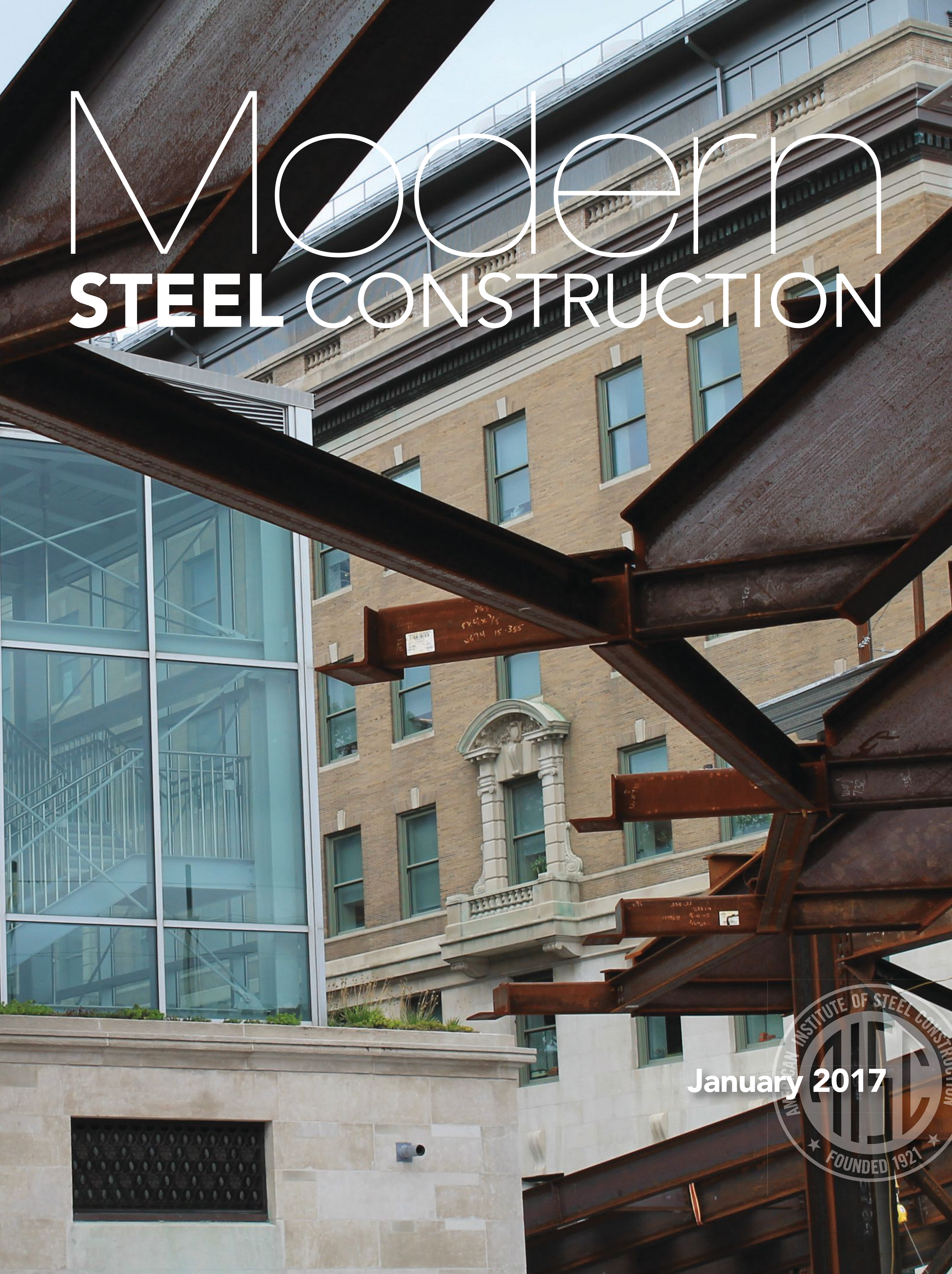
Modern STEEL CONSTRUCTION



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MODERN STEEL CONSTRUCTION (Volume 57, Number 1) ISSN (print) 0026-8445: ISSN (online) 1945-0737. Published monthly by the American Institute of Steel Construction (AISC), 130 E Randolph Street, Suite 2000, Chicago, IL 60601. Subscriptions: Within the U.S.—single issues $6.00; 1 year, $44. Outside the U.S. (Canada and Mexico)—single issues $9.00; 1 year $88. Periodicals postage paid at Chicago, IL and at additional mailing offices. Postmaster: Please send address changes to MODERN STEEL CONSTRUCTION, 130 E Randolph Street, Suite 2000, Chicago, IL 60601.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION AISC FOUNDED 1921

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editor's note

AS MANY HOME BUYERS MUST, MY WIFE AND I NEEDED TO REPLACE MOST OF OUR APPLIANCES WHEN WE MOVED INTO OUR HOUSE NEARLY 20 YEARS AGO. AS A RESULT, IT SHOULDN'T BE SURPRISING THAT DURING THE PAST YEAR, WE'VE HAD TO REPLACE ALMOST ALL OF THEM AGAIN.

Up until now, I haven't spent much time looking at washing machines and refrigerators. But while the form has remained mostly the same, the bells and whistles have certainly improved. My washing machine has no agitator, uses much less water and detergent, and does a better job. The new refrigerator features French doors, a freezer on the bottom and an impressive array of controls for almost every compartment.

The steel industry has undergone a similar evolution. Today's wide flange looks pretty much the same as it did two decades ago, but a lot has changed. For starters, it's about 40% stronger (we've moved from a standard steel with a yield strength of 36 ksi to one with a yield strength of 50 ksi). But it's not just the material itself that's improved. It's also the machines and tools that process the material as well as the designs and systems that utilize this primarily domestically produced material.

This March, I encourage you to travel to San Antonio for the 2017 NASCC: The Steel Conference and see for yourself.

Stop by the booth of one of the steel producers and chat with them about what's new and better about their product. Visit a buckling-restrained brace manufacturer. Check out the latest machines that cut steel like butter and use computers to guide the process for unrivaled accuracy. Visit a structural engineering software provider to see how fast and easy it is to design with steel. With more than 200 exhibitors, it's easy to spend your day learning about a myriad of new systems and products.

But don't forget the more than 130 technical sessions. Into bridges? Check out one of the sessions on "Understanding and Reevaluating Fracture Critical." Concerned about business issues? Don't miss the session on "Case Studies in How to Use the Code of Standard Practice to Prevent and Resolve Disputes." Need a technical refresher? How about "Practical Advice for Reviewing Software Generated Connection Designs" or maybe "Secrets of the Manual to Get it Done."

And don't forget the opportunity to meet industry notables ranging from the nation's most prestigious structural engineering professors to our country's leading designers and fabricators. More than 4,500 attendees are expected and it's always a great opportunity to meet and discuss issues with your peers. And it's a great place to chat with AISC's new president, Charlie Carter.

Visit **www.aisc.org/nascc** and download the advance program. You'll want to register as soon as possible—registration fees increase $10 every week. But as always, AISC members receive a substantial discount.

I hope to see you in San Antonio!

Scott Melnick

SCOTT MELNICK
EDITOR

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

steel interchange

Headed Stud Anchor Diameter for Composite Beams

What should be taken into account for selecting headed stud anchor diameter for composite steel beams? Are there any limitations on using ¾-in.- or ½-in.-diameter studs welded through metal deck to create composite action?

There are some considerations in selecting headed stud anchor (stud) diameter and a few limitations that are independent of diameter.

Size selection: Per the AISC *Specification* (a free download at **www.aisc.org/specifications**) Section I3.2c (1)(2), studs shall be ¾-in. or less in diameter. Also, per Section I8.1, the diameter of the stud shall not be greater than 2.5 times the thickness of the beam flange unless the stud is welded directly over the beam web. Per Section A3.6, headed studs shall conform to AWS D1.1, which only addresses studs ½-in. in diameter and larger, which therefore defines the lower size limit. Those are the specific code provisions pertaining to diameter limitations. Beyond that, it becomes an engineering assessment by you as to what is the more economical or practical solution to transfer the shear for your specific beam.

The cross-sectional area of a ¾-in. diameter stud is more than double the area of a ½-in. diameter stud. Since stud shear capacity is directly proportional to stud cross-sectional area, you will need more than double the quantity of ½-in. diameter studs to provide the same strength as ¾-in. diameter studs. You could confirm with a local steel fabricator or erector in your area, but I would expect that the labor cost of installing a larger quantity of smaller-diameter studs would exceed any cost benefit associated with using smaller-diameter studs unless you have a floor system that uses small beams and doesn't demand very much in the way of shear transfer between the steel and concrete.

With respect to welding through deck, ICC-ES report ESR-1094 provides some good information. This may not be the only report available. Section 4.1 provides some guidelines for when studs can be welded through two layers of decking versus only one layer. This report does not distinguish between various stud diameters in defining the limitations on welding through deck.

Studs should not be welded through coated sheet metal other than typical steel decking. Welding through other materials, such as galvanized architectural flashing materials and even paint on the beam flange, can introduce contaminants into the weld that could affect the weld performance. Standard structural composite decks have controls in place to limit the potential of contaminants from the coatings.

Susan Burmeister, PE

Developing Flexural Strength of Spliced Wide-Flange Members

***Specification* Section J6 has always vexed me because it seems impossible to satisfy the requirement for groove-welded splices. By my calculation, for a W36×160 the weld-access hole reduces the flexural strength of the member to less than 70% of the flexural strength of the member without the weld access hole. My calculation is based on the moment being resisted only by the flanges, which are governed by yielding on the area of the flange. The shear strength is likewise reduced due to the presence of the weld access hole to 85% to 90% of the shear strength of the member. How does the spec intend for the designer to "develop the strength" of the shape?**

Relative to the flexural strength, the situation is similar to directly welded beam-to-column moment connections. The argument is sometimes made that one cannot develop the strength of the beam by connecting only the flanges while at the same time reducing the overall area by including weld access holes. An explanation is provided in the May 2012 article "Developing M_p" (available at **www.modernsteel.com**).

Though I am not aware of a document that addresses shear in this manner, a similar argument could be made. In fact, I suspect you make a similar assumption all of the time without giving it a second thought. It is not common for engineers to perform a yielding check on an uncoped beam with a bolted connection. Even a net area check is not typically considered necessary, since the flanges will tend to prevent such a failure. The only check commonly made is gross shear (yielding) based on the full area of the web. Regardless of the size of the weld access holes, they certainly remove less area than a full depth bolted connection.

It is also important to note *Specification* Section J6 is not requiring the connection to develop the actual strength of the member or the expected strength of the member (as we might in seismic design) but rather the "nominal strength of the smaller spliced section."

The case of the expected strength is an interesting one. The welded unreinforced flange-welded web moment connection in AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (a free download for members at **www.aisc.org/seismic**) has been shown through physical tests not only to develop the nominal strength of the beam but also to develop the actual beam strength and force hinging of the beam outside the connection. This is accomplished even with the larger weld access holes required in AWS D1.8.

Larry S. Muir, PE

Tributary Length for Prying Action

The article "A Slightly Longer Look at Prying" (an online supplement to the July 2016 article "A Quick Look at Prying," available at www.modernsteel.com) states that the effective width, *p*, for prying action can be conservatively taken as 3.5*b* but cannot exceed the spacing between the bolts. This conflicts with Part 9 of the *Manual*, which indicates that *p* is limited to twice *b*. Can the effective width exceed 2 times *b*?

Yes. The statement in the article is based on the upcoming 15th Edition *Manual*, which will revise the default effective width from $p = 2b$ to $p = 3.5b$. The new default value is based on guidance provided by the South African Institute of Steel Construction that was evaluated by the AISC Manual Committee and deemed to be adequate. The new assumed distribution angle is 60°, which is conservative but not as conservative as the assumed 45° angle used in the 14th Edition *Manual*.

It should be noted that the 2*p* limit was not intended to be a requirement. Even though it is not stated, it was only a recommendation. The recommendation was established because it was brought to the attention of the Manual Committee that there was a wide range of assumed tributary lengths being used in practice. It was felt that the *Manual* should provide guidance. As is often the case when engineers are forced to provide guidance, the first pass was conservative. Given the lack of data available at the time, the committee felt that the 2*b* guidance was a safe lower bound. With a closer look at the South African Institute of Steel Construction data the 3.5*b* limit was adopted. Note that the *Manual* also allows that a larger tributary length may be justified based upon testing or rational analysis.

Carlo Lini, PE

Fatigue and Removal of Backing for Fatigue

We have received shop drawings for a steel structure with moment frame connections using complete joint penetration groove welds. It is not a high-seismic project but we do have fatigue design considerations. The contractor has indicated on the shop drawings that backing bars will be used. We requested that the backing be removed in our review comments. The contractor is asserting that this is an unusual requirement and is treating this as a change in the contract. We believe that since the structure is subjected to fatigue, the contractor should be required to remove the backing at no additional cost. How is this situation treated in AISC documents?

This situation is not directly addressed by any AISC document. However, it is addressed in AWS D1.1, which is adopted by reference in Section J2 of the *Specification*.

Clause 2.17.2 of AWS D1.1 addresses backing and directly addresses the removal of backing, which often can generally be left in place. The treatment of backing is tied to fatigue considerations as you indicate. Clause 2.17.2.1 requires the engineer to provide the fatigue stress category in the contract drawings. If you provide the applicable fatigue stress category in the contract documents and AWS D1.1 Clause 2.17.2 requires removal for that fatigue stress category, backing removal is required. Otherwise, adding a requirement to remove backing with comments during shop drawing approval or by RFI response may represent a change to the contract. Section 4.4.3 and 9.3 of the AISC *Code of Standard Practice* (a free download from www.aisc.org/specifications) addresses revisions to the contract documents.

Carlo Lini, PE

Short-Headed Stud Anchors

Chapter I of the AISC *Specification* requires that "stud shear connectors, after installation, shall extend not less than 1½ in. above the top of the steel deck." I have an existing building and the original design documents indicate the shear studs extend only 1 in. above the steel deck. When calculating the composite strength of this member, is there a reduction factor that can be used to account for shorter stud?

AISC does not have sufficient information to make a recommendation about the performance of composite flexural members when the stud projection above the deck flutes is less than 1½ in. Therefore, the *Specification* does not provide a reduction factor for use with the current equations. You would have to use your own engineering judgment. The parameter limitations noted in current Section I3.2c were established to ensure beam designs are performed within the margins of the available research data, largely summarized in the first quarter 1977 *Engineering Journal* article "Composite Beams with Formed Steel Deck" (available for free to AISC members at www.aisc.org).

Provisions for composite members with formed steel deck did not appear in the AISC *Specification* until 1978. At that time, the *Specification* required the same 1½ in. projection per the research in the above-mentioned article. However, Section 1.11.6 stated: "When composite construction does not conform to the requirements of Sects. 1.11.1 through 1.11.5, allowable load per shear connector must be established by a suitable test program." This may have permitted a designer to use a shorter stud projection if they had access to some other test data in order to establish their shear connector strength.

Susan Burmeister, PE

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

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

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

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

866.ASK.AISC • **solutions@aisc.org**

Steel SolutionsCenter

steel quiz

Steel Quiz made its first appearance in the November 1995 issue of *Modern Steel Construction*. This month's Quiz takes a look at some of the best questions from 2001.

1 Paint thickness is commonly measured in which unit?

a. cm **b.** feet
c. mils **d.** coats

2 Why would a bolt stick through requirement decrease the ductility (ability to stretch) of F3125 Grade A325 and A490 bolts?

3 What is the maximum acceptable wind velocity in the vicinity of the weld when the FCAW-G process is used?

4 What is the minimum thickness of a compact 10 in. wide A572 Gr. 50 flange cover plate welded to the top of a W24×131 beam?

5 **True or False:** Written WPSs are required for all prequalified shop and field welds.

6 **True or False:** The shear and tensile strengths of a bolt are not affected by pretension in the bolt.

7 What is meant by "firm contact" in a bolted connection?

8 What is the generally accepted minimum inside-bending radius for cold bent ½-in. (A36) plate when bending is transverse to the direction of rolling.

9 How is "grip" defined?

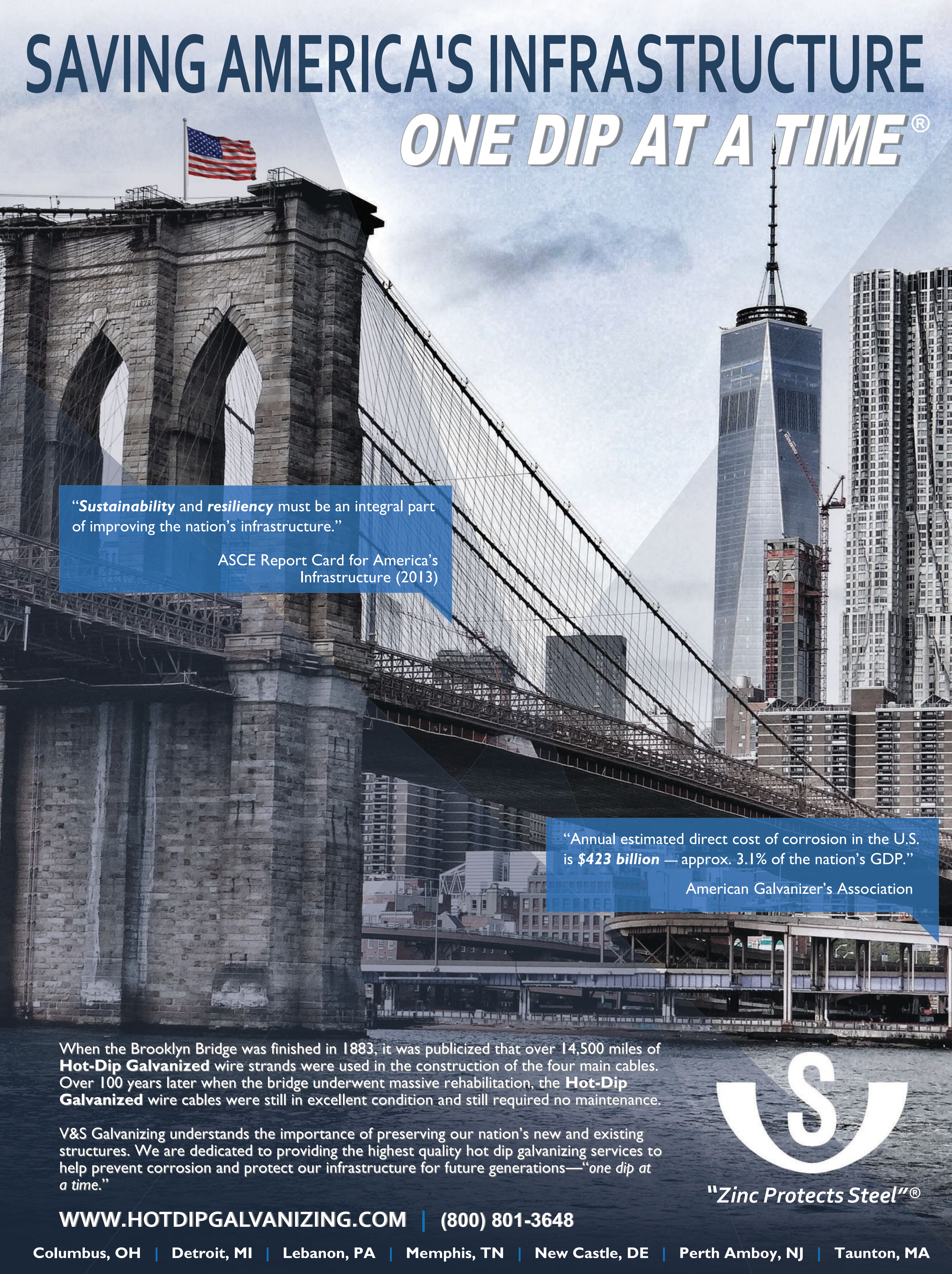
10 **True or False:** Spray-applied fire protection material should always be applied over primed steel.

TURN TO PAGE 14 FOR ANSWERS

Everyone is welcome to submit questions and answers for Steel Quiz.

Steel SolutionsCenter

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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1 **c.** A mil is $\frac{1}{1000}$ of an inch.

2 Ductility is related to the number of threads in the grip. Most of the bolt elongation occurs in the threaded portion below the nut. This relationship is described in the 2nd Edition of the *Guide to Design Criteria for Bolted and Riveted Joints* (a free download at **www.bolcouncil.org**) which states: "Since most of the elongation occurs in the threads, the length of thread between the thread run-out and the face of the nut will affect the load versus elongation relationship. The heavy head bolt has a short thread length, whereas the regular head bolt has the normal ASA thread length specified by ANSI standards. As a result, for a given thickness of gripped material, the heavy head bolt shows a decrease in deformation capacity."

3 Maximum acceptable velocity is 5 miles per hour. If expected wind velocity is higher, a temporary shelter can be used for protection. (See AWS D1.1 Clause 5.12.1)

4 $\frac{3}{8}$-in. See Table B4.1b in the AISC *Specification* for limiting width-thickness ratio for compact flange cover plates.

5 **True.**

6 **True.**

7 The glossary of the RCSC *Specification* (a free download at **www.boltcouncil.org**) defines firm contact as "the condition that exists on a faying surface when the plies are solidly seated against each other, but not necessarily in continuous contact."

8 It would be 1.5×0.5 = 0.75 in. See Table 10-13 in the 14th Edition *Steel Construction Manual*. Note that bent plates exhibit better ductility and require a smaller bending radius when bent perpendicular to their rolling direction.

9 Grip is defined in the glossary of the RCSC *Specification* as "the total thickness of the plies of a joint through which the bolt passes, exclusive of washers or direct-tension indicators."

10 **False.** Many shop-applied coatings (and field-applied coatings, for that matter) are incompatible with common fire-protection materials, causing them to adhere poorly to the steel. In most cases, unprimed steel is the best surface to receive applied fire-protection materials.

Steel Solutions Center

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steelwise

COMPREHENSIVE COVERAGE

The new UL D982 now covers all common steel-framed floor configurations.

BY CHARLES J. CARTER, SE, PE, PhD, AND FARID ALFAWAKHIRI, PEng, PhD

GOOD NEWS: Fire protection for steel framing just got much more economical.

New revisions to UL Design No. D982 will allow flooring fire-protection material and application costs to be cut nearly in half compared with old unrestrained applications. While UL D982 originally only applied to composite construction with normal-weight concrete, these limitations have been discarded as a result of testing that UL performed for AISI and AISC. (For background on what necessitated the testing, see "Restrained or Unrestrained?"—September 2013—and "UL Design Considerations"—October 2015—both available at **www.modernsteel.com**.)

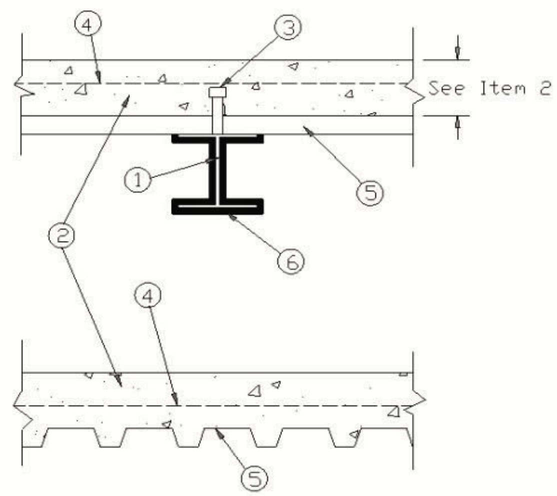
UL D982 used to refer only to unrestrained ratings and left it up to the designer to know that it could also be used for restrained conditions. Now, the design explicitly states that it is useful for both two-hour restrained and two-hour unrestrained assemblies. In addition:

- It covers both composite and non-composite designs
- It allows for the use of either normal-weight concrete or lightweight concrete
- It allows for any welded wire fabric placement location in the concrete slab
- It applies to metal deck thicknesses from 1½ in. to 3 in., inclusive

Also worth noting, but not new, is that UL Design No. D982 is not load-restricted (again, see "UL Design Considerations" as well as visit **www.aisc.org/ulclarity** for further information).

What does this all mean? In a nutshell, it means that the design now covers all common steel-framed floor configurations and provides two-hour assembly ratings with unprotected steel deck and spray-applied fire protective materials (SFRM) on the steel beam with thickness sufficient to obtain a one-hour unrestrained beam (temperature-based) rating. The design can be used in both the United States and Canada, it works with a wide range of steel deck products and it is valid for any SFRM material bearing the UL Classification Mark. See the full text of the updated UL Design No. D982 at **www.ul.com/firewizard**.

Our conclusions from 2013 ring all the more true today. These test results and the expansion of UL D982 they allow are great. They provide a solution that eliminates any need to argue about what fire protection thickness is required. In all common cases, UL D982 allows you to have the same fire protection thickness whether you choose a restrained rating or an unrestrained rating. ■



Design No. D982
October 21, 2016
Restrained Assembly Rating-2 Hr.
Unrestrained Assembly Rating-2 Hr.
Unrestrained Beam Rating-1 Hr.

Loading Determined by Allowable Stress Design Method or Load and Resistance Factor Design Method published by the American Institute of Steel Construction, or in accordance with the relevant Limit State Design provisions of Part 4 of the National Building Code of Canada

▲ The full version of Design No. D982, including descriptions of each numbered part, is available at **www.ul.com/firewizard** (search for "D982").

Charles J. Carter (**carter@aisc.org**) is president of AISC and **Farid Alfawakhiri** (**falfawakhiri@steel.org**) is senior engineer, Construction Codes and Standards, for the American Iron and Steel Institute.

economics

WHAT'S REALLY HAPPENING WITH THE CONSTRUCTION ECONOMY?

BY JOHN CROSS, PE

What to expect in 2017 and how to prepare for it.

I HEAR THE FOLLOWING STATEMENT a lot, lately: "But John, I heard that construction was back to prerecession levels!"

And inevitably after that statement comes this question: "So why are structural steel volumes lower than 2008?"

The assumptions behind each are true. Construction starts on a dollar basis are back to prerecession levels. Building construction starts in 2008 were valued at $199 billion and are projected to be $200 billion in 2016. Sounds good, right? At the same time, the apparent consumption of structural steel in 2008 was 8.5 million tons but is projected to be 7.3 million tons in 2016. Sounds bad, right? Absolutely!

Maybe the difference is a significant drop in industrial construction? Nope, industrial construction is actually up from $184 billion in 2008 to $235 billion in 2016.

Imports of mill material? Yes, imports have had an impact but not on apparent consumption. Apparent consumption includes the impact of imports.

So what is going on in the building sector? The fact is that even though construction has returned to 2008 levels on a per-dollar basis, the square footage being constructed has not. In 2008, construction starts for all nonresidential buildings and multistory residential buildings (five stories and higher) totaled 1.40 billion sq. ft. In 2016, the anticipated square footage of construction starts is only 1.19 billion sq. ft.

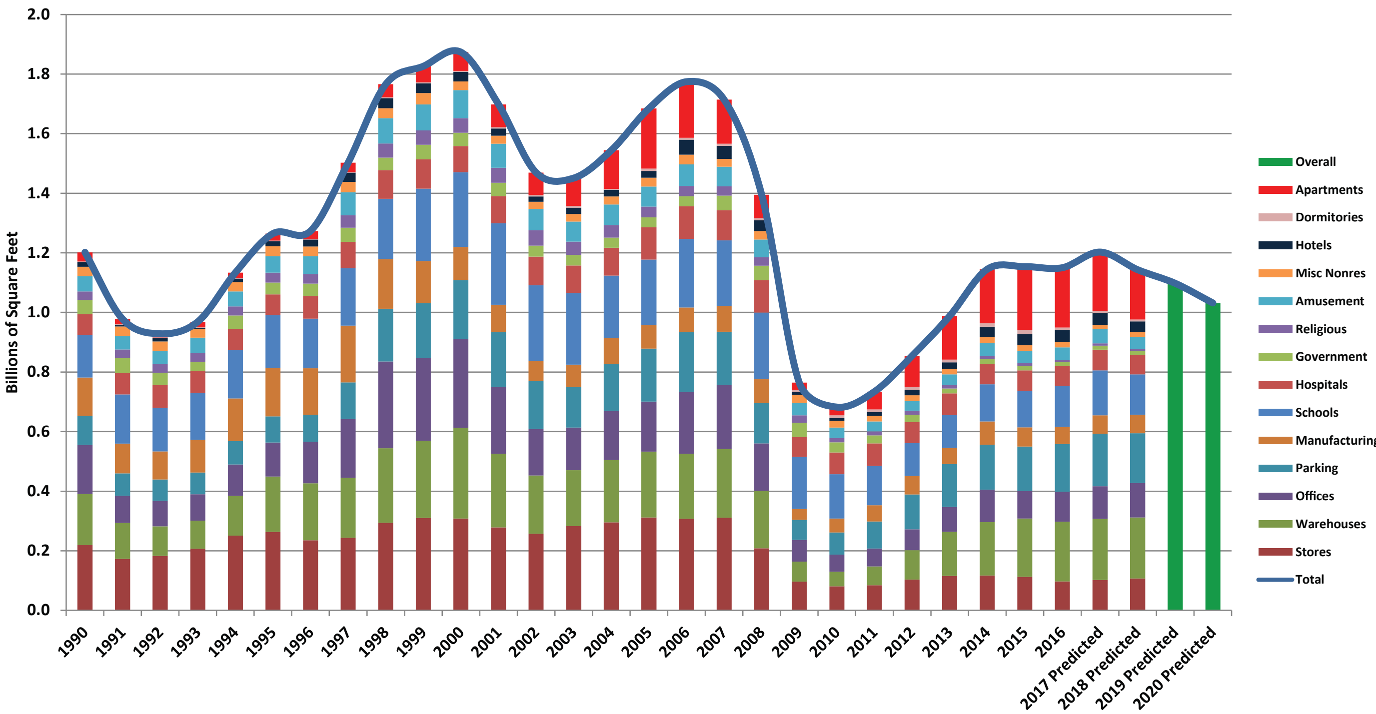
John Cross (cross@aisc.org) is an AISC vice president.

	2008	2016	Change
Billions of Dollars	199	200	1%
Billions of Square Feet	1.40	1.19	-15%
Number of Projects	45,347	22,634	-50%
Apparent Domestic Consumption *in millions of tons*	8.5	7.3	-14%

The simple fact is that structural steel consumption is not driven by project cost. Structural steel consumption is driven by square footage.

The increase in project costs is a function of the location and type of building being built as the cost per square foot increases by building height—and construction in urban areas is more expensive than in rural areas. Since 2008, the growth in construction can be credited to mid- to high-rise buildings being built in major urban areas. In 2008, only 21% of the square footage of construction starts was in buildings five stories and above, while by 2016 that percentage has grown to 37%. The trend to taller buildings is tied to an increasing percentage of construction in urban areas.

As we move into 2017, it is anticipated that we will see an overall growth of 4% in our prime building sector market on a square-footage basis. This will result in approximately 1.19 billion sq. ft of construction starts, which is still 14% below 2008 levels. The good news for the structural steel industry is that this growth will be in the nonresidential sector, where structural steel has a higher market share, as opposed to the multistory residential market, where a significant portion of the growth has occurred since 2008. The combined growth in the market and the types of projects where the growth will occur will result in an increase of 5% in the apparent demand for structural steel.



The increase in construction starts will not be felt uniformly across the country. The majority of the growth will continue to occur in urban areas, with suburban and rural construction continuing to lag. By region, the slowest growth will occur in the Midwest (1%) and the Northeast (2%), with higher growth taking place in the West (5%) and the South Atlantic and South Central regions (11%).

The increase in 2017 will be short-lived, with construction starts beginning a cyclical downturn in 2018. This downturn will be significantly more gradual and shallower than that experienced during the Great Recession after 2008.

all values in millions of square feet	2016	2017	% Change
Stores	101	110	9%
Warehouses	183	205	12%
Offices	113	122	8%
Parking	158	176	11%
Manufacturing	57	62	9%
Schools	141	150	7%
Health Care	71	69	-2%
Government	14	14	0%
Religious	8	7	-2%
Amusement	43	47	10%
Misc. Nonresidential	18	20	11%
Hotels	46	41	-12%
Dormitories	7	6	-13%
Apartments	225	197	-12%

So what do we need to plan for in the coming years? At a recent meeting of engineers and steel fabricators in Colorado, I shared the following points to consider for the future:

- A cyclical building construction market peaking in 2017
- Flat to declining construction volumes for several years, starting in 2018
- Growth in mid-rise (five to 19 stories) construction
- Continued urbanization and building demand in urban areas
- A slowing multistory residential market
- A moderate increase in nonresidential building construction
- Fewer projects to compete on
- An increase in renovation activity

So how can designers, constructors and the structural steel industry deal with the challenges these trends present? Each sector of the design and construction industry and each individual firm will need to answer that question for themselves. But the consensus answer from the Colorado meeting was that those firms, be they designers, constructors or steel fabricators, that focus on providing exceptional project outcomes will be the firms that prosper. And who will those firms be? The ones that become involved in the project early in the conceptual process and participate as a contributing team member throughout the design and construction of the project, independent of the project delivery method being used.

conference preview

MINIMIZING HAZARDS, BY DESIGN

Safety can and should be just as important in the design process as it is during manufacturing and construction.

BY WAYNE J. CREASAP II

BY NOW, ALMOST EVERYONE in the industrial maintenance and construction industry is familiar with OSHA's "Focus Four" hazards: falls, struck-by, caught-in-between and electrical.

These four culprits account for the majority of construction-related fatalities and severe injuries and can be easily encountered throughout the construction process—so much so that OSHA has developed several resources to help companies identify and eliminate them. These include specialized training modules dedicated to the Focus Four as part of the required training in OSHA's 10-hour and 30-hour training courses for construction, general industry and maritime work.

But remember, OSHA standards cover the minimum requirements for an employer to eliminate or control hazards on the job. By using these standards as a baseline to build from, companies can implement risk-management practices to help identify hazards and further reduce or eliminate them before they become problems on the job.

Safe for All

The focus of any building project is on who will be occupying the structure after it is built, but it often fails to account for the health and safety of those building it, those who have to maintain it or ultimately, those who have to remove it. The American Society of Safety Engineers (ASSE) has been working on American National Standards Institute (ANSI) standards aimed at implementing risk-management practices and prevention-through-design techniques to drill down on leading practices that will assist employers in gauging risk and eliminating common hazards on the job. Several of these components are aimed at reducing hazards throughout the life-cycle of a building, from design through construction, operation, demolition and waste treatment. Often, the hierarchy of controls is used to help eliminate or reduce hazards, with the use of personal protective equipment (PPE) as the last line of defense.

Steel fabricators and erectors are faced with several safety and health challenges when fabricating and building a project. The International Association of Bridge, Structural, Ornamental and Reinforcing Iron Workers Union developed a list of "Deadly Dozen" hazards for both steel fabrication and erection activities. Many of these hazards are related to various OSHA standards, and the majority of them are Focus Four-related. Here, we'll take a look at some of these hazards and determine how we can preplan and come up with design solutions that will better protect shop workers during fabrication and ironworkers during steel erection.

Shop Hazards

Here are the Deadly Dozen in the shop:

1. Exposure to toxic welding fumes that create serious health hazards
2. Striking hazards during material handling and loading and unloading of trucks
3. Dismemberment by shear presses, punch presses and other equipment
4. Rigging failure and use of chains, slings, plate dogs and other rigging equipment
5. Hazards related to overhead rail cranes, gantry cranes and other cranes
6. Hazards pertaining to use of forklifts and jacks
7. Exposure to toxic paints and chemicals through inhalation and skin absorption
8. Exposures to airborne metals, dust and compounds during grinding and hot-work operations
9. Electrical hazards, de-energizing equipment and lock-out tag-out systems
10. Improper signals, communication and clearances
11. Exposure to heat illness and dehydration
12. Lack of protective eyewear, leathers, gloves, hearing conservation equipment and other PPE

Several items on the list involve exposure to welding fumes and toxic metals from grinding and hot work, as well as chemical exposure to paints and other coatings. Employing the hierarchy of controls, can we design systems that will eliminate these hazards? If not, can we substitute a different product that will be just as effective, but not as harmful to the employees working on or around it? With a little preplanning, we can de-

Wayne Creasap (**wcreasap@tauc.org**) is senior director of environmental health and safety with The Association of Union Constructors (TAUC).

▲▼ Safety considerations in the shop run the gamut from proper and efficient material handling to exposure to and protection from airborne metals and compounds during hot-work operations.

sign a system that reduces employee risk to these exposures—and at the same time make them more productive.

Welding fumes are common in fabrication shops, and in recent years we have done a much better job of protecting employees from the toxins in these fumes. Instead of using natural ventilation or fans, or skipping straight to respirators, many companies have employed the use of ventilation systems with high-efficiency particulate air (HEPA) filtration to reduce or eliminate employee exposure to welding fumes. Mostly, these units are aftermarket installations where the building has been around for several years and the systems are installed later.

Even with the new installations, these systems require advanced planning to ensure the greatest efficiency and ease of use for the workers. Otherwise, if they are too difficult to use or maintain, employees will not use them and then the investment becomes worthless. Preplanning is the key, and employing hazard analysis will help to determine the best location for the system. Is it out of the way when materials arrive, yet accessible when needed? Is it easy to get to for filter changes and maintenance? You might find the need to discuss these issues with your employees or safety committee. They can offer valuable input on location and real-life use and will help prevent problems after the system is installed.

Additionally, are there other systems that can be used that are more automated? Can some of the welding be done remotely or with robots to reduce human exposure? The benefits here are reduced employee exposure to welding fumes and other metals and less dependence on and cost associated with implementing a respiratory protection program. Further, quality and productivity may increase under certain circumstances.

Shop hazards also include a lot of struck-by and caught-in-between hazards. Forklifts, overhead cranes and rigging play an important part of safe material handling. However, we should be asking what systems are in place to ensure the greatest efficiency and reducing the number of times an item is lifted or moved. Are things discussed and planned out in advance for the fabrication of each piece for a specific project? Or are managers and employees scrambling to make things fit and meet deadlines? Are those pieces scheduled and fabricated in a way that will enable them to be sequenced properly and erected more efficiently on-site?

Personal protective equipment is the last line of defense when it comes to safety.

Fall-arrest systems can be as simple or elaborate as needed but require training and engagement with employees.

Field Hazards

Here are the Deadly Dozen on-site:

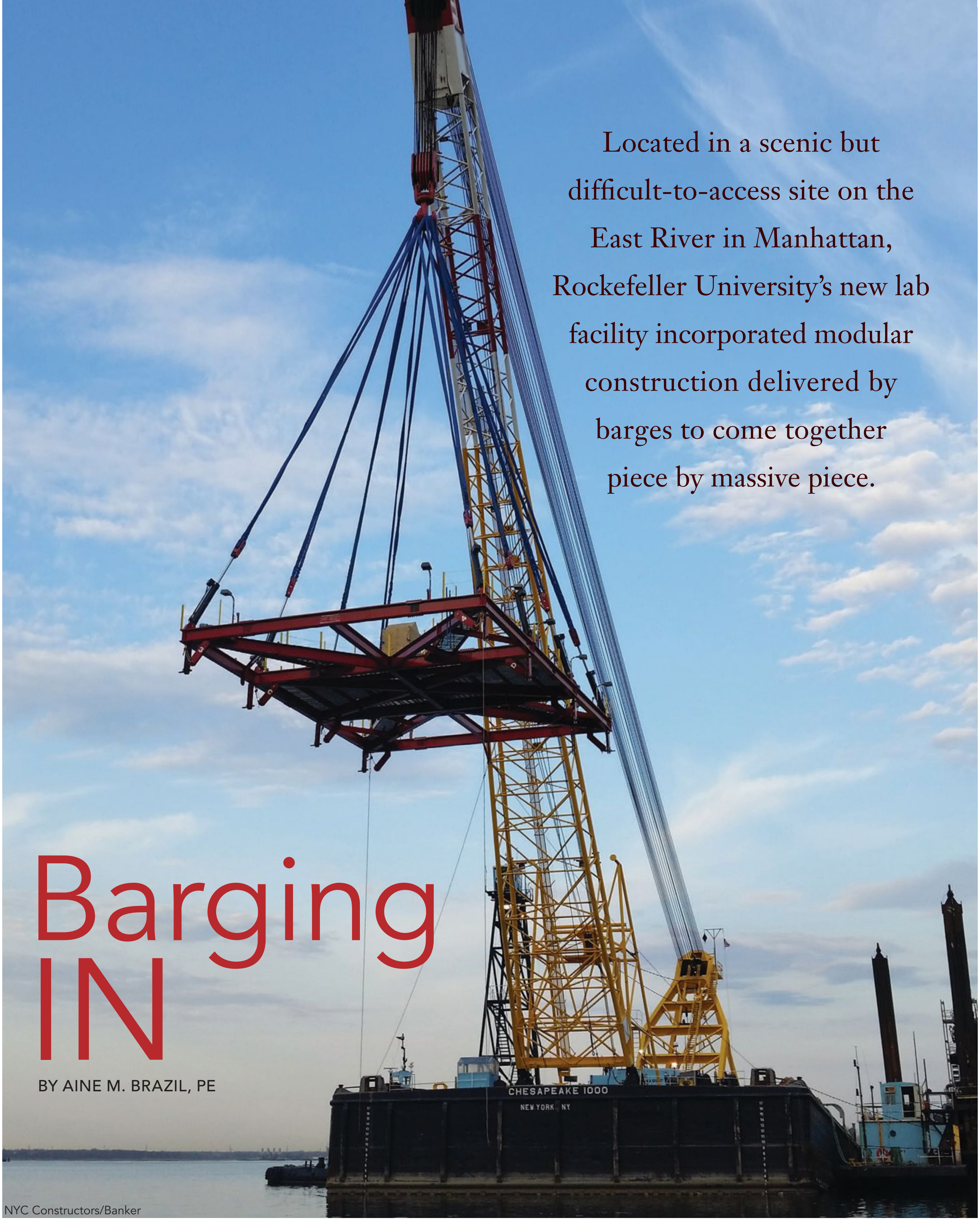
1. Falls through unprotected or inadequate floor opening covers
2. Collapse of unsecured open web steel joists
3. Lack of fall protection and inadequate use of fall-arrest equipment
4. Falls during installation of floor and roof decking
5. Material handling injuries during steel erection and reinforcing steel activities
6. Column collapse due to anchor bolt failure and/or insufficient concrete strength
7. Structural collapse of unsupported reinforcing steel columns, walls and decks
8. Struck-by injuries from falling objects, tools and materials.
9. Caught-between injuries during hoisting and rigging operations
10. Impalement from unprotected reinforcing dowels or other vertical projections
11. Electrical hazards and injuries from high-voltage power lines
12. Heat illness and toxic exposure to chemicals and air-borne contaminants

As with the list of shop hazards, almost every item here is associated with one of the Focus Four. Risk can be minimized by working with the fabricator to ensure that pieces are fabricated and sequenced properly. Additionally, preplanning and design can help eliminate a number of fall hazards. For example, assembling pieces on the ground can help minimize risk associated with falls, falling objects and material handling when erecting multiple pieces in the air; using aerial lifts can also help minimize fall hazards (though it should be noted that proper fall-arrest equipment needs to be used in aerial lifts as well). Again, PPE is the last line of defense in our hierarchy of controls and is only as good as how it is used. How often do we see employees on this equipment wearing harnesses but that aren't actually tethered to the lift? And more importantly, what do we do about it?

Additionally, working within OSHA's steel erection rules, contractors can save time and resources by preplanning and working with the engineer and fabricator to design and install proper fall-arrest systems for a specific job, such as horizontal lifeline and perimeter cable systems, that will protect ironworkers from fall hazards and increase their productivity. Systems can be as simple or elaborate as needed and will require a certain level of training and engagement with employees to ensure they are used properly. In addition, falls associated with decking operations can be minimized through sequencing.

While working with engineers to emphasize and design safety systems might seem like an additional, unnecessary stop, it's not. It benefits all workers in the shop and field, as well as those who will maintain the building. It saves money associated with potential injuries and illnesses. And most importantly, it's the right thing to do. Be an agent of change and keep the dialogue going. ■

*This article is a preview of Session R4 "Minimizing Hazards by Design" at NASCC: The Steel Conference, taking place March 22–24 in San Antonio. Learn more about the conference at **www.aisc.org/nascc**.*



NYC Constructors/Banker

Barging IN

Located in a scenic but difficult-to-access site on the East River in Manhattan, Rockefeller University's new lab facility incorporated modular construction delivered by barges to come together piece by massive piece.

BY AINE M. BRAZIL, PE

ROCKEFELLER UNIVERSITY is in a tight spot.

Founded by John D. Rockefeller and widely considered one of the best biomedical research institutions in the world, the university is located on Manhattan's Upper East side, adjacent to the East River and FDR Drive, one of the city's busiest roadways and a major north-south traffic artery.

When the university decided to expand its campus with a modern three-story building to house laboratory space, it was boxed in on the north, south and west sides. The only way to go was east—over FDR Drive.

Slated for completion next year, the building will house the university's research programs in genetics, neuroscience, biological imaging, cancer biology and other fields. The new 180,000-sq.-ft facility has laboratory space on two floors and will offer researchers a place to work and connect on the same floor, with the proper resources to pursue their projects. The layout encourages collaboration across disciplines and allows for maximum flexibility in rearranging current laboratory spaces. Two one-story glass pavilions are located atop the laboratories and provide office and cafeteria space, as well as a plaza with greenery and outdoor seating.

▲ A completed rendering view from Roosevelt Island.

▼ Framing for one of the pop-up assemblies on the roof level.

▲ Framing for the amphitheater.

◀ The Chesapeake 1000 crane, ready for the first pick.

▶ A rendering of the completed roof, showing the amphitheater.

When constructing laboratory space of any kind, special care must be taken to eliminate vibrations that would affect the daily activities of the lab, such as footfalls inside or rumbling vehicles outside, and long spans like those employed for this expansion are typically more susceptible to vibration issues. Understanding that standard rules frequently employed for vibration analysis would not be adequate for this project, project structural engineer Thornton Tomasetti approached the design by performing detailed dynamic analyses of the overall structure and tuned the sizes to satisfy the vibration limits required for a laboratory.

Aine M. Brazil is vice chairman of Thornton Tomasetti.

▲ The amphitheater faces the existing, adjacent building.

▼ Lay-down space between FDR Drive and the river promenade.

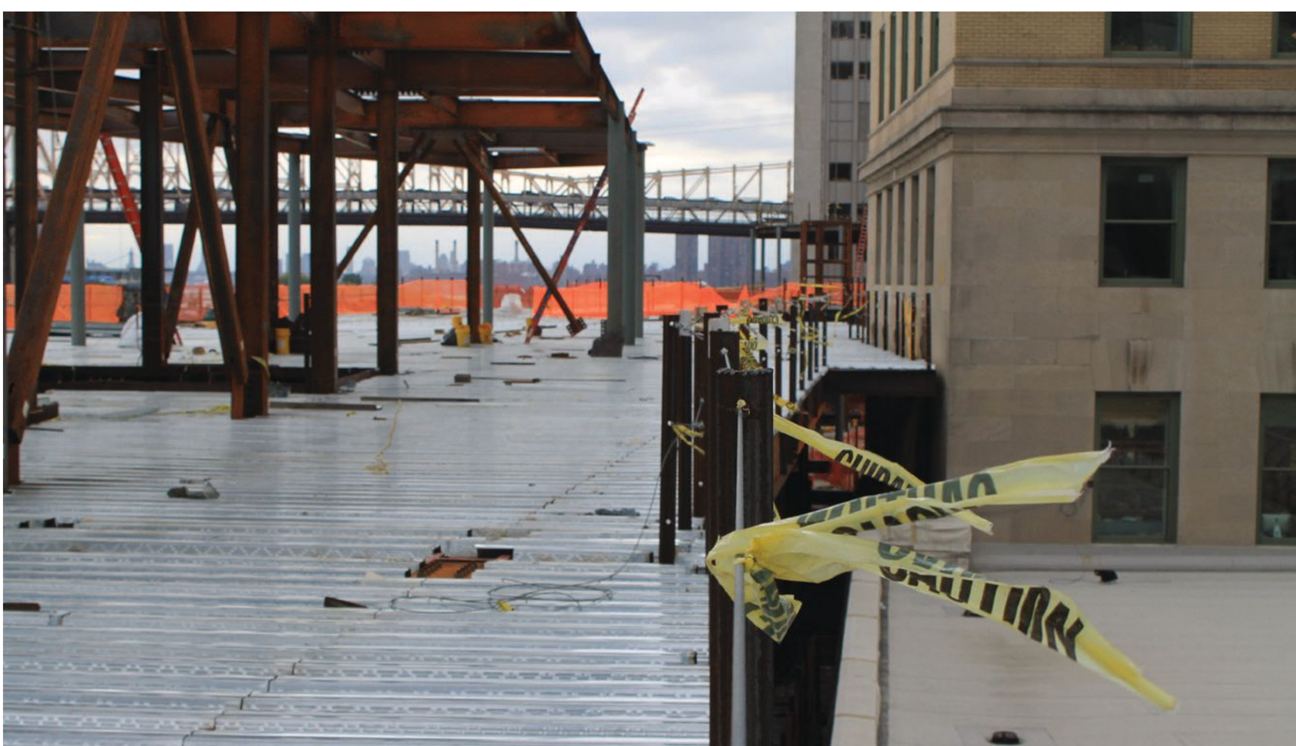
Over the Drive

Not only did the expansion need to meet vehicle clearance constraints on FDR Drive, it also had to match current campus elevations for connections back to existing buildings. Thus, construction depth needed to be minimized. The design called for columns along the east side of FDR Drive to be spaced at 96 ft on center. These two-story "Y" columns are founded on pile caps with multiple mini-piles rather than large caissons due to limited capacity for equipment on the esplanade. Columns on the west side of the roadway are spaced at 48 ft on center. The primary superstructure consists of two levels of high-strength plate girders, each approximately 5 ft deep, spanning up to 92 ft over FDR Drive. The girders are linked by diagonals such that they can act together like a truss; this assembly also supports the green roof on the third level. In addition, the 900-ft-long building was laterally supported at two mechanical and electrical rooms (located at modules 7 and 10) with drag struts (two at each of these rooms) that were embedded in the first-floor slab and welded to plates cast into the shear walls of the mechanical rooms.

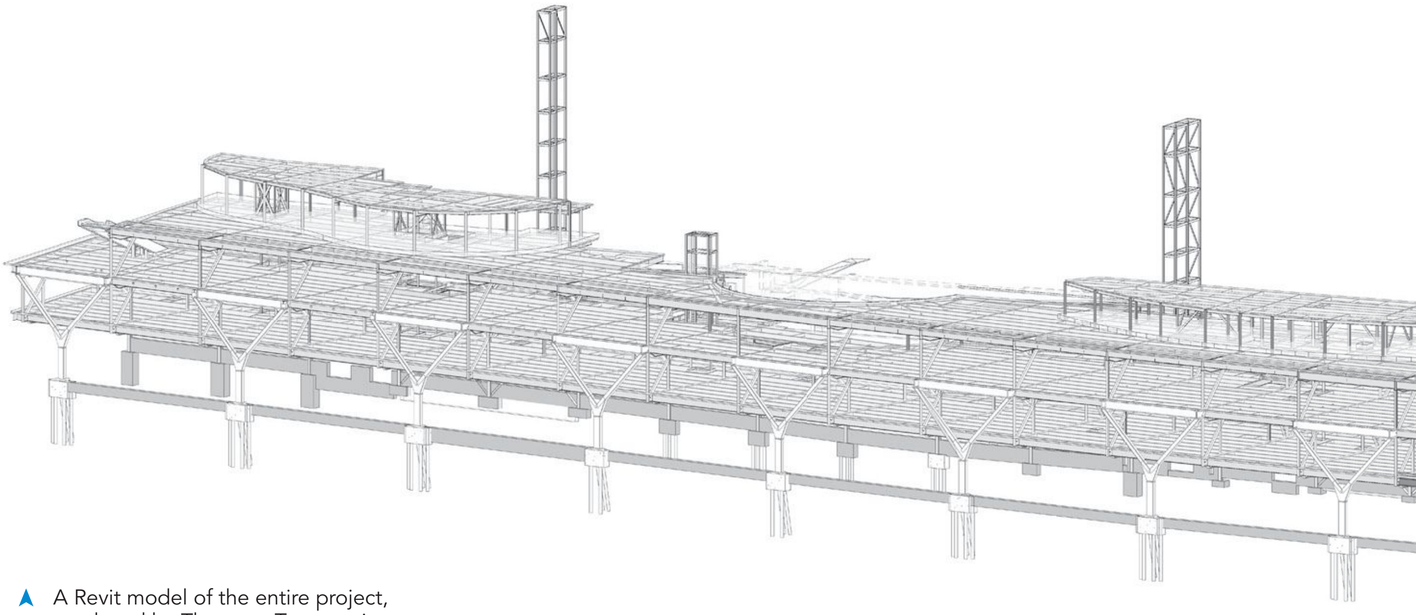
Thomas McLane, Thornton Tomasetti

▲ One of the plate girders being assembled off-site.

▼ The modules were set 2 in. from the existing structure.



Working on and over one of the busiest highways in New York City could have been an excessively expensive and time-consuming endeavor. In addition, laydown area for the job was limited to a 10-ft to 15-ft strip of land between FDR Drive and the river promenade. In an attempt to reduce the cost and schedule and address these space constraints, the design and construction team explored off-site modularization for the structural framing system. Fabricator Banker Steel further developed this plan and came up with the solution to split the building portion that would span FDR Drive into 19 individual modules—approximately 92 ft by 48 ft and three levels high. These modules were prefabricated at a site in Keasbey, N.J., and transported to the project site via barge. Several other trade components were also installed into the modules prior to shipping, including cast-in-place concrete on two levels, fireproofing sprinkler systems and conduits. The modules were set 2 in. from the existing structure, the façade of which was surveyed using a point cloud survey and incorporated into the 3D model of the new building at the design stage.



A Revit model of the entire project, produced by Thornton Tomasetti.

Modification for Modules

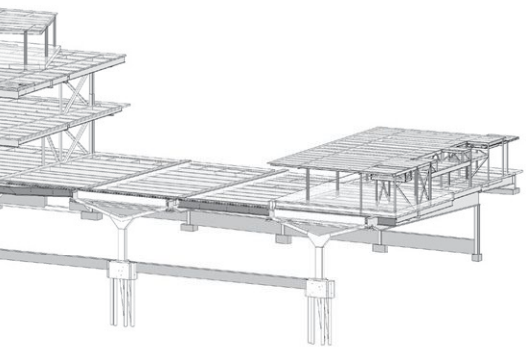
As the modules for Rockefeller University were assembled some 20 miles southwest of the job site in Keasbey, N.J., they had to be barged to the site. However, the barges that were employed weren't built specifically for such a job, so the construction team worked with the barges' owner to modify the vessels, with the promise that they would be returned to their original state following installation of the 19 modules.

The transformation involved removing cargo walls at the perimeter of a barge—known as "coaming"—and adding transverse beams to transfer the loads imposed back to the center of the barge. They also had to be modified to include a "pocket" for the Y-columns to sit in, so as to avoid the need to raise the module up to clear the deck—which would also raise the loaded barge's center of center of gravity and increase the risk of overturning. In addition, stair towers on 30-ft track beams were added to provide ironworker access for connecting the lift frame to the modules.

Modifications also had to be made to the Chesapeake 1000 barge crane. A mooring arm, which pivoted off the corner of the barge, had to be installed. While setting the modules at the site, this mooring arm connected to a 120-ft-long whaler beam (connected to two Y-column shafts) which provided horizontal stability and a rotation point for the barge when it had to rotate approximately 90° from pick position, taking the module off the transport barge to its final placement on the building site.

NYC Constructors/Banker

➤ The Chesapeake 1000 crane preparing to erect the first module.



Lorenzo Sanjuan, Thornton Tomasetti

Each module was lifted and installed from the barge during 19 separate five-hour periods during a two-and-a-half-month span in the summer of 2016. The lifts were performed via the Chesapeake 1000, a rare, 1,000-ton barge crane. The crane was only able to operate during a steady slack tide, and FDR Drive closures were limited to 12:00 a.m. – 5:00 a.m. in order to minimize traffic disruption, so there were specific days on which the lifts could take place. In the end, this modular construction and erection scheme shaved several months off the original schedule and minimized on-site steel assembly.

Constructing the building over FDR Drive was one of the project's major challenges, as the roadway needed to remain open with limited disruption to the traffic flow and minimal impact on the city. The project team also had to ensure the safety of the passenger cars that drove through the site each day on FDR Drive.

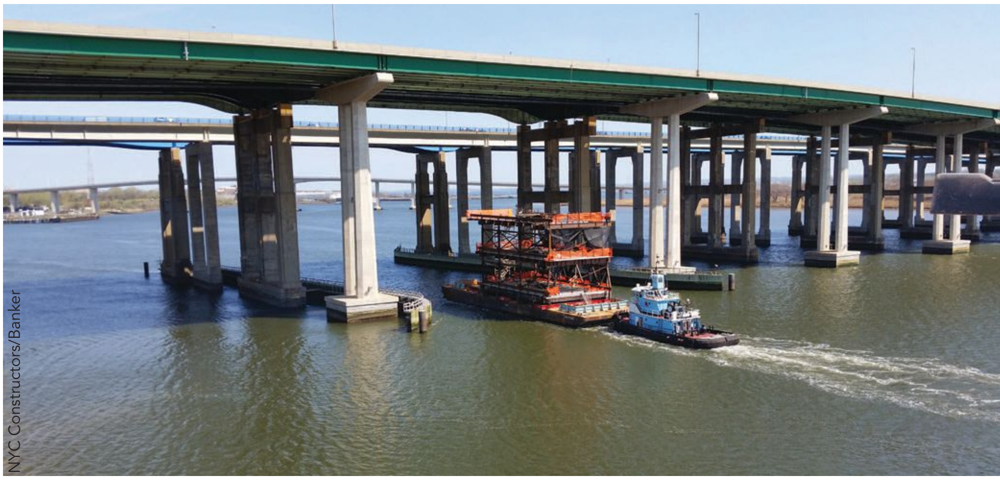
In and Around the Modules

Following installation of the main three-story modules, smaller but equally important framing assemblies have been taking shape on the site in and around the modules while using the modules themselves as lay-down area. The modules are split into two, alternating types: main modules and infill modules. The main modules incorporate the east-west spanning plate girders that the north-south beams connect to—e.g., module 2 (infill) contained north-south spanning beams that connect to the north plate girder on module 1 (main) and

NYC Constructors/Banker

▲ Picking one of the Y-columns.

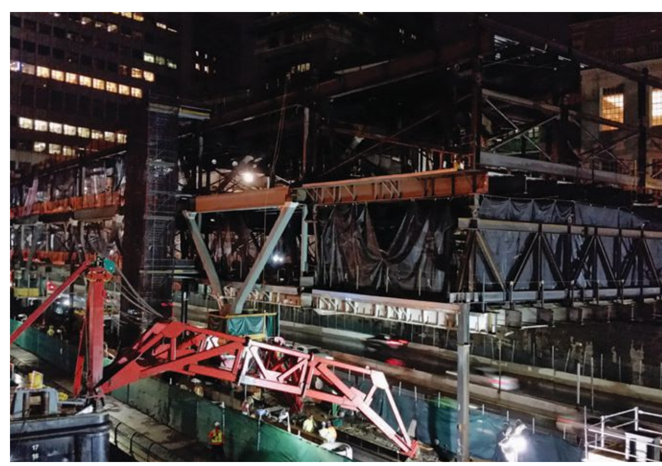
▼ A module in transit under the Garden State Parkway en route to Manhattan.



NYC Constructors/Banker

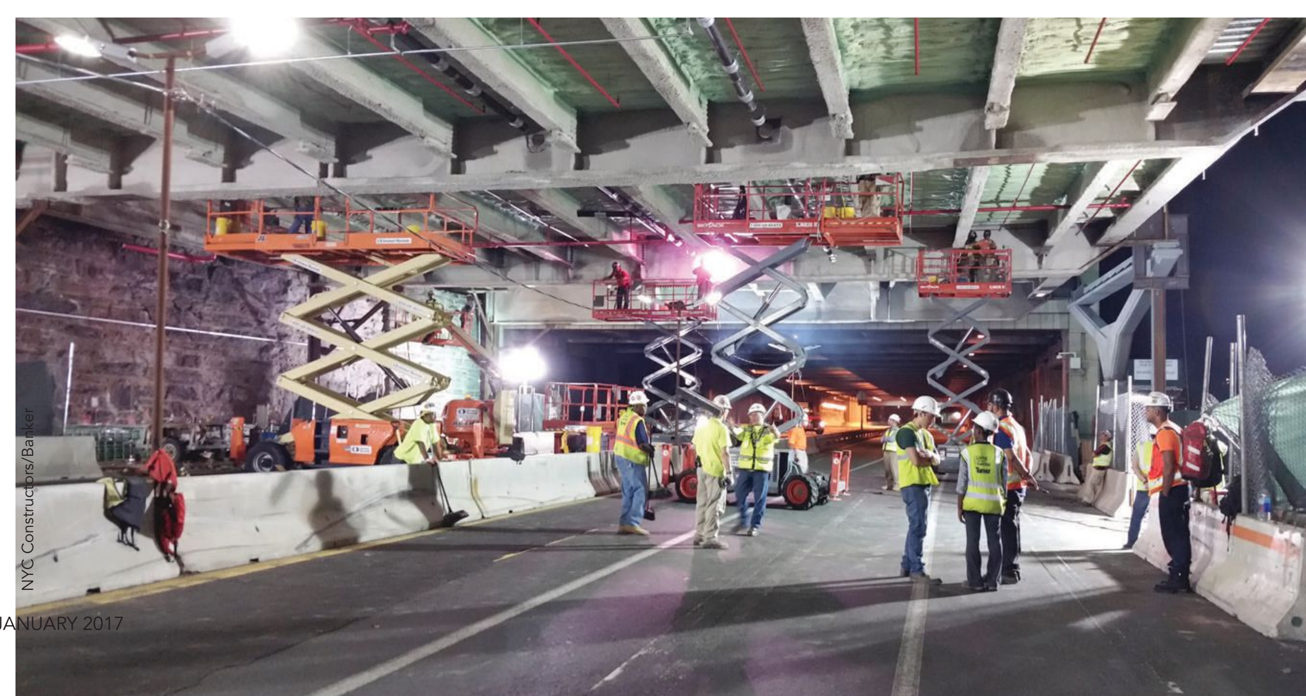
▲ Placing Module 1.

▼ A view of the mooring arm at the site (the red steel).



▲ Loading a module onto a barge.

▼ Work being performed underneath the project on FDR Drive.



the south plate girder on module 3 (also main). These beams required temporary steel to hold them in place from the time the modules were separated in New Jersey until they were connected to adjacent modules at the job site.

On top of the modules, near both ends of the building, are level-four steel-framed "pop-up" assemblies that serve as mechanical space and roof access. The steel members for these assemblies were loaded on top of the modules in the assembly yard in New Jersey and stick-built using a 15-ton-capacity crane that was hoisted from the street after the third module was set. The crane sat on top of crane mats that spanned the level 3 floor steel; concrete for this level was poured on-site due to weight restrictions on the barge.

The project also features somewhat of a contrast to the pop-up assemblies in the form of a rooftop amphitheater, which is sunken into the structure and faces the existing, adjacent building. The outrigger steel for the amphitheater was erected as part of the modules, with some of the minor beams between the outriggers erected by the 15-ton crane where they connected to two modules. The gap in the third floor in the modules where the amphitheater resides created additional work in terms of installing the modules; since there was effectively no third floor to attach to in these areas, the rigging had to be attached to the second-floor plate girders.

Thanks to a combination of off-site assembly, modular thinking and barge delivery, the framing for Rockefeller University's new expansion made the most of the small space it had to work with on the edge of the Upper East Side—and in doing so will give its occupants a larger space to expand their research. ■

Owner/Client
Rockefeller University, New York

General Contractor
Turner Construction, New York

Architect
Rafael Viñoly Architects, New York

Structural Engineer
Thornton Tomasetti, New York

Steel Team

Fabricator
Banker Steel Company, Lynchburg, Va.

Erector
NYCC, Inc.
(a Banker Steel Company), New York

HANGING OUT in Salt Lake City

BY PATRICK M. HASSETT, SE, AND JORIEN BAZA, PE

An elaborate erection and jacking scheme involving an innovative roof-top truss helps pull together the Utah capital's newest Class A high-rise.

FROM THE OUTSIDE—and from three directions—111 Main might look like any other high-rise.

But from a steel design and construction perspective, the project was on a whole new level when it came to erection challenges. For one thing, the 25-story, 502,000-sq.-ft Class A office tower, located in the heart of downtown Salt Lake City, extends 46 ft over an adjacent structure. All 18 perimeter columns are hung from roof trusses framed with jumbo W14 shapes (with heavy node weldments) that rest on six structural spherical bearings at the top of the concrete core, a new scenario for the erection engineer and steel erector.

Tightening Up

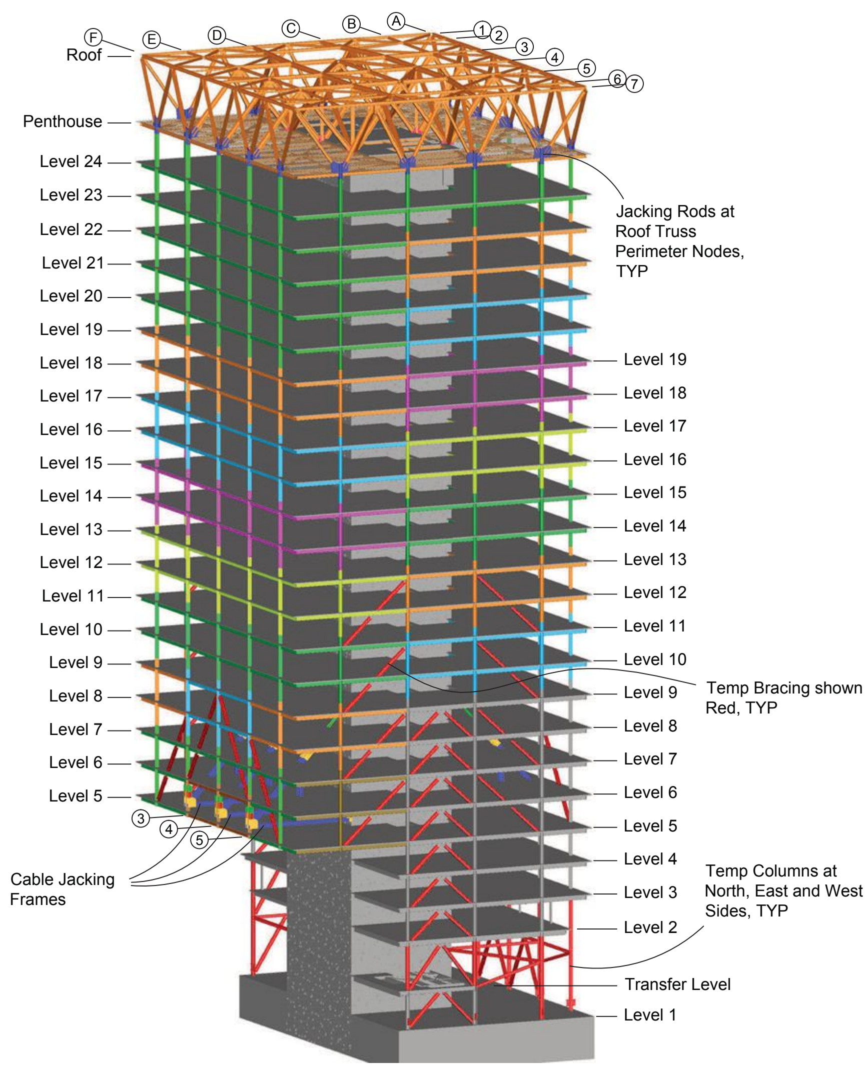
Temporary columns were the obvious solution for the north, east and west sides, but the south side required a different solution—one that would allow steel erection to start at level 5, which would then serve as the support base for the floors above.

The design team at architect and structural engineer Skidmore, Owings and Merrill, LLP (SOM) envisioned a "saddle cable system" that would employ cables through the core and act to balance the placement of construction loads while eliminating any eccentricity on the core that would affect the stress, strain and creep of the reinforced concrete core walls. The challenge for the steel erection team was to come up with a practical solution of framing and cable connections that allowed the tensioning of the cables to account for sag and stretch, as the cable loads increased while the building was being erected. Working with erector SME, erection engineer Hassett Engineering developed a steel-framed concept that minimized the cable length, thereby minimizing cable sag and stretch.

The temporary framing—referred to as "jacking trusses"—was designed to jack up the building columns while simultaneously

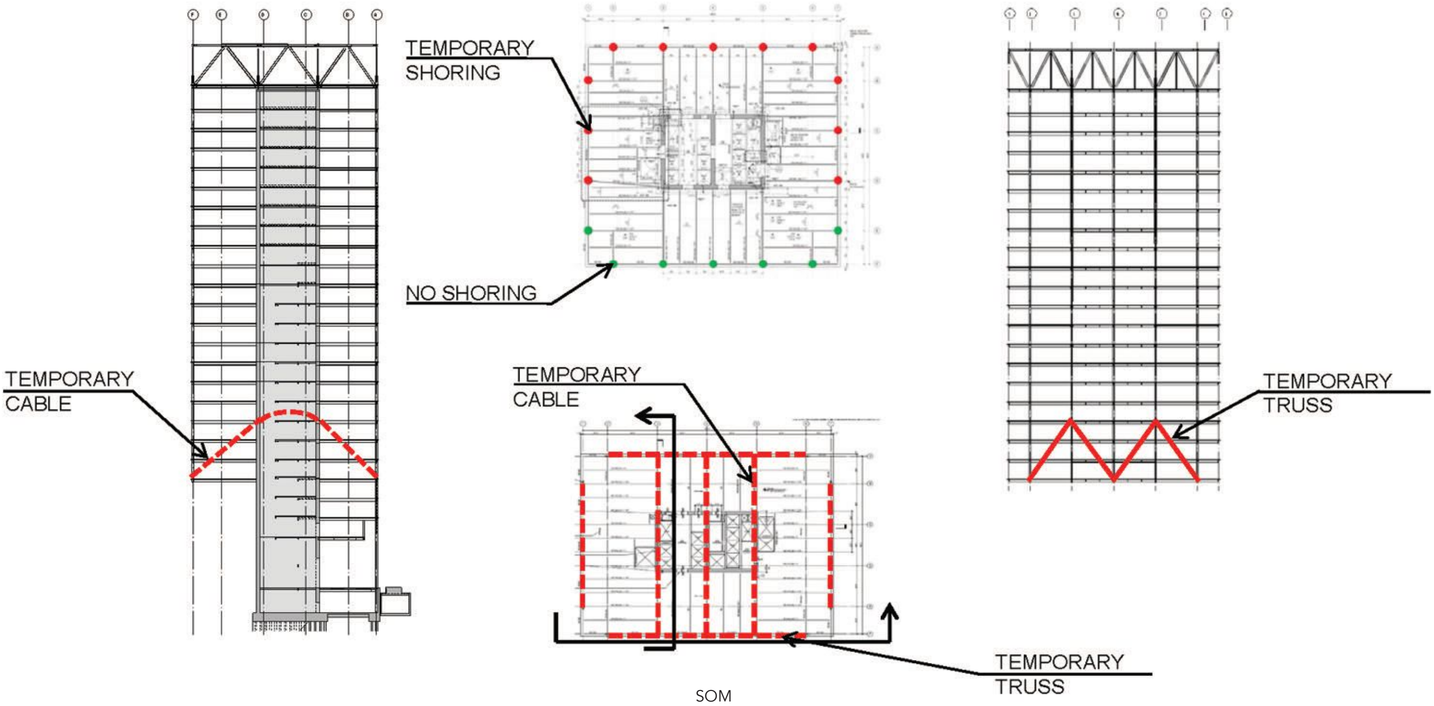
The south side of the building cantilevers 46 ft over an adjacent structure.

All images courtesy of Hassett Engineering unless otherwise noted.



The temporary truss scheme.

Pat Hassett (pat@hassettengineering.com) is president and **Jorien Baza** (jorien@hassettengineering.com) is a project engineer, both with Hassett Engineering, Inc.



The construction sequence of the saddle cable system.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Gen. Hutchison Pkwy
Highland Bridge (Denver, CO)
This award-winning bridge is both dramatic and economical. Chicago Metal Rolled Products' Kansas City facility was able to curve 153 tons of 18" outside diameter tubing up to 100' long, which reduced splicing costs.
Tempe Town Bridge (Tempe, AZ)
Winner of multiple prestigious awards, each of the four 228ft long arching sections consists of 16in diameter HSS pipe curved up to 46ft in good arc....a pedestrian bridge providing both function and aesthetics.
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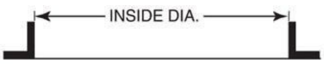
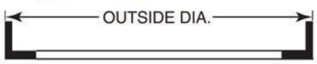
1 Angle Leg Out


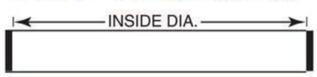
10" x 10" x 1" Angle

2 Angle Leg In


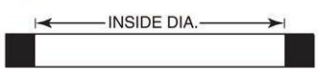
10" x 10" x 1" Angle

3 Flat Bar The Hard Way

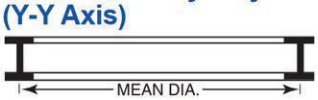

24" x 12" Flat

4 Flat Bar The Easy Way


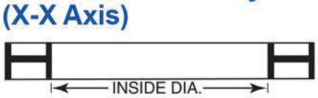
36" x 12" Flat

5 Square Bar


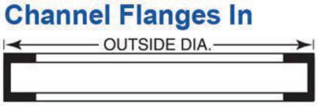
18" Square

6 Beam The Easy Way (Y-Y Axis)


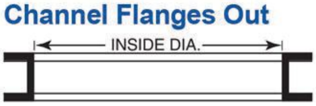
44" x 335#, 36" x 925#

7 Beam The Hard Way (X-X Axis)


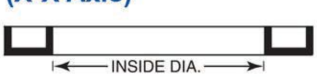
44" x 285#

8 Channel Flanges In


All Sizes

9 Channel Flanges Out


All Sizes

10 Channel The Hard Way (X-X Axis)


All Sizes

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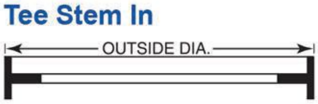
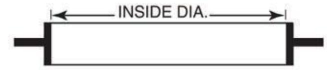


22" x $142\frac{1}{2}$# Tee

We bend ALL sizes up to:

12 Tee Stem Out


22" x $142\frac{1}{2}$# Tee

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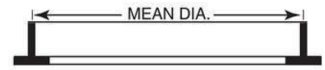

22" x $142\frac{1}{2}$# Tee

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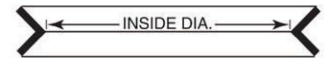
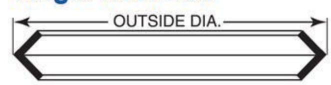
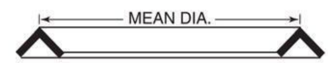


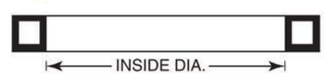
8" x 8" x 1" Angle

15 Angle Heel Out


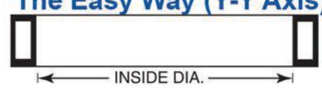
8" x 8" x 1" Angle

16 Angle Heel Up


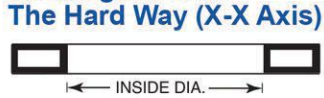
8" x 8"x1" Angle

17 Square Tube


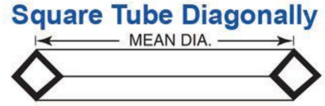
24" x $\frac{1}{2}$" Tube

18 Rectangular Tube The Easy Way (Y-Y Axis)


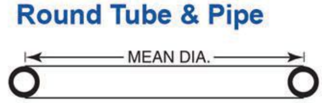
20" x 12" x $\frac{5}{8}$" Tube

19 Rectangular Tube The Hard Way (X-X Axis)


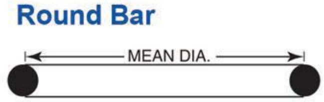
20" x 12" x $\frac{5}{8}$" Tube

20 Square Tube Diagonally


12" x $\frac{5}{8}$" Square Tube

21 Round Tube & Pipe


24" Sched. 80 Pipe

22 Round Bar


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➤ The cable jacking frames and building core.

tensioning the cables. Column lines 3, 4 and 5 would be framed with jacking trusses and the saddle cables supporting the south side, and anchored with similar framing on the north side to columns that extended down to the foundation.

After extensive review of available cable types, 4-in.-diameter ASTM A586-04 were chosen. The cables were pre-stretched to provide an effective modulus of elasticity of 23,000 ksi and rated with a breaking strength of 997 tons (which translated to an allowable capacity for construction loading of 997 kips). There were two cables per grid line for a total of six, and they were installed through the 30-in.-thick concrete core walls on a circular radius in a lubricated steel conduit. Column lines 2 and 6 were coupled to lines 3, 4 and 5 through temporary bracing on the south face of the building between levels 5 and 9, and were consequently supported by the jacking trusses as well. Lines 1 and 7 had a different problem: There was no core to anchor through, so a braced frame was designed to carry the hanging and the associated lateral component.

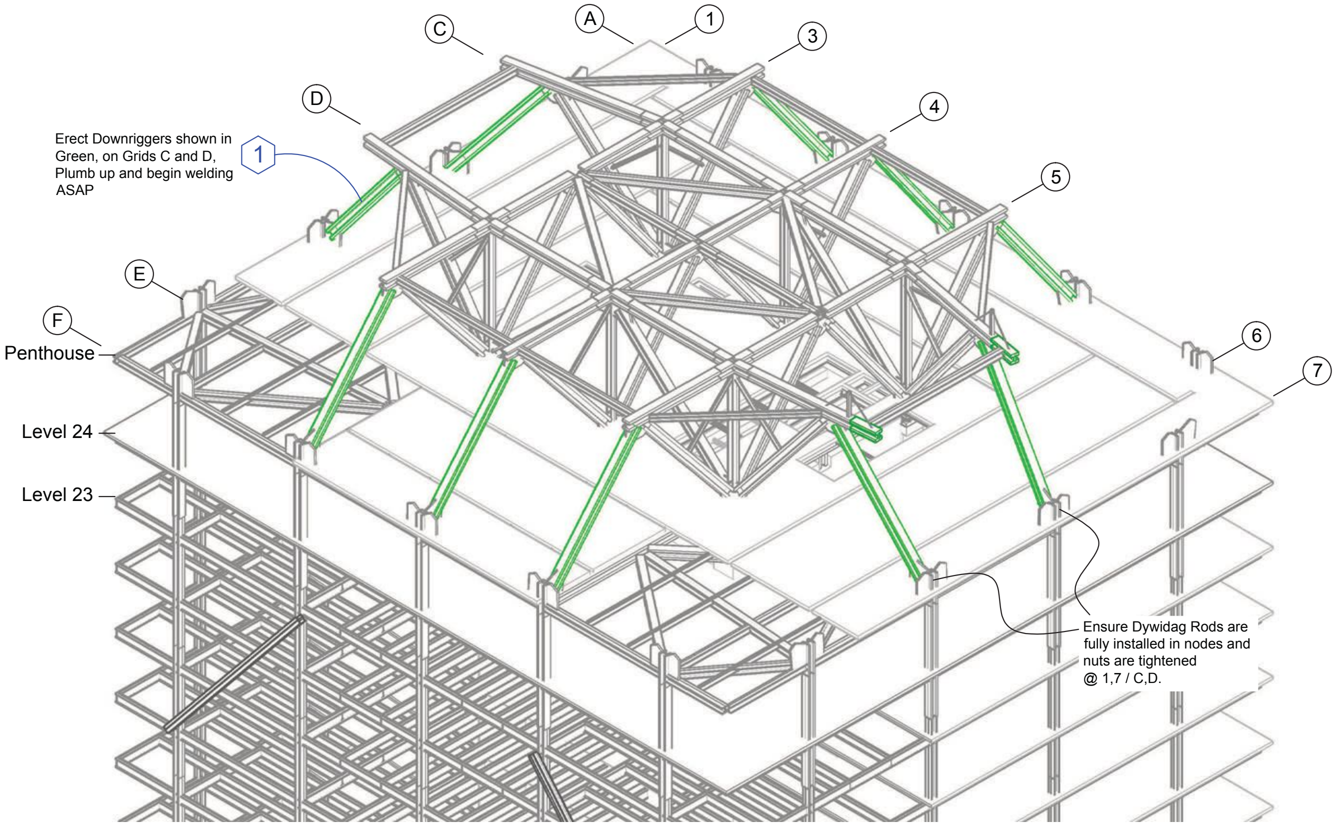
Transferring load from the temporary shoring system to the roof hat truss once the structure was complete was another challenge. The initial concept was to pull up the perimeter columns using jacks at the hat truss nodes. Through collaborative meetings between SOM, SME and Hassett Engineering, this scheme was changed to a "lowering" of the columns at the base. The solution allowed gravity do the work as opposed to pulling up, fighting against gravity and the stiffness of the building itself. With this scheme, the perimeter of the building needed to be erected to a higher point than the final theoretical elevation. Since the core does not move during load transfer, the slab and floor beams would rotate about the core walls. Concrete pour strips were implemented adjacent to the core, and special pinned beam connections were developed to allow rotation without adding extra stresses into the system.

As is typical for a building of this type, the permanent columns are designed largest at the top where the tension is the highest, and smaller at the bottom. During construction, however, the columns were supported from below and saw large compressive forces most notably in the lower columns. Through collaboration with SOM, the columns were checked and resized as necessary to handle the temporary construction loads.

Tier by Tier

Erection of the composite steel construction followed a typical sequence of two stories at a time, tier by tier, and concrete slabs were poured via normal sequencing four floors (give or take) behind the erection. The column lift

➤ A jacking frame and column bracket.



The main axial members of the hat truss (in green).

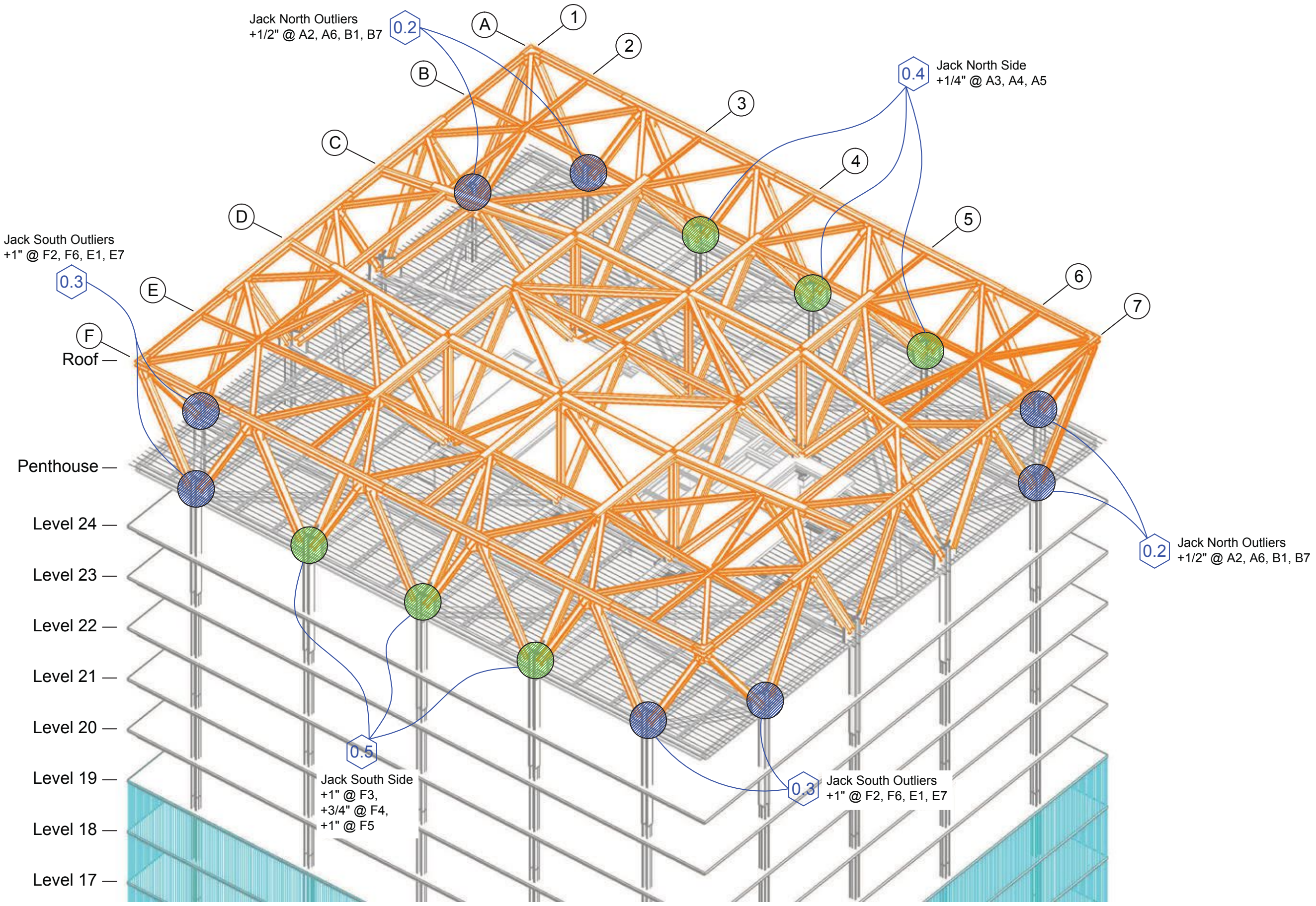
(via jacking) was estimated to require ⅜ in. at every tier, accounting for deflection due to two slabs and two floors of steel. The jacking frame was designed to deflect by rotating about a pin connection at the core. The two 500-ton jacks at each column were designed to push down on the jacking frame while pushing up on brackets welded to the flanges of the columns. This would provide the elevation adjustment for the cantilevered south side throughout the project. Loads were measured on the jacks and compared to the theoretical loads, which were calculated using an ETABS model.

However, after a few tiers were erected, the jack readings were showing higher loads than expected. Unable to capture the behavior in the ETABS model, the team determined that some load transfer (by plate action of the slabs) was occurring between jacking lines 3, 4 and 5 and braced frame lines 1 and 7. Connections for the jacking trusses and the bracing for lines 1 and 7 were reinforced to accommodate the additional loading, and the jacking magnitude at each adjustment was minimized to keep the floor elevations within acceptable tolerances while ensuring that the jacking frames would not be overstressed.

As with any high-rise project, the team had to pay close attention to the "differential shortening" or compression strains of the perimeter columns relative to the core structure. But in the erection scheme for this project, the columns were in compression, then finally in tension, and the floors lost elevation due to the deflections of the roof truss. Anticipating this in the design phase, the team performed analysis to determine what, if any, column length changes would be needed. Column lengths per tier would be required to be ⅛ in. shorter than theoretical. However, SME's field experience told us that there would be approximately ⅛ in. of weld shrinkage at the column splices. Therefore, the columns were detailed to their theoretical lengths, thus eliminating complexity in the modeling for the column shop drawings.

Since the columns would compress during shored construction, then stretch and deflect (due to hat truss deformation) after hanging, a specific, unique "set-high" elevation was required at each column of each floor that would vary at any given stage of construction. The erection plan also required the bottom level of each column line erected to an initial set-high elevation, varying between 3 in. and 4.5 in., to account for deformation of the temporary steel during construction, final column stretching and final hat truss deflection.

Similarly, the perimeter nodes of the roof hat truss were erected to a "tip-up" elevation between 1.5 in. to 2 in. to account for the final truss deflection alone. To complicate matters, the schedule required the curtain wall glass panels to be installed during steel erection, prior to jacking down the building and transferring the building weight from the shoring to the roof truss. The glass panels were designed to allow greater than normal tolerances due to movement after installation. As such, general contractor Okland Construction performed extensive survey monitoring on a weekly basis so that fine adjustments to the preset elevations of



Pre-jacking adjustments to the hat truss.

each column could be made in order to ensure the integrity of the panels. The target maximum differential vertical movement between adjacent columns was +/-½ in.

Making Adjusments

Erection proceeded quickly up to level 24, with the south side column elevation adjustments made close to the theoretical estimate of ⅜ in. per tier. As the stretch of the cable continued, it would slip through the core, occasionally breaking friction with a noticeable bang. As the roof truss began erection with the heavy members and nodes, the concept was to start at the core, landing the nodes on the structural spherical bearings and work out towards the four perimeter column lines. This procedure allowed welding to start at the core and work outward, thus allowing relatively symmetrical and unrestrained weld shrinkage. In addition, the main axial members of the trusses could soon be able to support their own weight, carrying load from the perimeter toward the core.

Jacking rods were installed on the bottom chord of the roof truss as a backup plan to make further elevation adjustments, which would involve pulling up the columns and pulling down the trusses. This backup scheme proved to be very useful in adjusting the floor elevations, as the floor elevations at the south side and outlier columns were lower relative to the other grid lines. These "outlier columns" were columns outside of the main truss lines and were supported by perimeter trusses off the main trusses at the roof. Because the measured loads during jacking indicated that more jacking would stress the saddle cables beyond their intended design loads, the decision was made to leave those three columns relatively low.

Relying on the carefully developed survey data provided by Okand Construction, and correlations made with ETABS erection sequence modeling, adjustments at the roof were done in four phases:

1. The four outliers on the north were jacked ½ in.; these column grids then could provide some pretension for the supporting hat truss, acting as an abutment
2. The four outliers on the south were jacked 1 in.
3. The columns on line A at 3, 4 and 5 were jacked ¼ in. to provide further pretension for the next jacking.
4. The columns on line F at 3, 4 and 5 were jacked 1 in., ¾ in. and 1 in., respectively. By making these small final adjustments at the roof level, the roof trusses were effectively preloaded and the erection team was able to set the geometry closer to the theoretical elevations prior to jacking down the building

Once the columns were welded to the nodes at the main truss lines (3, 4, 5, C and D) and the rods were engaged at the outlier columns, the building was ready to be lowered into position. These outlier columns were expected to deflect more than the main truss line columns. Therefore, they were left un-welded to the nodes so that they could be jacked later for a final floor elevation adjustment at those grids.

A tension release at a braced end-plate connection.

Once the roof welding and inspections were complete, the building was ready to be jacked down and the load transferred to the two-way roof "hat" truss system. A hydraulic control system was placed on level 5 at the south (line F) to control the six jacks at the three jacking frames, and another hydraulic control system was placed at level 1 (ground level) to control the 22 jacks and 11 columns. As the jacking-down process proceeded, surveyors were placed at level 5 and at the roof and were in radio contact with Okland's superintendent. SME, Hassett Engineering and SOM had continuous communication between the ground and level 5 jacking, giving the go-ahead for each simultaneous jacking-down of ⅛-in. increments. The glass curtain walls and MEP systems were being monitored by their respective subcontractors as well. As the load was transferred to the hat truss and the load on the temporary cables was reduced, cable slippage back through the core was observed as the cables occasionally broke friction with noticeable bangs, as with initial erection.

Winding Down

Jacking-down was performed in small increments until an accumulation of 1 in. at the ground and ¾ in. at level 5 was achieved. Column elevations and jacking loads were then recorded, reported to the team and reviewed; upon approval, jacking-down would recommence. A three-to-four ratio was used since previous fine adjustments at the roof jacking rods had preloaded the south side more than the north, resulting in less required jacking-down at the south for full load transfer. Furthermore, in order to achieve the required ¾ in. of jacking-down at level 5, a total jack movement of 3.5 in. was required. This was due to the jacking trusses moving upward (since they were unloaded) at the same time the building was moving downward. After each 1-in. lowering step, jacking forces and survey elevations at levels 5, 15 and 24 were reported.

Elevations and loads were compared to theoretical predictions, and the curtain wall deflections were checked to be within tolerance before proceeding to the next step. This process was repeated until the jacks were unloaded and the building was fully supported by the hat truss above. Total jacking-down at the columns on the ground varied between 3 in. and 4 in., and total jacking at level 5 was about 8 in. for a column movement of approximately 1.7 in. Cable movement through the core was between 3 in. and 3.4 in.

Further adjustments to column elevations were inevitable since the floors would continue to deflect due to the pouring of the level 25 concrete slab, the removal of the temporary braces and the completion of the façade and remaining dead load. The worst predicted deflection locations were at the outlier columns, as they were supported by the more flexible parts of the hat truss. After load transfer, about ¼ in. of jacking was performed at the roof level on the four outlier columns on lines A and F, north and south, respectively. Removing the temporary braces or releasing the connections caused a redistribution of forces throughout the building and resulted in further deflection, predominantly at the outlier columns on the south, where the braces were retaining much residual tension load. This was done by loosening the tension

Jacking adjustments at the roof node.

rod nuts in the end-plate connections of the braces, with a resulting gap spread of between 1⁄16 in. and 3⁄16 in. between end plates.

Finalizing the roof slab pour only added to the deflections. In response, and in anticipation of the remaining façade load and other dead loads, a final adjustment of 0.4 in. was performed at the two south outliers on line F in order to bring the floor elevations closer to theoretical. These outlier columns were then welded off, essentially securing the building into its final state and intended structural system load path. The façade, having survived several changes in elevation and geometry, was given a final adjustment.

The steel structure of 111 Main was built from the ground up and went from being supported at the bottom levels to hanging from the hat truss above—basically 1⁄8 in. at a time—proving that steel was ideal not only as a framing system, but also in terms of constructability for an especially intricate erection operation. ■

Owner
City Creek Reserve, Inc., Salt Lake City

General Contractor
Okland Construction Company, Salt Lake City

Architect and Structural Engineer
Skidmore, Owings and Merrill, LLP, San Francisco

Erection Engineer
Hassett Engineering, Inc., Castro Valley, Calif.

Steel Fabricator, Erector and Detailer
SME Steel Contractors, Inc., AISC CERTIFIED FABRICATOR
West Jordan, Utah

Leveling UP

BY PAUL TAYLOR SE, PE, AND MELISSA JURGENS

Strong, open steel design drove a university football facility project built to raise the profile of an already successful program.

Paul Taylor (ptaylor@saul.engineering) is a senior engineer and **Melissa Jurgens** (mjurgens@saul.engineering) is a client development engineer, both with Saul Engineering.

WHEN IT COMES TO ATHLETICS at the collegiate level, facilities are increasingly becoming part of the package to woo potential recruits to campus.

For the University of Iowa's Hawkeye football program, the school is banking on its new Stew and LeNore Hansen Football Performance Center to be one component in getting talented athletes from around the country and beyond to come play for Iowa. The new facility consists of three components: an indoor practice facility, the renovated Ronald and Margaret Kenyon Outdoor Practice Facility and the Richard O. Jacobson Football Operations Building.

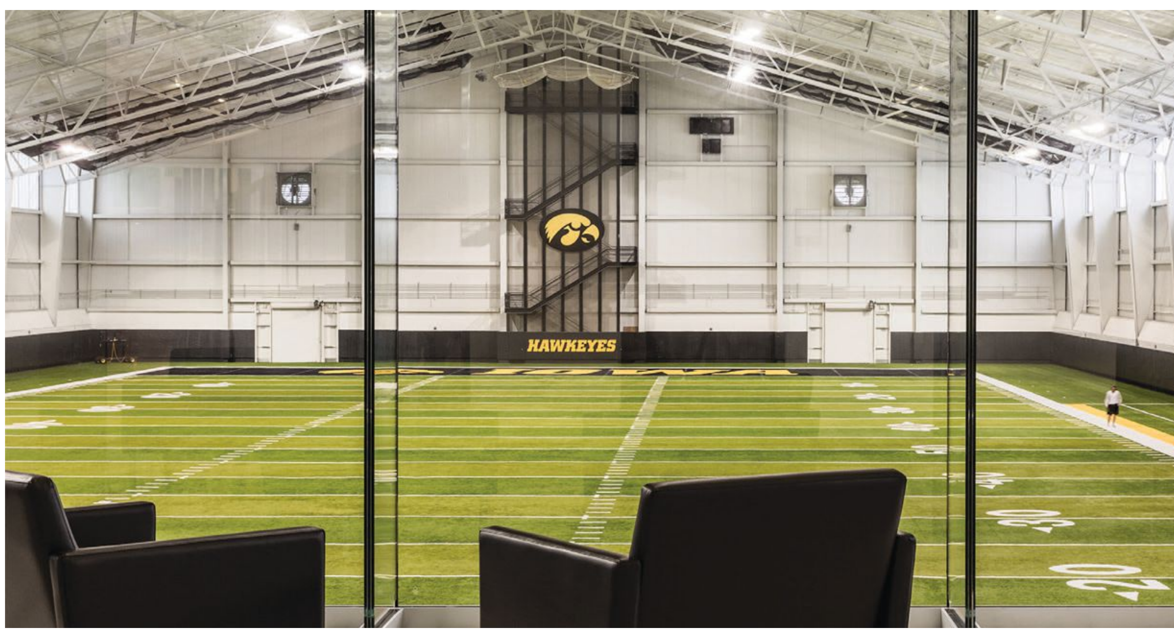
The indoor practice facility and supporting storage spaces replace an inefficient air-supported structure that had outlived its useful life. This facility is connected directly to the operations building, which provides updated training and classroom spaces for the team as well as exhibit space dedicated to the history and success of Iowa football. Public exhibits in the entry hall extend along a "recruiting

Photos by Paul Crosby
Architectural Photography

The operations building uses a steel framing system that addresses its aesthetic considerations.

The building's 42-ft-high roof cantilevers 14 ft, 6 in. above the main entrance.

The indoor practice facility replaces an inefficient air-supported structure that had outlived its useful life.



path" through the building, connecting the program's history with its current achievements and ongoing goals. The facility includes a large reception area opening onto a multiuse meeting and press space with views to Kinnick Stadium, which is located across the street. It also houses strength and conditioning space, athletic training facilities and hydrotherapy, team meeting rooms, position group rooms, equipment and locker rooms, video production and offices for coaches and staff members. All aspects of the football program are physically connected and planned around efficient interaction between student-athletes and coaches.

While the indoor practice facility uses a pre-engineered metal framing system, the bright, open operations building uses structural steel (roughly 650 tons) to address the project's design and aesthetic considerations—and the sleek, muscular framing system is prominently displayed. Branding and overall experience were important aspects of the design (the tiger hawk logo representing the school's athletic program is prevalent), and the expansive glass exterior entry is mirrored on the interior, welcoming people into a large two-story volume of glass and steel. The exposed steel columns in this space are W10×88, creating the appearance of an equal flange-to-depth ratio. This 10-in. rhythm is mirrored at the beam-to-column connections as well at the exposed framing of the W16×67 beams, matching the column's flange widths. The interior glazing system abuts the column flanges to visually incorporate the emphasized symmetry within the defined space.

To continue to meet the architectural intent of visual balance, the strength and conditioning space roof is framed with single-pitch open-web steel roof joists (7 ft at the low end; the high end depth varies by span), which provides a large, flexible column free space. Although the spans of the roof joists decrease in the triangular shaped room, the depths and web members of the joists are identical.

The interior glazing system abuts the column flanges to emphasize symmetry within the space and put the steel framing on prominent display.

One of the building's signature steel elements is the 42-ft-high roof over the reception and meeting space, which cantilevers 14 ft, 6 in. from the supporting interior column gridline above the main entrance. To maintain a thin roofline, cantilevering HSS18×6×½ LDH members were used, and an HSS8×2×⅜ LDH member was welded to the underside of the members to brace the top of the exterior glazing system. An additional canopy overhang was provided at the mezzanine height of the entry ramp. Independent columns support the lower steel and glass canopy, with a slotted connection bracing the canopy back to the mezzanine story. Tapered and sloping WT8×18 sections allowed the visible edge of the 4-ft, 3-in. cantilevered canopy at the entry to appear as a thin 3½ in.

On the exterior, the red brick and cast-in-stone sills and jambs of the windows required varied detailing. This included HSS members ranging from 10 in. to 14 in. in depth spanning the 24-ft column spacing to support the sill and jambs of the ribbon windows on the east façade of the building. The red brick façade echoes that of Kinnick Stadium, while the building's precision in design and execution echoes the goals of the team it supports.

The Hansen Football Performance Center is the final component of The Iowa Football Legacy Campaign, which began in 2002. With its strong, open aesthetic, it elevates the program's physical home to a

The multiuse meeting and press space.

Public exhibits in the entry hall connect the program's history with its current achievements (including a mostly full case displaying trophies from this year's rivalry games) and ongoing goals.

All aspects of the football program are physically connected and planned around efficient interaction between student-athletes and coaches.

new level, which will hopefully translate to increased success on the field. ■

Owner
University of Iowa, Iowa City

General Contractor
McComas-Lacina Construction, Iowa City

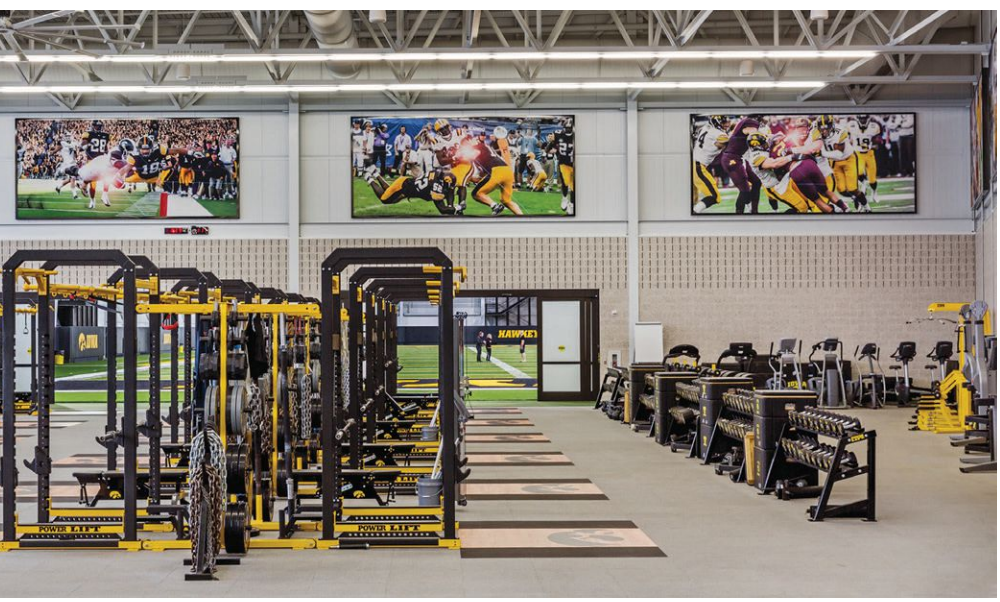
Architect
Substance Architecture, Des Moines

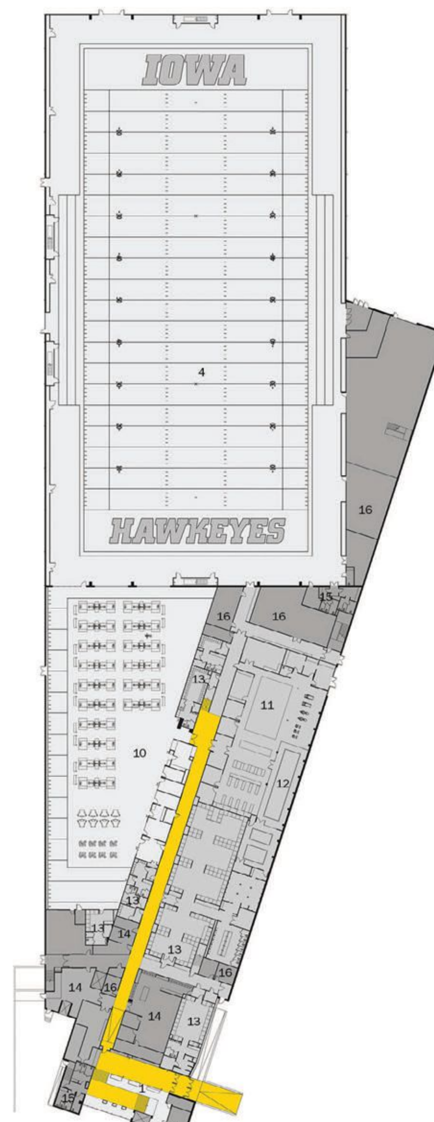
Structural Engineer
Saul Engineering, Des Moines

Steel Fabricator and Detailer
Johnson Machine Works, Inc., Chariton, Iowa

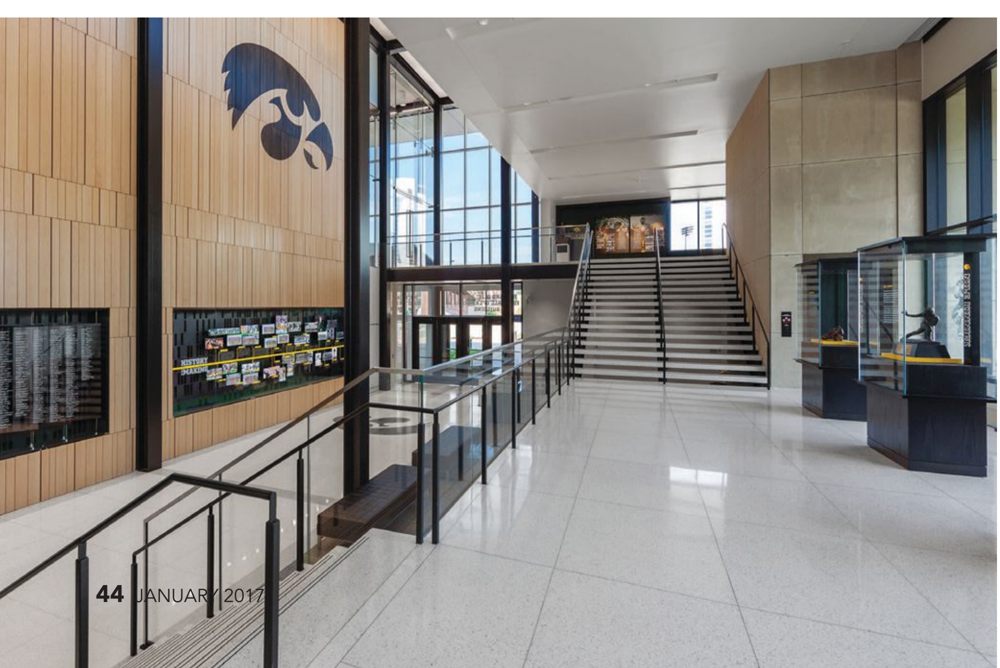
▲ The building's red brick façade echoes that of Kinnick Stadium.

▼ The weight and strength training area, topped with steel joists.





▲ The facility layout.

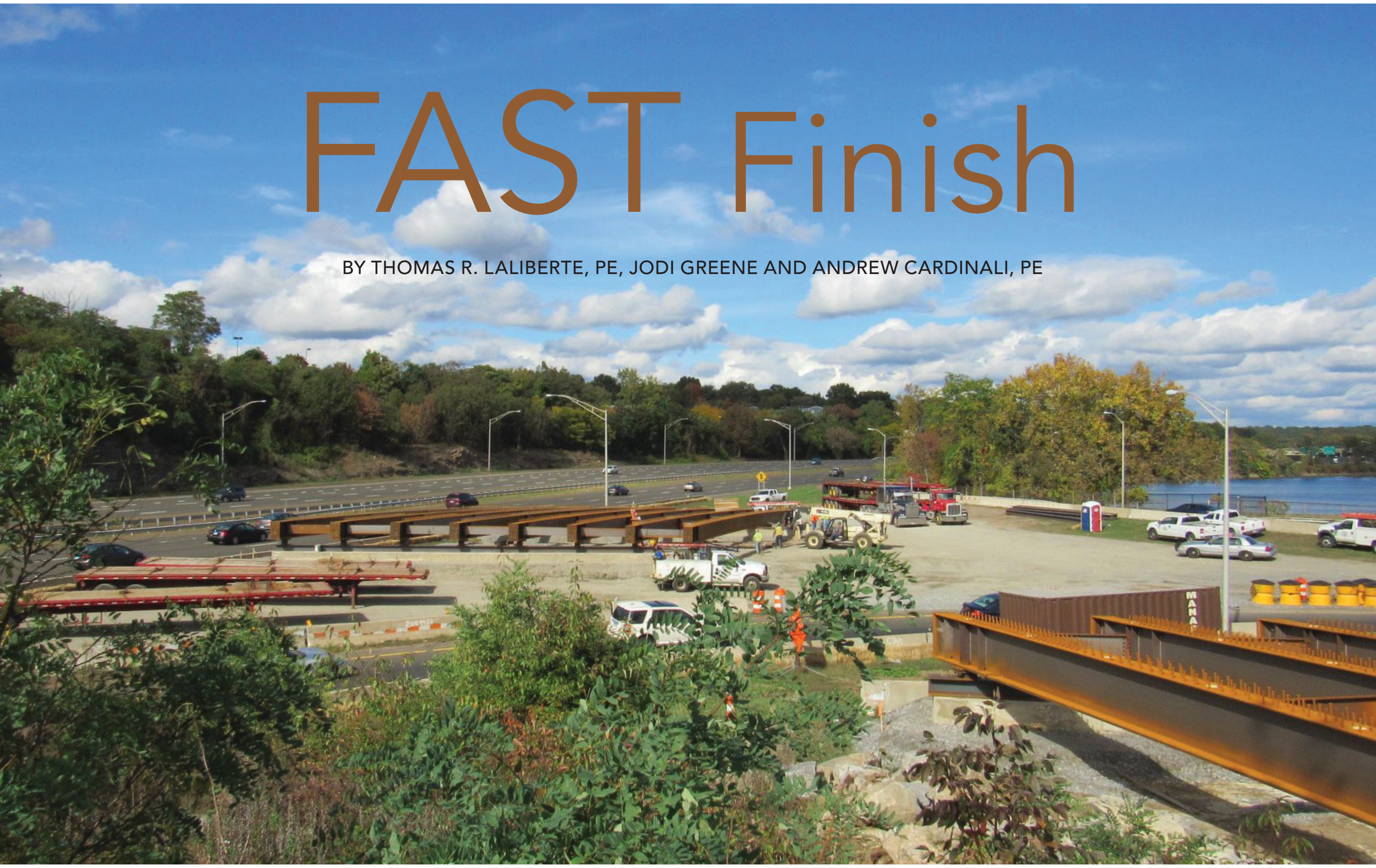


◄ The expansive glass exterior entry is mirrored on the interior, welcoming people into a large two-story volume of glass and steel.

The Connecticut DOT
makes quick work of its first-ever design-build project.

FAST Finish

BY THOMAS R. LALIBERTE, PE, JODI GREENE AND ANDREW CARDINALI, PE



At the lay-down area, temporary or "mock" abutments were constructed to match existing bridge substructures, which were reused for the proposed bridge.

WITHIN 15 MONTHS from the notice to proceed for design, four bridge superstructures carrying Route 8 over Lindley Street and Capitol Avenue in Bridgeport, Conn., were fully replaced and open to the public.

The bridges were originally built in 1970 with prestressed concrete girders. These bulb tee girders experienced premature deterioration caused by alkali-silica reactivity, and the failed bridge joints created sub-

Thomas R. Laliberte (lalibertet@pbworld.com) is the Connecticut structure lead and the lead bridge engineer for the Route 8 project, and **Jodi Greene** (greenej@pbworld.com) is a bridge engineer; both are with WSP|Parsons Brinckerhoff's Connecticut office. **Andrew Cardinali** is a supervising engineer for CTDOT and was the owner's project manager for the design of the Route 8 project.

structure spalls and bearing damage. The poor superstructure conditions are what prompted this rehabilitation project.

Hoping to complete the project within a shortened period, the Connecticut Department of Transportation (CTDOT) conducted a risk analysis that determined this project was appropriate for its first design-build endeavor. The design-build project delivery method allowed general contractor Manafort Brothers, Inc. (Manafort) and engineer WSP|Parsons Brinckerhoff (WSP|PB) to develop innovative design and construction solutions from the start of design through the end of construction. Design began in April 2015, girders were delivered to the site in October and all bridges were replaced and in service by July 2016.

Crossover

A significant portion of the project involved preparing for two 14-day crossover periods when the superstructures were replaced. Due to the geometry of the existing highway alignment, it was possible to shift both north- and southbound lanes onto the original northbound highway during the crossover period. This method was repeated in the opposite direction as well. When traffic was diverted to one side of the highway, demolition of the existing and construction of the new structures was performed on the opposite side. Various accelerated bridge construction (ABC) techniques were used to replace the superstructures in less than weeks, and one specific technique was the use of prefabricated bridge units (PBUs). The short construction schedule relieved highway users (the average daily traffic volume on the bridges is 89,300 vehicles) and the community of Bridgeport from long-term construction impacts.

The PBUs were designed to be constructed off-line, then erected during the crossover periods. They consisted of two steel plate girders and a concrete deck, and the magnitude of the units made transportation and erection challenging. The PBUs were up to 90 ft long by 17 ft wide and weighed up to 260 kips. Manafort assembled the units at a site less than one mile from the bridges, then they were transported to the project site with 16-axle highly specialized steerable and adjustable trailers, then finally tandem-picked into final position with 300-and 500-ton cranes.

➤ WSP | PB designed prefabricated bridge units for the Route 8 bridge project.

➤ Traffic was shifted to one bound during each crossover period. The two sides were replaced in under 14 days.

At the assembly area, temporary or "mock" abutments were constructed to match existing bridge substructures, which were reused for the new bridges. Each bridge span included four PBUs that were constructed together full width and length to ensure fit-up during erection. All steel members were erected prior to casting the deck, then the diaphragms between the PBUs at the closure joints were removed when it was time to transport the units. As a result, minimal adjustment occurred at the final site.

Switch to Steel

The bridge design focused on reducing the weight of the superstructures for both final condition and construction purposes. For this reason, steel girders were used instead of the concrete scheme of the original bridges. The overall superstructure weight reduction prevented the need for advanced geotechnical analysis on the existing abutments and piers, which were reused for the new bridges. The switch from concrete to steel girders resulted in lighter PBUs, ultimately saving on the costs associated with larger capacity cranes and transportation devices.

The design-build delivery method prompted innovative use of steel girders that resulted in benefits to safety, schedule and budget. The steel girder design reduced superstructure depths; at controlling locations, minimum vertical clearances increased from 14 ft, 5 in. to 15 ft, 11 in. for local roads below, improving safety to the traveling public. The design also allowed for an alteration that reduced the number of girders per span from twelve to eight, which decreased the number of PBUs and required cast-in-place deck closure pours. The ripple effect of this change saved both cost and time for the required materials, fabrication, erection and construction of each bridge superstructure.

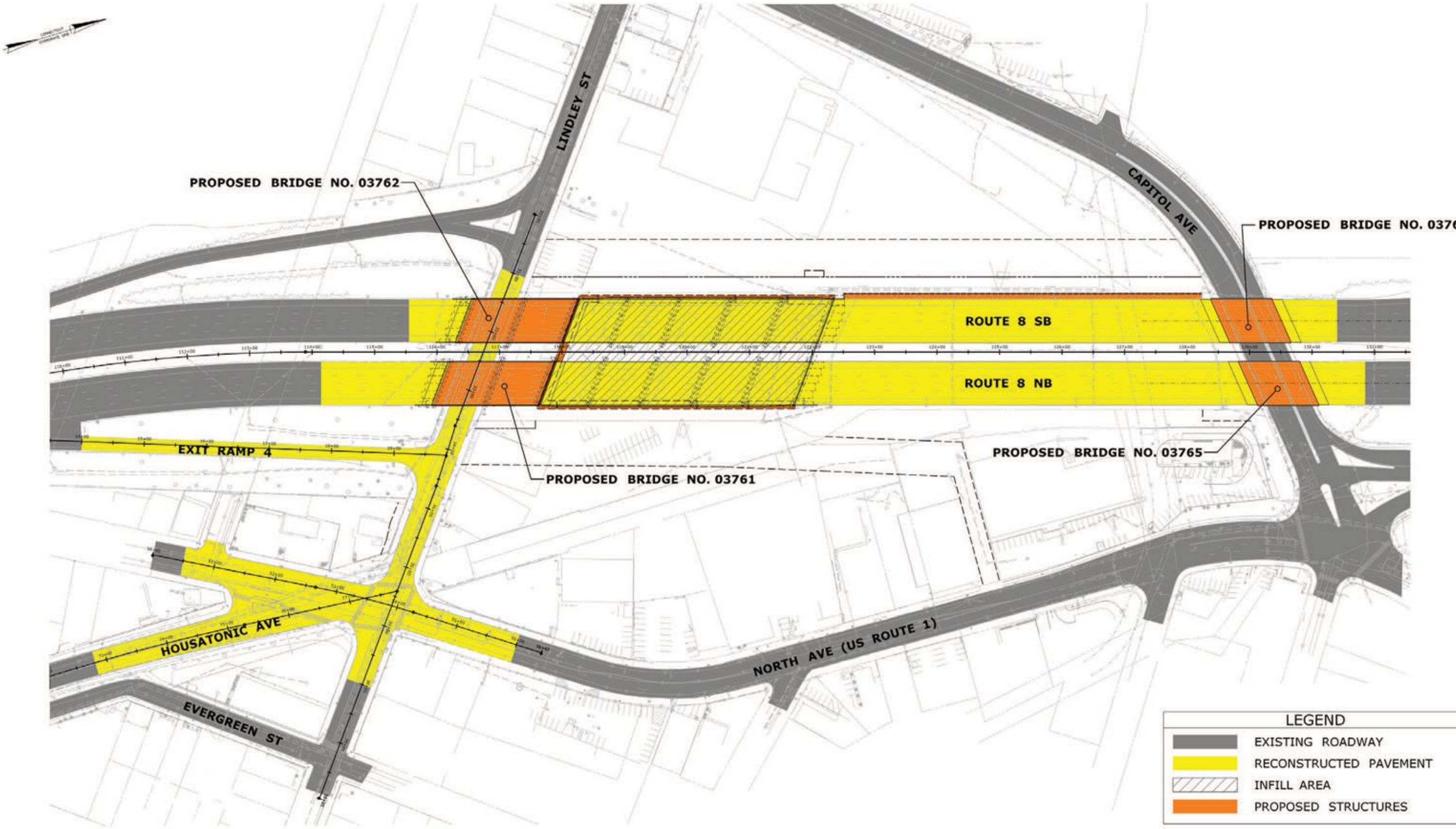
Reduced Maintenance

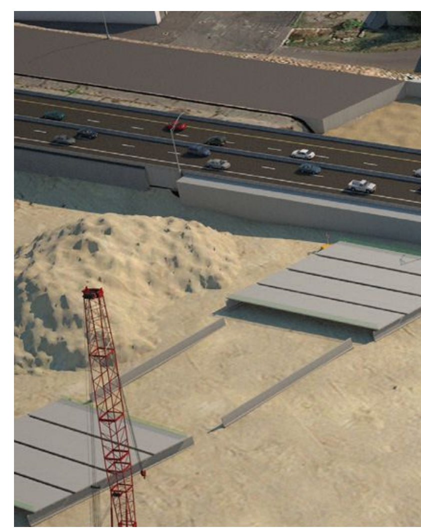
In the long term, the replacement project will reduce bridge maintenance. The existing bridges over Lindley St. consisted of seven spans, and the new configuration filled in five spans and replaced the final two, resulting in ten fewer spans to inspect and maintain; retaining walls were constructed alongside and below the existing bridge prior to the crossover period to accomplish this. The existing bridges were carefully demolished adjacent to these walls, and fill brought the grade up to match the elevation of the bridge approach.

In addition, the bridge superstructeres are built from weathering steel, which will eliminate the need for future painting. The design incorporates link slabs and semi-integral abutments, which will eliminate bridge joints and associated damage to bearings and concrete, a problem on the original bridge. The existing structures only served for 46 years, and CTDOT was keen on avoiding premature replacement in the future. The new bridges have an expected service life of 75 years.

The two crossover periods required round-the-clock construction activity, with more than 100 employees working to further accelerate the schedule. Manafort completed the closure periods a total of four days ahead of the 28-day schedule,

Highly specialized steerable and adjustable trailers were used to transport the PBUs on local roads to the bridge location.





- The design team produced visualizations for public information meetings to communicate the project sequencing to the community.
- Construction activities were schedule around the clock during the crossover periods.
- The bridge cross section was composed of four PBUs with closure pours between units.

An overhead view of the project layout.

and one local paper noted that the project was "likely the fastest bridge replacement project ever seen in Fairfield County." ■

Owner
Connecticut Department of Transportation

General Contractor
Manafort Brothers, Inc., Plainville, Conn.

Structural Engineer
WSP | Parsons Brinckerhoff, Glastonbury, Conn.

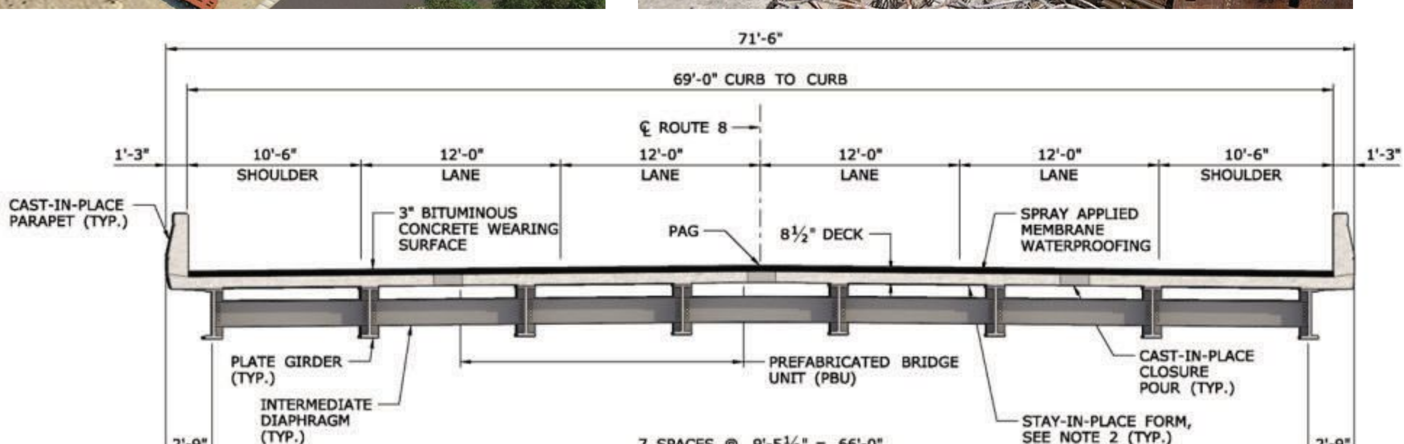
Steel Team

Fabricator
High Steel Structures, LLC, Lancaster, Pa.

Erector
Hartland Building and Restoration Co., East Granby, Conn.

The PBUs were assembled about a mile from the final bridge locations.

Seven spans of the existing bridge were demolished within hours of the start of crossover.



TYPICAL CROSS SECTION

A Utah bridge team turns to structural steel solutions for expanding an Interstate crossing to accommodate high-occupancy vehicle lanes.

HIGH Volume, LOW Impact

BY JASON KLOPHAUS, MICHAEL GOODMAN AND CORIN PIACENTI

Jason Klophaus (jason@klophausllc.com) is the owner of Klophaus and Associates. **Michael Goodman** (goodmanmp@pbworld.com) and **Corin Piacenti** (piacentice@pbworld.com) are senior bridge engineers with WSP|Parsons Brinkerhoff.

◄ The sliding system involved an end diaphragm with the stainless steel slide shoe on Teflon bearing pads.

▲ The permanent abutment between the temporary abutments provided a level sliding path.

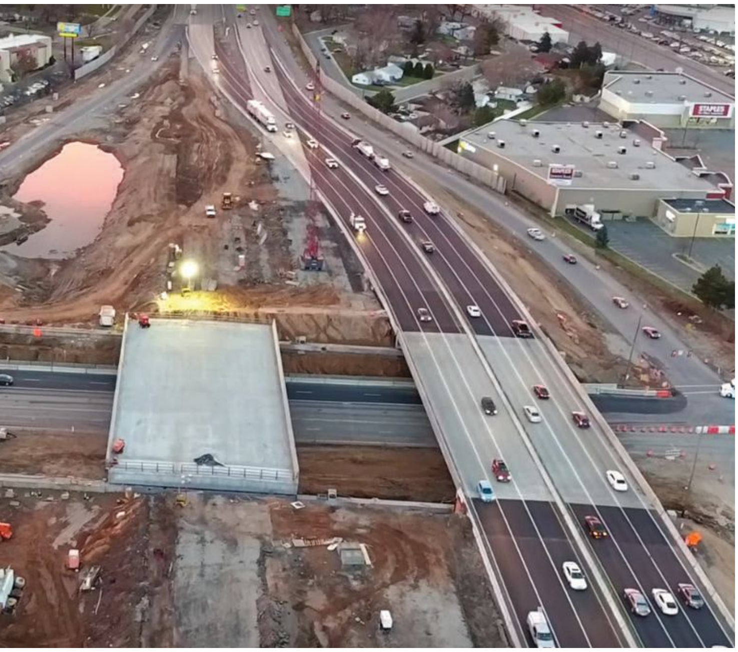
AS UTAH'S ONLY north-south Interstate, I-15 is critical to the state's infrastructure.

Average annual daily traffic in 2012 was approximately 65,000 vehicles in each direction and is predicted to rise to over 80,000 by 2040. To brace for this increase and help address the potential for the delays it would likely incur, the Utah Department of Transportation (UDOT) has implemented systematic capacity improvements through the addition of high-occupancy vehicle (HOV) lanes in the Salt Lake City vicinity.

One prominent interchange where HOV lanes were recently implemented is the I-15–Hill Field Road interchange located 25 minutes north of Salt Lake City. A pair of existing three-span bridges carried the I-15 traffic while local traffic moved along Hill Field Road below. The goal of the project was to provide additional structure width for HOV lanes as well as to upgrade the intersection to a single-point urban interchange (SPUI) configuration. This configuration allows large volumes of traffic to move through limited space by providing multiple turning movements below the bridges while high-volume traffic flows on I-15 above.

Using the design-build method, the design and construction team believed that building the new substructure behind the existing bridge abutments, then sliding the superstructures into place would be the most cost-effective and least disruptive approach. The construction would take place in three phases and maintain three lanes of I-15 traffic in each direction by using the southbound bridge as a "shoofly" (a solution in which an existing bridge is temporarily used as a detour in both directions).

▼ The southbound bridge was used as a shoofly while the northbound bridge was slid into its final position.



▼ The new superstructures were built next to the existing ones.

Wing wall construction, following the slide.

The prestressing resistance anchor block used in the slide.

The project came with challenging construction requirements:

- The existing bridges were three spans, each 56 ft wide. Clearances over Hill Field Road to the bottom of girders were substandard
- Lowering the Hill Field Road profile would require extensive utility (storm and water line) relocations, create new drainage issues and require more roadway reconstruction
- The RFP allowed only two full 12-hour closures of each direction of I-15. Additional full or partial closures would be charged to the contractor at $20,000 per lane per hour
- Hill Field Road was allowed only 12 off-peak, 12-hour full closures, with ramps used to maintain I-15 traffic during that time. Any additional full or partial closures would be charged to the contractor at $10,000 per lane per hour

Sliding Solution

The team explored multiple accelerated bridge construction (ABC) concepts. Sequential phasing of the superstructures was immediately ruled out due to strict maintenance of traffic (MOT) requirements and closure constraints. Self-propelled modular transporters (SPMTs) were also examined but ended up not being cost-effective or convenient when considering the grading that would need to occur to accommodate the new interchange configuration. However, as the new structures were adjacent and similar in geometry, a third ABC concept—bridge sliding—proved to be a winner and was selected based on estimated completion time, feasibility and cost.

Temporary pile-founded abutments would be driven adjacent to either side of the existing structures and connected to future permanent pile-founded abutments, which were to be built once the existing structure was removed. This long continuous section of temporary and permanent abutments would be the level surface that would support the superstructure during the slide. The wider typical section of the new bridges allowed for part of the permanent abutments to be used in the temporary construction location. The southbound superstructure (in the temporary location) would carry I-15 traffic in a shoofly condition while the existing structures could be removed.

Once the shoofly was in place, the existing bridges could be removed, the northbound bridge would be slid into place and all traffic would move to the northbound structure. Finally, the southbound bridge would be slid into place, approaches would be completed and the remaining civil work could occur. The temporary abutments would then be removed prior to substantial completion.

Slide Shoes

To move the superstructure, the team employed two, 13-ft-long concrete blocks with polished stainless steel surfaces at the bottom of each end diaphragm. These shoes would slide on Teflon bearing pads, with the superstructure pulled by prestressing jacks at each abutment line. The Teflon pads had a lubricated coefficient of friction of about 5% during the slide and were replaced with permanent bearing pads in the final location.

The team initially considered a prestressed concrete solution, but this would have required a two- span bridge and a bent that would be overly expensive to build. This approach would also have increased user cost penalties thanks to the additional lane closures needed to construct a pile cap, columns and bent cap. It would also increase the overall length of the structure to accommodate the SPUI configuration. Overall, an equivalent concrete option for this project would have weighed 437.5 tons more and required a structure that was 3 ft deeper.

All of these considerations led the team to select a single-span steel superstructure. Steel allowed for a shallow girder section that closely matched the existing structure depth. The I-15 northbound and southbound profiles matched the existing profiles at the bridges, which minimized expensive interstate roadway reconstruction. The Hill Field Road profile was lowered only 2 ft; this was 2 ft less than what the RFP concept plans called for and required fewer utility relocations. Plus, the lightweight single-span steel solution was simply easier and quicker to slide. A single span meant one less temporary support, thus reducing traffic interference when shifting lanes under the new and existing structures.

Due to vertical clearance requirements, the northbound bridge was built 2 ft higher than the final condition. This provided temporary clearances that would ensure the new steel girders would not be damaged by over-height vehicle traffic below the bridge prior to the bridge slides. Once again, the single-span solution proved beneficial, as it simplified lowering the structures.

Fast Start

The schedule dictated that the project needed to be substantially completed by August 1, 2016, and the design and construction schedules had to be coordinated to allow lead time when ordering materials, particularly the steel girders. The design team provided an early steel package one month after the notice to proceed (NTP) to allow for the 12-week fabrication time. Complete bridge plans were released for construction by mid-July 2015, just three months after the NTP.

Pile driving for the temporary abutments began immediately. The end diaphragms with slide shoes were formed on the temporary abutments, girders were set on these end diaphragms and both superstructures were complete by September. As the southbound bridge functioned as a shoofly, northbound and southbound lane capacity was maintained, avoiding work through the winter.

On March 9, 2016, one year after design began, the I-15 northbound bridge was scheduled to be moved into place. Preparations began several weeks earlier with the removal of the existing structures and placement of permanent piling and pile caps. Several days prior to the slide, prestressed cable was threaded through the end diaphragm blockouts.

The day of the slide, the northbound structure was lowered 2 ft onto sliding pads and Hill Field Road was closed to traffic. The horizontal movements began at midnight and the 1,600-ton bridge—both bridge spans are 178.5 ft long—was slid 74 ft in five hours. (Also, these girders were fabricated and erected at full length, thus avoiding the need for field-bolted splices.) Following the slide, earthwork and approach wingwalls were constructed.

On May 1, the second bridge slide was completed, following the same principles. The team switched to hardwood, instead of the more compressible plywood, to support the Teflon slide bearings. They also tensioned the prestressing strands more uniformly for smoother jack advancement. These improvements, along with the crew's familiarity with the system, resulted in a slightly faster completion time for the second slide. Wing walls, approach slab construction, backfill and civil work associated with the second bridge were delayed due to a wet spring season. Despite the complications, the project opened to traffic by the end of August.

Thanks to the design-build process and the ABC component of bridge sliding, the team was able to work together to optimize construction. And for this project (and many others) steel played a critical role in the ability to design longer spans and minimize reconstruction. ABC, design-build and structural steel were the perfect match for this prominent Interstate overpass. ■

Owner
Utah Department of Transportation

General Contractor
Ames Construction,
West Valley City, Utah

Structural Engineers
WSP | Parsons Brinckerhoff, Murray, Utah
Klophaus and Associates, Salt Lake City

Steel Fabricator and Erector
Utah Pacific Bridge and Steel Corp.,
Lindon, Utah

A new accelerated bridge construction solution makes its commercial debut in a statewide bridge replacement project in Pennsylvania.

A NEW TAKE on Plate Girders

BY TOM STOCKHAUSEN

Tom Stockhausen (thomas.stockhausen@cdrmaguire.com) is president of CDR Bridge Systems, LLC.

THE PENNSYLVANIA Rapid Bridge Replacement Project (PRBRP) is ambitious in scope, to say the least.

The project involves replacing 558 structurally deficient bridges over a three-year span, and the majority of the bridges in the project are short spans proposed to be replaced with concrete structures. Approximately 10% of the bridges restricted the general contractor (Walsh/Granite) to a five-week maximum detour, thereby necessitating accelerated bridge construction (ABC).

As an alternative to concrete, general contractor Walsh/Granite turned to the folded steel plate girder (FSPG) system for multiple restricted-detour bridge projects under the PRBRP. FSPG bridges were first constructed as demonstration projects in Massachusetts and Nebraska using Accelerated Innovation Deployment Grants from the Federal Highway Administration (FHWA). When compared to concrete, they can be erected faster, last longer, require roughly the same level of

◄ Installing the first section.

▼ The folded steel plate girders are fabricated from a single steel plate of uniform thickness that is bent along multiple lines using a hydraulic metal press brake, forming a trapezoidal-shaped section that is open at the bottom

maintenance and compete in terms of cost. As of publication, four of the seven bridges ordered by Walsh/Granite have been manufactured and two have been erected. The three others are currently being manufactured.

Concurrent Construction

Conventional construction of a typical precast 60-ft bridge was estimated by Walsh/Granite to take about 11 weeks following design and manufacturing of the bridge beams. In many rural areas where detours are quite lengthy, the 11-week detour placed a severe impact on local residents, in particular emergency vehicles and school buses. However, the FSPG solution moved much of the construction process off-site, where it was performed concurrently with demolition of the old bridge and construction of new abutments—which reduced the typical detour time frame by more than 50% as well as accelerated design and manufacturing. An added benefit was the improved quality of the deck concrete because it was constructed in a controlled environment.

The ability to standardize the FSPG process is part of what significantly reduced the design and manufacturing time. With 11 standard sizes, design basically involves selecting the appropriate size girder for the span length and opening, then detailing the horizontal and vertical geometry of the specified deck. This allowed superstructure design for the Pennsylvania projects to be accomplished in a matter of days.

CDR Bridge, which designs and manufactures the FSPG system, was also able to reduce total production time for the Pennsylvania bridges to as little as 14 weeks, including steel procurement. Manufacturing took place in three steps: forming

Installing the third section.

> **Folded Steel Plate Girders**
>
> What is a folded steel plate girder (FSPG)?
>
> Developed at the University of Nebraska-Lincoln, the girders are fabricated from a single steel plate of uniform thickness that is bent along multiple lines using a hydraulic metal press brake, forming a trapezoidal-shaped section that is open at the bottom. The plate thickness of either ⅜ in. or ½ in. can accommodate all span lengths by simply changing the location of the bends. Only the width of the top and bottom flanges and the depth of the web vary depending on span length. However, the maximum span length for FSPG bridges is currently limited to 60 ft.
>
> The FSPG system eliminates the need for internal and external cross frames due to the large amount of lateral stiffness generated by the design. The absence of cross frames in the system results in less costly details, and the need for welding is significantly reduced. The open bottom geometry of the girders simplifies inspection, and the hot-dip galvanization process is used for corrosion protection.
>
> Two companies—CDR Bridge Systems and HBS—have been granted exclusive distribution rights, and the steel is fabricated by approved fabricators. (You can view standard drawings at **www.cdrbridges.com**.)

the steel, galvanizing it and then precasting the deck panels. For this project, the girders were formed by cold bending ½-in. plate steel. Shear studs, sole plates and bearing stiffeners were welded and flange separators were bolted to complete the first step. The steel fabrication process took less than three weeks for four girders of a typical two-lane bridge.

The second step in the process was applying corrosion protection via hot-dip galvanizing, which took only a few days to complete. The galvanizing has a guarantee of 25 years and reduces maintenance over that time frame to a level equivalent to or better than that of concrete. Removable, galvanized bird screens were added to the open bottom of the FSPG girders and allow easy inspection of the girders while still keeping birds and other critters from nesting on the bottom flanges.

The final step in the manufacturing process was precasting the deck panels to form the composite system. Precasting the panels (for the four folded steel plate girders) took about three weeks. Once the precast decks were completed, the prefabricated units making up the superstructure were ready for shipment to the construction site for erection.

The first of the Pennsylvania FSPG bridges—a two-lane, 50-ft bridge near Bradford, Pa.—was erected this past October in less than three hours. Closure pours (to complete construction of the superstructure) took only a few days, which enabled the bridge to be reopened in 30 calendar days, five days ahead of the accelerated, five-week schedule for the project detour. This is a full month faster than it would have taken using conventional cast construction. The second Pennsylvania FSPG bridge was erected near Lewiston, Pa., just two weeks later—and took only 2½ hours.

Enhanced Accelerated Construction

These first few FSPG units have provided our steel fabrication and precast concrete partners with valuable experience that has led to improvements in their processes and further reductions in manufacturing time for future bridges. It also reinforced the need for communication throughout the process. The team maintained constant communication with the contractor, so that everyone knew precisely where each bridge was in the design and production process. We were able to adjust schedules through the supply chain without interrupting our other team members' business. The responsiveness of the team resulted in the completion of production in advance of scheduled erection for every bridge despite changing erection dates.

In addition, the Walsh/Granite design-build team proactively managed the schedule, thanks to such practices as a weekly discussion of detailed preproduction tasks between the teams. CDR shared changes with its team and coordinated adjustments to its production priorities to meet Walsh/Granite's changing schedule. That proactive coordination enabled the team to meet every erection schedule—especially important on an ongoing project involving multiple bridges spread over a large area.

Another benefit of this proactive approach was that each member of the team brought ideas and energy to the table that improved the process and the system. The engineers aggressively responded to design changes to substructure and roadway design, shop drawings and RFIs. The fabricator procured extra material so that it was prepared for any changes during fabrication. And the design and precast teams worked together to solve an issue with lifting the exterior girders, adding inserts into the barriers to balance the lift and keep the exterior units from rolling during erection.

The entire team worked together to create a better, faster and more constructable product. It also proved that ABC is not just about accelerating the schedule at the job site but rather accelerating the *entire* process of design, manufacturing and construction. ■

Owner
Plenary Walsh Keystone Partners

General Contractor
Walsh/Granite JV

Architect and Engineer
HDR, Inc.

Superstructure Engineer/Supplier
CDR Bridge Systems, LLC, Pittsburgh

Proper use and maintenance of hard hats on the job site.

Topped OUT, Part Two

BY DONALD GARVEY

WHILE CHOOSING THE APPROPRIATE EQUIPMENT for a specific job is certainly important, maintaining that equipment so that it performs its job properly is equally important.

In the previous article, we discussed features to look for in selecting an appropriate hard hat for the job site (see "Topped Out" in the October 2016 issue, available at **www.modernsteel.com**). This time around, we'll discuss some of the things employers and workers should do to help keep the hard hat functional and serviceable.

Don't Mix and Match

Hard hats are designed as a system; the shell and suspension work together to provide protection to the wearer. Therefore, hard hats should always be maintained using the manufacturer's original parts. Parts from different manufacturers should not be mixed (e.g., the suspension from one manufacturer and the shell from another) as this could create a situation where the protection offered by the hard hat is compromised.

In addition, manufacturers are required to test their hard hats with any attachments they offer (e.g., ear muffs or a face shield) to ensure the hat still meets ANSI Z89.1: *American National Standard for Industrial Head Protection* design requirements. However, there is no requirement to test with other manufacturers' attachments—yet another reason to avoid mixing and matching.

When it comes to accessories (like scarfs or cold-weather liners) that are typically worn under hard hats, keep in mind that they may compromise the functionality of the hard hat and in some cases may create a violation of U.S. OSHA regulations. Consult the April 17, 2006 OSHA Letter of Interpretation (available **www.osha.gov**) for a further discussion of this issue.

As noted, the shell and suspension are designed to work together. The clearance between the shell and the top of the suspension allows for the hard hat to compress during impact and absorb some of the shock without transferring it to the wearer's head or neck. This clearance space must not be used as a handy storage space. Storing items like gloves or disposable respirators in this space can reduce the shock absorption capacity and increase risk to the wearer.

Sticker Shock

Many companies use hard hat stickers as motivational tools or to identify that a person has completed training or is authorized to work in a specific area. Typically, pressure-sensitive sticker adhesives will not create any problems for the hard hat material. However, stickers should not be allowed to cover up significant portions of the hard hat surface as this may hide cracks or other signs of deterioration. Stickers shouldn't be used as a method for repairing areas of shell damage. They should be placed away from the edge of the hard hat (at least ½ in.) and not wrapped around to the inside. This is to prevent the sticker from possibly transmitting an electrical current to the inside of the shell.

In addition, hard hats should *not* be:

- Used as seating or a step. The compression may damage the shell
- Cleaned with solvents or abrasive cleaners. Follow the recommendations of the hard hat manufacturer. A pH-neutral soap and warm water (120 °F) will usually get the job done
- Painted, drilled or otherwise customized
- Stored in areas subject to environmental extremes or repeated, prolonged exposure to sunlight. While it's true that work itself often takes place in prolonged sunny conditions (see maintenance suggestions below), storing a hard hat in such conditions just accelerates degradation

Donald Garvey (**djgarvey@mmm.com**) provides construction technical service in 3M's Personal Safety Division.

Maintenance

Like all personal protective equipment (PPE), hard hats need to be inspected on a regular basis and repaired or replaced when parts become damaged or worn. Hard hats that are subject to forceful impact—from something falling on them or a hard hat itself falling—should be immediately taken out of service and replaced regardless of visual appearance—i.e., it's best to take a "one and done" approach. The inconvenience and expense of replacing a hard hat are nothing compared to the potential risk of injury due to a damaged or less-than-optimal hard hat. For hard hats currently in service, some signs to look for during routine inspection include:

- **Shells.** Cracks, dents, deep abrasions, brittleness, chalky appearance, penetrations or fading of color. These may indicate damage to the hard hat or the start of plastic degradation. Hard hats exposed to harsh environments such as airborne chemical exposure, extreme temperatures (hot or cold) or repeated, prolonged exposure to sunlight (UV) should be checked on a more frequent basis, as these conditions can accelerate degradation. While there is no regulatory requirement for when to replace the shell, most manufacturers recommend routine replacement within five years depending on use conditions. This may be based on service life (when the hard hat was actually put into service) or the manufacture date. Check the manufacturer's user instructions for guidance
- **Suspensions.** Frayed, torn or damaged crown straps. The keys (the tabs on the crown strap that fit into slots on the hard hat) should be intact and firmly locked into place. The headband should be flexible and the size adjustment mechanism should be able to maintain the selected size. At least one manufacturer recommends replacing the entire suspension system annually. Again, the user instructions provide the manufacturer's specific recommendations

As with all PPE, it is critical to follow the manufacturer's user instructions, which will provide specific details on assembly, maintenance, inspection and precautions to take with the product. Doing so will help maximize the service life and protection offered by the hard hat and ensure that it's doing its job to keep the wearer safe. ■

For further reading on PPE, see:

- Occupational Safety and Health Administration (OSHA). Personal Protective Equipment Publication 3151-12R 2003
- Occupational Safety and Health Administration (OSHA). Letter of Interpretation: 29 CFR 1926.31 and 1926.100; wearing caps or other apparel under a hard hat for cold weather protection
- Occupational Safety and Health Administration (OSHA) 29 CFR 1926.100 Head Protection

People and Firms

Carter Lake, Iowa-based **Owen Industries** (an AISC member) is now an official registered apprenticeship sponsor and offers a three-year, competency-based welder apprenticeship program. Late last year, the company began an interview and selection process, seeking four welder apprentices to offer on-the-job training as well as classroom training, in partnership with Iowa Western Community College, at no cost to the apprentice.

Tyler Owen, president of Owen Industries, commented, "We are very excited to offer an ongoing welding apprenticeship program to individuals interested in establishing their career path." After successful completion of Owen's program, welder apprentices will receive a nationally recognized Journeyman apprenticeship certificate from the United States Department of Labor (DOL) Office of Apprenticeship (OA).

Owen Industries offers full-time employment to their apprentices, thus an opportunity to earn a good wage while learning a new skill. "Our goal is to ensure that this new program provides high-quality training and produces skilled, competent workers who may not otherwise have an entry-level opportunity in a skilled trade of their interest," said Owen. "Finding and training dedicated workers is a win-win for our community and necessary for growing both our business and our local economy."

Owen Industries' welder apprenticeship program is ongoing and open to anyone who applies at the company's Carter Lake location, regardless of where they live. For questions or more information about the program, please contact **Ronald D. DeBord**, Owen Industries' vice president of human resources, at 712.347.5500 or **rdebord@owenind.com**.

BRIDGES

Updated I-Girder Bridge Resource Available

The National Steel Bridge Alliance (NSBA) has released an updated version of its *Skewed and Curved Steel I-Girder Bridge Fit* document at **www.steelbridges.org/bridgefit**. This free resource offers additional guidance on proper detailing of bridges to ensure constructability.

"Fit" or "fit condition" refers to the deflected girder shape under specific loading conditions. This is important to steel bridge fabricators because it determines how the cross frames and diaphragms are detailed to fit to the girders. In addition, fit affects bridge constructability by determining how much force is necessary to align the girders and cross-frame elevations.

The new document offers additional and expanded insights into how bridges should be detailed. Updates include minor changes to recommended fit conditions for horizontally curved bridges based on the recently finalized *Guidelines for Reliable Fit-Up of Steel I-Girder Bridges*.

IN MEMORIAM

Vincent J. DeSimone, Founder of DeSimone Consulting Engineers, Dies

Vincent J. DeSimone, PE, founder and chairman of New York–based DeSimone Consulting Engineers, passed away in November following a battle with cancer. He was 78 years old.

DeSimone established DeSimone Consulting Engineers in 1969 and has led the firm as senior principal-in-charge of design ever since. The firm has designed a vast range of projects over the decades, including the Fisher Center for the Performing Arts at Bard College in Annandale-on-Hudson, N.Y., and the Cosmopolitan Resort and Casino in Las Vegas (see the articles "Setting the Stage" and "Filling the Gap" at **www.modernsteel.com**).

Following DeSimone's death, his firm stated, "True to his life's calling, Vincent continued to work every day, despite his illness, to advance his engineering vision."

DeSimone is survived by his fiancée, Stefany Koo, two children and 15 grandchildren.

BRIDGES

Press-Brake Tub Girder System Introduced for Short-Span Bridges

The press-brake tub girder (PBTG) system has evolved from a research project to a viable solution for short-span bridges. The new technology—developed by the Short Span Steel Bridge Alliance (SSSBA) in conjunction with West Virginia University and Marshall University—was recently used to build the Amish Sawmill Bridge in Buchanan County, Iowa. In collaboration with members of SSSBA, the Iowa Department of Transportation and the Buchanan County Engineer worked to construct the bridge with funding through the Innovative Bridge Research and Deployment Program (IBRD).

According to SSSBA, the PBTG system was designed specifically for the short-span bridge market and offers a number of advantages over traditional systems used in short-span bridge construction. A standard plate size is folded into a trapezoidal shape using a press brake, similar to a larger steel tub girder; using standard plate sizes ensures that material is readily available and pricing is economical. Unlike the larger steel tub girder, the press-brake tub girder does not require welding, so fabrication time is significantly reduced. One girder can be produced in as little as 45 minutes, according to SSSBA. And because of the girder's lighter weight, a precast deck can be placed on the girder and shipped to the job site, a significant advantage in accelerated bridge construction (ABC). The weight savings also allows for installation with smaller cranes or by county engineering crews. (Through conversations with county engineers, the steel industry has determined that county engineers can better control costs and find savings by using their own crews for construction.)

The Federal Highway Administration recently awarded the Ohio Department of Transportation and Muskingum County, Ohio, an Accelerated Innovation Deployment (AID) grant for the installation of a PBTG system with a sandwich plate deck system. West Virginia is also evaluating two locations for the system.

You can find more details on the PBTG system at SSSBA's website, **www.shortspansteelbridges.org**.

NASCC

2017 NASCC Registration Now Open

Registration for the 2017 NASCC: The Steel Conference is now open.

The Steel Conference is your once-a-year opportunity to engage with more than 4,00 of your peers involved in structural steel design and construction and learn the latest design concepts, construction techniques and cutting-edge research for steel buildings and bridges. The 2017 conference takes place March 22-24 in San Antonio at the Henry B. Gonzalez Convention Center and will offer more than 130 technical sessions and feature more than 220 exhibitors showcasing the latest equipment, software and tools.

Attendees can earn up to 17 PDHs by attending the conference's dynamic, expert-led sessions (plus an additional 4 PDHs if they attend the optional pre-conference short course). Topics will range from "Practical Advice for Reviewing Software Generated Connection Designs" to "Lateral Torsional Buckling and its Influence on the Strength of Beams" to "Keeping OSHA out of your Bank Account." In addition, AISC has created a new program to explore "Solutions for Equity in the Workforce." This special 2½-hour session is being held on the opening day of the conference and offers a look at what diversity means for the design community and construction industry, with an emphasis on what works and what doesn't when creating solutions that can increase equity within the workplace.

You won't want to miss the keynote address, " The Neuroscience of Decision Making," presented by Carmen Simon, PhD. Whether it's an email, brochure or presentation, a well-crafted message needs to be informative, reusable and adaptable. But according to Simon, it also needs to be memorable. Audiences typically only remember 10% of your presentation—and which 10% they remember varies from person to person. Using lessons learned from neuroscience, this session provides a unique approach to controlling what audiences remember about your message.

One registration fee gains you access to all of the technical sessions, the keynote address, the T.R. Higgins Lecture and the exhibition hall. Visit **www.aisc.org/nascc** to register or view more conference information, including the advance program. But be sure to register soon, as the price increases weekly between now and the conference.

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RECRUITER IN STRUCTURAL MISCELLANEOUS STEEL FABRICATION

AISC ASSOCIATE

ProCounsel, a member of AISC, can market your skills and achievements (without identifying you) to any city or state in the United States. We communicate with over 3,000 steel fabricators nationwide. The employer pays the employment fee and the interviewing and relocation expenses. If you've been thinking of making a change, now is the time to do it. Our target, for you, is the right job, in the right location, at the right money.

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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, Ficep Minosse CNC, 10' x 20' x 5" Max. Plate, 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 FPB500-3C CNC Plate Punch with Plasma Torch, 177 Ton, 20.8" x 40' Plate, Triple Gag Punch, Hypertherm, Fagor 8035 CNC **#25885**
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 AFCPS 823/B CNC Anglemaster Angle Punch & Shear Line, 8" x 8" x 3/4", 130 Ton Punch, 400 Ton Shear, Marking Press, 1998 **#26594**

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Basden Steel Corporation

An ENR Top 600 Specialty Contractor and AISC Certified Fabrication and Erection company is actively recruiting for Estimating / Project Management positions for its locations in Tulsa, Fort Worth and Houston. Basden Steel offers a competitive compensation package, positive work environment and growth potential. Interested? Inquire at: **careers@basdensteel.com**
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Lincoln Engineering Group is one of the fastest growing steel detailing firms in the country located in Chicago suburbs. We currently have immediate openings for experienced detailers, checkers, and Project Managers.

Ideal candidate would have experience in Structural and Miscellaneous steel detailing and checking. He/she should be a team leader with excellent communication skills. We offer a competitive compensation and benefits package. May consider relocation allowance for the right candidate.

Please submit your Résumé to: **jobs@lincolnengineering.com** or Contact Terri Peters at (630) 445-2111 structurally sound

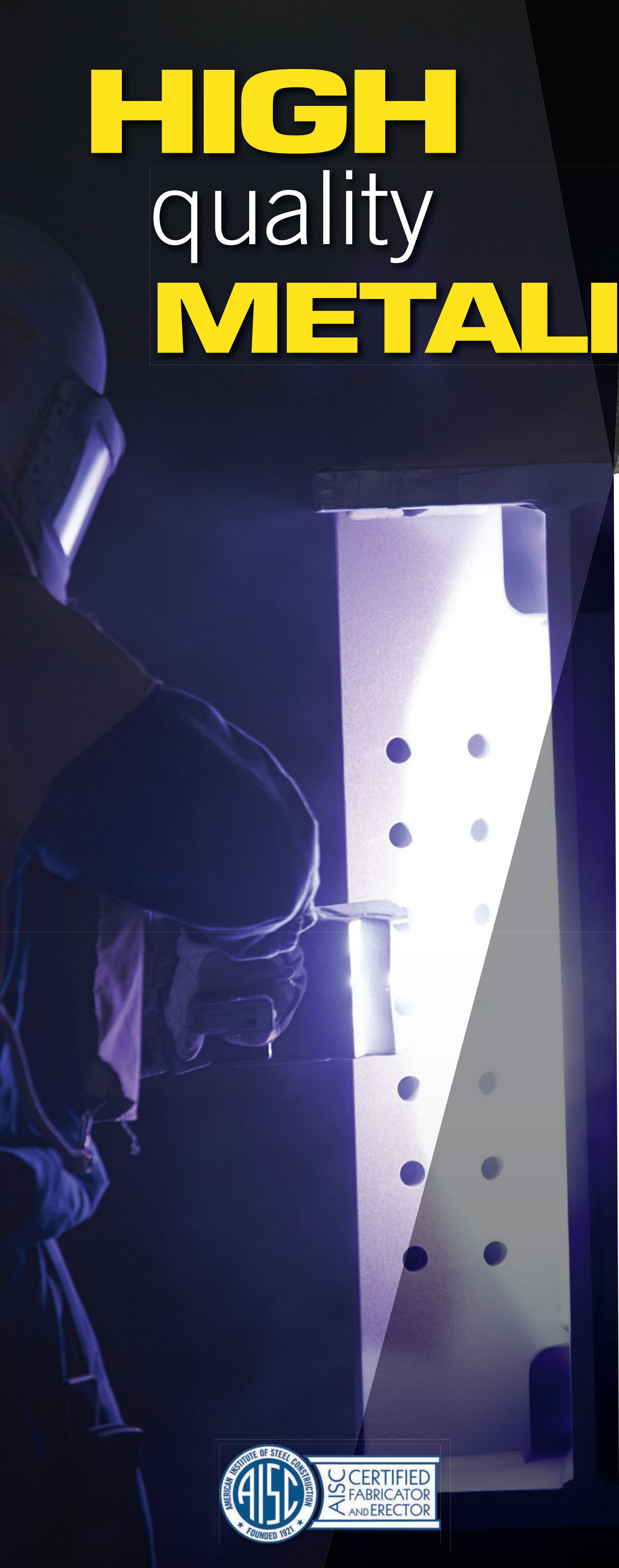
SCULPTURALLY SOUND

AS PART OF 2016'S STEELDAY, students entered photos of their campus steel sculpture, which were posted to AISC's Facebook page and voted on by fans during the week of SteelDay. The winners are: (top photo) Lauren Santullo from The College of New Jersey; (bottom row, left to right) CJ Powell from Rochester Institute of Technology; Christie Moore and Reggie Raney from Christian Brothers University (the two submitted nearly identical photos); and Eric Bellville from the University of Central Florida.

The original AISC Steel Sculpture was created by Duane Ellifritt, PE, PhD, professor emeritus of civil engineering at the University of Florida, and this past October marked its 30th year of existence. There are now more than 170 versions of the teaching sculpture on campuses around the country and the world (see **www.aisc.org/steelsculpture** for more information and a list of locations). You can learn more about Ellifritt in "Creating Art in Unlikely Places" in the October 2011 issue (available at **www.modernsteel.com**).

And be sure to mark your calendars for this year's SteelDay, which is scheduled to take place September 15 (for more on SteelDay, visit **www.steelday.org**).

HIGH quality METALIZING.



Among the industry's most valuable tools against corrosion, metalizing provides a protective coating by means of a thermal spray of molten zinc or zinc/aluminum alloy. Incorporating metalized structural steel into your project can increase the structure's lifespan while reducing long-term maintenance costs.

High Steel Structures is one of a select few fabricators who have made the investment to provide metalized structural steel directly from the fabrication facility. To meet increasing industry demand, High Steel recently added two dedicated chambers to its Williamsport, PA facility with state of the art equipment for blasting and metalizing steel components.

We offer our full range of steel coatings as individual services or as part of a turnkey package including fabrication and shipping.

CONTACT US TO DISCUSS YOUR PROJECT

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