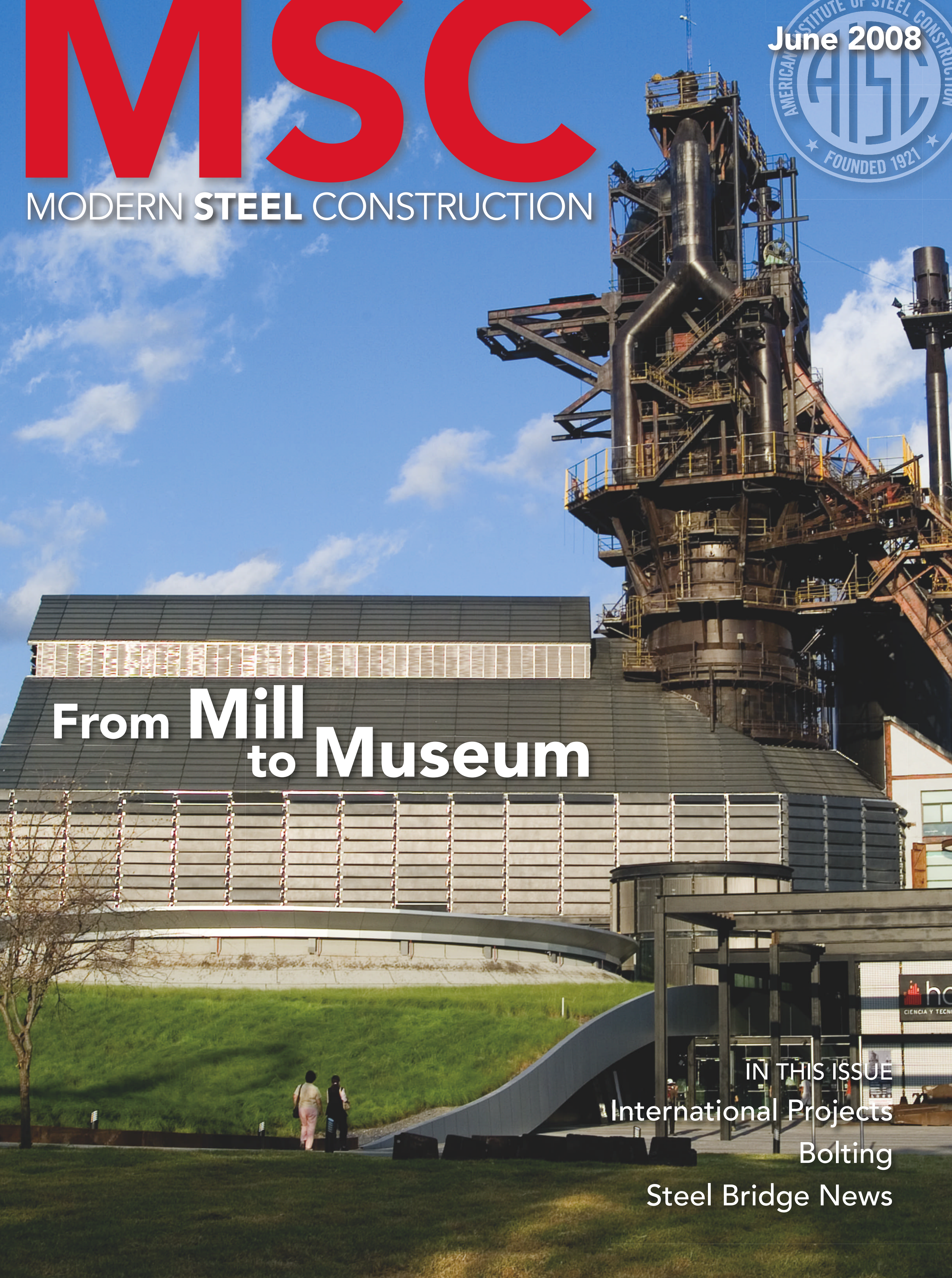
MSC

MODERN **STEEL** CONSTRUCTION

June 2008



From Mill to Museum

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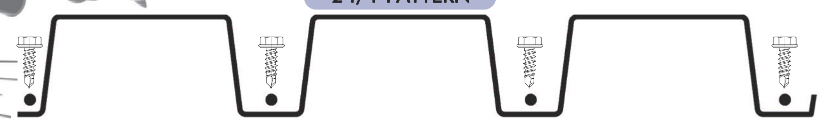
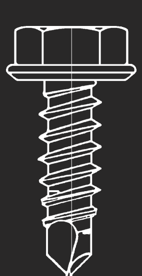


DECK DESIGN DATA SHEET 49R

WIND EFFECTS ON DIAPHRAGM SHEARS



SHEAR AND UPLIFT INTERACTION'S IMPACT ON DIAPHRAGM SHEAR CAPACITY

NS, NI 3" Roof Deck Structure Fasteners = #12 or #14 Screws — 24/4 Pattern Stitch Fasteners = #10 Screws

Allowable Shear Strength is in PLF and Includes a Shear Safety Factor = 2.35.

THICK-NESS	SPAN	SCREWS PER SPAN	UPLIFT PRESSURE IN PSF 0	10	20	30	40	50
.0295"	9'	2	122	114	96	75	4	*
		3	152	144	125	96	4	*
		4	182	174	149	114	4	*
		5	209	199	172	128	4	*
		6	234	224	192	140	4	*
	10'	3	136	128	109	76	*	*
		4	163	155	131	89	*	*
		5	190	179	151	98	*	*
		6	213	201	170	105	*	*
	11'	3	124	115	95	52	*	*
		4	148	139	115	57	*	*
		5	173	162	134	60	*	*
		6	196	183	150	62	*	*

THICK-NESS	SPAN	SCREWS PER SPAN	UPLIFT PRESSURE IN PSF 0	10	20	30	40	50
.0358"	10'	3	166	159	141	117	65	*
		4	198	192	170	141	71	*
		5	231	223	197	161	75	*
		6	259	250	221	179	77	*
		7	286	277	244	194	78	*
	11'	3	150	143	125	99	*	*
		4	180	173	152	120	*	*
		5	210	202	176	137	*	*
		6	238	228	199	151	*	*
	12'	3	138	130	112	83	*	*
		4	165	157	136	99	*	*
		5	193	184	158	112	*	*
		6	219	208	179	122	*	*

THICK-NESS	SPAN	SCREWS PER SPAN	UPLIFT PRESSURE IN PSF 0	10	20	30	40	50
.0474"	11'	3	199	195	177	158	131	59
		4	239	235	217	191	157	62
		5	278	274	251	222	179	64
		6	315	310	283	251	198	64
		7	348	343	314	276	213	65
	12'	3	183	178	160	140	109	*
		4	219	214	196	170	129	*
		5	255	250	228	198	146	*
		6	291	284	258	224	158	*
	13'	4	202	196	178	152	96	*
		5	235	229	208	177	105	*
		6	269	262	236	200	111	*
		7	299	291	263	221	115	*

THICK-NESS	SPAN	SCREWS PER SPAN	UPLIFT PRESSURE IN PSF 0	10	20	30	40	50
.0598"	12'	3	230	229	211	193	171	139
		4	276	275	257	236	209	166
		5	322	320	300	275	243	188
		6	367	365	340	311	273	205
		7	406	404	377	345	301	218
	13'	4	255	252	234	214	185	127
		5	297	294	275	249	215	139
		6	339	336	312	283	242	147
		7	378	374	347	315	266	153
	14'	4	237	233	215	194	163	65
		5	276	272	254	227	189	67
		6	315	311	288	259	212	68
		7	353	348	321	288	232	69

The structural fasteners are loaded in both shear and uplift (tension) during wind events.
The uplift capacity is based on a screw head diameter of 0.4"; other diameters are available.
Interaction affects diaphragm shear strength. Diaphragm stiffness is independent of uplift.
Table is based on F_y = 33 ksi & F_u = 45 ksi. Table is based on a multispan application.
The support must be sufficiently thick so that bearing controls screw shear capacity.
Spans were chosen to provide shear capacities between 100 PLF and 400 PLF. These are not absolute limits. Capacity is not rapidly reduced due to uplift pressure. Interpolation is allowable.
* The uplift capacity of the fastener group is exceeded at this pressure.

For technical assistance please contact our Summit, NJ Engineering Office at 1.800.631.1215 or visit us on the web at www.cmcjd.com - *United Steel Deck.*

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For further information on any of our products or for the nearest CMC Joist & Deck sales representative, contact us at:

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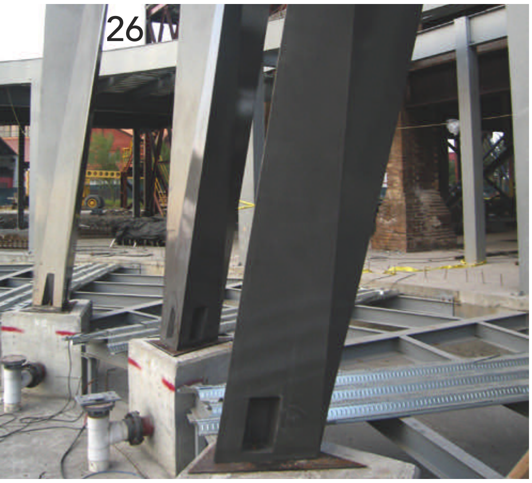

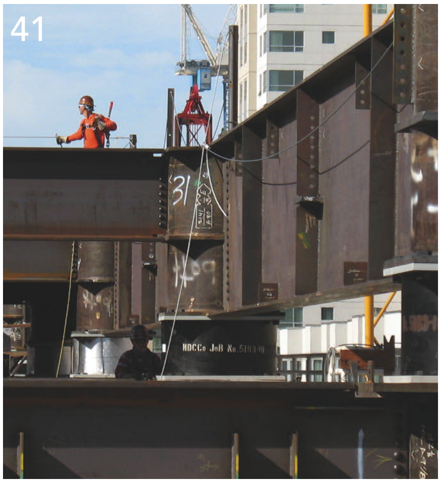

ON THE COVER: The Museo del Acero, Monterrey, Mexico (Photo: Miguel Fuentes, San Pedro).

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editor's note

MY ALMOST-12-YEAR-OLD DAUGHTER IS VERY IMAGE-CONSCIOUS. On those occasions when she accompanies me to the office, she never wears her typical school outfit.

No, she agonizes over which clothes are formal enough to convey the grown-up—and perhaps even professional—image she wants to project. She carefully selects jewelry and even makes sure her hair is done perfectly.

Of course, it's not just my daughter that recognizes that first impressions count. On a recent trip to the west coast, I was fascinated by the wholly diverse styles of the engineering firms I visited.

One medium-sized firm I went to was what I think of as the prototypical engineering firm. The office was highly functional, vaguely corporate, and incredibly egalitarian. It consisted of an open plan with nearly identical cubes and private spaces for conferences. And everyone had the same space whether you were a principal or an entry-level EIT. The company projected the image of technical competence rather than marketing flair, and their niche is medium to large projects with seismic issues.

A completely different impression was made by a visit to a another firm's office. Their clients tended to be larger corporations and their offices presented a more formal corporate appearance—attractive reception area, a mix of cubicles and offices, and a very high-tech conference room. They participate in a lot of video conference calls with their clients and other offices, and their facility included both a camera for the participants and an overhead camera to focus in on drawings. The company projects both the corporate and technical image in keeping with their clientele.

On the opposite end of the spectrum was a much smaller firm that specializes in smaller, architect-driven projects. Their offices were in a converted loft building in a hip part of town. The "green" conversion went so far as to leave the original paint untouched (including bathroom graffiti) wherever possible. Mechanical and structural systems were exposed, and a large bike rack along one wall was heavily used. The steel manuals on many desks were a hint that this was an engineering firm, but in appearance it could easily have housed a cutting edge architectural practice.

The use of design to reflect your corporate identity isn't only the domain of design firms, though. A couple of years ago I visited two steel service centers located just minutes apart. The first only sold structural steel, and they prided themselves on being a friendly and casual family-owned business, the type that does business with a handshake. Sure enough, their offices were in a building that from the street looked like a nice house. In the back was a fishing pond—and sure enough, during lunch employees regularly grabbed a rod and relaxed with some catch-and-release.

The other firm was one of the largest in the country, with a reputation of having a top-notch management structure. Their board room was expansive and impressive. A lot of their work involved OEM fabrication and they had an incredibly organized material handling department.

Interestingly, both companies have the reputation for excellent customer service and both are very profitable and successful businesses. It's not the particular image they project; it's that they all project a clear image, and one that's reflected in their actual core beliefs.

Look around your office. Listen to the way your telephones are answered. Check out some outgoing e-mails from your staff. What's the image that you project? Does it reflect your core philosophies? Image doesn't just reflect reality; it often represents it.

Scott Melnick

SCOTT MELNICK
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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.

Turn-of-Nut Method

One of the recommended methods for installing bolts is the turn-of-nut method. The RSCS specification indicates that turn-of-bolt can be used if it is impractical to use the turn-of-nut method. How is the turn-of-bolt method different than the turn-of-nut method?

This seems to be more a question of terminology rather than requirements. Whether the nut is turned or the head is turned, the method is the same. That is, the same requirements apply.

Amanuel Gebremeskel, P.E.

Web Slenderness Ratio

Please confirm that h/t_w ratio in footnote [a] of Table B4.1 (page 16.1-18 in the 13th edition manual) is the height of the web (i.e., clear distance between flanges) over the web thickness (see Case 2). We just want to verify that this ratio should be used, and not the *b/t* ratio of the flange.

Yes, K_c is present to account for web slenderness h/t_w and h is the clear distance between the flanges. Note that per Section E7.1, K_c is between 0.35 and 0.76. See the Commentary to Section E7.1 for discussion.

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

Comparison of Historic Shapes to Current Shapes

Does AISC provide any written material that compares the properties of old designated beams sizes, such as 16B26, to the current properties of a standard wide-flange (W shape)?

AISC's Design Guide 15 provides the section properties of historic shapes, as well as a summary of historic AISC specifications and applicable standards. AISC has also developed a Shapes Database v13.0 and 13.0H, where the H stands for historic. Both of these resources are available from the AISC bookstore at **www.aisc.org/bookstore** (and are free to members at **www.aisc.org/epubs**). With this data, you should be able to make comparisons between historic and current shapes of your choosing.

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

Two-Story X-Bracing

I have a question on AISC 341 Section 14.3 regarding the unbalanced earthquake load acting on beam due to a buckled brace. We have a two-story OCBF where the V and inverted V braces meet at the second floor beams, forming a two-story X. The braces above and below the beam tend to balance each other (opposing forces). We are not clear if this requirement applies to this configuration. Does it apply only for Chevron-type configurations where the braces are located only below (inverted V configuration) or only above (V configuration)?

The intent of this section is to cover V or inverted V bracing, not the combination of these as a two-story X-braced configuration. In the case of two-story X braced systems, there is no unbalanced load on the intermediate floor beam.

Amanuel Gebremeskel, P.E.

Restrained or Unrestrained?

I am looking into the difference between "restrained" and "unrestrained" ratings for a steel-framed building. I have read the AISC engineering FAQs at www.aisc.org/faq, as well as the ASTM E119 and other literature, and all seem to point me in the direction that a standard steel building is classified as restrained.

That said, we have had some discussion in our office that the restrained classification depends on the continuity of the structure. While I understand that continuous beams spanning over more than two supports will offer more rotational restraint that a simple shear connection, are the simple shear connections enough? Could a single-bay structure (with simple shear connections) be considered restrained? Are there any special restraint requirements for perimeter beams? If one part of the structure becomes unrestrained, does that mean that the entire structure must be classified as unrestrained?

The important type of restraint is rotational restraint, not axial or in-plane expansion. Only a moderate amount of rotational restraint is needed for an assembly to perform as restrained. Beams framed with typical shear connections provide enough restraint. Other factors influence the degree of rotational restraint in large steel-framed floor assemblies. If continuity and/or composite action are part of the floor system, fire tests have shown that the concrete slab plays a significant role in providing rotational restraint and improves fire resistance.

The ability of standard shear connections to provide sufficient rotational restraint was tested by UL and independently under AISI sponsorship.

These findings are discussed in a paper "Restrained Fire Resistance Rating for Buildings" by Gewain and Troup, published in the AISC *Engineering Journal*, Second Quarter 2001 (available free to AISC members at *www.aisc.org/epubs*.)

The conclusion is that steel-framed buildings should be considered thermally restrained.

John L. Ruddy, P.E.

Shape Group Numbers

In preparation for an ICC bolting examination, one of our technicians found a question asking what structural group number a W14×426 member was. We found the answer in the AISC 9th edition ASC manual, but not in the 13th edition *Steel Construction Manual*. The ICC test now references the newer manual, and I was wondering if this table is included in it, or if it was left out intentionally?

steel interchange

It was left out intentionally. ASTM dropped the group number classification system several years ago. For further information on this subject, there was a Steel Interchange article in the July 2006 edition of MSC. You can browse previous Steel Interchange questions by visiting **www.modernsteel.com** and clicking on the Steel Interchange link on the right.

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

Delamination

We have encountered a situation where it appears that the carbon steel framing is delaminating, for lack of a better word. I have searched the Internet, but have not found many articles or data concerning this specific issue. Do you know of the correct terminology to use when the steel is separating in layers?

Your description of laminations may be indicative of what is referred to as "rust pack." In such a case, you will see what appears to be an expansion of the material thickness, with a bulging of the plies in contact. The material thickness can be several times the thickness of the original material.

I have seen this phenomenon quite often in old masonry-clad buildings, where steel lintels were used over exterior doors and windows—in conditions where moisture could accumulate, resulting in severe rust pack after many years of exposure to the elements. When such a condition is encountered, an evaluation should be made to ascertain if the load-carrying capacity of the lintel has been severely compromised. Removal of the loose rust pack, measurement of the remaining sound material, and calculation of the resulting section properties is a common procedure to perform such an evaluation.

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

Seismic Design for Horizontal Bracing Members

Do horizontal bracing members need to meet the requirements of Sections 13.2a and 13.2d of the 2005 AISC seismic provisions? It is not clear whether "bracing" applies to horizontal and vertical, or just vertical bracing.

Section 13 of the 2005 AISC seismic provisions is meant to address the requirements for vertical bracing. In practice, horizontal bracing is typically dealt with similar to how one would deal with diaphragms.

Amanuel Gebremeskel, P.E.

Specifying Reactions

What is the best way to specify simple shear connection reactions on the design drawings? On a non-composite steel framing system, we typically specify that "All beam connections shall develop the full uniform load capacity the member can carry..." The connection designer can then easily obtain a design load using the AISC allowable load tables. This method also ensures that the connection will not be the limiting design element. We have found this procedure to be an efficient method to specify design loads for typical framing.

We would like to specify composite beam connection design loads in a similar manner. Is there a design aid available that would allow a connection designer to easily obtain the maximum uniform load for a composite beam?

Do you have any other recommendations to efficiently provide simple shear connection design loads for composite framing?

Your stated method of specifying reactions for non-composite beams is one method used, but not necessarily the most accurate. Also, it may not always ensure that the connection will not be the limiting design element. It may generally be fairly accurate—if the beams are loaded uniformly, and if the beams are rather close to the design or allowable strength for flexure, and if the limit state of flexure controls the design of the beam. These are a lot of "ifs," and it is generally more desirable and accurate to define the actual design end reactions for the beam on the contract documents.

In the design of composite connections, the process gets more complicated, because one also needs to consider the level of composite action and resulting plastic neutral axis (PNA) location for the design of the particular beam, in order to define a flexural moment capacity for the shape and slab/deck configuration. This flexural capacity would then be correlated back to the superimposed load arrangement and span length of the beam. One may be able to make a lot of conservative assumptions in order to come up with a coefficient that may work in most cases; but again, it may be more prudent to show the actual required design reactions on the contract documents.

Why not just put the actual reactions on the drawings?

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 and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is AISC's chief structural engineer, and Lou Geschwindner is AISC's vice president of engineering and research.

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

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

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From advanced aging...

To advanced engineering.

steel quiz

LOOKING FOR A CHALLENGE? *Modern Steel Construction*'s monthly Steel Quiz tests your knowledge of steel design and construction. Most answers can be found in the 2005 *Specification for Structural Steel Buildings*, available as a free download from AISC's web site, **www.aisc.org/2005spec**. Where appropriate, other industry standards are also referenced.

This month's Steel Quiz was developed by AISC's Steel Solutions Center. Sharpen your pencils and go!

1 Which manufacturing process is most commonly used in the U.S. to melt the steel used to make ASTM A992 wide-flange shapes?

2 Which steelmaking process is most commonly used in the U.S. to melt the steel used to make the coil steel used in the production of ASTM A500 HSS?

3 Do steel members contain VOCs (volatile organic compounds)?

4 What happens to the lateral torsional buckling strength of a beam when the load is applied above the shear center?
a. It stays the same
b. It increases
c. It decreases

5 What types of restraints are addressed in the 2005 AISC specification to prevent lateral torsional buckling?

6 When should eccentricity be accounted for in the outstanding leg of a single-angle shear connection?

7 For a conventional single plate shear connection as defined in the 13th edition AISC *Steel Construction Manual*, what is the maximum distance from the bolt line to the weld line?

8 Can a single plate connection be used with an eccentricity larger than this value?

9 What grade of steel is the usual material for an HSS 5.563×0.375, and what is its value of F_y?

10 What grade of steel is the usual material for a Pipe 5 X-Strong, and what is its value of F_y?

TURN PAGE FOR ANSWERS

steel quiz

ANSWERS

1 W-shapes are most commonly produced in the U.S. using the electric arc furnace (EAF) process.

2 More than half of the coil steel in the U.S. is produced using the basic oxygen furnace (BOF) process, but a growing number of HSS producers also use coil steel produced using the EAF process.

3 No. The abbreviation "VOCs" stands for volatile organic compounds, and steel, being inorganic, does not contain any VOCs. VOCs may be present in coatings used with steel, however.

4 **c.** The lateral torsional buckling strength of a beam decreases when the load is raised from the shear center, due to the destabilizing effect of the load location; it increases if applied below the shear center. Most common structural details are suitable to transmit the load to the shear center.

5 Appendix 6 of the 2005 AISC specification (available for free download at www.aisc.org/2005spec) permits either restraint of the compression flange against lateral translation or torsional restraint of the section to achieve bracing against lateral torsional buckling.

6 Eccentricity should always be accounted for in the outstanding leg of a single-angle shear connection. See page 10-122 in the 13th edition AISC *Steel Construction Manual* (available at www.aisc.org/bookstore).

7 The maximum distance from the weld line to the bolt line for a conventional single plate shear connection is 3½ in.

8 Yes, though the extended configuration procedure found on page 10-102 of the 13th edition *Steel Construction Manual* is applicable.

9 As shown in Table 2-3 in the 13th edition AISC *Steel Construction Manual*, ASTM A500 grade B is the usual material for an HSS 5.563×0.375. It has F_y = 42 ksi (and F_u = 58 ksi).

10 From the same table mentioned in the answer to question 9, ASTM A53 grade B is the usual material for a Pipe 5 X-Strong. It has F_y = 35 ksi (and F_u = 60 ksi). Note that the Pipe 5 X-Strong and HSS 5.563x0.375 have essentially identical cross-sectional dimensions, but different strength levels.

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**.

Steel SolutionsCenter

Proposed ASHRAE Standard Delves into Structural Issues

The structural steel industry has a long track record of success in sustainable development, consistently leading the way in implementing energy, carbon, and resource utilization improvements for the past 25 years. Today, however, the structural steel industry is faced with a significant challenge, one that originates not from a lack of accomplishment with respect to sustainability, but rather from being too successful in this area.

AISC has significant concerns with some provisions in the recently published second draft of the proposed ASHRAE (American Society of Heating, Refrigerating and Air-Conditioning Engineers) Standard 189.1, *Standard for the Design of High-Performance Green Buildings Except Low-Rise Residential Buildings*. AISC's concerns with this proposed standard are that:

1. It is outside the scope and expertise for which ASHRAE is ANSI-accredited.
2. It will result in adverse environmental impacts.
3. It includes provisions that are unfair to steel, and inappropriately preferential to the interests of the concrete industry.
4. It adversely restricts the freedom of design professionals in their selection of structural framing materials.

The proposed ASHRAE standard being developed jointly with the U.S. Green Building Council and the Illuminating Engineering Society of North America is intended to provide minimum requirements for high-performance green buildings.

While addressing HVAC and lighting issues, the standard also establishes prescriptive requirements for construction materials, an area that AISC believes to be outside the scope and expertise of ASHRAE, USGBC, and IESNA. In addition, the committee responsible for the proposed 189.1 standard was not constituted in a balanced manner under appropriate ANSI consensus protocol. AISC also believes that the committee lacks expertise in the area of construction materials, particularly as they relate to structural framing systems.

This apparent lack of balance and expertise has resulted in provisions that appear to be significantly slanted toward the interests of the cement/concrete industry under the guise of encouraging less sustainable industries to become more sustainable.

For example, today a typical structural steel frame provides an 11% credit towards the overall recycled content of a building. A concrete frame may provide one to two percent. At the same time, the reinforcing steel in the concrete structure will provide an additional 5% credit. The proposed standard will limit the contribution for any material at 5%. The result: structural steel gets capped at 5%, while concrete still gets its full credit AND the 5% credit for reinforcing steel.

Similarly, the definition of recycled content is that portion of a material by mass that originates in either pre- or post-consumer waste streams. But the ASHRAE committee has decided to allow the calculation of the recycled content of concrete to violate that definition. Instead of reflecting the actual recycled content of the concrete, ASHRAE 189.1 allows the recycled content of the cementitious portion of the concrete to be used as the recycled content of the entire concrete mix.

For example, at present, substituting 25% fly ash for Portland cement in concrete with no other recycled content, results in an actual recycled content of 3%. Under ASHRAE 189.1 the cement and concrete industries are allowed to claim a 25% recycled content.

The committee's justification is that they wish to encourage the use of fly ash in concrete. In reality, they are discouraging the use of recycled aggregates, removing over 50% of the mass of a concrete building from green considerations and providing the cement/concrete industry with an unfair advantage in the marketplace.

It is not the role of a standard to provide incentives and to favor particular products. The selection of structural framing materials should be based on the merits of the materials as judged against a consistent metric.

The inclusion of this standard in building codes is being encouraged by ASHRAE as an appendix at the national level available for local adoption. Including these provisions in a local building code will significantly limit the opportunity of design professionals to select construction materials for high-performance green buildings that properly balance economic, environmental, and design issues.

The structural steel industry believes strongly in the need for high-performance green buildings. AISC also believes that standards for the selection and optimization of structural framing materials should be developed in a balanced, consensus-based ANSI process that engages design professionals, industry associations, and interested parties with the required level of expertise to develop a fair and environmentally sound standard. AISC would welcome the participation of the concrete, cement, masonry, wood, precast, light-gauge steel, per-engineered building, and any other affected industries in that process.

AISC's objection to 189.1 is not a rejection of sustainable construction practices or the need for green buildings. Much to the contrary, AISC's commitment is to continue to be the leader in sustainable construction materials and to actively pursue additional sustainable practices within our industry.

When the domestic structural steel industry experienced a rebirth 25 years ago, purposeful decisions were made to create a sustainable industry. A central decision was the transition from basic oxygen furnaces (using iron ore and coke) to electric arc furnaces (using scrap as the primary raw material and electricity and natural gas as energy sources).

The gains from this transition have positively impacted sustainable construction:

- ✓ Wide-flange structural steel products average in excess of 90% recycled content.
- ✓ A 96% recycling/reuse rate for structural steel members removed from existing structures.
- ✓ An increase in mill productivity by a factor of 20 moving from 10 to 12 man-hours per ton to 0.6 man-hours per ton.
- ✓ A reduction in energy consumption per ton of product by 30%.
- ✓ A reduction in carbon emissions by 47% since 1990; by comparison, the Kyoto protocol would have mandated a 5.2% reduction by 2012.
- ✓ A recycling rate for automobiles now exceeding 100%, emptying out salvage yards.
- ✓ The elimination of all production water discharges and the minimization of water utilization.
- ✓ An increase in the strength of structural steel by 38% over the past 10 years, reducing the quantity of structural steel required in a typical building.

For more information, contact Scott Melnick, AISC's vice president of communications, at melnick@aisc.org. MSC

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Keeping Up with the AISC Spec

BY MATTHEW FADDEN AND JILL RAJEK

Important changes to the AISC specification can be easy to lose track of, and often the challenge is to understand when the changes were made. *Design Guide 15: AISC Rehabilitation and Retrofit Guide* (DG 15), Appendix A1 provides a comprehensive source of historical information that references changes made to the specification.

Currently, DG 15, Appendix A1 provides a list of changes to the specification through the 1999 *LRFD Specification for Structural Steel Buildings*. An update to this list, including the changes from the 1999 LRFD specification to the 2005 specification, is now available at **www.aisc.org/crossref99**. An overview of some of the more prominent revisions is outlined below.

The 2005 specification contains numerous unifying changes, which can be seen in Chapter A, General Provisions. While this list of updates only pertains to the LRFD portion of the specification, the most noticeable change is the combination of the ASD and LRFD provisions. Additionally, the scope of the specification has been expanded to include "other structures", which are defined as "those structures designed, fabricated, and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting elements." Less noticeable but equally useful to the 2005 specification is the inclusion of the specifications for single angles and HSS sections. Incorporating these specifications has mitigated the need for other provisions.

Revisions to Chapter C, Stability Analysis and Design, reveal major organizational and substantive changes; most notable is the requirement to address second-order effects in the analysis and design. A new procedure, the Direct Analysis Method, which is described in Appendix 7, will satisfy the requirements of Chapter C. Additionally, stability based on plastic design must follow Appendix 1, which also includes other provisions for inelastic analysis and design.

Chapter F, Design of Members for Flexure, has also been renamed and reorganized. The chapter is now divided into sections based on member type and the axis of bending. Table User Note F1.1, Selection Table for the Application of Chapter F Sections provides a summary of the chapter by illustrating each cross-section addressed and stating the applicable limit states for that member.

The updates to Chapter I, Design of Composite Members, reflect research results and allow the use of higher strength materials, as well as provide better consistency with ACI 318-05, *Building Code Requirements for Structural Concrete*. Shear stud strength is now dependent on the location of the stud in the flute of the metal deck, the number of studs welded within one flute, and the orientation of the metal deck with respect to the beam. Composite column design is based on new interaction formulas that better reflect behavior and strength.

Some changes have been made to Chapter J, Design of Connections, and one of the most notable organizational revisions is the inclusion of the effects of concentrated forces previously appearing in Chapter K of the 1999 LRFD specification. The 1999 LRFD specification combined concentrated forces, fatigue, and ponding into one chapter. The 2005 specification separates these sections where Design for Ponding is located in Appendix 2 and Design for Fatigue is located in Appendix 3.

The all-new Appendix 4, Structural Design for Fire Conditions, provides much-needed criteria for the design and evaluation of structural components for fire conditions. This appendix discusses the effects of elevated temperature on materials and accounts for these changes in the design.

A review of the complete list of changes will help engineers using LRFD to become more acquainted with the 2005 specification. To get up to speed with all the changes to the specification, be sure to visit **www.aisc.org/crossref99**. MSC

Matthew Fadden is an engineering graduate student at the University of Michigan in Ann Arbor. Jill Rajek is a recent engineering graduate of the University of Wisconsin–Platteville and plans to attend graduate school in the fall. Both were summer interns at AISC in 2007.

letters

Continuing Education

I found Steve Kurtz's article "Learning by Doing" (April, p. 66) extremely interesting. We train the structural Ironworkers of New York in a 24,000-sq.-ft training school that houses nine classrooms and a 12,000-sq.-ft, 50-ft-high indoor work area. The work area includes a five-ton overhead crane, a structural steel frame, 12 burning stations for oxy-acetylene, and 33 welding booths for a variety of welding procedures (stick, automatic, pipe, and stainless).

We would be more than happy to have engineering students visit our school at any time to further showcase the complexities of structural steel erection that our students learn during their three years of training. I can be reached at director@nycironworkers.org.

Bryan Brady II, Director of Training
Ironworkers Locals 40 & 361, New York

Do they Really?

I found Anne Scarlett's article "Engineers Can—and do—Communicate Well" (April, p. 51) quite interesting, but I do not fully agree with her reasoning and her conclusion.

During their university training, engineering students write many term papers and other reports for their professors, who are senior specialist in the subject. When these students enter the industry, they continue to communicate with the assumption that the readers or listeners are also experts in their field. They fail to recognize that engineers have to communicate with other people such as workers, money managers, governmental officials, and the general public, and not only with specialists in their field. It is just as important to recognize to whom one communicates as what is communicated.

To be successful communicators, engineers must evaluate their target audience and tailor their presentations to the specific target audience.

Harry W. Ebert, P.E.
Madison, N.J.

Anne Scarlett responds:
Excellent points! It was definitely an oversight on my part to not include that factor in the article (by all means, when I'm coaching folks, we do an audience analysis first and foremost). Mainly, I wanted to be clear that engineers self-proclaiming that they are rotten communicators (and/or engineers who are just tossed aside with preconceived notions that they are ineffective at communicating) are both displaying cop-out attitudes. Rather, they can (and do) have the skill sets in them. But they need to work at it, and they need to have much confidence. (Confidence is half the battle; knowing your audience and your main message is the other half, yes?)

news & events

ENGINEERING JOURNAL

Call for EJ Papers

AISC is always looking for *Engineering Journal* articles on interesting topics pertinent to steel design, research, steel fabrication methods, or new products of significance to the uses of steel in construction. We are especially seeking technical articles with practical applications in the steel industry.

If you have a new idea or an improvement on an old idea, please submit a paper to AISC for publication in the *Engineering Journal*. All published papers are eligible for the Best EJ Paper of the Year award. The winning author of this annual award is selected by our readership and receives a free trip to the North American Steel Construction Conference as well as acknowledgment at the conference.

Please send your paper in duplicate to:

Engineering Journal
Editor, Cynthia Duncan
AISC
1 E. Wacker Dr., Suite 700
Chicago, Illinois 60601
duncan@aisc.org

Detailed information on our review process and requirements for submittals can be found on the inside back cover of each *Engineering Journal* issue.

ONLINE RESOURCES

Why Steel?

In a continuing effort to answer this seemingly simple question, AISC has recently launched a specialized web site as part of its industry mobilization program. Created exclusively for the structural steel industry, **www.WhySteel.org** is a controlled-access web site; visitors must join the site to access its content.

Users of the new site are not necessarily members of AISC; they just have to be part of the structural steel industry. The site provides access to educational articles, newsletters, discussion forums, and tools that the industry can use to learn about and promote structural steel.

Perhaps most importantly, it provides the steel industry with reasons why any project should be built with steel and includes hints, tips, and ideas for convincing those in the design and development community that "There's always a solution in steel."

Some of the sections of **www.WhySteel.org** include:

- ✓ **Industry Mobilization.** Find out how you can help your industry—and why you should. You'll also find hints and tips to get started and answers to frequently asked questions. Get started in helping your industry be even stronger than it is today.

- ✓ **Structural Steel Benefits.** Learn about the advantages and benefits of building with steel. One area of this section is devoted to understanding the benefits of steel as it relates to different types of projects, enabling you to talk sensibly to the local design community about steel and provide them with handouts and case studies of specific project types.
- ✓ **Education/Learning.** Access talking points about the industry and learn about the other players in the steel supply chain. Gain an understanding of terms and terminology, and increase your knowledge of steel's competition.

Guitars and Girders

BY GEOFF WEISENBERGER

The information-packed 2008 Steel Conference in Nashville breaks NASCC attendance records.

FOR A LONG TIME NOW, NASHVILLE HAS BEEN KNOWN AS THE PLACE WHERE COUNTRY MUSICIANS GO TO MAKE IT BIG. This year, the same can be said of the North American Steel Construction Conference. The show brought nearly 3,800 people to the Music City, making it the best-attended Steel Conference ever.

While Nashville received a substantial amount of rain during the show (April showers…), it certainly didn't dampen the spirits of those in attendance. This year's Steel Conference took place at the Nashville Convention Center in the heart of downtown. The compact, more vertical layout of the show created a hive of activity, as attendees seamlessly moved between the sessions and the exhibit floor, greeting fellow colleagues and old acquaintances along the way.

In addition, many of the show's 90+ sessions and short courses were very well-attended, some of them even reaching standing-room-only status—as was the case with the Erector track session "Sporting Opportunities: Design, Fabrication, and Erection Issues on the New Dallas Cowboys Stadium." Perhaps it was because of the two world-record longest single-span trusses (1,225 ft each) that the will support the stadium's new roof, or maybe there are just a lot of Cowboys fans out there, but the session was packed to the gills, with several attendees standing in the back or near the front.

The exhibit hall was equally busy, hosting more than 220 exhibitors. In a setup that was part Cracker Barrel and part Old West, AISC's booth welcomed visitors with candy sticks, rocking chairs, and iced tea (and yes, it was sweet tea).

Away from the booth, AISC made a couple of major announcements. One focused on the findings of a full-scale blast test of a steel wide-flange column that was conducted for and funded by AISC. The test investigated the behavior of a W14×233 column of ASTM A992, Gr. 50 structural steel subjected to an explosive charge similar to that experienced during the terrorist attack on the Alfred P. Murrah Federal Building in Oklahoma City in 1995. The result showed that while the column suffered plastic deformation, the use of a steel column—as opposed to the concrete column that was destroyed in the Murrah Building attack—would have resulted in much less structural damage to the building. (Go to **www.aisc.org/blast** for the full report.)

AISC also announced its response to the recently published second draft of the proposed ASHRAE (American Society of Heating, Refrigerating and Air-Conditioning Engineers) Standard 189.1, *Standard for the Design of High-Performance Green Buildings Except Low-Rise Residential Buildings* (see page 18 for more information).

Pushing Green

The idea of sustainability echoed throughout the conference and also served as one of AISC's top priorities for the show. Conference bags were made from 51% post-consumer recycled content, the final program was printed on recycled paper with soy-based inks, and attendees were presented with reusable water bottles, which they were able to fill at one of the many water coolers throughout the center. A number of sessions, including "Greening the Shop: Strategies for Managing Your Environmental Footprint" and "Green Design: Beyond Material Issues", also focused on sustainable building practices and material recycling.

While not focusing directly on environmental friendliness, many other sessions did put an emphasis on efficiency. "Designing Low-Cost Steel Structures" provided dozens of tips on efficient steel design. One of the speakers, John Rolfes, P.E., S.E., of Computerized Structural Design, pointed out that while not all of the tips were new, it's important to push the old ones until people really start following them. Suggestions ranged from

The exhibit hall at the Nashville Convention Center boasted more than 220 exhibitors, showcasing everything from software to services to fabrication machinery.

Both the exhibit hall and technical sessions were packed throughout the conference. More than 90 technical sessions provided ample opportunities for attendees to earn continuing education credits.

using joist girders only where appropriate to considering cantilevered columns for the top story of a multi-story frame to practicing timely service. "Don't let shop drawings sit in your office for three weeks," he stressed. Co-speaker Jay Ruby, P.E., of Ruby and Associates, compared construction documents to confusing assembly instructions for items such as bicycles and grills, saying, "As designers, we need to make it easier for the builders."

Wednesday's keynote speaker, Stephen Kieran, AIA, a partner at KieranTimberlake Associates LLP, also touched upon sustainability in his address, noting that the concept is becoming more desired in the construction industry. "Our clients are starting to demand the same performance from us as is taking place everywhere else," he said.

At the center of his address was a call for innovation, and he noted the building industry's tendency to shun liability—and risk. "We have been marginalized because we haven't assumed risk," he said, stressing that playing it too safe squelches innovation and doesn't allow the industry to reap its benefits.

He also suggested a more proactive, long-term approach to buildings. Instead of planning, designing, and building a structure, then walking away from it, architects and engineers should monitor a building to learn how to make the next project better.

Bringing the discussion back to sustainability, Kieran predicted that within 10 years, we will no longer be able to construct "throwaway" buildings. They will be built with full life cycle in mind, and when they reach the end of their useful lives, they will have to be disassembled and "taken back"—much like BMW does with its "retired" automobiles.

More than 1,400 attendees enjoyed networking at the conference dinner, held on Nashville's Honky Tonk Highway.

Recognizing Excellence

Also at the Wednesday keynote session, Joseph Burns, P.E., S.E., FAIA, LEED AP, managing principal in the Chicago office of Thornton Tomasetti, Inc., was presented with the Special Achievement Award for notable accomplishments in structural steel design, research and education. Specifically, he was honored for his significant efforts in promoting and advancing the use of building information modeling (BIM) and interoperability in the design and construction of major steel structures.

At the Friday keynote address, Walterio Lopez, S.E., a senior associate with Rutherford & Chekene in San Francisco, and Rafael Sabelli, a senior project manager with Walter P Moore in Los Angeles, were honored with the T.R. Higgins Lectureship Award for their paper "Seismic Design of Buckling-Restrained Braced Frames." Their work on BFRBs—an increasingly popular new steel seismic load resisting system—has already been published by the Structural Steel Education Council and has helped BRBFs be accepted in ANSI/AISC 341-05 Seismic Provisions for Structural Steel Buildings and in the International Building Code.

Nashville at Night

A hard day's work at the Steel Conference warranted a relaxing night on the town. This year's conference dinner gave attendees a taste of two things that Nashville does best: barbeque and country music. Guests were given exclusive use of six adjacent country music bars that comprise the well-known Honky Tonk Highway, just steps away from the convention center. Guests could amble from one music venue to the next with relative ease, networking in the laid-back, fun, and friendly atmosphere.

Of course, plenty of barbeque, from local favorite Jack's, was on hand too, and everyone was able to dig into their hickory-smoked meat of choice, topping it off with one of several BBQ sauce options and complementing it with delicious and BBQ-appropriate, if not exactly healthy, sides. (This editor opted for brisket, the hottest sauce available, homemade macaroni and cheese, iced tea, and Shiner Bock; I call it the "Honky Tonk Diet".)

Next year, NASSC leaves the country behind but keeps the western, as the 2009 show will take place April 1-4 in Phoenix. Information for the 2009 Steel Conference will be posted soon at **www.aisg.org/nascc**. MSC

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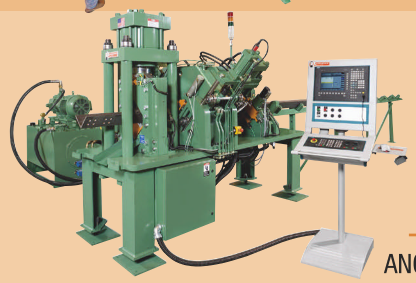

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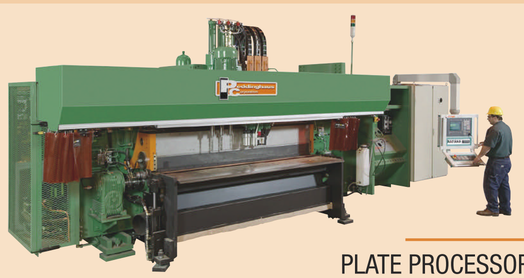

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ENGINEERING JOURNAL

Second Quarter 2008 Article Abstracts

The following papers appear in the first quarter 2008 issue of AISC's *Engineering Journal*. EJ is available online to AISC members and ePubs subscribers at **www.aisc.org/epubs**.

Reduced Beam Section Spring Constants

BART MORTENSEN, JANICE J. CHAMBERS, AND TONY C. BARTLEY

A moment connection that includes a wide-flange beam with trimmed flanges is commonly known as a reduced beam section (RBS) connection. Accurate analysis of frames incorporating RBS beams requires knowledge of the elastic stiffness matrix of RBS beams. In lieu of using this stiffness matrix, an RBS beam can be modeled as an Euler-Bernoulli frame element with rotational springs at each end, which can be easily implemented in structural analysis software. This paper presents the derivation of the formula for the spring constants of an RBS beam, and validates it. From a study of the spring constants for a plethora of RBS beams, it was found that a strong linear relationship exists between the minimum plastic section modulus of RBS beams and their spring constants. This paper has direct applicability to the practical and accurate determination of the elastic drift of a moment frame with RBS connections.

Topics: Analysis, Connections-Moment, Lateral Systems, Seismic Design

Bending Strength of Steel Bracket and Splice Plates

BENJAMIN A. MOHR AND THOMAS M. MURRAY

The primary purpose of this study was to determine the ultimate behavior of bracket and splice plates. The study consisted of experimental testing and comparison of test results with various design methods. The experimental testing consisted of connecting two beams together with web splice plates to form a simple span, then loading the span symmetrically to induce pure moment at the location of the splice, with the goal of achieving plate flexural rupture. This study indicates that design models used prior to the publication of the 13th Edition AISC *Steel Construction Manual* for determining bracket plate and web splice nominal moment strength are overly conservative.

Topics: Connections-Simple Shear, Research, Splices

A Modified Equation for Expected Maximum Shear Strength of the Special Segment for Design of Special Truss Moment Frames

SHIH-HO CHAO AND SUBHASH C. GOEL

Special truss moment frame (STMF) is a relatively new type of steel structural system that was developed for resisting forces and deformations induced by severe earthquake ground motions. The system dissipates earthquake energy through ductile special segments located near the mid-span of the truss girders. The other elements outside the special segments, such as truss members, girder-to-column connections, and columns, are designed based on the expected vertical shear strength (V_{ne}) of the special segment and are expected to remain elastic during a major earthquake. As a consequence, overestimation of V_{ne} in the special segment can result in significant over-design of the elements outside the special segment. This study shows that the equation for expected shear strength in the current AISC seismic provisions can be quite conservative, thereby leading to considerable over-design of members outside the special segment. Based on more realistic assumptions, a modified expression for V_{ne} is proposed in this paper, which results in a better estimation of the expected shear strength while maintaining an adequate safety margin. The proposed expression was validated by using previous experimental results as well as nonlinear static and dynamic analyses (to determine the seismic demand in the special segment). A design equation of V_{ne} for STMF using multiple Vierendeel panels in the special segment is also proposed.

Topics: Seismic Design, Structural and Building Systems

Performance-Based Plastic Design of Special Truss Moment Frames

SHIH-HO CHAO AND SUBHASH C. GOEL

This paper presents the results of a study in which a recently developed performance-based plastic design (PBPD) methodology was used to design the special truss moment frame (STMF) system rather than conventional elastic method. This newly developed performance-based method has been successfully applied to moment frames and also extended to eccentrically braced frames, buckling-restrained braced frames, and concentrically braced frames. The procedure begins by selecting a desired yield mechanism for the frame. Design base shear and lateral forces are determined from input spectral energy for a given hazard level needed to monotonically push the structure in the yielded state up to a pre-selected target drift. The frame members are then designed by following the plastic design method in order to develop the needed strength and the intended yield mechanism. A new seismic design lateral force distribution based on nonlinear dynamic behavior is also presented. The proposed design procedure was validated by extensive nonlinear dynamic analyses for a number of ground motion records. The results confirm the validity of the proposed method for the study STMFs in terms of meeting all the performance design objectives, such as target drifts and intended yield mechanism. An important advantage of the PBPD method is that, generally, no nonlinear analysis is needed to check the structural performance after the initial design.

Topics: Seismic Design, Structural and Building Systems, Plastic Design

Current Steel Structures Research

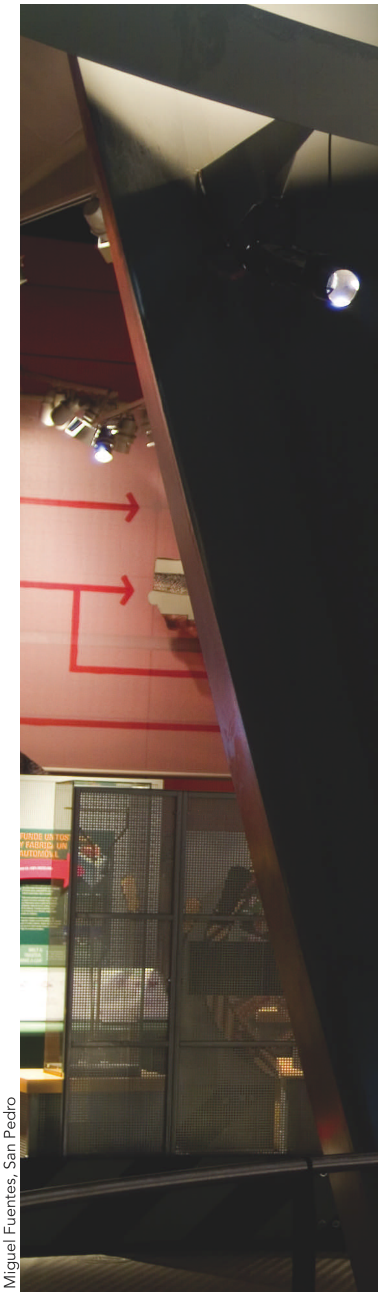
REIDAR BJORHOVDE

This regular feature of the *Engineering Journal* provides information on new and ongoing research around the world. In the 14th installment, research projects are summarized on the following topics: three-dimensional behavior of semi-rigid connections, behavior of longitudinal double plates-to-rectangular hollow section connections, welded steel beam design using particle swarm analysis, modeling of micro- and macro-structural size effects for fatigue of welded tubular structures, advanced engineering for orthotropic bridge decks and surfacing solutions, and ponding of roof structures.

Topics: Research

From Mill to

Miguel Fuentes, San Pedro



Miguel Fuentes, San Pedro

Above, left: Parque Fundidora is the setting for the old steel mill and the new museum building.

Above, right: Triangular chevron-shaped steel columns in the Steel Gallery.

A former Mexican steel mill lives on as a museum, invoking the past, present, and future of the material.

THERE ARE SOME PLACES WHERE THE PAST LINGERS. And that can be a good thing.

At Parque Fundidora in Monterrey, Mexico, the area's industrial past is apparent. Part of the site, a defunct steel mill, was turned into a steel museum, and the remaining blast furnace leaves no question as to the building's original use. The Museo del Acero (Steel Museum) opened to the public last fall and was immediately designated a Monumento Nacional de Mexico.

A portion of the museum complex is the old steel mill that was refurbished and made accessible by open-air walkways. A new building is connected to the existing structure at ground level; due to the site's topography, the new building is partially located underground.

Two structural elements are particularly noteworthy when looking at the new building: the faceted roof of the Steel Gallery and a "floating" helical staircase.

Multi-faceted

The Steel Gallery is an underground exhibition space demonstrating the structural potential of steel. The roof covering the Steel Gallery is itself also a demonstration of this potential. The roof is constructed of relatively thin steel plates (maximum thickness is approximately 0.5 in.) with a maximum span of about 42.5 ft and a total diameter of about 98 ft. Seen from above, the steel plates span from an outside circle made of concrete walls to an inner circle of 12 angular steel columns. The bottom of each column is triangular-shaped. At the top, there are two triangles side by side in plan, meaning that from every corner at the top, two edges run towards one corner at the bottom. Finally, the complete column consists of six triangular-shaped plates; together, they form a "chevron." Above the inner circle of columns is composite construction carrying a terrace and a small bridge connecting the terrace to the main building.

In order to avoid local buckling, multiple folds were introduced into the roof geometry. These folds allow for a variation of the structural height, depending on the span of the steel planes and the moment distribution. This helps to minimize stresses due to vertical loading; additional beams or stiffeners were not required. Finally, the originally designed plate thicknesses were adapted to market availability in Mexico.

The columns of the inner circle have a unique shape. They support the roof by deforming their top plates into the folded roof elements, hence forming a rigid moment connection at the top. In addition, the columns have chevron-shaped horizontal cantilevers at their top; these cantilevers support the terrace structure above. At their bases, the columns are hinged to the concrete floor.

Welded plate steel columns are supported on adjustable triangular base plates flush with the finished floor level. The base plates rest on concrete pedestals, with the roof drainage integrated into

Museum

BY WERNER SOBEK, ANGELIKA SCHMID, AND FRANK HEINLEIN

the columns. The tapered and cantilevering central terrace supports provide sliding layers to allow for horizontal movement of the terrace above, and thus avoid restraining the steel roof structure. In the horizontal direction, the terrace is supported by the bridge linking the terrace to the main museum building. Thus, the architectural idea of tapered triangular steel elements is converted into a structural system suitable to support both the roof and the terrace.

A 3D finite element model employing iso-parametric shell elements (RFEM by DLUBAL) was used to model the roof and the columns of the Steel Gallery. Even for thin steel plates (between 0.4 in. and 0.5 in. thick), the maximum deflection due to dead load is less than 1.14 in. When using the most critical ultimate limit state factored load combinations, stresses in all steel plates (material grade A572 Grade 50) remain within allowable limits. Stability checks were performed using second-order nonlinear analysis. To ensure that plates remain stable under compression, additional hand calculations were performed for individual plates, using plate buckling theory.

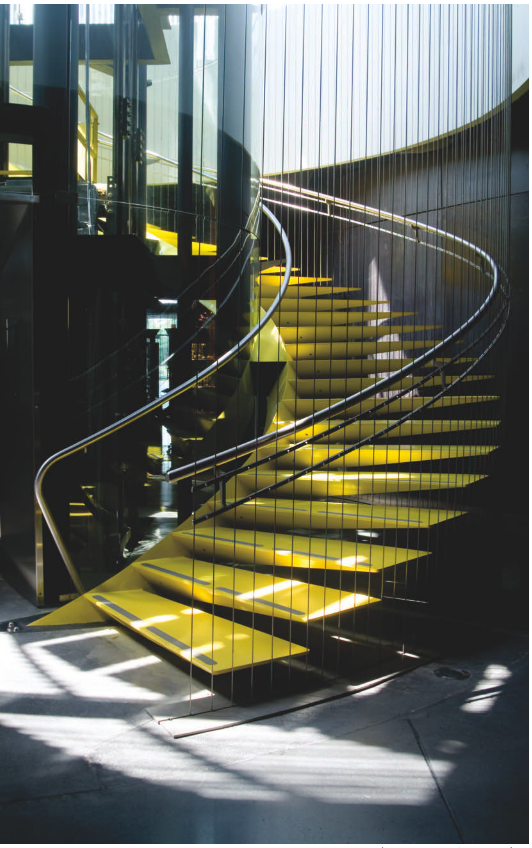
It was also shown that under an accidental load case situation (with reduced partial safety factors), the steel roof remains stable, even if one column fails entirely. This high safety potential results from the possibility of plastic load-redistribution.

In order to minimize welding on-site and to optimize quality, larger steel pieces were shop-welded, inspected, and then

Right: The roof of the Steel Gallery.

Below: Helical stair with cantilevered threads and pre-stressed cables.

Miguel Fuentes, San Pedro



Miguel Fuentes, San Pedro

transported to the site. After installation of the columns as well as the outer ring walls, large segments of the roof were lifted into place and then site-welded into a monolithic folded surface.

Ascending the Helix

The two levels of the new museum building are linked by an elevator and a flight of stairs winding around it. The steel columns arranged in a circle around the elevator support the elevator structure as well as the roof structure above. This roof structure cantilevers outwards from the columns to the façade edge beams and therefore spans over the stair structure. From those edge beams, thin cables are pre-stressed against the ground, holding up the handrail profile, which thus seems to "float" around the staircase without any uprights. The staircase stringer and treads are made of welded triangular steel plates; welds are ground flush for aesthetic appearance with clear geometry lines and sharp corners. The cantilevering treads are only supported by the stringer without touching the outer handrail cables. The custom-made stringer profile is helical and spans from a triangular base plate at the concrete floor (moment connection) to the top steel beams of the composite landing slab (also moment connection).

In order to minimize possible resonance problems, the stair was designed for a sufficiently high relevant natural frequency ($f_{1,\ staircase} = 2.7$ Hz). The stringer support is a rigid moment connection where support forces and moments are anchored into the ground. At the junction of stair tread, stringer, and triangular base plate, shear studs and concrete anchors develop sufficient lever arm to transfer forces to the concrete floor slab. In case of any unexpected vibrations during use (especially combined horizontal/twist modes), the inner stringer can easily be connected locally to the outer elevator shaft columns.

Like the Steel Gallery's roof, the elevator structure and the helical stair were modeled by means of a 3D finite element calculation (RSTAB by DLUBAL). Member sizes were chosen to keep stresses and deflections within clearly defined limits. The pre-camber of the staircase treads was also determined according to finite element modeling. As steel structures offer relatively little damping, the first natural frequency of the system was raised high enough so as to minimize the risk of resonance due to walking. On-site frequency measurements were carried out to check the staircase behavior under walking conditions. These measurements confirmed the correctness of the previous assumptions.

The staircase was pre-fabricated off-site using temporary scaffolding and then lifted onto a truck for transportation to the site. The final lift on-site was performed using a mobile crane. Anticipated deflections due to dead loading were pre-cambered and also compensated by using wedge-shaped shimming plates below the triangular base plate of the stringer profile. Thus it could be ensured that treads were in a perfectly horizontal position after erection.

Living History

For both the folded roof of the Steel Gallery and the helical staircase, the potential of steel was used to the fullest extent with regard to material strength, structural design approach, and construction technology. And this potential is invoked in a unique way—in a facility that used to produce steel and now not only teaches visitors about the history of the material, but also incorporates it in innovative ways.

Werner Sobek is director of the Institute for Lightweight Structures and Conceptual Design at the University of Stuttgart, Germany, and president of Werner Sobek. Angelika Schmid is senior engineer with Werner Sobek, Stuttgart. Frank Heinlein is communications director for Werner Sobek, Stuttgart. MSC

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Werner Sobek, New York

Engineer of Record
SOCSA, Monterrey, Mexico

Contractor
Sistemas Optimos Constructivos, Monterrey, Mexico

Construction Manager
Aconsa

Heavy Lifting

- There are 252 beams and 40 columns for each floor.
- More than 370 crane picks were made per floor for the steel and concrete contractors.
- There were three tower cranes on the project; each crane lifted 112 pieces of steel every week.
- The frame consists of 4,200 tonnes (4,629 tons) of steel and was erected in 24 weeks.

— Courtesy of Marcel Feuchter, Structural Project Manager, Canary Wharf Contractors Limited

CWCL

What Lies Beneath

Under-floor building services drive floor member design in a London office building.

BY JAMES CASSON

CANARY WHARF, FOR SOME TIME, HAS BEEN A LEADING EPICENTER OF OFFICE HIGH-RISE CONSTRUCTION. And a new development in the area continues this trend.

BP2, a new 15-story office building, is nearing completion on the northern edge of Canary Wharf over the former West India Import Dock. It is on the second of four plots in an area called Blackwall Place. The building will be the future home of an investment bank, securities trading, and brokerage firm.

There are considerable structural challenges associated with the project, which stands on an irregularly shaped site of approximately 4,690 sq. m (50,482 sq. ft). It is being built partially over a 1990s parking ramp and partially over water, with the podium access shared with an adjacent site.

The development will comprise a total internal area of approximately 36,450 m^2 (392,344 sq. ft), including two roof plant areas, a double-height reception area, and ancillary areas at ground level. Twelve office floors provide a total net internal area of approximately 29,150 m^2 (313,767 sq. ft).

Foundations

The pile foundations for BP2 were constructed in three separate stages. The first stage was built during the construction of Churchill Place car park (1989), located beneath the southern end of the plot. The second stage was installed across the northern end of the site during construction of the One Churchill Place car park and loading bay ramp (2002). The third stage took place during 2006, when a single 1,500-mm-diameter (4.9-ft) pile was constructed to allow the proposed new BP2 footprint to be fully supported without overloading of the existing piles.

Superstructure

The steel frame for the BP2 project springs from the reinforced concrete foundation at the ground-floor level.

"The column locations for the building vary from being on a uniform grid at the southern end of the building to a seemingly random arrangement elsewhere," states Graham Pocock, Senior Structural Engineer, WSP. "Essentially, columns had to be

HOK

Opposite and above: The building's cladding panels will be top hung and fixed off of channel inserts into the concrete floor slab.

Top, left: BP2's frame consists of 4,200 tonnes (4,629 tons) of structural steel.

Top, right: The building, offering views of the Millenium Dome, is part of the Canary Wharf development.

Courtesy of HOK International

The typical floor-to-floor height in BP2 is 4 m (13 ft).

located wherever they could, to avoid clashes with the access ramps to Blackwall Place and BP1 Car Parks. This also results in a number of significant cantilever details in many of the corners. There were no column transfers in the superstructure, as there just is not sufficient space in the typical floor zones."

The typical floor-to-floor height of an office floor in BP2 is 4 m (13 ft), the allowable zone for floor structure being 810 mm (2.65 ft), consisting of a 140-mm (0.46-ft) lightweight concrete floor slab on trapezoidal profiled metal decking, a 610-mm (2-ft) structural steel beam zone, and 60-mm (2.36-in.) allowance for sprayed fire protection, tolerance, and deflection. The steel beams are designed to act compositely with the concrete floor slab. The building's cladding panels are top hung and fixed off of channel inserts into the concrete floor slab.

The building systems are located within the structural zone under the floors, following the current trend with many commercial office buildings. The systems are actually routed through the beams, which have holes cut in them, rather than under them, resulting in a lower floor-to-floor height. The criteria for designing the steel floor members aim to achieve minimum depth rather than minimum weight. Because of the corner details, column arrangement, and the requirement for building service/structural integration, many floor beams are heavy plate girders with web openings. Where possible, the beams were kept to a minimum of 310 mm (1 ft) deep to allow for the passage of services below.

The level of complexity for design and detailing of the steelwork floors increases at the plant room and roof levels, the main reason for this being the increased story height at these levels. The architectural design called for joints in the cladding panel system every 4,050 mm (13.3 ft) vertically. Therefore, the cladding panels could not be supported off of the floor slabs. Instead, they are supported by a secondary steel framing system. This involved a significant amount of coordination between all the design and construction team members to ensure the whole variety of cladding panel supports was provided at the correct position.

At roof level there is a building maintenance unit (BMU) rail track for support around the building perimeter. Typically, stub posts pick up the rails at 3 m (9.8 ft) on center; these posts are at closer spacing around tight corners. Because of the long reach required of the BMU, the loadings from the unit are significant, and this—together with the strict deflection limits for the members supporting the posts and the fact that these members invariably also pick up the cladding—make the design, detailing, and construction quite complex. MSC

James Casson is an associate with WSP Cantor Seinuk's London office.

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HOK International, London

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Steelwork Contractor
Zuid-Nederlandse Staalbouw/Victor Buyck Joint Venture

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The American Institute of Steel Construction (AISC) has retained Dowco Consultants, Ltd., to develop a new online program to train entry-level steel detailers. The new program utilizes the industry's premier reference, *Detailing for Steel Construction*, and is designed to take a student with no knowledge of the steel industry or detailing and provide the basics to allow them to work as an entry-level steel detailer.

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American Institute of Steel Construction   There's always a solution in steel.   Dowco Consultants, Ltd.

For complete information, visit **www.structuraldetailertraining.com.**

A recent renovation project in Dresden, Germany sheds new light on the interior of the city's Main Station.

Shining Through

BY MIKE COOK

Adam Wilson/Buro Happold

IN THE UNITED STATES, IT IS COMMON TO TRAVEL LONG DISTANCES BY AIRPLANE. But in Europe, long distances are generally covered via train. So it is not surprising that one of the most urgent tasks for Germany, following its reunification, was the need to upgrade its national rail system.

Dresden Main Station, which was earmarked by the German national railway (Deutsche Bahn) for renovation in 1997, is one of the grandest pre-war train stations in Germany. It is also one of the country's busiest. Some 50,000 travelers pass through it on a daily basis on their way to urban, regional, and long-distance destinations.

Buro Happold helped execute the vision of London architectural firm Foster and Partners to replace the vast but gloomy spaces over the tracks with a soaring, translucent fabric roof that brings additional light into the station. To maintain the spirit of the original structure, Foster and Partners wanted to preserve the existing steel arches of the 1897 train halls.

A structural overhaul was necessary so that the new roof would not add excessive loads to the fragile supports, which had suffered damage at the end of World War II and inadequate repairs in subsequent years. To make the project work, Buro Happold collaborated closely with Foster and Partners and the local engineer in charge of the renovation, Schmitt Stumpf Frühauf und Partner.

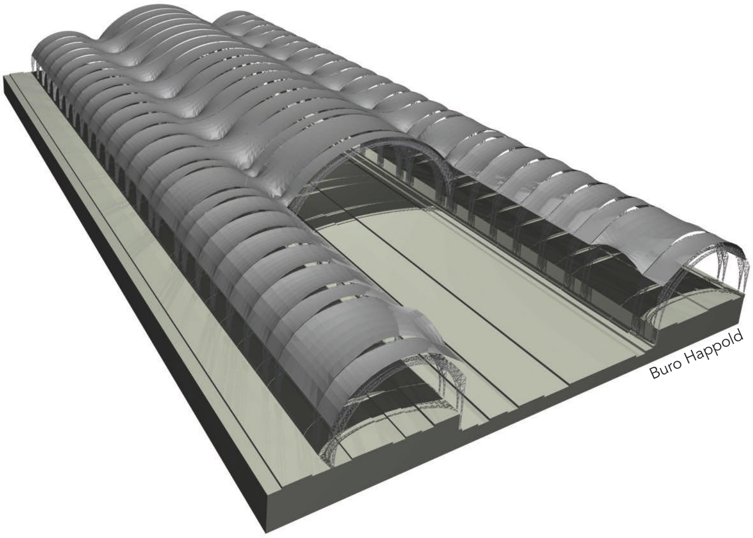
Structural Challenges

The station's footprint is 787 ft by 394 ft, which is divided into three parts: the entrance building and the middle hall (194 ft wide with a roof height of 98 ft) and two side halls. The north hall is 105 ft wide and contains four tracks, while the south hall has only three tracks and is slightly narrower. Both side halls rise to a height of 72 ft.

The existing roof cladding, a combination of corrugated metal and skylights, had been supported by steel arches with purlins spanning between them. The new 323,000-sq.-ft membrane roof imposed less weight, but the flexibility of the fabric material created horizontal forces on the steel arches, which were originally designed to carry only the vertical loads of the roof. The existing structure also had to transfer the loads of the fabric roof from the arches all the way down to the foundations.

Buro Happold helped to resolve these conditions through intensive studies in order to design and complete the installation of the roof. Major investigations were conducted during the design process to assess the strength of the steel and the existing structure of the station. These tests determined that the steel had significant structural weaknesses caused by years of poor maintenance and neglect—not to mention damage sustained by Allied bombing during World War II. In many places, it was corroded, deflected, and deformed.

To support the loads from the membrane roof, much of the steel had to be repaired or replaced, including entire bays of arches. Some of the existing foundations were also replaced and/or reinforced. In addition, the erection of the fabric roof



Adam Wilson/Buro Happold

At the junctions of the arches, a special fabric configuration was required to collect and drain rainwater.

necessitated the use of a secondary steel framing system to sustain the new roof, as well as new roof lights, drainage pipes, and lightning protection.

Testing Assumptions

The sophistication of the new roof structure required that the team reconsider the entire structure of the station. The Buro Happold team began the design process by carrying out extensive calculations using Tensyl, its proprietary computer software. Tensyl has a three-dimensional interface that allows the firm to build and then analyze models to determine the information required for construction and installation. (Originally developed in 1978, Tensyl has since been used successfully by the company to design non-linear structures, such as fabric and cable-net, and assess how they will behave under a variety of load conditions.)

Another major challenge the team faced was the choice of a translucent fabric for the roof, that was not fully approved as a building material—particularly for train stations—because of its potential fire hazard. During the planning process, several possible membrane fabrics were considered. The team decided on a Teflon-coated glass fiber fabric over PVC-coated polyester because of its strong resistance to fire, diesel fumes, and staining, as well as its self-cleaning properties.

Buro Happold investigated the ways fabric could be stressed between the existing steel arches, while identifying the problem zones, such as those with high tension stresses or excessive deflections. The total roof area was divided into nine different bay types in order to test all relevant load cases and load case combinations. This analysis allowed the design team to develop the shape of the membrane and its points of connection to the existing structure.

The fabric structure and the design assumptions were also tested in relation to the existing structural arches, since the original steel was old and damaged and therefore much less ductile than a contemporary material. The design had to account for such things as temperature changes and the failure of the membrane panels in the event of an unexpected catastrophe or the impact of point loads (from maintenance personnel, for example). Any of one of these conditions alone could disrupt the integrity of the structure and the strength of the roof.

Raising the Roof

After reviewing many different options during the design phase, Buro Happold developed a structural solution that added a secondary, triangulated steel structure directly on top of the existing steel arches. To prevent further damage to the existing steel due to welding, the two structures were bolted together for stability.

The fabric membrane was lifted 3 to 6 ft above the main arches, based on a study of the forces on the fabric structure, then fixed to the secondary steel framing system. Transfer loads from the membrane were then passed from the secondary structure to the top chords of the old steel arches.

To make the fabric roof structure resistant to wind and snow loads, the design team developed a double-curved fabric form that created the required stiffness to span the maximum distances between arches, up to 46 ft. The team linked pairs of arches in the secondary structure with rigid purlins that act as lateral trusses, to provide additional stiffness. This design avoided the need for secondary purlins, which were present in the previous roof structure. The spaces in between the paired arches were linked only by the fabric, so that the whole structure would flex to accommodate the elongation that normally occurs in steel when temperatures change.

Since the fragile arches had little resistance to horizontal forces, the longitudinal loads of the roof were transferred to the end bays of the station. The bays were then braced, turning them into 33-ft-wide trusses. To prevent catastrophic failure of the membrane, additional horizontal cables were inserted underneath the fabric to support the membrane, if needed, under extreme load cases.

Creating new Connections

Although the fabric membrane was much less expensive and lighter than glass, the roofing material made necessary the design of a customized solution for linear membrane clamps to attach the fabric roof to the secondary steel structure via steel plates. This detail had to work with the membrane roof and the top chord of the secondary structure, yet remain cost-effective to install and produce.

Whereas the steel plates that connect the fabric to the secondary steel structure were fixed, the clamps themselves had to accommodate for movement of the membrane structure. Once a clamp was installed and stressed, there were no further methods available to adjust the membrane tension. This meant that the fabrication had to be extremely accurate.

Each bay had to be modeled individually, using the computer model to determine the angle of the clamps at each point of connection. The team supplied this three-dimensional information to the steel fabricator for inclusion on their shop drawings.

Since the clamps and the fabric materials had never been used before by Deutsche Bahn, the team had to obtain approval for their use. Buro Happold submitted all structural calculations and drawings, as well as material tests that described the behavior of the material and structure. This involved laboratory testing of the fabric to verify the long-term characteristics of the material, including flex tests to demonstrate the durability of the clamped fabric edges under repeated load cycles.

From Darkness to Light

Dresden Main Station's official reopening in November 2006 coincided with the 800th anniversary of the founding of Dresden. Today, the station that was once so dark and neglected is now bathed in natural light. Within the halls, the membrane roof appears to float over the 19th-century steel arches. From above, the fabric creates a gently undulating impression in Dresden's cityscape, an indication of how the city is rapidly moving forward into the 21st century. **MSC**

Mike Cook is a partner with Buro Happold.

Rudi Meisel/Foster and Partners

Stretched between the existing steel arches (circa 1897), the translucent fabric brings daylight into the main train shed.

Owner
Deutsche Bahn Station & Service AG
Regionalbüro für Großprojekte
Dresden

Architect
Foster and Partners, London

Planner and Structural Engineer
Schmitt Stumpf Frühauf und Partner

Structural Engineer (Membrane Roof)
Buro Happold

Project Manager
Homola AYH AG

General Contractor
ARGE Dywidag und Heitkamp

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On the Grid

BY JOHN SUMNICHT

Photos: John Sumnicht/SGH

Seismic isolators, sandwiched between two steel grids, separate an existing San Francisco building from its vertical addition.

THIS PAST DECEMBER, CONSTRUCTION REACHED SUBSTANTIAL COMPLETION on a technically innovative seismic isolation project. The building: 185 Berry Street in San Francisco's China Basin district. Simpson Gumpertz & Heger, Inc. (SGH) helped meet the owner's goal of expanding the existing three-story, 216,000-sq.-ft building by 150,000 sq. ft in two new stories, with minimal disruption to tenants.

The existing building is 825 ft long and 110 ft wide. In order to accommodate the length of the building, two expansion joints are located approximately at the one-third points along the building's length, dividing it into three separate structures. The second and third floors and roof are post-tensioned, concrete flat slabs. The grade level slab is a conventionally reinforced, structurally supported concrete flat slab. Foundation support is provided by prestressed, precast concrete pile groups beneath columns.

The building was designed in 1988 to the requirements of the 1984 San Francisco Building Code, which was based on the 1979 Uniform Building Code (UBC). The design of the building included provisions for a light, one-story addition of 50,000 sq. ft above the roof. It thus had nominally more lateral strength than required to support the three stories, and the foundations had additional vertical load-carrying capacity.

A Heavy Addition

The addition of the two new floors triggered an upgrade of the entire building to meet the requirements for new construction under the 2001 California Building Code (CBC). Initial analysis considering a conventional fixed-base addition demonstrated that the building did not satisfy some of the prescriptive requirements necessary under the currently enforceable 1997 UBC (on which the 2001 CBC is based). Conventional upgrade approaches, involving the addition of reinforced concrete shear walls, steel braced frames, or dampers throughout the structure, would have been highly disruptive and intolerable to the bio-science laboratories operated by University of California at San Francisco within the building.

SGH suggested that the two new stories be constructed on seismic isolation bearings over the roof of the existing structure, essentially converting the addition into a mass damper. This concept had never previously been implemented in any building in the U.S. In initial feasibility studies, SGH demonstrated that this technique enabled the new construction to perform like a mass damper, which would reduce the seismic response of the existing structure below. This permitted the new space to be constructed without requiring a major structural upgrade of the existing building, thereby eliminating the need for an intrusive and disruptive construction in the currently occupied space.

The project's challenges were significant. The existing columns and foundations had only a limited amount of additional vertical load-carrying capacity. The addition is a continuous, approximately 800-ft-long structure bridging the three, independent structures below. Underlying soils are liquefiable. There were large amounts of active HVAC equipment on the existing roof that needed to remain functional during and after construction. The stair and elevator towers needed to be extended through, but isolated from, the new floors. And, all of the construction work had to be completed while the building remained fully occupied.

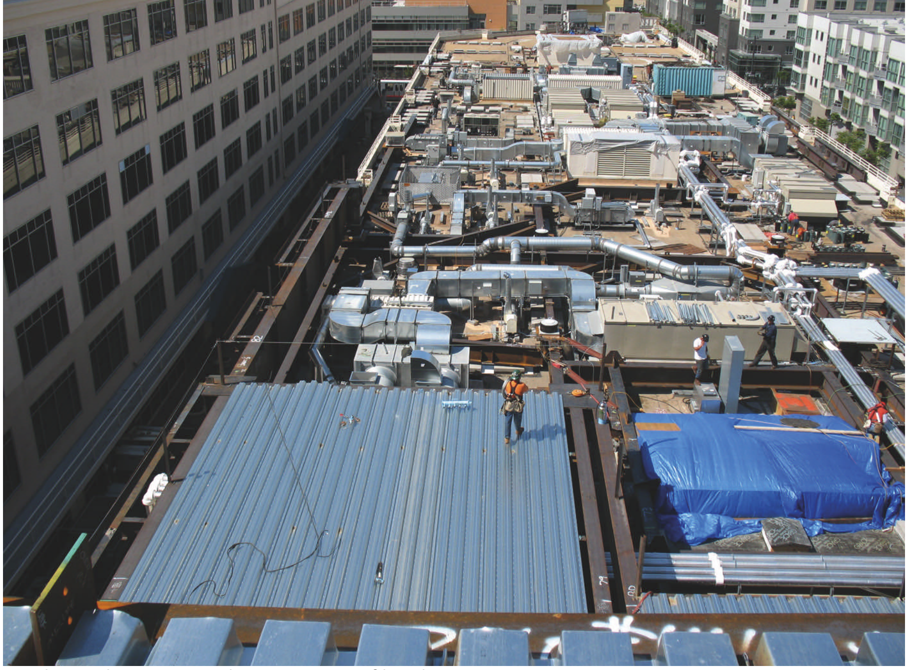
In order to minimize the weight of the addition and maintain Type I construction, steel framing was the logical choice. The addition consists of a new fourth and fifth floor and roof constructed over an interstitial space formed between the existing building roof and the new fourth floor. The fourth and fifth floors consist of lightweight concrete topping over metal deck on top of steel framing. The roof consists of lightweight topping over metal deck in the center bay for most of the length of the building, and untopped metal deck in the outer bays. The new superstructure uses approximately 3,000 tons of structural steel.

Performance-Based Design

The seismic isolation provisions contained in the 2001 CBC require that the structure below the plane of isolation remain essentially elastic under design earthquake shaking; the existing structure had insufficient strength to ensure elastic behavior. Upgrade of the structure to provide such strength was impractical due to cost and tenant disruption. Therefore, SGH used a performance-based approach to demonstrate that although the substructure would not remain elastic, it would perform to acceptable standards. The City of San Francisco required a strict peer review to oversee the design process on behalf of the city. The agreed-upon performance objective was that the existing building and addition would provide a similar level of reliability against collapse or life safety endangerment to that of a new building designed to the current code. The FEMA 356 provisions for seismic evaluation and rehabilitation of existing buildings were followed as a basic guideline; however, these were modified with project-specific criteria applicable to this groundbreaking technical achievement.

The engineers constructed a full three-dimensional, non-linear model of the three wings in RAM Perform (now CSI's PERFORM-3D); this software allows users to model full material and geometric non-linearities in the existing beams, columns, and shear walls. The frame non-linearities were modeled using discrete plastic hinges with properties based on relevant tabulated values in FEMA 356, modified using moment curvature analyses. Using the composite action provided by the floor slab along

The seismic isolator can be seen in between the two structural steel grids.



Mechanical systems at the existing roof level.

with the existing prestress gave the model its frame beam flexural capacities. Since SGH expected pounding between the existing three structures, we added contact-only elements to the model to capture the effects of pounding. To help reduce the amount of pounding, we added dampers across the existing expansion joints at the existing roof.

The non-linear time-history analysis justified that the base-isolated addition was not detrimental to the existing structure. However, the peer review team wanted us to demonstrate that the building, along with the addition, possessed the necessary toughness of a code-compliant structure. To demonstrate that, we performed a reliability analysis using the results from incremental dynamic analyses, in order to estimate a confidence level associated with the existing building's ability to resist global collapse at maximum considered earthquake (MCE) level shaking.

The height of the interstitial space, measured from the top of the existing roof to the top of the fourth floor, was set at approximately 11 ft. This height was adequate to accommodate a grid of new steel beams above the existing roof but below the isolators, the isolators themselves, and another grid of steel beams as part of the fourth-floor framing above the isolators. It also allowed many pieces of the existing, active HVAC equipment to remain on the existing roof, saving money and potential downtime.

The grid of large structural steel members above and below the isolators was required to resist the moments from the displaced isolators. We took advantage of the inherent stiffness of the heavy steel grid at the fourth floor to provide enhanced vibration characteristics—4,000 micro-inches per second—since the owner was pursuing biotech and laboratory tenants for the new space.

In order to protect the relatively thin prestressed concrete roof slab, we utilized an interlocking shear transfer system consisting of concrete pads connected to the roof, and steel shear lugs connected to the lower steel grid. The connection of the concrete pads to the existing roof employed 6-in.-diameter pipes cored into the slab, as well as threaded rods epoxy grouted into the slab.

Coring and drilling into the existing post-tensioned slab was not the only challenge faced by the contractor. There were large amounts of existing HVAC equipment and piping on the roof. The contractor had to raise approximately three miles of existing piping to install some of the lower steel grid, and special care and detailing was required to thread the steel members—some weighing as much as 7.5 tons—around the units, ducts, and piping.

The project employs 87 seismic isolation bearings, including 33 lead-rubber bearings and 54 combined elastomeric/slider bearings. The design of the isolation system presented a significant challenge: isolating a relatively light superstructure while keeping the isolators stable at a displacement of +/- 45.5 in., which was 1.5 times the code-required maximum displacement of +/- 30 in. This required an isolation system consisting of 45-in.-diameter lead-rubber bearings and a new combined system of sliders in series with elastomeric bearings, where the PTFE sliders provided +/- 30 in. of displacement and the additional +/- 15 in. of displacement was accommodated in the 24-in.-diameter elastomeric bearing. Prototype testing demonstrated the stability of both bearings.

The use of seismic isolation as a means of providing new floors to an existing building and improving its seismic performance is truly an innovative application that set a high bar for technical accomplishment, advanced structural design and review, and collaboration among the team members. Most importantly, by applying seismic isolation in this way, we helped the owners achieve their scheduling and cost objectives on a highly constrained project. MSC

John Sumnicht is a senior principal with Simpson Gumpertz & Heger, Inc. He can be reached at jfsumnicht@sgh.com.

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McCarthy Cook

Structural Engineer
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Seismic Isolation Bearings
Dynamic Isolation Systems, Sparks, Nev.

On the Brink

BY JAMES O. MALLEY, S.E.

Degenkolb Engineers

Degenkolb Engineers

Advanced seismic analysis and design keep a crucial 15-story building near the Hayward Fault from becoming history.

JUST FIVE MILES FROM CALIFORNIA'S HAYWARD FAULT STANDS the California Department of Transportation's (Caltrans) District 4 headquarters, a 15-story steel moment frame structure that houses the San Francisco Bay Area's transportation operations, including its emergency response team.

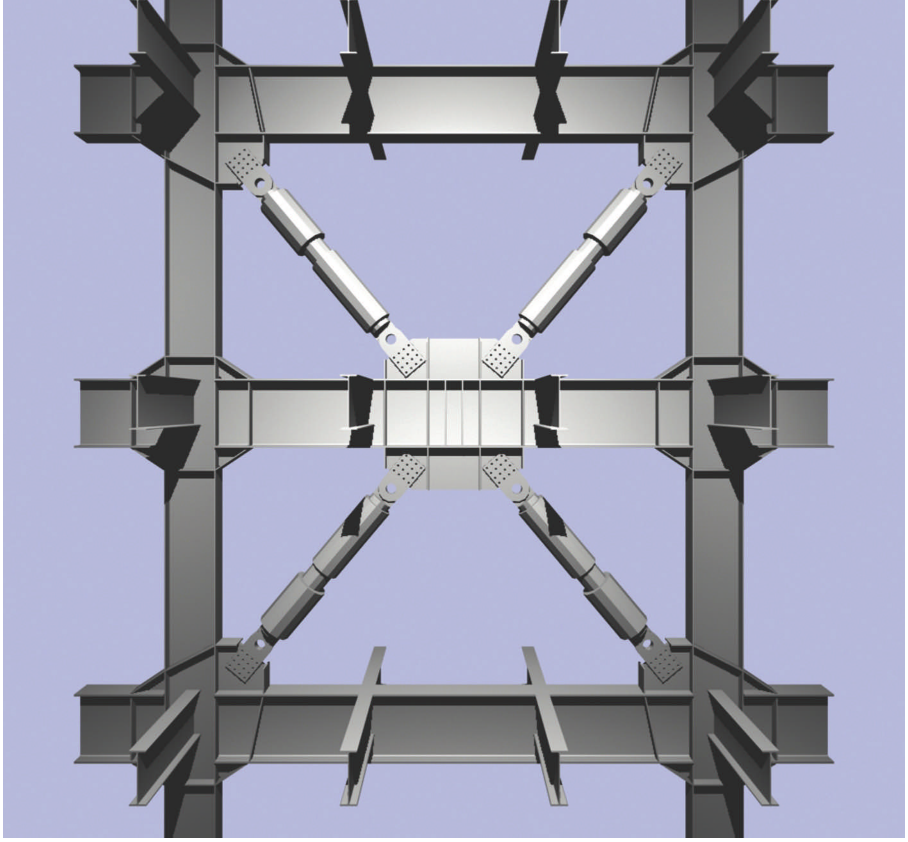
The structure was built two years after the destructive 1989 Loma Prieta earthquake. Its integrity was not questioned until three years after Caltrans' operations moved in, when the 1994 Northridge earthquake damaged numerous steel moment frame buildings in the Los Angeles area. As a consequence, seismic retrofitting became mandatory. A team under the direction of Degenkolb Engineers, and including The Crosby Group, employed extensive state-of-the-art analysis techniques to design and test a retrofit scheme that will protect the building.

Existing Building Performance

Every 150 years or so, the Hayward Fault generates severe quakes. This puts the Caltrans District 4 building in an especially vulnerable period, since the last major event on the fault occurred in the 1868. Experts forecast that the next major earthquake on the Hayward Fault will be in the destructive 6.7 to 7.2 range.

The Caltrans District 4 structure was designed to meet the 1988 Uniform Building Code. It has one basement level, a first-story lobby with public space, four levels of above-grade parking, and ten stories of office space. The building also has a large atrium above the parking levels. Full-height moment frames are located along the perimeter frame lines. There are also two interior transverse moment frames adjacent to the atrium on either side.

Recognizing the potential vulnerability of the existing SMRF system as a result of the Northridge damage, the State of California commissioned a study that included both a preliminary evaluation of the building's structural system and laboratory testing on a few moment connections similar to that of the existing building. These tests were performed by the University of California, Berkeley at the Pacific Earthquake Engineering Research Center. The results confirmed that the existing connections were vulnerable to fracture and demonstrated even less rotation capacity than smaller specimens tested for the FEMA-sponsored SAC Steel Program. The results also led to the conclusion that a seismic upgrade was required. In accordance with the state guidelines, the building needed to be seismically upgraded to an expected performance level that includes minor repairable structural damage, moderate



Degenkolb Engineers

By co-locating the connection upgrades with the damper locations, the seismic strengthening scheme limited the number of work locations.

non-structural damage that may entail extensive repair, minor risk to life, and the ability to return to operations within weeks of the earthquake.

Analysis and Development of Upgrade Schemes

Site-specific response spectra were developed to represent the Design Base Earthquake (DBE, BSE-1) and the Maximum Considered Earthquake (MCE, BSE-2) in accordance with FEMA 356, the document used as the basis of the retrofit design criteria. These spectra represent anticipated earthquakes of Richter magnitude 7.0 and 7.25. At the DBE level this resulted in a first-mode spectral acceleration of approximately 0.4 g. Seven pairs of time-histories for use in the nonlinear response history analysis were also developed and scaled in accordance with FEMA requirements.

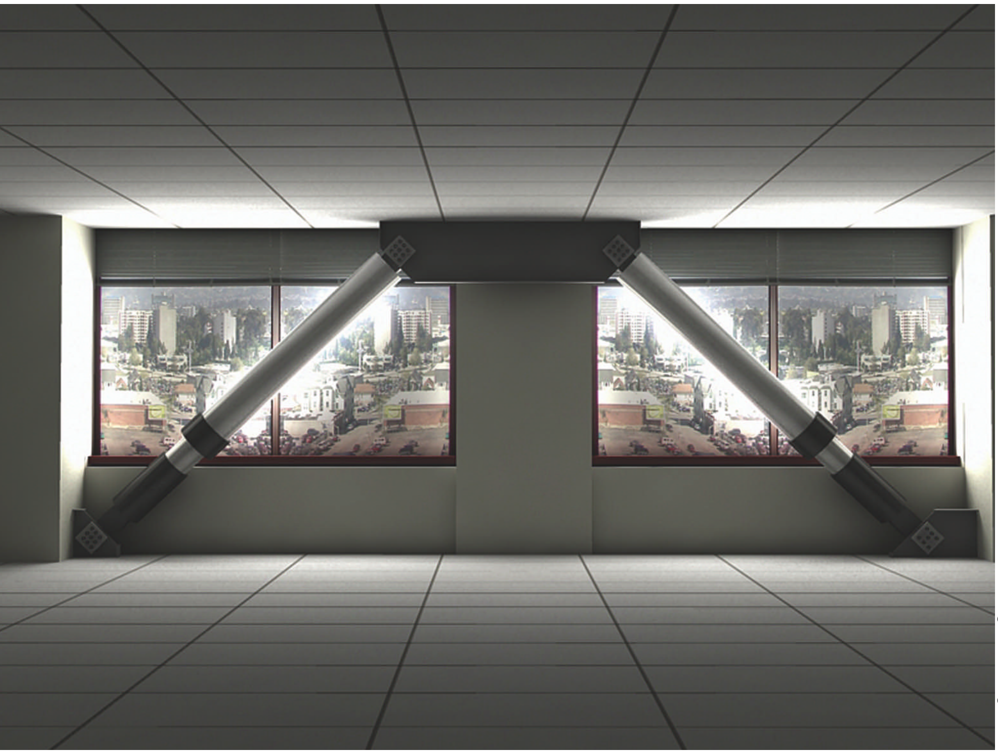
In the early evaluation stages, the team led by Degenkolb performed multi-mode, two-dimensional nonlinear pushover analyses and single-degree-of-freedom nonlinear dynamic time-history analyses to estimate the necessary upgrade measures.

Four strengthening schemes were developed: an all-connection-strengthening scheme, a connection-strengthening-plus-dampers scheme, a buckling restrained brace scheme, and a base isolation scheme. Each scheme was designed to meet the design criteria and was compared on the basis of construction cost and associated "soft" costs such as construction phasing, long-term and short-term staff relocation, and other facility downtime. Ultimately, the scheme that combined connection upgrades with the addition of viscous dampers was selected for the upgrade.

By selecting maximum practical damper sizes and placing the connection upgrades and dampers at the same locations, the selected strengthening scheme minimizes the number of work locations. The damper layout avoids interference with major points of entry and interior building flow and will add 228 dampers to the building's perimeter. Of 1,218 existing moment-resisting connections in building, 746 are being strengthened. Column splices will be strengthened in selected locations where connection strengthening occurs.

In the working drawings phase of the project, two RAM Perform (now CSI's PERFORM-3D) models were constructed: one two-dimensional model for each primary direction of motion. In each model the moment frames were modeled completely, and the gravity columns and orthogonal moment frame columns were modeled for secondary effects. Rigid diaphragms were assumed.

The moment frames were built with compound elements with elastic and inelastic components. Beam elements were



Degenkolb Engineers

Dampers are visible in the interior spaces of the building.

built from an elastic beam section and a nonlinear moment-rotation hinge for strengthened connections or a nonlinear fiber section for the existing connections. Since not all existing connections were to be retrofitted (for economic reasons), a model of the existing connections that simulates their fracture behavior needed to be developed. The existing connection model took advantage of a fiber model technique, with the connection model being comprised of three different types of fibers: one fiber representing the top flange, one fiber representing the bottom flange, and one fiber for each of the bolts in the shear tab. Using the fiber section allowed the existing connection model to closely mimic tested behavior.

The fiber model captured the top and bottom flanges fracturing at different moments, both of which are significantly below the expected moment strength of the beam. It also captured the post-flange-fracture effect, where the bending capacity of the connection relies on the couple between the shear tab bolts in shear and the fractured flange in bearing, and the individual fracture of each shear tab bolt at the expected bolt ultimate strength.

Full-scale Testing to Confirm the Upgrade Design

Deep column sections and large beam sizes were beyond the scope of previous testing on the connection upgrade schemes that were considered, and therefore, connection strengthening schemes involving deep columns and very large beam sections were experimentally tested to validate the proposed rehabilitation scheme.

Four full-scale, double-sided steel moment connection tests were commissioned so that the proposed strengthening scheme could be properly validated. Specimens included a representative width of composite steel deck and concrete slab. Various connection upgrade schemes were considered, based on previous research results and retrofit designs. The scheme considered and tested included a single-welded haunch (WBH), a double-welded haunch (WTBH), a double haunch on one side of the column and a double gusset plate on the other, and a bolted bracket (BB). The BB was considered because its installation could be performed without welding. This would shorten the construction schedule and reduce welding fume containment issues. The tests were conducted at University of California, San Diego (UCSD) under the direction of Professor Chia-Ming Uang. The size of the specimens did lead to upgrade connection performance that differed from previous testing of both the WBH and BB approaches. As a result, the WTBH scheme was selected for the connection upgrades. A final test that simulated the application of damper gusset plates into the moment connection confirmed that these plates could meet the project's drift demands. (More details on the testing portion of the project can be found in a 2006 report by Uang and Newell: *Cyclic Testing of Steel Moment Connections for the Caltrans District 4 Office Building Seismic Upgrade, Report No. SSRP-05/03*, Department of Structural Engineering, University of California, San Diego, La Jolla, Calif.)

Lessons Learned

The simplified analysis used to estimate the performance of the proposed retrofit scheme was reasonably successful in estimating the overall extent of strengthening work required to achieve the desired performance. However, this analysis substantially underestimated the drift in the lower stories and overestimated the drift in the upper stories. This resulted in a significant revision to the damper and connection strengthening scheme configuration.

The extra steps taken beyond typical engineering practices in both the analytical techniques and the full-scale testing were intended to provide better assurance that the project's performance goals would be met during the design-basis seismic event. They also helped to produce an economical scheme (approximately $50 per sq. ft for construction) that will allow the building to remain operational during the entire construction process.

Currently, construction is underway at Caltrans District 4, with little disruption to the agency's operations thanks to the extensive planning that the team undertook to establish an orderly phasing program to orchestrate the work. MSC

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Steel Bridge NEWS
NATIONAL STEEL BRIDGE ALLIANCE
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On Opposite Coasts

BY CRAIG FINLEY, P.E., JERRY PFUNTNER, P.E., AND MATTHEW ADAMS, P.E.

This rendering shows the MIC-MIA bridge, which will provide access to Miami International Airport's rental car facility.

The redesign of the Estero Parkway Flyover project replaces twin, cast-in-place concrete box girders with with a design using four steel box girders.

Two Florida bridges—on either side of the state—will deliver multiple benefits to their owners and users, thanks to value engineering redesign.

LOCATED ON OPPOSITE COASTS OF FLORIDA, TWO CURRENT BRIDGE PROJECTS WILL SERVE VASTLY DIFFERENT PURPOSES.

The Estero Parkway Flyover, near Fort Myers on Florida's west coast, will ease traffic congestion on the parkway and offer travelers an alternate east-west route on the Tamiami Trail and I-75. The Miami Intermodal Center Terminal Access Roadway Project—nicknamed MIC-MIA—will provide access to a rental car facility as part of a major upgrade of Miami International Airport.

But both projects have one thing in common (besides being in Florida): they were both initially designed to use concrete for the superstructure. Both were redesigned in steel by Finley Engineering Group in a value engineering process. And both will now be built faster and will save their respective owners approximately $2.5 million combined.

Out of the Comfort Zone

Tampa Steel is the steel fabricator on both the Estero and the MIC-MIA projects. Robert "Bob" Clark, Jr., the company's president, says that most bridge superstructures are designed in concrete because many bridge engineers are more comfortable and experienced with concrete than they are with steel. "Most colleges teach concrete design in their core courses, whereas steel design is an elective in advanced courses. So many engineers choose concrete because of an absence of knowledge about steel."

Not every project benefits from a conversion to steel from concrete, of course. And despite what some may think, the savings aren't strictly linked to the material costs of the former versus the latter. Donald Deberry, P.E., public works operations manager for Lee County, notes that the recent cost fluctuation for all kinds of construction materials underscores the need for good, solid engineering design, because chasing material prices is a losing game.

"It might look like you're saving money when you evaluate price during development of the project or the bridge development report," he says. "But later, when you actually go to buy it, you might find that you would have been better off using something else because of price fluctuations in the materials market."

On both the Estero and MIC-MIA projects, three factors other than material price dictated that steel was the better choice:

Site conditions. The original design for the MIC-MIA project called for a cast-in-place concrete-on-falsework section combined with concrete U-beam superstructure to make up the 584-ft-long bridge. As designed, the construction would have been excessively complicated and labor-intensive. The value engineering redesign included only one superstructure type. And by converting the superstructure to steel, the redesign eliminated the need for falsework, greatly simplifying construction and minimizing the impact on ongoing operations and construction projects at the rental car facility.

According to one of the subcontractors working on the FDOT MIC-MIA project, the original design would have required substantially more temporary shoring towers on the site, thereby impeding the principal access to the site. As such, the shift to steel box girders

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KC Crossing

Construction is scheduled to start this spring on the $245 million KCICON (Interstate Connections 29/35) Project in Kansas City. This is the second of three design-build projects being undertaken by the Missouri Department of Transportation (MoDOT).

Paseo Corridor Constructors, a joint venture partnership of Clarkson Construction, Massman Construction, and Kiewit Construction, will lead construction efforts. Design is being handled by Parsons Transportation Group and TranSystems. PDM Bridge (AISC/NSBA Member) is the steel fabricator for the project.

The project will reconstruct/rehabilitate a four-mile section of Interstate 29/35 leading into downtown Kansas City. Along with highway widening and interchange reconstruction work, the project will replace the existing Paseo Bridge over the Missouri River with the Christopher S. Bond Bridge, a 124-ft-wide, 1,676-ft-long cable-stay structure.

The cables will stretch across a 550-ft main span and 451-ft side span, with a single composite steel plate girder span connecting to the structure's south end and a multi-girder span arrangement on the north end.

A closed, delta-shaped pylon extending 250 ft above the roadway surface will anchor the 40 parallel strand cable stays, creating a unified focal point for the structure and a gateway experience for those driving across it.

To encourage the creation of an aesthetically pleasing bridge, representatives of a community advisory group were given 20 points with which to score the two design-build proposals. This marked the first time a state transportation agency has allocated selection points to community members. MSC

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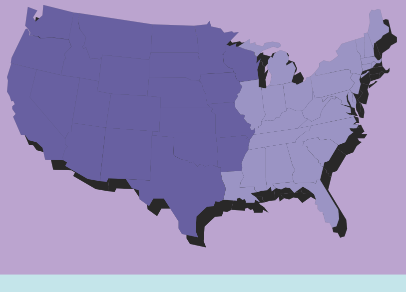

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significantly reduced the amount of shoring required.

The Estero site had similar site restrictions—and design solutions. The redesign replaces twin, cast-in-place concrete box girders with a single bridge using four steel box girders.

This eliminates a large falsework support system, reduces foundation design requirements, and simplifies construction. The overall result is a shorter time frame to complete the bridge, an obvious benefit to the traveling public.

Falsework in the original design would have also more significantly affected a design-build project to widen I-75, which runs beneath the bridge site. It would have created more safety hazards for motorists, and the falsework erection and cast-in-place pours would have slowed traffic far more often than will occur with the placement of steel girders on the superstructure.

Contractor means and methods. Value engineering redesigns are often driven by the preferences and experiences of the contractor chosen for the work. Some contractors are simply better with one material than the other.

Contractors also prefer designs that are not overly complex and that allow them to make money on the job. On both the MIC-MIA and the Estero projects, elimination of the falsework reduced the complexity and the amount of labor that would be required to complete the work. Particularly in a construction climate where qualified workers have been difficult to find, this is a welcome change for the contractors.

Material cost did play some role in the Estero project. Jovan Zepcevski, president of Estero contractor Zep Construction, says steel prices were relatively high during the initial design period, but that they eventually receded enough to make steel the clearly better choice during the value engineering portion of the project.

The less complex construction process will also help the contractors meet their schedules. For example, in the case of the Estero project, Zep must finish the job in less than two years. For the Miami project, completing the structure quickly alleviates coordination issues associated with large, integrated construction projects.

Owner requirements. Lee County will pay a minimum of $1.85 million less for the redesigned Estero Parkway Flyover than it budgeted under the original design. The county coffers may get even more back, as the contractor and the county will split any additional savings.

The redesigns saved money in a number of ways. On the Estero project, the conversion to steel resulted in a reduced superstructure depth. This shortened the required approach embankment height, so the project won't need as much fill on each approach. A reduction of about 4 ft in the fill height at the beginning and end of the bridge, tapered over the length of the nearly 700-ft-long approach embankments, means massive savings in mechanically stabilized earth (MSE) fill.

In both projects, the change to steel reduced the number of piles required, and in the case of the MIC-MIA project, one of the piers could be eliminated. Because the steel boxes produce a lighter superstructure, the redesign resulted in a more efficient substructure design, meaning that fewer piles were necessary. In the Estero project, the number of 24-in. precast piles dropped from 130 to 76. On the MIC-MIA, the redesign increased the precast piles from 18 in. to 24 in., but reduced the number of piles from 163 to 60.

A New Perspective

An important lesson to take away from both projects is that value engineering provides the opportunity for a new look at a project by involving the contractor. That new look may very well uncover a better way to build the bridge—as it did in these two cases. Taking the opportunity to look at a project, with the engineer and contractor both contributing ideas on how to best accomplish the project's objectives, can add *better*, *faster*, and *less costly* to the overall project accomplishments. By incorporating a value engineering redesign, the project team is loudly and clearly stating that the client's needs are paramount. MSC

Craig Finley is managing principal, Jerry Pfuntner is a principal, and Matthew Adams is a bridge engineer, all with Finley Engineering Group.

For more on the Estero Parkway Flyover project, see December 2007's Steel Bridge News, available online at ***www.modernsteel.com/archives****.*

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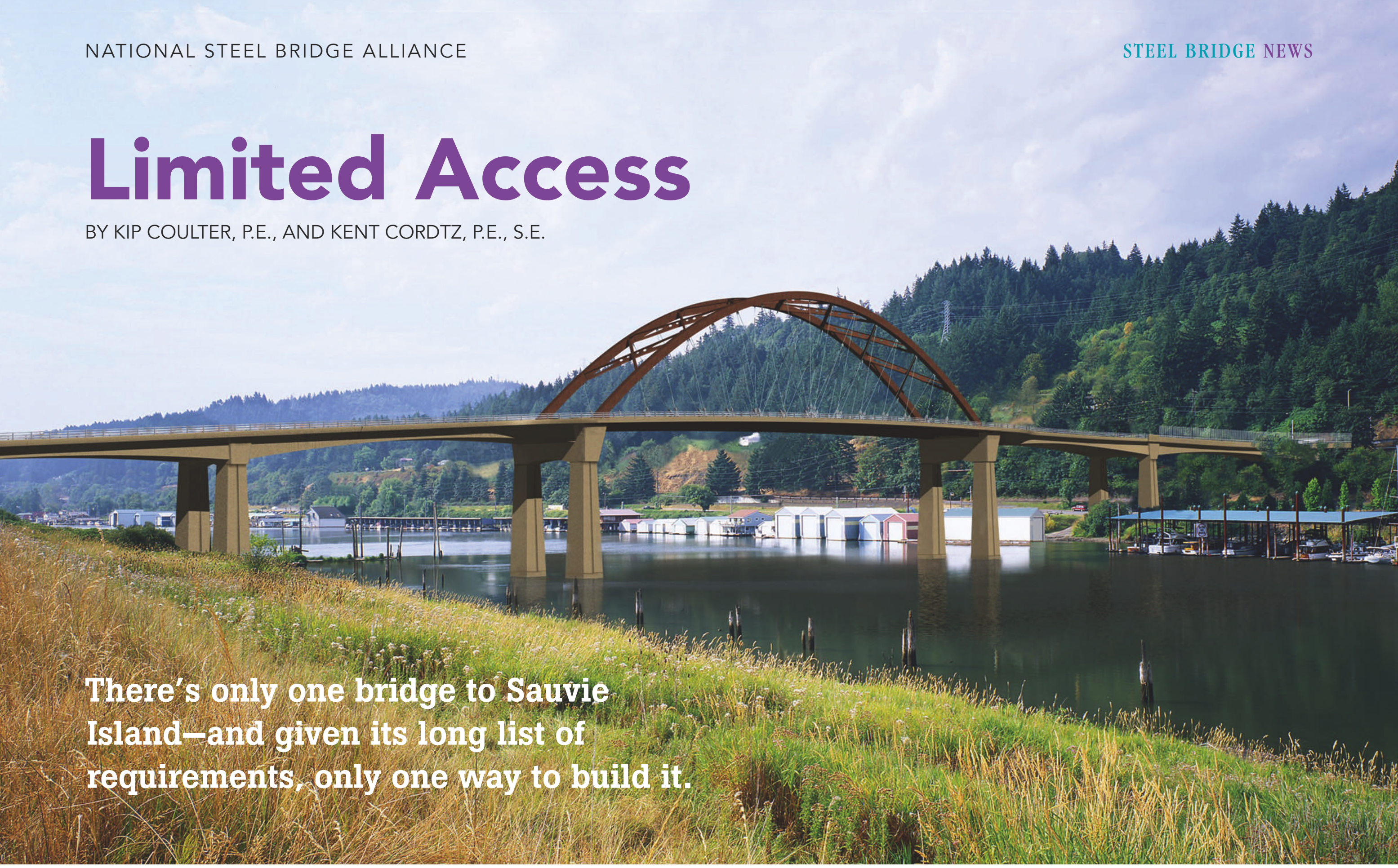
Limited Access

BY KIP COULTER, P.E., AND KENT CORDTZ, P.E., S.E.



There's only one bridge to Sauvie Island—and given its long list of requirements, only one way to build it.

Photo simulation of the five-span replacement bridge. (Courtesy 2L2 Architects/Planners LLC)

THE IDEA OF "A TOUGH ACT TO FOLLOW" isn't limited to the entertainment business. In the construction world, for example, such a situation might come up in the form of having to replace a structure that's eligible for listing on the National Register of Historic Places—while also having to pay homage to it.

Such is the case with the Sauvie Island Bridge in Oregon. Located approximately 10 miles northwest of downtown Portland, the bridge provides the only vehicular access to Sauvie Island, a largely agricultural 24,000-acre island bounded by the Columbia River, the Willamette River, and Multnomah Channel.

The original 1,198-ft long bridge, constructed in 1950, featured 14 spans. It was constructed using concrete girder and steel deck truss approach spans, and featured a steel through truss over the main navigation span. Despite its historical significance, the bridge required emergency structural repairs and was eventually classified as functionally obsolete and structurally deficient—and slated for replacement.

Considering Configurations

Led by project owner Multnomah County and with the active participation of the public and various stakeholders, viable alternative configurations for the replacement bridge and its approaches were developed to meet the following key criteria:

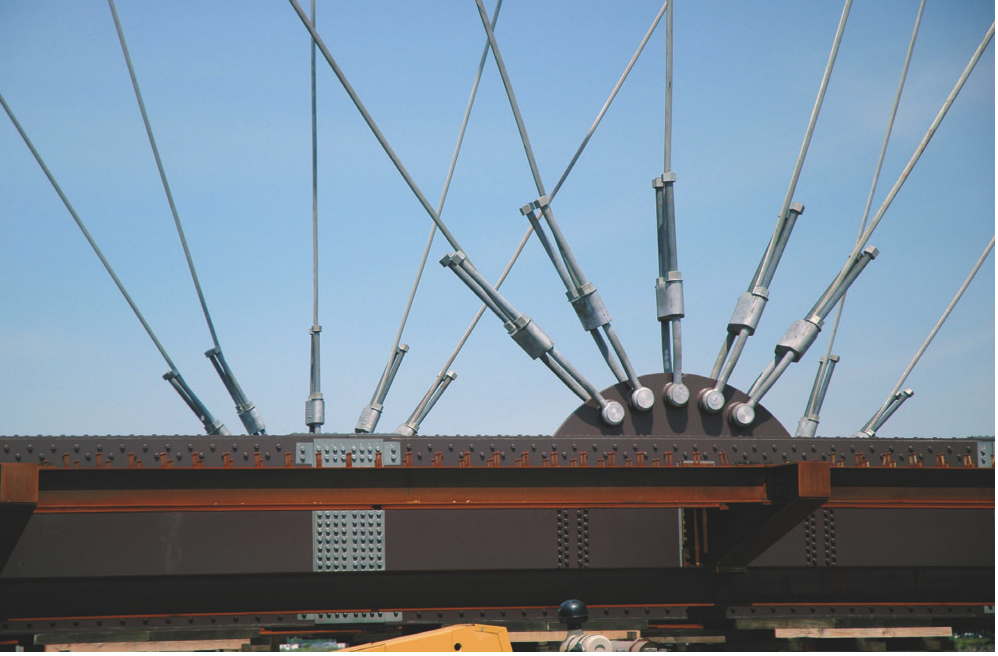
- The State Historic Preservation Office (SHPO) required that a portion of the replacement structure be above the bridge deck (i.e., a through truss or arch) in order to memorialize the historic existing through-truss bridge.
- The main bridge span was required to clear a 175-ft-wide by 52.5-ft-high navigation envelope (above the 100-year high-water scenario).
- The maximum grade on the bridge and its approaches was limited to 6%.
- The bridge must be constructed without falsework in the channel, and with minimal disruption to navigation.
- Life cycle costs, environmental impacts, construction duration, aesthetics—and, of course, budget—were important considerations.

A total of 21 structure alternatives were initially identified, and a multi-step process was employed to screen these down to the options that best met all project criteria. The approach grade constraints, coupled with the required vertical navigational clearance, quickly led to the conclusion that a structure with a very shallow floor system was required; this would also support the SHPO requirement for a through structure. A half through-steel arch with a 425-ft center span and twin 155-ft side spans was initially preferred for the main span, but rising construction costs forced the team to consider other similar but less costly alternatives.

Steel Tied-Arch

The team ultimately selected a 5-span, 1,177-ft-long replacement bridge featuring a 365-ft Grade 50W weathering steel tied-arch main span. The tied-arch and its unique radial cable pattern satisfied the stakeholders' desire for an aesthetically pleasing bridge while meeting the stated project and site constraints. The graceful steel tied-arch span reduces the number of piers in the channel, permits an increased navigation opening, meets vertical clearance requirements, and could readily be constructed without requiring temporary falsework.

Haunched post-tensioned concrete box-girder approach spans, constructed on falsework, complement the slender tied-arch main



David Evans and Associates, Inc.

Than hanger cables are 2.5-in.-diameter galvanized structural strand. The cast-steel sockets for the lower strand connections use molten zinc to fuse the strands into a conical "basket," locking in the cable tension.

span, and are reminiscent of the existing concrete approach spans.

Tied-arch bridges employ tension-tie girders in the plane of each arch to resist all arch thrust forces; no horizontal thrust forces are transmitted externally from the arch span to the supporting piers. The tie girders also support the transverse floor beams that carry the roadway deck structure, and resist local bending moments and deflections resulting from dead loads and moving live loads. Since complete fracture of either of the two tension tie girders could result in structure collapse, these important main members are considered to be fracture-critical. The fracture-critical nature of the main tie girders was addressed by detailing the tie girders as fully bolted members without any welding. The tie girders are built-up steel box sections consisting of web and flange plates connected by bolted corner angles. This provides for internal redundancy, as a fracture in one plate cannot propagate to the entire cross-section and lead to collapse. The bolted built-up tie girder was designed for the loss of a single web or flange plate using a special LRFD Extreme Event load combination.

The arch ribs consist of welded box sections with internal diaphragms at the hanger locations. The depth of the ribs were dictated by the minimum access openings through the diaphragms required by Multnomah County, to facilitate internal access for periodic inspection.

Hanger Cables

The hanger cables consist of 2.5-in.-diameter galvanized structural strand, per ASTM A586. Non-adjustable cast steel open-strand sockets are provided at the upper connection to the arch rib. Cable tensioning is performed at the open bridge sockets provided at the lower connection to the tie girder. Zinc spelter sockets are employed to attach the structural strand to the anchor sockets. This method of attaching structural strand cables to cast steel sockets has been in use for many years, and employs molten zinc to permanently join the individual wires of the structural strand to a conical "basket" in the cast steel socket. The resulting hanger cable assemblies provide a dependable and internally redundant tension member.

Bridge hanger cable assemblies, and the hanger plates to which they are attached, are designed with a minimum factor of safety of 4.0 for breaking strength versus unfactored dead load plus live load and impact. Bridge hanger cable assemblies are also designed for the loss or replacement of any one cable under traffic, with a minimum factor of safety of 3.0.

The unique radial cable pattern is not as structurally efficient or as stiff as a traditional vertical cable pattern or a crossed-cable pattern, but it was selected during the public involvement process primarily on aesthetic value. In other recent tied-arch projects, a crossed-cable pattern has been found to be most effective in stiffening the entire structural system and minimizing differential live load deflections, particularly when the live load is placed at the one-quarter point of the arch span.

An iterative process was employed to develop the most efficient arch shape, as the cable forces, arch rib and tie girder moments, and deflections are extremely sen-

David Evans and Associates, Inc.

David Evans and Associates, Inc.

The tied arches and floor system were assembled eight miles from the bridge site and delivered to the site on barges.

sitive to the arch-rib geometry with this unique cable pattern. The final shape of the arch differs somewhat from the classic shape of a uniformly loaded arch, and also from the common approximation of that shape by a second-order parabola.

Floor System

The floor system consists of longitudinal stringers supported by transverse floor beams. Due to vertical clearance requirements over the navigation channel and roadway approach grade restrictions, the top of the stringers and floor beams coincide. This results in the least structure depth and the lowest roadway profile. The stringers are composite wide-flange sections with moment connections at the floor beams. The floor beams are composite welded plate girders with moment connections to the tie girder that occur at each hanger cable node.

Erection Schemes

A significant challenge of the project was to develop a feasible means of erecting such a large steel structure. To attract the maximum number of bidders, the tied-arch span was designed to be erected by either of two methods. The first method was cantilever erection using temporary towers and stay cables, with material delivery and erection by barges. The towers would be located on the piers adjacent to the channel, with backstays anchored to the approach spans. The second method was to assemble the tied-arch and floor system off-site, deliver it to the site on barges, and erect it on the piers. Both methods were presented in the plans as suggested erection schemes only; the contractor was responsible for performing the final erection engineering for his chosen scheme. The contractor selected the float-in erection method, because it allowed for concurrent construction of the approaches and main span, as well as a savings in schedule.

Fabrication and Erection

The steel fabricator employed a progressive shop-assembly technique to complete final fit-up of all major steel members. The components were then dismantled and shipped to the Port of Portland dock on the Willamette River, approximately eight miles from the project site, where the final assembly was completed. The tie girder and floor system were assembled to the correct cambered geometry on timber blocking supported on the dock, followed by arch assembly from temporary shoring towers supported on the tie girder. Temporary compression struts between the arch and tie girder were installed to stiffen the structure during load-out and erection. Following steel assembly, the hanger cables were installed and tensioned to the specified initial tension.

The 365-ft arch structure was raised on the barge at the assembly dock, and the bridge span was eventually floated the eight miles to the project site. Once at the site, the span was carefully lowered and guided onto temporary bearing pedestals at its final vertical and horizontal location.

When completed this year, the new Sauvie Island Bridge will include 1,250 tons of steel and will provide the required capacity to support the heavy vehicles operated by the island's agricultural and industrial businesses, while also providing for safe bicycle, pedestrian, and truck use. And the chosen steel tied-arch meets the project's stringent engineering and permitting requirements while also satisfying the aesthetic and historical desires of the stakeholders. MSC

Kent Cordtz is the bridge discipline director for David Evans and Associates, Inc. and is the Engineer of Record for the Sauvie Island Bridge. Kip Coulter is the bridge discipline leader for the Denver office of David Evans and Associates, Inc. and is the tied-arch design leader for the project.

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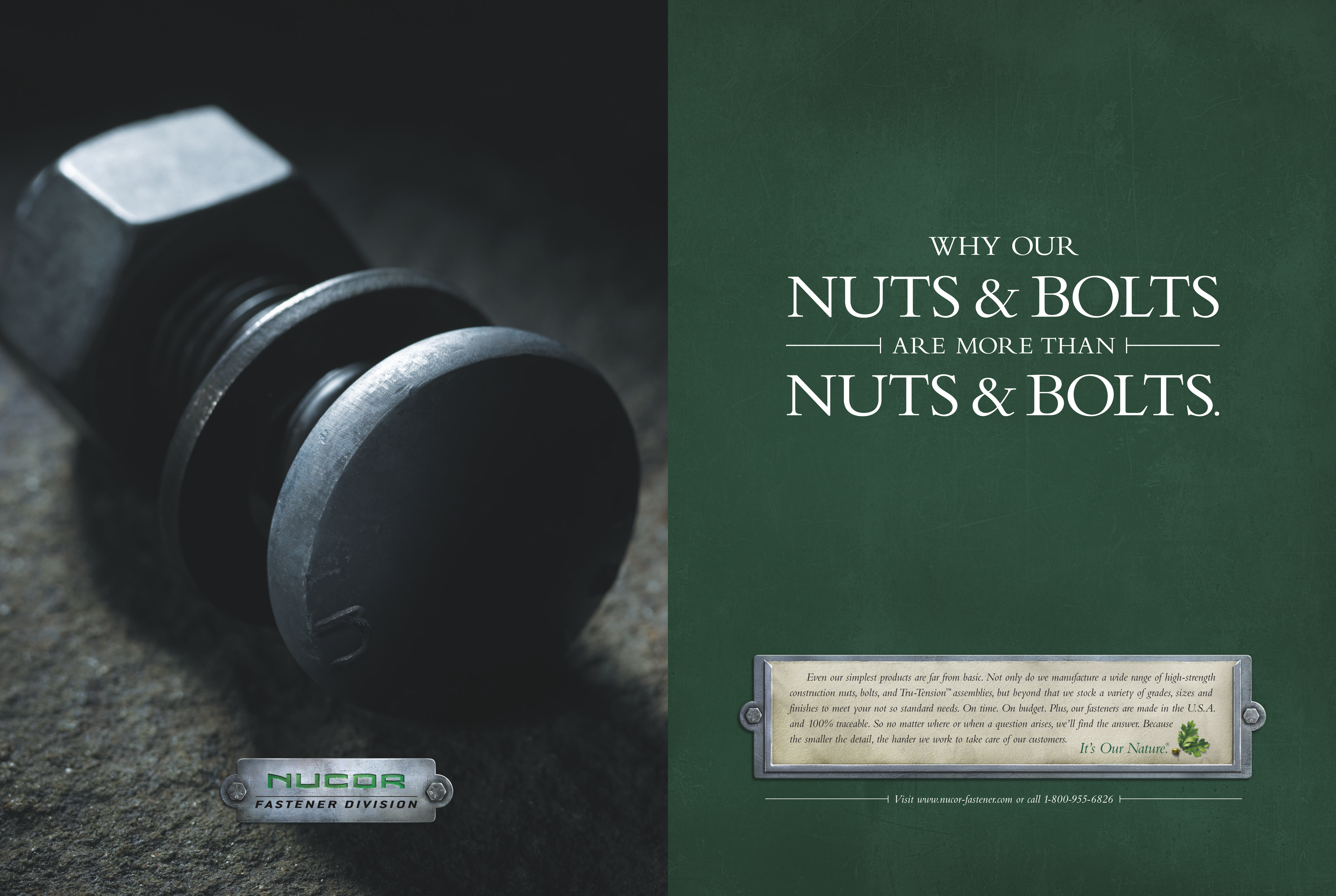
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Getting the Most from Certification

BY RUSS PANICO

AISC Certification gives fabricators a foundation for building problem-prevention strategies.

AISC CERTIFICATION IS A NATIONALLY AND INTERNATIONALLY RECOGNIZED PROGRAM whose development has followed the format of the ISO Program. AISC Certification requirements are steel-specific, focusing on fitting, welding, bolting, and coating issues, among other things; the program takes aim only at those specific areas that engineers and builders care about. It provides a process for not only improving the manufacturing and material-handling activities, but also improving engineering, sales, purchasing, and management.

Components of AISC's *Standard for Steel Building Structures* that can be directly applied to prevent rework and errors include:

Fabrication

A fabricator is required to provide a comprehensive and effective calibration system. Although this calibration system is not a very visible benefit, calibrated equipment (including welding machines) is critical to accurate fabrication. Bolting methods will be reviewed for completeness and accuracy during the audit. With a review of written procedure and application, fabricators are confident that their connections are sound and consistently made. Fracture-critical welding programs will receive a complete review. The very definition of fracture-critical demands accurate and thorough compliance. Our shop's fracture-critical program was built using the AISC Certification criteria as a resource. Why reinvent the wheel when you don't have to?

Materials

Materials are fundamental to the fabrication process. Steel plate, welding consumables, paint, and many other material components must be controlled and stored properly in order to prevent material-related issues from reaching the jobsite. Traceability can also be critical, depending on the type of project. The Certification program addresses how a fabricator will handle and store material, along with how they will control non-conforming material. These measures are required in order to prevent problems of traceability and confusion.

Documentation

Meeting AISC Certification requirements helps develop a robust quality system. AISC Certified fabricators will have a great deal more documentation control for consistency and trainability of their staff. Providing objective evidence of what you've done in your shop can prove significant when someone asks to see it. How do you train? What do you train with? Using documented procedures as training material can help keep your training programs consistent. An AISC Certified fabricator can easily provide answers to the above questions.

Contract Review

AISC Certification supports a quality management system that ensures customer requirements are fully defined prior to accepting an order. The program also enables fabricators to ensure that procedures are in place to resolve discrepancies between customer requirements and fabricator capabilities. If conflicts can be resolved early in the bid process, there is less chance of confusion regarding the fabricator's responsibilities. We have found that using systems that review and address customer requirements has significantly improved the internal communication among our sales, engineering, and manufacturing departments.

Supporting Project Design

Certification improves the consistency of the instructions we provide to manufacturing. It accomplishes this by ensuring that an effective review system is in place to support project design and prevent design errors. Certification criteria provide detailed feedback through the "request for information" (RFI) process to question and resolve design issues. Whether the Fabricator details jobs themselves or sublets detailing, the *Standard* addresses qualification and accuracy of the detailing. It will also help you set up a method for qualifying and maintaining sublet detailers.

Vendors

AISC Certification criteria reinforce the importance of accuracy when communicating material requirements to your vendors, and also allow the fabricator to communicate more effectively. The program can help initiate formal actions that are meant to improve your vendors' level of service. In our company, non-conformance reports let us track and categorize mill errors to provide strong and accurate feedback.

Management

There are many benefits to a fabricator's quality management system, such as the management review. A review meeting, which must be held on a regular basis, gives the fabricator's management team the opportunity to review the overall effectiveness of the quality system. Problems and needs are discussed at the highest

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**.

level, and these meetings provide our quality department with improved visibility to the management team.

Company-wide corrective action requests (CARs) are established to document problems, their root causes, and what measures are taken to prevent these problems from occurring again. As an example, our own CAR system is a valuable tool for preventing rework issues. Personally, our system has helped me to better organize and follow up with recurring problems and errors.

A comprehensive training system, including the development of standard work processes for consistency and improved quality, has been developed in response to the AISC Certification requirements. Consistent training, along with established written procedures, provides the antidote to many of our problems.

The AISC Certification criteria also focus on internal auditing. There is an old saying, "You get what you inspect, not what you expect," which certainly applies in this situation. The fabricator checks himself in all areas of the program criteria at least once per year. This enables the fabricator to find and self-correct any problems that are discovered in the self-audit. In our company, self audits have disclosed many areas for improvement, including calibration and documentation issues.

Goals

Setting goals is one of the most unique and valuable components of the AISC Certification program. The fabricator sets his own goals for improvement, but it is actually the progress towards achieving these goals that is audited. This concept is a real driver toward continuous improvement. Once initial goals are met, new ones are established and the cycle of improvement continues. Our company has specifically set goals to reduce paint rework and weld rejects. With the support of management, we have made real, quantifiable progress in both areas.

Overall Communications

The *Building Standard* has provided a framework to help improve communication between owners, designers, and fabricators:

- ➜ A more thorough and consistent review of drawings and specifications brings problems to light sooner in the bidding process
- ➜ Consistent, organized RFIs track and resolve design and other issues
- ➜ Standardized documentation of corrective actions, preventive actions, (CPARs), travelers, and other reports provides objective evidence of meeting requirements
- ➜ Standardized fabrication procedures
- ➜ Standardized repair procedures

For owners, contractors, and fabricators, AISC Certification focuses on smooth contract fulfillment. By establishing and maintaining communications between all parties, the *Building Standard's* framework enables the fabricator to prevent design and material issues, among others, which could hinder fabrication. AISC Certification shifts the focus to the customer. A strong, consistent quality system emphasizes the importance and effectiveness of problem prevention and improves the way the business operates.

Quality Management Company's auditors work with your company to improve your quality management system. The focus of these audits has changed from passing inspections to developing a customer-driven, process-based quality system. With management leadership and defined goals, your company can increase productivity like ours has. MSC

Russ Panico is Director - Quality at High Steel Structures, Inc. in Lancaster, Pa., a company that has been involved with AISC Certification for more than 25 years.

THE FINANCIAL ESSENTIALS OF STRATEGIC PLANNING

BY IAN RUSK

A good business planning effort should begin with a thorough financial analysis of the firm's performance.

I'VE HAD THE PRIVILEGE of participating in many strategic business planning sessions for A/E/C clients over the years. One thing I've noticed in many such sessions is that analysis of the firm's financial performance and setting financial goals usually get short shrift. In fact, more often than not, discussions of financial matters are often limited to a brief presentation by the CFO or controller, often at the tail end of the planning meeting (assuming any time is left). This is a mistake!

Let's back up a step and ask ourselves what the purpose of the business planning session is. The planning session, and the preparation that leads up to it, is our opportunity as business leaders to lift our heads up from our day-to-day work and assess what we are trying to accomplish as a business enterprise (our vision or mission), objectively assess how we are doing, identify ways that we may improve, and ultimately lay out an action plan with measurable goals and clear responsibilities.

An Objective Perspective

That first step—the objective assessment of how the firm is doing—can take many forms. We might survey our clients about their level of satisfaction with our services, or we might look inward and survey our own staff to identify any managerial or workplace issues that need to be addressed. But the most quantitative answers to the question of "how we are performing as a company" lie in the numbers, and as the saying goes, numbers don't lie.

To illustrate my point, let's pick a problem that many firms might find themselves dealing with in our current economic climate: employee turnover. Employee turnover is actually not the problem itself, but rather an observable symptom of an underlying issue. That issue might actually be a lack of perceived upward mobility on the part of your staff, overwhelming workloads, interpersonal conflicts, etc. Calculating the turnover rate itself neither identifies the issue nor quantifies its impact on the organization. However, you should be able to quantify the impact of this and other "issues" your firm may be facing through some simple financial analysis.

The impact (if any) of employee turnover should manifest itself in measures such as the firm's utilization rate (chargeability), training and development costs, effective labor multiplier, and ultimately its profit margins. These are all financial metrics—that is, metrics derived from the firm's financial statements (income statement and balance sheet). And, unlike a simple employee turnover rate calculation, these metrics will allow you to quantify the economic cost of the issue, which in turn will advise how much time, effort, and expense you ought to be devoting to correcting it. You might find, for example, that utilization has dropped three percentage points over the last year as a result of the increased turnover. And that, based on your size and billing rates, translates into $50,000 in lost revenue.

A good business planning effort should begin with a thorough financial analysis of the firm's performance. We recommend looking back over the last three to five years, even if you perform such analysis annually, as it will allow you to identify trends and put your financial performance and condition into an historical perspective. For further perspective, we recommend benchmarking your firm against your peers. Peer data is readily available through a variety of sources. When assisting our clients, we like to use benchmarking data from ZweigWhite's annual survey publications because of the level of confidence we have in how the data has been compiled, as well as our ability to access the underlying database to make the most meaningful peer-to-peer comparisons. In this sort of analysis, an "apples-to-apples" comparison—or as close to it as you can get—is crucial.

Peer benchmarking data will tell you where you lag behind industry norms and where you excel. This sort of analysis, when overlaid with more qualitative data, can be truly insightful, helping you to identify the issues that are having the most economic impact on the firm. This will allow you to prioritize your business planning efforts and resources. Financial metrics can then be used to set goals and measure real progress over the course of the year.

Ian Rusk is an executive vice president at ZweigWhite and the leader of its Advisory Services and Management Education groups. He can be reached at irusk@zweigwhite.com.

Analyze This!

Below is a guide to the sort of financial analysis that should precede your business planning efforts:

Revenue growth. Begin with the income statement. How have net service revenue levels (gross revenue less reimbursable and sub-consultant expenses) changed over the course of the year? What factors or efforts are driving the revenue growth, or causing a decline? If external economic factors are an influence, how can you capitalize on the opportunities they represent, or mitigate the negative impact of a poor economy?

Profit margins. The most meaningful measure of profitability is on a pre-tax, pre-discretionary distribution basis, stated as a percentage of net service revenue. How does the firm's profit margin compare to its peers? What has the trend been (increasing or declining)? What factors have been impacting profits? Sometimes the answers are obvious, but other times it's difficult to understand what factors are having the most impact on profits. When this is the case, analysis of several financial metrics can help.

Revenue factor. This is the ultimate litmus test of an A/E/C firm's performance. It represents the amount of net service revenue earned for every dollar of payroll (not including payroll benefits and taxes), which represents your largest expense category. How does this compare to your peers? What has been the trend within your firm? Determining the cause of a high or low revenue factor can be easier if the metric is broken down into its two components: utilization and labor multiplier.

Utilization, or chargeability, is the measure of direct (billable) labor cost to total labor cost. Higher utilization rates increase the revenue factor and typically result in above-average profit margins. A low utilization rate can be a symptom of many things. It could reflect a top-heavy firm with too many principals in unbillable roles. It could also reflect a firm that's simply overstaffed.

Very high utilization rates can also reveal potential issues. Firm-wide utilization rates well in excess of industry norms can point to tough working conditions and an unsustainable workload. While the firm may be enjoying the economic benefits at the moment, it could see negative ramifications in the long run.

The other element of the revenue factor, the labor multiplier, measures the relationship between net service revenue and direct (billable) labor costs. The multiplier a firm is able to achieve can reflect the value of its services in the marketplace, the competitive environment, and the efficiency of its staff.

We've found that firms with very high multipliers are often operating in a niche where they enjoy a significant competitive advantage or have established themselves as experts in one or more very specialized services or project types. In contrast, firms that have low multipliers are often providing undistinguished commodity services or operating in markets with very high competitive pressures. Understanding what factors are influencing your firm's multiplier will help guide your strategic planning—allowing you to sustain a strong multiplier—or take steps to address a weak one.

Overhead rate. This ratio measures the relationship between overhead expenses (including non-billable labor) and direct (billable) labor. If you do public-sector work, don't confuse this metric with your audited overhead rate under Federal Acquisition Regulations (FARs). What this metric illustrates is your firm's level of indirect (nonbillable) expenses in relation to direct labor cost. There are many issues that can contribute to a high or low overhead rate, and you may have to take a deeper look at the various categories of operating expenses on your income statement to understand what's influencing your overhead rate.

We often find that small firms with multiple offices have high overhead rates due to the difficulty in keeping multiple offices fully occupied and the inefficiencies of managing staff in multiple locations. A low utilization rate will also have a big negative impact on a firm's overhead rate. The overhead rate is a metric that tends to fluctuate significantly as a firm grows and hits points where it must make significant investments in overhead expenses to support continued growth, such as hiring an HR director or investing in new software or hardware.

Average collection period. This metric is an indicator of cash flow. It measures the amount of time it takes to convert accrued revenue to cash. We've observed many cases of otherwise successful firms being crippled by poor cash flow, so don't ignore this aspect of your financial performance. A poor average collection period may point to procedural inefficiencies with invoicing and collections, poor project management discipline, or even project delivery problems.

Balance sheet analysis. Take a hard look at the makeup of your balance sheet and how your firm is capitalized. How much debt are you carrying in comparison to your equity and/or your total asset base? What is the ratio of your current assets (assets that can be converted to cash within one year) to current liabilities (obligations that must be paid within one year)? This is sometimes referred to as liquidity.

How to capitalize a growing firm is one of the most important strategic decisions you can make. Growth inevitably requires investment in both working capital and fixed assets. Analysis of the current state of your balance sheet will help you make an informed decision as to what sources of capital to use. Should you make more use of bank financing, or should you seek to raise equity capital by creating more ownership opportunities for key staff?

You should also examine what liabilities the firm may have that do not appear on the balance sheet. For example, does the firm have upcoming stock-redemption liabilities (i.e., retiring shareholders), and how do you plan to manage those obligations?

Other Indicators

Of course, there are many other financial indicators that you could and should be examining as part of the business planning process. We've only scratched the surface here. But hopefully, this illustrates the value of conducting a thorough and thoughtful financial analysis of your firm as part of your business planning preparation. Doing so can help you assess your relative strengths and weaknesses, identify underlying issues to be addressed in the planning process, and set measurable goals for tracking the effectiveness of the initiatives that you take. MSC

This article first appeared in The Zweig Letter, Issue # 700, originally published February 19, 2007.

TurnaSure introduces the greatest aid to structural bolting since the DTI

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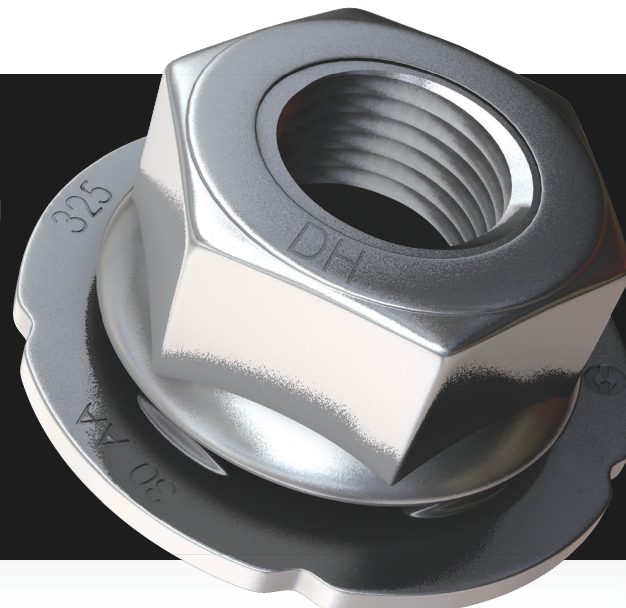

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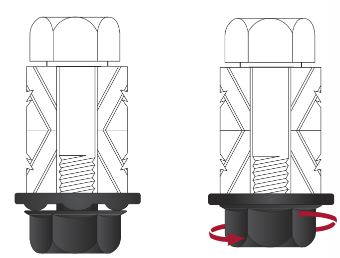
BEFORE TIGHTENING AFTER TIGHTENING



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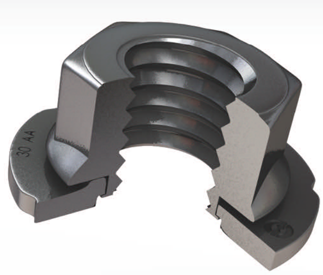

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BOLTING STANDARDS UPDATE

A look at recent additions and updates to industry standards for structural bolts.

COMPILED BY GEOFF WEISENBERGER

MSC ASKED A HANDFUL OF BOLTING EXPERTS to comment on recent changes in fastener standards and how they will affect the steel industry. Here's what we found.

ASTM A307

Lindsay: One recent change that will have a significant impact on the construction fastener industry is the 2007 elimination of the grade C designation within the ASTM A307 specification. ASTM A307 is the standard specification for low-carbon steel construction fasteners. Until recently, A307 had three grades: A, B, and C. Grade A covers bolts for general applications, grade B covers heavy hex bolts and studs for cast iron flanges, and grade C covered unheaded threaded rods, either bent or straight, intended for structural anchorage purposes. Last year's elimination of A307 grade C is the result of a virtually identical specification: F1554 grade 36. Developed in 1994, this specification replaces A307 grade C. It has taken structural engineers many years to become familiar with the F1554 specification, but now as F1554 grade 36 has become more commonplace, ASTM didn't see the need to have two specifications that covered the same item and thus eliminated the grade C designation of ASTM A307.

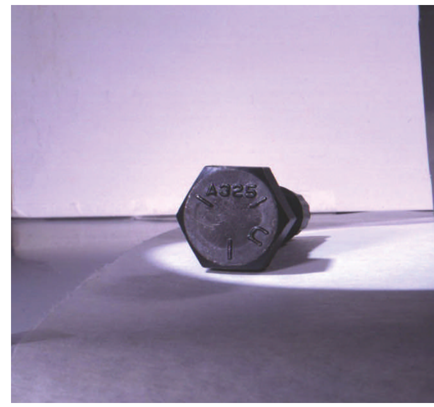
Although A307 grade C and F1554 grade 36 are virtually identical, there are some subtle yet very important differences. The ASTM F1554 specification was introduced in 1994 and covers anchor rods† designed to anchor structural supports to concrete foundations. There are three grades—36, 55, and 105—with the grade corresponding to the minimum yield strength of the anchor rod. F1554 grade 36 is manufactured from low-carbon steel just like ASTM A307 grade C was. But in addition to being a bent or straight anchor rod, it can also be a headed bolt that is embedded in concrete and used for anchoring purposes.

There are also some important differences with regard to mechanical values of both specifications, with F1554 grade 36 possessing more stringent requirements than ASTM A307 grade C. Most commercially available all-thread rod that meets ASTM A307 and is used for anchor rods will not meet ASTM F1554 grade 36.

Additionally, imported hex bolts that meet A307 and are commonly embedded in concrete and used as anchor rods will also not meet F1554 grade 36. However, many fastener distributors, and even some manufacturers who do not have a thorough understanding of the differences between ASTM A307 grade C and F1554 grade 36, continue to provide A307 bolts, believing they will cross-certify to ASTM F1554 grade 36, which is simply not the case in most instances. This practice, which has become commonplace in our industry, is exposing contractors to a significant amount of liability, since contractors rely on fastener companies to have a thorough comprehension of the ASTM specifications and provide products that meet all requirements of the ASTM F1554 grade 36 specification. This is also a continued source of frustration for Portland Bolt, since we routinely compete against companies substituting less expensive and readily available imported items that simply do not meet the requirements of ASTM F1554 grade 36.

ASTM A490

Hamilton: The ASTM A490 specification was just revised in the online version in February of this year. This -08a revision allows ASTM F1136 Grade 3 to be applied to A490 bolts. This is the first coating that has been allowed by the ASTM standard to be pre-applied to A490 bolts prior to installation, so this is a big change in the standard since its creation in 1964. The evaluation and subsequent determination of acceptability of the coating was the result of hydrogen embrittlement research that is currently being supported at McGill University by the RCSC, IFI, CIFI, AESF, Ifastgroupe, the U.S. Navy, Nucor Fastener, and the Boeing Company.



Nucor Fastener

An ASTM A325 bolt.

The concept of using any type of electroplating process on A490 structural bolts without having conducted a finish/coating qualification testing process, as outlined in IFI-144 and per the McGill University research project to qualify the ASTM F1136 coating, would be in violation of ASTM A490 (per Section 4.3, Protective Coatings). Consult your fastener manufacturer for further information or to put you in touch with a finish/coating expert if you have specific questions regarding the ASTM F1136 finish/coating.

Lindsay: Hot-dip galvanizing does not affect the strength of the ASTM A490 bolt.

†Editor's Note: The title of ASTM F1554 is *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength.* Even though the standard refers to "anchor bolts," AISC uses the term "anchor rods" when referring to materials used to anchor structural supports to concrete foundations—even if those materials have the same appearance as headed bolts. The reason for the distinction is to discourage the inadvertent (and incorrect) application of structural bolt installation and pre-tensioning requirements to foundation connections.

The problem with galvanizing or plating A490 bolts is the potential for hydrogen embrittlement. This scenario may occur when atomic hydrogen is absorbed by the steel during the acid pickling process that takes place prior to galvanizing or plating. This embrittlement can potentially lead to the loss or partial loss of ductility in the steel and consequently result in the premature failure of the fastener in the field.

Common practice in the industry is to bake bolts after plating to reduce the potential embrittlement, but ASTM does not recommend this procedure and simply states that A490 bolts should not be galvanized or electroplated.

A better option to reduce corrosion on an A490 bolt, besides painting them after installation, is to order an ASTM A490 Type 3 in lieu of a standard A490 (Type 1) bolt. The Type 3 version of this specification uses weathering steel, which is naturally corrosion-resistant. The result is a high-strength structural bolt with the same mechanical properties as a Type 1, but with corrosion-resistant properties.

Pfeifer: Following extensive research, conducted according to the IFI-144 test method, the DACROMET finish (the brand name for the finish applied per ASTM F1136) has been added to the A490 specification as an approved finish. At this time, DACROMET is the only finish permitted on A490 bolts. The DACROMET process is non-electrolytic and it is applied via dip-spin or spray application method.

ASTM F2280

Hamilton: ASTM F2280 is also a relatively new standard for twist-off tension-controlled bolt assemblies, having been first approved in 2006. The current revision of the standard is 2008. This standard is the corresponding ASTM A490-strength version of the twist-off tension control assembly, similar to ASTM F1852 (ASTM A325-strength version) that has been around for several years without an official ASTM standard. Dimensions are covered by the same ASME B18.2.6 standard that defines dimensional requirements for the A325, A490, and F1852 standards. This ensures the user of dimensional uniformity by standard and allows the user to choose their preferred installation method without having to worry if the body length and thread lengths will be different between the hex and tension-controlled products they purchase. MSC

Participants

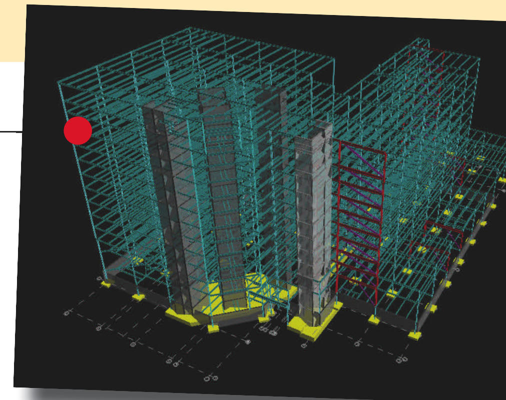
Roger Hamilton, Applications Specialist, Nucor Fastener Division

Greg Lindsay, General Manager, Portland Bolt & Manufacturing Company

Andrew R. Pfeifer, Assistant Manager, Product Management, Metal Coatings International, Inc.

Each month, MSC's product section features items from all areas of the steel construction industry.

In general, these products have been introduced within the past six months. If you're looking for a specific product, visit MSC's online product directory at **www.modernsteel.com/products**. You can browse by product category or search on any term to help find the products you need, fast.

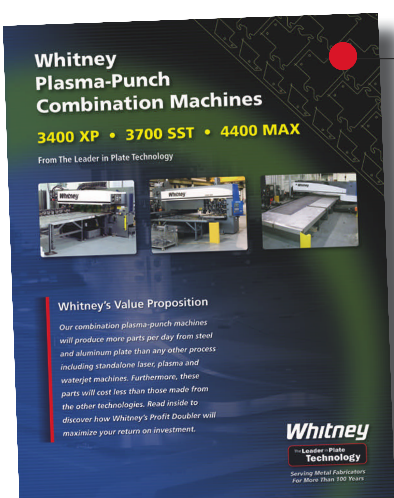


Integrated and Intuitive

Bentley's RAM International Solutions Center has released Version 11.3 of the RAM Structural System, Bentley's modeling, analysis, and design software for structural engineers involved in the design of building structures of all types. The software provides powerful automated and integrated tools that enhance structural engineers' productivity, allowing them to be more profitable and to more quickly produce economical designs. RAM Structural System V11.3 includes several new design and usability features, including the following:

- The latest steel design specifications—AISC 360-05 as published in the AISC *Steel Construction Manual*, 13th Edition—have been implemented.
- The requirements of the Direct Analysis Method of AISC 360-05 have been implemented.
- An RSS Feed on the main screen provides up-to-date announcements of program releases, downloads, seminars, and training events.
- The steel design tables for the U.S. have been updated to reflect the properties of the currently available steel sections as outlined in the AISC manual.

For more information, visit www.bentley.com or call 800.726.7789.



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For more information, visit www.gatewaysafety.com or call 800.822.5347.

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 Geoff Weisenberger (weisenberger@modernsteel.com). To be included in MSC's online products directory, contact Louis Gurthet (gurthet@modernsteel.com).

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This is a career opportunity where you will be recognized for your contributions, compensated for your skills, and appreciated for your hard work!! We need team members and leaders who know how to work hard and work smart yet still maintain an enjoyable and fun workplace (life is too short for anything less). You don't have to be ambitious but if you are, expect plenty of opportunities for advancement.

Additionally, we offer a full benefits package after 90 days.

If this sounds right for you, then come join us in Texas for an in-depth interview at our north Dallas office.

To set up your confidential interview please forward your resume via email to:
xyzaxisdetailing@gmail.com

AISC Seeks Senior Engineer

Are you the person that fellow engineers come to with questions about the AISC Specification or AISC Seismic Provisions? If you are looking for the next step in your career, have excellent people skills, and like new challenges, AISC is the place for you.

AISC seeks a Senior Engineer in the AISC Steel Solutions Center. This is a unique opportunity to apply your design experience in helping other steel designers better understand the nuances of AISC documents and steel design and construction. You will work with the top engineers, fabricators, educators, and leaders in the North American steel design community and construction industry. You will provide technical assistance to the entire structural engineering community and structural steel industry. As you serve as the connection between people with questions and people with answers, you will become nationally known for your own expertise in steel design and construction.

Applicants should have a BS degree in architectural or civil engineering with structural emphasis; MS or M. Eng. preferred. A minimum of 5 years of design experience is required, and more is better; also, having experience in construction is a plus. AISC provides a great working environment offering professional development opportunities, flexibility, excellent resources, and a competitive salary with excellent benefits.

Send resumes to Janet Cummins at **cummins@aisc.org**.

General Manager, Steel Fabrication & Coatings – New England

The General Manager will have full P&L responsibility of two steel fabrication facilities with revenue totaling $50-60M. Will be responsible for efficient operational control and organizational development, policy and procedure deployment, financial cost management, quality management, marketing and sales management, effective planning of goals, objectives and tactics of the organization.

Specific duties include but are not limited to:

- Work with the President to develop short and long-term goals and objectives for the corporation.
- Gain understanding of the market dynamics. Develop sales and marketing strategies to allow growth in key areas.
- Develop a full understanding of all fabrication and operational processes in both facilities. Be able to assess current production and business processes. Develop and implement methods to improve efficiencies and returns.
- Continue development of our professional management team focusing on refining corporate culture and achieving predetermined goals.
- Development and management of annual business plan and other plans as necessary, that allow for sustained growth of the company, as well as above average return on investments.
- Ensure the accuracy and timeliness of all financial reporting.
- Accurately analyze a variety of business and related opportunities and recommendations to the President.
- Participate in professional and community associations.

Requirements: Must have 10+ years of experience in an executive level role within the steel or metal fabrication industry.

Location: Position is base at the New England facility with responsibilities also for a facility in the mid-Atlantic.

Please Contact:
Brian Shuppe
412.355.8200
bshuppe@specialtyconsultants.com

SALES MANAGER

The ideal sales manager will have experience in the steel fabrication/detailing industry, with several years working with in-house or outside detailers; reading contract drawings; preparing detailing estimates; and successfully selling. He/She must have the capability to travel extensively to meet with senior management at the customer's offices and conduct meetings in a professional manner.

This position offers excellent bonuses for the candidate who can meet or exceed targets. Engineering or college degree is a definite plus. Location is open.

CHECKER

The checker will have several years experience checking structural steel and miscellaneous drawings. The checker will be responsible for reviewing the contract drawings to ensure that detail drawings conform to the intent of contract drawings. He/She will highlight discrepancies or errors immediately to the detailer and ensure that drawings are corrected in a timely manner. Some coordination with customers is required. Must be familiar with 3D detailing software like Tekla Structures or SDS/2. Location is central New Jersey. This position involves 30% travel.

Creative Engineering Services is based in central New Jersey and serves structural and industrial fabricators around the country. We offer excellent salary, benefits, relocation and paid vacation.

Send resumes to: **jobs-usa@crenser.com**

Arch. Misc. Steel Detailing Firm Needed

Due to rapid growth of our detailing firm we have an immediate need to hire a firm to handle most of our detailing of architectural misc. steel and site work. SDS2 capabilities are preferred however SSDCP would be acceptable. We work with Design/Build fabricators on most projects so all schedules are accelerated.

If interested and qualified, please contact George Dilks at **gdilks@dtls.com.**

Structural & Misc. Steel Fabrication

Our organization has been recruiting for the Structural and Misc. Steel Fabricating industry for over 20 years. Current positions include:

- **Project Manager**
- **General Manager**
- **Quality Control**
- **Plant Superintendents**
- **Estimators**
- **Detailers**
- **Chief Draftsman**
- **Checkers**

Please send resume to:
Richard Stauffer
United Employment Associates, P.O. Box 8, East Texas, PA 18046
phone: (610) 437-5040 fax: (610) 437-9650
e-mail: rstauffer@unitedemployment.com www.unitedemployment.com

Steel Detailer/Checker

Bender's Technical is looking for experienced detailers and checkers. Located in Western New York along the Niagara River, we have been serving the steel fabrication industry for 11 years. We are seeking self-motivated professional steel people with a minimum of 3 years experience in low to mid rise commercial/industrial and structural or miscellaneous fields. Candidates must be experienced in Tekla or Stru-CAD and have knowledge of AISC and OSHA Standards. We offer a competitive salary based on experience, medical insurance, vacation, Simple IRA plan and performance bonuses. Relocation to the water front of Western New York required. Qualified candidates please submit resumes via email or fax to:

Bender's Technical Detailing
Attn: Jim Pieper
Fax: 716-695-0239
Email: **jpieper@benderstech.com**

BENDER'S
TECHNICAL DETAILING

AISC Engineering & Research Dept.

Staff Engineer

If you are looking for an entry-level position, have good people skills, and like new challenges, AISC has an opportunity available. AISC is now seeking applicants for a Staff Engineer position in the AISC Engineering & Research Dept. This is a unique opportunity to work with issues in structural research as well as steel fabrication. You will work with the top educators in the North American steel design and construction industry and will be in a position to establish yourself as an expert in steel design. The position will allow you to use your technical training to develop and promote research and industry issues that benefit the entire structural engineering community and structural steel industry.

Applicants should have a BS degree, with a structural emphasis, in architectural or civil engineering, and must be qualified to work in the U.S. AISC provides a great working environment offering professional development opportunities and a competitive salary and benefits package.

Send resumes to C. Becker at **becker@aisc.org**.

THE CARE AND FEEDING OF DETAILERS

Overlooked, misunderstood, and often maligned, structural steel detailers nevertheless play a critical role in bringing projects to life.

BY JACK METCALFE

WHEN ASKED BY MSC TO CONTRIBUTE A COLUMN, I, like so many novice writers, began to panic and search for a subject. But then it struck me: I'm a detailer! So allow me to introduce you to "What you've always wanted to know about detailers—in one page or less."

Usually far from the excitement of steel members being hoisted by a crane, or far from the thunder and sparks that come from the fabrication shop, detailing is accomplished in relative obscurity. The detailer is more often than not in a different city than his client, and perhaps even a different country. Unless there is a schedule crisis, an RFI, or a revision, many in the construction field don't even know that detailers exist—or if they do, they think that it is actually a computer that does all the drafting and the detailers just push the buttons. Many old-time contractors and some inexperienced project managers believe that detailer drawing rooms are a part of the fabricator's in-house organization. Project owners and GCs believe the detailer is controlled by the fabricator and is subject to his dictates. They are often astonished to discover that we detailers are now, more often than not, an independent business organization with our own clients, schedules, overhead, employees, and sales to worry about. Times have changed, and the detailer must now wear many hats, not just that of a "produce or perish" draftsperson serving one master. But whether an independent entity or part of a fab shop, the detailer must still be a part of the construction team.

Jack Metcalfe is president of John Metcalfe Company, a 57-year-old detailing firm specializing in bridge and bridge rehabilitation projects. He also serves as an active director of NISD and SEAA. He can be reached at metcalfe51@aol.com.

There is an old adage about how the more things change, the more they remain the same. As a detailer who has been privileged to serve the fabricated steel construction industry for more than just a few years, it is tempting to believe that. Sure, the callus on my finger has diminished and has been replaced with carpal tunnel, but the holes in mating material still have to match. Erection diagrams are now frequently called "member placement plans," but the detailer still has to show the erector where, and sometimes how, the member is to be properly located in the structure. The design drawings may now be issued in version "eleventy-seven" of someone's software, but it is still the detailer who has to interpret the intent of the design into a set of zero-defect shop and field drawings. Changes in design now often come guised as "approval comments" instead of revisions, but making the necessary changes to mill orders or shop and erection drawings still costs time—and time is still money no matter how fast electronic changes are supposed to be. The electronic RFI and clarification sketch may have replaced the fax, which replaced snail mail, which replaced the phone. But all of us in the construction business still struggle with costs and delays caused by incomplete or erroneous design information.

Back in the day, any detailing errors the checker found would have to be "scrubbed" by the detailer from his/her drawing, using a rubber eraser, and the corrections made manually with a pencil—an arduous task that made drafters strive for perfect drawings and consequently made for better detailers. Now, in most drawing rooms, it's done with the CAD system, but the work still requires a checker, and the drafter still has to "back check" and agree with any checker's corrections before fixing the drawing.

And I'm sure that many high-tech gurus would be surprised to learn that there are still a number of highly skilled manual drawing rooms in existence, turning out some very sophisticated work. (I can remember when a mouse was something that snuck into your drafting table drawer and made a nest in the pencil shavings.)

At one time, the only names we had to be familiar with were Smolley, Bruhn, Inskip, Webster, our squad boss, and the paymaster. Now, we have to know HP, Dell, Jobber, Blodgett, Gates, and Google. But regardless of who you know, what you know is still the trademark of a quality detailer. How many other craftspeople have to be knowledgeable in estimating, welding, scheduling, mathematics, scheduling, connections development, spatial concepts, camber, scheduling, mill practices, scheduling, graphics, tension, compression, scheduling, overhead, erection techniques, sales, computers, scheduling, estimating, paint systems, partnering, design-build, and OSHA—not to mention scheduling?

A few issues ago, MSC ran an enlightening article on visiting a fabricator ("A Complete Fabrication", 3/08, p. 68). I would suggest that you visit a detailer—or talk to a group like AISC, NISD, or SEAA, all of whom will be happy to share with you their knowledge of this little-known but highly important member of the steel construction team. MSC

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