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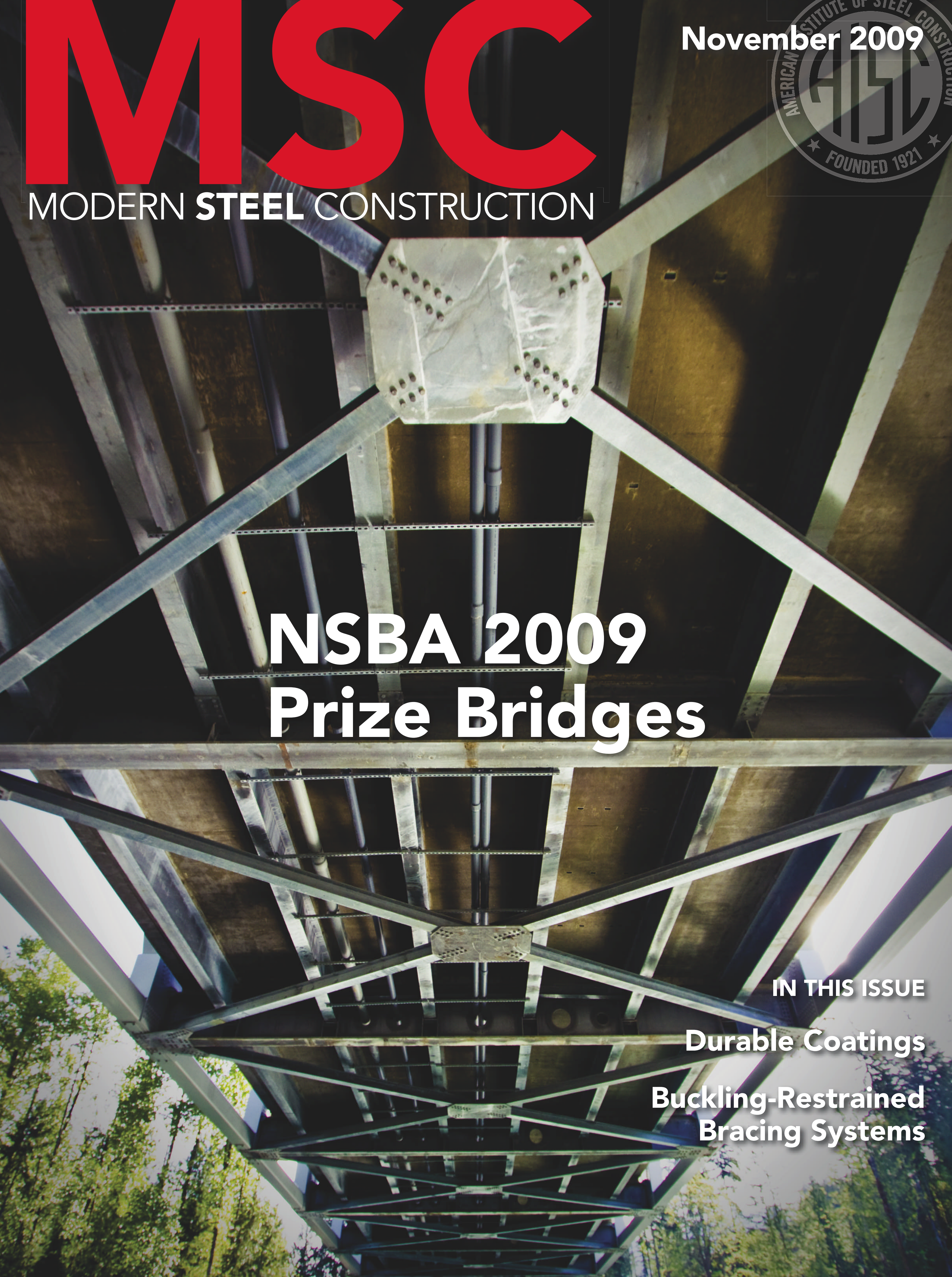
MODERN **STEEL** CONSTRUCTION

November 2009



NSBA 2009 Prize Bridges

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November 2009

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ON THE COVER: Truss steel on the Sauk River Bridge was powder-coated with paint over galvanizing material, preventing zinc from leaching into the river below.

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editor's note

RECENTLY MY DAUGHTER'S CLARINET BROKE. Our rental agreement with Evanston Band & Orchestra said they would repair any problems so we dutifully drove over to the store. The company's response? They immediately took the damaged clarinet and handed us a nearly identical loaner. But they also went one step further: They said it would be a couple of weeks to repair the clarinet and at that time, we could either come back and exchange the clarinet, or we could just keep using the loaner and avoid having to make another trip to the store (we made the trip; my daughter wanted "her clarinet" back).

With today's poor economy, the natural inclination of many companies is to pull back on such niceties as customer service. But your current customers are also your best prospects and retaining them may be one of the most critical factors in your company's success.

Consider three other customer service experiences I've had recently.

My wife's Blackberry has been a lemon since we got it (in contrast I love my Blackberry and it was on my recommendation that she got hers). T-Mobile had previously replaced the SIM card and upgraded the software but the problems with the phone have only gotten worse. My wife was finally fed up so I called T-Mobile and recommended they replace the phone. They were very pleasant and responsive and told me they would ship another phone out and I'd only have to pay the shipping cost. I asked if I could simply go to one of the seemingly hundreds of T-Mobile stores located within walking distance of my house to exchange it (maybe even the one where it was originally purchased) but they said that wasn't allowed.

Last year I bought a computer from HP for home and for the most part I've been pretty satisfied: it's fast, has plenty of capacity for my family's needs, and it was relatively inexpensive. But over time it developed an extremely noisy fan. I opened the case but couldn't determine which of the fans was the troublemaker. After some online research I discovered reports of noisy fans in the video card that came with this particular model—and a link to MSI which was offering a replacement at no charge (not even the shipping cost). Sure enough, the replacement card came last week and now my computer hums along quietly. I only wish HP had thought to notify me; I wonder how many people had the same problem and didn't find the information online?

I store a lot of photos and music on an external hard drive. The other night, the plug connector broke off inside the Maxtor One Touch 4 unit. At a loss as to what to do, I turned to Seagate's (the parent company for Maxtor) website. I couldn't find any helpful information so I emailed customer service, who advised me to "get in touch with local technicians or someone to check if they can help you fix since only the connection is broken." A suggestion on who to contact would have been a lot more useful, however.

If you were the customer, which of these companies would you be most likely to use again? (My middle child just started oboe lessons. Guess where we rented his oboe!) Now is a good time to look at your own company's customer service and the impact it has on your business. Are your clients likely to be one-time customers or repeat business?

SCOTT MELNICK
EDITOR

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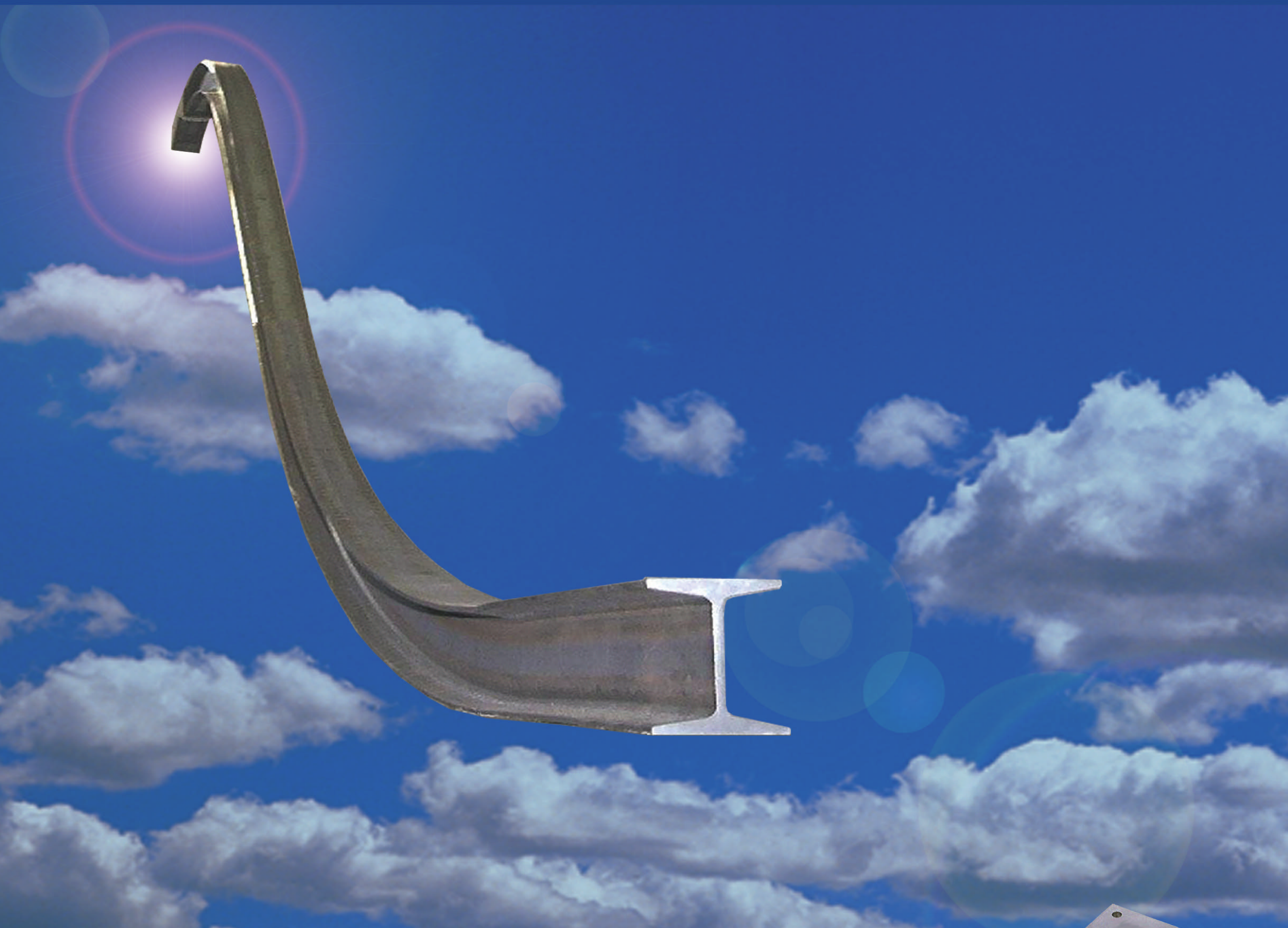

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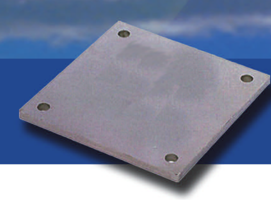

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IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to solutions@aisc.org.

Calculation of Weights

If I purchase steel plate as a raw material for a project and the customer is quoted a price per pound for the product, is he required to pay for the remaining skeleton of the steel plate if it is not useable in the project? Example: I purchase sheets that are 4 ft by 10 ft and burn two pieces that are 44 in. by 58 in. from each piece. I have a skeleton left over that is not useable. Does the customer pay for 40 sq. ft of material or only the weight of the two pieces?

The calculation of weight for this example is stated in Section 9.2.2(c) of the AISC *Code of Standard Practice* (a free download at **www.aisc.org/code**) as follows:

"When parts can be economically cut in multiples from material of larger dimensions, the weight shall be calculated on the basis of the theoretical rectangular dimensions of the material from which the parts are cut."

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

Document Discrepancies

In case of a discrepancy between plans and specifications for buildings, which one governs?

The subject of document discrepancies is covered in Section 3.3 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (a free download at **www.aisc.org/code**) as follows:

"When discrepancies exist between the Design Drawings and Specifications, the Design Drawings shall govern."

Note that this section also states that any discrepancies that are discovered must be reported for resolution. It also states that it is not the responsibility of the construction team to discover discrepancies.

This may seem like a confusing answer, so let's go further. If a discrepancy is noted by the fabricator or detailer, it should be reported so that the design team can advise what information is correct and the work can be performed with the correct information. However, the fabricator, detailer, and others on the construction team are not expected to find discrepancies. Sometimes, the presence of a discrepancy only comes to light after a piece has been detailed, fabricated and/or erected. The quoted sentence provides a way to resolve if the work already performed has been performed properly, and who should pay for any re-work that is needed.

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

Pipe Design

Section F8 of the 2005 AISC *Specification* addresses flexural design of round HSS. Can Section F8 be used for the flexural design of steel pipe?

Yes, it is common to do so, and the AISC *Specification* explicitly includes steel pipe complying with ASTM A53 Gr. B. The Glossary of the 2005 AISC *Specification* defines HSS as a square, rectangular or round hollow structural section produced in accordance with a pipe or tubing product specification. Section A3.1a(3) of the *Specification* lists pipe as meeting the ASTM A53/A53M, Gr. B standard.

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

Allowable Stresses in 1967

What was the allowable stress for A36 steel, fabricated in 1967?

Like today, the allowable stresses in 1967 were based on the limit state being investigated. F_y for ASTM A36 steel was and still is 36 ksi. Some example cases are as follows:

Flexure

The allowable strong-axis bending stress for a compact shape braced at a small enough interval to preclude lateral-torsional buckling was:

$F_b = 0.66F_y = 23.8$ ksi (use of 24 ksi was common)

The allowable strong-axis bending stress for a non-compact shape braced at a small enough interval to preclude lateral-torsional buckling was:

$F_b = 0.60F_y = 21.7$ ksi (use of 22 ksi was common)

The allowable bending stress for bracing at larger intervals was lower than these values.

Compression

The allowable axial compression stress was based on the slenderness ratio with a maximum of $F_a = 0.60F_y$. The actual allowable was much lower for any typical column length, of course.

Tension

The allowable tension on the net section, except at pin holes, was

$F_t = 0.60F_y$.

See the 1963 AISC *Specification* for further information.

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

Large Bolted Connections

We currently have a job with 1¼-in. diameter bolts (approximately 50 per connection) with 2 plies of 3-in.-thick steel. The hole size specified is 1⁵⁄₁₆ in. Needless to say all of the holes do not exactly line up perfect. What are the dimensional tolerances for the locations of holes in large bolted connections?

Neither the AISC *Code of Standard Practice* nor the AISC *Specification* provides tolerances on the locations of bolt holes. The holes however must be placed such that the other tolerances given in the AISC *Code of Standard Practice* can be maintained and the bolts can be installed in the holes. That is, the only requirement is that the joint must fit up, and it is up to the fabricator to employ a method that will achieve this. Some suggestions for how to do this follow.

When dealing with thick plates, consideration must be given to the use of oversized holes and slip-critical connections. The use of slip-critical connections will usually require more bolts. This may be detrimental to economy in both the shop and the field, due to the greater number of holes to be drilled, and bolts to be installed. There will also be additional cost involved in surface preparations and inspections.

The use of bearing bolts in standard holes will mean fewer holes to drill and bolts to install, but may require reaming if things do not fit-up. This reaming can be both time-consuming and costly in the field.

Larry S. Muir, P.E.

steel interchange

High-Seismic Column Splice

Section 8.4a(2) of AISC 341 requires the available strength for each flange (LRFD) is noted as $0.5R_yF_yA_f$. Is the term A_f the area of one flange or the total area of the two flanges?

The term A_f refers to the area of one flange of the smaller column connected.

Thanks for your question—this has been clarified in the draft of the 2010 Seismic Provisions where the term A_f is proposed to be replaced by the term $b_f t_f$, which is defined as the area of one flange of the smaller column connected.

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

Steel Properties at Elevated Temperatures

What is the reasoning between the different material ratios vs. temperature (i.e. Modulus of Elasticity vs. Temperature & Yield Strength vs. Temperature) given in the AISC 13th ed. Table A-4.2.1 versus the graphs (Figures 3.2 and 3.3) published in ASCE, *The Structural Design of Air and Gas Ducts for Power Stations and Industrial Boiler Applications*? When comparing the ratios for yield strength, AISC gives a reduction beginning at 800 °F (i.e. 0.94), and the ASCE publication shows almost a linear reduction in yield strength beginning at 100 °F. Conversely, when comparing the ratios for Modulus of Elasticity, the AISC ratios drop much faster than what is given by ASCE. I would appreciate your help in understanding the discrepancies between these reported values.

The properties in Appendix 4 of the AISC *Specification* are deemed suitable for the ultimate state (pre-collapse) conditions of structures exposed to severe fires. Such conditions involve very high levels of thermal and structural strains, very large deformations, and implicitly assume irreparable damage. Therefore, the associated properties are not suitable for the design of in-service ducts at elevated temperatures.

There is more than one way to test steel for mechanical properties at elevated temperatures. Even from the same set of tests at elevated temperatures, there are several ways to derive mechanical properties. The steel properties at elevated temperatures reported in the literature often vary considerably due to these variations in testing and derivation methods.

The yield strength values/ratios in Appendix 4 of the AISC *Specification* could be associated with stress at 2% strain (this is quite different from the usual 0.2% offset method), and they are essentially equal to the ultimate/tensile strength values/ratios (note that ultimate strength ratios are normalized by the yield strength in the *Specification*) at 750 °F and higher temperatures.

Extra conservatism of elasticity modulus values/ratios in the AISC *Specification* follows from concerns about column stability. Part of the difference may also be due to differences in testing and derivation methods.

Farid Alfawakhiri, Ph.D.
American Iron and Steel Institute

U-Factor in 1989 Column Tables

How was the factor *U* that was tabulated in the 9th edition ASD *Manual* column tables calculated? This factor is used to determine an equivalent axial load for beam-columns.

The *U* value for beam-columns published in the 9th edition column tables was a hold-over from that factor first shown in the 8th edition *Manual*. Unfortunately, I could find no explanation in either manual as to how these values were derived. I did some searching in my library of old publications, however, and found a derivation in my U.S. Steel *Column Design Curve* book from 1969 that gave the Equation $U = 0.66S_x/0.75S_y$. This is the ratio of the strong-axis to weak-axis allowable bending stresses for a compact shape. I checked a few numbers in the 9th edition and found them accurate for the compact shapes. The Equation $U = 0.60S_x/0.75S_y$ was used for non-compact shapes.

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

High-Seismic Column Splice Location

AISC 341 requires column splices to be no closer than 4 ft, 0 in. from beam to column connection. What is the basis of this requirement? Can the splices be closer than 4 ft, 0 in. if complete-joint-penetration groove welds are used?

This 4-ft requirement is intended to keep the column splice away from the beam/column intersection, and locate it closer to the point of inflection between stories. The OSHA requirement for safety erection of exterior columns also influenced the 4-ft dimension.

In the draft of the 2010 AISC *Seismic Provisions*, it is proposed to relax this requirement for columns that are spliced with CJP groove welds. In such cases, it is proposed that the splice be permitted to be located closer to the beam-to-column flange connections, but not less than the depth of the column. Note that the OSHA requirements for fall protection still must be met, and if the column is not extended up high enough, another means of attaching the perimeter cables must be provided.

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

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

Kurt Gustafson is the director of technical assistance in AISC's Steel Solutions Center. Larry Muir is a part-time consultant to AISC. Farid Alfawakhiri is senior engineer at the American Iron and Steel Institute.

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

The opinions expressed in Steel Interchange do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

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

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steel quiz

LOOKING FOR A CHALLENGE? *Modern Steel Construction*'s monthly Steel Quiz tests your knowledge of steel design and construction.

According to Mies Van der Rohe, "God is in the details". Accordingly, this Steel Quiz highlights information that can be found in …well, see Question 1.

1. True/False: AISC provides a manual that illustrates common details and formats for the preparation of shop and erection drawings.

2. What are some of the operations that one commonly finds taking place in a structural steel fabrication shop?

3. As per the AISC *Code of Standard Practice* are fabricators and detailers obligated to discover discrepancies in contract documents?

4. In order to optimize the erection process, how often should columns be spliced?
   (a) Every two floors
   (b) Every three floors
   (c) Every four floors
   (d) Both (a) and (c) are correct

5. True/False: Weld length does not limit the effective size of a fillet weld.

6. True/False: A weld symbol on the bottom side of the line signifies a weld on the arrow side.

7. According to the American Welding Society what is the standard symbol with which to specify dye penetrant testing on a drawing?
   (a) PT (b) MT
   (c) RT (d) UT

8. What is CIS/2?

9. Which of the following material types is corrosion resistant?
   (a) ASTM A992
   (b) ASTM A500
   (c) ASTM A588
   (d) ASTM A514

10. True/False: Batten plate is the term used to describe a column base leveling plate.

TURN TO PAGE 14 FOR ANSWERS


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steel quiz ANSWERS

1 True. The recently released AISC *Detailing for Steel Construction*, 3rd Edition provides guidance on the proper detailing and formatting of shop and erection drawings. It is similar to the 2nd edition, but updated to conform to the 2005 AISC *Specification*. You can purchase this manual at **www.aisc.org/bookstore**.

2 Fabrication of structural steel includes a wide variety of operations, some of which vary based upon equipment. Computer-numeric-controlled equipment has rendered template making and laying out a rare activity these days. Still there are cutting, punching and drilling, grinding, fitting, welding, bolt installation, and similar operations. Chapter 1 of the 3rd Ed. AISC *Detailing Manual* provides information on these and many other activities.

3 No. The accuracy of the contract documents is the responsibility of the Owner's Designated Representative for Design. Section 3.3 of the 2005 AISC *Code of Standard Practice* [a free download at **www.aisc.org/code**] requires that discrepancies must be reported when they are discovered, but also notes that the fabricator and steel detailer are not obligated to find them. Chapter 2 of the 3rd Ed. AISC *Detailing Manual* addresses topics related to the quality of construction drawings.

4 (d) Since erectors are required by OSHA to tie off when the fall distance exceeds 30 ft, placing column splices every three floors is an inefficient choice for the purposes of erection. Other common OSHA requirements that impact steel design and detailing are discussed in Chapter 2 of the AISC *Detailing Manual*.

5 False. According to Section J2.2b of the 2005 AISC *Specification*, the maximum effective fillet weld size can be no greater than 25% of the weld length. Connection detailing requirements like this one are covered in Chapter 3 of the AISC *Detailing Manual*.

6 True. Welding symbols and other aspects of welding are covered in Chapter 4 of the AISC *Detailing Manual*.

7 (a) Basic symbols for nondestructive testing have been developed by AWS and published in AWS A2.4. Dye penetrant testing is symbolized by PT. PT and other methods of NDT also are covered in Chapter 4 of the AISC *Detailing Manual*.

8 CIS/2 is a neutral data file format for the exchange of design and construction information. AISC has endorsed this format as the method to accurately convey information between all members of the project team. Appendix C of the AISC *Detailing Manual* provides more information on data formats.

9 (c) ASTM A588 is also known as weathering steel. It gets its corrosion resistance by forming an adherent protective layer on its surface as it weathers. Chapter 1 in the AISC *Detailing Manual* (and Part 2 of the 13th Edition AISC *Steel Construction Manual*) provide a summary of materials.

10 False. A batten plate is an element that is used to join two parallel components of a built-up column, girder or strut. It is rigidly connected to the parallel components and designed to transmit the shear between them. See the glossary in the AISC *Detailing Manual* for common steel terms.

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

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news

NETWORKING

The New steelTOOLS.org Takes Off

The activity on the new steelTOOLS website is exceeding expectations. Launched by the AISC Steel Solutions Center on September 18, the site is home to more than 135 software utilities and programs related to steel design and construction that have been contributed by site members.

AISC's director of technical marketing Tabitha Stine reports that in its first week alone the site saw nearly 5,000 separate user visits, 50,000 page views, and more than 2,200 downloads. By mid-October, the number of "tools" downloaded had climbed to 6,800. Among the most popular is the wind load calculation spreadsheet program posted by Alex Tomanovich.

In addition to file sharing, the new steelTOOLS site provides free social and business networking opportunities for design and construction professionals in the form of a multi-faceted online community. Capabilities include discussion boards, blogs, and other connectivity tools.

"People have been very busy logging in and setting up profiles to blog on various hot topics," Stine says. She says special interest groups have been set up focusing on bridge design and construction, sustainability, and detailing, to name just a few. Each group is outfitted with its own library which, as documents and other files are added, will serve as a valuable reference repository.

Everyone involved in the structural steel industry is encouraged to participate. Registration is free at **www.steeltools.org**.

RESOURCES

Performance-Based Plastic Design Explained

A new book published by Schuff Steel Company, the National Council of Structural Engineers Associations (NCSEA) and the International Code Council (ICC) explains how to design buildings for desired seismic performance using the new Performance-Based Plastic Design (PBPD) method. The authors of "Performance-Based Plastic Design Earthquake-Resistant Steel Structures" are Subhash C. Goel, Ph.D., P.E., professor emeritus in structures and materials engineering at the University of Michigan, Ann Arbor, Mich., and Shih-Ho Chao, Ph.D., assistant professor of civil engineering at the University of Texas, Arlington, Texas. Professor Goel will give a presentation on this subject at NASCC in May, 2010.

The book is a collaborative effort based on research conducted by Goel and Chao at the University of Michigan and sponsored by the American Institute of Steel Construction; NUCOR Research and Development; and Nabih Youssef Associates, Structural Engineers. An advisory group consisting of 13 leading academics and engineers from across the country read and commented on the manuscript.

"This book offers clear explanations of design methodology. It also applies the methodology to different structural systems while clearly demonstrating each step of the process through numerous design drawings, formulas and tables," said Jay Allen, S.E., executive vice president of sales and engineering for Schuff International, which helped underwrite the book's publication.

The book is available at **www.iccsafe.org** or by calling 888-422-7233.

EDUCATION

Live Webinars Launch and Hit the Mark

The first AISC live webinar, presented on SteelDay, September 18, was well received by engineers across the country. Well over 1,000 people logged in to hear AISC vice president and professor emeritus at Penn State University, Louis F. Geschwindner, present *Design for Stability Using the 2005 AISC Specification*. This relevant topic garnered good questions and participation from the audience.

AISC's next live webinar will occur on December 10. *Introduction to Seismic Steel Design and the AISC Seismic Provisions* will be presented by Thomas A. Sabol, Ph.D., S.E. Sabol is a principal with Englekirk & Sabol Consulting Structural Engineers, Inc., Los Angeles, and adjunct professor in the civil engineering department at UCLA.

AISC live webinars are presented using an internet and phone connection. Purchase a single site connection and an unlimited number of people within your office can attend at that site. All attendees will earn continuing education credits. More information on registration and pricing can be found at **www.aisc.org/webinars**.

People and Firms

- **ESAB Welding & Cutting Products** has added two new wire drawing lines at its Ashtabula, Ohio, manufacturing facility. The new lines will produce wire for high production submerged arc welding (SAW) applications, significantly increasing the company's SAW wire capacity. To learn more, visit **www.esabna.com**.
- **Raymond F. Messer**, P.E., president and chairman of the board of Walter P Moore, has been presented with The Alumni Hall of Fame Award of Carroll College. The award honors alumni of the Helena, Mont., institution who have given outstanding contributions of stewardship—sharing of time, talent and treasure in distinguished service to the community and/or to Carroll College. One of three alumni to receive the honor this year, Messer was presented the award on Friday, September 25 at Carroll College's Centennial Homecoming.
- **Independence Tube Corporation** is expanding its Marseilles, Ill., manufacturing facility, with production to begin in the fourth quarter of 2009. The new mill will produce HSS from 1.66-in. OD to 5-in. OD. Headquartered in Chicago, Independence Tube also has operations in Decatur, Ala. **www.independencetube.com**.
- **Delbert F. Boring**, P.E., has been selected as the 2009 recipient of the Outstanding Civil Engineering Alumni Award presented by The Ohio State University (OSU) Civil Engineering Alumni Association. Boring joined the American Iron and Steel Institute (AISI) in 1976 as regional director of construction codes and standards. In 2003, he was promoted to vice president of construction market development, where he was responsible for developing and implementing the Market Development Strategic Plan for AISI's Construction Market program. He is a recognized authority on structural fire endurance and co-authored the book *Fire Protection Through Modern Building Codes*, published by AISI in 1981.
- **American Punch Company** is now certified to ISO 9001:2008. Based in Euclid, Ohio, the firm manufactures punches, dies and shear blades used in steel fabrication. More information is available at **www.americanpunchco.com**.
- **Trilogy Machinery, Inc.**, Belcamp, Md., is now the exclusive North American agent for Belgium-based DERATECH, offering the company's press brakes and shears. For more information, visit **www.deratech.be** and **www.trilogy-machinery.com**.

AWARDS

Malley Selected as 2010 T.R. Higgins Lecturer

James O. Malley, P.E., S.E., senior principal of Degenkolb Engineers, San Francisco, is the 2010 recipient of the prestigious AISC T.R. Higgins Lectureship Award. Malley is being honored for his paper on *The 2005 AISC Seismic Provisions for Structural Steel Buildings*, which was published in the First Quarter 2007 AISC *Engineering Journal*.

The award is presented annually by the American Institute of Steel Construction (AISC) and recognizes an outstanding lecturer and author whose technical papers are considered an outstanding contribution to the engineering literature on fabricated structural steel.

Malley has more than 26 years of experience in structural design, seismic evaluation, and seismic strengthening of existing buildings. His projects include a variety of healthcare facilities and several research efforts funded by institutions like the Federal Emergency Management Agency (FEMA) and the National Science Foundation (NSF).

He has made numerous presentations on earthquake effects, seismic design of steel structures and the AISC *Seismic Provisions*. The author of more than 50 technical papers, Malley was co-recipient with the late Egor Popov of the 1986 American Society of Civil Engineers (ASCE) Raymond C. Reese Research Prize for their paper *Shear Links in Eccentrically Braced Frames*.

In addition to his role at Degenkolb Engineers, Malley also served as the Project Director for Topical Investigations of the SAC Joint Venture. In that position, he was responsible for directing studies of the steel frame buildings damaged by the Northridge Earthquake and all of the analytical and testing investigations performed as part of the SAC Steel Project. SAC has published guides on how to design, inspect, evaluate, and repair steel moment frame connections, reflecting Malley's nationally recognized expertise in the seismic design of steel structures. Much of this work formed the basis for the development of the AISC *Seismic Provisions* and related standards and publications.

Malley is a member of the AISC Committee on Specifications and chair of its subcommittee responsible for developing the AISC *Seismic Provisions*. He has served as a member of the Structural Engineers Association of Northern California (SEAONC) board of directors, including a term as president from 2000-2001. He also has served as a member of the Structural Engineers Association of California (SEAOC) board of directors, also including a term as president from 2003-2004. He was named a SEAOC Fellow in 2007. Currently, he is a member of the National Council of Structural Engineers Associations (NCSEA) board of directors, and will become its president in 2010.

news

AWARDS

Student Photos Capture the SteelDay Spirit

Jennifer Jernigan, a student at North Lake College, Irving, Texas, is the winner of AISC's 2009 Student Photo Contest. Students were invited to submit three photos demonstrating the combination of structural steel and the SteelDay theme, "Interact. Learn. Build." Jernigan's photos to the right show two welders working together, a lesson in layout, and the fit-up assembly of a truss in the fabrication shop prior to shipping the pieces.

Jernigan, who will receive her associates degree in construction management next May, says she has enjoyed photography as a hobby for many years. In addition to being a student at North Lake, she also works full time as an estimator at Hirschfeld Steel Group in Irving. She selected these photos from recent visits to Hirschfeld facilities where she was photographing various operations to update the company's brochures and website.

Above, top: Interact
Two welders work together to attach a large base plate to a column, an operation where teamwork is critical.

Above, middle: Learn
A long-time employee teaches the art of marking up steel plate for fabrication, a very important part of the process.

Above, bottom: Build
After being fabricated in three separate parts, a 185-ft rolled truss is assembled in the shop to ensure a perfect fit in the field.

All photos by Jennifer Jernigan, North Lake College, Irving, Texas. Photos taken at Hirshfield Industries facilities, San Angelo and Abilene, Texas.

letters

More Agreement on the Value of Hand-Checks

I want to compliment you for printing, and Matt Thomas for writing, the Topping Out article ["If You Want it Done Right, Do it Yourself"] in the July 2009 edition of MSC.

As a "senior" member of the structural engineering profession, I heartily agree with Matt's opinion. He is on target when it comes to "feeling a structure." I am very impressed with his insight and practical approach.

I graduated in 1970, and during my early years calculations were done by hand, which helped to develop a "feel for structures." As my career progressed I used analysis programs which reinforced my "feel for structures." I envy the young keyboard jocks, but would not trade my apprenticeship years for anything. There are times when the most obvious item is entrusted to a program, but in fact not included in the analysis checking.

As I mentor young engineers, I continually convey embracing their craft and understand why structures behave as they do. Now I have a great article to give to them to bring that concept home.

Please forward my gratitude to Matt for expressing his thoughts so well. It's comforting to know that there are Matts out there to continue the profession.

Dennis Schiffer, P.E., S.E.

Great Advice on Double Checking

As a corrosion consultant for URS Washington Division in Princeton, N.J., I work a lot with consulting structural engineers, probably more than any other engineering discipline. While not an S.E. myself—I'm a chemical engineer—I work with structural steel and reinforced concrete daily. They are two of my favorite materials of construction.

The article "If You Want it Done Right, Do it Yourself" by Matt Thomas (July MSC, page 66) is a good reminder of the great benefits of hand calculations in checking computer models and assumptions. In discussing the article with a P.E. I know who worked on the Hartford Civic Center, he replied, "Thanks for the great article by Matt Thomas. This brings back memories in my work with Ewing Cole Engineers who designed the HCC." He and I fully agree with Matt that young engineers should be wary of trusting the "black boxes" without double checking them by doing hand calcs. After all, the computer analysis is only as good as the engineers' assumptions and data. Good job, Matt!

Robert E. Moore, P.E.

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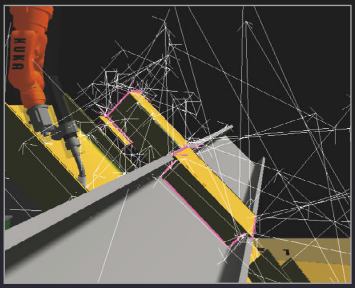

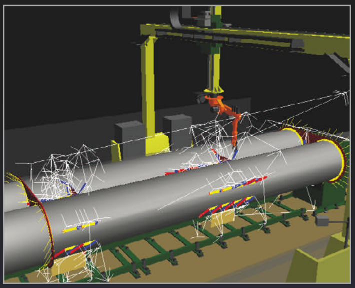

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NATIONAL STEEL BRIDGE ALLIANCE 2009 PRIZE BRIDGE COMPETITION

THE NSBA PRIZE BRIDGE COMPETITION honors significant and innovative steel bridges constructed in the United States. Awards are presented in a variety of categories, including long span, medium span, short span, movable span, major span, reconstructed, and special purpose.

The National Steel Bridge Alliance thanks the submitters of all of the outstanding entries for their participation in the 2009 Prize Bridge Competition. The projects were judged on:

- ➜ Innovation
- ➜ Aesthetics
- ➜ Design and engineering solutions

Designers of the winning Prize Bridge projects will receive award plaques during an award reception at the 2009 World Steel Bridge Symposium in San Antonio, Texas, on November 19, 2009. Owners of winning bridges will receive award plaques at a dinner banquet during the 2010 AASHTO Bridge Subcommittee meeting.

Jurors for this year's competition:

Ralph Anderson
Chief of Bridges & Structures, Illinois Department of Transportation, Springfield, Ill.

Nancy Kennedy
Principal Bridge Engineer, Nevada Department of Transportation, Reno, Nev.

John Elwell
Senior Supervising Engineer, Senior Project Manager, Parsons Brinkerhoff, Minneapolis

Bill Wilson
Editor, Roads & Bridges Magazine, Arlington Heights, Ill.

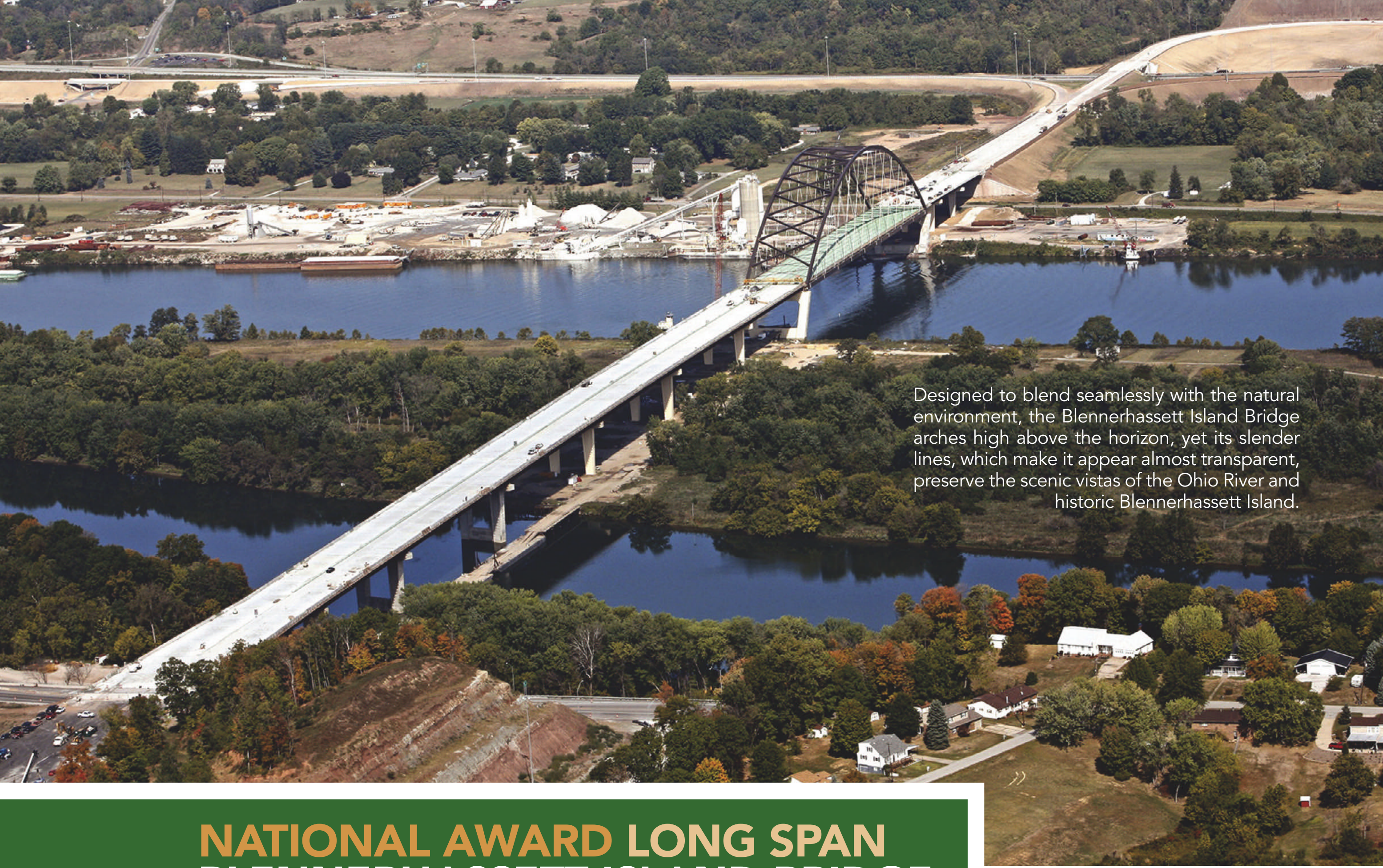
2009 Prize Bridge Awards

National Awards

Long Span	Blennerhassett Island Bridge Wood County, W.Va. and Washington County, Ohio
Major Span	Tempe Town Lake Light Rail Bridge Tempe, Ariz.
Medium Span	Mount Si Bridge Replacement King County, Wash.
Short Span	Roslyn Road Grade Crossing Elimination Mineola, N.Y.
Special Award	Woodrow Wilson Memorial Bridge Washington D.C.
Special Purpose	Bob Kerrey Pedestrian Bridge Omaha, Neb.
Movable Span	Hamilton Avenue Bridge Brooklyn, N.Y.
Reconstructed	Thurston Avenue Bridge over Fall Creek Ithaca, N.Y.

Merit Awards

Long Span	Route 151 over the Salmon River East Haddam, Conn.
Medium Span	Sauk River Bridge Darrington, Wash.
Medium Span	Three Springs Drive Bridge Weirton, W.Va.
Special Purpose	Dr. Martin Luther King, Jr. Memorial Bridge Roanoke, Va.
Reconstructed	MacArthur Maze Ramp Reconstruction Oakland, Calif.



Designed to blend seamlessly with the natural environment, the Blennerhassett Island Bridge arches high above the horizon, yet its slender lines, which make it appear almost transparent, preserve the scenic vistas of the Ohio River and historic Blennerhassett Island.

NATIONAL AWARD LONG SPAN
BLENNERHASSETT ISLAND BRIDGE
WOOD COUNTY, W.VA. AND WASHINGTON COUNTY, OHIO

The Blennerhassett Island Bridge, which spans the Ohio River between West Virginia and Ohio, was the critical remaining "missing link" of the final segment of Appalachian Highway Corridor D. This major economic development highway traverses approximately 240 miles along U.S. 50 from Cincinnati, Ohio, to Clarksburg, W.Va.

The 4,008-ft bridge includes an 878-ft-long, tied-arch main span with a rise of 175 ft and approach spans that consist of variably spaced steel-plate girders with spans up to 401 ft in length. To minimize the size and weight of the approach span superstructure, the design uses hybrid girders and high-strength steel. Post-tensioned concrete pier caps support the main tied-arch span and contribute to the structure's cost efficiency.

The bridge's tied arch ranks as the longest networked tied-arch structure in the United States and is among the longest in the world. The bridge spans historic Blennerhassett Island, an environmentally sensitive area and designated Historic District, and the main and back channels of the Ohio River.

Numerous innovative approaches were employed during the planning, design, and construction of the Blennerhassett Island Bridge. A total of 16 alternatives were studied to arrive at the alternative with the least environmental impact.

Archaelogical concerns led to the use of trench shields during project excavation to protect investigators as they searched for prehistoric deposits as much as 40 ft under ground. Coincidentally that reduced the amount of excavation required, compared to benching, saving both time and money.

Another example of the efforts undertaken to protect the environment throughout construction, the team performed tree-topping as an alternative to tree removal. Removing the trees within the bridge alignment would have been a quicker and more easily implemented solution, but the tree-topping technique saved more than 200 trees that are an integral part of a valuable forested wetland complex.

From the earliest planning stages, accommodating the massive size of the Blennerhassett Island Bridge posed a significant design challenge. Engineers sought to develop a design that would optimize structural integrity and user safety, but that would also control costs by minimizing the size and weight of the approach span superstructure. They departed from traditional bridge design methodology by designing a tied-arch structure that integrates a key truss-type element—post-tensioned steel networked cables that improve structural strength and flexibility and enhance safety. This hybrid "arch-truss" design approach made it possible to leverage the benefits of both bridge types.

The bridge's arch span is strengthened by post-tensioned, seven-wire-strand steel cables configured in a unique X-shaped network, which enhances stiffness and redundancy in the bridge's superstructure. These cables allow the structure to redistribute some of the arch rib horizontal load, so that the members function similarly to those in a truss structure.

To evaluate stress distribution within the structure under normal conditions, as well as during catastrophic events such as cable loss, a 3D finite element model of the bridge was created. The 3D model was used to refine the construction sequence. Each time the survey points on the arch were measured, the 3D model was updated to obtain data on the actual stresses to the members. The networked cables were carefully adjusted to optimize deck elevations and stress distribution for the structure, based on the results of the 3D model.

The arch tie itself, normally a fracture-critical member, is a box-shaped tension tie that was specially designed to withstand cracking and not collapse. The tension tie was mechanically fastened together with bolts for redundancy, rather than welded together, which enables it to withstand loads, even if one of the four plates that comprise the box fractures.

Plans specified the use of the stringent mill-to-bear method for fabrication of the steel for the arch ribs. The intent was to use the most precise fabrication process available to reduce construction cost. It succeeded in reducing the number of bolts required at the arch rib splices by 50%.

The design also included hybrid steel members of high-strength, high-performance 70 ksi weathering steel, for maximum durability and improved ductility. This also helped to minimize the size and weight of the approach span superstructure, reduce material quantities and construction cost, and defray long-term maintenance costs.

The bridge deck is longitudinally post-tensioned to prevent cracking caused by the lengthening of the tied arch under load. The piers are also post-tensioned to resist cracking.

Construction

Innovative construction methods and techniques were required to account for the challenges presented by the extraordinary weight and size of the structural members, the mountainous terrain, and the riverine environment. In preparation for construction of the tie girders and arch, the contractor constructed eight temporary drilled caissons in the river. The tie girders and arch were constructed in segments, from each main river pier, halfway across the channel. The most efficient method was to construct a significant portion of the tie girders and use this as a base for building out the arch ribs until the cantilevers reached the center of the span.

The construction of the arch was the most complicated task of the entire project because of its size and the need to ensure that the arch segments fit together perfectly. Temporary adjustable stays were used to brace the arch segments during erection, prior to installation of the cable hangers. As each cable hanger was installed, the supporting temporary stay was removed.

The position of the arch was monitored very closely. Elevations were taken after every segment was erected, and the position of the structure was adjusted through the use of the temporary falsework stays provided by the contractor.

The Ohio side of the arch was constructed six inches out of position longitudinally, and then jacked into place during installation of the arch's keystone section. The ends of the arch were temporarily post tensioned to the pier caps to ensure the stability of the cantilevered sections during jacking. Sand jacks with steel shims and polytetrafluorethylene sliders were mounted on top of the river caissons and served as temporary supports. The jacks and sliders also could be quickly and easily removed after the arch was constructed. Large, barge-mounted cranes were used to install the heavy steel segments (which weighed up to 60 tons) for the arch and the West Virginia approach.

The contractor designed a temporary bridge and used a "barge bridge" to cross the back channel of the Ohio River to access the island from the West Virginia shore. On the island, 70-ft-high falsework towers, designed to withstand a 75-mph wind load, supported the girder segments. The towers were anchored by guy wires connected to concrete deadmen embedded in the island soil.

> The use of network cables was very innovative. This bridge showcases complexity on a grand scale and should make a lasting impression over the Ohio River. *—Jury Comments*

The original plans called for the design of a suspension bridge with no piers on the island. To reduce construction costs, the client decided instead to proceed with the design alternative that included piers on the island. Pier placement required extensive archaeological and environmental investigation.

The innovative approach to the design of this project, including the hybrid design of the arch and the efficiencies in material quantities, resulted in significant cost savings to the client and taxpayers, and will also deliver long-term benefits by reducing maintenance costs and extending service life.

The client realized significant cost savings from the design of a networked tied-arch structure. Employing this bridge type further demonstrates its application in the engineering field as a viable, cost-effective alternative to the more commonly used design options. The use of the networked cables enabled engineers to reduce the arch rib size by approximately half and thereby significantly reduce overall construction cost. Also, the use of inclined hangers with stay cables will facilitate future cable replacement.

Owner
West Virginia Department of Transportation, Charleston, W.Va.

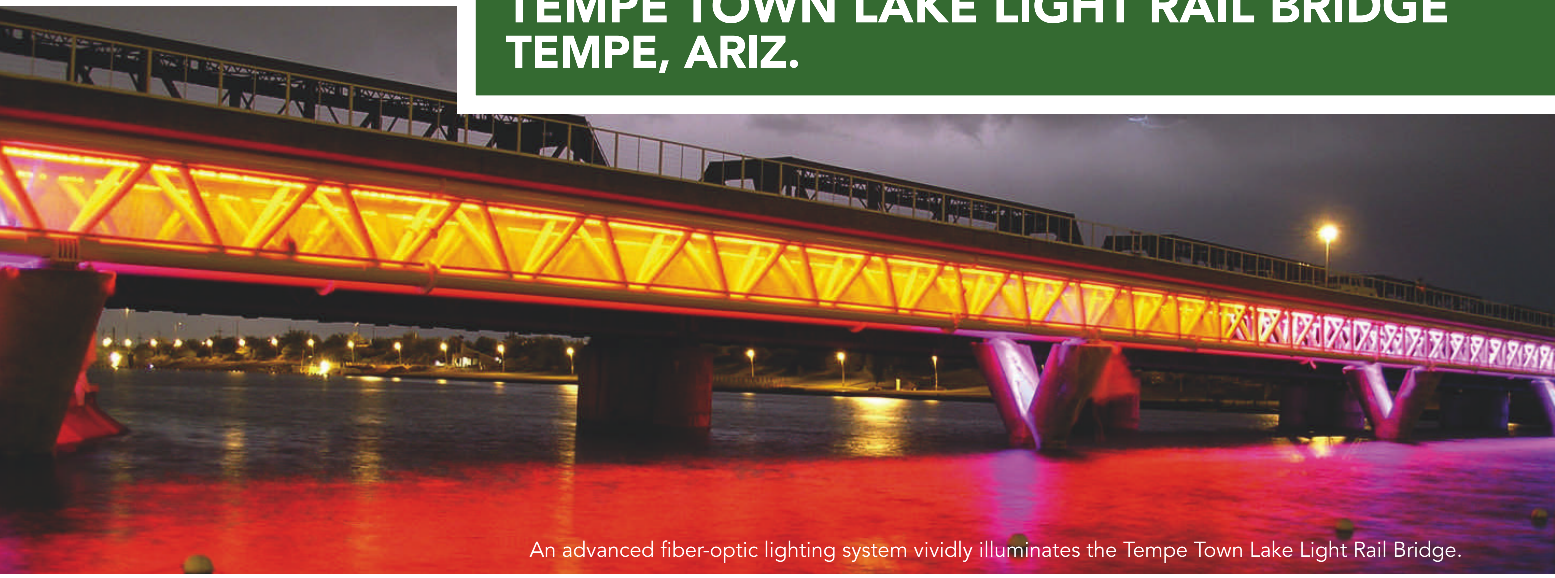
Designers
Michael Baker, Jr., Inc., Charleston, W.Va.; HNTB, New York

General Contractor
Walsh Construction Company, Canonsburg, Pa.

Steel Fabricator (approaches)
Hirshfeld Group, Greensboro, N.C. (AISC/NSBA Member)

Detailer
CanDraft, Coquitlam, B.C. (AISC/NSBA Member)

NATIONAL AWARD MAJOR SPAN
TEMPE TOWN LAKE LIGHT RAIL BRIDGE TEMPE, ARIZ.



An advanced fiber-optic lighting system vividly illuminates the Tempe Town Lake Light Rail Bridge.

Built to withstand a 500-year flood, the Tempe Town Lake Light Rail Bridge spans 1,535 ft and consists of two abutments, 10 Y-shaped piers and 42 steel trusses that feature diagonal pipe bracing connecting top and bottom pipe chords. It was built to provide the Valley Metro Light Rail project, a high-capacity transit system, with a crossing over Tempe Town Lake to connect Phoenix with Tempe and its neighboring communities.

This bridge represents true design innovation because it combines the past with the present. During the day, one can see that the superstructure mimics the truss design of the adjacent historic Union Pacific Railroad Bridge. At night, modern technology emerges for a visual spectacle. Below the 30-ft-wide cast-in-place concrete deck is an advanced fiber-optic lighting system that vividly illuminates the structure with various colors.

During construction, the unique bridge design created several technical challenges, one of which dealt with the complexity of the site conditions. The location of the south approach and abutment had obstacles above and below. The high-voltage power lines overhead presented a significant hazard during construction. Because the large drill rigs on the barge could not fit under the power lines and the power lines could not be de-energized, the abutment design was switched from the original plan of using drilled shafts to a single spread footing. In addition to solving the constructability challenge, this redesign also reduced costs.

Below the abutment, a major water line that supplies downtown Phoenix with 60% of its potable water posed a potential challenge. It was crucial to not disrupt the underground 72-in. diameter pipeline. The project team solved this challenge by carefully excavating and encasing the waterline with concrete reinforcement to withstand the mass of the south approach structures.

Throughout the project, the project team worked closely with local historical and transportation commissions to ensure that the bridge did not detract from the adjacent historical railroad bridges built in 1912. Valley Metro Rail worked with the State Historic Preservation Office through several design concepts from a cable-stayed bridge to the constructed concept where the design consultant borrowed from the old rail bridge design while adding modern touches. Despite the unpredictable issues that came up during construction, the team was able to overcome the challenges with the development and implementation of innovative solutions. Detailed and strategic coordination among design and construction teams enabled the delivery of an on-time and on-budget project.

> Its innovative use of steel pipes creates a powerful and efficient structural system. It is "beauty over water," and the lighting makes it a piece of art. *—Jury Comments*

The bridge follows AASHTO Load Factor Design guidelines, using 50 ksi yield strength steel pipe. A 24-in. diameter steel pipe with a 1-in. wall thickness forms the bottom apex of each triangular truss. Two 18-in. diameter steel pipes about 5 ft apart lie at the top vertices of each truss, directly under a track rail. Wall thicknesses of the pipes vary, depending on whether the truss is in a positive or negative loading configuration. A series of 880 10-in. diameter steel pipe braces in sets of four run diagonally upward from a steel saddle on the bottom pipe to a saddle on the two pipes above, creating the truss. Spacing of the bottom saddles ranges from about 10 ft to 15 ft between longitudinal centers. Truss segments between piers range from about 75 ft to 160 ft long.

The lighting system hangs from horizontal 8-in. pipes, positioned directly above the bottom chord and connecting the two upper pipe chords. Tubular cross-diaphragms connect the parallel trusses at both abutments and at the nine piers. Disc bearings, two on each abutment and two atop each pier, control bridge movement.

The cylindrical piers have the look of being "wood chopped" at the top center, where they form a Y. The bottom pipe of each truss rests on one arm of the Y. All but the center pier accommodate expansion. Total movement for expansion at each abutment is about 5 in. A 46,000-sq.-ft continuous concrete deck placed on stay-in-place decking forms the tops of the two trusses, providing a 30-ft width for the two tracks and emergency walkways. Depth of the truss is 9.25 ft and the overall depth is 11 ft at the top of the rail.

Fabrication was a critical challenge. The main obstacles were weld configurations, open-root welds, rework and material.

Open-root welds, which were required by code, had to be made out-of-position without backing bars. Welders first had to be qualified to make these welds. The welding fabricator's testing left only six qualified for the job. Usually this type of project would require backing strips, but in this case, backing strips would not have made the structure sturdier. They also would have added 20% to 30% more time and cost to the project. Additionally, any necessary rework was limited to two reworks on each weld joint.

The original design specified ASTM A618 pipe in various diameters and wall thicknesses, and because Federal Transit Administration funding was involved, domestic materials were required. By contract, the bridge was to be fabricated in six months—including material procurement. However, the lead time for A618 for several of the size and wall thickness combinations was at least one year. Sources often would not quote some sizes at all or would require a huge mill purchase for each size and wall thickness. Eventually the Federal Transit Administration approved the use of imported materials for two size combinations, which before purchase, were subject to on-site inspection by the fabricator to verify dimensions, form, and traceability.

Another aspect of this construction that at first might have been considered a hindrance, actually was a benefit: segmented fabrication. To optimize structural strength versus weight, meet budget, enable transportation to the work site, and enable a modular erection, the fabricator constructed the bridge in segments. The design consultant decided on the segmented fabrication method as a way to help ensure the bridge would be fabricated on time.

The design consultant also invited the fabricator for an interview and to provide comments and recommendations that eventually helped write the welding specifications. Because Arizona generally uses concrete more than steel, the fabricator helped educate others on the team about welding and developed the majority of the weld design.

Owner
Valley Metro Rail, Phoenix

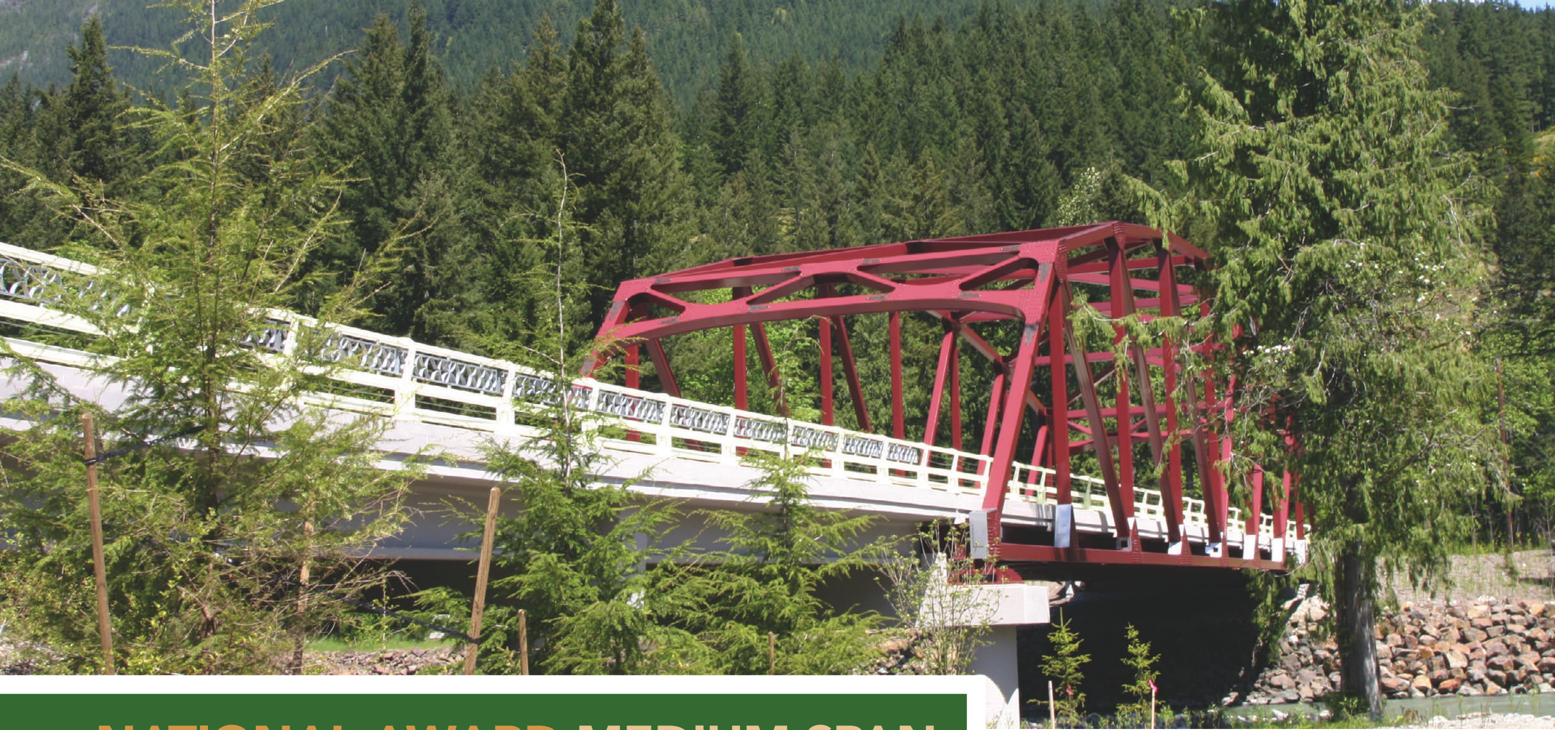
Designer
T.Y. Lin International, Tempa, Ariz.

General Contractor
PCL Civil Engineering, Inc.

Fabricator/Detailer
Stinger Welding, Inc., Coolidge, Ariz.
(AISC/NSBA Member)

Architect
Buster Simpson, Seattle

Electrical/Lighting
R.A. Alcala & Associates, Inc., Tucson, Ariz.



Using built-up box members and HSS sections created a neat and clean appearance for the new Mount Si Bridge. Built-up members in the top and bottom chords are connected with minimally-obstructive, slender hollow structural section tube web members.

NATIONAL AWARD MEDIUM SPAN
MOUNT SI BRIDGE REPLACEMENT KING COUNTY, WASH.

The Mount Si Bridge serves as a vital link across the Snoqualmie River for local residents and as a gateway to regional outdoor activities within the Mount Si Natural Resources Conservation Area, southwest of Seattle.

For more than half a century, the original bridge provided the only access to the community and recreation areas north of the Snoqualmie River. As the second-oldest bridge in King County, and one of its few remaining steel Pratt truss bridges, the Mount Si bridge symbolized the rural community and was designated a county landmark. It also was on the National Register of Historic Places for its engineering and architectural significance.

The structure had severely deteriorated over the years and was listed as a high priority for replacement in the county's 2001 annual bridge report. The structural design team presented eight bridge alternatives for evaluation. Ultimately, another steel Pratt truss bridge was chosen based on cost, ease of construction, and maintenance requirements.

Located in one of the state's most popular outdoor recreation areas, the new bridge had to blend with the natural environment and not be an eyesore, while keeping the scenic attraction of Mount Si in the background. To accomplish this, the design had to be as open as possible.

The design team used built-up box members and HSS sections to create a neat and clean appearance. These built-up members, with top and bottom chords, are connected with minimally-obstructive, slender hollow structural section tube web members.

Other innovations included using rigid moment sway frames with slip-critical type bolt connections and optimizing panel spans at 30 ft, rather than the usual 20 ft to 25 ft, which resulted in using less steel and reducing fabrication requirements.

The new Mount Si Bridge also incorporates art in many bridge elements, including:

- ➜ Ornamental in-fill panels on the approach span railings
- ➜ Landscaping elements surrounding the bridge
- ➜ Decorative bronze plates attached to the bridge structure
- ➜ Bridge and railing paint colors
- ➜ Special finish and color applied to the bridge's sidewalk

Owner
King County Department of Transportation, Seattle

Designer
Andersen Bjornstad Kane Jacobs (ABKJ), Seattle

General Contractor
Mowat Construction Company, Woodinville, Wash.

Fabricator
Jesse Engineering Company, Tacoma, Wash. (AISC Member)

Detailer
MKE Detailing Service, Seattle (AISC Member)

Consulting Firm
3 Ring Services, Seattle

Art elements, such as the decorative bronze plates attached to the bridge structure, combine with the bolt connections to make the Mount Si Bridge a distinctive and aesthetically pleasing structure.

The construction staging to accommodate working with a busy railroad was impressive. Aesthetically, it fits into the neighborhood while providing a unique solution to a unique situation. *—Jury Comments*

NATIONAL AWARD SHORT SPAN
ROSLYN ROAD GRADE CROSSING ELIMINATION
MINEOLA, N.Y.

The tragic death of nine teenagers in 1982, when they drove their van around a properly functioning crossing gate onto the Long Island Rail Road (LIRR) main line tracks and into the path of an oncoming train, gave the village of Mineola, N.Y., the impetus to reinvigorate a 24-year-old desire to eliminate several at-grade LIRR crossings in the community. Today, almost 50 years after the village's initial petition to eliminate the grade crossings, the final chapter of the $180 million plus grade crossing elimination project—the Roslyn Road Grade Crossing Elimination—is complete. The project, including designing and constructing both a new steel structure for the LIRR main line and a depressed Roslyn Road beneath the tracks, has enhanced safety and traffic operations as well as improved the quality of life for village residents.

The LIRR is the most active commuter rail line in the country, and so included challenging design issues. A number of alternatives were investigated that involved raising or lowering either the LIRR tracks or the roadway. Because nearby residents were concerned that raising the railroad would increase noise levels, the railroad was kept at grade and Roslyn Road was depressed to pass below. That decision made designing and constructing the structure for the steel bridge that supports the tracks, as well as the depressed roadway, extremely complex.

The 73-ft steel through-girder bridge, which carries the tracks over Roslyn Road, was extremely challenging to construct, especially because the longest period allowed for a track outage was a long weekend. Three steel through-girders support the two existing main line tracks and allow for a future third track. These girders govern the critical vertical clearance, which is 14½ ft for the bridge.

A unique construction phasing solution was developed that allowed the new bridge and substructures to be built with only four weekend single track outages and two weekend double track outages. After each outage, the tracks were returned to service.

During the first weekend double track outage, large diameter steel casings were augered into the ground, then filled with concrete to act as foundation piles. At a much later time, during

Rolling this new bridge into place eliminated the most dangerous at-grade crossing on a busy commuter line and completed the effort begun 50 years earlier.

four weekend single track outages, four temporary steel trestles were installed atop the concrete piles. These temporary trestles provided support for the tracks while excavation and construction of the abutments occurred below the trestles.

After the substructures were completed, the second weekend double track outage was implemented. The new bridge had been constructed adjacent to the tracks while the substructure work was progressing. During this weekend, the new bridge was rolled into position and placed onto the new abutments. After the final weekend outage, the tracks were returned to service.

The public's safety, welfare and quality of life all have been improved, either as primary or secondary benefits of this project. The project's most significant benefit is its elimination of the very serious safety issue of rail and vehicle conflicts, particularly important given more than 200 trains per day running through the village—some at speeds exceeding 80 mph.

Prior to the project, peak hour gate closures caused major traffic backups, which, in turn, resulted in a breakdown of function at the adjacent intersections. This traffic congestion, with cars idling, caused pollution, excessive energy consumption, aggravation for those caught in the backup, and a temptation for some to avert the gates. Additionally, the at-grade crossing caused noise pollution, with train horns blasting each time a train approached the crossing. Had the project not been undertaken, anticipated expansion of LIRR operations would only have exacerbated these problems. Instead, the project completion has eliminated them. With gasoline prices at today's levels, the removal of traffic congestion and idling makes the project even more cost-effective, bringing economic as well as environmental benefits to the community.

Owners
Long Island Rail Road and New York State Department of Transportation

Structural Engineer
Stantec Consulting Services Inc., New York

General Contractor
Posillico Civil Inc., Farmingdale, N.Y.

Fabricator
Francis A. Lee Company, Syosset, N.Y. (AISC/NSBA Member)

Prior to the grade separation both traffic and safety were serious issues at this crossing.

courtesy of Top Shots

The monumental $680 million Woodrow Wilson Memorial Bridge project, under construction on the Capital Beltway (I-495) for more than a decade, was undertaken to eliminate one of the nation's worst traffic bottlenecks.

NATIONAL AWARD SPECIAL AWARD
WOODROW WILSON MEMORIAL BRIDGE
WASHINGTON D.C.

The Woodrow Wilson Memorial Bridge is truly a new icon in a city of monuments. The $680 million project replaces an outdated bridge carrying I-95 across the Potomac River connecting Maryland and Virginia at the southern tip of the District of Columbia. It is a vital link on I-95 and the Capital Beltway (I-495), the circumferential freeway surrounding the core of the Washington metropolitan area. The new state-of-the-art structure eliminates one of the nation's worst traffic bottlenecks. The 12-lane bridge has separate local and express lanes, and capacity for future mass transit expansion. It also contains America's largest movable span.

The previous bridge had a vertical clearance of only 50 ft, but its drawspan over the Potomac River's navigational channel allowed larger marine vessels access to Washington, Alexandria, and other points north of the bridge. The decision was made to build new drawbridges rather than a higher fixed-span structure because many commercial, Navy, Coast Guard and recreational vessels on the river require high clearances. A fixed bridge would have required a vertical clearance of 135 ft.

The previous double-leaf bascule span bridge opened an average of five times per week. The new drawbridge is about 20 ft higher than its predecessor, reducing the number of bridge openings each year from approximately 260 to less than 60.

> This monumental bridge is packed full of innovation and is a trailblazer in the land of leaders. The engineering elements are amazing. This was a stimulus package before there was a need. The number of jobs created was incredible. It is an elegant, visually stunning bridge with good lines that enhances the surrounding architecture.
>
> *—Jury Comments*

The project includes two parallel bridges, each consisting of eight plate girders and three to four substringers to accommodate widths of up to 148 ft. Each bridge consists of two parallel double-leaf bascule spans for a total of eight leaves, which keeps the floor system and mechanical and electrical systems economical. By not connecting adjacent leaves, and providing separate machinery with the ability to operate each leaf independently, any one of the leaves can be taken out of service, if required, while maintaining a minimum of three lanes of traffic in each direction.

Each of eight drawspan leaves weighs approximately 2,000 tons and is designed to close within a ⅛-inch tolerance. Thirty-four million pounds of structure will move to clear a ship through the channel, representing the largest moving mass of any bridge in America and possibly the world. With 270 ft between trunnions, this span is among the longest in the world. It also is extremely wide: 249 ft from fascia to fascia.

The bascule span is a simple trunnion Chicago-type bascule. The front transverse beams of the piers serve as supports for the forward live load bearings at each bascule girder. The fixed deck beam of the bascule pier also serves as the rear live load anchor. Other design features of the bascule include a fully-composite lightweight concrete deck, fully counterweighted leaves, shear and moment-transferring span locks, and tail locks.

The span lock arrangement for the new Woodrow Wilson Bridge is unique in that the locks transfer moment as well as shear between the leaves of each double-leaf span. The tail locks work in conjunction with the span locks and relieve the operating machinery of live load transferred through rack into the main pinions. This will significantly reduce wear on the operating machinery.

The design of the bridge was decided by competition. The signature bridge that resulted from this process is an elegant, curving, haunched plate girder bridge supported by V-shaped piers. The combination of the curved V-piers and the girder haunches highlights the architectural motif of arches desired by the public. The steel plate girder/diaphragm/substringer framing system was used for overall economy, aesthetics and compatibility with the V-pier configuration.

The floor system framing and detailing were kept as simple as possible. Each bascule leaf consists of two bascule girders that support floor beams and stringers. Girder-to-

courtesy of Potomac Crossing Consultants

Thirty-four million pounds of structure move to clear a ship through the eight-leaf bascule arrangement on the Woodrow Wilson Memorial Bridge over the Potomac River.

girder distances vary for different leaves, ranging from 35 ft to 40 ft, 6 in. The typical floor beam spacing is 20 ft, 9 in. and stringer spacing is kept under 6 ft. Girders and floor beams are welded I-shaped members, and the stringers are rolled sections. Bolted connections are used throughout the span.

In all, 16 bascule girders are required. These girders are very large, with webs varying in depth from nearly 12 ft at the toes to 20 ft in the vicinity of the trunnions, and with 28-in.-wide flanges that range between 1½ in. and 4 in. thick. The overall length of each girder is 215 ft. To keep girder segments within sizes and weights that could be fabricated and to provide shipping and erection options, the girder design included two field splices. Each bascule girder weighs between 350 tons and 400 tons.

Approach Spans

The approaches on each end of the bridge consist of two continuous units, with 13 individual spans on the Virginia side and 19 spans on the Maryland side. They use haunched plate girders having a depth of 11 ft, 9 in. at the support points and 6 ft, 10 in. at midspan. The parabolic shape was developed to provide the continuous curved line of the V-pier and the superstructure varies with the span length.

The variable-depth girders in conjunction with the V-shaped piers provide the arch-like appearance that was desired in order to be visually similar to the other great bridges in the capital city. The plate girder spans vary from 100 ft to 209 ft. This variation in span length is due, in part, to the height of the structure above the ground surface. Plate diaphragms support the substringers and provide a clean appearance from the historic park below the bridge.

The plate girders were designed as hybrid girders. They were primarily fabricated from ASTM A709 Grade 50 steel, but some flanges used Grade 70 HPS steel to minimize the plate sizes, reduce girder weight and minimize constructed cost.

Co-Owners
Maryland State Highway Administration, Baltimore
Virginia Department of Transportation, Chantilly, Va.

Designer
Parsons, Baltimore

General Contractor (bascule)
American Bridge (AISC/NSBA, IMPACT and TAUC Member)/Edward Kraemer & Sons (IMPACT Member) Joint Venture, Coraopolis, Pa.

Detailer (bascule)
Tensor Engineering, Indian Harbour Beach, Fla. (AISC/NSBA and NISD Member)

Consulting Engineer (bascule superstructure design)
Hardesty & Hanover LLP, Annapolis, Md.

General Engineering Consultants
Potomac Crossing Consultants, Alexandria, Va.

Fabricator – Virginia Approach
Williams Bridge Company, Manassas, Va. (AISC/NSBA Member)

Fabricator/Detailer – Maryland Approach
High Steel Structures Inc., Lancaster, Pa. (AISC/NSBA and IMPACT Member)

It's a unique structure, a graceful and attractive bridge, created with a one-of-a-kind approach. The construction of a curved bridge with straight members added an interesting element. *—Jury Comments*


© Mark McCabe, 2009, courtesy HNTB

The Bob Kerrey Pedestrian Bridge provides 53 ft of clearance for Missouri River traffic and serves as a landmark for riverfront development in both Omaha, Neb., and Cedar Bluffs, Iowa.

NATIONAL AWARD SPECIAL PURPOSE
BOB KERREY PEDESTRIAN BRIDGE
OMAHA, NEB.

The Bob Kerrey Pedestrian Bridge spans the Missouri River connecting the cities of Omaha, Neb., and Council Bluffs, Iowa. At more than 2,300 ft, the structure is one of the longest pedestrian-only bridges in the U.S. Visually transparent but dynamic and innovative, its curvilinear design gives it a signature look and makes it a visual icon for the area.

The bridge's length made design and construction a complex matter, with wind-induced vibrations a particular concern. By keeping the girders relatively shallow and allowing a 4-ft gap between the edge girders and deck, designers provided for aerodynamic stability and kept wind-induced vibrations to a minimum. The girders are approximately 23 in. deep on a 24-ft-wide steel frame, and the 16-ft-wide deck is 12 in. deep at the curbs and 3 in. elsewhere. Had the girders been deeper, they would have been stiffer and stronger, but they also would have increased the potential for wind-induced problems.

The shallow girders reduce the wind load, but require closely-spaced cables to further support the deck. Because the cables are more steeply angled than traditional cable-stayed bridges, the axial load on the girders is reduced.

Another innovation used by the design-build team was the foundation design, with each pylon supported by a single drilled shaft. Minimizing the use of temporary works, the team drilled shafts that are 13 ft in diameter and extend roughly 85 ft into the riverbed. And because the foundations were constructed within the steel casing that forms the shaft, no cofferdams were required.

With a strict $22 million budget, the design-build team made each of its decisions within the context of economic feasibility. That started, quite literally, from the foundation up: Rather than take additional borings, the team chose to place the foundations in locations where subsurface investigations had already been completed. Other cost-saving measures included the use of the balanced-cantilever method, which reduced the need for falsework, and foregoing traditional cofferdam construction in the river.

But no decision was more important than the decision to use steel. The design-build team needed an efficient and lightweight structure to stay within budget and determined that choosing steel over concrete was the answer for many reasons, including a reduction in:

- ➜ The size of the foundation
- ➜ The size of the cables
- ➜ The magnitude of the windload
- ➜ The amount of falsework

The bridge's signature look is its curvilinear design, which is complemented by a pair of three-sided pylons that pierce 203 ft into the air and transform the area's skyline. Two planes of cables are connected to those pylons, and those 80 cables—40 per pylon—range in diameter from 1.25 in. to 2.3 in. Spacing the cables closely together—roughly 23 ft apart—and increasing the height of the pylons to decrease the cable angles provided enough clearance along the curved deck for a wide variety of users as well as a maintenance vehicle.

The LED lights at the top of each pylon make the bridge an aesthetic marvel. The lighting is dramatic, with different colors and effects at different times of the night.

The first bids on the project, in 2004, came in at more than twice the budget. City officials later re-issued the RFP, specifically asking for a design-build contract and a bridge that was "architecturally significant." However, the team had less than six months to design it.

The first step was choosing Grade 50 fracture-critical steel to create the superstructure. The final design consists of a horizontally curved bridge with a 506-ft main span and two 253-ft back spans. The superstructure bends from one side of the first pylon to the opposite side of the second pylon, spanning a total of about 2,300 ft.

The bridge's innovative design enabled builders to use straight steel segments instead of curved segments. The bridge is set on a radius, and all dimensions are based on that radius. Although the bridge alignment is curved, the superstructure segments and pre-cast deck panels are not. Instead, steel sections are straight-edged and identical in size and shape—with one side slightly longer than the other—and arranged to create the "S" curve.

Each section of the bridge is fairly short—approximately 23 ft—and its concrete deck is skewed slightly to create the curves along the bridge length. The long superstructure segment consists of a 24-ft-wide steel frame and a 16-ft, 4-in. precast concrete deck panel.

In effect, the main span of the bridge is really a series of short spans. Even the railings, piecewise, are straight. The use of straight-piece sections is one of the most innovative features of the bridge, and it couldn't have been done with any material other than rolled steel. Foregoing the heat-curving process that would have been necessary with shaped steel segments, the design-build team saved time and money with the straight sections, assuring an on-time and on-budget delivery.

Construction of the bridge through a design-build contract began in October 2006. The bridge opened to the public in September 2008, two months ahead of schedule and within its strict $22 million budget.

Owner
City of Omaha, Neb.

Architect and Designer
HNTB Corporation, Kansas City, Mo.

General Contractor
APAC-Kansas, Inc., Kansas City, Kan.

Fabricator/Detailer
DeLong's Inc., Jefferson City, Mo.
(AISC/NSBA Member)

The two replacement skewed bascule spans include all new steel superstructures and mechanical and electrical systems.

NATIONAL AWARD MOVABLE SPAN
HAMILTON AVENUE BRIDGE
BROOKLYN, N.Y.

The rarely seen Hanover skew bascule, also known as a knee-girder bascule bridge, is a unique and complex movable structure in terms of both design and construction. The replacement of a movable bridge during an accelerated construction period is also an incredibly difficult task to engineer and construct. Either one of these constraints would make a project difficult to execute. For the Hamilton Avenue Bridge project in New York City, however, these two levels of complexity combined to create a one-of-a-kind project that would challenge the owner, designers and constructor to achieve a near impossible goal: to replace a skewed bascule bridge with a new, fully operational span in 64 days.

The existing Hamilton Avenue Bridge was constructed in 1945 based on the novel knee-girder bascule design. The bridge comprises two separate single-leaf bascule bridges with each leaf carrying four lanes of Hamilton Avenue over the Gowanus Canal, a fixed span at the north pier which carries the roadway over the bascule pit, and a fixed arch span at the south end. The approaches have steep grades to meet the existing local street network of Brooklyn before and after the canal crossing. The East Span carries traffic northbound to the Brooklyn Battery Tunnel and lower Manhattan and the West Span traffic southbound to lower Brooklyn. The bridge carries approximately 55,000 vehicles per day with a high percentage of truck traffic.

As part of its ongoing bridge evaluation and maintenance program, the New York City Department of Transportation (NYCDOT) conducted an in-depth evaluation of the existing bridge in 1998. The bridge superstructure possessed a number of nonconforming roadway features (lane widths, bridge railings, etc.) making it functionally obsolete, so NYCDOT decided to replace the two skewed bascule spans with new steel superstructures and mechanical and electrical systems.

One of the key unique aspects of this project is the structure type itself. Only four knee-girder bascule spans were constructed in the U.S. based on the patented design of Clinton D. Hanover. The Hamilton Avenue Bridge was the first and is one of only two remaining in existence. The knee-girder framing provides an efficient means to span a skewed waterway with a single-span bascule bridge and alleviates a number of the disadvantages of this type of bridge, such as large differential loads in the supports and a non-uniform counterweight.

The project included tight time and work zone constraints. During two closure periods in July and August of consecutive years, the existing bascule span and approach superstructure of each span was demolished and replaced with the new structure.

To meet the schedule requirements, the contractor used an innovative temporary operating system for the bascule spans. The use of hydraulic cylinders and a hydraulic power unit enabled the contractor to decommission the existing bridge's electrical control system and machinery in advance of the roadway closure period, permitting many components to be partially or completely disassembled in advance. Much of the wiring and electrical components contained asbestos, so the contractor used this initial phase for the abatement of these hazardous materials. Their removal at this early and non-critical-path phase permitted the critical path tasks to proceed without delays due to worker safety or environmental issues.

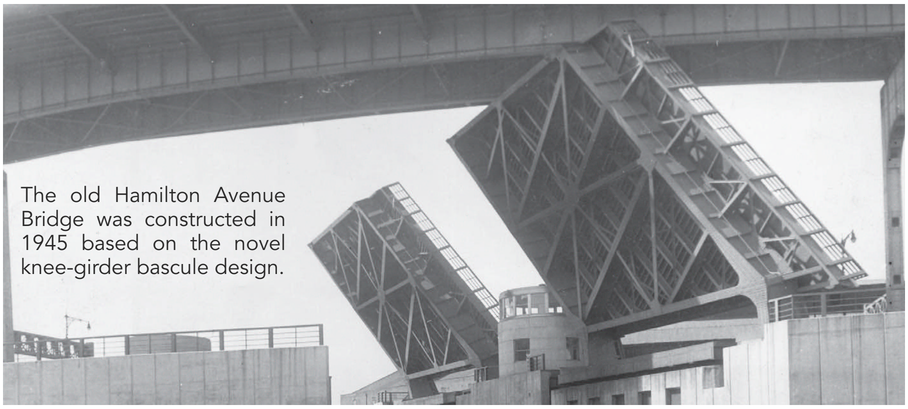
The temporary hydraulic drive system was also used for the new bridges to ensure span operation at the end of the closure period while allowing the time-intensive gear and machinery alignment to be performed outside of the two-month closure period.

Owner
New York City Department of Transportation

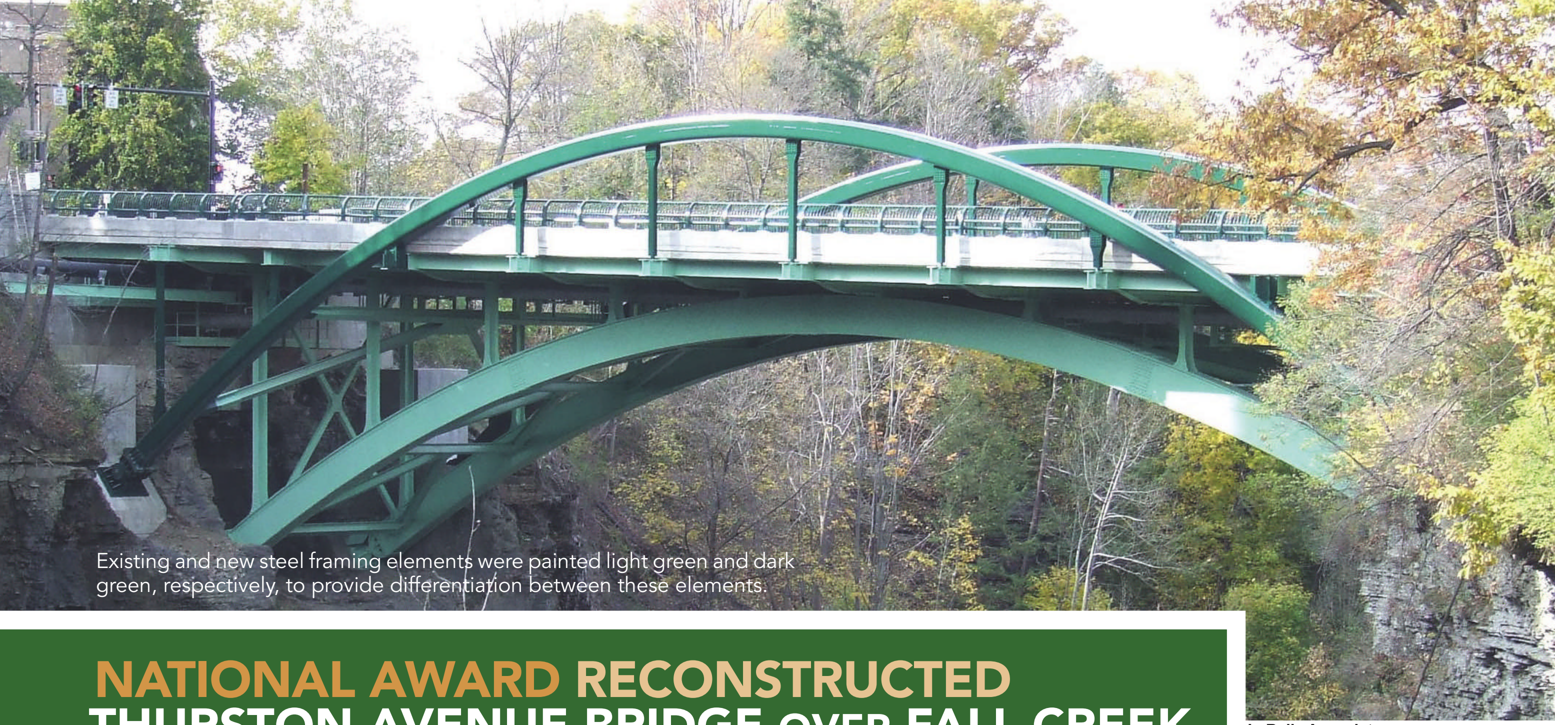
Designer
Hardesty & Hanover, LLP, New York

Consultant
Greenman-Pedersen, Inc., Babylon, N.Y.

General Contractor
Kiewit Constructors, Inc., Park Ridge, N.J. (IMPACT and TAUC Member)



The old Hamilton Avenue Bridge was constructed in 1945 based on the novel knee-girder bascule design.


Existing and new steel framing elements were painted light green and dark green, respectively, to provide differentiation between these elements.

LaBella Associates

NATIONAL AWARD RECONSTRUCTED
THURSTON AVENUE BRIDGE OVER FALL CREEK ITHACA, N.Y.

The Thurston Avenue Bridge over Fall Creek in Ithaca, N.Y., is built over scenic gorges and has a long and interesting history dating to the late 1800s. Ezra Cornell and his associate Andrew Dickson White capitalized on the famous "Ithaca is Gorges" slogan to bring students to their new university in 1868. Recognizing the significance of the setting and reputation of Cornell University, the City of Ithaca and the New York State Department of Transportation (NYSDOT) implemented a first-of-its-kind design to retain a bit of history in combination with a bit of invention for the rehabilitation of Cornell's primary link.

Originally a trolley bridge, the 215-ft long crossing now serves more than 34,000 students, faculty, and staff as the "gateway" between the residential and academic campuses of the university. However, severe congestion was causing pedestrians to walk in the travel lanes as well as vehicle delays at the approach intersection. The bridge's capacity had to be increased, but with due respect to its heritage.

The solution was to widen the bridge by 12 ft by adding new induction bent tubular arches at each fascia to provide for 10-ft-wide sidewalks and 5-ft-wide bicycle lanes. The new arches were elevated so that the existing arches remained visible.

The final parabolic curvature of the new arches was designed to meet constraints posed by a number of factors. The location of existing floor beams for column and hanger connections helped determine the locations where the arches rise above the deck. The height of the crown was determined by the owner's desire to allow views to the gorge and to discourage climbing.

The 32-in. by 30-in., 1-in.-thick tubular shape the designer arrived at was larger than any standard tube section produced in the U.S. and so had to be custom fabricated. The tubes also had to be bent into a parabolic curve, incorporating field-welded splices to maintain a continuously smooth appearance for the entire length of the arch rib.

Fabrication began by cold bending two 50 ksi, 1-in.-thick flat plates into U shapes that were then welded together with complete joint penetration seam welds. The 20-ft tube sections were fed through an induction bending machine that heated the steel to 1,850 °F. The curvature was introduced as it was pushed through at 1.5 in. per minute.

Each arch was erected by first setting the end pieces followed by the center piece. The splice ends were fabricated with a backing tube that allowed the crown section to be dropped in without springing the two sections.

Three cranes held the arch sections in place for approximately 16 hours until temporary stand-offs and new bracing struts were connected and complete joint penetration butt welds at the splices were finished and tested.

The new arches are filled with nitrogen gas to provide an internal corrosion protection system. The gas was pumped into the arch replacing all of the air inside and sealed with a slightly positive pressure. Permanent pressure gages ensure pressure loss does not occur.

> The great lengths that were taken to mesh a new structure with the historic one should be considered legendary. The bent tubular arches are a graceful element and used an innovative fabrication solution. The color treatments were exceptional. *—Jury Comments*

Owner
City of Ithaca, N.Y., Department of Public Works

Designer
LaBella Associates PC, Rochester, N.Y.

General Contractor
Economy Paving, Inc., Cortland, N.Y.

Fabricator
BendTech, Inc., Duluth, Minn. (AISC/NSBA Member)

Reducing the skew and increasing the span for the new Salmon River Bridge reduced ice jamming potential but also required innovative design features.

MERIT AWARD LONG SPAN
ROUTE 151 OVER THE SALMON RIVER
EAST HADDAM, CONN.

The original three-span through-girder steel bridge at this challenging river crossing had served well for many years but was the site of frequent ice jams. However, the more serious problem was that riverbed scour had undermined its spread footings. Rather than disturb the sensitive environment with scour countermeasures, the decision was made to replace the bridge with a new structure that would span the entire river channel, thus preventing ice jams and reducing scour potential. Even so, the new foundations were designed to accommodate an anticipated 26 ft of scour and included end-bearing steel H-piles driven to bedrock.

Although the original bridge alignment spanned a total of 219 ft and was skewed 60 degrees to the channel, the new span was set at 250 ft with a 20 degree skew. Reducing the skew angle meant the cross frames could be laid out on the skew, which would greatly decrease the potential for skew-induced torsion. It also enabled the use of skewed deck reinforcement, which simplified construction.

Weathering steel was selected for the corrosion protection system, which at the time required and received public approval.

The new bridge girders were the first in Connecticut to be designed according to LRFD bridge design specifications. The girders had a large span-to-depth ratio, which would have required very thick bottom flanges. However, designing them as hybrid sections using Grade 70 steel for the bottom flange reduced the size of the flanges to a maximum thickness of 2.25 in. The web design was optimized so that transverse stiffeners were required only in the first 25 ft of the span.

Many of the design's key features were based on NSBA Steel Bridge Collaboration documents, including the following details:

- ➜ Inverted K-type cross frames, used without top horizontals
- ➜ Skewed cross frame connection plates
- ➜ Weathering steel drip bars
- ➜ Bolted splice plates
- ➜ Elastomeric bearings

The result of the design and detailing was that the cost for the structural steel was very reasonable. The bid price for the steel, fabricated and erected, was $1.28 per pound, which was very reasonable for bridges in Connecticut.

The innovative design did not stop at the girders. Large-scale round elastomeric bearings were designed to accommodate potential torsional rotation brought on by the large deflections and skew. Anchor bolts were only used at the fixed bearings. The expansion bearings are connected to the girders, but simply rest on the abutment seats. Lateral restraint is provided by concrete keeper blocks between two of the girder flanges. The 25-in.-diameter bearings were most likely the largest elastomeric bearings ever used in the state. The bid price for the bearings was much less than an equivalent high load multi-rotational bearing.

Another innovation was the use of the empirical design method for the composite deck design. The LRFD design greatly reduces the amount of deck reinforcing, which was run along the skew of the bridge.

The overall cost of the bridge portion of the project was approximately $3.9 million for a deck area of 9,159 sq. ft. This results in a unit bridge cost of $425 per sq. ft—a very high value, even for bridges in Connecticut.

Based on the strict scour criteria, the cost of the substructure and foundations was significant. The unit cost of the superstructure alone was $133 per sq. ft, approximately 25% less than typical superstructure costs in Connecticut. This is especially significant considering the size of the girders in the bridge section.

The original steel bridge lasted more than 73 years, serving well with virtually no maintenance, and was replaced only because of scour issues. The new Salmon River Bridge is the next generation of steel bridges that have the potential to serve the department for the next 100 years with minimal maintenance.

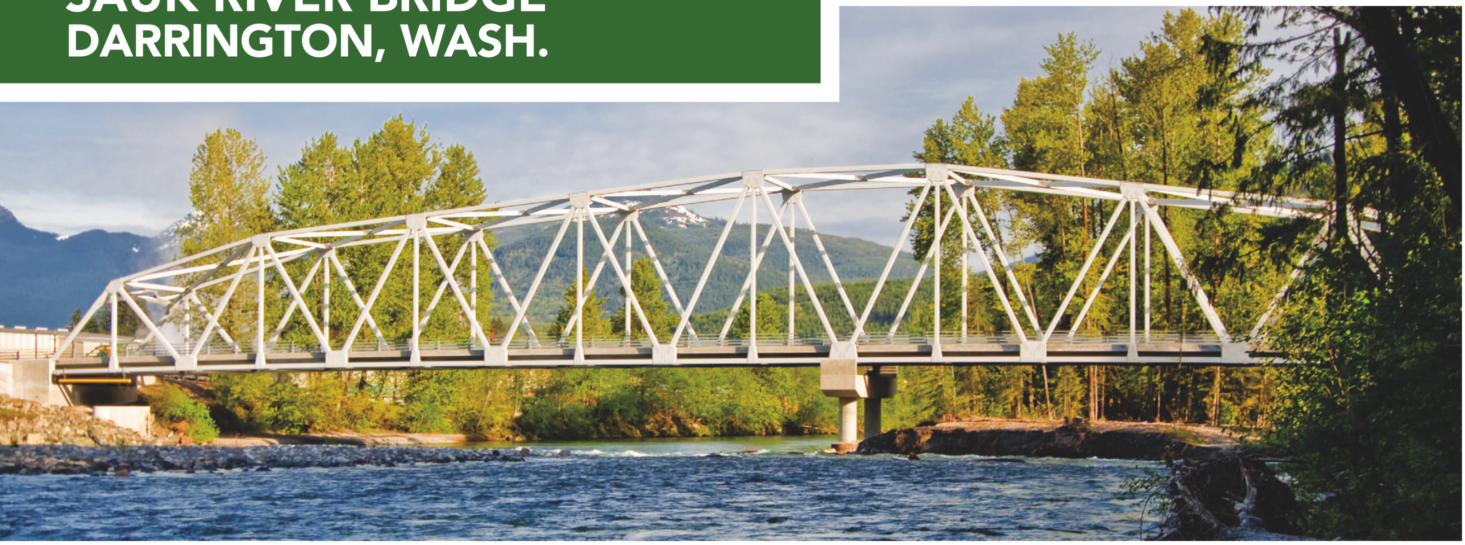
Owner
Connecticut Department of Transportation, Newington, Conn.

Designer
CME Associates, Inc., East Hartford, Conn.

General Contractor
Baier Construction Company, Inc., Bloomfield, Conn.

MERIT AWARD MEDIUM SPAN
SAUK RIVER BRIDGE
DARRINGTON, WASH.

Replacing the Sauk River Bridge near Darrington, Wash., required protecting one of the most wild and scenic rivers in the country.



Spanning a federally designated "wild and scenic" river, the Sauk River Bridge fords one of the most spectacular whitewater rafting and fishing stretches in the country. Built in 1930, the existing two-truss steel bridge served as the only access to Darrington, Wash., and its main employer, the Hampton Logging Mill, from the Sauk Prairie area east of the river.

But the bridge was extremely narrow and dangerous, especially for truck traffic. Determined to be both functionally obsolete and structurally deficient, its overhead clearance, bridge curb-to-curb width, and structural load carrying capacity did not meet current standards. In addition, the west pier of the bridge was scour critical and was considered extremely vulnerable to one of the most energetic hydraulic environments in the state.

Carrying two lanes of traffic and providing a wide pedestrian shoulder, the new two-span steel truss bridge is the county's longest at nearly 479 ft and is composed of a continuous truss with a main span of 266 ft. Built on a new alignment just downstream from the existing bridge, it features drilled-shaft, scour-proof foundations as deep as 125 ft to ensure survival during extreme spring and winter floods. It now handles an average daily traffic of 750 vehicles, 25% of which are heavy logging trucks. It also provides a dramatic stopping point for tourists on the Mountain Loop Highway, viewed against a backdrop of snowcapped Whitehorse Mountain and other nearby Cascade peaks.

Numerous challenges faced the project team. Rugged, cramped conditions, raging water, an adjacent lumber mill, and river migration patterns severely limited placement and construction options for a new bridge. Environmental regulations required that the bridge be built without any temporary supports in the river, and within an unusually tight fish window for in-water work. Road traffic would have to be maintained at all times because the bridge provided sole access to and from Sauk Prairie. Additionally, the existing bridge was eligible for being listed on the National Register of Historic Places (NRHP), complicating the removal process.

Many original solutions make this project both successful and noteworthy. Designing the bridge to be continuous for the structure self-weight (dead loads) and the forces imparted by traffic and the environment (live loads) allowed longer spans to be achieved and improved material efficiency. Advanced 3D modeling techniques incorporated into both design and construction allowed members to be optimized for cost reduction and resulted in greater geometrical precision during fabrication, which greatly reduced the potential for erection and launching difficulties.

Careful siting and design minimized right-of-way issues, yet will also accommodate future river movement. An innovative launch technique allowed the bridge to be constructed on shore and cantilevered into place, which minimized environmental impacts. Other environmental considerations included hot-dip galvanizing and powder coating the bridge, a first for the region; temporary erosion control measures during construction; longer pier spans to accommodate river migration; and a suspended access work platform and protective system to keep debris from entering the river during new bridge construction and as the old bridge was demolished.

An interpretive kiosk at the bridge site mitigates loss of the previous bridge, increases historic awareness, and enhances the town's status as a destination on the Mountain Loop Highway.

Design innovations incorporated into the launching scheme saved approximately $1 million in construction costs and about five months in construction time.

Owner
Snohomish County Public Works, Everett, Wash.

Designer
Berger/ABAM, Federal Way, Wash.

General Contractor
Mowat Construction Company, Woodinville, Wash. (IMPACT Member)

Fabricator
Rainier Welding, Inc., Redmond, Wash. (AISC Member)

Detailer
Pro Draft, Inc., Surrey, B.C. (AISC, NISD Member)

Having the girder continuity splices for the Three Springs Drive Bridge on the intermediate support simplified bearing details and removed making the splices from the critical path.

Owner
West Virginia Department of Transportation, Division of Highways, Charleston, W.Va.

MERIT AWARD MEDIUM SPAN
THREE SPRINGS DRIVE BRIDGE WEIRTON, W.VA.

The Three Springs Drive Bridge over U.S. Route 22 in Weirton, W.Va. was designed using simple for dead load, continuous for live load (SDCL) steel girder construction. The project involved replacing the existing structure with one carrying five 12-ft traffic lanes, two 3-ft-wide shoulders and a 5-ft-wide, raised sidewalk with an 8-in. concrete deck for a total width of 73 ft, 4 in. The deck is supported by seven 54-in.-deep weathering steel plate girders spaced at 11 ft, 2-in. with spans of 125 ft, 6 in. and 95 ft. Span lengths were dictated by the configuration of U.S. Route 22. The girder depth was based on preliminary depth studies. K-type cross frames were provided at intermediate locations and temporary X-type cross frames were used at the supports until the deck was cured.

The steel girders were placed as simple spans to resist non-composite forces. After placement of the deck in both spans, flange splices and a concrete continuity diaphragm at the pier were constructed to provide continuity for composite dead and live loads. The structure is supported at the ends by jointless, integral abutments founded on steel H-piles.

SDCL construction was accomplished by splicing the top and bottom flanges of the simple span girders at the interior support location after placement and curing of the deck. The deck was placed to within 5 ft of the centerline of bearing at the abutments and centerline of the pier, which minimized non-composite forces on the continuity splice.

Constructing the new bridge using staged construction adjacent to and overlapping the existing bridge was determined to be the most practical and economical option to maintain traffic for this on-alignment replacement. This option reduced traffic congestion during construction and eliminated the need to construct a temporary bridge, thereby reducing time and cost.

Due to simplified and expedited fabrication, erection, simplified traffic control, and cost effective design and detailing, both schedule and cost were minimized. The SDCL detailing in conjunction with the pier continuity splice, weathering steel and fully integral abutments will help minimize maintenance and extend the life over conventional fully-continuous steel plate girder structures of similar size.

Designer
HDR Engineering, Inc., Pittsburgh

General Contractor
Ohio-West Virginia Excavating Company, Shadyside, Ohio

Fabricator/Detailer
Ohio Structures, Inc., Canfield, Ohio (AISC/NSBA Member)

MERIT AWARD SPECIAL PURPOSE
DR. MARTIN LUTHER KING, JR. MEMORIAL BRIDGE
ROANOKE, VA.

Roanoke's historic First Street Bridge was removed, refurbished, reinstalled and rechristened as a memorial to Dr. Martin Luther King, Jr.

The City of Roanoke, Va., wanted to preserve a historic but deteriorating 19th century steel truss bridge that spans the Norfolk Southern rail lines by renovating the existing vehicular structure for pedestrian use only and enhancing its approaches to create a memorial to civil rights leader Dr. Martin Luther King, Jr.

Formerly known as the First Street Bridge, the structure consists of three 53 ft, 10-in. approach spans on the south, a 100-ft main truss span, and a single 53-ft, 3-in. approach span on the north. The deck surface was an asphaltic concrete overlay of 3-in. timber decking supported on 6-in. by 14-in. timber stringers and built-up steel floor beams. The main load-carrying superstructure members of the approach spans consist of built-up steel through-girder sections while the main span is a steel Warren pony truss.

After bridge ownership was transferred from the railroad to the city in the 1990s, the city decided to replace the existing First Street crossing with a new bridge at Second Street and convert the existing bridge to pedestrian use only.

The built-up floor beams were the critical members, so they were replaced with new rolled W-sections. All other components retained more than the necessary capacity for the pedestrian live load.

In-place rehabilitation was impractical because of the active railway below the main span. Engineers developed a plan for removing the deck and stringers, carefully dismantling the steel members, and repairing, then strengthening and painting them off-site. This also kept the removal of lead-based paint in a completely controlled and monitored environment.

Owner
City of Roanke, Va.

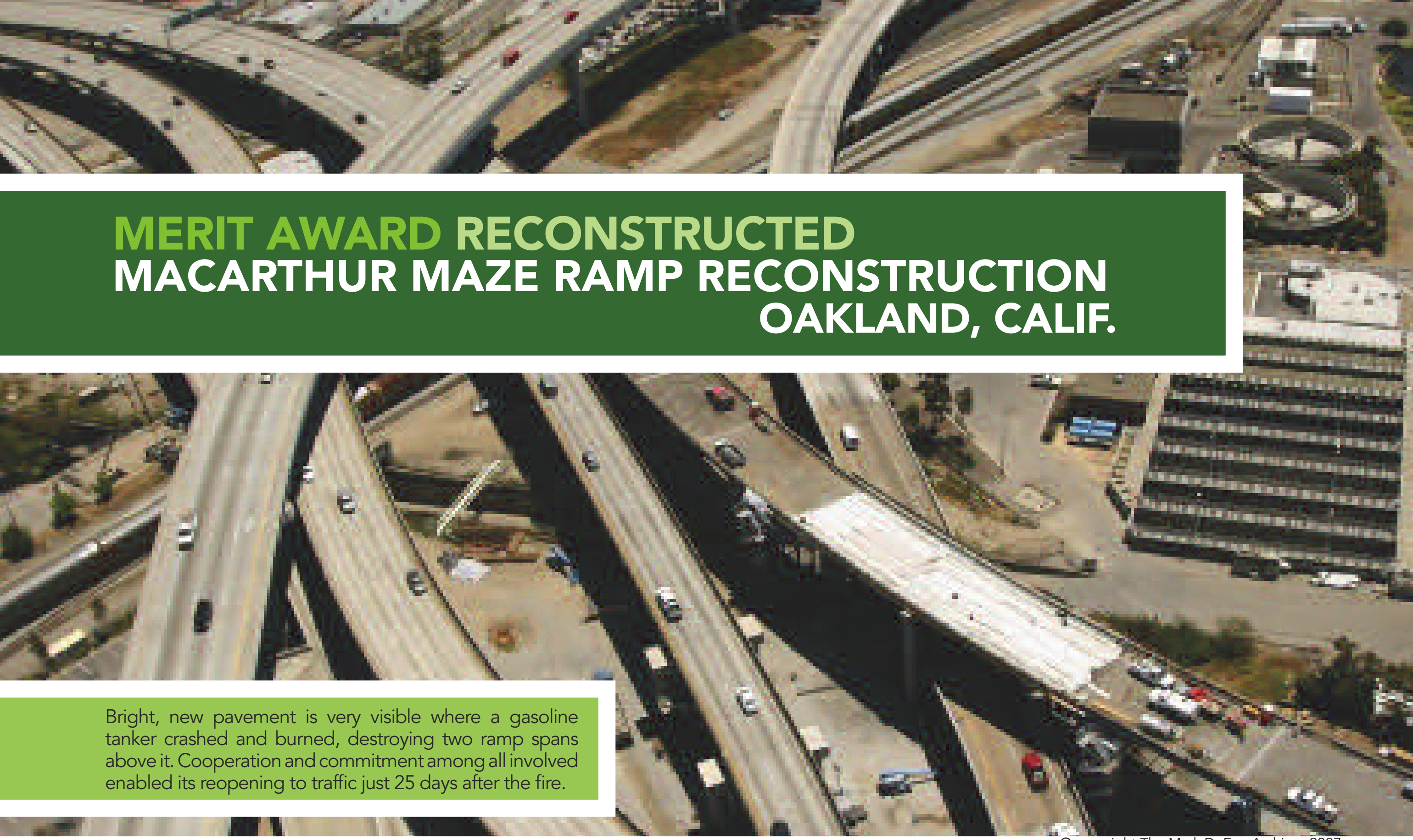
Designer
AECOM, Roanoke, Va.

General Contractor
Allegheny Construction Company, Roanoke, Va.

Fabricator/Detailer
Structural Steel Products Corporation, Clayton, N.C. (AISC/NSBA Member)

Consulting Firm
Hill Studio, Roanoke, Va.

MERIT AWARD RECONSTRUCTED
MACARTHUR MAZE RAMP RECONSTRUCTION
OAKLAND, CALIF.



Bright, new pavement is very visible where a gasoline tanker crashed and burned, destroying two ramp spans above it. Cooperation and commitment among all involved enabled its reopening to traffic just 25 days after the fire.

The I-80/I-580 MacArthur Maze Ramp is a vital link between Oakland, Calif., and San Francisco. At 3:41 a.m., on April 29, 2007, a tanker truck, carrying 8,600 gallons of fuel and traveling southbound on the lower ramp, overturned on the bridge deck and skidded directly beneath the upper level connector ramp.

The 1,500+ °F heat from the free-burning gas fire caused the steel box bent cap as well as adjacent spans to collapse onto the lower level connector ramps directly below. The collapsed portion, a total of 160 ft long and 45 ft wide, included the six steel girders in both spans and the steel bent cap.

Within hours, bridge officials were meeting to set priorities and engineers were on site assessing the damage. Steel plate girders and a precast prestressed concrete bent cap were designed to replace the collapsed portion of the structure. Heat straightening would be used to repair the lower ramp.

The reconstruction plans, specifications, and engineer's estimate ($5,140,070) were completed by the design team within three days. Caltrans was motivated to complete the project as safely and quickly as possible, so the project was advertised with a $200,000 per day incentive/disincentive clause capped at $5 million to reward contractor innovation. In addition, the contractor would be fined $200,000 for every 10 minutes that lane closures were picked up late.

Bids were opened May 7, nine days after the accident. The contract was awarded on the same day to general contractor C.C. Myers, who had arranged to work with Stinger Welding, for the bid price of $867,075.

Within two hours Caltrans began discussions with the steel fabricator on its first critical path item. Within 24 hours, Caltrans had a senior reviewer full-time at the fabricator's shop to provide immediate guidance for welding and shop plans.

Three days later, representatives from Caltrans, C.C. Myers, and Stinger, the AISC/NSBA fabricator, conducted a pre-welding meeting to discuss steel welding and fabrication quality. By the end of the meeting, Caltrans was satisfied with the fabricator's plan and fabrication began.

Stinger fabricated the 12 girders in eight days. Six truckloads took the girders and diaphragms to Oakland for construction. The concrete deck was designated for a 96-hour compressive strength of 3,600 psi prior to directly supporting construction loads, allowing fast track deck placement and a bridge re-opening earlier than originally scheduled.

Caltrans' contract set a construction completion deadline of re-opening on June 27. The work was completed on May 24, 2007, after a mere 15 days on site, earning the contractor the maximum incentive of $5 million.

Innovative concepts incorporated into this project include:

- ➜ The girder web thicknesses were increased to reduce the number of stiffeners required for local buckling checks and the amount of welding required on the built-up girders.
- ➜ The web depth was adjusted to ensure that the overall depth would not require adjustment of the existing bearings that were to be reused.
- ➜ The flange plates were kept to one size to simplify the fabrication.

A professionally made 29-minute documentary on the reconstruction is available at **www.amazingmaze.org**. MSC

Owner and Designer
California Department of Transportation, Sacramento, Calif.

General Contractor
C.C. Myers, Inc., Rancho Cordova, Calif.

Fabricator/Detailer
Stinger Welding Inc., Coolidge, Ariz. (AISC/NSBA Member)

Photo used with permission from Golden Gate Bridge, www.goldengate.org

Durable **Bridge** Coatings

Successful Today and Getting Even Better for Tomorrow

BY ERIC KLINE

COATINGS NOW BEING SPECIFIED AND APPLIED to steel bridges are completely different from and perform much better than those applied until about 1965. That said, there is still the often articulated perception that bridge painting is expensive, troublesome, and that bridge coatings simply "do not last." However, one does not have to look far to find strong evidence to the contrary. In 1965, the Golden Gate Bridge in San Francisco became one the first large bridges to be painted using a modern zinc-rich, steel bridge paint. It provides an outstanding example of how well today's coatings protect bridges across America and around the world.

Prior to 1965 coatings were generally oil- or alkyd-based and contained pigments using lead and/or chromium compounds as the corrosion inhibitors. In addition, they often were applied directly to steel covered with shiny, slick mill scale that had been subjected to only power tool cleaning (SSPC-SP 3 "Power Tool Cleaning") for surface preparation. The old axiom was "the more paint the better," as additional coating thickness meant that more inhibitive pigment was applied to resist corrosion. These old-technology coatings were expected to last about eight to ten years before requiring some level of maintenance intervention. As a result, there were so many coating layers on some bridges that apart from other forces, the sheer weight of the paint would overcome the adhesion of the coating layers to one another and/or to the smooth mill-scale-covered steel beneath. Subsequently, the coating would simply fall off, sometimes in sheets. Coatings with an overall thickness of ¼ in. or more have been encountered.

Bridge owners and maintenance engineers still are living with the complex issues brought about by the 100-plus years of use of this long-ago-discontinued coatings technology. A recent survey of 20 state departments of transportation (DOTs) regarding bridge painting practices revealed that, in those states, only about half of the bridges originally painted with lead-based paint have been repainted. Repainting in this context refers to complete removal and replacement of all old coating. That leaves thousands of bridges with the "old" coatings that must be addressed in the future. Accordingly, the removal/replacement expense, often for

Beginning about 1965, the lead-based paint that had protected the Golden Gate Bridge in its early years was removed and replaced with an inorganic zinc-rich paint system.

Photo used with permission from Golden Gate Bridge, www.goldengate.org

Four decades after being primed with the inorganic zinc-rich paint but never topcoated, these members on the north end of the bridge continue to be in very good condition.

coatings containing lead, will continue in many states. In many areas, the current cost is around $12 per sq. ft of steel surface area, including access, lead removal, containment, worker protection, disposal, repainting, etc.

Modern Era Technology

There is good news for the owners of the bridges built or repainted with modern-era coatings, meaning those made available since about 1965. Around that time, many DOTs began specifying the use of blast cleaning to a near white condition (SSPC-SP 10) in order to completely remove mill scale. They also began applying a "new generation" primer.

The coatings introduced at that time employed an entirely different technology than earlier products. They contained metallic zinc powder as the pigment providing corrosion resistance. Why zinc? When zinc and iron (or steel) are joined in presence of moisture and oxygen (air)—a corrosive environment—zinc will be consumed first, and the iron (and steel) will be protected from corrosion. This consumption of the zinc will continue until the available zinc is depleted.

The innate ability of zinc to protect steel from corrosion is referred to as "galvanic" protection. This provides long-lasting protection because the zinc reaction normally occurs at a fraction of the rate of corrosion of bare steel in the same environment.Many everyday items are galvanized, including fencing, guard rails, sign structures, light standards, and even automobile parts.

Basically, zinc can be applied to steel in three ways. Galvanizing is a process in which the bare piece of steel is dipped in molten zinc. It is limited by the size of the "kettle" in which the article is immersed. Metallizing requires melting wire containing zinc and spraying the molten metal onto the steel surface, with a stream of air. Both galvanizing and metallizing are excellent means of protecting steel from corrosion. However, many steel bridge members and components are best protected by the use of zinc-rich paint, which is the focus of this article.

Service Life of Zinc

The metallic zinc pigment in zinc-rich paint is able to provide galvanic protection for the steel until the zinc itself is consumed. When the zinc is depleted, the steel will eventually rust. Therefore one important consideration is how long the zinc will last. The time it takes for zinc to be consumed is affected by many variables, e.g., weather, duration of wetness, the number of wet/dry cycles encountered, etc. The service life of zinc-coated items often is measured in decades. In a recent article, the American Galvanizers Association (AGA) projected that hot-dipped galvanized (HDG) items will last 75 to 100 years in an aggressive marine environment.

There are a number of important differences between zinc-rich coatings and galvanizing. In zinc-rich paint part of the coating consists of binder materials whereas galvanizing is 100% zinc. The AGA data for HDG are based on a bare zinc coating. A plus factor in terms of service life for zinc-rich coatings is that they are often paired with additional coating layers (topcoats). These additional layers protect the zinc by limiting the amount of moisture and oxygen in direct contact with the zinc. The extensive and impressive 40-plus-year field performance history of zinc-rich coatings, in combination with the AGA calculations, suggests strongly that steel which has been properly coated with a zinc coating and which has additional coating layers can provide permanent or nearly permanent protection for the steel beneath.

The Gold Standard

The current "gold standard" for bridge coating entails the use of a three-coat system consisting of an inorganic zinc-rich primer, an epoxy midcoat, and a urethane topcoat (IOZ/E/U). Literally thousands of steel bridges constructed since about 1965 are coated with a zinc-rich primer paint as a part of a paint system and are in excellent condition.

One such structure is the world famous Golden Gate Bridge (GGB). This structure measures 8,981 ft long (1.7 miles), weighs about 887,000 tons, has two massive towers that stand 746 ft-tall and a roadway about 220 ft above the Golden Gate Strait. When the bridge was built, from 1933 to 1937, it was coated with lead-based paint. Through an extensive undertaking from 1965 to 1995, the lead paint was removed and an inorganic zinc-rich paint system was applied.

Some areas on the north end of this iconic suspension bridge structure were primed with IOZ, but never topcoated. These areas were recently examined and were in very good condition. According to Dennis Dellarocca, the bridge's paint

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KTA-Tator Inc.

This coating damage is the result of destructive adhesion tests on the east exterior fascia girder of the Windgap Bridge, near the north abutment. All indicate excellent adhesion.

superintendent, there are no plans to disturb the corrosion-free coating that has been in place for more than 44 years.

Another example of good long-term coating performance is the Windgap Bridge near Pittsburgh, Pa. This 849-ft-long, seven-span, composite steel, multi-girder bridge carries Windgap Road across Chartiers Creek. This Allegheny County-owned bridge is being protected by the 23-year-old coating system applied during its construction in 1986.

The coating system consists of an organic zinc-rich primer, an epoxy midcoat, and a urethane topcoat (OZ/E/U). When the bridge was evaluated in 2007, the coating was found to be in excellent condition. The overall rate of coating breakdown was very low and confined to areas beneath leaking joints plus a few tiny areas damaged by rock-wielding vandals. Some minor graffiti was also noted. Accordingly, the 23-year-old bridge paint was in need of only a small amount of touch-up around the bearings.

The Martin Luther King Bridge in Richmond, Va., provides another example of excellent long-term IOZ-based coating system performance. This 2,000-ft long bridge has six lanes plus two sidewalks and rises 100 ft as it crosses Interstate 95 and the Shockoe Valley. When the bridge was constructed in 1975, the orthotropic steel girders were painted with an inorganic zinc-rich primer and vinyl coatings layers.

A recent examination of this structure revealed that the coating system was in excellent condition overall. There are a few areas with apparent loose or debonded topcoats, aggregating a tiny percentage of the steel surface. These small areas are in need of touch-up attention, but only from a cosmetic perspective as only a small amount of rust is evident, indicating that the IOZ coating material is still performing its intended function—corrosion protection.

An example of one of the older zinc-coated steel bridges is in Franklin County, Mo. Known as MoDOT Bridge No. A2107, this two-lane, 185-ft-long bridge on Route E crosses Pin Oak Creek. It was painted in October 1969 with an inorganic zinc-rich system; the coating was examined in 1999. At that time the coating condition was very good.

The bridge was overcoated in 2000 with a calcium sulfonate topcoat as part of the state's bridge maintenance program. Having been recently overcoated, the bridge should be well protected for decades to come.

These long-ago painted bridges are illustrative of the thousands of painted structures constructed over the past 40-plus years whose coatings have already stood the test of time. With periodic paint touch-up and overcoating the primer will be able to provide complete corrosion protection for decades to come, likely for the life of the structure, perhaps extending a century, or more.

Modern Painting Costs

The cost of painting in the shop as part of the initial fabrication is about $1.50 per sq. ft, far less than the cost of full lead paint removal and repainting in the field. Maintenance overcoating in the field, where no lead paint remediation is required, currently costs about $1 per sq. ft.

After overcoating a zinc-rich primer based coating system, it is expected that the bridge will not need to be painted for 15-25 years. At that time, after a now-total service life of about 55 years, another overcoating is possibly required, costing an additional $1 per square foot.

The costs for such a bridge, in 2009 U.S. dollars, are summarized below in Table 1:

Lifetime Cost Per Sq. Ft (Three-Coat System)	
Year 1	$1.50 for the initial blast cleaning surface preparation and prime coating in the shop during fabrication.
Year 1	$2 for the application of the second/third coating layers at the construction site after steel erection.
Year 30	$1 for the first touch-up/overcoating.
Year 45-55	$1 for the second touch-up/overcoating.

Table 1

There are ways to reduce even these modest costs. U.S. Federal Highway Administration research and other testing has shown that the performance of newer two-coat paint systems, while lacking the 44-year field history of the three-coat "gold standard" coating systems, are possibly capable of equaling its performance. If a two-coat system were to be widely used, the lifetime costs would be expected to be similar to those shown in Table 2:

Lifetime Cost Per Sq. Ft (Two-Coat System)	
Year 1	$2.50 for the initial blast cleaning surface preparation and application of both coats of paint in the shop.
Year 30	$1 for the first touch-up/overcoating.
Year 45-55	$1 for the second touch-up/overcoating.

Table 2

Caution: Periodic Maintenance is Required

It is unlikely that any steel bridge can be painted and simply remain untouched for its entire service life, extending perhaps a century or more. No epoxy or urethane coating currently known to the author is likely to be able to perform well for that long. Current topcoat materials generally will serve very well for about 20-30 years, at which time at least a first touch-up/overcoat is expected.

There are three good reasons for adding additional protection at that time. First, there likely will be some reduction of the gloss and/or fading of the color of the topcoat due to weather (sunshine, rain, air pollution). Second, there likely will be locations on the bridge where traffic or wind-blown debris have nicked, scratched, or otherwise damaged the coating. Finally, girders and bearings beneath leaking joints often are bathed in corrosive salt-laden water from storms or from winter deicing activities.

During the touch-up/overcoating operation, all such locations can be repaired and the entire structure can then be completely overcoated. In this scenario the zinc-rich coating, which provides the basic corrosion protection, is not disturbed in the repair/overcoating process. Consequently, the zinc layer will remain beneath the existing coating and any new coating(s) applied during the touch-up/overcoating process. It is expected that this zinc-rich paint layer should be able to perform its corrosion resistance function for the life of the structure.

Note that other very good zinc-rich primer coating systems currently are available, and widely used, in addition to the IOZ/E/U system discussed above; however, it is that "gold standard" system that already has stood the test of time, since at least 1965, and is widely specified and successfully used.

Rapid Deployment

Two areas of technology have been developed in recent years to assist in field cleaning and repainting low-rise overpass bridges located in high traffic areas that cannot be shut down for long periods. These practices, dubbed "rapid deployment," involve the use of mobile, blast cleaning equipment and containment platforms—usually flat-bed trailer mounted—that are used to completely enclose the area to be cleaned and painted. The mobile platform can be deployed overnight to enable cleaning and painting to occur. The trailer is removed from the roadway before rush hour begins the next morning.

The second aspect of a rapid deployment approach is the use of a matching two-coat coating system consisting of a fast-curing organic zinc-rich primer along with a fast-curing high-build topcoat. Using this tandem approach allows the contractor to mobilize the platform, clean an area and apply both coating layers in an overnight shift, thereby completing the work on that area. Economies associated with rapid deployment are readily apparent.

Coatings Prequalification

In days gone by, the myriad vendors in the coatings industry offered the bridge engineering community many materials. Each state was forced to provide its own prequalification test program and to develop and maintain a qualified products list (QPL). Testing by every state was expensive and duplicative.

Since then standard test protocols have been developed under the auspices of the AASHTO National Testing Product

Tom Calzone

This 1999 photo shows the excellent condition of the inorganic zinc-rich coating on bridge A2107 in Missouri 30 years after its application.

Evaluation Program (NTPEP). Suppliers of coatings to the bridge painting industry are now required to have their products tested in accordance with the AASHTO NTPEP testing standard for Structural Steel Coatings (SSC). In these laboratory "torture tests," the performance of candidate bridge coating systems is evaluated using tests identical to those required to qualify the IOZ/E/U system described above.

Test results are accumulated in the AASHTO DataMine which is available to state DOTs for the purpose of coating system comparison. Each state can apply its own performance criteria. For example, a state with a mild, less-corrosive environment may have different criteria for adding a coating system to its QPL. Information about the AASHTO NTPEP coating testing program can be found online at **http://www.ntpep.org/ContentManagement/PageBody.asp?PAGE_ID=30**.

The New England states also have their own separate prequalification testing standard by which materials are prequalified and listed on a QPL accepted by the member states and several others. The NEPCOAT member states are: Connecticut, Massachusetts, Maine, New Hampshire, New Jersey, New York, Pennsylvania, Rhode Island, and Vermont. The NEPCOAT Qualified Products Lists can be found online at **http://www.state.me.us/mdot/nepcoat/qualprod.htm**. Approved two-coat systems like that discussed above for rapid deployment are presented in NEPCOAT Appendix C.

Many states use either/or the AASHTO DataMine or NEPCOAT for coating performance data. See **http://www.state.me.us/mdot/nepcoat/**.

Conclusion

Bridge engineers are properly cautious professionals who are charged with safely building and protecting our modern infrastructure from attacks of all kinds, including corrosion. For nearly a century following construction of the Eads Bridge in St. Louis, which heralded the beginning of the steel bridge era, bridge engineers did the best they could to protect steel bridges with various coatings systems. Since the advent of the modern age of bridge coatings, in 1965, many improved user-friendly, color-retentive, adherent, corrosion-preventing and durable coatings have emerged from the coatings industry. Literally thousands of improvements have been made in every aspect of bridge paint and painting leading to improved durability.

Effective means of corrosion protection via corrosion preventive protective coatings have proved themselves in the field for more than 44 years. Progress is steadily being made toward the development of even better, more durable, safer, and more cost-effective coatings, ensuring that there always is a solution in steel. MSC

Keast & Hood Co.

Keast & Hood Co.

Surrounded by Science

BY THOMAS J. NORMILE, P.E.,
AND AMANDA GIBNEY WEKO

An urban research campus expands its knowledge base with a new, steel-framed addition.

Keast & Hood Co.

Supporting connections for the precast panels were welded to columns in the fabricator's shop, greatly reducing field welding requirements.

Keast & Hood Co.

With the seats for the wall panels already in place on the columns the panels could be put in place much earlier in the process.

opposite page: Selecting an all-steel frame rather than using concrete framing for the garage and below, as originally considered, saved approximately $4 million and 10 weeks of construction time. The building uses about 2,200 tons of structural steel in all.

Jeffrey Totaro

All the steel columns in the enclosed and mechanically ventilated garage were encased in concrete for impact protection.

PHILADELPHIA'S UNIVERSITY CITY SCIENCE CENTER has been, in a word, prolific. Set within the University City neighborhood near leading science institutions the University of Pennsylvania, Drexel University, Children's Hospital of Philadelphia, and the Wistar Institute, the nation's first urban research campus has created more than 16,000 jobs. Its resident companies have raised more than $400 million in venture capital and other funding in the last five years alone, and the Science Center has grown to 15 buildings on 2 million sq. ft of real estate.

The latest component of the complex, 3711 Market Street, was completed last year. This urban redevelopment project is a flexible wet laboratory facility with science and technology incubator services available on-site. The new building contains ground-level retail and five stories of structured parking capped with four more stories of flexible laboratory and office space. The building's state-of-the-art laboratory infrastructure, coupled with the Science Center's unique business incubator services, help small and growing life-sciences and high-technology companies get to market faster and more efficiently

Time and Money

Initial design concepts for 3711 Market Street suggested two types of structural framing: steel for the upper laboratory floors and concrete for the garage levels and below. Comparative options were prepared by structural engineers Keast & Hood Co. for both concrete and steel framing on the lower levels. The construction manager's estimates showed that the steel scheme would save approximately $4 million and 10 weeks of construction time. Given the need for economy and speed-to-occupancy, the steel structure was immediately more attractive to the entire team. The steel design also was quite efficient, with an average of 10 lb of steel per sq. ft of building area. The typical column bay size is 31 ft 6 in. by 31 ft 6 in. This works well for both the typical lab bench module and parking stall. On the garage levels, these typical bays are framed with W18×35 beams at 10 ft 6 in. on center and W21×55 girders on the column centerlines. On the laboratory floors, beam sizes jump to W18×40 and girders to W24×84. Beams and girders were initially cambered between ¾ in. and 1 in.

Speed to Occupancy

Schedule was one of several challenges faced by the project team. Complex funding and land-use agreements resulted in a short window of time between concept and occupancy. Once the structural system was selected, the design team was charged with producing a full steel bid package in seven weeks, beginning in June 2006—no small task for a 420,000-sq.-ft, nine-story, mixed-use building. Successfully meeting the schedule required a multi-faceted approach, including a streamlined design process and the use of building information modeling (BIM) software.

Integrated project delivery contributed to the project's efficiency and speed. Keast & Hood Co. used RAM Structural System and RAM CADstudio, BIM software packages from Bentley designed to streamline development of analytical models and construction documents. Structural engineers used RAM Structural System to prepare a complete, three-dimensional model of the structure; perform analysis and design for gravity, wind, and seismic forces; and evaluate floor vibrations. The model was exported to AutoCAD using RAM CADstudio for creation of two-dimensional construction documents. Once the design was optimized, a structural team went to work finalizing details. Structural engineers created floor plans, a column schedule, and braced-frame elevations straight from the computer model.

Design Complexity

Another project challenge was the accommodation of demanding architectural design features. The front entrance canopy cantilevers approximately 12 ft over the sidewalk, does not align with any floor framing or interior building structure, and features a very thin architectural profile. To meet the architects' expectations, Keast & Hood Co. developed an innovative framing solution through extensive finite element modeling with STAAD software.

Several building skin systems added to the design complexity. There are two types of curtain walls with different profiles and window mullions on the building. A composite, insulated metal "sandwich" panel encloses the penthouse while a single-skin panel system is used on the north wall. Complementing the metal panel systems are brick-clad precast concrete and thin-stone veneer on

Jeffrey Totaro

The 3711 Market Street entrance canopy cantilevers 12 ft out over the sidewalk, requiring an innovative framing solution.

Keast & Hood Co.

To accommodate the variety of building skin systems on this project, numerous details were developed, such as this galvanized tube for the front façade.

structural steel framing, as well as stucco on cold-formed steel framing. The slab edge varies in distance from 1 ft, 6 in. to 2 ft from column centerlines.

Extensive edge details were developed to accommodate the different skin systems. To maintain project economy, the edge details used small steel angles and tees to efficiently support the decks during slab placement. Once the slab cured, it acted compositely with the light steel sections to support live loads and the curtain wall attachments. This detailing saved more than 80 tons of steel that would have been required if additional beams at the slab edge had been incorporated. In total, the building used approximately 2,200 tons of steel.

Preventive Measures

As steel was also chosen for the garage framing, attention turned to preventing the deterioration so common in parking garages. Empirical research showed that some of the most common corrosion-related deterioration in steel garages occurs near the perimeter where garage floors take on rainwater. Coincidentally, the owner and architects were interested in a more refined aesthetic than would have been possible with the conventional open-sided garage approach. Hence, enclosing and mechanically ventilating the garage made sense from a variety of perspectives and was readily adopted.

Additional protection methods included partial reinforcement of the slabs on steel decking and a silane sealer applied on all the garage slabs. By using epoxy-coated reinforcing bars at the top of the slab over all the beams and girders, crack widths were kept to a minimum, thereby enhancing the water-resisting performance of the silane. Within the garage, all columns were encased with concrete for impact protection, and HSS $12\times6\times\frac{5}{16}$ steel tubes were employed as car barriers at all perimeter slab edges.

In the Shop

The steel fabricator, Cives Steel Company, contracted Ted F. Duggan and Sons, Inc. to detail the garage's complex geometry. Duggan's computer drawings were used to generate CNC files for precise cutting, burning, drilling, and punching of materials within the Cives shop; accuracy and efficiency were maintained throughout as a result. To further accelerate the project, Cives was contracted to install precast wall panel connections in the shop, saving valuable field labor and allowing the precast panels to be installed on site much earlier.

In the Field

While the rest of the design team was finalizing its drawings, steel fabrication was under way and foundation construction sped along in the field. By spring 2007, steel erection took place in four, full-height lifts. Limited available space on the urban site required careful sequencing. Material deliveries were scheduled only as needed to avoid unnecessary congestion on the site.

All members of the project team leveraged their experience and expertise to optimize design and construction without compromising safety or serviceability. Optimization resulted in both tonnage and bids well below pre-construction estimates. The project's sustainability features were recognized with U.S. Green Building Council Leadership in Energy and Environmental Design (LEED) registration, and 3711 Market Street is a LEED Registered Building at Level 2.0 for Core and Shell. The project was awarded LEED silver certification in October 2009. MSC

Thomas J. Normile is a structural engineer and principal with Keast & Hood Co. and served as principal in charge of the 3711 Market Street project. Amanda Gibney Weko is a design writer and communications consultant based in Philadelphia.

Owner
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Architect of Record
Zimmer Gunsul Frasca Architects LLP, Los Angeles

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UJMN Architects + Designers, Philadelphia

Structural Engineer
Keast & Hood Co., Philadelphia (AISC Member)

Steel Fabricator
Cives Steel Co., Winchester, Va. (AISC Member)

Steel Erector
Steel Suppliers Erectors, Wilmington, Del. (AISC Member)

Steel Detailer
Ted F. Duggan and Sons, Inc., Chattanooga, Tenn. (AISC Member)

Construction Manager
Intech Construction, Philadelphia

Specifying Buckling-Restrained Brace Systems

BY KIMBERLEY ROBINSON, S.E.

Using the ductility of steel effectively in concentrically braced frames.

THE TERM BUCKLING-RESTRAINED BRACE (BRB) has become more common in the past few years, appearing in construction magazine articles and conference presentations. The system, the buckling-restrained braced frame (BRBF), has been used more frequently in seismic applications.

The 2008 AISC T.R. Higgins lectureship awardees were honored for their paper on the topic. BRBFs are a codified system covered by both ASCE/SEI 7-05 and ANSI/AISC 341-05. Yet even after so much recent information has appeared on this topic, many engineers still ask: "What is a BRB? Why consider using a BRBF? How do you specify this system?"

Anatomy of a BRB

The main characteristic of a BRB is its ability to yield both in compression and in tension. It is manufactured with two main components that perform distinct tasks while remaining de-coupled. The load-resisting component of a BRB is a steel core restrained against overall buckling by an outer casing filled with concrete, which is the stability component or restraining mechanism. These elements are illustrated in Figure 1. Bonding of the steel core to the concrete is prevented in the manufacturing process to ensure that the BRB components remain separate to prevent composite action that would change the behavior. Otherwise, the BRB would behave like a composite brace, which would still be expected to buckle.

The BRB is placed in a concentrically braced frame, which thus becomes a buckling-restrained braced frame (BRBF) lateral force resisting system. This system typically is used for structures in seismic demand category D, E, or F, regardless of whether wind or seismic loads govern the design of the structure. BRBF systems also have been explored for low seismic applications.

BRBF systems exhibit robust cyclic performance and have large ductility capacity, which is reflected in its seismic response factor R of 8 when the beams in the lateral force resisting frame are moment connected to the columns; R of 7 is applicable when they are not. Testing performed on BRBs to date has suggested that BRBs may even be capable of withstanding multiple seismic events without failure.

Designing and Specifying a BRBF

The design of a BRBF system is straightforward. Engineers typically use the Equivalent Lateral Force procedure provided in ASCE/SEI 7, unless a more rigorous analysis method is selected. The approximation of the structural period T_a should use C_r and x values from Appendix R of ANSI/AISC 341-05, because these values were mistakenly omitted from ASCE/SEI 7-05. A good reference on the methodology of designing with BRBs is *Seismic Design of Buckling-Restrained Braced Frames*, the paper that merited its authors Walterio López and Rafael Sabelli, the 2008 AISC T.R. Higgins lectureship award.

What Should be Included in BRBF Design Drawings?

One of the questions frequently asked on BRBF projects is what information must the structural engineer of record (SER) include in the design drawings to obtain the intended performance. Certain information is necessary to ensure that BRBs can be accurately estimated, priced, detailed, and erected. This includes BRB quantities, sizes, lengths and end connection types. Other information is necessary to ensure that the BRBs provided meet the design intent and are adequate for the seismic response of the structure. This includes design factors and maximum allowable strength adjustment factors. Clearly, it is in the best interest of the SER to communicate design assumptions, acceptance criteria, and interpretation of the requirements of ANSI/AISC 341-05.

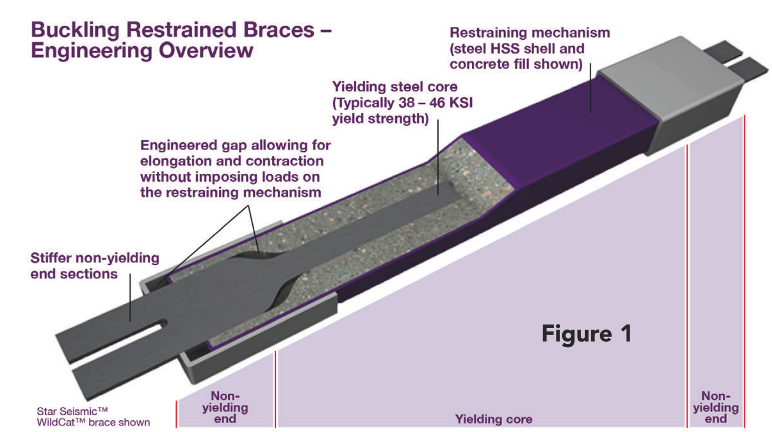


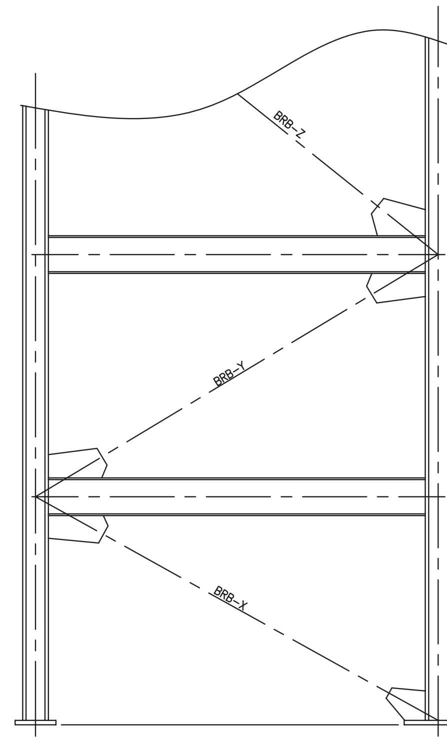
Figure 1

Kimberley Robinson, S.E., is the chief engineer with Star Seismic, Park City, Utah. The company designs and builds buckling-restrained braces for earthquake and seismic resistance for all types of structures.

The following list describes what to include in the design drawings to help make the project a success. Figure 2 provides an example of a BRB Schedule that effectively communicates several of these items.

1. *Seismic design parameters and analysis procedure employed.* Information such as the values of R, C_d, I, and ρ used, and that the analysis was conducted using the equivalent lateral force procedure or nonlinear dynamic analysis, is important in the accurate determination of design brace strains.
2. *Permissible range of steel core yield strength,* F_{ysc}. A range of 38 ksi ≤ F_{ysc} ≤ 46 ksi is generally the accepted practice. However, it is advisable to contact a BRB manufacturer to discuss the recommended range. See Figure 2, note 3.
3. *Permissible variability in BRB required strength.* There are two options for complying with the BRB strength requirements in AISC 341. Option 1 involves maintaining a constant steel core area (A_{sc}) and allowing F_{ysc} (and P_{ysc}) to vary as stated above. Option 2 involves allowing F_{ysc} to vary and compensating by adjusting A_{sc} such that P_{ysc} remains constant. Option 2 results in lower BRB overstrength but also results in a wider variation of BRB stiffnesses. BRBs with identical specified strengths may have stiffnesses that vary by as much as 15 to 20%. If not controlled, this may result in a different load distribution than what was assumed in the design phase, which can lead to unintentional soft stories or torsional behavior. See the table in Figure 2 and schedule note 2.
4. *Permissible variability in BRB stiffness.* Specify either a minimum stiffness or both a minimum and a maximum stiffness. This can be given as a stiffness modification factor (KF) in the drawings, or as a K_{eff} value. Whatever approach is taken to present the stiffness, the SER should provide guidance on how the BRB manufacturer should use the information given. See Figure 2, note 4 for one possible method.
5. *Definition of methodology for determining BRB strains.* Calculated BRB strains should be smaller than those associated with successfully-tested braces. As a result, the BRB manufacturer determines BRB strains to verify code compliance and should be required to document submit proof of this compliance (see Figure 2, note 1). The most common methods used to determine brace deformations are noted below, but there are certainly other ways that this information can be conveyed. See Figure 2, note 5.

Figure 2– Braced Frame BF-1



Braced Frame	Brace Type	P_u (kips)	A_{sc}	Stiffness Modification Factor (KF)
BF-1	BRB-X		X	
	BRB-Y		Y	
	BRB-Z		Z	

BRB Schedule Notes

1. Buckling restrained braces are to be tested per the provisions of AISC 341-05. Supplier to submit proof of each brace's compliance with the qualified load and strain ranges.

2. P_u given is the governing code level force in the brace, using LRFD force levels $P_u \le 0.9\, A_{sc}\, F_{y\,min}$.

3. F_{ysc} is the actual yield stress of the steel core as determined by a coupon test. 38 ksi ≤ F_{ysc} ≤ 46 ksi. Charpy testing required when thickness of the core material exceeds 2 in.

4. Brace stiffness K_{eff} to be $KF \times (A_{sc}E\,/L)$ ±10%, where the values for Stiffness Modification Factor (KF) and A_{sc} are taken from the table and L is the workpoint–workpoint length of the brace.

5. Brace strains to be calculated as $P_{service}\,/\,K_{eff}$, where $P_{service} = P_u/\rho I$ (ρ = code redundancy factor and I = code importance factor).

6. Maximum ωβ not to exceed X.XX. Maximum β not to exceed X.XX.

a) Use the relationship: $\Delta_{bservice}=P_{service}/K_{eff}$. $P_{service}$ can either be obtained from the SER during the design process or approximated by the BRB manufacturer if the importance and redundancy factors are shown in the design drawings.

b) The BRB manufacturer can calculate Δ_{bm} from building drifts. It is important to note that compliance with code drift limits is the responsibility of the SER and that the BRB manufacturer is only a user of the building drift data. The SER has control of and responsibility for the structural analysis model including accurate modeling of feasible BRB stiffnesses.

6. *Maximum permissible BRB strength adjustment factors.* Frame beams, frame columns, and BRBF connections are checked using BRB-dependent strength adjustment factors ω, β, and ωβ. These factors can be obtained from BRB manufacturers early in the design of the structure. To guard against imposed forces that are greater than those assumed during design, maximum permissible values for β and ωβ factors should be shown in the design drawings. See Figure 2, note 6.

7. *BRB connection details (even in skeleton format) that include work-point location and beam/column connection configuration.* If requested by the SER, BRB manufacturers will design and detail the connection of the brace to the gusset plate and may design and detail the entire gusset plate connection. To accomplish that, a minimum level of information on the design drawings is required. Connection limit states that include gravity and drag loads remain the responsibility of the engineer providing connection design for the structure.

Lessons Learned From BRBF Projects

Although the process of designing and specifying BRBFs is generally straightforward, all parties can benefit from heeding the lessons of past projects to avoid re-learning those lessons at further expense. With that in mind, two recommendations are presented below.

1. Clearly state the force level for any forces given in the design drawings. Problems with design or pricing of BRB projects have been encountered because the force level given in the documents was ambiguous. Sometimes this force level is stated as a P_u value, or the actual load taken from the model and perhaps rounded up to make fewer brace types. The value may be a P_{ysc} force level, or the actual force level at which the engineer requires the brace to yield (which must be greater than or equal to P_u/ϕ). P_u or P_{ysc} may be obtained using either ASD or LRFD design. It is recommended that the design drawings include both the design approach used

Accounting for BRB Stiffness

In the modeling of any structural system, simplifying assumptions are made that will yield results that are considered close enough to predicting the actual performance of a structure. Connections that are semi-rigid may be considered stiff enough to be treated as rigid; brace lengths are considered to extend from work-point to work-point; panel zone flexibility may be accounted for in an approximate way; etc. With a buckling-restrained brace (BRB) project, it is possible to arrive at very accurate modeling parameters that closely reflect the linear-elastic (or post-elastic) behavior of a structure. It is also possible to model a structure in such a manner that the actual behavior varies significantly from what was assumed during the modeling process (see Figure 3). The ability to correctly model the stiffness of the BRBs usually depends on the communication between the structural engineer of record (SER) and the BRB manufacturer during the modeling process.

When modeling the BRB elements in structural analysis software, the stiffness of the braces used in the frame should be taken into account. Overall brace stiffness is determined by analyzing the two stiffer end segments that are "non-yielding" and the less stiff center yielding core segment (see Figure 1, previous page). The steel core area (A_{sc}) can be selected based on the brace load using the equation: $A_{sc} \geq P_u/(\phi F_{ysc\text{-}min})$. However, if A_{sc} is input into the analysis software with the typical modulus of elasticity of steel, E = 29,000 ksi, building drifts will be overestimated by the model, and the seismic forces will potentially be underestimated.

Modeling programs use either an input spring stiffness K or the stiffness equation $K=AE/L$. If the brace is modeled using an area of steel and modulus of elasticity (as is usually done), engineers working on BRB projects usually incorporate the stiffness of the braces and connections by providing either a larger steel area than the steel core area or a higher modulus of elasticity than 29,000 ksi. The factor that is used to increase either A_{sc} or E is sometimes referred to as a stiffness modification factor, KF. This factor is determined based on bay geometry, connection size, brace type and length of the yielding core. Figure 4 demonstrates how this factor can vary from frame to frame and brace type to brace type (note that two different brace types are shown). Generally, the brace stiffness will be expected to vary slightly from the model and only a few KF factors will be used to simplify the modeling process.

It is not expected that the SER determine what the KF factors or the brace stiffness K should be. This is even discouraged. All brace manufacturers currently producing in the United States provide this service free of charge and engineers are encouraged to contact them to discuss their models. Some building officials even require this coordination to take place prior to approving the structure for permit.

Figure 3

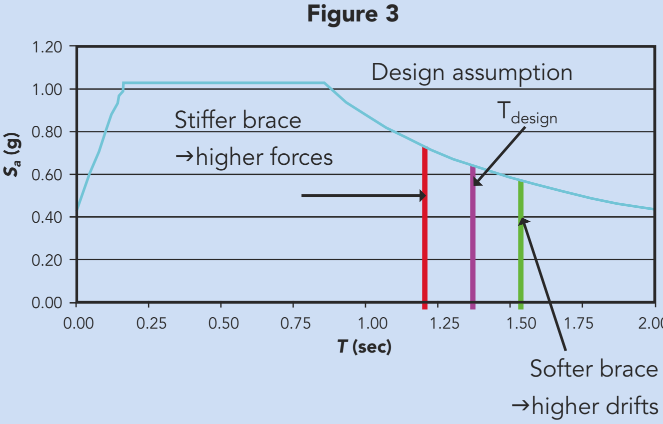
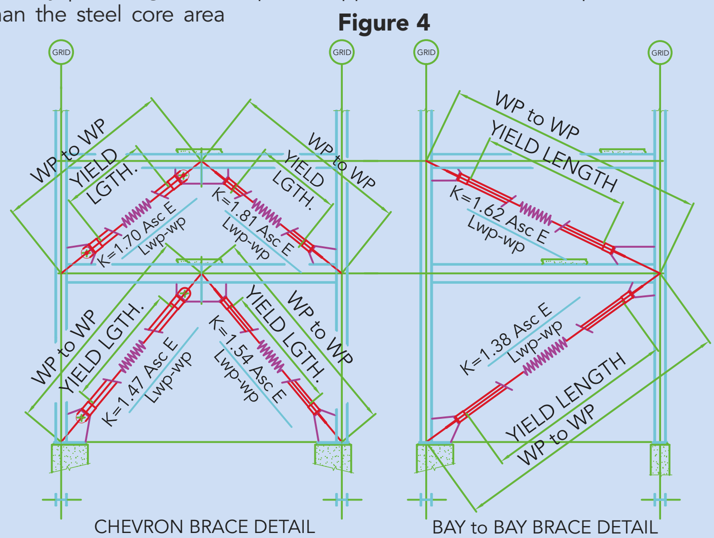


Figure 4



(ASD vs. LRFD) and an equation showing the manufacturer how it is intended that the loads given are to be used. For example, see Figure 2, note 2.

2. During the design phase, verify with the BRB manufacturer that BRB stiffnesses specified are feasible. Occasionally, the engineer specifies a BRB stiffness that cannot be accomplished at the required BRB strength. Sometimes the steel core area specified results in a BRB stiffness that is much higher or much lower than what is specified in the design drawings. The lack of understanding of what is achievable in terms of stiffness has resulted in the SER having to redo analyses with more accurate BRB stiffness values. See additional discussion in the sidebar "Accounting for BRB Stiffness."

Although BRBF design and specification is not complex, there are always things to learn with any new structural system. On a regular basis BRB manufacturers work with engineers who are unfamiliar with BRBF design. BRB manufacturers are eager to assist in any way possible to make the process easier for the design professional. MSC

Auditing in the 21st Century

BY DUKE OKES

Personal competence and integrity are as important as ever.

SOME FOLKS (THIS AUTHOR INCLUDED) believed the 21st century offered the potential for mankind to step away from the horrors of the 20th century and before. That is, we believed that perhaps we were beginning to mature in ways that would allow individuals, organizations and society to operate in more humane and ethical ways, bringing the visions of many from the past into fruition.

But then we saw the collapse of Enron and WorldCom, and more recently the failures of Fannie Mae and Freddie Mac, many large banks, mortgage companies and rating agencies. The list goes on, but it would not be complete without including the havoc wrought by Bernard L. Madoff. It appears the first decade of the 21st century has destroyed many lives, both present and future, mostly due to greed and lack of ethics.

Management systems such as the *AISC Standard for Bridge and Highway Metal Component Manufacturers* are intended to prevent failures of critical organizational processes by defining the processes that should exist as well as some aspects of how the process should work. Audits are then intended to detect whether or not the processes are well implemented, and take corrective action to address any deficiencies. The management review process then provides the highest level of accountability by ensuring that corrective actions are adequately addressed.

Systems were in place in the aforementioned organizations, and audits were also being conducted. In the case of Freddie and Fannie there was even the Office of Federal Housing Oversight (now renamed Federal Housing Financing Agency) whose sole role was to ensure these two organizations operated appropriately. So what happened? In many cases, perverse incentives and slow, gradual deviations built up to become large systemic failures.

Cynthia Cooper, former vice president of internal audit at WorldCom, describes her experiences in finding and reporting major financial reporting problems in her book *Extraordinary Circumstances*. It should be required reading for anyone involved, internally or externally, in any type of auditing. While the focus is on audits of financial accounting and reporting systems, the lessons are just as relevant to other management systems (e.g., quality, safety, environmental).

Management systems can begin to slip especially when things aren't going as desired and there is pressure to make them "right." Also, during restructuring, downsizing and/or to cut costs, organizational resources may be constrained in ways that (at least implicitly) indicate that shortcuts have to be made. Variances might include reducing margins for error in product/process design, ignoring certain legal/regulatory/contractual requirements believed to not be critical, and/or increased management overrides of decisions.

The intent of audits is to detect variances from policies or procedures and to identify any processes which are not meeting objectives and have no related action plans. Auditors should then not just look at a single audit, but also for trends across audits. If a significant shift in compliance or performance (either positive or negative) is observed, further digging should be done to determine what led to the change. Internal auditors within your organization should be especially vigilant after any significant change in organizational structure, resources, products or processes.

Auditors should never accept the findings from a single source for assessing the process. Effective auditors triangulate data from interviews, observations and reviews of records in order to gain confidence in the degree of compliance or noncompliance. Even though it requires more effort, it is generally better to do a deep audit than a shallow one. Use of a statistically based sampling scheme should be considered when auditing critical processes.

Duke Okes helps organizations implement systems to manage and measure organizational performance, and respond to problems. He is the author of Root Cause Analysis: The Core of Problem Solving and Corrective Action, *and has trained thousands of individuals how to perform internal quality audits and root cause analysis. He can be reached at* ***www.aplomet.com***.

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

The internal auditor must also be careful of messages inadvertently given to coworkers during the audit. For example, suppose a process owner indicates that an issue that was found is minor and will be fixed, but prefers that "you not document it as part of the audit findings." Accepting such a request communicates the wrong thing to the process owner both about the audit process and the internal auditor.

It is most helpful if your internal quality auditors are trained in auditing principles and practices. However, it is most critical that they are extremely well versed in the requirements of the management systems to be audited. In addition, they should familiarize themselves with auditing standards such as ISO 19011, which describes not only the methods of auditing but also the personal attributes of effective auditors. The personal credibility of an auditor will have a significant impact on his ability to gain and report the necessary information.

Auditors are of course subject to pressure from management. However, in addition to auditing standards, there are several resources available which might help them. The lead auditor or audit manager is the most logical place to go first. It is hoped that most auditors will never need to seek such channels, but they exist specifically for helping ensure effective governance of the organization.

Personal barriers also may affect whether or not internal auditors are willing to do what is right. One is how well the individual can relate to and communicate with management. How the individual views personnel in management, as well as management's attitude toward quality, auditors, etc., will affect his degree of personal confidence. Another, of course, is how willing he is to do the right thing, regardless of the consequences. The author was recently impressed hearing of an engineer who refused to sign off on defective parts, even though it meant he would lose his job. Auditors are there to report whether or not processes comply with requirements, and to not do so is a professional and ethical lapse. Such lapses impact on self, the profession, the company and society. MSC

Increasing Profitability with Project Cost Data

BY CURT FINCH

A structured approach to collecting good information helps you make better-informed decisions.

INDUSTRIES LIKE MANUFACTURING and farming have had effective accounting systems for thousands of years, but accounting for modern engineering projects can be tricky. Unfortunately, a poor grasp of project costs and return on investment (ROI) can be dangerous to an engineering firm's overall profitability and competitive edge. The only way to avoid this problem is to ensure you have the right project accounting methods in place.

Why Project Accounting?

Years ago, I learned the significance of project accounting firsthand. I was working at a consultancy called The Kernel Group (TKG), primarily fixing software bugs for IBM. Initially, we had a contract to fix the bugs for $2,000 each, regardless of the level of difficulty. Eventually, however, someone in our company decided we should apply the fixed-cost concept to our work. He believed that tracking the time we took to fix each bug would reveal the per-bug profitability, allowing us to set a fair price and gain competitive advantage. As it turns out, he was right.

As we tracked our time, we began to learn which bugs were most profitable, making the most money for the company. Not only that, but we extended this concept to product development, tracking time against each new debugging tool we developed. The data showed us which tools were successful and which were not, based on how much money and time were spent on developing them and how that affected the ROI. In one instance, this process alone resulted in $1 million of revenue for TKG.

When IBM began to bid out the work to multiple vendors, our project time data gave us an advantage over our competitors. We knew exactly how much to bid on each bug because we knew our costs so well. We also knew which ones were extremely difficult to fix, and we were able to bid those high because we had a technical monopoly on them.

Accounting for Engineering Projects

Tracking time for project accounting purposes also brings these benefits to engineering firms. Novar, one of the largest global energy management firms in the world, is a great example. The company began tracking project costs in greater detail in 2002. The project accounting data it collects not only enables staff to execute projects on time and on budget, but also helps them in planning and estimating future engineering projects. "It has saved us more than $10,000 in capital expenditures," says Trevor Porter, Novar's engineering manager.

The best way for engineering firms to begin implementing project accounting processes is simply to jump right in. Guiding your firm to a tracking environment requires changes to your corporate culture. Start by explaining the benefits to the company of doing this. If you understand your costs, you can run your business. Otherwise, you're flying blind.

Step 1. Prepare. Get a complete list of the projects or processes whose costs you want to understand, along with a list of the employees who spend their time on that work. Begin with the end in mind. What will the reports that you want to get out of the system ultimately need to look like? Decide who will own the system. Begin populating the system by adding new employees and projects to it.

Step 2. Get (at least) most of your people to start tracking time. Start measuring adoption by seeing how many employees you can get to track their hours and how often they enter it. Your data will be best if people track their time daily or even more frequently. Recording what you did a week ago is useless. Who remembers what they ate for lunch last Thursday?

Step 3. Get everyone tracking time and expenses with pay rates. Now that you've entered data in the system and employees are tracking their time, you have an accurate and complete list of your firm's projects. Say, for example, you review the data and see that some projects required more time than you expected. Real data has already surprised you. Now it's time to guide your company to the next level. Compute a pay rate for each person in the company and enter it in the system. Ask employees, where appropriate, to record mileage and travel expense data. Further enforce adoption across your entire employee base. At the end of this step, you will receive per-project, per-person direct cost data.

Curt Finch is the CEO of Journyx (http://pr.journyx.com), an Austin, Texas-based provider of Web-based software for tracking time and managing resources. An avid speaker and author, Finch recently published "All Your Money Won't Another Minute Buy: Valuing Time as a Business Resource." He also authors a project management blog at www.project-management-blog.com.

Step 4. Provide for calculation of indirect costs and choose formulas for spreading that cost. Expenses incurred in your company in areas like accounting, marketing and office space can be organized by project cost, project revenue, per person, by square footage of office space used, or in countless other ways. Often, two levels of indirect cost may be necessary. There should be a formula for spreading "partially indirect" cost over multiple customer projects.

Step 5. Revenue Integration. Tools like Salesforce.com or SugarCRM provide a great way to track bookings (and depending on your business, even revenue). Integrating your time accounting system into these systems can give you a profit report, or at least an approximation of one. Integrating the system into QuickBooks or other accounting systems can connect time periods for revenue recognition to cost, giving a good estimation of profit on a per-person-per-project basis. For knowledge worker organizations internal to a company, proxies for revenue—like business value delivered—can be used.

Now you are successfully measuring per-person, per-project profitability. At every step, your situation is better than it was before. Once all of the above steps are complete, you'll know which employees are making money for the firm and which aren't, which engineering projects are profitable and which aren't, and which customers are profitable and which aren't. This is powerful information that can affect your strategy going forward, your rewards and compensation systems, and many other aspects of your business, just as it did for TKG.

Conclusion

In his book *The Seven Habits of Highly Effective People* Stephen Covey argues persuasively that you should track your time even if it's just for yourself. If you do, you will certainly be surprised by the data. However, the business value really starts to get delivered in terms of understanding profitability for organizations of five people or more. Engineering firms that track project costs and ROI will always take the lead, leaving other firms in the dust. Perhaps it's time to evaluate your firm's project accounting methods and how they are working for you.

MSC

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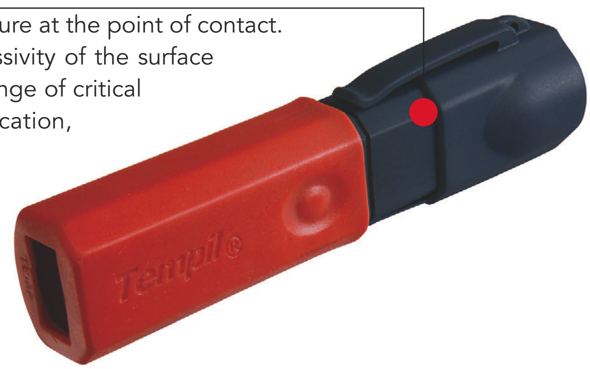
Electronic Surface Thermometer



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

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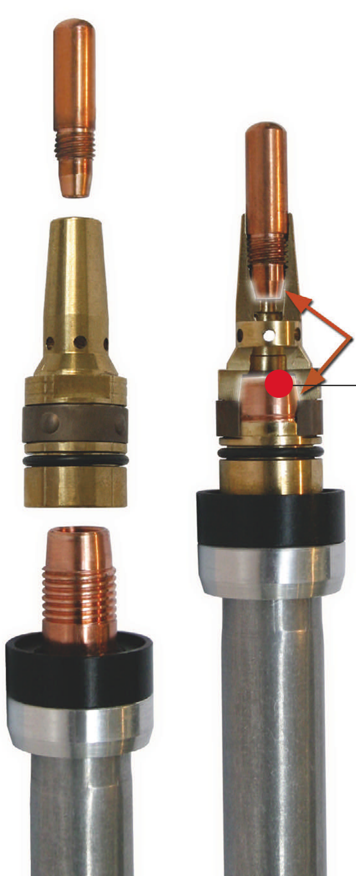

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Ignoring the Wisdom of Crowds

BY JASON COHEN

Let's start with some fascinating, unassailable facts. Then we'll assail them.

IN 2007 MICHAEL MAUBOUSSIN presented a big jar of jelly beans to his 73 Columbia Business School students. How many beans did they think it contained?

Guesses ranged from 250 to 4,100; the actual number was 1,116. The average error was 700—a massive 62%—demonstrating that the students were awful estimators.

Now here comes the weird part. Even with all these wildly incorrect guesses, the average guess was 1,151—just 3% off the mark. Not only that, only two of the 73 students guessed better than this group average. So although individually everyone was woefully inaccurate, collectively the group was incredibly accurate.

Was this a fluke? Hardly. The experiment was made famous in 1987 by financial economist Jack Treynor. In his case it was 850 jelly beans and 56 students. The group estimate was 2.5% off; only one student guessed better. The study has been repeated many times since with similar results, and this eerie effect goes beyond jelly beans.

The Best Multiple-Choice Test, Ever

A contestant on the game show "Who Wants to be a Millionaire" can win $1 million if she answers 15 consecutive multiple-choice questions. If she's stumped along the way, she has three "life-lines:" (1) eliminate two of the four choices, (2) telephone a friend, or (3) poll the audience. The jelly bean experiments imply that this third choice might be pretty good. Is there as much wisdom in the crowd for pop culture and science as there is in counting jelly beans?

The TV studio audience predicts the correct answer an astonishing 91% of the time. Remember, these are questions from all domains of knowledge, all ranges of difficulty, polling a group of people who happened to spend the (weekday) afternoon in a TV studio.

To quantify how amazing that is, compare with the accuracy of the "phone a friend" life-line where the contestant gets 30 seconds with a pre-determined person. This accomplice is probably considered to be "the smartest person I know," plus undoubtedly has access to the various "information" sources such as Google and Wikipedia.

The intelligent friend with broadband access to the entirety of human knowledge gets it right only 65% of the time. Crowd wins again.

There's seemingly no end to studies like these, all showing that the crowd is smarter than the individual, but is this a universal rule? Should we be leveraging this power more often?

Big companies do use crowd wisdom. You always hear about advertising campaigns being honed by focus groups of "real people." (I'd like to see the questionnaire that distinguishes "real people" from that elusive other kind of person.)

However, company messaging, product features, advertising layouts, and the other creative aspects of business require innovation, and we know that design-by-committee is the antithesis of innovation. Average products designed for the average consumer is the path to small business failure. So what should we do? Can we rely on the wisdom of the collective or should we trust a stroke of inspiration?

Let's take another look at "Who Wants to be a Millionaire." Suppose there are 100 people in the audience and only 16 of them know that "A" is the correct answer. Of the rest, none knows the answer and they vote randomly. The result of the vote will be: 37, 21, 21, 21. Oh gee, that's awfully similar to the earlier results of a real audience poll.

(For those of you so inclined, it's fun to try more complex scenarios, although you'll find the result is always similar. For instance, what if only 11 know the answer is A, 15 each know that B, C, or D are certainly not the answer (and vote randomly for the other three), and the remaining 44 have no clue and vote randomly. In this scenario, the vote distribution is the same as in the given example!)

So we have the interesting result that a mere 16% of the voters were able to make choice A the clear winner—nearly double the next closest answer. The reason? The ignorant people vote randomly and *their votes cancel out*, leaving the rest in control of the result.

The Crowd Vetoes Innovation

Now that we understand how crowds can be right, let's see why this same process doesn't work for creative endeavors.

Consider what happens when you're planning a holiday meal. There's a range of fantastic things you could cook, but wait: Some people can't take spicy food, Uncle Bill is allergic to garlic, Aunt Sarah doesn't eat red meat, Timmy doesn't eat anything green...

Eventually you realize there's only one way to please everyone: Cook something bland, mild, and safe, like chicken and rice. But does chicken and rice actually *please* anyone? Not really, it was just what everyone *hated the least*.

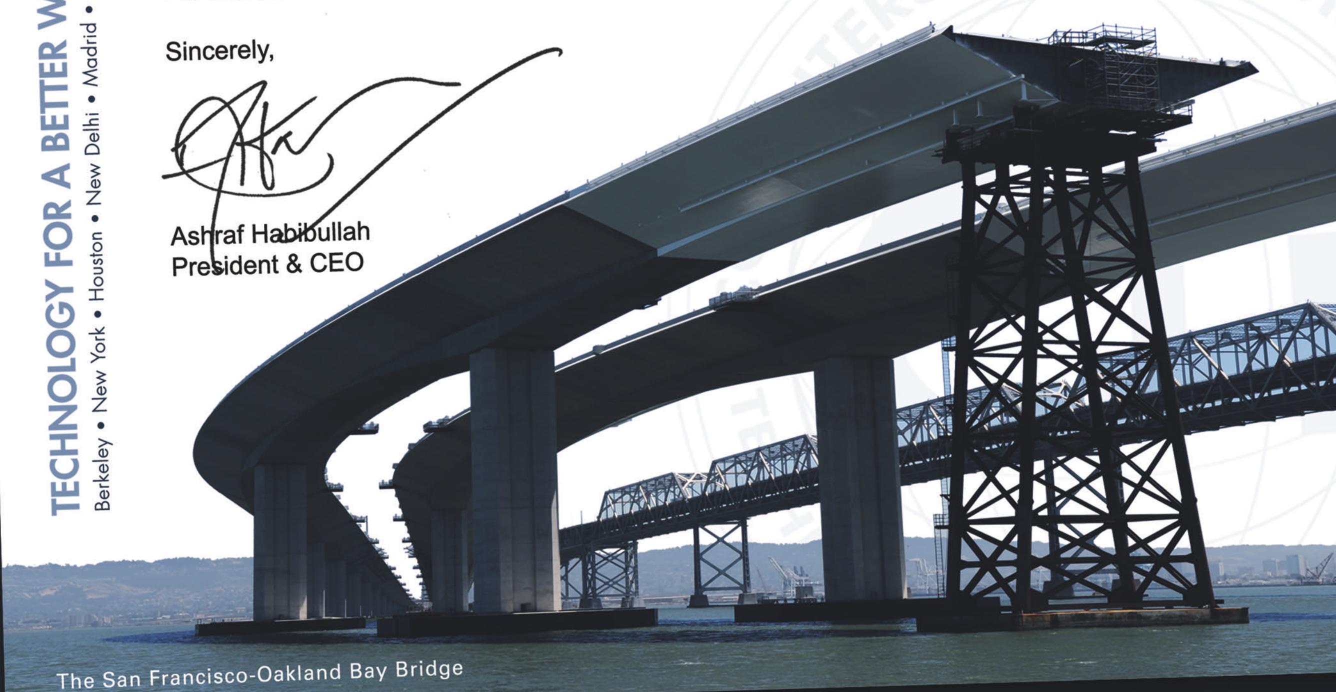
Votes don't converge on something wonderful. Rather, votes are vetoes. Of course if you're a catering company for weddings, chicken and rice might be the way to go. After all, no one goes to weddings for the food, so your primary goal is to irritate as few of the 300 guests as possible. Come to think of it, chicken and rice does seem to be popular at those sorts of functions.

But this isn't a strategy for startups. Little companies need a niche—a market space they can completely, unquestionably own, not some gray middle-ground where your attempt to offend no one also means exciting no one.

There is "wisdom in the crowd" only when errors cancel out, like when estimating jelly beans or answering pop culture questions. In creative work, votes eliminate the interesting edges, leaving only the boring residue that no one hated enough to vote off the island. And that's not how great products are made, or how great structures are designed and built. **MSC**

Jason Cohen is the founder of Smart Bear Software, the industry thought leader on lightweight software code review, and the author of Best Kept Secrets of Peer Code Review. *He blogs about marketing and small business at* ***http://blog.asmartbear.com****, where this essay first appeared.*

Have an opinion you'd like to share in "Topping Out"? Send your feedback to Tom Klemens, senior editor, at klemens@modernsteel.com.



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