visit **www.aisc.org/benders** for contact information for each of these companies.



➤ ➤ Steel for two sets of curved stairs. A layout for the steps is shown above while a more completed set is shown below. Both use special jigs in the shop to mimic the actual assemblies in the field.

involves bracing steel at two ends and applying pressure at a third point via a hydraulic ram or press and is particularly useful for curving steel to high radii. The second uses a specialized process to extrude steel from the straight condition into a bend and is the most flexible when it comes to radius parameters.

Bending Inquiries

Given the various methods of bending and the infinite possibilities that are achievable, designers are always looking for insight. Below is a handful of questions that AISC's Steel Solutions Center has received recently regarding bending steel. While AISC can provide some general guidance, it's usually best to contact a bender-roller to tap their expertise.

I am looking for a data table giving the minimum rolled radius for wide-flange steel. Does AISC have any information on wide-flange steel radius minimums for rolling?

Rigid guidelines for the minimum bending radius are not available because it is dependent on several variables, such as:

- Axis of curvature
- Cross-sectional shape of the member
- Bending method used by the bender-roller
- The equipment limitations of the bender-roller
- Level of acceptable cross-sectional distortion
- Level of acceptable cold-working of the material

These limitations should be discussed with the bender-roller who will provide the service; however, some general guidelines are on page 2-37 of the AISC 14th Edition *Steel Construction Manual*. The AISC website also has a publications page for information related to curved members at **www.aisc.org/curvedsteel**.

According to the general guidelines for cold bending on page 2-37 of the 14th Edition *Steel Construction Manual*, sweep (curving about the weak axis) can be provided to "practically any radius desired." The minimum radius for camber (curving about the strong axis) by cold bending of members up to a nominal depth of 30 in. is between 10 and 14 times the member depth.

Are there any tolerances on curved beams imposed by AISC? The ASTM A6 tolerances are primarily for straight beams and really do not cover the web deformation. Are there any guidelines for ensuring that deformation does not happen?

There are limited tolerances for curved members in the 2010 AISC *Code of Standard Practice* (a free download at **www.aisc.org/code**). According to Section 6.4.2, "For curved structural members, the variation from the theoretical curvature shall be equal to or less than the variation in sweep that is specified for an equivalent straight member of the same length in ASTM A6/A6M." Other acceptable tolerances, such as any cross-sectional distortion, are not generally available because they are dependent on whether the member is architecturally exposed structural steel (AESS) and any effect they may have on the member strength. AESS tolerances are discussed in Section 10 of the *Code of Standard Practice*. The actual geometric imperfections for rolled members are dependent on several factors, including:

- Cross-sectional shape of the beam
- Bending radius
- Bending axis
- Bending method used by the bender-roller
- Equipment limitations of the bender-roller

It is best to discuss the required tolerances with the bender-roller who will provide the service—and be sure to add the required tolerances to the contract documents to ensure that you get what you are asking for.

What is the maximum geometric camber that I can specify for a W27 rolled beam?

The capabilities of bender-rollers and fabricators vary, as does the equipment used and the cost. A tighter radius can often be obtained using a more sophisticated and costly process. It is best to speak with a bender-roller and a fabricator to get their opinions.

Is it possible to put a 90° bend in a pipe?

Yes. The key factor is the radius and the bending method used. You will need to contact a bender-roller or a fabricator to discuss the limits, options and costs.

Best Bend

What's the best way to bend steel? Several factors determine the best technique, including the overall member size, web and flange thickness or HSS wall thickness, radius requirement and end application of the material. Also, keep in mind that varying amounts of extra material are required at one or both ends of the member, depending on the process used; you don't want to have to splice additional material to one or both ends. Talk to a bender-roller about the best options for your particular application as well as their capabilities.

As with steel fabricators, getting benders involved early in the process will help you achieve the best results for your next curved steel masterpiece.

"Including benders early in a project can help assist with what is and isn't feasible concerning a design, and can help save time and money as a project moves forward," says Brian Smith, president of Albina Company, Inc. "And many benders have the necessary equipment and abilities to bend whatever is required in very fast time frames. We are used to working and performing under considerable pressure." ■

Something BOLTED this Way Comes

BY TOBY ANDERSON

The toil of revising the F1554 standard will help reduce the trouble of specifying anchor bolts.

THERE ARE A LOT of memorable scenes from *Macbeth*.

The knife floating through the air, the "Tomorrow and tomorrow and tomorrow" soliloquy and Lady Macbeth trying to scrub her hands clean all come to mind.

For me, the scene with three witches and their bubbling cauldron is the one that stands out the most. And it's not just because it's such a great scene, but also because the brew's disparate and unfamiliar ingredients mimicked the disparate components that came together in a recently completed standard. This past November, colleagues representing bolt suppliers have completed a revision of ASTM F1554-15, *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*. The group took into consideration strength, ductility, the effects of heat treatment and cold work, the capabilities of producers and the needs of designers—albeit with fewer poisoned entrails and eyes of newt.

Prior to 1994, designers commonly used other bolt standards to call out anchor rods, because there was no standard written specifically for them. ASTM adopted F1554 that year and AISC soon recommended it for anchor rods, but it has seen only minor revisions since then. As design codes adopted it, engineers and other specifiers designed with it, and suppliers naturally have seen increased requests. However, over the last few years, designers and producers noticed a few inconsistencies that complicated what ought to be clear requirements. Hearing such concerns, the ASTM-led task group met, drafted and completed an important update to this increasingly vital standard. With the goal in mind of a clear and economical standard that meets engineering requirements, the group made many changes; a few of the important ones are described here.

Toby Anderson (**baybolt@pacbell.net**) is general manager of Bay Bolt.

New Ingredients

First, ACI 318-11 Appendix D design provisions defined ductile anchor rods with multiple parameters: reduction of area (ROA) that varied by grade and diameter and two measures of elongation. Two grades within F1554 straddled the requirements, with some parameters meeting the ductility limits while others did not. This led to designers using the lower resistance factors required for non-ductile materials, a 15% decrease in available strength. The "brittle" designation (vs. "ductile" anchors) may have necessitated additional design considerations, unless users specified higher reduction-of-area requirements than those in F1554. After consulting mills, manufacturers, bolting suppliers and ACI, the task group found that Grade 55 bars in most diameters were certified already to ROA's higher than F1554 required. To avoid the complications of the "brittle" designation, then, stakeholders agreed to raise F1554 Grade 55 ROA requirements to a minimum of 30% for all diameters.

The more recent standard, ACI 318-14, moved the definition of "steel element, ductile" to Section 2.3 ("Terminology"), but kept it essentially the same: "element with a tensile test elongation of at least 14% and reduction in area of at least 30%."

While the actual term "brittle" is not contained in the ACI 318, one can assume that if an anchor does not meet the definition of "steel element, ductile," it may be classified as brittle.

Due to ACI 318's definition, it still is possible, technically, to have a brittle Grade 105 bolt in ASTM F1554. Therefore, if a ductile Grade 105 is required, designers need to modify their contract specifications to require a 14% minimum elongation. (All other F1554 grades are ductile.)

Second, the original standard did not anticipate the growth of cold-drawn threaded rod ("all-thread") as an anchoring product, and certain sections allowed such rods' *predrawn* tensile properties to qualify its *finished* tensile properties. Conceivably, a finished Grade 36 all-thread rod—e.g., after it had been drawn and threaded—could have exceeded the standard's upper limit for ultimate tensile strength, even though its original raw material did not. The task group revised and clarified these sections to make the requirement clear: All tensile properties are to be determined only after all cold-drawing (and heat-treating, if performed) is completed. (What is not new is that Grades 36 and 55 anchor rods in sizes through 1½ in. and Grade 105 rods through 1¼ in., when tested full-size, still need to meet only yield strength and ultimate tensile requirements. ROA and elongation need not be reported for those sizes.)

Next, answering frequent queries, the group clarified that Grade 36 anchors are considered weldable, and to help ensure such weldability for even cold-drawn all-thread (which are often cut into shorter studs for anchoring), they lowered the grade's maximum carbon content by 0.01% (to 0.25%). In addition, recognizing a current industry process, the members revised the standard to allow all-thread rod manufacturers to perform qualifying tensile tests on coupons that have been drawn to pitch diameter, but not yet threaded. (Tensile properties are then extrapolated from the standard's full-size test requirements.)

The task group made minor revisions as well. They provided dimensional guidance for hex-head anchors, deleted extraneous metric equivalents, replaced references to "certifications" with "test reports" (the ASTM Fastener Committee's preferred usage) and consolidated and standardized certain sections (which had no other than mere editorial impact). Finally, the group deleted sizes under ½ in., having found no structural anchoring applications that specify such small diameters.

Ghosts of the Past

In addition to the changes in F1554-15, this is a good place to review some other areas of this specification. Like other ASTM product standards, F1554 includes an "ordering information" section (Section 5) that includes: quantity, product name, ASTM Designation and year, grade and class, copper content, nominal diameter and thread pitch, bolt length, thread length, head type (if required), hook angle and hook length (if required), coatings, number of nuts, number of washers, source inspection requirements, color marking (if different from the standard's), test reports, supplementary requirements (if required) and special packaging requirements (if required).

Something else that is not new: The actual minimum body diameters are smaller than the nominal diameters, and they are smaller for rolled threads than for cut threads. Designers ought to account for this variance. (See the adjacent table.) AISC and ACI have different ways to design threaded rod and in light of the minimum diameter for F1554, yield on the body of the rod might need to be checked. On the next page is an example using a 2-in.-diameter Gr 105 threaded rod:

F1554 Anchor Rod Minimum Body Diameters[A]

Nominal Diameter, in.	Threads/in.	Body Diameter, min, in.		
		Rolled Threads[B]		Cut Threads[C] Classes 1A and 2A
		Class1A	Class2A	
Unified Coarse Thread Series (UNC)				
½	13 UNC	0.4411	0.4435	0.4822
⅝	11 UNC	0.5561	0.5589	0.6052
¾	10 UNC	0.6744	0.6773	0.7288
⅞	9 UNC	0.7914	0.7946	0.8523
1	8 UNC	0.9067	0.9100	0.9755
1⅛	7 UNC	1.0191	1.0228	1.0982
1¼	7 UNC	1.1439	1.1476	1.2232
1½	6 UNC	1.3772	1.3812	1.4703
1¾	5 UNC	1.6040	1.6085	1.7165
2	4½ UNC	1.8385	1.8433	1.9641
2¼	4½ UNC	2.0882	2.0931	2.2141
2½	4 UNC	2.3190	2.3241	2.4612
2¾	4 UNC	2.5686	2.5739	2.7111
3	4 UNC	2.8183	2.8237	2.9611
3¼	4 UNC	3.0680	3.0734	3.2110
3½	4 UNC	3.2110	3.3233	3.4610
3¾	4 UNC	3.5674	3.5730	3.7109
4	4 UNC	3.8172	3.8229	3.9609
8 Thread Series (8 UN)				
1⅛	8 UN	. . .	1.0348	1.1004
1¼	8 UN	. . .	1.1597	1.2254
1½	8 UN	. . .	1.4093	1.4753
1¾	8 UN	. . .	1.6590	1.7252
2	8 UN	. . .	1.9087	1.9752
2¼	8 UN	. . .	2.1584	2.2251
2½	8 UN	. . .	2.4082	2.4751
2¾	8 UN	. . .	2.6580	2.7250
3	8 UN	. . .	2.9077	2.9749
3¼	8 UN	. . .	3.1575	3.2249
3½	8 UN	. . .	3.4074	3.4749
3¾	8 UN	. . .	3.6571	3.7248
4	8 UN	. . .	3.9070	3.9748

[A] Extracted from ASME B 1.1.
[B] Minimum body diameter is the same as minimum pitch diameter.
[C] Minimum body diameter is the same as minimum major diameter.

AISC:

Tensile from Table J3.2 $F_{nt} = 0.75\ F_u$

$R_n = F_n A_b = 0.75\ (125)(3.14) = 294$

Design Strength $= \phi R_n = 0.75(294) = 221\text{k}$

ACI:

$R_n = F_{nt}(A_b)$ where $F_{nt} = F_u$ and $A_b =$

Net tension area of the threaded bolt = 125 (2.5) = 312

(see *Steel Construction Manual* Table 7-17 for A_b value.)

Design strength $= \phi R_n = 0.65(262) = 203\text{k}$

AISC yield on the body:

$R_n = F_y\ A_{min} = 105[(\pi\ (1.83852/4)] = 278.7$

Design strength $= \phi R_n = 0.90\ (278.7) = 250\text{k}$

During the F1554 revision process, ASTM, AISC and ACI members endeavored to gather many industry voices, realizing that creating and amending product standards is much like creating a forging (though generally at lower temperatures and with less slag). The best standards are clear, concise and forged by groups that have all the information-consumers—mills, manufacturers, distributors, specifiers, and end users—represented, and where each listens, collaborates and contributes their bit to the chemistry (or the hammering).

Writing the Play

This new publication is the result of disparate industry players joining forces to coordinate, unify and rationalize the standard for anchor rods. I encourage all industry members to join the standards organizations linked to their specialty and to participate in similar collaborative processes, if only to combat the occasional impression that standards are brewed up with dragon scales and wolf teeth and foisted on an industry to create toil and trouble. ■

F1554 Anchor Rod Tensile Properties[A]

	Grade		
	36	55	105
Tensile strength, ksi	58–80	75–95	125–150
Yield strength, min, ksi (0.2% offset)	36	55	105
Elongation in 2 in., min, % (machined specimens)[A]	23	21	15
Elongation in 8 in., min, % (bar stock)[A]	20	18	12
Reduction of Area, min %	40	30	45

[A] Elongation and ROA need not be reported for rods tested full-size.

F1554 Anchor Rod Welding Colors

Grade	Color
36	Blue
55	Yellow
55- Weldable[A]	Yellow (projecting end) and White (encased end)
105	Red

[A] When Supplementary S1 is used.

news

CERTIFICATION

Updated Fee Schedule for AISC Certification Programs

AISC has introduced a new fee schedule for its Certification programs. "This new fee schedule reflects changes and continuing improvements to the program and is the first time we've increased fees in more than five years," said Roger Ferch, president of AISC.

In the past, fees were based on program type (such as steel building structures or steel bridges). The new fee schedule allows participants to pay for a base certificate with a supplementary fee for additional certificates/program certifications. "This is a more equitable solution that better reflects the actual costs for all program participants," Ferch explained. For an AISC Member certified to the building fabrication program, the increase will range from 2% to 7% over the three-year certification cycle.

"Recognizing the higher cost of auditing a larger company, the fee schedule includes higher fees for larger companies," stated Jacques Cattan, the AISC vice president responsible for Certification. "And reflecting the contributions of AISC members, the fee schedule includes a 35% discount for them."

The new fee schedule can be found at **www.aisc.org/certification**. For questions, please contact Jacques Cattan at **cattan@aisc.org** or 312.670.5436.

SUSTAINABILITY

AISC and Steel Organizations Represent the Material of Choice at Greenbuild

AISC, the Steel Recycling Institute (SRI) and the Steel Market Development Institute (SMDI) teamed up to showcase the sustainable benefits of steel at this year's Greenbuild. The premier event for sustainable building, the show took place in Washington, D.C., in November and drew close to 20,000 attendees—and many of them visited the steel booth to discuss how they design and build with steel as well as to learn about its sustainable attributes.

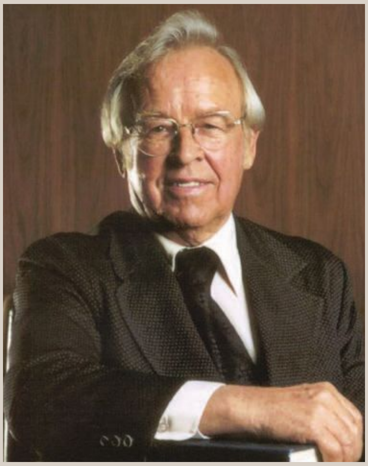
"Roughly a third of the steel in North America—and half of the steel produced worldwide—is used in construction," said SMDI's vice president of sustainability, Mark Thimons. "Steel is so ubiquitous that even people who spend their days immersed in green building topics rarely stop to consider how well it demonstrates meaningful sustainability throughout its life cycle. That's one of the things we're trying to emphasize here."

AISC vice president John Cross commented that while attendees are clearly focused on the environmental performance of buildings, they were still wary of how upcoming elections may affect the green building movement. "Many attendees seemed to have an unsettled attitude toward the twin concerns of the level of traction that will be gained by LEED V4 in the marketplace and the impact that the changing administration will have on government policy related to environmental and sustainable concerns—particularly if a candidate is elected that rejects the relationship between CO_2 emissions and global climate change," he said.

One prominent topic at the show was the continuing trend of millennials moving into cities—a clear opportunity for new multistory construction as well as structural rehabilitation and expansion of existing buildings. Another was that of resiliency/environmentalism as a societal issue. Academy Award-winning director James Cameron, a keynote speaker, stressed the idea that climate change could further destabilize the parts of the world where it will cause the most damage, but noted that business can lead the way in addressing the problem and that the bottom line can't be a company's only consideration.

USGBC and GBCI CEO Rick Fedrizzi, who will be succeeded by current COO Mahesh Ramanujam at the end of 2016, echoed that sentiment at the show's opening plenary session, pointing out that business/manufacturing and green are not mutually exclusive. "The environment and the economy are tied together," he stated. "They share a common enemy: waste."

People and Firms



Sanford H. High (1907-1983) has been posthumously recognized by the **American Road and Transportation Builders (ARTBA) Foundation** with an induction into the organization's Transportation Development Hall of Fame; he was inducted in the Innovators category. The posthumous honor was one of three bestowed at the ARTBA annual meeting.

High was the founder of **High Welding Company** in Lancaster, Pa. (the forerunner to **High Steel Structures**) and pioneered the concept of welded, rather than riveted, bridges. ARTBA recognized High for "pioneering the welded bridge concept, saving time and money for cash-strapped highway departments during the Great Depression." High convinced skeptical engineers that highway bridges presented a new frontier for welding instead of riveting.

Founded in 1931, High Welding Company grew with regard to the number of workers and job complexity. In the late 1950s automated welding equipment was adopted, revolutionizing heavy girder construction and leading the movement to faster, lower-cost submerged arc welding, the predominant process used by High Steel today. (High Steel is an AISC/NSBA Member, Certified fabricator and Advanced Certified steel erector.)

PUBLICATIONS

New Nuclear Spec Now Available

A new Supplement No.1 (ANSI/AISC N690s1-15) to the AISC *Specification for Safety-Related Steel Structures for Nuclear Facilities* (ANSI/AISC N690-12) is now available. ANSI/AISC N690, a companion standard to the AISC 2010 *Specification for Structural Steel Buildings*, applies to the design of safety-related steel structures and steel elements in nuclear facilities.

"The new AISC N690 supplement adds specifications for the design of steel-plate composite (SC) walls, which is one of the most important developments for the nuclear industry in several years," said Amit Varma, professor in the School of Civil Engineering at Purdue University and vice chair of the AISC Task Committee 12 Ad Hoc Subcommittee, which developed the new supplement. "Several new nuclear plant designs are using SC walls for their safety-related structures. The publication of this industry consensus code/standard, developed based on international research, will facilitate the design, regulatory review and eventual licensing of new nuclear plants in the U.S. and abroad."

The 2015 supplement has been seamlessly integrated into the 2012 standard for ease of use. The complete document is available as a free download at AISC's website (please visit **www.aisc.org/specifications**). A limited number of printed copies are also available for purchase in the AISC bookstore for $12.50 (AISC members) and $25.00 (nonmembers), plus shipping and handling.

For more on the new specification, see "Nuclear Design Development" on page 48.

NASCC

NASCC Registration Now Open

Registration is now open for the 2016 NASCC: The Steel Conference, which will take place April 13-15 in Orlando at the Gaylord Palms Convention Center.

The Steel Conference is the ideal place for structural engineers, steel fabricators, detailers and erectors to learn about structural steel design and construction, to interact with their peers and to see the latest products for steel buildings and bridges. It offers more than 100 technical sessions and is the premier educational event for structural engineers, fabricators, erectors and detailers.

In addition to practical seminars on the latest design concepts and construction techniques, the conference features an extensive trade show (displaying products ranging from structural software to machinery for cutting steel beams) and plentiful networking opportunities. It's a once-a-year opportunity to learn the latest techniques, 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.

The March issue of *Modern Steel* will feature a handful of session preview papers, and the April issue will feature the full exhibitors list. To view the complete schedule, review travel information, register and more, visit **www.aisc.org/nascc**.

STUDENT COMPETITIONS

Registration Open for Global Steelmaking Competition

Registration is open for the 10th annual steelChallenge, hosted by the World Steel Association. The competition challenges students and young industry professionals from around the globe to test their skills in steelmaking to win various prizes and recognition in the steel industry.

Participants will first compete in four regions: North and South America; Europe, CIS, Middle East and Africa; Asia and Oceania; and China. This regional round of the competition will take place online at **www.steeluniversity.org** over a 24-hour period on January 20, where competitors will be tasked to use an electric arc furnace steelmaking simulation to produce a grade of steel that meets technical requirements at the lowest total cost.

The regional champions and best performing runners-up from the "Student" and "Industry" categories will be invited to compete in the World Championship, scheduled for April 11 in London, where they'll use a designated simulation during a two-hour period to produce the single best result for the simulation. At the end of the competition, the awards ceremony will take place in the presence of leaders from steelmaking companies worldwide.

Competitors must register for the steelChallenge by January 19. More information about the competition is available at **www.steeluniversity.org.**

CORRECTION

A photo in the December 2015 article "Up-Tempo Bridge Construction" (lower-right photo on page 45) was incorrectly identified as the Massachusetts 93Fast14 project and credited to CME Associates. The photo is actually of the Milton-Madison Bridge over the Ohio River between Kentucky and Indiana and should be credited to Walsh Construction.

ENGINEERING JOURNAL

First Quarter 2016 *EJ* Now Available

The first quarter 2016 issue of AISC's *Engineering Journal* is now available at **www.aisc.org/ej**, where you can view, download and print the current digital edition. Articles in this issue include:

➤ **Analysis and Design of Stabilizer Plates in Single-Plate Shear Connections**
By Patrick J. Fortney and William A. Thornton

Single-plate shear connections experience some magnitude of torsional moment, either due to the lateral torsional buckling phenomena or due to the effects of lap eccentricity. When the required torsional strength of the connection exceeds the available torsional strength of the connection, the designer has two options: alter the geometry of the connection to increase the torsional resistance of the connecting plate or provide stabilizer plates. This paper presents recommendations for the analysis with regard to appropriate stabilizer plate cross-sectional dimensions and the attachment of the stabilizer plate to the connecting material and support. Three different types of stabilizer plates are presented along with recommendations for the design and detailing of the stabilizer plates; the impact that each type has on the design of the single-plate shear connection and the supporting column is presented as well.

Keywords: nodal bracing, single-plate shear connections, stabilizer plates, stiffener plates

➤ **Connection Design Recommendations for Improved BRBF Performance**
By Keith D. Palmer, Charles W. Roeder and Dawn E. Lehman

Numerous component tests on buckling-restrained braces (BRBs) have demonstrated their approximately symmetric tension and compressive capacities, stable cyclic behavior and large (component) ductility prior to core fracture. These properties make them suitable ductile fuses for seismic design. Experiments on buckling-restrained braced frame (BRBF) systems show that the inelastic axial deformation capacity of BRBs may be compromised by system performance demands. Prior test results are reviewed, and an analytical study using high-resolution models, which were validated with prior test results, is used to develop mitigation strategies for the damage. Design recommendations to mitigate damage and improve system performance are developed.

Keywords: buckling-restrained braced frames, ductile fuse, gusset-plate connection, core fracture

➤ **Finite Element Modeling of Steel Moment Connections with Fracture for Structural Fire Analyses**
By Mina Seif, Therese Mcallister, Joseph Main and William Luecke

Performance-based methodologies to evaluate the fire performance of structures are needed to move beyond the prescriptive procedures currently in use, which cannot be used to determine actual structural performance in fire. Analytical methods are needed for simulating the performance of structural systems, including connections, subject to realistic fire effects. Framing connections may be subject to large, unanticipated deformations and loads during fire events, and connection failure may lead to other failures or local collapse. This paper presents the development of detailed finite element models of typical moment connections for steel-framed structures. These detailed models incorporate temperature-dependent material models that have been calibrated against available test data from tensile coupons, including the modeling of necking behavior and fracture. Connection performance at ambient and elevated temperatures is evaluated, and dominant failure modes are identified.

Keywords: plastic strain, fracture, erosion strain, finite element analysis, material modeling, structural fire effects

➤ **Fatigue Testing and Retrofit Details of High-Mast Lighting Towers**
By Ryan J. Sherman, Matthew H. Hebdon and Robert J. Connor

Fatigue cracking has been the cause of a number of high-mast lighting tower (HMLT) failures throughout the United States. In almost every case, forensic evaluations have shown cracking initiates and propagates due to wind-induced fatigue at mainly the base plate–to–tube wall connection detail or the hand-hole weld detail. Simply replacing the towers is not an economically feasible alternative because thousands of HMLTs are in use along major highways across the United States. As a result, strategies to retrofit existing HMLTs are needed. Results from laboratory testing performed on two HMLT retrofit configurations are presented. The retrofit strategies are employed without removing the pole from the foundation using simple bolting techniques and moderately skilled labor, providing cost savings for owners and increasing safety for the motoring public.

Keywords: high-mast lighting tower (HLMT), fatigue, retrofit, sign structure

PUBLICATIONS

Spec and Seismic Provisions Available for Public Review

The drafts of the 2016 AISC *Specification for Structural Steel Buildings* and the 2016 AISC *Seismic Provisions for Structural Steel Buildings* will be available for public review until February 1. Both specifications, along with the review forms, are available for download at AISC's website; visit **www.aisc.org/publicreview**. (You can also order a hard copy—for a $35 charge—by calling Janet Cummins at 312.670.5411.)

Please submit comments to Cynthia J. Duncan, director of engineering, at **duncan@aisc.org** by February 1 for consideration.

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LATE MODEL STRUCTURAL STEEL FABRICATING EQUIPMENT

Ficep 2004 DTT CNC Drilling & Thermal Coping Line, 78-3/4" x 24" Max. Beam, 3-Drill, Ficep Arianna CNC Control, 2003 **#20382**
Ficep TIPO A31 CNC Drill & Thermal Cutting System, 10' x 20' x 5" Max. Plate, Ficep Minosse CNC, 2009 **#25937**
Controlled Automation ABL-100-B CNC Flat Bar Detail Line, 143 Ton Punch, 400 Ton Single Cut Shear, 40' Infeed, 1999 **#24216**
Controlled Automation 2AT-175 CNC Plate Punch, 175 Ton, 30" x 60" Travel, 1-1/2" Max. Plate, PC CNC, 1996 **#23503**
Peddinghaus F1170B CNC Plate Punching Machine, 170 Ton, Ext Tables, Fagor CNC, 30" x 60" Trvl., Triple Gag Head, 2005 **#19659**
Peddinghaus FPB1500-3E CNC Plate Punch with Plasma, 177 Ton, Fagor 8025 CNC, 60" Max. Width, 1-1/4" Plate, 1999 **#25161**
Controlled Automation BT1-1433 CNC Oxy/Plasma Cutting System, 14' x 33', Oxy, (2) Hy-Def 200 Amp Plasma, 2002 **#20654**
Peddinghaus Ocean Avenger II 1000/1B CNC Beam Drill Line, 40" Max. Beam, 60' Table, Siemens CNC, 2006 **#25539**
Peddinghaus FDB1500B CNC Plate Drill with Oxy Cutting Torches, 177 Ton, 60" Plate Width, Fagor CNC, (3) Drill Heads, 2001 **#25718**

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Commercial Metals Company is seeking an Assistant Structural Detailing Manager who will play a key role by assisting in scheduling work, working within budgets and managing projects with other departments including Project Management and the Fabrication Shop. Requirements are the ability to read and interpret Architectural and Structural drawings, 6 plus years of steel detailing experience, and 2 plus years in a detailing, project management, coordination role. Experience in Tekla, SDS/2 and AutoCAD software preferred.

The job location is Taylors SC Candidates can apply by going to the below website: http://stage.cmc.com/Careers/ViewJobOpening.aspx?JP_ID=58DD4EA6-BB36-46CB-929B-727D7FB2A092

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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**.

Summary: A Large U.S fabricator seeks a highly motivated Vice President to oversee the general management for several production facilities within the U.S. Position leads and directs the management of each facility to achieve Company goals and financial objectives in the areas of shop production. The position has responsibilities for each facilities P&L, fabrication thru-put, quality, safety, and process improvement.

Requirements: Bachelor's Degree from an accredited college or university in the following fields of study: Manufacturing Engineering, Industrial Engineering, Civil/Structural Engineering, or Mechanical Engineering. A degree in a business related field may be acceptable if accompanied by proven track record of managing Fabrication / Manufacturing facilities.

Ten (10) years related experience (Director Level or above) with a fabrication company or Original Equipment Manufacturer (OEM). Steel and heavy plate fabrication/production experience required. Experience must encompass P&L responsibility along with supervisory/management responsibility for large plant facilities regulated by OSHA, EPA and other governmental agencies. Prefer experience with an AISC, AWS and/or ISO regulated facility. Travel time requirement is 50-75%.

Qualified applicants should send their resume to **resume3@cox.net**. Resumes will be handled with strict confidentiality.

U.S. Department of Transportation

Federal Highway Administration

The FHWA is seeking a Senior Bridge Engineer to fill a key technical position at the HQ Office of Bridges and Structures (Washington DC) as Steel Bridge Specialist. This person will develop, advance, and implement new and improved construction techniques in steel bridges and other highway structures; and associated engineering practices. This person will provide technical review and assistance with plans and specs for complex structures across the U.S. The ideal candidate will have an advanced degree in Civil/Structural Engineering, and will have experience and competency to improve national engineering practice as defined in policy, codes, standards, and guidelines.

If you are interested in this position, prepare your profile and resume on **USAJOBS.GOV** and submit an application as soon as the announcement is made, which is expected in early January 2016. For additional information contact Brian Kozy at **brian.kozy@dot.gov**.

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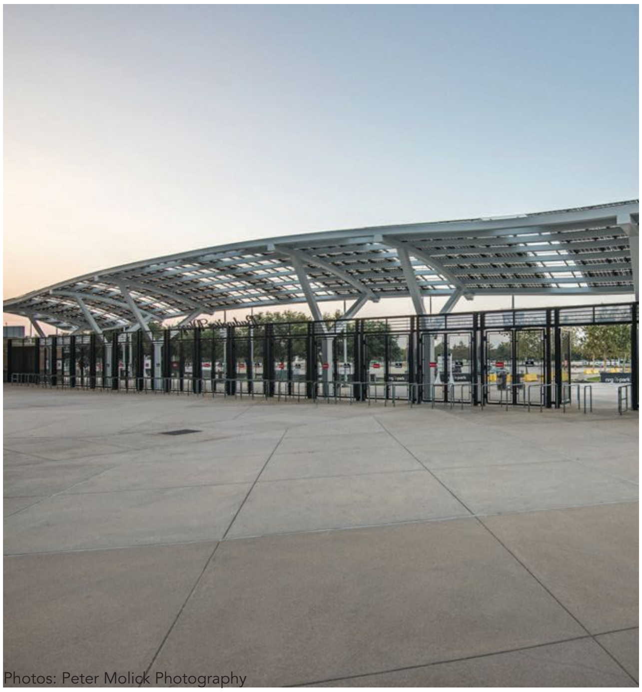

Photos: Peter Molick Photography

SOLAR PANELS HAVE long been attractive from an energy-saving standpoint, but they typically haven't been held in high regard for their aesthetics. More often than not, they are relegated to rooftop arrays, out of sight and out of mind.

Not so at NRG Stadium, home of the Houston Texans. The stadium's solar installation consists of 600 solar panels spread across the stadium for a total system size of 180 kW, with the structural elements being designed by DLR Group and Walter P Moore. The project is the result of close collaboration between DLR's engineering and architectural teams and energy provider NRG to support the latter's mission to get people to "think differently about energy."

NRG Stadium's original pedestrian bridge (constructed for its opening in 2002 as Reliant Stadium) is a steel truss arch that spans the main road in front of the stadium. The addition of new "stealth wings" (as dubbed by the design team) on each end of the bridge facilitated a unique entry point into the stadium while showing off the new addition of sustainable solar panels mounted on top. The flared wings play on the existing braced steel structure of the bridge and the braced roof structure of the stadium. The new structure dips down before curving back up to cantilever out in a sweeping flare using steel wide-flange beams rolled in both the hard way and the easy way by AISC Member Chicago Metal Rolled Products.

The weatherproof entry canopy in front of the stadium uses rolled HSS purlins that are seamlessly welded to create 200-ft-long undulating ribbons. These are splayed along a radiused HSS girder to create an hourglass shape to the canopy. The purlins cantilever 24 ft at each end to extend over the ticket booths in front of the Stadium, and the HSS14×14 "Y" columns tip the entire canopy 10° to provide an incline angle for the solar panels. ■