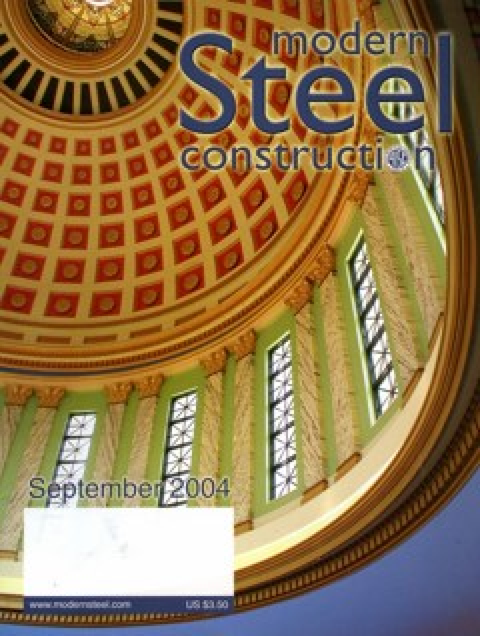
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notes from the editor

Scott L. Melnick

Not too long ago, my oldest brother's youngest daughter came to me with a question about something I had written more than a dozen years earlier. Of all my sibling's progeny, Beth Ann is probably the most similar to me (though smarter, more athletic, and much more personable), so, unlike her parents, she appreciated my attempt at philosophical humor: Essentially, on her first birthday, I gave her a card with a long diatribe questioning why anyone would write a long message to someone too young to read or comprehend language.

But words have power—though all too often possess more than a modicum of ambiguity.

And the power of words was much in evidence during a recent discussion of the green attributes of steel.

While everyone can agree on the meaning of "recycled content" (which, for wide-flange beams and columns, incidentally, is at least 90%) there is another term out there—"recycling rate"—the meaning of which is less clear. In fact, as our recent discussion revealed, the government's definition is radically different than what I had assumed the phrase to mean: the amount of steel which was diverted from the waste stream into recycled products (and for that matter, my definition—which I thought was the obvious layman's definition—didn't agree with what my wife's best friend Sandee assumed the term meant). So given the extensive confusion over the term, and so as not to get into an argument with the Environmental Protection Agency, at *MSC* we've decided to use the more understandable term "reclamation rate".

The reclamation rate, as we define it, is the amount of steel diverted from the waste stream and made into recycled products such as beams and columns (in other words, what I had incorrectly interpreted "recycling rate" to mean). Overall, the reclamation rate for steel is about 88%—but it's even higher for structural members and plate. According to the Steel Recycling Institute (**www.recycle-steel.org**) the reclamation rate for structural beams and plates is 95%. (Interestingly, the reclamation rate for rebar is only about 50%, presumably due to the difficulty in separating it from its embedment in concrete.)

What's fascinating to me is the combination of high recycled content with high reclamation rate. While the LEED™ rating system doesn't consider reclamation rates, I believe anyone concerned with the future—and as a parent how can I not be?—must take into consideration what becomes of a material at the end of its useful life.

A Fond Farewell; A Warm Welcome

After two years with *MSC*, Beth Pollak is leaving to participate in a 10-month-long volunteer program in Israel. During her too-brief tenure, Beth has been a great contributor and has written a number of notable articles (my favorite was her coverage of the Student Steel Bridge Competition; see this year's coverage in this issue).

Beth's successor is Hillary Lichtenstein. Hillary is a graduate of DePaul University in Chicago (and of Niles North High School—coincidentally Beth's alma mater) and comes to *MSC* after a brief stint at a small consumer magazine (*Illinois Now!*). In addition to writing and editing, Hillary will have primary responsibility for article acquisition and for the new products section. She can be reached at 312.670.8316 or at **lichtenstein@modernsteel.com**. She'll also be attending various industry trade shows (including NASCC: The Steel Conference)—so be sure to say hello when you see her!

Scott Melnick

Editorial Offices
Modern Steel Construction
One E. Wacker Dr., Suite 3100
Chicago, IL 60601
312.670.2400 tel
312.670.5403 fax

Editorial Contacts

Editor & Publisher
Scott L. Melnick
312.670.8314
melnick@modernsteel.com

Managing Editor
Keith A. Grubb, P.E., S.E.
312.670.8318
grubb@modernsteel.com

Assistant Editor
Hillary A. Lichtenstein
312.670.8316
lichtenstein@modernsteel.com

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Advertising Contact

Account Manager
John A. Byrne
847.699.6049 tel
847.699.8681 fax
byrne@modernsteel.com

2400 E. Devon Ave., Suite 380
Des Plaines, IL 60018

To receive advertising information, contact John Byrne or view it online:
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312.670.5444 tel
312.670.5403 fax
admin@modernsteel.com

Modern Steel Construction's monthly *Steel Quiz* allows you to test your knowledge of steel design and construction. All references to LRFD specifications pertain to the 1999 *LRFD Specification for Structural Steel Buildings*, available as a free download from AISC's web site:

www.aisc.org/lrfdspec

ASD references pertain to the 1989 *ASD Specification for Structural Steel Buildings*. Where appropriate, other industry standards are also referenced.

Anyone is welcome to submit questions for *Steel Quiz*—one question or 10! If you or your firm are interested in submitting a *Steel Quiz* question or column, contact ▶

Steel Solutions Center
One E. Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.670.9032
solutions@aisc.org

This month's quiz was developed by AISC's Engineering and Research department. Sharpen your pencils and go!

1. What does the quantity $1.1R_yF_yA_g$ represent in the design of a Special Concentrically Braced Frame (SCBF) per the AISC *Seismic Provisions for Structural Steel Buildings*?

2. How much of a bolt must project beyond the nut when properly installed?

3. What flange thickness is required for welding shear stud connectors in a composite beam?

4. What is the purpose of doing CVN (Charpy V-notch) tests on members and plates?

5. Why is CVN testing only required when thickness exceeds 2 in. (1.5 in. in some case in the AISC *Seismic Provisions*)?

6. What notch toughness requirements are appropriate for exterior exposed steel?

7. AWS D1.1 specifies dimensions for weld access holes. What is the origin of these dimensions?

8. When TC bolts are used, does this automatically require that the joint be made slip-critical?

9. When can a conventional shear tab be used in lieu of a through-plate connection to the face of an HSS column?

10. Which is the proper specification of the material: "ASTM A992 grade 50" or "ASTM A992"?

Turn page for answers

Answers

1. This is the design force corresponding to tension yielding of the brace, including the effects of material overstrength (R_y) and strain hardening (1.1). In a special concentrically braced frame (SCBF), the goal is to have a system that can undergo significant controlled inelastic deformations through tension yielding and compression buckling of the bracing elements. The quantity $1.1R_yF_yA_g$ provides the means of determining the axial force in the brace that the rest of the system will have to withstand.

2. *RCSC Specification* Section 2.3.2 states that "The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed." So as long as the point of the bolt is flush with or projects beyond the face of the nut, the bolt length is acceptable.

3. From *LRFD Specification* Section I5.6, the shear stud diameter cannot exceed 2.5 times the flange thickness, unless the stud is over the web. However, the best way to apply this limit is to establish a minimum flange thickness for selection of beam sizes, which means that flange thickness must be greater than 0.4 times the stud diameter when studs are not centered over the beam web.

4. When thick plates and heavy shapes are used in applications loaded in tension, the core area has to be notch tough to ensure brittle fracture will not occur.

5. Essentially, plates thinner or shapes lighter than the threshold value will have toughness without a special requirement. Above the threshold, it may not be a guarantee, hence the requirement. Some additional conservatism is taken in the AISC *Seismic Provisions* due to the more extensive demands predicted in those applications.

6. In cold-weather applications, notch toughness is the primary means of ensuring that the steel will perform properly. If the steel is exposed, this may be necessary and the specifier should consult ASTM A709 (Tables S1.1, S1.2 and S1.3).

7. Weld access holes serve multiple functions. As one function, they permit the access needed to continue welds past the web. As another, they provide a transition that accommodates shrinkage strains from weld cooling. The weld access hole extends to a location where the end of the hole is in compression and therefore cannot crack. The dimensions indicated in AWS D1.1 satisfy this distance requirement.

8. No. TC bolts can be used in snug-tightened, pretensioned and slip-critical joints. In a snug-tightened joint, we don't care how high the level of pretension is. The use of this product here is merely for the convenience or preference of the installer. In pretensioned or slip-critical joints, TC bolts can be used as the installation method to achieve the proper pretension in the bolt.

9. There are two criteria. The first is a wall slenderness check — $b/t \leq 1.40(E/F_y)^{0.5}$ — that is required for the second check to be valid. The second check is a punching shear calculation to ensure that the plate will not slice through the wall. This is covered in detail in the AISC *Hollow Structural Sections Connections Manual*.

10. Because ASTM A992 is a 50 ksi material only, it is properly specified as "ASTM A992". ★

10 Tips for Designing with Steel Joists

by Timothy J. Holtermann, P.E.

You don't have to know how to dance in order to sidestep some of the problems that can surface when designing with steel joists and joist girders. Here are a few ways to avoid common missteps—and make your project (and your joists) more efficient.

1 **Situation:** Setting the joist seat depth too shallow for sloped ends. The typical K-series joist seat depth of 2½ inches is not adequate once the roof slope is ½:12 or greater. At the high end of the joist, it becomes difficult to place the end web member to deliver the end reaction over the support. As shown in Figure 1, the chord angle may not clear the support and would need to be coped.

Solution: Increase the seat depth anywhere that more than a nominal roof slope is used. The joist manufacturer can readily provide a variety of seat depths other than the typical 2½-inch and 5-inch depths.

2 **Situation:** Including a web configuration in load diagrams or joist profiles, when there are not any specific geometric requirements. Drafters will often "fill-in" a joist profile with random web members just to make it look presentable. The joist manufacturer then loses the option of quoting the most efficient geometry.

Solution: Leave the web area blank or add a note stating that the "web may vary." If there is a specific constraint for the placement of the web members, such as a large duct passing through the joist, show this on the joist profile.

3 **Situation:** Limiting the use of joist girders to only those designations found in the Weight Tables. The purpose of the Weight Tables is to provide the approximate self-weight of the joist girder to be included in the structural design. The Weight Table is not a comprehensive listing of all possibilities.

Solution: Use joist girders for any reasonable combination of depth, length, kip loading, and number of panels. In general, most manufacturers can provide joist girders with chord angles of up to 6×6×¾. Several manufacturers have published more extensive Weight Tables. (Steel Joist Institute is planning an expansion of its current Weight Tables to include limits beyond 72 inches deep, 20 kips per panel point, and 60 feet long.)

4 **Situation:** Showing a weld at a joist bottom-chord extension when one is not required, as shown in Figure 2. A typical joist is designed as simply supported with an underslung end. Welding the bottom chord extensions develop a fixed-end moment, which should not be done unless it is considered in the design of both the joist and the overall frame.

Solution: Unless required, don't call for a weld on a bottom-chord extension. The column stabilizer plate in the gap between the chord angles provides the same resistance to the joist overturning without being welded, as if it were welded. If a weld is required, show the appropriate design requirement—either an end moment or an axial bracing force to the bottom-chord extension.

5 **Situation:** Don't call for welds to joists that are bigger than the typical chord-angle sizes. As shown in Figure 2, a ¼ inch fillet weld is specified from the bottom chord to the column stabilizer plate. For a typical K-series joist, this is thicker than the bottom chord angle sizes that would ordinarily be used.

Solution: Use thinner, longer welds. For smaller K-series joists, assume a maximum angle thickness and weld size of ⅛ inch. For larger K-series, and smaller LH-series joists, assume a maximum angle thickness and weld size of 3/16 inch. Call only for a ¼ inch weld at heavy long-span joists, or joist girders.

6 **Situation:** Making the joist manufacturer responsible for tracking down loading information that is not available yet. Structural contract drawings that refer the joist manufacturer to the mechanical contractor or equipment supplier for the joist design loads can often delay joist fabrication.

Solution: When mechanical information is not available, consider designing a roof top unit "zone" with extra capacity. This is far cheaper than reinforcing joists later on in the field.

7 **Situation:** Using joists that are too short. Short joists that are less than 8-feet long are not practical to fabricate. If the drafter is not given other instructions, a minimum joist size may be used even for very short spans, such as the 8K1 joists in the corner of the framing plan of Figure 3.

Solution: Use joist substitutes. The joist manufacturer can build a joist substitute more efficiently than a very short joist. The joist substitute may consist of two angles welded together to make a channel section. It may be made of tube steel or another shape. The Steel Joist Institute recently published a straight-forward Load Table for three standardized joist substitutes with the designations 2.5K1, 2.5K2, and 2.5K3.

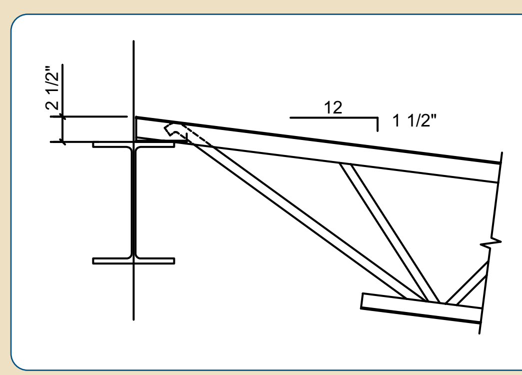


Figure 1.

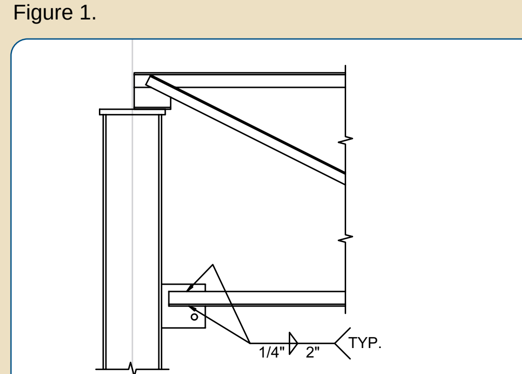


Figure 2.

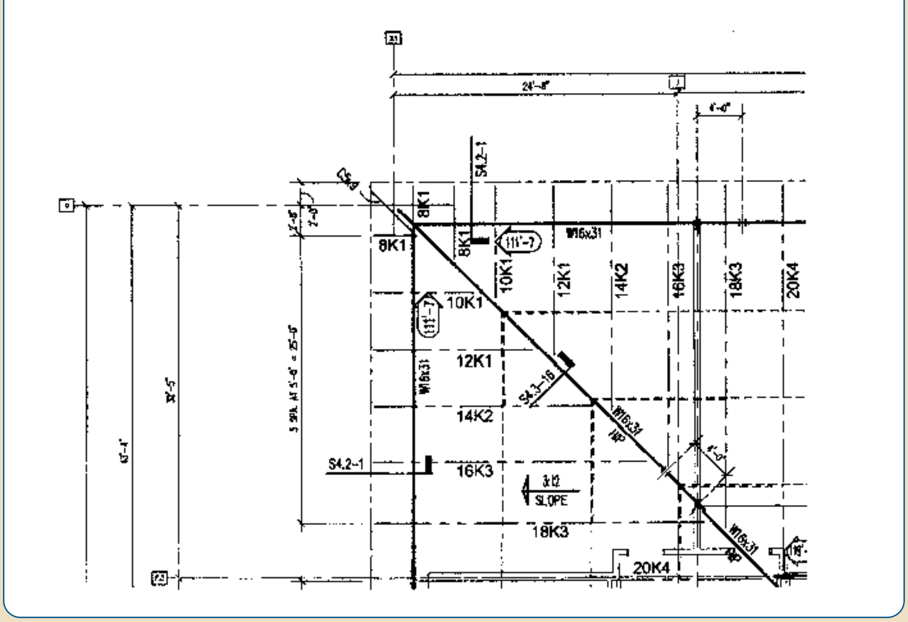


Figure 3.

8 **Situation:** Corner joists or substitutes that are not simply supported at both ends. Notice in Figure 3 that while the typical joist has two supports and a cantilevered top chord extension, the shortest 8K1 joists in the corner are simply cantilevers with no "base" span. This is a problem, whether these joists are designated 8K1 or if they are joist substitutes as discussed above.

Solution: Make sure all the framing members have support at each end. Where necessary, design the edge angle or other edge steel to provide support spanning from the corner to the first non-cantilevered framing member. The size of the edge-framing member, as well as the resultant point loads where that member is supported, should be shown as in Figure 3.

9 **Situation:** "Over-designed" top-chord extensions on small joists. A short 8K1 or 10K1 will have a Load Table capacity of 550 plf total and live load. This capacity may be more than is needed but comes at no great cost for the main span. As with a roof overhang (see Figure 3) or a top chord extension of any length, providing a capacity of 550/550 is difficult for both strength and deflection.

Solution: Use the Steel Joist Institute's Extended End Load Tables and specify a top chord extension type such as "S7" or "R3."

10 **Situation:** Using too many joist depths in a skewed bay. While the economy table might suggest a different joist depth for every span in a skewed bay, the joist manufacturer builds, bundles, and ships by depth.

Solution: Refer to Figure 3. Instead of changing the joist depth several times going into the corner, make only one or two steps down in depth, combining several span lengths in each depth. Then use joist subs for the smallest spans.

For more information about steel joists, to order any of the above mentioned guides or manuals, or to contact an individual manufacturer, please visit the Steel Joist Institute web site at **www.steeljoist.org**. ★

Timothy J. Holtermann, P.E., is engineering manager for Canam Steel Corporation, Washington, MO and chairman of the Steel Joist Institute Engineering Practice Committee.

Bridge **Builders**

The 2004 National Student Steel Bridge Competition

By Beth S. Pollak and Dan Swiatek

A record 44 teams participated in the 2004 National Student Steel Bridge Competition Finals (co-sponsored by AISC and ASCE) at the Colorado School of Mines in Golden, CO.

Members of North Dakota State University's student steel bridge competition team knew what it was like to have a prize-winning model bridge—the team won the ASCE/AISC national competition in 2002. After dropping to 12th place in the 2003 competition, the team was determined to win again at this year's finals, held May 28-29, 2004 at the Colorado School of Mines in Golden, CO. With careful planning and an innovative bridge design, NDSU's captain Mitch Okeson led his teammates to capture their second win in just two years.

"We really enjoy the competition, and we work really hard with it," said Okeson, who is completing his master's degree at NDSU. "The most rewarding part about it was knowing that we had something that was very competitive, and that we saw our ideas work out in a good economical design. For four of the members of the team, it was our last year, and we wanted to give it our best chance. Our team advisor also is leaving the university, and we wanted to win it for him too. We had a lot of motivation to do well and to go out with a bang."

A record 44 student teams participated in National Student Steel Bridge Competition finals, which are co-sponsored by ASCE and the American Institute of Steel Construction (AISC). During the competition, teams race the clock to

Team members from the Illinois Institute of Technology worked swiftly to assemble their bridge.

2004 National Winners

1. North Dakota State University
2. University of Michigan
3. Southern Polytechnic State University

2004 Category Winners

Construction Speed
1. Illinois Institute of Technology
2. Iowa State University
3. University of Wisconsin – Madison

Lightness
1. North Dakota State University
2. University of Michigan
3. University of Wisconsin – Madison

Aesthetics
1. North Dakota State University
2. University of Puerto Rico – Mayagüez
3. University of Wisconsin – Madison

Stiffness
1. North Dakota State University
2. Southern Polytechnic State University
3. University of Michigan

Economy
1. Illinois Institute of Technology
2. Iowa State University
3. University of Wisconsin – Madison

Efficiency
1. North Dakota State University
2. University of Michigan
3. Southern Polytechnic State University

Bridges were load-tested under the watchful eyes of competition judges.

erect their bridges as quickly as possible. Rules specify site conditions, member sizes, weight limitations, design loads, and erection and safety procedures. Penalties are assessed for stepping into the "water," dropping tools or equipment, and other violations. Once constructed, bridges must meet standards for load-bearing capacities and deflection.

"The competition brings together everything students have learned in the classroom," said competition organizer Fromy Rosenberg, AISC's director of university relations. "Students practice basic steel design and fabrication, project scheduling and management, and gain hands-on appreciation for the strength and versatility of structural steel."

This year, new deflection-measuring equipment helped the competition run smoothly. "There were lots of unique designs this year, and the Colorado School of Mines did a great job running the event," said Head National Judge John M. Parucki. "We also had more judges than ever before, and they did an excellent job."

Prizes were awarded for overall performance as well as in six specific categories: lightness, aesthetics, stiffness, economy, efficiency, and construction speed. North Dakota State University took first place overall, followed by the University of Michigan-Ann Arbor and Southern Polytechnic State University.

NDSU's bridge, which also won the aesthetics competition, was composed entirely of square and round steel hollow structural sections (HSS), which Okeson says gave it an elegant look. "It looked like it had a fourth dimension," he said. "It was a continuous joist from end to end, and over the long span, there was a 2'-6" straight truss. In our design analysis, we determined that uplift on the long span would be necessary. So we created a big point load on the end of a cantilever over that continuous span, which helped give uplift deflection on the short span."

NDSU's team of about 15 active members began design work for their bridge at the beginning of the academic year in August 2003. "Everyone reads the rules, and then we sit down and analyze structural efficiency and construction economy to consider what works and what doesn't," Okeson said. "After we determine the basics that we think will give us a score that will be competitive, we move on to the actual structural design. We try all our ideas, and by process of elimination, we come up with what we hope is the best one."

Team members did all of the fabrication for the bridge on their own. "We do all the cutting, all the welding and all the machine shopping. We're self-taught. I grew up on a farm, I learned it from home, and I took welding classes in high school."

Okeson, who plans to work for the Minnesota Department of Transportation when he completes his studies, says this year's win marks the end of an era for him. "It's been a part of my life. The bridge competition has provided a lot of fun times and has been a good experience. I'll miss working in the metal shop, brainstorming in a bar at night, and the camaraderie of a great group of people."

Tough Competition

Enthusiasm and competition was high this year, and teams like New Mexico State practiced the night before the competition in the hotel parking lot. "Steel Bridge gives you a real-world experience," said team member Max Lopez. "You learn that planning is important to get everything done on time."

Bridge designs also were creative: Students from the University of Missouri – Rolla used structural-design software program SAP 2000 to model and create a bridge with a circular curved arch supporting a long span. "We began brainstorming in October, and after considering more than 20 different designs, we began fabricating," said team member Jared Brewd.

"The curved steel arches for their long span had to be sent out to a professional fabricator to bend the steel," said teammate Bryan Madson. "The first time it was sent out, UPS lost one of the 15' arches!"

Many students spent vacations, nights and weekends designing, welding and practicing the bridge assembly, even during exam periods. Team members from Illinois Institute of Technology (IIT), who took first place for speed and economy, constructed their bridge during their spring break. "The bridge was finished just two weeks before the regional competition," said Captain Jorge Cobo. "We didn't know if we would make it to nationals."

At the national competition, the team assembled their bridge in just 1 minute, 48 seconds, but two penalties brought their final time up to 2 minutes, 9 seconds.

Cobo said that practicing to build the bridge quickly was a process of trial and error. "Every time you practice assembling the bridge, you find something new that you can improve. Once you find the best way to do something, you repeat it. We tried to coordinate the construction activities to make sure that all team members were working and that there was no idle time."

Iowa State, whose bridge failed at the 2003 competition, came back strong this year, finishing in eighth place overall, and second place in both economy and speed. The team, clad in gray coveralls, assembled their bridge in just 2.383 minutes. "This year we built a beefier bridge and used a much higher factor of safety," said team member Chris Bauer.

Steel fabrication shops helped many teams fabricate and construct their bridges, and corporate sponsors lent financial support for tools, equipment and travel. Firms also are important connections for the soon-to-be graduating students. "I would hire any of these guys on the spot!" said University of Michigan professor Nikolaos D. Katopodes.

Iowa State's team co-chair Julie Maher said that the team worked hard at its fundraising efforts in addition to its bridge design. "Company sponsors helped provide money and supplies. A local fabricator donated the steel, helped with laser cutting, and custom-bent channel sections that otherwise would not have been available to the team. We also sold pizza every Wednesday to raise extra funds for the team." She said the bridge competition is a great way to put engineering lessons into practice. "[Steel Bridge] really intermingles the students [of different ages] and helps them see the class work and how to apply it."

Penn State also did a good job with fundraising: The team's letter-writing efforts earned about 25 sponsors. With the sponsors' support, as well as supplies and mechanical resources available at the university, the team was able to devote their full attention to the competition. "Work is never over, and there is always something to do to improve the bridge," said team member Nick Remington.

Rutgers University's bridge team started from scratch this year—and their bridge still made it to the national competition. "The entire bridge team for the last few years had been the same age, and they graduated together, leaving no one with experience behind," said Captain Douglas Eichenbluff.

The Rutgers team had not constructed their bridge with its name attached, so competition judges ruled that the team would have to place the name on their bridge and add the additional placement time to the total construction time. The team made a sign, fitted it with double-sided scotch tape, and chose their fastest runner to place it. As his teammates cheered him on, runner Michael Solar sprinted from the staging area to place the sign, adding just 3.87 seconds to Rutgers' total.

Returning Champs

University of Michigan (2003 Champion and this year's second-place winner) tried to incorporate aspects of last year's bridge into their 2004 model. "Some of the connections used were the same," said Captain Cordelle Thomasma. "But we also developed new connections specifically for this year's bridge."

Michigan's team made an impressive mark on the competition, taking second in both the lightness and efficiency competitions, third in the stiffness competition, and eighth in speed. To design this year's bridge, team members split up into "mini-groups" and competed against each other to design the best bridge. The team used ideas from each group's designs to construct their final competition bridge.

About half of Michigan's team from last year returned to compete again, and recruiting new members was easier after last year's big win. "The department threw a huge party at the beginning of first semester for the championship team," Thomasma said. "Lots of people came out to the party and got involved that way. It also was easier to get sponsors after winning."

Michigan's civil engineering department helps with money and other aspects of the competition, such as equipment and facilities. "The University also has the Wilson Student Team Project Center, which is shared by many teams on campus that need special tools and machines," Thomasma said. "The center contains all kinds of tools and machines, including mills, welders, saws, and grinders."

Some University of Michigan professors attended the competition to show their support. "The enthusiasm [for the Steel Bridge Team] stems from the leadership of the ASCE student chapter," said professor Radoslaw Michalowski. "But what you see is absolutely their work. We can give broad suggestions, but the students do all the specific design work on their own."

A Super Tool

The University of Wisconsin-Madison's "super tool" helped the team stay at the top of the competition. The tool is an irregular-shaped red steel plate with different notches, grooves, and holes to perform different tasks aiding in the construction of the bridge. It can be used as a counterweight and to hold up half of a truss while it is being constructed.

"We used statistics from previous classes to determine how long the tool had to be to properly act as a counterweight for the bridge legs," said team co-chairman Drew Agosto. "After some quick calculations to find the center of gravity and the moment, we came up with the proper size. It worked on the first try!"

UW-Madison's team took fourth overall, third in construction speed, lightness, aesthetics and economy, and fifth in efficiency. Agosto said the team's three co-chairs tried hard this year to recruit new members and generate enthusiasm for the team.

"At the first ASCE general meeting, the three of us presented information on the bridge competition while wearing big boots and hard hats," he said. "We did one-armed push ups and other stunts. It created a lot of buzz around the team."

Many students enjoyed the setting of the Colorado School of Mines just outside of Denver, where they had the chance to do some hiking in the mountains and take tours of the nearby Coors Brewery. Some students from schools whose bridges failed at regional competitions made the trip to Colorado to gain insight for their own teams.

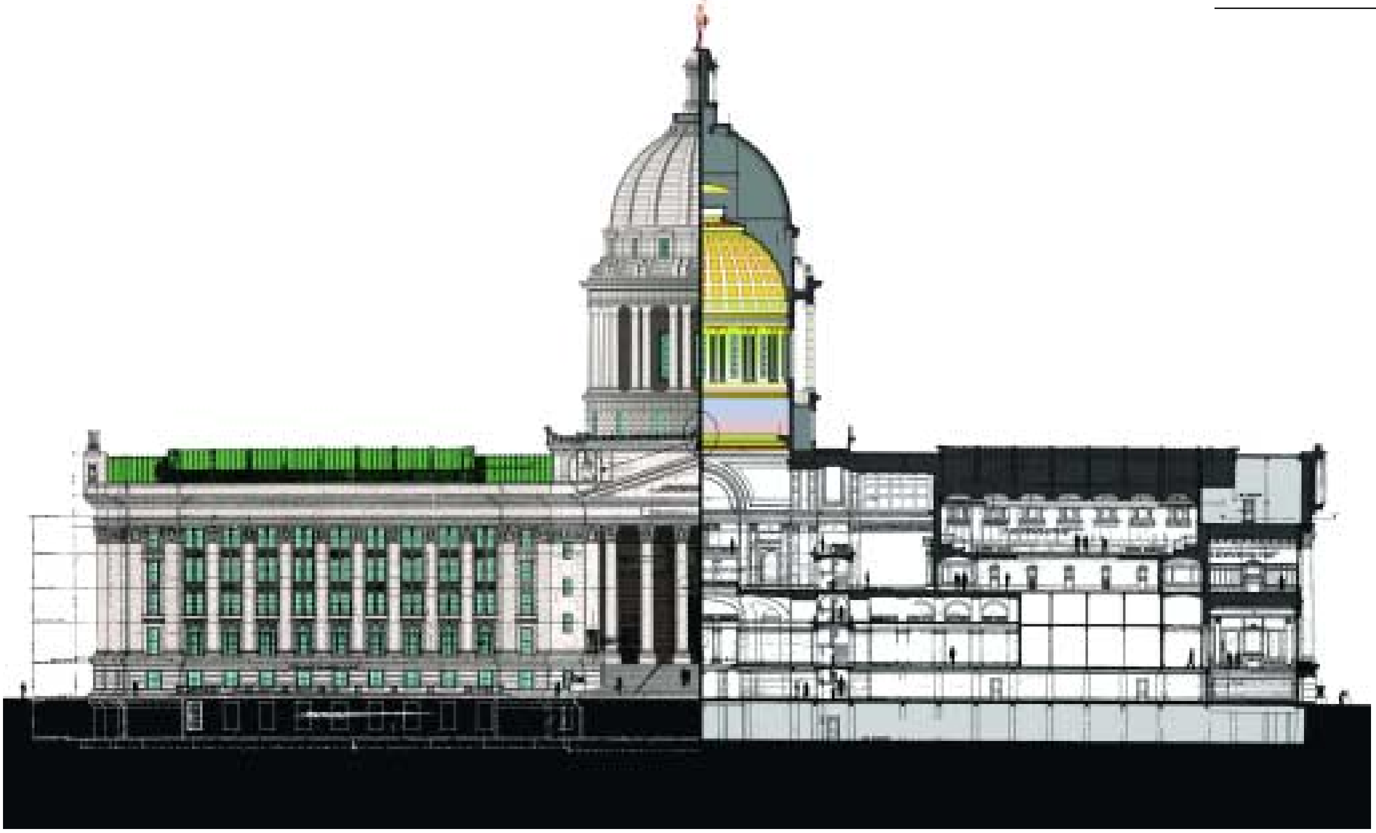
In addition to ASCE and the AISC, other sponsors included the American Iron and Steel Institute; the James F. Lincoln Arc Welding Foundation; the National Steel Bridge Alliance; and Nucor Corporation of Charlotte, North Carolina.

Next year's competition will be held at the University of Central Florida (Orlando), May 27-28, 2005. For more information about the competition, visit **www.aisc.org/steelbridge.html**. ★

Beth S. Pollak is associate editor of Modern Steel Construction. Dan Swiatek is a senior structural engineering student at University of Illinois-Urbana-Champaign and an intern at the American Institute of Steel Construction, Inc.

Capitol Improvements

By Gene O. Brown, P.E. and Timothy J. Dolf, P.E.



Graphic courtesy Frankfurt-Short-Bruza.

A design-build effort topped off Oklahoma's nearly 90-year-old State Capitol building with a crown of steel.

In late 2000, the Oklahoma Department of Central Services developed a $21 million project to construct a new dome for the State Capitol building—the State's first, major design-build contract. Oklahoma's State Capitol originally was constructed between 1914 and 1917, northeast of downtown Oklahoma City. While original plans for the Capitol indicated a large dome, a small saucer dome was constructed instead, since, at the time, World War I efforts drew money, labor and materials from domestic projects. The saucer dome was intended to be temporary, but it remained for nearly 90 years.

For the new dome project, the Department of Central Services and the Oklahoma Historical Society wanted a structure that closely resembled the original plans and matched the existing weathered limestone. The Capitol had to remain fully operational during construction, and artwork located below the saucer dome had to be protected from construction activities and the weather. A design-build approach allowed team members to share their expertise early in the planning stages to identify construction sequences and methods that would accommodate the owner's requirements (see design-build sidebar on p. 33 for more). The construction schedule allowed six months for site investigation, architectural design and engineering, and 19 months for construction. The completion date was scheduled to fall on Statehood Day, November 16, 2002.

Structural Exploration

Prior to architectural design and engineering, the existing condition of the Capitol had to be verified. A thorough, in-depth review had never taken place, but The Oklahoma Historical Society had preserved the original project specifications, construction drawings (ink on linen), construction photographs and Capitol Commission meeting minutes. Among the original construction drawings were five architectural sheets and a single structural sheet depicting the originally planned large dome, even though it was never constructed.

A detailed structural exploration was conducted using this information. Existing framing members and structural systems were verified, core samples were taken and steel reinforcing bars were removed for testing. Core samples indicated consistent concrete compressive strengths of about 1,610 psi in the superstructure, and 3,270 psi in the foundations. Steel reinforcing bars varied from square, twisted square, deformed round and smooth round bars, and consistently indicated a yield strength of approximately 49,000 psi. The investigation confirmed that the original structure was constructed to support the large dome.

The general plan consisted of an 80′-diameter exterior dome rising 140′ high above the existing roof. The dome would be capped with a 17′-tall, 6,000-lb bronze statue entitled "The Guardian," and would house an inner, coffered dome of glass-fiber-reinforced plaster panels. Access would be provided to the base of the lantern atop the dome through a series of ship ladders, catwalks and spiral staircases.

Photo courtesy R. Greg Hursley Incorporated.

Photo courtesy Capitol Dome Builders.

The concrete ring-beam from the existing dome served as the base for the new structure's 48 columns. Anchor rods were field-located to avoid disturbing the existing rebar in the ring beam. The field-located anchorages required that each base plate be custom-fitted at each location.

Since the original structural system was a cast-in-place-concrete frame, project architect/engineer Frankfurt-Short-Bruza (FSB) first explored concrete options. However, FSB realized that the original structure could not accommodate the additional weight of a concrete dome and meet current building codes. A steel-framed solution wrapped in architectural pre-cast concrete and cast stone (to match the appearance of the adjacent weathered limestone) provided a building system that was 60% lighter. After demolition of the small saucer dome and construction of the new dome, the net increase in overall building weight was approximately three million pounds (4% of the overall building weight). Minimizing the additional weight was important since the existing structure did not meet current seismic detailing requirements, and was not believed to be capable of supporting design seismic loads.

A structural steel frame was affordable and readily available from local sources. Steel connections and splices primarily used tension-control bolts. The steel framing could be easily assembled in sections on the ground and then lifted into place by the largest, free-standing tower crane ever erected in Oklahoma—a Comedil CTT 561-20. The crane could reach 230′ away from its tower with a height of 270′ above the ground. During times of high wind (when the crane could not be used) assembly of steel sections on the ground allowed construction to progress.

Designing the Dome

To meet the original dome design, the architectural features and proportions dictated the configuration of the structural steel framing system. The exterior colonnade around the drum of the dome consists of 16 pairs of 12″-diameter standard pipe columns wrapped in pre-cast concrete. The interior drum of the dome consists of 16 W14x176 wide-flange columns vertically braced above and below tall, narrow windows. Above these columns, at the base of the exterior dome, is a concrete-on-steel-deck tension ring that supports a mechanical mezzanine and forms the oculus of the interior dome. From this tension ring, 16 arched W10x49 wide-flange sections form the dome and terminate at the compression ring near the base of the lantern. A 36″-diameter steel pipe with a large cap plate extends 30′ upward through the lantern to support the base of the statue. Including connections, the structural steel totaled approximately 250 tons.

The lateral-force-resisting systems varied, since moment frames, vertically braced frames and inverted pendulums were all used at different elevations. Due to architectural features, vertical bracing could not be used exclusively. The largest portion of the lateral drift occurs within the drum section where vertical bracing had to be discontinued at the tall, slender windows, which in effect created a series of vertical Vierendeel trusses. The total lateral design drift of the new dome was limited to its height divided by 300. The final structural models indicated that the dome's horizontal drift would be approximately 1.6″ under a design seismic event and 1.4″ under the design wind loads.

The new dome attached to an existing 5′-thick-by-9½′ wide concrete ring beam (100′ above the foundation/basement level) which was in turn supported by columns that formed the central core area of the Capitol. In order to preserve the strength and integrity of the concrete ring beam, care was taken to not damage any of the existing rebar. Workers determined each anchor-bolt location by locally chipping down to the top mat of reinforcement, then drilling small pilot holes to locate and avoid lower mats of reinforcement. The 48 columns each have a unique base plate since all 352 post-installed anchors were field-located to avoid the existing rebar.

Designers modeled the new structural steel framing atop the original central core area of the Capitol building and existing spread-footing foundation. They used three-dimensional structural modeling software, and due to the complexity of the construction sequencing, several models were required to analyze the structure. The computer models used a P-Delta analysis and were simplified by taking advantage of the inherent radial symmetry of the dome. The final structural model required 1,704 joints, 452 spring supports and 2,389 elements to model the existing foundation, existing concrete members and the new steel members.

Capitol Construction

Two temporary work platforms were incorporated into the dome design. The lower platform was located within the existing structure, on the fourth floor of the Capitol, to support interior demolition of the small saucer dome and later to support construction efforts on the interior of the dome. The upper platform attached to the new structure, creating a work-staging area. It also created a weather-tight enclosure with temporary walls, forming a white box that extended downward to seal off the opening in the Capitol's roof, 26′ below. This helped protect the original artwork and allowed the Capitol to remain operational during construction.

The radial beams of the platform projected through the 16 future window openings, and the columns extended

Why Choose Design-Build?

For the Oklahoma Capitol Dome, the design-build approach made sense due to the project's unknown conditions and the need to incorporate complex construction sequences into the design. Modern construction materials had to match the original materials—within budget, on a tight schedule and under intense scrutiny.

Capitol Dome Builders worked with FSB to develop construction sequencing and the concept of temporary, elevated work platforms. These platforms served as weather-tight enclosures and had to be incorporated into the structural design since they represented the largest load that the structure would experience. Had this been a traditional design-bid-build project, the design team might not have accounted for these work platforms, and the contractor would have had to stage all work on the ground, hundreds of feet below. While this would have made the structural component of the dome more economical, the contractor determined that additional materials were not as costly as the potential costs of additional time, money and labor.

Some other benefits of the design-build team:

- ➜ Team communication played a positive role in identifying potential complications and challenges before they became a construction problem.
- ➜ When unexpected field conditions were encountered, engineers arrived quickly on-site to help determine a solution.
- ➜ Design-build contracts can offer a less adversarial relationship between contractors and engineers, since they are team members under the same contract. They allow direct communication between the engineers and contractors. For example, incorporating the elevated work (staging) platforms saved time and money by making construction/staging more convenient.
- ➜ Since the contractor was on-board and involved during design, early steel packages allowed construction to follow directly behind design without any delay. Site mobilization occurred prior to the completion of design. Steel shop drawings arrived just as 100% construction documents for other disciplines were completed.
- ➜ Being part of the design-build team meant FSB could aid in connection design, tower-crane foundation design, platform/scaffolding checks on the existing structure, and other tasks.
- ➜ FSB's involvement in connection design drastically reduced shop-drawing review time. ★

Photo courtesy Capitol Dome Builders.

through the platform and were sealed. Hatches and a penthouse were constructed to allow workers, demolition debris and construction materials to pass back and forth through the weather-tight enclosure during construction. The top of the platform was approximately 12,000 sq. ft and could support a 50 psf live load. It was constructed with 2″ of polypropylene-fiber-reinforced concrete over a water-proofing membrane, over another 2″ of reinforced concrete. Scaffolding was erected from the perimeter of this platform as the exterior construction progressed. As the exterior of the dome neared completion, interior portions of the platform were removed so the interior dome could be constructed through the platform. When the exterior portion of the dome was completed and became weather-tight, the entire platform was removed through the window openings. This allowed interior construction to progress upward with a 140′-tall interior scaffolding system from the existing fourth floor below.

The governing gravity-load combination was anticipated to occur near the completion of exterior construction, when the building was complete and the platforms were still supporting 120,000 lb of scaffolding and other construction materials and debris. The design-build process helped the architect, engineer and contractor incorporate realistic construction loads and needs into the structural design—cutting the construction schedule and lowering the construction cost.

Since designing and constructing a dome on an existing Capitol had not been attempted in the United States since the U.S. Capitol in 1865, this was a once-in-a-lifetime project. Many Oklahomans watched in person or on live television as the dome was dedicated in an Oklahoma-themed show on Statehood Day, with the largest fireworks display in State history. ★

Gene O. Brown, P.E. and Timothy J. Dolf, P.E. are project structural engineers for Frankfurt-Short-Bruza Associates, P.C., in Oklahoma City.

Owner
The State of Oklahoma, Department of Central Services

Architect and Engineer of Record
Frankfurt-Short-Bruza Associates, P.C., Architects Engineers and Planners, Oklahoma City

Contractors
Capitol Dome Builders: (Manhattan Construction Company, Oklahoma City and Flintco Incorporated, Oklahoma City)

Structural Steel Fabricator
H&M Steel Corporation, Luther, OK (AISC member)

Detailing
Connection Design: Frankfurt-Short-Bruza Associates, P.C., Oklahoma City
Shop Drawings: H&M Steel Corporation, Luther, OK (AISC member)

Structural Software
STAAD.Pro

Classic Papers from AISC's
Engineering Journal

The AISC *Engineering Journal* is a practical journal designed to present papers with information that is useful and relevant. Many of the questions received by the AISC Steel Solutions Center can be answered by referencing one of the papers published in the journal. *EJ* is available to all by subscription and is available to members through free electronic access using AISC's ePubs feature at **www.aisc.org/epubs**.

While all papers in the AISC *Engineering Journal* make a contribution to the state of knowledge of steel design and construction, some papers rise above the rest and stand as seminal in their importance. This regular feature in *Modern Steel Construction* magazine will highlight those most notable of works in the AISC *Engineering Journal*.

Cambering Steel Beams

By David T. Ricker
From the 4th Quarter 1989 issue

The following highlights can be examined in greater detail by reading the full paper, available at **www.aisc.org/epubs**.

Types of Camber: The author distinguishes between natural camber (the out-of-straightness remaining after the initial rolling, cooling, and straightening of the member at the mill) and induced camber (the curvature that is applied subsequent to the initial rolling and straightening process, usually in the fabricating shop).

The Camber Curve: The deflection curve for a horizontal, uniformly loaded member of constant cross section theoretically approximates a parabola. However, since the sag ratio for beams, that is, the ratio of mid-ordinate to chord, is so small, it generally is accepted that adequate accuracy results if the camber curve is considered to be a segment of a circular curve. Some camber calculations are based on circular curves. Camber usually is expressed in terms of the maximum ordinate at mid-span. The full paper gives a method to determine the approximate camber at other points along this curve.

Sage Advice, with Humor to Boot: Once the deflections are determined, the desired amount of camber can be selected. The selection of camber is often arbitrary. The methods of cambering are relatively crude, and the results are less than precise. There is little need nor justification in meticulous mathematical manipulation or methodical multifarious meditation when it comes to determining camber requirements.

Cold Cambering Using Force: The types of steel used for structural purposes are ductile, that is, they have the property of deforming extensively under substantially constant stress…When we cold-camber a beam, the extreme fibers reach stresses and deformations that are on the horizontal portion of the stress-strain curve. With most cold-cambering devices, it is impossible or difficult to maintain a constant stress. The rams which deliver the force advance when the button is pushed and halt when the button is released. When the ram is halted, the deformation and stresses come to balance in a brief moment, and equilibrium is reached. When the ram is retracted, the beam relaxes and some residual deformation is evident by the fact that the beam is no longer straight. This essentially is the cold-cambering process.

We all know that bending a wire back and forth enough times will weaken it to the point where it will fail with very little effort. What about the cold-cambered beam, which we bent in one direction to make the camber curve and are now loading in the opposite direction with its service loads? Another of the seemingly endless wonderful properties of structural steel is that, if allowed to rest for a few hours at room temperature, steel has the tendency to recover its elastic properties. The application of mild heat, about 225°F for a few minutes, will accelerate the period of recovery.

Mention should be made here of the term "strain hardened." This consists of an alteration of the elastic properties of cold-worked steel and a raising of the proportional-limit stress, as a result of the aforementioned aging or application of mild heat. Two facts emerge from this brief discussion of cold bending:

1. The same allowable stresses (or load factors) can be applied to cold-cambered beams as to uncambered beams, provided that they are allowed to "age" for a few hours.
2. Never attempt to reduce the camber in an overcambered beam by immediately applying force in the opposite direction. If this caution is ignored, strain weakening will result and the elastic-limit will be lowered. If normal, allowable stresses subsequently are assigned to the member, the factor of safety will be reduced.

When a beam is to be cold-cambered by force, it is usually mounted in some type of rigid frame which holds the beam secure while the force is delivered. The ends of the beam must be laterally supported in addition to the compression flange. The fulcrums must be devised so that the ends are free to rotate because of the severe arc to which the beam is forced in order to produce the residual strain. There are usually two points at which the load is delivered to the beam. These points might be about 6′ or 8′ apart, while the resisting supports can be 22′ or so apart. This will produce a curve that is very close to duplicating a parabola.

When the operator activates the cambering device, the rams advance and the beam deflects, often as much as two or three times the amount of desired camber. The rams are halted and kept in place for a few seconds, during which the steel microstructure undergoes rearrangement and the stresses relax somewhat. When the rams are retracted, the beam springs back, stresses drop to zero, and a permanent set is left in the beam. If the deformation is insufficient, the rams can be immediately reactivated to add more camber. Contrary to popular belief, cold cambering does not result in significant residual stresses, at least in the flange areas. A bar shape, if bent cold to a tight radius, will contain residual stresses but, when a beam is bent, the radius of the bend is so large that the tension flange undergoes nearly pure tension while the compression flange is subject to nearly pure compression. Pure tension "stretching" and compression "squeezing" do not produce residual stresses; therefore, there should be no concern over this phenomenon when cold-cambering beams.

Cambering Using Heat: The heat application must not exceed 1100°F for ASTM A514 steel nor 1200°F for other structural-type steels. The temperatures should be monitored by heat-sensitive crayons or other suitable means. There is no reason to exceed these temperatures. In fact, most cambering can be done at temperatures lower than these maximums. The consequences of overheating are not readily apparent to the naked eye, but nonetheless they are present in the form of microstructure changes in the steel.

Most heat cambering is accomplished by heating wedge-shaped segments at intervals along the length of the member. The number of wedge-shaped heated segments varies depending on length and size of the member and the amount of camber required. For starters, try two heated areas at the 3/8 and 5/8 locations. Experience will be the best teacher. Before heating, install a plumb line or other device so that the movement can be monitored. Once the heating starts, it should not be interrupted until the serpentine path described below is completed. As the heating proceeds, the member will start to bend in the direction opposite to that intended. However, after the heating is finished and the beam starts to cool, the beam will commence to straighten and then continue to bend in the desired direction... For best results, let the heated member cool by itself... Heat also can be used to straighten members. The heat source can be anything that works—natural gas, propane, and oxy-acetylene mixtures all work well. A commonly used torch nozzle is a medium to large (approximately 1½" diameter) "rosebud" type.

Members that Lend Themselves to Cambering: Filler beams, girder beams, composite floor beams, and members with uniform cross section.

Members that Do Not Lend Themselves to Cambering: crane beams or crane girders, spandrel beams, especially those supporting facia materials, c. beams with single or double cantilevers, beams braced with knee braces, beams with full moment connections or significant semi-rigid moment connections, beams with welded cover plates, especially if the cover plate does not extend full length, members of non-uniform cross section, beams with significant non-symmetrical loading, short beams—less than 20′ in length, shallow beams—wide-flange shapes less than nominal 14" depth and standard beams less than 12" depth, beams subject to significant torsion loads, and beams which would require less than 1" of camber. (Small camber requirements can often be satisfied by natural mill camber.)

Establishing the Amount of Camber: Beams can be cambered to accommodate part of the dead-load deflection, the full dead-load deflection, or dead-load deflection plus part of the live-load deflection, at the discretion of the engineer. This can be influenced by the relative percentages of dead and live load, the probable frequency and intensity of live load, the performance history of similar members, aesthetics, or other pertinent factors.

As previously mentioned, determining the amount of camber is a very inexact process. After the cambering process, performance of the member often is not according to the script. In general, the anticipated amount of beam deflection does not occur. This probably is due to some degree of end fixity of the beam connections.

When to Camber: Usually the cambering, if performed by the fabricator, is done after the member has been cut to length and punched or drilled. Beams that require square and parallel ends, such as for end-plate or welded-moment connections, must be cut after cambering. Any interior hole groups will be perpendicular to the flanges at their locations. The fact that the beams that frame to these hole groups will not be exactly vertical is of small consequence.

Camber Cautions

a. Don't over-camber beams which receive shear studs for composite action. The over-cambering could result in the heads of the studs protruding from the top of the concrete slab.

b. Don't cold-camber beams to which a cover plate will subsequently be welded. The heat thus generated at one flange will generally be enough to significantly alter the camber curve.

c. When cold-cambering by force, make sure that there is sufficient length of bearing at the load points in order to prevent local flange and web buckling. The fulcrums must be free to rotate so as to conform to the camber curve.

d. When heat cambering, do not overheat the steel.

e. Do not force-cool heated members with water or air spray if their temperature is greater than 700°F.

f. Columns comprised of section sizes normally associated with beams should, when ordered from the mill, be noted "no camber permitted." Otherwise, the mill may provide the members with a small amount of "natural" mill camber.

g. Heat cambering should be performed only on low-carbon steels. Application of heat to medium-and high-carbon steels increases the danger of embrittlement. ASTM A36, A572 Gr. 50, A588, A441, and A242 are popular low-carbon steels. [Note ASTM A992 is also a low-carbon steel.]

h. When using the heat-cambering method, it is not necessary to reheat previously heated areas when following the serpentine path. Allow the steel to cool naturally behind the torch. ★

Complex Apartments

By Beth S. Pollak and Michael Gustafson

An innovative staggered truss system and a design-build effort sped the completion of an apartment complex in Ames, IA.

"Everyone thought that the staggered truss was a good concept, considering cost, schedule and simplicity," said Gene Arnold, P.E. structural engineer for the new Legacy Tower apartment complex in Ames, IA. It was Arnold's first staggered truss project, but he was familiar with the system through AISC's educational efforts from years past.

"We started designing the job about the time that *Modern Steel Construction* published its "Anatomy of a Staggered Truss" insert (Sept. 2002), and the sample set of plans helped," he said. "We drew up some models of what the building might look like and presented them to the owner during the preliminary design stages. The architect came up with a floor plan, and everybody liked the layout of the system. The owner had experience with structures made out of steel and precast concrete before, and since we have good fabricators and good precasters locally, the staggered truss would use both materials quite well."

Fifty-six staggered trusses frame the 96-room luxury apartment complex, located near Iowa State University. The structure consists of four levels (each with a main and mezzanine level) of residential space and three levels of pre-cast concrete parking structure below.

At each grid line, two 57′-6″-long staggered trusses span each side of a 7′-0″ wide corridor. Due to limited site access and crane-lift capacities, designers split the building width by using two trusses instead of one. The use of two trusses allowed a stiffer truss section, which enabled easier fit-up of the structure.

An accelerated schedule and design-build project delivery helped complete the building in less than two years. "The whole team worked together exceptionally well," said Project Manager Lee Welch. "Design-build facilitates communication between members. It's definitely the way to go. If you get your team put together early on, everyone has good input and works out problems in advance."

Robert Pyle and Michael Gustafson,

Having the fabricator, erector, precaster and general contractor on-board from the beginning helped reduce project cost and shorten the schedule. We worked with all of the main trades including electrical and mechanical in schematic design to determine the schedule so as to meet the short time frame available to build the project. Having everyone involved from day-one worked very well for the owner and the project.

In contrast, with the design-bid-build method of delivery design professionals can overlook the realities of construction methods and schedules. I don't profess to know everything there is about building a building, that is what the construction team is about. By working with the expertise of the entire team of designers and major subcontractors the finer details could be looked at more closely resulting in faster delivery and lower cost.

—Mike Sloter
Senior Project Manager
Stott & Associates Architects, P.C.

regional engineers with AISC Marketing, worked with the design and construction team early in the project. They hosted pre-design and pre-erection meetings with team members to educate them on the lessons learned from previous staggered-truss construction projects.

"The major design requirement for this project was to fit as large of a building with as many apartment units as possible on the given site and to erect the building as quickly as possible," said Architect Michael J. Sloter, AIA. "The staggered truss system seemed to meet the needs of the short construction time by providing a repetitive template for the fabricator. Other benefits included repetitive components, column-free spaces, speed of erection and flexibility."

Due to height restrictions on high-rise construction in Ames, the owner maximized floor-plan size while minimizing the overall height of the structure to approximately 111′. The floor plan is a double-loaded layout, where two units are accessed on each side of the central corridor. The building stories consist of alternating main and mezzanine floors.

The mezzanine floor layout permits a creative, two-level, loft approach that is probably the first of its kind for staggered truss construction. At each mezzanine floor, the precast concrete floor plank is removed within the interior of the floor plan. This allows each apartment to have a two-level occupancy, where tenants access the mezzanine from their main floor via staircases. The mezzanine level hosts a balcony at the exterior of the building.

Typically, a great strength of staggered-truss construction is low floor-to-floor heights. However, in this case, the staggered truss system was flexible enough to allow an 18′ ceiling height in some areas while maintaining a low frame floor-to-floor height of 9′-0″.

"Steel, for this project, was the best in terms of schedule, cost, ease of assembly, and large open spaces," Arnold said. "With the column spacing, you couldn't have built this out of all concrete. Trusses allowed us to eliminate the floor beams and use the precast planks for the ceilings."

The building required Type 1–Fire Resistance (1997 UBC), a three-hour rating, on all structural components. As such, the trusses were enclosed in three layers of 5/8″ gypsum board.

The Legacy Towers' mezzanine loft approach is probably the first of its kind for staggered-truss construction. At each mezzanine floor, the pre-cast concrete floor planks are removed within the interior of the floor plan. This allows each apartment to have a two-level occupancy, where tenants access the mezzanine from their main floor via staircases.

Some benefits of the early-involvement were:

- A good relationship with the structural engineer meant we could see the designs up-front and talk about the efficiencies of the steel sections. We knew what sections were being rolled, what was in stock, and what could be applied in the field, so there was good communication with engineering, detailing and purchasing.
- Because we had that information up front, we had a good comfort level that our take-off list was the correct material. We could order material early knowing we had the right stock coming.
- Ordering materials early allowed us to take advantage of mill pricing and receive stock with less costly rail delivery, reducing overall cost for the owner.
- With steel arriving early, even though it took some of our space, we had it all in stock, nothing was missing, and we were not waiting for odd members when we got started.
- We only run one shift, so it is important for us to manage our workload. By starting early on this project, we were able to manage shop-fabrication hours well and stage fabricated material before it was needed at the job site. The early start allowed us to get ahead, so we didn't have to worry about delaying the schedule. It also allowed us some flexibility in production planning.

—Doug Kollbaum
President, Mid-States Steel

Teamwork

Early involvement of all steel specialty contractors was essential to the success of the project. All key construction and design team members had a direct contract with the owner, including the steel fabricator. This true design-build approach provided many benefits for the project.

"Before they even broke ground, there were several meetings early on with the fabricator, detailer, precast supplier/erector, construction manager, engineer and architect," said detailer Jeff Drake, of Mid-States Steel. "We had worked with the structural engineer on other projects. We knew the basic design and gave ideas to each other. When there were design changes, communication between parties was very quick."

The steel fabricator and detailer worked with the structural engineer to minimize the overall frame costs of the structure. The number of different trusses, connection details, truss-member sizes and types, and temporary erection systems were all discussed and resolved in the project's design phase. Early on, the steel fabricator suggested making design changes to the trusses based on three-member-bundle quantity orders. Recognizing the benefits of economies of

The footprint of Legacy Towers takes up about 97% of the site, making construction challenging.

scale helped minimize the overall cost impact of changes to the project.

"We had several meetings trying to get the material sizes nailed down, so we could get it ordered from mill sources and delivered by rail—the most economical way to transport the material," Drake said. "The trusses are almost 60' long. Originally the design called for the trusses to be three-chord, 18'-high trusses—that would be too deep and expensive to ship. So the designers scaled it back, and went with 9'-high trusses."

Also during planning stages, the steel erector and the precast supplier (Iowa Prestresed Concrete, Inc.) discussed what size crane to use, so that the precast members could be designed around the capacity of the proposed crane. The team chose a PECCO SK400 Tower Crane with a 162' radius and maximum capacity of 40,000 lb. For this project, the erector selected a 20,000-lb maximum capacity, and the precast members were sized accordingly.

Another benefit of the early involvement of the steel fabricator with the design team was the ability to place a mill order at 75% design completion with minimal risk to the owner. Typical design-bid-build models that conduct such early mill orders impose additional risk to the owner because of the potential steel frame changes that occur during the remainder of the design. For this project, the early steel order saved money for the fabricator, who could purchase steel according to mill schedules, ship steel by rail rather than pay trucking costs, and fabricate the steel with time to spare.

"I can't emphasize enough how important early involvement was for us in terms of material handling and costs, rail delivery, giving us the ability to work in our schedule, and other savings," said Doug Kollbaum, president of Mid-States Steel. "This really worked well from the fabricator's standpoint. I wish I had 10 more jobs like that right now."

Erection

The owner believed that structural steel would simplify the construction process, especially with the proposed mezzanine loft floor plan. Although precast concrete was used for the parking structure, it could not offer the low floor-to-floor heights of the staggered-truss frame.

"The structural engineer really convinced us that we ought to try the staggered truss system," Welch said. "From an erection standpoint, the building went up very fast, even faster than if we had a conventional steel frame and poured concrete floors, especially because we were building during the winter months. It saved us a lot of time. The staggered truss system allowed us flexibility as far as the spans between the trusses, and in how we could lay out the interior walls of the building."

Fabrication began in August 2003. "Because of the staggered truss system, there was a lot of steel, but not a lot of pieces, so it was very easy to manage the actual piece-mark numbers, compared to conventional column framing," Kollbaum said. "There were 56 trusses, 72 columns, and approximately 600 beams. The structure erected quickly, like an erector set."

Erection began Dec. 15, 2003 and ended April 1, 2004. It took 14 weeks as planned, even with some days lost during the coldest winter months. The erector was able to make up lost time, partially due to the ease of erection of the staggered trusses and precast plank. The steel and precast un-topped system minimized the risk of tripping and other hazards on site, especially during winter. Temporary bracing of the staggered truss was provided through coordination between the structural engineer and erector.

The limited staging area created a construction challenge, with 97% of the site used for the building footprint. Truss

I think communication with a design-build team is always better [than with a design-bid-build team].

From connections to piece sizes, and to the sequence of putting it together, everyone can sit down and share their perspective of the project. That's what happened here.

Also, with design-build, you have a ballpark figure of what materials and services will cost. You have to have a high level of trust by picking team members, bringing them on board and assuming you'll be able to negotiate a price that is fair. All of us have worked together on several projects and all the players knew each other quite well.

We have a great deal of trust with Project Manager Lee Welch and Superintendent Dennis Oppedal. They ask great questions and can work with every discipline. From beginning to end, they kept everyone working together.

—Gene Arnold, P.E.
Arnold Engineering

and precast concrete deliveries were coordinated to establish a quick-flowing erection process.

"Lee Welch and his staff, with the direction of the erector, handled the sequencing well," Kollbaum said. "We broke down the sequences and staged them. They handled changes in the field. We are only 15 miles to the site, so they would call us if there were any construction delays. Since we're close to the site, we were able to use our own tractor, and we didn't have to spend money on a separate carrier."

There are different ways to erect the staggered truss on site: either connecting the columns to the trusses on the ground and then erecting, or erecting the columns first and then sliding in the trusses. The latter was done in this case. However, due to rotation of the trusses after installation and therefore rotation of the columns, the erection of additional trusses above required additional fit-up requirements. Using one-story column splices instead of two-story column splices would minimize the amount of rotational effect on the extension of the columns, thereby minimizing fit-up issues.

"One thing we discovered in hindsight was that it would have been helpful to assemble more of the frames in the shop," Arnold said. "Some small x-braced frames and horizontal frames would have been better assembled in the shop. The more assembly you can do in the shop, the better, and this system lends itself to that."

Welch said that the staggered truss system met the project requirements on time, giving Ames an attractive new residential complex. "There was no other method or material that could have been used on this building that would have worked in our schedule." ★

Owner/Contractor
Ev Cochrane & Associates, Ames, IA

Architect
Stott & Associates Architects, P.C., Ames, IA

Structural Engineer
Arnold Engineering, Waukee, IA

Steel Fabricator/Detailer
Mid-States Steel Corp., Boone, IA (AISC-member)

Structural Software
RISA

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Over the Top

By Richard C. Pennock, P.E. and Ronald E. Thompson, P.E.

Steel canopies offer a lighter and brighter view at several new stations for the Washington, D.C.-area rapid transit system.

From its opening day in March 1976, to the present, Washington, D.C.'s Metro commuter rail has maintained a uniform architectural appearance. The at- and above-grade stations feature distinctively arched, reinforced-concrete canopies at trackside to protect commuters from rain, snow and sun. The canopies' design mimics the column-free openness of the barrel-arch vault design of Metro's underground stations. Some Metro riders, however, will soon notice a subtle change: new, above-ground stations scheduled to open in December 2004, use steel "umbrella" canopies instead of the cast-in-place or pre-cast concrete canopies traditionally built at the system's 31 other surface stations.

The use of steel canopies at the new stations constituted a big change in engineering design but only a subtle departure from the original architectural concept. The change was partly a response by the Washington Metropolitan Area Transit Authority (WMATA) to feedback from commuters and elected officials that some of the concrete-canopy stations were dark and uninviting. Acordingly, WMATA's architects worked to develop a new station design to address community concerns.

WMATA's consultants, Capital Transit Consultants (CTC) began considering steel for canopy design in order to achieve a lighter and more delicate structural appearance. The conceptual design that developed called for exposed arched trusses spaced on 6'-8" centers. Although the structures were to be designed for the code-mandated loading conditions, WMATA conducted wind tunnel tests and determined load forces in the truss members using STAAD.Pro 2002.

Because one of the three new stations would be built within the District of Columbia, the new design concept required review and approval by the U.S. Commission of Fine Arts and the National Capitol Planning Commission.

With their approval in hand, WMATA was ready to move forward with the detailed design and construction of the steel-canopy concept for the new stations, providing bridging-type documents in its "Design-Build Request-For-Proposal." The two new stations discussed here (Morgan Boulevard and Largo Town Center), are part of a 3.1 mile lengthening of the commuter rail system's Blue Line that extends Metro beyond the Capital Beltway east of the city.

In fall 2002, WMATA awarded a design-build contract to Clark-Kiewit Largo, a joint venture of Clark Construction Group, Inc. of Bethesda, MD, and Kiewit Construction of Omaha, NE. The $92 million contract includes designing and constructing two stations, ancillary parking facilities, and, at Morgan Boulevard, a childcare center. Morgan Boulevard is located just outside Washington's eastern city limits, while the next station, Largo Town Center, is the new line terminus at Largo, MD, just outside the Capital Beltway. Clark-Kiewit subcontracted the project's design to HSMM/STV Largo, a joint venture between Hayes, Seay, Mattern & Mattern, Inc., of Roanoke, VA, and STV Incorporated, New York City.

The HSMM/STV Largo team tackled the challenge of developing construction documents from the WMATA-provided design-build documents. Working closely with the design-builder and the structural steel subcontractor, the design-build team created a workable design, fabrication, and erection solution that met WMATA's vision for the canopy. Largo's platform canopy covers 385′ of the 600′-long structure and spans 44′-6″ over the 34′-wide platform, providing approximately 5′ over-the-track protection on each side. The Largo station design also included a 69′-2″ by 84′-8″ pavilion-roof structure, providing a station gateway from the south parking area. This stand-alone structure's design was relatively straightforward.

In contrast, at Morgan Boulevard, the pavilion design is cruciform. It overlaps with and draws commuters to the linear concourse canopy. Morgan's cruciform pavilion is 184′-8″ long by 69′-2″ wide on its long axis parallel to the concourse, while the right-angle intersecting section runs 111′-4″ by 69′-2." The concourse canopy is 251′-4″ by 40′-8."

The design-build team had to address critical fabrication and erection issues along with the complex design requirements. HSMM/STV structural engineers worked hand-in-hand with the design-builder, the structural steel fabricator, the steel erectors and WMATA to achieve an optimal design and construction solution at each station.

Structurally, the design called for triangular box girders to support trusses on the canopies' long axis. The unusual shape and relatively long length (from 33′-4″ to more than 75′), coupled with their thin (½″) plate sections, made handling the girders during fabrication, shipping/handling, and erection a tricky process at best. Controlling distortions in the fabrication shop was also a major concern. On-site erection, handled by AISC-member Williams Steel Erection Company, called for extensive field welding to provide an essentially seamless sightline for on-site appearance by Metro users, with erection connections cut-off and ground to a smooth finish for final coating. To avoid extensive field welds, the girders were fabricated to longer-than-usual lengths where possible.

Strict specifications applied to other tolerances affecting visual impact, including welds, cambers, member sweep and final alignments. The combined "forces" made erection a difficult exercise. Moreover, the design-build team had to carefully coordinate erection logistics at the Largo Town Center site, since the platform steel was erected simultaneously with erection of pre-cast panels and structural members of two multi-level garages flanking the platform.

Fabrication differed between the stations, but in general, the canopies are made from a pair of parallel, hollow triangular steel girders on the long axis, with a series of plate trusses spanning between the girders, and cantilevering beyond the girders and over the tracks. The girders are made from ½″ plate, with either a 1′-4″ or 2′ width at the base, depending on location. Each side tapers inward as it rises by 10 degrees from the vertical.

The plate trusses are, for the most part, 1¼″ thick at the platforms, and 1½″ at the pavilions The "valley" trusses for the Morgan cruciform are 2″ thick. The plate trusses also have a ½″-by-5″ flange along the top edge to provide a surface for attaching the steel roof deck. As with the other trusses, these are also set on 6′-8″ centers along the girder length.

The girders are supported by 14″- and 20″-diameter steel pipe columns that are, in turn, supported by an understructure of reinforced or pre-cast concrete. Strips of skylights located between the plate trusses run most of the length of the canopy to help create the brighter, more welcoming atmosphere sought by the public. The rest of the canopy is covered by exposed-to-view, painted acoustical Versa-Deck and topped with standing-seam roof decking. The Largo platform also uses acoustical Versa-Deck to minimize the reflective noise from approaching and departing trains.

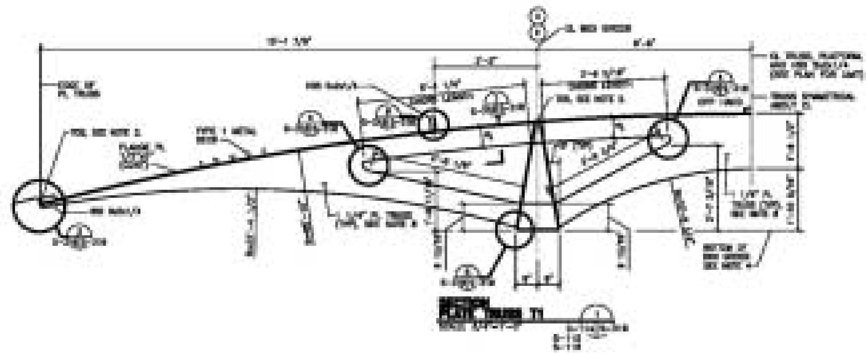
The hollow girders serve another important function in addition to their structural role. They offer an excellent location for recessed lighting, public address system speakers, mounting space for CCTV security cameras, and the electronic passenger information display system signage. The hollow columns provide out-of-sight conduit to support the various safety functions.

HSMM developed the detailed structural design for the Largo station and shared it with STV for the Morgan Boulevard station. STV modified the design for the canopy and detailed it to meet the needs of the cruciform pavilion.

Key to the project's success was the close design-build collaboration by the engineers and contractors with AISC-member fabricator Owen Steel Company. Owen personnel worked with engineers and contractors through every detail of design, fabrication and erection to resolve issues before construction. Some concerns included how to weld the relatively thin girder plates without causing warping; transferring tension and compression forces from the plate truss chords through the hollow girders; and how to provide temporary support to the truss plates before welding them in place to the girders.

The collaborative process led to minimizing weld sizes and to using fillet rather than full-penetration welds to reduce the possibility of heat warping. Contractor Clark-Kiewit demonstrated to the client that the fillet-weld offered the required strength and a better-looking finished product than the client-specified full-penetration welds.

This was particularly significant since the girder side plates were fairly flexible to begin with and lost some stiffness when holes were cut in the plate to accommodate light fixtures. The designers worked with Owen to attack this problem by adding internal stiffeners inside the girders to minimize the possibility of plate warping. The stiffeners



The triangular box girders (at column lines D and E) run parallel to the rail tracks, while the perpendicular plate trusses are attached every 6’-8” along the girder length.

themselves served a dual purpose. The cantilevered roof trusses, spaced on 6’-8” centers along the girder length, created significant tension (top) and compression (bottom) forces in the chords of the plate trusses that had to be transferred through the triangular, hollow girder. The stiffeners were placed inside the girders to match the locations of the truss chords, with a center hole cut to accommodate various system wiring. This solution was chosen after verifying that the welded stiffeners could hold the loads sufficiently.

Owen used Design Data’s SDS/2 software to detail the project and used the software’s automated downloading capabilities to export data into the SigmaNEST fabrication package. The software dictated the mill plate layout to cut the maximum number of truss plates from a single steel sheet. This “nested” program was imported into the cutting machines to yield a series of identical plate trusses. The equipment used to cut the plate trusses were L-Tec gantry burning tables outfitted with ‘Burny’ computer numerical controls (CNC). In all, some 1,234 tons of U.S.-made steel was consumed for canopies at the two stations, with much more used in the garages, rails and other features at the two sites.

When the new stations open, the public will enjoy these inviting above-ground stations that maintain the older stations’ signature canopy but with a more user-friendly appearance and a brighter appearance. Refined public architecture is meeting the public’s will with an “umbrella of steel” that took many collaborative hands to bring to realization.

Richard C. Pennock, P.E., is an associate with Hayes, Seay, Mattern & Mattern, Inc., in Roanoke, VA. Ronald E. Thompson, P.E., is a senior vice president of STV Inc., and heads the firm’s Vienna, VA, office.

Project Owner
Washington Metropolitan Area Transit Authority (WMATA), Washington, D.C.

Design-Build Contractor
Clarke-Kiewit, A Joint Venture, Bethesda, MD

Architect/Structural Engineer
HSMM/STV Largo, A Joint Venture
HSMM, Inc., Roanoke, VA
STV, Incorporated, New York City

Steel Fabricator/Detailer
Owen Steel Company, Columbia, SC (AISC member)

Steel Erector
Williams Steel Erection Company, Manassas, VA (AISC member)

Structural Engineering Software
STAAD.Pro 2002

Detailing Software
SDS/2

Fabrication Software
SigmaNEST

Fabrication Machinery
ESAB L-Tec Steel Industry Products

Rapid Replay

By Gene Martin, P.E. and Beth S. Pollak, with David R. Simpson, P.E.

Trust, teamwork, and technology boost a quick switch from concrete to steel in the expansion of West Virginia University's Mountaineer Stadium in Morgantown, WV.

In less than a year, 18 luxury suites and 648 seats were to be added at the north end zone of West Virginia University's Mountaineer Stadium in time for the team's opening game on September 4, 2004.

The project design team initially chose concrete for the expansion. However, during the bidding process, the eventual general contractor, March-Westin Company, suggested to the owner that a steel option should be investigated because of the tight schedule, difficulties facing winter construction, and the possible costs savings of steel versus concrete. March-Westin was the successful bidder on the original design, but after receiving the contract they worked with Allegheny Design Services and AISC-member fabricator Contracting Engineering Consultants (CEC) to investigate the steel option. By early November, they submitted a proposal for the steel-framed option that gave the owner a 5% total project savings. The steel option also lowered the risk of schedule overruns with winter approaching and a December 1, 2003 start-of-construction date. The owner accepted, and March-Westin, Allegheny and CEC worked as a team to design and build the project.

"We like to bring the fabricator and the engineer on at the same time," said Jamie Ridgeway, project manager for March-Westin. "We do it all the time on projects, because it can save time and money."

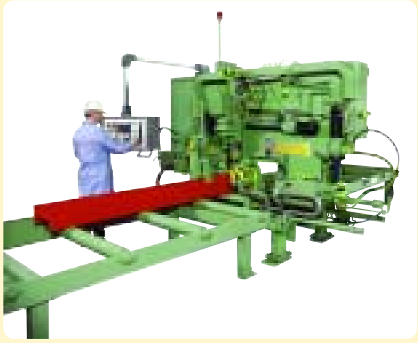
The transition from concrete to steel went smoothly, said structural engineer David R. Simpson, P.E. "We worked directly for March-Westin while maintaining coordination with HOK and Thornton-Tomasetti. To complete the design in a three-month period, we had to feed information to the fabricator as we were going along so that they could meet the mill schedules to order steel—while maintaining communication with HOK and their architectural requirements—and while communicating with Thornton-Tomasetti to determine what

Six Secrets of Success at WVU

1. The contractor's experience in construction methods and materials equaled an opportunity for cost savings on a tight schedule through the use of steel.
2. The owner was willing to entertain design options.
3. Working relationships that were cultivated on past projects helped the contractor assemble a unified team with design and construction expertise.
4. Excellent communication between the architect, engineers, and contractors facilitated the redesign process.
5. Excellent communication between the engineer, detailer and fabricator helped expedite the detailing and drawing-approval process.
6. The fabricator's use of technology helped move seamlessly between detailing software, production control software and CNC-controlled equipment. This saved weeks on the schedule and greatly reduced the chance of shop errors.

Machinery Makes a Difference

The Peddinghaus Beam Drill Line BDL 760 used on this project is a compact structural drilling machine suited for short or long production runs of sections 30" wide or less. It is a computer-numerically controlled (CNC) machine—and fabrication information could be directly imported into the machine from Xsteel, the detailing software program used for the WVU project. The combined use of Xsteel and the BDL 760 shaved as many as three weeks from the project schedule. "The Peddinghaus machine is pretty impressive," said Tanner Johnson, project detailer for fabricator Contracting Engineering Consultants. "Depending on the difficulty of the pieces that we have to run, and how much work gets done to them, with some projects we can squeeze in as many as 45 tons a day. It'll chew up beams and columns as fast as you can keep it running. It's a great piece of machinery." The drill line's carbide marking system, Signoscript, is used for fast part identification.



their foundations could and could not take." Simpson also worked carefully with HOK to make sure that frame layout and geometry would work well with fascia and curtain-wall attachments.

Allegheny had worked with the contractor and the fabricator on many projects before, some of which were design-build projects. "Because of our relationship with the builder and the fabricator, we were able to provide a quick response," Simpson said. "A lot of barriers that we might have faced were already out of the way."

Tanner Johnson, CEC's project detailer, gathered information daily from Allegheny Design Services and detailed areas using Tekla Xsteel as soon as sections and layout were available.

"The fabricator is two miles down the road from us," Simpson said. "So sometimes we were able to sit at their desk as they were doing their detailing."

Johnson agreed that the proximity and trust between team members facilitated the quick design and detailing process. "We've worked together a lot before, and our companies have a good flow of information when we work together. Even if there were RFIs, there was a quick response, usually just a phone call away. With other engineers, it might take days or weeks to send an RFI out and receive a response, which can make the project excruciating. We were on the phone daily with each other—we all have the same cell phone service, so it's as if we were on walkie-talkies."

Working closely with the engineer and general contractor also meant that the team members could consider each other's input in the design, fabrication and construction process. "One of the greatest assets of the relationship between CEC, Allegheny Design, and the general contractor is that everyone is open to suggestions that will make the project faster to build, quicker to ship, or easier to erect," Johnson said. "As fast as we're doing this project, it's good to have a lot of trust. There's no one pointing fingers. There's no haggling back and forth about who owes what. Everyone's focused on the main goal."

Johnson started detailing December 2, 2003. Working closely with Simpson on submittals and approvals, he was able to complete the detailing and approval process by January 22, 2004. As soon as material lengths were available, steel was ordered from various steel service centers, including AISC-members Infra-Metals (Baltimore) and Metals USA (Pittsburgh). Fabrication began a few days later.

Using the Xsteel software, Johnson converted the connection information to computer-numerically controlled (CNC) files that were sent directly to a Peddinghaus BDL 760 drill line—which he says saved up to three weeks on the delivery schedule. "What one person can do in two days with Xsteel would take a person two or three weeks to program by hand. It's quite a lifesaver. Plus, it eliminates most human error. As compared to a job where you would have to hand-program each and every file—Xsteel is seamless."

Johnson also used FabTrol fabrication and project management software to manage pieces, layouts and parts necessary for the project. "FabTrol manages our ordering, shipping and receiving. It keeps track of our specs, and just about anything else you'd want to do with it," he said. "We also use Master Builder, an accounting software, to keeps track of our billing."

The first load of material was shipped to the job site on March 1, three weeks ahead of foundation completion.

Steel Stadium

The steel-framed expansion used W-shape columns and beams with knee-braced, back–to-back double angles for lateral stability. The roof is a steel-beam and metal-deck system, and the floor is composed of 8" hollow-core pre-cast concrete with structural stopping. The second level of the facility houses the private luxury suites with general seating and concessions on the lower levels. All suites are enclosed with motorized windows that rise for warm weather games. The motorized windows required tight construction tolerances. Close attention and coordination had to be paid to the fascia and curtain-wall attachments due to the change in structural frames.

A cantilever, primarily composed of a W27×94 with an end-plate connection to the W27×94 columns, accommodates the allowable deflection limits imposed by the curtain-wall system and a motorized

Project Timeline

October 2003: March-Westin wins the bid for the project; organizes a team to consider design in structural steel.

November, 2003: March-Westin submits a proposal for the steel-framed option that offers the owner a 5% total project savings.

December 1, 2003: Construction begins.

December 2, 2003: Structural steel detailing begins.

January 22, 2004: Detailing and shop-drawing approval process is complete.

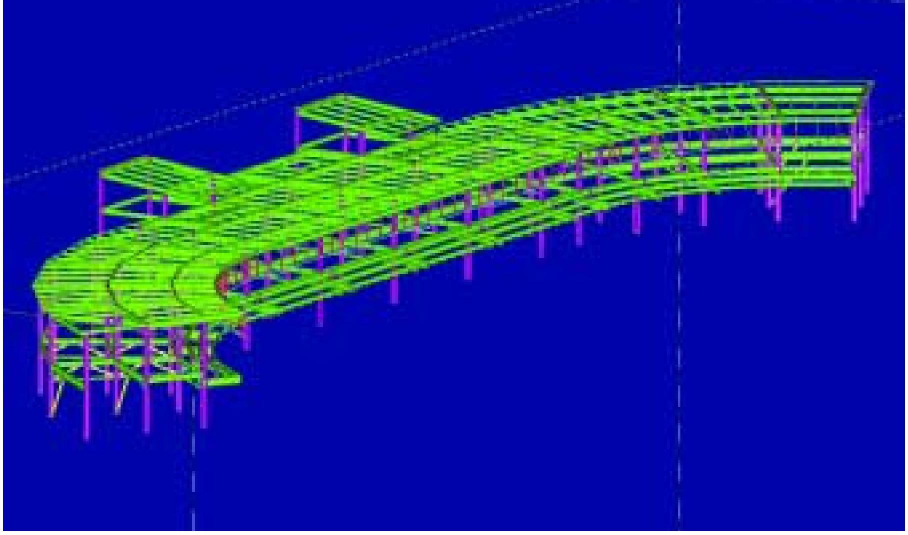
March 1, 2004: The first load of structural steel is shipped to the job site.

March 22, 2004: Foundations are complete.

May 20, 2004: Structural steel erection complete.

September 4, 2004: Kick-off!

Tekla Xsteel software was used to created CNC files for use by the fabrication equipment. FabTrol fabrication and project management software helped out in the shop.



window-operating system. The driving frequencies of excited football fans in the stadium also were taken into consideration. For the cantilevers, Simpson designed a bolted moment connection with 1½″ ASTM A325N bolts, for ease of erection.

"As opposed to concrete, the appearance as you're walking underneath the cantilevered sections is really impressive," Johnson said. "Especially to see those W27×94 beams that are cantilevered out with bolts on them."

An error in the soils report was discovered, and the expected differential settlement was increased from 1″ to 2″-3″. The solution was to temporarily brace the frame with cables and install the angle knee bracing after most of the dead loads were applied. Under temporary bracing, the frames were allowed to rotate at the beam-column connections during settlement. After an approximately 1.5″ settlement and no sign of significant further settlement, the knee braces were secured, thus greatly reducing any residual stresses.

At the ends of the building, where it joins the existing stadium, considerations were made to provide underpinning to the existing stadium structure. Due to poor soil conditions, it was not advisable or economical to use soil nailing for this area. This led to the use of the superstructure as support for the 20′-high retaining walls. HSS 8×8s were used to direct the top-of-wall reactions into the Thornton-Tomasetti-designed foundations.

The project contained 325 tons of structural steel members. A spray-applied fire-protective vermiculite coating was used on the interior structural members of the steel frame. The exterior steel was coated with a Tnemec SSPC-SP6 system, with zinc-rich shop-applied primer and a two-coat system of field-applied paint.

Gene Martin, P.E., is a regional engineer for the Upper Midwest Region for AISC Marketing, LLC. David R. Simpson, P.E., is president of Allegheny Design Services.

Owner
West Virginia University, Morgantown, WV

General Contractor
March-Westin Company, Inc., Morgantown, WV

Architect
HOK, Inc., Kansas City, MO

Structural Engineer
Thornton-Tomasetti Engineers, New York City (Original Concrete Design)

Allegheny Design Services, Star City, WV (For the Steel Design Option)

Steel Fabricator/Detailer
Contracting Engineering Consultants (CEC) (AISC member), Maidsville, WV

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RAM Structural System

Detailing Software
Tekla Xsteel

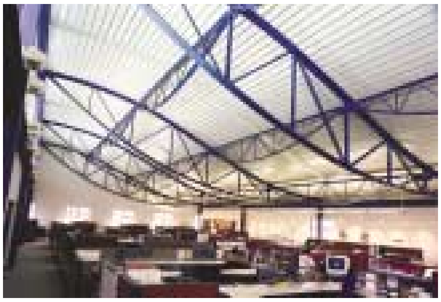
Project Management Software
FabTrol, Intuit Master Builder

Coating Supplier
Tnemec

Steel Joists

Canam Steel Corporation

4010 Clay Street
P.O. Box C-285
Point of Rocks, MD 21777
Ph: 800.638.4293 or 301.874.5141
Fax: 301.874.5685
John.love@canamsteel.com
www.canamsteel.com



Canam Steel produces specialty trusses, open-web steel joists and extra-long joists. Along with the open-web steel joists, Canam specializes in composite joists, super-long spans and specialty trusses such as barrel, bowstring, gable and scissor trusses. Canam Steel has 13 plants in North America and a total production capacity of 524,000 tons. Canam's welders, inspectors and quality assurance technicians are AWS or CWB certified. To locate the nearest Canam office, call or visit the web site to download Canam's joist catalog.

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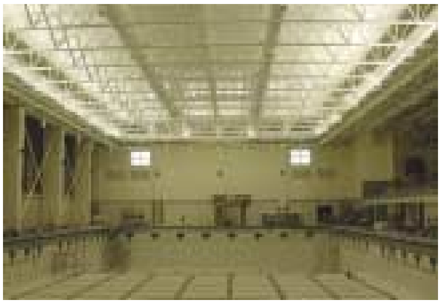
25 Deforest Ave.
Summit, New Jersey, 07901
Ph: 908.277.1617
Fax: 908.277.1619
njbsales@bourasind.com
www.njb-united.com



Nicholas J. Bouras, Inc. is the sales, engineering and detailing subsidiary of Bouras Industries Inc., a manufacturer of steel construction products, with plants and sales offices in 17 states. The New Columbia Joist Co., Bouras' joist-manufacturing subsidiary, produces Steel Joist Institute-specified K, LH, DLH series joists, joist girders and specialty trusses. For more information or to locate your local sales representative, call or visit the web site.

Nucor Vulcraft Group

P.O. Box 637
Brigham City, UT 84302
Ph: 435.734.9433
Fax: 435.723.5423
roy@vulcraft-ut.com

P.O. Box 280
Chemung, NY 14825
Ph: 607.529.9000
Fax: 607.529.9001
sfitts@vulcraft-ny.com (Vulcraft of NY, Inc.)

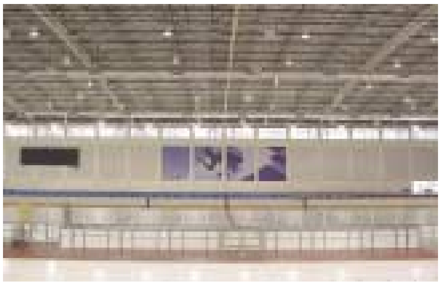
P.O. Box 100520
Florence, SC 29501
Ph: 843.662.0381
Fax: 843.662.3132
asears@vulcraft-sc.com

P.O. Box 680169
Fort Payne, AL 35968-0169
Ph: 256.845.2460
Fax: 256.845.1090
kgeesaman@vulcraft-al.com

P.O. Box 186
Grapeland, TX 75844
Ph: 936.687.4665
Fax: 936.687.3168
grapelandsales@vulcraft-tx.com

P.O. Box 59
Norfolk, NE 68702
Ph: 402.644.8500
Fax: 402.644.8528
sales@vulcraft-ne.com

P.O. Box 1000
St. Joe, IN 46785
Ph: 260.337.1800
Fax: 260.337.1801
sales@vulcraft-in.com
www.vulcraft.com



Nucor's Vulcraft Group produces steel joists, joist girders and steel deck. Vulcraft also produces highly engineered products, such as composite floor joist. Vulcraft supplies products for a range of structures: from high-rise office buildings and large industrial complexes, to pump house buildings and schools. A variety of joist and joist-girder configurations are available for architectural consideration, including arched chord, bowstring, scissor and single- and double-pitched designs. Vulcraft products, made from more than 90% recycled materials, have been part of green buildings. They support venues such as the Olympic Speed Skating Oval in Utah, home of "The Fastest Ice on Earth," and the second-largest skylight in the nation at the Opryland Hotel in Nashville, TN.

Quincy Joist Company

520 S. Virginia Street
Quincy, FL 32351
Ph: 800.277.1075
Fax: 850.875.1277

22253 W. Southern Avenue
Buckeye, AZ 85326
Ph: 623.386.1900
Fax: 623.386.1888
www.sales@quincyjoist.com
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Quincy Joist Company manufactures open-web steel joists and joists girders. Quincy Joist also produces engineered and customized products, such as composite floor joists, bowstring, arched and scissor joists. Engineering capabilities and production capacity permit timely fabrication and delivery of projects of all sizes and complex engineering challenges. One example is St. Timothy's Catholic Church in Hillsborough County, FL. The design required two domes, one spanning 64' with a radius of 45' and the other spanning 124' with a radius of 105'. The use of joists saved the project almost $200,000 and cut four weeks out of the schedule.

Steel Joist Institute

3127 10th Ave. North Ext.
Myrtle Beach, SC 29577-6760
Ph: 843.626.1995
Fax: 843.626.5565
rhackworth@steeljoist.org
www.steeljoist.org

The Steel Joist Institute offers the new 75-Year Manual with a complete listing of commonly used live and dead loads throughout the 75-year period since 1928. It offers step-by-step investigative procedures and time-saving data for use when analyzing exiting structures as well as building-site information and equipment needed. This manual includes a complete chronological listing of all Institute specification and load tables. A new section features examples for calculating moment of inertia of older joists.

Steel Joists

SMI Joists

2305 Ridge Road
Rockwall, TX 75087
Ph: 800.308.9925
Fax: 800.308.9339
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Valley Joist, Inc.

3019 Gault Ave. North
Fort Payne, AL 35968
Ph: 800.633.2258
Fax: 256.845.2597
dmackey@valleyjoist.com
www.valleyjoist.com

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By Roumen V. Mladjov, S.E.

The Most Expensive Bridge in the World

This commentary is a response to the "Notes from the Editor" in the June 2004 issue of *Modern Steel Construction*. Recent information for the continuing increase in the cost of the new San Francisco-Oakland Bay Bridge East Span has reopened the discussion of this already controversial project. Vital for everyday commutes, this bridge is the main highway link in the San Francisco Bay Area, and its retrofit or replacement is long overdue.

The California Department of Transportation (Caltrans) made this replacement the largest and most expensive bridge project ever undertaken in California. Caltrans tries to explain the cost increase from $1.4 billion at project selection in 1998 to the current estimate of $4.0 billion with the uniqueness of the suspension bridge section and the recent increase in the cost of steel.

A more careful analysis and comparison with other large bridge projects shows that, unfortunately, the only thing extraordinary about this bridge is its cost. The new East Span has two distinctive sections, one being a suspended structure with 385-m span and other a "Skyway" with multiple concrete 160-m spans. The costs for both sections significantly exceed the cost estimate presented in 1998, which could indicate inefficient design and project management.

Comparing the cost of the new bridge against others, one can see that almost all super-long span bridges were built for significantly less than the new Bay Bridge. The increase in the cost of steel could account for about $120-150 per ton structure, but it can in no way explain $30,000 per ton structure.

Caltrans must respond to many questions related to the bridge design and project management, and in spite of the current late state in the project development there are still opportunities for saving a lot of California taxpayers' money.

—R.V.M.

Piers for the replacement of the almost 70-year-old East Span already were appearing above the water when discussion of the project started anew on the pages of technical magazines. Discussion was spurred by the updated cost estimate for the entire new span, published by the California Department of Transportation (Caltrans), based on the bid for the suspension bridge portion (*San Francisco Chronicle*, 5/27/04). The new cost estimate comes to $4 billion, which is almost three times Caltrans' original estimate of $1.4 billion at the final design selection in 1998.

According to Caltrans, the increase in cost is due to the unique design of the suspension portion of the bridge and the recent increase in the cost of steel. Not mentioned in this explanation, the concrete "Skyway," about 84% of the length of the bridge, also significantly exceeds the early cost estimates.

It is therefore important to reconsider major parts of this controversial project.

During the 1989 Loma Prieta Earthquake, the East Span of the Bay Bridge sustained relatively little damage. The bridge was quickly repaired, but the event triggered a long overdue study of the structure's capability to resist strong earthquakes. The bridge is located about 12 km from the San Andreas Fault and less than 10 km from the Hayward Fault, each of them capable of producing earthquakes with magnitude exceeding 7.0.

The San Francisco-Oakland Bay Bridge is one of the busiest crossings on the West Coast. When built in 1936, the bridge was the longest in the world (4.36 miles). It also had the deepest foundation and the third-longest cantilever span ever built. The tunnel on Yerba Buena Island divides the Bay Bridge in two. The West Span suspension structures connect the island with San Francisco, and the East Span connects it with Oakland. Built in just 3½ years during the Great Depression, the Bay Bridge is one of the engineering marvels of the 20th century. The double-decker bridge has five vehicle lanes on each deck and 280,000 vehicles use it daily. It is considered the "workhorse" of the Northern California transportation system.

Bridge Replacement and Renovation

Controversy plagued the replacement of the East Span from the beginning. The public expected a fast decision for fixing the bridge, but following the Loma Prieta Earthquake, it took Caltrans seven years to complete an engineering study and to publish recommendations. The recommendations called for reinforcing the suspended portion (the West Span) and for replacing the East Span. The higher cost of reinforcing the East Span, and the significant traffic difficulties of such a reconstruction, motivated Caltrans' recommendation to replace the East Span. On this point, most specialists have concurred.

The public disliked Caltrans' initial proposals for replacing the East Span. Subsequently, the Metropolitan Transportation Commission (MTC) created a Design Task Force and an Engineering and Design Advisory Panel (EDAP) to review alternatives and to advise MTC. EDAP accepted the concept to retrofit the West Span and to replace the East Span but rejected Caltrans' and other hastily prepared proposals. A national or even international competition for the new bridge was then proposed.

Ignoring this, Caltrans hired T.Y. Lin International and Moffat & Nichol to design the new East Span. Despite critical comments and objections during the EDAP review, and explicit concerns that no valuable solutions had been presented, MTC approved a final design concept in 1998.

The final design calls for a suspended "self-anchored" bridge just east of Yerba Buena Island, attached to the so-called "Skyway," a concrete multi-span portion with 160 meter-bays. This suspended bridge is asymmetric, with one main (385 m) span and one side (180 m) span. Unlike the existing double-decked Bay Bridge, the replacement East Span will be built with two parallel single-level decks. The cost of the replacement bridge was the most controversial issue at the early stages. By not providing construction quantities and therefore a reliable cost estimate, Caltrans and its design team repeatedly resisted any attempts by EDAP to challenge the efficiency of the final design.

The cost estimate for the entire East Span project presented by the Design team and Caltrans in 1998 totalled $1.4 billion. Some of EDAP's members were already arguing that this cost was too high in comparison to similar bridges built in the previous years both in the United States and overseas. In spite of the conflict of interest, Caltrans had the opportunity to play the combined roles of Owner, General Contractor and Supervisor of the whole process by setting the bridge systems, the budget and the construction time. Not surprisingly, the cost escalated, growing to $2.4 billion in 2001 and reaching $4 billion in June 2004—while the projected construction time continued to be extended.

Time alone can serve as the final judge of the engineering accomplishment. However, we evaluate a bridge achievement by length of the major span, the quantity of material and the cost used per unit deck.

Bay Bridge Statistics	Existing Span	Replacement/New Span
Total Length (feet)	11,287	11,525
Main Span (feet)	1,400	1,263
Traffic Lanes	10	10
Vehicles per day (average)	280,000	300,000+
Construction Time (years)	3.5	5.0?
Completed/ Estimate Completion	1936	2008?
Total steel for superstructure (tons)	167,100	104,200 (with concrete for 84% of the bridge)
Steel (psf)	119	289
Cost (in million dollars)	78	3,600+

Uniqueness of structure…?

Length of main span. The longer the bridge, the greater the achievement. The longest span of the new bridge is 385 m, a relatively modest span next to the 1930s Golden Gate Bridge (main span of 1280 m) and the West Span of the Bay Bridge (main span of 704 m). It is even shorter than the 427 m main span of the East Span that is to be replaced. Today there are hundreds of bridges around the world with longer spans. The 160-m continuous "Skyway" structure also does not present anything that qualifies for uniqueness.

Structural System. A self-anchored bridge, which is a hybrid between a suspension and a cable-stayed bridge, is considered a novelty. However, the fact that few such bridges are built should have made the designers more careful in their system selection. The "Skyway" is a segmental concrete continuous system—the most commonly used system for concrete highway bridges with spans between 80 m and 200 m.

Efficiency of material use. For both the suspension and the concrete "Skyway," the quantity of material use exceeds almost all contemporary bridges, including those with spans several times longer.

Cost Efficiency. The cost per square meter and the cost efficiency factor exceed significantly almost all other bridges.

It is clear that nothing is unique in this bridge. Both sections are designed with significantly more material (steel and/or concrete) and for a higher cost than almost all long-span bridges in the world. The location, near active seismic faults and deep water, does not provide a valid excuse. Many bridges are built in similar or more violent seismic areas than the San Francisco Bay and in even deeper water. These include the Golden Gate Bridge, the existing Bay Bridge, the Akashi-Kaikyo, the "25 of April" Bridge at Lisbon, and the Gulf of Corinth Bridge.

The main span for the suspension bridge of the East Span is modest when compared to other bridges, and therefore the steel and cost should have been proportionally lower. In light of these considerations, the new East Span will be unique only its excessive quantity of steel and its excessive cost.

The increase in cost of steel?

Can the recent increases in the cost of steel material explain the extremely high unit cost for the East Span? The cost of "raw" structural steel products has increased by 22% from 2003 to May 2004. However, the cost difference is $122 (rising from $567 to $689) per ton. The cost of the steel material, plates, wide-flange beams and others do not have an impact on production cost, welding, labor, assembling, transportation, erection and finish work. Thus, the significant increase of steel material can explain only an increase in the range of $120 to $150

Material	Unit	2001	2002	2003	March, 2004	May, 2004	May, 2004/ 2001	May, 2004/ 2003
		$	$	$	$	$	%	%
Reinforcing Bars	cwt	24.60	24.00	24.52	30.28	34.20	39.0	39.5
Structural Shapes Avg.	cwt	26.57	27.00	25.71	29.04	31.27	17.7	21.6
Reinforcing Bars	ton	492.00	480.00	490.40	605.60	684.00	39.0	39.5
Structural Shapes Avg.	ton	531.40	540.00	514.20	580.80	625.40	17.7	21.6
Reinforcing Bars	metric ton	542.33	529.10	540.56	667.55	753.97	39.0	39.5
Structural Shapes Avg.	metric ton	585.76	595.24	566.80	640.22	689.38	17.7	21.6
							Cost Difference	
Reinforcing Bars	metric ton	542.33	529.10	540.56	667.55	753.97	211.64	213.40
Structural Shapes Avg.	metric ton	585.76	595.24	566.80	640.22	689.38	103.62	122.58

The costs listed in table above are based on McGrow-Hill, *Construction Economics*, *ENR*—given originally in $/cwt or $/100 lbs; 1 ton = 2000 lbs; 1 ton = 20 cwt; 1 metric ton = 1000 kg; 1 metric ton = 2204.6 lbs.

Bridge	Year	L max	Total Steel	Steel	Steel Efficiency	Cost	Cost	Cost Efficiency	Cost Str. Steel
		m	Tons	kg/m²	kg/m³	$ million	$/m²	$/m³	$/ton
SUSPENSION BRIDGES									
Akashi-Kaikyo, Japan	1998	**1990**	193300	**1648**	**0.83**	4300	**36658**	**18**	**22245**
Great Belt East, Denmark	1997	**1624**				950	**13061**	**8**	
Humber Estuary, UK	1981	1410		451	0.32				
Verrazano Narrows, US	1964	1298		584	0.45				
Golden Gate, US	1937	1280	66040	1226	0.96	35	non comparable		non comparable
Hoga Kusten, Sweden	1997	1210	14000	437	0.36	125	3901		8929
Bosphorus First, Turkey	1973	1074		365	0.34				
April 25th, Portugal	1966	1013		476	0.47				
Kwang Ahn, South Korea	2000	500	29710	688	1.38	196	4537	9	6597
Yongjong, South Korea	2000	**300**	35100	**1140**	**3.80**	270	**8766**	**29**	**7692**
(Self-anchored)									
CABLE-STAYED BRIDGES									
Tatara, Japan	1999	890				606	15165	17	
Gulf of Corinth, Greece	2004	**560**				740	**9957**	**18**	
Fred Hartman, US	1996	381	5043	141	0.37	91.3	2549	7	**18104**
Existing Bay Bridge, US	1936	704	151593	579	0.82	78	non comparable		non comparable
Existing Bay Bridge, US	1936	427	151593	579	1.36	78	non comparable		non comparable
Bay Bridge East Span	2008	**385**	43420	**1410**	**3.66**	1575	**51148**	**133**	**36274**
Bay Bridge (ES) Skyway	2008	**160**	*60780	***573**	***3.58**	1500	**14140**	**88**	not applicable

Notes:
1. Bay Bridge East Span is the San Francisco - Oakland Bay Bridge
2. **Steel Efficiency** and **Cost Efficiency** are accounting the length of the main span and are equal to the Steel/m² and Cost/m² divided by the length of main span
3. Bay Bridge (ES) Skyway: *573, *3.58 and *60,780 tons: these values are for reinforcing steel of superstructure.
4. ES indicates East Span
5. The Structural Steel cost is total cost of steel bridge including foundations, temporary structures, etc.

per-ton completed structure, or 1% of $15,000 per ton. While the East Span unit cost is around $32,000 per ton (the highest of all bridges), the increase in the cost of steel represents less than 1% of the total cost and therefore can hardly be accepted as an excuse for the most recent Caltrans estimate.

The "Skyway" Bridge

The mostly concrete "Skyway" section uses 623 kg/m² reinforcing steel for the superstructure alone. If designed with steel, the same bridge would use about 420 kg/m² with orthotropic steel deck or about 260 kg/m² with composite concrete deck. Both steel options could have saved tens of thousand of tons of steel (and dollars), without even considering the cost of concrete. The considerable saving of dead weight could have simplified and reduced foundations and substructure—and saved an additional good percentage of the cost.

The total cost of the "Skyway" will probably exceed $1.5 billion, because the bid for its superstructure alone is $1.1 billion. This also means that, by itself, the "Skyway" will exceed the 1998 estimate projected by Caltrans and the design team for the entire new East Span.

It appears that the new Bay Bridge East Span will be the most expensive bridge ever built, for years to come. While a bridge's main function is to transport people and vehicles in the most efficient way, the current exorbitantly expensive plan for replacing the East Span of the already-congested Bay Bridge does not envision the addition of even one extra traffic lane. Uniqueness should not be a self-serving goal if it fails to deliver higher performance.

These are questions that should be addressed by MTC, Caltrans and the design team, and the answers should be supported by engineering data. One might think this is merely an academic exercise, as the bridge is already under construction. However, it is very important to understand why the bridge is so expensive so the same mistakes are not repeated. Moreover, the bridge has two separate parallel decks and it is not too late to revise the project and replace at least the extremely expensive "Skyway" with one deck. The suspension portion also can be redesigned. Caltrans has protracted construction time for years. There still is the chance that a better-designed option will overcome the delays—and maybe even save some time—and still reduce the cost significantly. ★

Roumen V. Mladjov, S.E. is a structural engineer and served as member of EDAP during 1997 and 1998. He also served as Chair of the Seismology Committee of SEAONC 1996-1997 and is a member of the International Association for Bridge and Structural Engineering. The opinions included in this article are representative of the author alone and do not represent those of his engineering firm, Modern Steel Construction *magazine, the American Institute of Steel Construction, Inc., or any of its affiliated organizations.*

New Seismic Moment Frame Joint

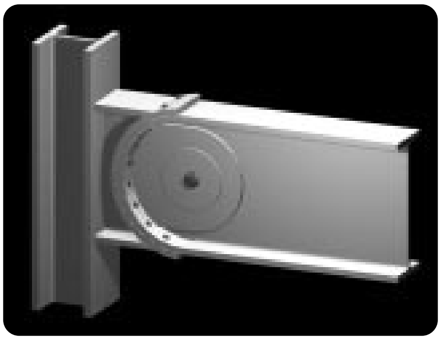
The article in the July *Modern Steel Construction* about the new moment frame joint developed by Mark Sarkisian of SOM was very interesting. However, there were two points in the article that were misleading and perhaps confusing to readers. The article says that, "When the earthquake is over, the Pin-Fuse Joints return to their usual positions, with the torque in the bolts maintained."

There is no mention in the article that the joint has any restoring force mechanism. If there is none, the joint will remain distorted at the angle at which it was at the time the seismic forces got small enough to be unable to slip the bolts. Additionally, it is unclear why builders could "rely on smaller frame members than otherwise would be required to withstand an extreme seismic event" since moment frame member sizing is invariably governed by stiffness (building drift) considerations which would ostensibly be only slightly changed by this joint. Notwithstanding these comments, it is an interesting invention that would appear to make post-earthquake repair simpler than for conventional joints.

C. Mark Saunders, President
Rutherford & Chekene Consulting Engineers
Oakland, CA

The following is a response to Mr. Saunders's comments:

The restoring force mechanism of the Pin-Fuse Joint relates directly to the overall behavior of the frame. The beams and columns are designed to remain essentially elastic, with the Pin-Fuse Joint controlling joint



slip to tolerable limits. Some beam-to-column joints within the frame may be designed without the fused joint so they can provide an elastic restoring force to return the frame to its original pre-seismic position. Moreover, as not all of the Pin-Fuse Joints are likely to slip during a seismic event, they will provide additional restoring stiffness. If under some conditions the frame does not complete re-align, it would be possible to loosen the bolts at any critical connections locked in an off-center position allowing the frame to re-position itself.

Stiffness (building drift) of the frame certainly may control final member sizing. However, using a performance-based design approach, the overall ductility and deformation demands of the system will be confirmed with studies compared to current code requirements. The expectation is that increased (and more reliable) joint ductility will lead to reduced structural demands and smaller member sizes.

Studies are presently underway to investigate both of these important design considerations. Thank you for your thoughtful comments.

Mark Sarkisian, P.E., S.E., Partner
Skidmore, Owings & Merrill LLP
San Francisco, CA

2004-2005 AISC Educator Career Enhancement Awards

AISC invites you to participate in a new award program designed to provide career development opportunities to help educators succeed at their university. AISC supports the development and enhancement of educators dedicated to the teaching of steel-related topics. The award is intended to foster creativity and promote the implementation of new ideas in teaching related to structural steel. Applicants must submit a proposal for how the award would be used to extend or create programs that enhance teaching activities related to structural steel.

The award recipient will receive a $20,000 grant, national recognition, and present his/her project at the Educator Session of an upcoming North American Steel Construction Conference. For more information or to enter, contact Fromy Rosenberg, AISC's director of university relations at 312.670.5408 or rosenberg@aisc.org. ★

AISC 2004 Fall Seminar Series

AISC'S fall 2004 seminar series is coming to a city near you! AISC offers cutting-edge continuing education courses at reasonable prices. And with the "Bring a Buddy" program, each paid registrant can bring one colleague or friend along for only $100 extra! AISC seminars are an excellent opportunity to learn about everything from basic field fixes to advanced seismic design with structural steel. This fall, AISC offers five continuing education seminars in cities across the nation through December 2004. Don't miss the chance to hear from the expert—-and bring your friends too!

For more information on dates, locations, and accommodations, and to register, please visit **www.aisc.org/seminars**, or contact Carol Pivonka, director of continuing education at 406.652.9787 or pivonka@aisc.org.

Seismic Design and 2002 AISC Seismic Provisions
This seminar focuses on changes to the Seismic Provisions and provides designers with practical knowledge on how to apply these changes to their current and future projects.

Basic Design for Stability—Columns and Frames
This six-hour program provides a practical understanding of the stability provisions in the AISC *Specification*. With design examples and case studies, the program gives designers a fundamental understanding of buckling. The course focuses on the compressive strength of columns and frames.

Field Fixes: Common Problems in Design, Fabrication and Erection-Solutions and Prevention
This five-hour seminar covers a wide range of topics, including material specifications, connection design requirements, standard details, sizing material for constructability, use of mill reports, perimeter details for tiltup and pre-cast concrete walls, design procedures for fast-track construction, electronic data transfer, and shop-drawing approval procedures.

Botling and Welding
This is for those who want to learn more about bolted and welded connection design. You will leave the seminar better equipped to solve bolting and welding problems and learn cost- and time-saving ideas on bolting and welding, which will permit you to put these ideas into practice and quickly reap the benefits of this course.

Fire, Blast, and Progressive Collapse
This program focuses on the prescriptive procedures included in model building codes for the determination of fire-protection requirements and provides an introduction to the evaluation of structural steel capacities at elevated temperatures. Included in the seminar will be a segment on blast and progressive-collapse resistance in steel. You will learn about three basic levels of approach-strategies for typical buildings, prescriptive approaches, and performance-based approaches. You will also be given engineering guidance for each approach. ★

Second Public Review of the 2005 AISC Specification

The AISC *Specification for Structural Steel Buildings* is now available for a second public review. The first public review was held in December 2003. The purpose of the second public review will be to consider additional technical revisions that have occurred in the document. The draft specification is available for free downloading on the AISC web site at **www.aisc.org**. Copies are also available (for a nominal $12 fee) by calling 312.670.5411.

To submit comments, please contact Cynthia J. Duncan, director of specifications, at duncan@aisc.org and request a public review form or download the form on the AISC web site. Comments must be received by September 20, 2004 for consideration. ★

Call for Papers: 2005 NASCC

Designers, contractors, detailers, erectors, and educators are invited to submit proposals for the poster session at the 2005 North American Steel Construction Conference. All poster presenters will receive free registration to the NASCC. The 2005 Conference will be held April 6-9 in Montreal, Canada. Abstracts are due October 1, 2004; accepted presenters will be notified by November 15.

Presentations should provide practical information on state-of-the-art design or construction methods, or present information on the relevant research activities. Final presentations should be mounted on 24"x36" boards (maximum size), which will be displayed in the main exhibit hall at the conference. Presenters should be available to discuss their presentations April 6 (5-6 p.m.), April 7 (noon-1 p.m.), and April 8 (noon-1 p.m.). Please submit your abstract by October 1, 2004 to:

Scott Melnick
AISC, Inc.
One East Wacker Dr., Suite 3100
Chicago, IL 60601
melnick@aisc.org ★

OSHA Joist Erection Update

The OSHA enforcement policy with respect to the column joist requirements in section 1926.757(a)(3) that had been extended to July 18, 2004, has now been extended indefinitely (ref OSHA CPL 02-01-040).

Section 1926.757(a)(3) requires that "where steel joists at or near columns span 60 feet or less, the joist shall be designed with sufficient strength to allow one employee to release the hoisting cable without the need for erection bridging."

There are many standard joists that are unable to meet this requirement either due to their span or due to their configuration, such as sloped or pitched chords. In addition, Steel Joist Institute research has determined that for any joist to meet the requirement, the erector would need to follow anchorage instructions that are more detailed than what is in the current OSHA standard.

Therefore, the enforcement policy that was scheduled to expire on July 18, 2004, will remain in effect indefinitely. That policy is as follows: for all joists at or near columns that span 60 feet or less, employers will be considered to be in compliance with 1926.757(a)(3) if they erect these joists either by: (1) installing bridging or otherwise stabilizing the joist prior to releasing the hoisting cable, or (2) releasing the cable without having a worker on the joists.

Steel joist manufacturers will continue to expect erectors to follow that policy. ★

Keeping Tabs on Structural Health Via the Internet

When the New York City Department of Transportation decided to launch a stiffening campaign on the Manhattan Bridge after observing movements between the suspension cables and the vertical truss members, it needed to monitor the structural behavior of the bridge while the work was being completed.

The solution was brought in by OSMOS, a French company that, for more than 15 years, has been developing monitoring systems for safety surveillance and fiber-optic technology for structural health monitoring (SHM). Its sensor-aided system for monitoring industrial installations, bridges, historic buildings, hydraulic engineering and other structures is used by building contractors and owners, operators, structural engineers, architects, and inspection authorities around the world. Users can monitor both short- and long-term structural changes. It provides ongoing risk surveillance, quality assurance during construction work and early detection of defects.

The OSMOS system is used to continually measure static and dynamic structural loads and stress cycles and the resulting effects on material and buildings. Changes in shape and position can be monitored, making it possible to forecast the behavior of the structure being inspected.

The complete system consists of a monitoring station that captures, transmits and records the incoming data—without any amplification or transformation—from sensors installed in or on the structure to be inspected. This station contains a master unit that performs all the necessary network functions and a slave unit that carries out data measurement. Measured values can be displayed on X-Y graphs, polar graphs, bar charts or as dashboards (threshold values with alarm levels).

In additon, all data can be saved, evaluated and displayed in real time, and information can be communicated to users by phone, fax, e-mail, the Internet, SMS (cell phone Short Message Service) or SNMP (Simple Network Management Protocol). Data can also be exported for further processing and managed directly via an Internet browser. OSMOS has developed specific fiber-optic sensors that reach the level of measurement of the micron and have a 20-year warranty. The system, however, is universal and can integrate all other sensors available on the market. The station will capture all data.

Currently, the company monitors 475 sites on a permanent basis. The OSMOS system has been used to monitor the Eiffel Tower since 1993 and the Nagoya power plant in Japan. The system also is being used to observe the Federal Hall structure in New York during geotechnical work in the immediate vicinity. Because the monitoring system was installed and operational for several months before the geotechnical work began, OSMOS was able to gather the data necessary to understand

the structure's behavior in "neutral" circumstances. This reference state will serve as a basis for the interpretation of any possible changes in its behavior during the geotechnical work.

Lastly, American Express used the system to monitor its 53-story building following heavy structural damage sustained by the collapse of the adjacent World Trade Center September 11, 2001. OSMOS installed one temperature sensor, one monitoring statio, and four optical strands on corner-bay spandrel beams on the 25th and 26th floors to detect both east-west and north-south movements. This instrumentation enabled American Express to evaluate the structure's behavior to determine immediate safety concerns and, from a long-term perspective, to produce reliable and precise data as a basis for a repair and rebuilding program.

For more information, visit www.osmos-group.com or call 312.327.5260. ★

Emergency Steel Bridge Goes Up in Four Days

On July 12, 2004, Burlington County, NJ was devastated with a 1,000-year storm. More than 13" of rain fell within a 10-hour time frame in some areas. Twelve dams burst, resulting in major flooding throughout the area and the closing of more than 25 roads and bridges. Friendship Creek in Southampton surged and the 90′-long Route 70 Bridge over the creek collapsed as the churning waters undermined its foundations.

The next morning, Matt Hummel, project engineer for AISC-member Acrow Corporation, an international bridge engineering and supply company based in Carlstadt, NJ, visited the bridge site. He promptly met with New Jersey Department of Transportation (NJDOT) officials and gave an assessment of the situation. The NJDOT would need a 130′ bridge to safely span Friendship Creek. NJDOT had an 80′ Acrow Panel Bridge in its storage yard that was used during an emergency on Interstate 80 in 2001. NJDOT dispatched the bridge for delivery by the following day. Meanwhile, Acrow proceeded with new drawings and shipped an additional 40′ bridge from its yard. Bridge assembly began July 1, and the final 130′ clear-span bridge was lifted into place July 16 —only four days after the storm.

"Our emergency response team worked in unison with the State engineers and everything organized in a matter of hours," said Bill Killeen, president of Acrow Corporation. "The panel bridge system is versatile and can be erected quickly, enabling the construction crews to get in and out. This helps minimize the inconvenience to the motoring public."

The new bridge includes two 12′-wide lanes with 3′ shoulders for traffic on the bridge. It is a major thoroughfare connecting the western part of New Jersey to the resorts along the New Jersey coastline.

For more information on Acrow Corporation or the Acrow® 700XS® Panel Bridging System, please visit www.acrowusa.com. ★

Second Public Review of AISC Prequalified Connection Standard

The AISC standard *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* is now available for its second public review. The new standard provides design, detailing, quality, and inspection requirements for prequalified beam-to-column connections in Special Moment Frames (SMFs) and Intermediate Moment Frames (IMFs). Copies of the proposed standard can be downloaded for free by visiting www.aisc.org/AISC353. Or, you can purchase a copy for a nominal fee of $12 by calling 312.670.5411 or by e-mailing cummins@aisc.org. Deadline for submitting comments is October 18, 2004. Review forms are available at www.aisc.org/prdoc. E-mail completed forms to hewitt@aisc.org, or mail to: Christopher Hewitt, American Institute of Steel Construction, Inc., Suite 3100, One East Wacker Dr., Chicago, IL 60601-2000. ★

Excellence in Hot-Dip Galvanizing Awards Competition

The American Galvanizers Association (AGA) has announced that the organization is now accepting applications for its 2005 Excellence in Hot-Dip Galvanizing Awards.

Presented annually, these awards recognize projects that utilize hot-dip galvanizing in an ideal, creative, innovative, or monumental fashion.

Winning projects will be announced at the AGA's annual conference awards dinner Wednesday, April 6, 2005 at the Biltmore Hotel in Coral Gables, FL. For each award presented, the galvanizer and all parties involved with the project will receive recognition. Winners will receive their own four-color project brochure and an awards plaque. They will also be featured on the AGA website and mentioned during various seminars, editorials, and publications throughout 2005 and beyond.

Non-winning projects may be publicized as well. All submissions will be considered for the 2005 Most Distinguished Project of the Year.

While membership is not required to enter, projects submitted must have been galvanized by an AGA member galvanizer, with construction completed during 2004. This free competition is open to all architects, engineers, fabricators, and other members of the specifying community. For project guidelines, rules, and applications, contact Madison Sterling at the AGA (720.554.0900 x15, or msterling@galvanizeit.org). Forms may also be downloaded at www.galvanizeit.org/awards. All entries must be received or postmarked on or before Friday, January 28, 2005; no extensions will be provided. ★

Steel Bridge Longevity

The 100-year old steel bridge that crosses the Delaware River and connects New Hope, PA with Lambertville, NJ reopened in late July after minor repairs. The bridge typically carries about 16,000 vehicles a day but had been closed for repairs on weekdays since the beginning of the year.

Conn Abnee, executive director of the National Steel Bridge Alliance, cites the New Hope/Lambertville free bridge as an example of the excellent life cycle performance possible with steel bridges.

According to the Delaware River Joint Toll Bridge Commission, repairs included the replacement of the flooring systems, sidewalks and handrails, miscellaneous steel repairs, cleaning and painting and various safety improvements. J.D. Eckman Company of Atglen, PA completed the repairs at a cost of $6.3 million.

The bridge, which contains 962 tons of steel, is 1046 ft long and 23 ft wide. It consists of six-span, pin-connected Pratt-type trusses, which are 27 ft high. Floor beams support roadway stringers with a 5″-deep open grid steel flooring. A cantilevered, well-used sidewalk resides outside the downstream truss.

The original bridge stone substructure, completed September 12, 1814, remains in place today. In 1904 the present steel superstructure replaced a succession of wooden superstructures washed away by floods. ★

AISC Staff News

Kurt D. Gustafson, S.E., P.E., has been named Director of Technical Assistance at the American Institute of Steel Construction, Inc. His primary responsibility is to lead the free technical assistance services provided by AISC's Steel Solutions Center.

Prior to joining AISC, Gustafson was Principal with Tylk Gustafson Reckers Wilson Andrews, LLC in Chicago. "Kurt's retirement from TGRWA created quite an opportunity for AISC," said Charlie Carter, AISC's Chief Structural Engineer. "His 30 years in design offices and eight years of construction experience with American Bridge before that make him uniquely suited to lead AISC's Steel Solutions Center efforts in technical assistance."

Jason R. Ericksen, S.E., has been promoted to the position of Director of the AISC Steel Solutions Center. He will be responsible for the coordination of all activities of the center and promoting the services provided by the center to architects, owners, general contractors and structural engineers.

Ericksen joined the staff of AISC Marketing, LLC, in August of 2001 as a Solutions Center Advisor. Over the past three years he has been instrumental in providing over 150 conceptual solutions demonstrating the cost and schedule advantages of utilizing steel framing systems to project decision makers. In addition, he is the primary author of many of the SteelTools™ available for free download from the AISC web site for assistance in the design of steel structures.

Prior to joining AISC, Ericksen was with Campbell & Associates Consulting Engineers in Chicago. He holds a masters degree in structural engineering and a bachelors degree in civil engineering from the University of Illinois at Urbana-Champaign.

The Steel Solutions Center is for people who need technical assistance, innovative solutions, or tools to make structural steel design easier. Developed by AISC specifically for steel fabricators, structural engineers, architects, owners, developers and other professionals, the Steel Solutions Center is the number one source of information for structural steel. Find out all the Steel Solutions Center offers at **www.aisc.org** or by calling 866.ASK.AISC. ★

Steel Girder Launch Video Available

The solution for the U.S. 20 river valley crossing in Hardin County, IA, was a launched steel I-girder bridge. (See coverage in the February 2004 issue of *Modern Steel Construction* at **www.modernsteel.com**.) Because the site was environmentally sensitive, the bridge frame was constructed span by span in a launching pit behind one abutment and rolled out over piers in the river valley. At the time of construction, the bridge was the first launched steel girder bridge in the United States.

The Iowa Department of Transportation, in cooperation with the Federal Highway Administration, Iowa State University, Jensen Construction, and HNTB Corporation, extensively instrumented and documented behavior of the bridge during construction. Both a 32-minute video of the launching process on DVD and a technical report on CD are available at no charge. For a copy, write to Max G. Grogg, P.E., Programs Engineer, FHWA, 105 Sixth Street, Ames, IA 50010; call 515.233.7306; or e-mail max.grogg@fhwa.dot.gov. ★