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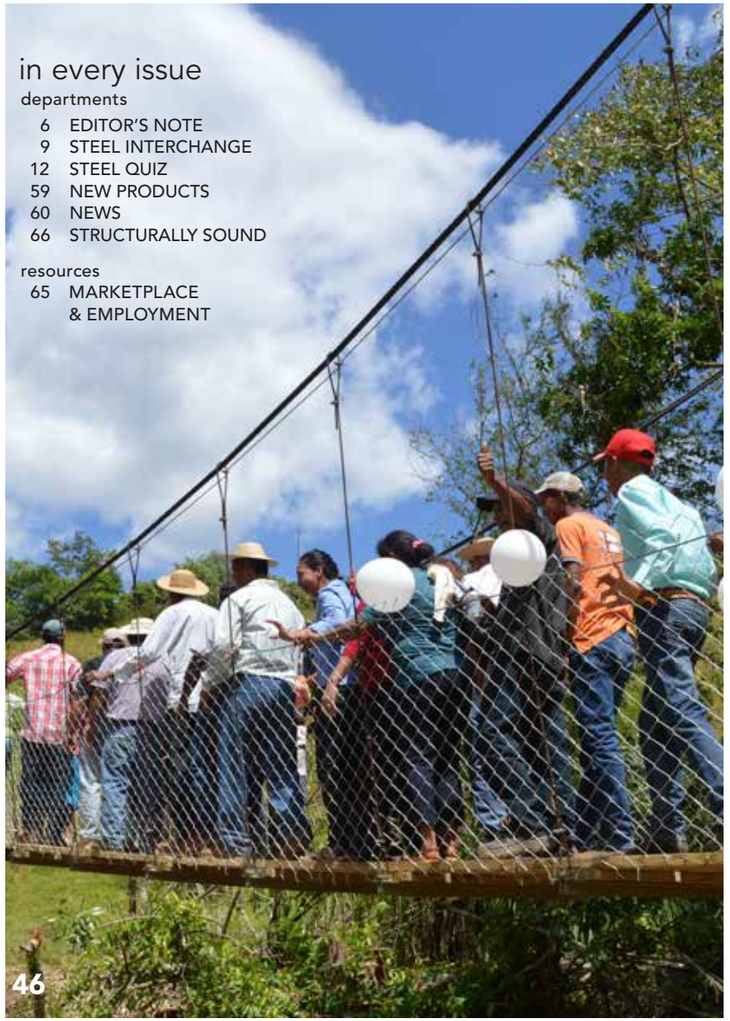

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ON THE COVER:
Tight quarters in the form of a narrow canyon and sharply curving road didn't stop this Idaho river span from coming together quickly, p. 26. (Photo: Ken Saindon)

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editor's note

My family doesn't travel with me that often, but sometimes the destination is so interesting that we make an exception.

Such was the case around 15 years ago when I was invited to attend a Canadian Institute of Steel Construction event in Jasper (up in the Canadian province of Alberta).

Bob Owen, an AISC board member and a gentleman in his own right, was also attending. Bob, the then president of Paxton and Vierling Steel, is an avid pilot and asked if I wanted a "ride." I explained that my whole family was going, but he simply replied that he loved kids and they were all welcome!

Also flying with us was Terry Peshia and his lovely wife, Connie. As we sat down and settled in, Terry pulled out a Chinese checkers set and invited my daughter, Julia, who was around 7 at the time, to play. Terry and Julia had a merry time on the flight, whether playing games or having Terry explain the nuances of aircraft navigation.

When we returned to Chicago, Terry handed the game to my daughter. I thanked him but said it really wasn't necessary. Terry responded that he had only brought the game to play with Julia on the flight.

That was the first time I learned what type of heart Terry had.

Terry, who passed away last month, was always a go-to expert for me. When I had a question about the fabrication business, he was the first person I turned to. And my questions were always very broad; I can remember calling to discuss topics ranging from the type of gasses he purchased to insurance issues to erection procedures. He was a calm voice on the board who always took a thoughtful and long-range position and was willing to reconsider when there was enough evidence to the contrary.

I have so many great memories of Terry, his wife and his two sons, who are both intimately involved with the fabrication business (if you have ever attended a Future Leaders Ideas Lab, you were sure to meet Ted and John).

Early on, he was a notable holdout who opposed the AISC Fabricator Certification Program. But over time, he grew to realize the value of the program and became one of its staunchest advocates and a leader in the development of the AISC Erector Certification Program.

His fabrication shop, Garbe Iron Works in Aurora, Ill., was not the largest nor the most modern facility. But they worked on a lot of interesting projects, including building a floating casino. Terry always advocated for the future, and his shop tours were incredible. Rather than just routing groups through the space, he actually posted signs explaining what was happening at each station.

He also was a huge supporter of students and donated multiple teaching steel sculptures to nearby universities (see www.aisc.org/steelsculpture). In fact, visitors often noticed a steel sculpture in front of his shop—which he maintained so he always had a sculpture ready to donate.

Terry represented the best of the steel industry, and I'll miss him dearly.

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steel interchange

*Unless specifically stated, all AISC publications mentioned in the questions and/or answers reference the current edition and can be found at **www.aisc.org/specifications**.*

Weld Inspection Acceptance Criteria

AWS D1.1 provides acceptance criteria for both statically and cyclically loaded connections. Since the criteria for cyclically loaded connections are more stringent, should they always be used unless stated otherwise? Is the inspector responsible for deciding between statically and cyclically loaded acceptance criteria?

The acceptance criteria for cyclically loaded connections should not be assumed to be the default requirement. AWS D1.1 indicates that cyclic requirements apply when the joints are "subjected to cyclic loads of sufficient magnitude and frequency to cause the potential for fatigue failure." Section 3.1 of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) states: "The fatigue resistance of members consisting of shapes or plate shall be determined when the number of cycles of application of live load exceeds 20,000. No evaluation of fatigue resistance of members consisting of HSS in building-type structures subject to code-mandated wind loads is required." This means that generally welded connections in structures within the scope of the *Specification* will be subject to acceptance criteria for statically loaded connections.

The engineer of record (the owner's designated representatives for design) is responsible for defining the acceptance criteria. If the requirements are not clear, clarification should be requested. Clause 6.7 of AWS D1.1 states: "The extent of examination and the acceptance criteria shall be specified in the contract documents on information furnished to the bidder." Section 8.5.6 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303) states: "The inspector shall not suggest, direct or approve the fabricator or erector to deviate from the contract documents or the approved approval documents, or approve such deviation, without the written approval of the owner's designated representatives for design and construction." Both statements indicate that the requirements must be provided in the contract documents.

Larry S. Muir, PE

Connection to Supports

Section 1.4.4 of ASCE-7 states: "A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss… The connection shall have the strength to resist a force of 5% of the unfactored dead load plus live load reaction imposed by the supported member on the supporting member." I am being asked to check every connection on a current project for this axial end reaction. Is this common?

No. Note that there is a requirement that the connections must be able to resist this force. There is no requirement to provide an explicit check. Engineers commonly judge some conditions as okay by inspection based on their own engineering judgment.

The required strength (in the horizontal direction) is only 5% of the unfactored vertical loads. This is quite small. Relative to many connection-related limit states, the load described by Section 1.4.4 would be 2.5% of the vertical design load. In practice, most engineers simply conclude that typical steel connections can resist this load. I think it would be difficult to find a reasonable connection that does not satisfy this requirement.

Since these are ASCE, not AISC, requirements, you may also want to contact ASCE relative to their intent.

Larry S. Muir, PE

Galvanized Architecturally Exposed Structural Steel (AESS)

The November 2017 *Modern Steel Construction* article "Maximum Exposure" addresses changes that occurred in Section 10 of the 2016 AISC *Code* and provides other useful advice. A caption to one of the photos states: "AESS can also be galvanized. Design teams should be aware that galvanizing steel does not provide a 'chrome' finish, and no two pieces of galvanized steel will look exactly the same." An editor's note in Section 2.9 of the Sample Specification further cautions about expectations for AESS finish when hot-dip galvanizing is specified, and also explains the possible causes of such finish irregularity.

If the level of dullness/shininess is of concern, is sample/mock-up the only way to establish the acceptable level of dull or bright finish, in lieu of any other descriptive verbiage in the project specification?

A mock-up may be a means of establishing acceptable and expected finish for the galvanizing. Section 10.1.2 of the *Code* requires a mock-up for AESS categories 3, 4 and C. If a mock-up is to be used in other AESS categories, it must be specified in the contract documents.

Regardless of whether a mock-up is used, you should work with the galvanizer and fabricator to come up with specification language that will result in an end product that meets your expectations. The chemistry of the steel influences the appearance of the galvanized coating. It may be necessary to impose tighter controls on chemistry, which could impact the cost and schedule of the project. Also keep in mind that the mock-up will reflect only the appearance of the coating at a particular time. As indicated in the article (www.modernsteel.com) the appear-

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The complete collection of Steel Interchange questions and answers is available online at **www.modernsteel.com**.

ance may change to a more uniform matte gray finish over time even if the mock-up appears bright or blotchy.

You might be able to refer to existing structures to get a better feel for the final appearance. The galvanizer may also be able to provide guidance relative to steps that were taken in an individual project to achieve a certain appearance.

You might want to reach out to the American Galvanizers Association (www.galvanizeit.org). Its site provides a good bit of information about the appearance of galvanized steel. This information is useful but should not be viewed as a substitute for direct interaction with those performing the work.

Carlo Lini, PE

Pretension in Snug-Tight Connections

I have a few questions about pretension in snug-tight connections:

1. **Do the RCSC or AISC specifications recommend that snug-tight connections not be pretensioned?**
2. **Has this recommendation changed over time?**
3. **What is the level of pretension that is expected in snug-tight connections?**

Your questions are addressed below:

1. No. There is no such recommendation. Neither the RCSC *Specification* nor the AISC *Specification* place an upper limit on installed pretension for snug-tight joints. Snug-tightened joints are defined as having all plies in firm contact and bolts tightened with a few impacts of an impact wrench or the full effort of an ironworker. To reach this condition, the bolts could be installed to the pretensions indicated in Table J3.1 of the AISC *Specification*—or even higher—in order to satisfy the requirements for a snug-tight joint.

2. No. To my knowledge, there has never been a recommended upper limit on the installed pretension for snug-tight joints.

3. As stated above, there is no upper limit on the installed pretension. AISC Design Guide 16: *Flush and Extended Multiple-Row Moment End-Plate Connections* (a free download for members at www.aisc.org/dg) does provide some guidance and states:

"The study by Kline, et al. (1989) observed that the pretension force measured in the snug-tightened bolts is directly proportional to the bolt diameter (d_b). Based on this data, a recommendation for the assumed pretension force in snug-tightened bolts to be used in the design procedure is:

$d_b \le 5/8$ in., use 75% of specified AISC full pretension
$d_b = 3/4$ in., use 50% of specified AISC full pretension
$d_b = 7/8$ in., use 37.5% of specified AISC full pretension
$d_b \ge 1$ in., use 25% of specified AISC full pretension"

This is just a guide to what pretension can be expected. It is not something to be measured when bolts are installed.

Carlo Lini, PE

Small Section Sizes

The smallest angle found in the AISC *Steel Construction Manual* is L2×2. Why doesn't the *Manual* include sizes such as L1½×1½ or L1¼×1¼?

The AISC Committee on Manuals determines what information to include in the *Manual* based on a number of factors, including the relevance to steel building structures. L1½×1½ and L1¼×1¼ are likely too small to be considered for use in building structures. If you need properties or strengths for these shapes, you may conservatively calculate the properties by hand and determine the strength manually using the provisions of the *Specification*.

Jonathan Tavarez

steel quiz

This month's Steel Quiz is based on fatigue requirements in Appendix 3 of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360, **www.aisc.org/specifications**). The questions and answers were contributed by Hamza Sekkak, a PhD student at the Illinois Institute of Technology. Thank you, Hamza!

1 Per the AISC *Specification*, fatigue resistance of members consisting of shapes or plate shall be determined when the number of live load application cycles exceeds:
a. 5,000 **b.** 20,000
c. 100,000 **d.** 500,000

2 **True or False:** When the applied cyclic stress range is less than the threshold allowable stress range, F_{TH}, no further evaluation of fatigue resistance is required.

3 Does fatigue typically need to be considered for seismic or wind loading?

4 **True or False:** Stress ranges that are completely in compression need to be investigated for fatigue.

5 From Appendix 3 in the *Specification*, stresses are calculated on the basis of …… . The maximum permitted stress is …….. due to peak cyclic loads.
a. live loads / F_y
b. amplified loads / $0.66F_y$
c. live loads / $0.66F_y$
d. amplified loads / F_y

6 Which of the two below details (with weld reinforcement removed) would be assigned fatigue Category B, and which would be assigned fatigue Category E?

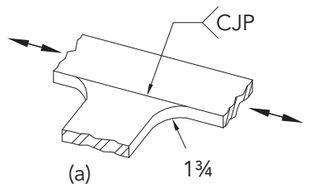

Figure 1a

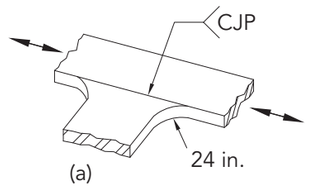

Figure 1b

TURN TO PAGE 14 FOR THE ANSWERS

steel quiz ANSWERS

1 **b.** The appendix on fatigue in the *Specification* (Appendix 3) deals with high cycle fatigue (i.e., >20,000 cycles).

2 **True.** At low levels of cyclic tensile stress, a point is reached where the stress range is so low that fatigue cracking will not initiate regardless of the number of loading cycles. This level of stress is defined as the fatigue threshold, F_{TH}.

3 **No.** Section B3.11 in the *Specification* states: "Fatigue need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components."

4 **False.** Fluctuations in stress that do not involve tensile stresses do not cause crack propagation and are not considered to be a fatigue condition. For a member subjected to compression only, cracks may initiate only in regions of high tensile residual stress and do not propagate because residuals stresses are relieved by the crack. Thus, stress ranges that are completely in compression do not need to be investigated for fatigue.

5 **c.** The provisions of this Appendix apply to stresses calculated on the basis of live loads. Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities. The maximum permitted stress due to live loads is $0.66F_y$.

6 The detail shown in Figure 1a would be assigned to Category E.

The detail shown in Figure 1b would be assigned to Category B.

From Table A-3.1 in Appendix 3, the ranges of radiuses and their corresponding categories are as follows:

Description	Stress Category
$R \geq 24$ in. (600 mm)	B
6 in. $\leq R <$ 24 in. (150 mm $\leq R <$ 600 mm)	C
2 in. $\leq R <$ 6 in. (50 mm $\leq R <$ 500 mm)	D
$R <$ 2 in. (50 mm)	E

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steelwise

UNLISTED MATERIALS – PART 1

BY LARRY S. MUIR, PE, AND THOMAS J. SCHLAFLY

A look at materials and products that are not included in the AISC *Specification's* list of approved materials.

THE AISC *SPECIFICATION* approves the use of several materials—but what about materials that it doesn't include?

Generally, the *Specification for Structural Steel Buildings* (ANSI/AISC 360, available as a free download at www.aisc.org/specifications) does not prohibit the use of any material. The use of unlisted materials and products—those that are not included in the list of approved materials—is left to the discretion of the engineer and can be viewed as a substitution of an unlisted material for an approved material.

An article about material substitutions (called "Material Substitutions") appeared in the August 2011 issue (www.modernsteel.com) and much of the information provided then is still applicable today. However, material substitutions seem to be more common today than they were in 2011, and familiarity may breed complacency if not contempt. Some engineers may erroneously believe that all steels and steel products are created equal and that material substitutions can be made with little thought. With this in mind, a fresh look is in order.

In this first of a series of three articles on the topic, we'll discuss the reasons for the treatment of materials that have been adopted by AISC *Specification*. We'll also include a discussion about the evaluation of unlisted materials based on a list of factors provided in the Commentary to Section A3 of the *Specification*. Let's start by addressing some basic questions.

Why does the AISC *Specification* include a list of approved materials?

The *Specification* has existed under various titles and in somewhat different forms for nearly a hundred years. Thousands of engineers around the world turn to the *Specification* on a daily basis to aid them with their designs. The *Specification* is also commonly referenced by other standards and specifications, sometimes relative to applications well outside the intended scope of the AISC Committee on Specifications.

Given the ubiquity and prominence of the *Specification*, engineers sometimes incorrectly believe that it can be used to design all steel structures using any material that can conceivably be classified as steel. This is not the case.

As stated in its scope, "The *Specification* shall apply to the design, fabrication and erection of the structural steel system or systems with structural steel acting compositely with reinforced concrete, where the steel elements are defined in Section 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303)… This *Specification* sets forth criteria for the design, fabrication and erection of structural steel buildings and other structures, where other structures are defined as structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting elements."

It is interesting to note that the *Specification* does not even include a definition of steel as a material. Instead, the range of materials that can be used is defined by two factors: application and references to ASTM specifications.

The scope of the *Specification* is limited to building design and is further limited by the definition of structural steel provided in the *Code*, which also defines steel elements not based on the physical properties of the material from which they are made, but rather based on their intended use.

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Limits are set based on material properties through references to ASTM specifications. Section A3 of the *Specification* lists ASTM specifications that are approved for use under the *Specification*. As indicated in the Commentary, the materials listed "are commonly useful to structural engineers" and "have a history of satisfactory performance."

The scope of the *Specification* is limited to certain applications and certain materials because these are the applications and materials that were considered when the various provisions of the publication were written or evaluated during updates. The standards in Section A3 are those representing materials commonly used for typical applications in building-type structures. Most of them are available in the supply chain from producers and service centers that routinely participate in the U.S. structural steel industry. The plethora of specifications for materials suitable for other structure types, unusual applications or from other countries is immense. It is beyond the capability of AISC committees to maintain such a comprehensive list.

Why are other materials not prohibited?

The simple answer is provided in the Commentary: "Other materials may be suitable for specific applications." There is no reason in building design to limit materials when other materials may be suitable.

It also has to be recognized that the structural steel industry, despite its age, is still a vibrant and evolving industry. The Commentary to the 1963 *Specification* states: "The increasing use of high-strength steels no longer permits the continuation of a standard design specification based upon the exclusive use of one strength grade of steel." The 1963 version required that structural steel conform to one of six listed ASTM specifications. The 1969 edition doubled the number of listed steels, and for the first time stated that the listed materials were "approved for use under this *Specification*."

Sometimes materials are developed to serve specific purposes related to building design that might provide a significant benefit, but that have not yet been adopted by the *Specification*—because of the length of the code cycle or for other reasons. There is no good reason for the *Specification* to stand in the way of such innovation. History indicates that innovation generally starts with industry and engineering, and the *Specification* simply follows suit.

How should unlisted materials be evaluated?

The evaluation of unlisted materials is the responsibility of the engineer specifying or approving them. The Commentary provides a list of some (but certainly not all) of the considerations. These include:

- Typical strength properties – F_y and F_u
- Strength properties in transverse directions
- Ductility
- Formability
- Soundness
- Weldability, including sensitivity to thermal cycles
- Notch toughness
- Other forms of crack sensitivity
- Coatings
- Corrosivity
- Effects of production
- Tolerances
- Testing
- Reporting
- Surface profiles

Some of these items are discussed further in the below sections. This should not be viewed as a complete list of factors that must be considered, but it will provide a start.

Typical strength properties

When considering a proposed material substitution, perhaps the two most requested properties of the substituting material are the yield strength and the tensile strength, but one should remember that outwardly similar materials can behave very differently in ways that are not always apparent from the numerical values of F_y and F_u.

The way in which these values are determined can also determine whether apples are being compared to apples or to oranges. This is discussed further relative to testing.

Strength properties in transverse directions

Engineers often assume steel is homogeneous and isotropic, and this assumption is adequate for most designs using approved materials that are addressed in the *Specification*. However, steel is a manufactured material, and steel shapes and plates are manufactured products. The manufacturing process introduces and modifies inclusions in the steel. The manufacturing process for long products includes hot rolling, which elongates the grains in one direction and can also elongate inclusions. This results in different properties in the direction of rolling as compared to properties in the transverse direction or through thickness directions (anisotropy).

Lamellar tearing is one consideration related to this anisotropy. Several factors play a part in lamellar tearing, including joint configuration and steel chemistry. The manufacturing process itself also plays a role. The current steelmaking practice of continuous casting places demands

on the producer that have the benefit of controlling the shape of inclusions and improving through-thickness strength. Therefore, current continuous cast products have less likelihood of lamellar tearing than did older ingot cast products. Engineers can reduce the likelihood of lamellar tearing through good design practice, as described in Design Guide 21: *Welded Connections—A Primer for Engineers* (www.aisc.org/dg). Material specifications define some aspects of chemical composition, such as limits on sulfur, which can be objectively evaluated assuming the proper expertise, but the details of the manufacturing process may be more difficult to know and to evaluate.

Ductility

Ductility is the ability of a material to deform plastically before fracturing. In many respects, it is the magic ingredient that makes structural design practical. Ductility can be measured or quantified in various ways. The *Specification* defines rotation capacity as "incremental angular rotation defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield prior to significant load shedding." The *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341, available as a free download at www.aisc.org/specifications) quantifies ductility demand based on story drift or building drift. Such measures of ductility can include factors other than material properties. Chapter K of the *Specification* uses the ratio of the specified minimum yield strength to the specified minimum tensile strength (yield/tensile ratio) as a measure of ductility. Ductility as it relates solely to material is usually defined in terms of elongation or reduction of area in tension tests, and minimum values are specified in ASTM standards.

Appendix 1 in the *Specification* directly addresses ductility when engineers implement design by inelastic analysis. A level of ductility consistent with the approved materials is assumed relative to the rest of the *Specification* as well. Ductility is required in all structures designed using the *Specification* because common design methods use approximation to calculate available and required strengths of elements that may not be justified if those elements are not sufficiently ductile to redistribute uneven stresses.

Formability

Formability is the ability of the steel to be plastically deformed without fracture. Formability is often not considered in design under the *Specification*, but it plays in role in such operations as cambering and cold bending of plates. Some standard shop practices may no longer be appropriate if the formability of the material specified differs from that of the materials approved for use in the *Specification*.

Increased inspection of cambered beams and bent plates may be appropriate until the formability limits of the specified material have been established.

Weldability

Not all steel can be effectively welded. The heat associated with welding is detrimental to the properties of some steels, and some steel is inherently prone to weld-induced cracking. Weldability is a significant consideration when evaluating unlisted materials.

AWS D1.1, which is adopted by reference in the *Specification*, categorizes materials by groups, which are then used to establish things like base and filler metal combinations for prequalified welds. The range of materials addressed in AWS D1.1 is much larger than the approved materials in the *Specification*, but not all materials are addressed. Welding procedure specifications (WPS) for materials not listed in AWS D1.1 must be qualified. This process can be very time-consuming and expensive.

Chemical composition also has a significant effect on the weldability of steel. Design Guide 21 contains a good discussion of weld cracking, including the effect of steel chemistry.

Notch toughness

Toughness is the ability of a material to deform and absorb energy before fracturing. Though it is not always obvious many of the standard U.S. fabrication practices rely on a certain level of toughness. Experience has shown that the approved materials in the *Specification* have sufficient toughness for use in buildings using typical U.S. fabrication and design practices. Where toughness is more of a concern, such as for heavy shapes in certain applications, the *Specification* imposes explicit toughness requirements. The ability to leave backing in place, the lack of a defined

Listed Consumables for Welding

- AWS A5.1
- AWS A5.5
- AWS A5.17
- AWS A5.18
- AWS A5.20
- AWS A5.23
- AWS A5.25
- AWS A5.26
- AWS A5.28
- AWS A5.29
- AWS A5.32
- AWS A5.36

Listed Fasters

Bolts:

- ASTM A307
- ASTM A354
- ASTM A449
- ASTM F3043
- ASTM F3111
- ASTM F3125

Nuts:

- ASTM A194
- ASTM A563
- ASTM A563M

Washers:

- ASTM F436
- ASTM F844

Compressible-Washer-Type DTIs:

- ASTM F959

Anchor Rods and Threaded Rods:

- ASTM A36
- ASTM A193
- ASTM A354
- ASTM A449
- ASTM A572
- ASTM A588
- ASTM F1554

Listed Plates and Shapes

Hot-rolled structural shapes:

- ASTM A36
- ASTM A572
- ASTM A709
- ASTM A992
- ASTM A529
- ASTM A588
- ASTM A913
- ASTM A1043

Hollow structural sections:

- ASTM A53 Grade B
- ASTM A847
- ASTM A1065
- ASTM A1085
- ASTM A500
- ASTM A501
- ASTM A618

Plates:

- ASTM A36
- ASTM A242
- ASTM A283
- ASTM A514
- ASTM A529
- ASTM A572
- ASTM A588
- ASTM A709
- ASTM A1043
- ASTM A1066

Bars:

- ASTM A36
- ASTM A529
- A572
- ASTM A709

Sheets:

- ASTM A606
- ASTM A1011 SS, HSLAS and HSLAS-F

radius for reentrant corners and the range of methods permitted to form bolt holes are all tied to toughness.

The Commentary to Section A3.1a states: "For especially demanding service conditions such as structures exposed to low temperatures, particularly those with impact loading, the specification of steels with superior notch toughness may be warranted. However, for most buildings, the steel is relatively warm, strain rates are essentially static and the stress intensity and number of cycles of full design stress are low. Accordingly, the probability of fracture in most building structures is low. Good workmanship and good design details incorporating joint geometry that avoids severe stress concentrations are generally the most effective means of providing fracture-resistant construction." This guidance is only applicable to steel similar to those approved for use with the *Specification*.

Special detailing, explicit consideration of fatigue and/or increased inspections in building applications might be necessary if steels with low toughness are to be used. ■

The discussion on evaluating unlisted materials will continue in Part 2 of this three-part series, which will appear in next month's issue.

CHOOSE STRATEGY

BY ANDY SLIPHER

Answer the magic question of "How?" with a solid strategy.

Andy Slipher is founder of Slipher Marketing, a consultancy where strategy comes first, followed by tangible marketing results. He is an accomplished strategist, interim CMO, speaker and writer on marketing strategy and the author of *The Big How: Where Strategy Meets Success*. For more information on Andy, please visit **www.thebighow.com**.

HOW DO WE GET IT DONE? What's our next move? Now that we know what we want and why we're here, where do we begin?

You've likely heard variations of these questions in your organization—particularly if you're at any level of planning how to achieve favorable outcomes.

It's one thing to know why you're doing something, who you're serving or even what makes your product or service better than the next company. But until you can adequately and effectively answer the *how* question, your idea, product, sales or whatever you endeavor to achieve may not become all you hope for.

The biggest how you can ask begs for a coherent approach. It means building a distinct advantage toward a favorable end. This level of how is best answered with strategy.

Strategy exists to solve problems. More often than not, calling upon strategic planning means that your problem is significant and complex and comes with higher-than-average stakes. That's why we call upon strategy. It is the means to simplify and unify activity to get you from Point A to Point B with greater clarity, effectiveness confidence and efficiency.

Planning without strategy is like feeling around in the dark. You may eventually find what you're looking for, but it will most certainly be unpredictable and take longer than anticipated, plus you run a greater risk of falling on your face along the way.

Here are three things you need to know about strategy in order to adequately answer any big *how* and to improve your planning process, no matter what the challenge.

Planning without strategy is like feeling around in the dark.

Strategy is about choice. Strategy is a word and concept that is abused today. People love to use it because it sounds, well, strategic. Unfortunately, calling something a strategy doesn't make it one. Strategy, in order to function as it's intended, means making significant choices throughout the planning process. In any complex or challenging situation, such choices are hard. Something must be sacrificed in order to move in a true and distinct direction. If you're not making hard choices in your planning, you need to ask yourself and others how distinct, clear and achievable your approach is.

Consider this example: When Steve Jobs returned to a struggling Apple in 1997, one of the first things he chose to do was to stop selling so many products. He put an end to more than 70% of Apple's products (laying off more than 3,000 employees in the process) in order to focus on a handful of truly innovative offerings. This hard choice allowed Apple to focus its resources on innovation—developing something truly game-changing. The result? The iPod. There's little doubt that Jobs' efforts would have been significantly more difficult and unclear if he had not made this critical strategic choice.

Strategy fits between your goals and plans. Strategy is not the most important thing. But good strategy is necessary and often critical in order to be successful. Once you've defined your goals, strategy comes next. Let's delineate between goals, strategy and plans:

- Goals answer "What is the end for the effort?"
- Plans, which follow strategy, answer "What are the blueprints for success?"
- Strategy is the point in between that answers "How will we coordinate our efforts to get there?"

Strategy marries strength with opportunity. The beauty of strategy is that it coordinates and integrates activities around a common goal. What's more, good strategy finds the sweet spot where strengths meet opportunity. If you identify an opportunity yet have no strengths to take advantage, how effective will you be? Likewise, if your strengths abound in a certain area yet no opportunities exist, your strategy could come up short.

Know that in order to improve the odds of achieving your goals, your strategy will need to amplify your strengths while playing to the opportunities at hand. A great example of this can be seen in the way Procter & Gamble (P&G) has nearly cornered the consumer package goods market. With its humble beginnings in soap and candles in the 1800s, P&G slowly and methodically built a strength producing, packaging, marketing and selling packaged dry goods of all types. Over the years, the company has taken advantage of opportunities to both develop new products and acquire its way into new product categories. Today, the company's holdings cover close to 80 products spanning many of the typical product we buy every day. P&G has employed different business strategies over the years but has always weighed opportunity in light of its inherent strengths.

Whatever your challenge, follow these three fundamental principles for better strategic planning. Your strategy will be both more clear and coherent. What's more, you will be incrementally farther down the road toward more successful outcomes in a more timely fashion. ■

Narrow Margin

BY KEN SAINDON, SE, PE, AND ALEX WHITNEY, PE

It's a tight—but successful—squeeze for a replacement steel span in a remote Idaho canyon.

Ken Saindon

The new bridge spans 300 ft.

Roadway curvature at both ends was required to allow a logging truck to cross the bridge.

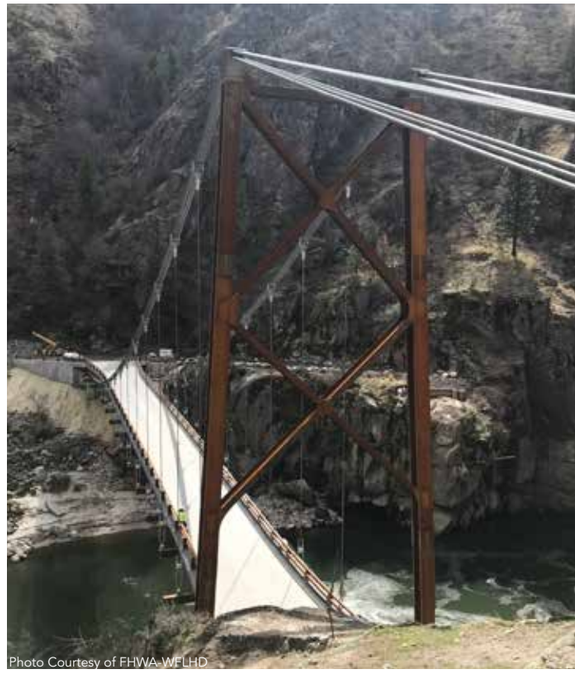


Photo Courtesy of FHWA-WFLHD

Ken Saindon (**kens@estinc.com**) is Colorado Bridge Group manager with EST and served as the technical lead and engineer of record for this project. **Alex Whitney** (**alexander.whitney@hdrinc.com**) is senior bridge project manager with HDR and served as consultant project manager. Both were formerly with Atkins.

WHILE CERTAINLY SCENIC, the steep nature of a V-shaped canyon near Riggins, Idaho, created quite the challenge for the designers of a replacement bridge over the Salmon River.

The original Manning Crevice Bridge carried Salmon River Road over the river at this location, providing access to residences, resorts and commercial rafting ventures and acting as a main artery for recreational users of the river and surrounding forest lands.

By 2010, the bridge (built in 1938) had reached the end of its service life, and the decision was made to replace it. But this would be no easy feat. The site, located in a steep canyon, had limited access for trucks and limited space available to stage construction equipment and materials, not to mention sharp bends in the road. The choice of steel for temporary and permanent works was crucial to developing a feasible erection scheme on this difficult site and addressed the following requirements for the replacement project:

- A bridge deck clear width of 16 ft for a single lane
- A minimum vertical clearance of 18 ft
- A minimum load capacity of AASHTO HL-93 and a 45-ton logging vehicle
- Roadway curvature at the bridge ends had to be able to accommodate a logging truck crossing the bridge
- No permanent construction could be placed within the 100-year flood plain
- Traffic had to be maintained on the existing bridge during construction
- The river had to remain open to rafters during construction
- Construction equipment was not allowed in the river

Not-So-Easy Access

After evaluating six different structural configurations, a single-tower, asymmetric suspension bridge scheme was chosen. Competent bedrock at the site provided ample capacity for anchoring large horizontal forces, thus favoring arch and suspension bridge types over cable-stayed structures. Given the limited access for construction equipment, cable suspension was judged to be more constructable than an arch because of the light weight and flexibility of steel cables. The bridge span length is 300 ft and with a cable sag of 18.5 ft at mid-span, the resulting sag ratio (span/sag) of 16.2 is much flatter than the classical suspension bridge sag ratio of 10. The bridge uses a total of 180 tons of structural steel.

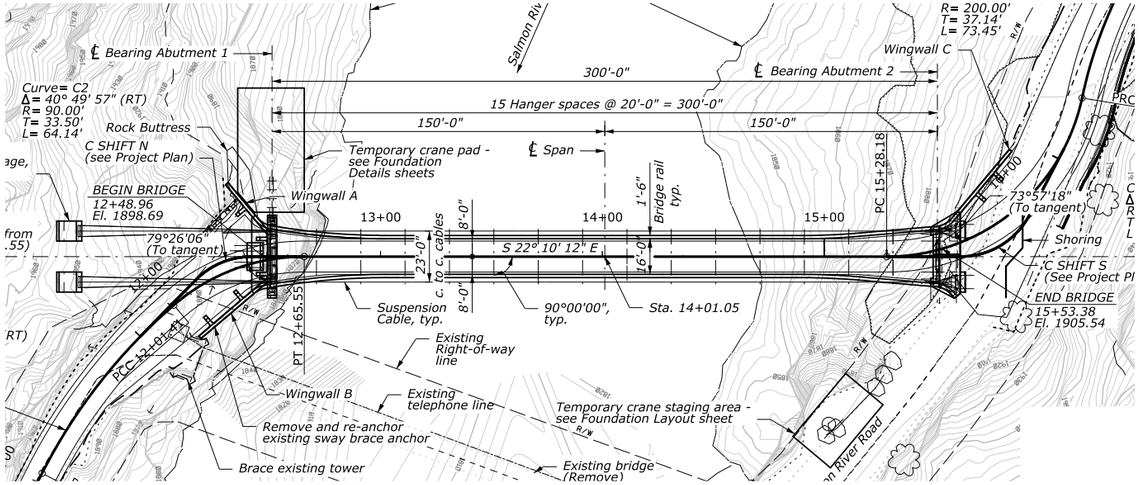
A number of factors led to the single-tower configuration. For one, the rock face adjacent to the north tower of the existing bridge required a minimum tower height of at least 60 ft to place anchorages on favorable rock geometry. A large debris flow zone and a continual water seep on the south hillside made this an unfavorable location for a new tower and anchorage. Finally, the size of crane that could be placed on the south side of the river was highly uncertain given that the only two access routes to the south side are either over an unpaved high mountain pass with very tight switchbacks or across the existing bridge, which had neither the geometry nor load capacity to handle a large crane. (Note that the CM/GC was able to deliver a large lattice crane over the high mountain pass to the south side of the structure.) As such, a tower on the south side of the river would not be feasible.

Orienting the new bridge was a balance between providing roadway alignment geometry to allow a WB-62 vehicle to negotiate the approaches, providing the shortest overall bridge length, maintaining the existing bridge in operation during construction and choosing a favorable tower and anchorage location on the north side of the river. The south abutment and anchorage were placed close to the river and, being below the road surface, has protection from hillside debris flows. The south abutment and anchorage placement also struck a balance between keeping all permanent construction outside the 100-year floodplain and providing sufficient room beyond the anchorage to allow traffic to pass during construction.

The site features a narrow shelf road with steep drop-offs in hard rock terrain. Standard construction techniques for such steep sites typically involve temporary benching. However, the hard rock site and pristine canyon location made benching both cost-prohibitive and inappropriate at the north abutment. Luckily, the presence of soil overburden on the south river bank allowed a cost-effective cut bench to be used at the south abutment. During the design phase, a temporary crane platform was located on the north side of the river for erection of the tower and cable anchorages. Additional temporary platforms were also used for construction at the north anchorage and behind the tower base. The existing south-side roadway bench was wide enough to accommodate a crane for erection and still allow vehicles to pass, and all construction materials were staged and delivered from Riggins to the north end of the bridge.

Steel Simplifies Erection Scheme

Helically wound galvanized wire (ASTM A586) was used for the main cables and hangers. The main cable and hanger cable con-



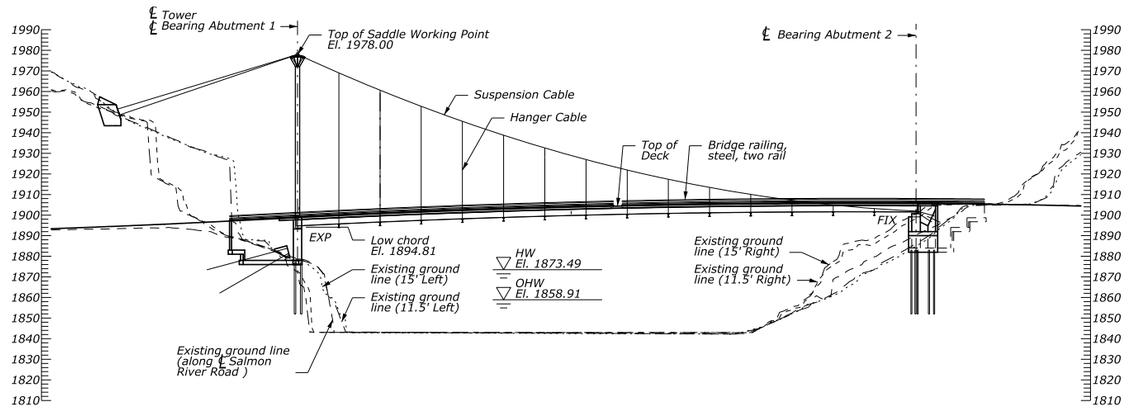
An overhead view of the tight project site and sharply turning roadway.

Photo Courtesy of FHWA-WFLHD

A view (looking west) of logistics on the south bank of the river, with the existing bridge in background.

Photo Courtesy of FHWA-WFLHD

Cable installation from the tower to the south abutment anchorage.



An elevation drawing of the new bridge.

Photo Courtesy of FHWA-WFLHD

A minimum tower height of at least 60 ft was required in order to place anchorages on favorable rock geometry.

nections consist of heavy steel castings with molten zinc spelter sockets, and the cable system saddles consist of 1-in.-thick steel plates and steel castings with groove and fillet welds throughout. The tower consists of welded I-sections for the battered legs and rolled W-shapes for the diagonal bracing. The superstructure framing was designed for simplicity and economy, and all members are rolled steel sections with W-shapes for the stiffening girders and floor beams and WT shapes for the lateral bracing. The stacked superstructure framing configuration was conceived to permit easy assembly from the bottom up, starting with the floor beams followed by the lateral bracing and then the stiffening girders. High-strength bolts were used in all field connections.

Tower erection was a breeze given the small reach and piece weights of about 9.5 tons. The main cables were erected using a cableway accordion sling (designed and patent-pending by Inland Crane) to support each strand at regular intervals on the temporary cables as it was pulled across the river from the tower to the south abutment. Erecting the cable hangers and bridge superstructure framing from the two fixed crane locations required crane reaches of up to 160 ft at mid-span. Hangers, floor beams and lateral bracing had piece weights of 2.25 tons or less, so the

long crane reach was not a problem for these items. The stiffening girder piece weights varied with the exterior 50-ft-long sections weighing around 5.5 tons and the interior 40-ft-long sections weighing 4.5 tons. Splice locations and piece weights were designed to reduce the demands on the cranes, and superstructure erection was completed in less than three weeks.

The new single-tower bridge opened this past June, bringing a touch of uniqueness to the canyon and respecting the constraints of the site with its force layout. With longevity in mind, especially considering the winter climate, Class C galvanizing was specified for the steel cables, and Grade 50 weathering steel was used for the towers and superstructure—not only for corrosion resistance but also to reduce visual contrast with the weathered granite prevalent at the site. The project's reception by the community has been overwhelmingly positive, and it is anticipated to last well beyond the century mark. ■

Owner
FHWA-Western Federal Lands, Vancouver, Wash.

Construction Manager/General Contractor
Record Steel Construction, Inc., Boise

Structural Engineer
Atkins, Denver

Steel Team

Fabricator
Rule Steel, Caldwell, Idaho

Detailer
ABS Structural, Melbourne, Fla.


Photo Courtesy of FHWA-WFLHD

Erecting the tower from a temporary crane platform.


Ken Saindon

Cable installation.


Inland Foundation Specialties

A view of the fixed crane positions for superstructure erection.

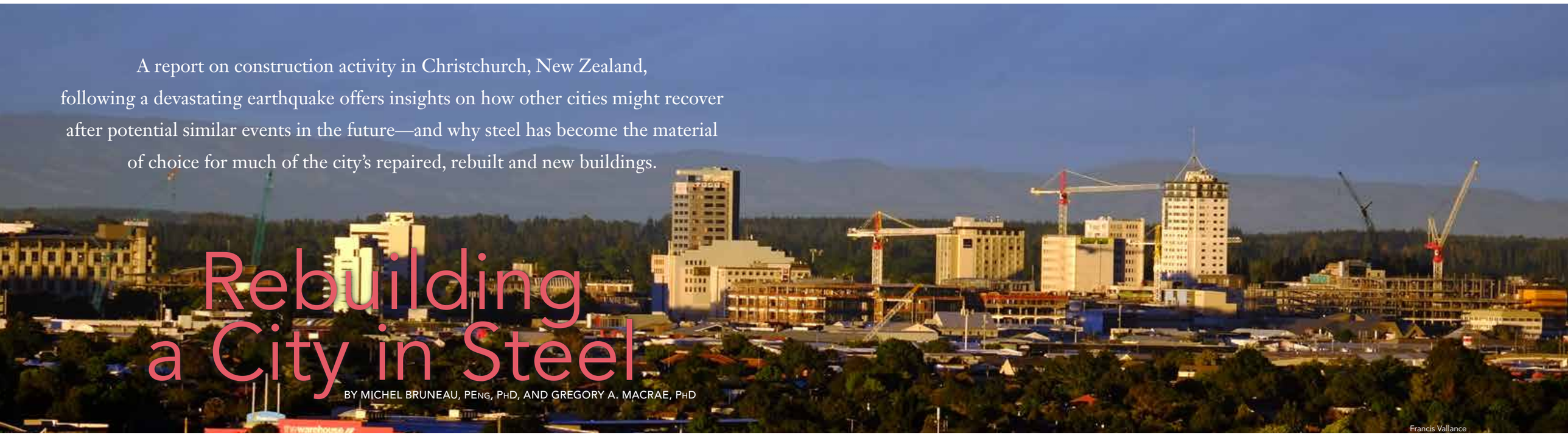

Photo Courtesy of FHWA-WFLHD

Temporary erection platforms on the north side of the river.

A report on construction activity in Christchurch, New Zealand, following a devastating earthquake offers insights on how other cities might recover after potential similar events in the future—and why steel has become the material of choice for much of the city's repaired, rebuilt and new buildings.

Rebuilding a City in Steel

BY MICHEL BRUNEAU, PEng, PhD, AND GREGORY A. MACRAE, PhD



Francis Vallance

The ongoing revitalization of the Christchurch skyline following the devastating earthquake of 2011.

Michel Bruneau (**bruneau@buffalo.edu**) is a professor in the Department of Civil, Structural and Environmental Engineering at the University at Buffalo, N.Y., and **Gregory MacRae** (**gregory.macrae@canterbury.ac.nz**) is an associate professor in the Department of Civil and Natural Resources Engineering at the University of Canterbury in Christchurch, New Zealand.

FOR THE PAST SEVEN-ODD YEARS, Christchurch, New Zealand's, central business district (CBD) has been—and continues to be—a landscape of sprawling construction sites, with multiple new buildings being constructed, a few existing ones being repaired, some still in the process of being demolished and a number of damaged structures boarded up awaiting their fate.

This flurry of activity is the result of the magnitude 6.3 earthquake that occurred on February 22, 2011, at a depth of a little over three miles and a horizontal distance of less than six miles from the CBD. The earthquake turned the CBD into a "red zone" with severely restricted access for many months.

Anyone walking through the heart of the city can witness the hustle and bustle of the rebuilding activity taking place. However, to structural engineers—who can't miss the fact that a large number of structural systems are being used in the process—the predominance of structural steel over that landscape can be striking. Where reinforced concrete structures dominated the building inventory prior to the earthquake, the "new Christchurch" that is emerging is a city with a variety of structural forms. The structural steel systems being used are diverse, ranging from traditional systems like eccentrically braced frames (EBF) to structures with replaceable EBF links, buckling restrained brace frames (BRBF), friction connections, viscous dampers, rocking frames and base isolators—a dramatic departure from past practices.

Why Steel?

But just how extensive is the shift in construction practice taking place in Christchurch—and, more importantly, what are the major factors that have driven decisions about structural materials and specific structural systems? To answer these questions, we conducted a series of interviews with the structural designers of more than 60% of the post-earthquake buildings constructed to date in Christchurch's CBD, as well as with a local architect, project manager and developer. Data was also collected from various sources, including Christchurch's City Council database, and quantitative information on structural forms and decision drivers has also been assembled. The interviews also provided a valuable overarching narrative on the reconstruction process that goes beyond the quantification process.

The findings from this study are presented in *Reconstructing Christchurch: A Seismic Shift in Building Structural System*, a 170-page report that can be downloaded for free from the Quake Centre's website (visit www.aisc.org/nzsteel). The information collected covers a total of 74 buildings, collectively adding to a total of 5,191,617 sq. ft of floor space. Results shows that as part of the reconstruction, structural steel has been used in the lateral force-resisting system (LFRS) of about half of the buildings. However, because this approach has been employed at a high rate in the larger structures, steel lateral force-resisting buildings account for 80% of the total square footage of all new construction encompassed in the study (as shown in Figure 1, right). Also, in buildings having a reinforced concrete LFRS, steel has been used for the gravity flooring system in about 75% of all cases. This results in approximately 95% of the total supported floor areas in new buildings relying on steel framing. Figure 1 also presents information as a function of year of consent—i.e., year of building permit—showing trends over time as part of Christchurch's ongoing reconstruction activities. Note that results for 2017 are only for the first three months of the year, as data was collected and last interviews were conducted in March of that year.

Subdividing the data into the various types of LFRS, the following results were obtained, in terms of number of buildings, floor areas and percentage of the total floor area, as indicated in Figure 1:

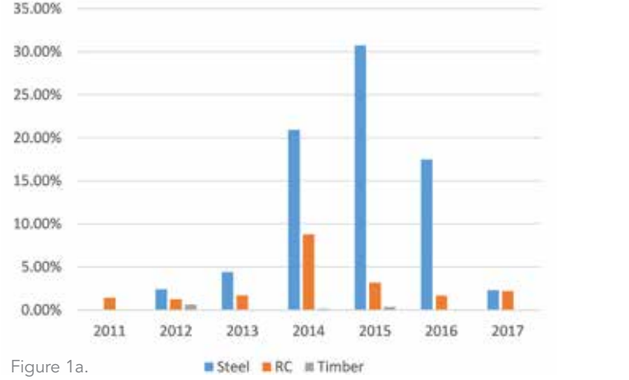


Figure 1a.

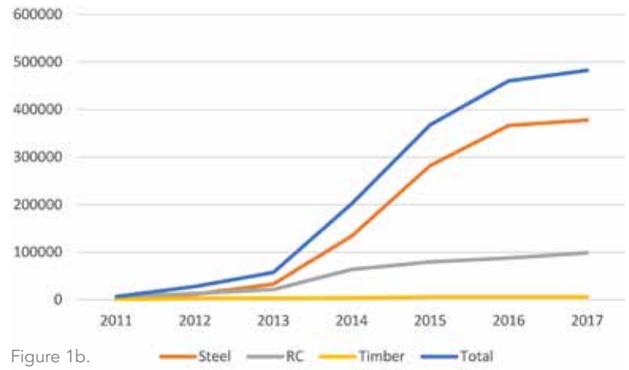


Figure 1b.

- MRF = steel moment resisting frames (9.5), MFF = steel moment resisting frames with friction connections (1) and MRD = steel moment resisting frames with reduced beam sections ("dog-bones") (4.5): 2,175,000 sq. ft (42%)
- BRB = buckling restrained braces (11): 1,195,000 sq. ft (23%)
- RCW = reinforced concrete walls (32.5): 865,500 sq. ft (17%)
- CBF = concentrically braced frames (3): 414,500 sq. ft (8%)
- EBF = eccentrically braced frames (2) and EBR = eccentrically braced frames with replaceable links (4): 296,000 sq. ft (6%)
- Other systems (such as rocking frames): 161,5000 sq. ft (4%)

Interestingly, the 11 base-isolated buildings (15% of the total number of buildings) alone provide a total 2,045,000 sq. ft, equivalent to 40% of the total floor area of the buildings considered in this study. This indicates that the base-isolated buildings have generally been large buildings. Indeed, the two largest base-isolated buildings alone, built specifically for public sector tenants, together add up to more 1,098,000 sq. ft (21% of the total floor area of the buildings considered). Note that the three largest buildings add up to 1,388,500 sq. ft (and 27% of the total floor area). A strong correlation was also observed between floor areas for base-isolated buildings and steel MRFs, although not exclusively.

An EBF with replaceable links (left) and a close-up of a link in an inverted-V braced frame (right).

A rocking frame system with energy-dissipating couplers between the frames.

To better understand the design trends, Figure 2 (below) shows results for all structures that have not been base-isolated, as it is interesting to identify which structural systems have been used more dominantly when buildings have not been base-isolated. Results, in terms of floor area indicated for each type of LFRS used, are as follows:

- BRB: 1,194,800 sq. ft (38%)
- RCW: 839,600 sq. ft (27%)
- MRF+MFF+MDF: 613,500 sq. ft (20%)
- EBF+EBR: 296,000 sq. ft (9.5%)
- CBF: 0 sq. ft (0%)
- Other: 169,000 sq. ft (5.5%)

In summary, 68% of all new non-base-isolated building area incorporates a steel LFRS.

Results from the qualitative part of the report indicate that the factors used to select specific structural systems are diverse and include costs, construction speed, perceptions of damage and structural performance, tenant requirements, local engineer-

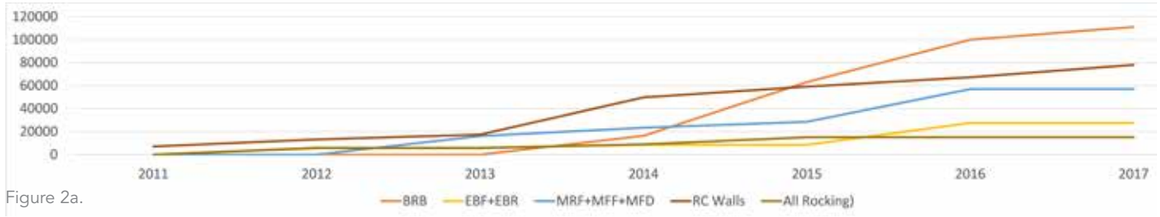


Figure 2a.

Figure 2b.

A BRB frame (left) and a column connection at mid-bay of the frame (right).

ing culture and other factors. These are explained through the narratives obtained from the interviews. This critical part of the report (i.e., 75 of the total 170 pages) cannot be summarized without losing critical perspective of: the breadth of opinions; the reasons that sustained decisions; and important nuances that impacted decisions from case to case. However, it can be drawn from this narrative that:

- Preventing loss of life is less frequently the most significant seismic performance objective for modern building
- The professional opinions of structural engineers drive the adoption of low-damage systems, but tenant expectations have a significant direct or indirect impact on the choice of structural systems for individual buildings
- Context directly affects these decisions
- While the reconstruction experience has paralleled an increase in stakeholders' knowledge, government regulations would still be required if the objective was to achieve an across-the-board increase in seismic performance for all buildings in a community—something unforeseen to occur at this time

It is noteworthy that the report also contains an Appendix showcasing a number of case studies that were provided by consultants to provide project-specific information and illustrate the decisions that led to selection of the chosen structural systems.

It is significant that New Zealand's building codes and seismic design requirements are similar to those in North America and other developed countries, and that Christchurch's mix and vintage of construction types before the earthquake was similarly comparable. As such, the Christchurch experience may be unique today, but it is likely to repeat itself in other similarly developed urban centers worldwide and provides unique insight into some of the mechanisms that can dictate structural engineering decisions during the post-earthquake reconstruction of a modern city. ■

This work was supported by the Quake Centre, based at the University of Canterbury, and made possible by the contributions of many consultants, steel fabricators, contractors and other individuals (listed in the report) who have met with the authors and have generously shared their experiences of the Christchurch reconstruction process. This work also benefitted from the Christchurch City Council, which provided information on building consents from the city database, and Steel Construction New Zealand, which kindly shared information from its own database.

A space moment frame (left) and a close-up of an RBS connection in the form of a bolted end-plate to moment-resisting connection to a square steel section (right).

A base friction connection (left) and a completed bidirectional moment friction connection (right).

Polyaspartic Coatings

BY AHREN OLSON, TODD WILLIAMS AND RONNIE MEDLOCK, PE

Reducing the cost of shop-painted steel bridges by improving painting efficiency.

PROTECTIVE COATINGS HAVE been used to mitigate corrosion on steel bridges for more than a century.

The state-of-the-art for the past several decades now has been a three-layer system consisting of an organic or inorganic zinc-rich primer, an epoxy intermediate coat and a polyurethane finish coat (commonly abbreviated as ZEU). Each layer provides specific protection mechanisms working in unity to prevent corrosion:

1. The zinc-rich primer provides galvanic protection, with the zinc preferentially "sacrificing" itself to protect the steel.
2. The epoxy layer provides barrier properties by reducing the permeability of water, oxygen and salts through the coating.
3. The polyurethane topcoat's main function is to protect the underlying coatings from the sun's ultraviolet rays while also providing abrasion and chemical resistance.

Economics and schedule impacts have driven multiple state and local departments of transportation (DOTs) to apply all three coats in the shop for new steel bridges. This has shifted the painting responsibility to steel fabricators or blast and paint shops. For fabricators, painting provides value-added work but can also create additional scheduling complications.

Applying three coats of paint is a time-intensive process. Each layer of paint has a minimum recoat time, which is the minimum amount of time before another layer can be applied. The recoat time is dependent on product chemistry and the degree of cure required before subsequent coatings can be applied. Environmental conditions also have a significant impact on recoat time. For instance, inorganic zinc-rich primers can require

Bridge #5160, which carries Main St. over the Little Madawaska River in Stockholm, Maine, was repainted with a PAS system.

Ahren Olson (ahren.olson@covestro.com) is the segment manager for corrosion protection and **Todd Williams** (todd.williams@covestro.com) is the protective and marine lab manager, both with Covestro, LLC. **Ronnie Medlock** (rmedlock@high.net) is vice president of technical services with High Steel Structures, LLC.

The Maine bridge project is a simple-span design with four steel girders spanning about 100 ft.

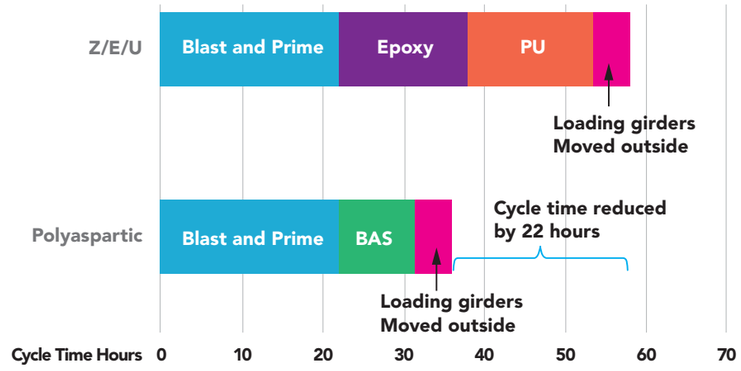


Figure 1. A graphical representation of the cycle time difference between the PAS system that was used on the Maine bridge project and a typical ZEU system.

more than 24 hours at low humidity to cure before subsequent coats can be applied, thus reducing productivity. In addition, the total time to apply a ZEU system in a shop setting can vary significantly depending on the available shop space and number of painting shifts per day. The longer the recoat time, the longer the product takes up space waiting, resulting in less product that is able to be handled. Depending on work load and scheduling, a fabricator may subcontract out painting due to the bottleneck that applying multi-layer coating creates in the paint shop.

Polyaspartic Solution

Advancements in coating resin technology have improved painting efficiency. More than 20 years ago, polyaspartic (PAS) coating resins were invented by Covestro. This new coating resin replaces the "polyol" or paint resin in the "A-side" of two-component polyurethanes.

PAS coatings bring two important application and physical property advantages:

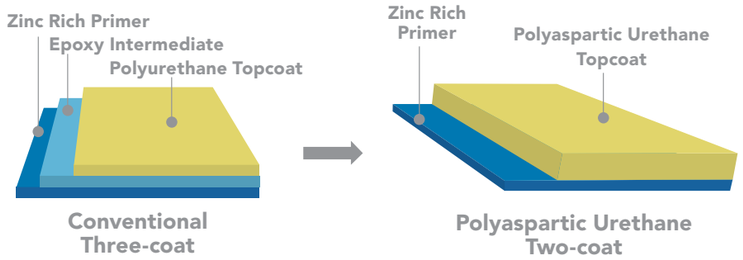
- In general, PAS coatings offer fast curing with a reasonable pot life (useable time to apply the coating). Typically, these coatings are dry-to-handle in one to two hours at 75 °F and 50% relative humidity, while having a pot life between two and three hours. By comparison, polyurethane coatings are dry-to-handle in six to eight hours, with a two- to four-hour pot life.
- They can be applied at higher dry film thicknesses (6-10 mils), which is much higher than polyurethanes (2-5 mils). The larger film build tolerance of PAS coatings allows for more forgiving application when painting complex geometries, as well as a reduction in the number of coats needed to provide corrosion protection. For instance, a ZEU three-coat system can be replaced by a two-coat system of zinc-rich primer with a PAS topcoat at the same overall film thickness.

PAS coatings are applied by the same means and methods as polyurethane coatings: spray, brush and roll. Their color and gloss retention is equivalent to polyurethanes, but they deliver better edge retention and cure significantly faster. These application and physical property advantages have been documented to increase painting productivity while reducing project costs without sacrificing corrosion protection. PAS coatings have become common in a number of different markets that shop-paint steel, including oil and gas, stadiums, railcars and structural steel.

PAS coatings have also been used in the steel bridge market for more than 15 years, and many of these applications have been in field maintenance painting. Since the early 2000s, a number of state DOTs have used PAS two-coat systems in this manner—e.g., Virginia, Maine, Connecticut, Michigan, Maryland, Pennsylvania, North Carolina and Kentucky—many of whom use salt liberally in the winter. In terms of total structures painted with PAS coatings, the Virginia DOT currently has the largest number for any one state, with more than 150 bridges.

The system has proven itself. The Connecticut DOT quantified the cost benefit for field applications of PAS coatings to show a cost reduction of up to 20% and a greater than 30% improvement in maintenance painting efficiency when compared to tradi-

A Virginia DOT project—I-64 over Simpson Creek in Clifton Forge—was repainted with a PAS system in 2005 and has experienced miminal rusting after 12 years in service (above photos and bottom-left photo on opposite page).



Layers of a standard three-coat ZEU system and a PAS two-coat system. Both systems have total dry film thicknesses ranging from 9 mils to 14 mils.

A close-up view of one of the painted beam ends on the Main St. bridge project.

tional ZEU systems. In addition, the long-term corrosion resistance of PAS coatings on steel bridges has been documented to show corrosion resistance equivalent to ZEU systems.

Spanning Main Street in Maine

While PAS coatings have predominantly been used for maintenance painting on steel bridges thus far, they are starting to see more use on new steel structures. One of these is bridge #5160, which carries Main St. over the Little Madawaska River in Stockholm, Maine, and was recently replaced with a new steel crossing (designed by HNTB and fabricated and detailed by NSBA member and AISC certified fabricator High Steel Structures). The design for the replacement structure is a simple-span bridge using four steel girders and spanning roughly 100 ft. The bridge was constructed with weathering steel girders with painted beam ends approximately 5 ft from both abutments. While the coating system was initially planned to be ZEU, the Maine DOT showed interest in PAS coatings after successfully using the technology for field maintenance painting, and as a result allowed a change order for the coating system. A two-coat system consisting of an organic zinc-rich primer with a PAS topcoat was eventually specified.

Beam ends were blasted to SSPC-SP 10 prior to primer application. Following surface preparation, the zinc-rich primer was applied per manufacturer requirements at 3-5 mils dry film thickness. After the primer was applied and inspection was complete, the PAS finish coat was applied using a single-component airless pump. The final inspection on the finish coat began four hours after completion of the application. After final inspection, the beams were loaded and moved outside to the lay-down yard. The total cycle time for blasting, painting and moving the finished product outside was 36 hours.

In order to provide a comparison between the two-coat PAS system and the traditional ZEU, a second timeline was put forward based on years of experience with ZEU systems. Both timelines assume the paint bay has three shifts. The total cycle time for the ZEU system for the same beam end project would be 58 hours (see Figure 1 on p. 40 for a graphical comparison of the time cycles between the PAS and ZEU systems). This timeline for the ZEU system also assumes ideal environmental conditions (temperature and humidity). Using the two-coat PAS system reduced the cycle time by 22 hours compared to the ZEU system. This 61% increase in throughput is attributed to reduced curing time and one less coating layer. The PAS system has a combined approximately six hours of curing "downtime" while ZEU has around 26 hours of curing downtime. One less layer for the PAS system also requires one less inspection, saving an additional two hours or so of cycle time. The PAS systems enables a significant improvement in the throughput and painting efficiency of the paint shop, essentially increasing a fabricator's painting capacity without having to add additional shop space or resources. In periods of high demand, PAS coatings can improve scheduling as well as require less painting work to be subcontracted out to third parties.

Reducing the number of paint layers improves the throughput and also generates cost savings through a reduction in coating application and steel handling costs in the painting process. While the material cost of a PAS system can be double that of a ZEU system, coating application and handling costs can be greatly reduced since, again, only two layers need to be applied versus three. In the case of the Maine project, the PAS system generated a 28% savings in coating application and steel handing in the painting opera-

A Michigan DOT project—West Road over I-75 in Woodhaven, Mich.—was repainted with a PAS system in 2017.

A Connecticut DOT project—I-75 over Starr Ave. in Danbury—was repainted in 2002 with a PAS system. After 15 years in service, minimal rust has been experienced.

tions. Considering both raw material cost increase and the coating application and steel handling savings, the PAS system created an overall cost reduction for painting of 14%, which factored to a 2% reduction in the total cost of the new fabricated and painted steel girders.

As the trend to shop-apply all coats of paint for new steel bridges continues, PAS coatings offer an option to deliver significant value to both fabricators and bridge owners requiring shop painting of new steel bridges. By reducing cycle time using PAS, steel bridge fabricators can gain additional painting capacity, and this can be very significant in periods of high painting demand. Ultimately, this will lead to time and cost savings for owners who can leverage the advantages of PAS systems into solutions for new steel bridges without having to sacrifice long-term corrosion resistance. ■

This article is a summary of Session B25 "Advanced Coating Systems" from the 2018 NASCC: The Steel Conference/World Steel Bridge Symposium in Baltimore. Next year's conference takes place April 3-5 in St. Louis. Learn more at ***www.aisc.org/nascc****.*

Spanning Generations—and Troubled Waters

BY VICTORIA CSERVENYAK

A team of bridge professionals connects a remote Panamanian village to nearby communities with a new bridge over a treacherous river.

IN AN ISOLATED jungle community 30 minutes from the nearest town, an 80-year-old woman hesitantly crosses a suspension bridge over the Tuancle River nearly 100 ft below to her family on the other side.

Following behind, two children laugh and romp along the bridge, gliding their little hands across the chain-link fences on the sides.

At the foot of the bridge, a man speaks about how going forward, this day will have a happy meaning for him and his family, who several years earlier had a son die on the same date.

The 100-ft-long El Macho Puente (puente is Spanish for bridge) opened this past spring and the nearly 200 residents of the village—for which the bridge is

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left: El Macho residents test their new bridge.

above: Building the bridge over the Tuancle River.

named—are celebrating their newfound freedom of safely traveling to and from their Panamanian hamlet over the river.

New Bridge, New Hope

But less than a week before this celebration, no bridge existed.

Spring is the dry season in El Macho, which lies about 200 miles west of Panama City. So the river, which geographically quarantines the remote area from larger towns, is only about 1 ft deep and even becomes a dry riverbed in some places.

But it's a different story during the rainy season, when villagers are sometimes forced to traverse the rocky riverbed to reach nearby communities—and are sometimes simply unable to make it. During the rainy season when flash floods are common, the water can surge to more than 6 ft high, making the river impassable. Recently, a few men were carrying a sick friend on a hammock, attempting to take him to the doctor. When they arrived at the river, the water was too high and before it could recede enough to cross, the man died.

Six months before the first bridge tower was installed, plans commenced to build a footbridge in El Macho across the river. And over the course of three months prior to the bridge's opening, Maria Rodriguez, the Panama country manager; Daniel Magallon, Bridges to Prosperity (B2P) mason; Chase Luckey, B2P fellow (volunteer); and the El Macho community worked to create the foundation.

Carter Bearden and Gary Kinchen installing decking near the middle of the bridge span.

Patrick Montgomery and team putting together scaffolding.

John Hastings and Jeff Carlson installing hanger assemblies.

Camaraderie and Colleagues

Since leading his first volunteer trip with B2P in 2016, Jeff Carlson, NSBA's director of market development, has been enthusiastically committed to the organization's mission. Whenever he meets with the AEC community, he evangelizes about the need for footbridges in rural areas throughout the world—which is how the El Macho team formed. In addition to Carlson, team members included Carter Bearden (HDR); John Hastings (Tennessee DOT); Marne Helbing (Tennessee DOT); Gary Kinchen (New Mexico DOT); Patrick Montgomery (Fought and Company, an AISC member and certified fabricator); Carlos Ramirez (WSP); Michelle Romage-Chambers (Texas DOT); and Scott Wilson (Palmer Engineering).

When the team began their trip, they were not familiar with each other, yet bonded as they worked seamlessly without construction issues or personality clashes.

"The most memorable part of the bridge for me was twofold," said Carlson. "First, everyone on the team worked well with one another. They were all respectful of their fellow teammates, the B2P staff and the local community. Second, I was impressed by how organized the B2P Panama staff was for our project."

"Our group had a lot of camaraderie," added Wilson. "We could have fun and at the same time all work hard towards the same goal, which was a benefit I didn't expect."

The completed El Macho Bridge before the inauguration ceremony.

Children excitedly skipping across the bridge for the first time.

The B2P team celebrating the completion of the El Macho Bridge—and holding up Jeff Carlson.

The group's gregariousness helped each team member to adroitly and quickly discover how to best use their individual skills to benefit the group as a whole. As a fluent Spanish-speaker, Ramirez harmoniously coordinated the community members and Kinchen cheerfully supervised the fabrication and rebar cutting on the ground, while the other team members constructed the towers and assembled the remaining pieces. Montgomery, as a fabricator, attempted not to heckle his team members, who were adjusting from their usual computer work to onerous manual labor.

"Most bridge designers are not used to hands-on experience, and to suddenly take a concept on a piece of paper and translate that into an actual built structure was a challenge at first," Kinchen explained.

However, Montgomery was happily astounded by his teammates' enthusiasm. "All the engineers were down-to-earth and ready to go to work," he said. "You hear that engineers are going to be finger-pointers. But every single one of them wanted to get their hands dirty. And they did."

Kind-Hearted Community

With both the temperature and humidity in the 90s (degrees and percent, respectively) the team spent the first day acclimating to the steamy climate, then dove in to work side by side with the El Macho residents to construct the bridge. It was essential to the B2P team that the community members take an active role in construction so they would know how to make future repairs to the bridge as necessary. Throughout the week, between 15 and 25 community members assisted with construction, spanning from 12-year-olds to octogenarians, and even more inhabitants made the American team feel extremely welcome. Each morning, two women walked for two hours to cook breakfast over a fire. Families invited the group into their concrete-walled, dirt-floor homes for lunch, and other community members cooked them dinner at the campsite at night. The villagers even built a hut made out of palm branches for the workers to take a reprieve from the blistering sun.

Once the crew arrived in El Macho, two weeks were allotted for building the bridge and despite a few minor injuries, they completed the project in six days.

"It's so gratifying to do something else that goes along with the skills that you have, especially in places where they're desperately needed," Ramirez said.

In addition to the gratifying work, the friendships formed also made the trip an unforgettable experience. Getting to know the community was the highlight of everyone's trip.

"It's really a neat relationship that you gain working with them and working with a lot of people you don't know; you get to know them well over that two-week period," Hastings said. "Everybody was wonderful. The whole experience was wonderful."

Although they started off as strangers, the team members were so invigorated by their journey to Panama that they have already begun to plan the next B2P opportunity. ■

Building Bridges

Through local engagement, from regional governments to members of each partner community, Bridges to Prosperity (B2P) is committed to a sustainable model that puts the focus on people and the opportunities that make it possible for them to thrive. In 2018, B2P will complete 39 new footbridges, increasing its overall total to 279 bridges and impacting more than 1,000,000 people since 2001.

To learn more about B2P, how you can become a volunteer or industry partner or to support its mission, visit www.bridgestoprosperity.org.

Design With a Twist

BY BO DOWSWELL, PE, PHD

AISC's new design guide provides much-needed advice on designing projects with curved steel.

Courtesy of AISC Bender-Roller Committee

Curved steel can turn a pedestrian bridge into an art installation.

Bo Dowswell (**bo@arcstructural.com**) is a principal with ARC International, LLC, in Birmingham, Ala., and also a consultant to AISC's Steel Solutions Center.

THERE'S A COMMON misconception that architectural appeal and structural efficiency are mutually exclusive, with the idea that one comes at the expense of the other.

Curved members do an elegant job of busting that myth by highlighting the beauty of structural steel while also offering structural efficiency. They are often chosen for exposed structures when aesthetics are a priority, but industrial buildings and non-building structures make use of curved members as well. For these structures, functionality is more important than aesthetics, and curved members are typically used in situations where they are more efficient than straight members.

A New Resource for Curved Steel

Although curved structures represent beauty and simplicity, the structural behavior of curved members can be quite different from their straight counterparts. Despite the widespread use of curved structural steel members, detailed guidance relative to United States design practice has been scarce—until now. The recently published AISC Design Guide 33: *Curved Member Design* (available at www.aisc.org/dg) provides design guidance and practical information on the fabrication, design and detailing of curved members. The contents of the new design guide are briefly summarized here:

Chapter 1 provides an introduction to curved members, with a discussion of typical applications in both commercial and industrial structures.

Chapter 2 describes various geometries available for curved members and the methods used to bend them. Due to the wide variety of bending equipment available, almost any structural shape can be curved, including wide-flanges, standard shapes, channels, angles, hollow structural sections (HSS) and welded built-up members. Bender-roller companies, who specialize in curving steel members, can provide further information on the fabrication of curved members. Because each bender-roller has different capabilities, discussing bending requirements as early in the project as possible will allow for a smooth design and construction process.

Chapter 3 discusses behavior during the bending operation. It provides information about curving mechanics and reducing the risk of fracture and excessive cross-sectional distortion during bending.

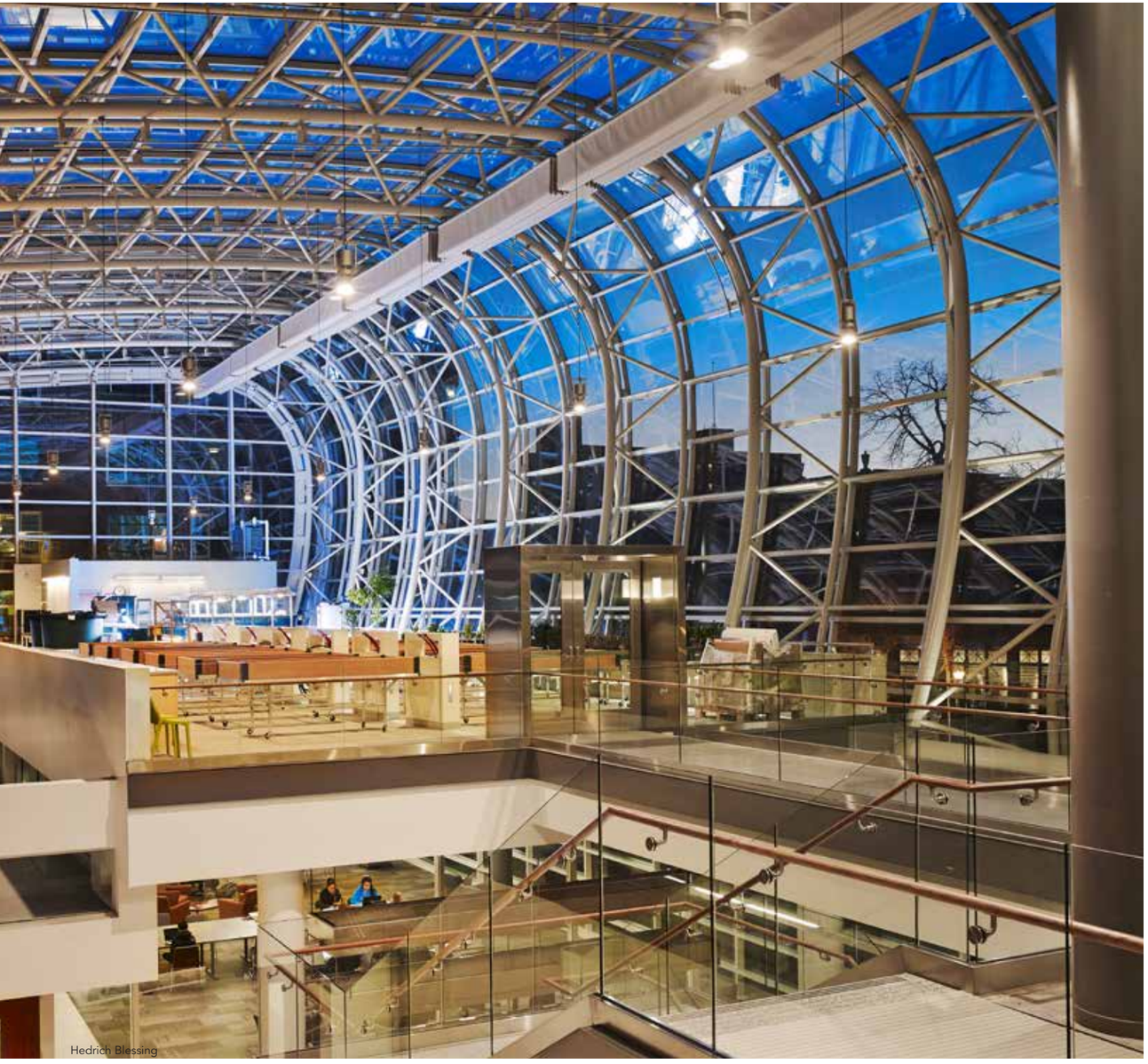
Chapter 4 focuses on detailing and fabrication requirements, including tolerances and dimensions required by the bender-roller for successfully bending members.

Chapter 5 discusses several considerations that, in some cases, may affect the design of curved members. These factors include potential changes in material properties caused by the bending process and their effects on the structural behavior of curved members. Other topics include residual stresses, nonlinear flexural stresses and cross-sectional distortion. The final section of Chapter 5 provides information that should be included in the contract documents to ensure the expected product is what is supplied.

Chapters 6 and 7 provide design methods and equations for vertically and horizontally curved members, respectively. These chapters discuss the strength, stability and serviceability of members, as well as connection design. All design equations comply with the 2016 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360, available at www.aisc.org/specifications).

Chapter 8 shows how to implement the equations in Chapters 6 and 7 via three extensive design examples.

The final parts of the publication include a glossary of common terms and a list of AISC associate member bender-roller member companies that can provide invaluable practical information in the conceptual and design stages of a project.



Hedrich Blessing

A curved HSS frame transforms a university building's atrium into a campus icon.

Curved Members

Here are a few of the more common types of curved steel:

Vertical. The ability of arches to span long distances provides an opportunity for large open spaces such as pedestrian bridges. A similar visual effect can be created with vertically curved roof beams. For industrial structures, vertically curved members may be used as circumferential shell stiffening rings for horizontal vessels, large industrial ducts and tubular conveyor galleries.

Horizontal. Although horizontally curved members are usually less efficient structurally than straight beams, they are often used to carry loads at curved floors and roofs. In some cases, such as for transportation and pedestrian bridges, horizontally curved structures are required due to geometrical constraints. For industrial structures, horizontally curved members can be used for monorail beams, chimney grillages, circumferential shell stiffeners and silo/tank roofs.

Specialty. Specialty bends are often required to form members to the proper geometry. Because parabolic curves are efficient for resisting gravity loads, many arches have a parabolic geometry, which requires a variable-radius specialty bend. Bender-rollers also have the capability to form complex curves with small, varying radii about multiple axes. For industrial structures, specialty bends are used primarily for spiral stairs providing access for circular vessels and for monorail beams with compound curves.

Sherman Takata

A curved roof steel in an airport project.

James Steincamp Photography

Curved HSS for a parking canopy.

S-shape curved canopy members.

Courtesy of AISC Bender-Roller Committee

Complex curves for a public sculpture.

Courtesy of AISC Bender-Roller Committee

Curved steel for a multistory spiral staircase.

Courtesy of AISC Bender-Roller Committee

Courtesy of AISC Bender-Roller Committee

Horizontally curved roof beams on a recreation center project.

Courtesy of AISC Bender-Roller Committee

An arch bridge, curved and painted for aesthetic purposes.

The design guide brings all of the latest information on curved members into a single document that is compatible with the 2016 AISC *Specification*. Although most of the guidance is focused on structural design, architects, fabricators and detailers will also find the document to be a great resource full of critical information on using curved members in steel structures. ■

Design Guide 33—and all other AISC design guides—is available at www.aisc.org/dg. For information related to curved members, visit ***www.aisc.org/curvedsteel****. Also see "There's More Than One Way to Bend a Beam" in the January 2016 issue, available at www.modernsteel.com.*

AISC Bender-Rollers

Here is a list of current AISC Bender-Roller Committee members:

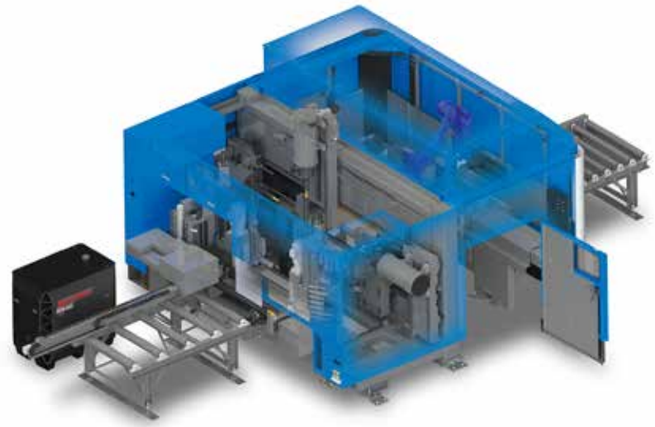
A-1 Roll Company
Albina Company, Inc.
Bendco, Inc.
BendTec, Inc.
Chicago Metal Rolled Products
Greiner Industries, Inc.
Hodgson Custom Rolling, Inc.
Holloway Company, Inc.
Hornsby Steel
Kottler Metal Products
Kubes Steel, Inc.
Max Weiss Company
Metals USA
Midwest Metal Products
Paramount Roll and Forming
Shaped Steel, Inc.
SIMS Steel, Inc.
Whitefab

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

DAITO DCM1050

Daito's DCM1050 is a multitasking machine combining a CNC Drill (with three spindles), a CNC robotic coper for standard or complicated plasma cuts and a marking machine for part number and welding/layout marking. Each drill has an automatic tool changer to cover tapping, milling, chamfering holes and boring blind holes. The wide range of functions makes the DCM the most versatile machine for non-repetitive structural steel processing.

For more information, visit **www.daitousa.com** or call 847.437.6788.



PRODEVCO PCR42

The PCR42 advanced robotic plasma steel cutting system combines CNC plasma cutting and torch technology with fully automated robotics, noncontact measuring and vision systems. The PCR42 now features Hypertherm's XPR300 plasma unit. For structural steel cutting, this delivers the perfect combination of speed, precision and full four-face operation. The PCR42 robotic plasma cutting system does beam coping, notches, holes and weld preps, splits beams and scribes, plus it marks on all four faces of H-beams, channels, angles, HSS and plates. The PCR42's cutting envelope covers all four faces of steel profiles, allowing for HSS cutting in a single pass and marking on all four sides. The PCR42 is the first system of its kind in the world to plasma cut standard structural steel profiles, pipe and round tube up to 26 in. in diameter with the same plasma coping equipment. Please check us out at Fabtech 2018 in Atlanta at Booth B5851!

For more information, visit **www.prodevcoind.com** or call 877.226.4501.

PEDDINGHAUS ANGLEMASTER-663

Serving as the industry's benchmark for productivity, the Anglemaster-663 caters to today's fabricators' need to automate punching, shearing and marking for angles and flat bar. The Anglemaster-663 can accommodate shaped holes such as squares, rectangles, obrounds and slots, including a feature that allows nibbling of material. This particular Anglemaster boasts a new punch tool design that allows for simplicity in tool changes, in addition to a misting feature that extends punch tool and shear life during operation. A new and improved patent-pending roller feed is designed to flex with the material in order to ensure a constant roll and accurate measurement, given deviations in material, with no stop in production.

For more information, visit **www.peddinghaus.com** or call 815.937.3800.

ENGINEERING JOURNAL

Fourth Quarter *EJ* Now Available

The fourth quarter 2018 issue of AISC's *Engineering Journal* is now available. You can access the current issue as well as past issues at **www.aisc.org/ej**. Below are summaries of this edition's articles:

- **Technical Note: Post-Fire Axial Load Resistance of Concrete-Filled, Double-Skin Tube (CFDST) Stub Columns**
  Reza Imani and Michel Bruneau

This technical note reports findings on a series of squash tests to investigate the effects of a significant fire loading history on the axial load strength of concrete-filled, double-skin tube (CFDST) stub columns. Axial loading tests were conducted on two stub columns that were previously subjected to the first 60 minutes of the standard ASTM E119 (ASTM, 2012) fire. Results were compared to the resistance of an identical virgin stub column. Comparisons indicated an average reduction of 28% in the axial load strength of stub columns when subjected to the mentioned fire loading history.

- **Quantifying Inelastic Force and Deformation Demands on Buckling Restrained Braces and Structural System Response**
  Justin D. Marshall, Brandt Saxey and Zhongliang Xie

Buckling-restrained braced frames (BRBFs) have become a very popular lateral-resisting system due to their balanced, full hysteresis and the ability to tailor stiffness, within limits, and strength to meet specific design requirements. This paper reports the results of an analytical investigation on the performance of buckling-restrained braces (BRBs) and the global performance of BRBFs, with a focus on the ductility and overstrength demands on the braces. Nonlinear analytical models of various three- and six-story steel frames were subjected to a suite of earthquake records to determine the demands on the BRB elements and the overall frame response. The structure variations include the location (i.e., seismic hazard), seismic importance factor, *Ie*, brace configuration (chevron versus single diagonal) and BRB yielding core length.

- **Local Strength of Single-Coped Beams**
  Bo Dowswell

In beam-to-beam connections, the top flange of the supported beam is usually coped to clear the supporting beam flange. Due to flexural and shear stresses in the coped portion of the web, the local strength can be limited by buckling. Design recommendations in previous editions of the AISC *Manual* imposed limits on the cope geometry and were based on an allowable stress philosophy, limiting the flexural strength to the first-yield moment. To eliminate the limits of applicability and provide equations that take advantage of any available post-yield strength, the design guidance in the 15th Edition AISC *Steel Construction Manual* has been revised from previous editions of the AISC *Manual*. This paper discusses the development of the revised design procedure and validates the equations with the results of 25 experimental tests from five independent research projects.

- **Steel Structures Research Update: Seismic Performance and Design of Steel Panel Dampers for Steel Moment Frames**
  Judy Liu

Ongoing work on the seismic performance and design of steel panel dampers for steel moment frames is highlighted. Dr. Keh-Chyuan Tsai, professor in the Department of Civil Engineering at National Taiwan University, leads the team from National Taiwan University and the National Center for Research on Earthquake Engineering (NCREE) in Taipei. In 2018 at NCREE, one recent collaboration with the University of Washington included cyclic tests of a three-story chevron special concentrically braced frame (SCBF). Current seismic design provisions require large beam sizes to resist the unbalanced forces from the chevron braces after brace buckling. The research has explored options for alternative ductile mechanisms and reduced beam sizes. Steel research at NCREE has also included studies on steel beam-to-box-column moment connections and electro-slag-welded (ESW) joints in those connections.

People and Companies

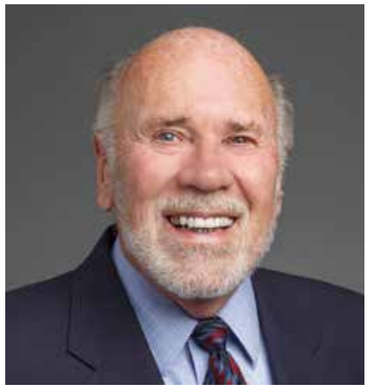
"Insatiable appetite to learn coupled with the fear of failure," says Ron Klemencic, SE, PE, chairman and CEO of Magnusson Klemencic Associates (MKA) on his motivation to innovate. Klemencic is interviewed in the latest episode of AISC's *Steel Profiles* podcast series at **www.aisc.org/podcasts**. Tune in to hear his fascinating hour-long conversation with Margaret A. Matthew, PE, AISC senior engineer and host of *Steel Profiles*. He gives an inside look at this life and career including his innovative ideas on tall building design, a revolutionary new composite structural steel framing system, his crusade to give back to the industry and even the scoop on his fear of heights.

Klemencic has led the development of the SpeedCore system, aiming to replace the reinforced concrete core in steel office-tower construction. The system is expected to take 40% less time to build than a steel frame with a reinforced concrete core. To learn more about this revolutionary system, visit **www.aisc.org/speedcore**.

You can play or download all 28 episodes of *Steel Profiles* at **www.aisc.org/podcasts** or on iTunes. In iTunes, simply search for "Steel Profiles" to access all of the episodes. You can also subscribe to the series for free, and each new episode will automatically be downloaded for you.

IN MEMORIAM

Steel Industry Mourns Terry Peshia, Former AISC Board Chair



Terry F. Peshia, CEO of Garbe Iron Works, Aurora, Ill. (an AISC member and certified fabricator), and former chair of the AISC Board of Directors, passed away on August 9 at the age of 80. Fittingly, his company provided the steel fabrication for the hospital, Rush-Copley Medical Center in Aurora, where he died.

"The AISC Board is saddened by the loss of our friend and mentor Terry Peshia," said David Zalesne, current AISC chair and president of Owen Steel Company (an AISC member and certified fabricator). "Terry joined the Board in 1996 and served as Board Chair from 2001 to 2003. He remained actively engaged in many Board activities over the years, including his current service as Chair of AISC Holdings, Inc., with oversight of AISC treasury and intellectual property assets. Even as the industry changed and evolved, Terry remained a strong advocate for its core values, which he lived out through his entire professional career at Garbe Iron Works, and his personal dedication to his wife, Connie, and their sons, Ted and John. The AISC Board will miss his presence, guidance and friendship."

A recognized leader in the structural steel industry, Peshia began as an ironworker and joined Garbe Iron Works in 1964, where he spent his entire career. In addition to his significant contributions to AISC, he served as chair of the Central Fabricators Association and was a member of the Associated Steel Erectors. In 2009 he received the prestigious AISC Stupp Award for Leadership Excellence. He was a 1961 graduate of Worsham College of Mortuary Science and served his country proudly as a member of the United States Army.

"Terry's influence and impact on the steel industry cannot be overstated," said Charlie Carter, AISC's president. "He served and led AISC, CFA and the Ironworkers faithfully for many decades, all with wisdom, vigor and dedication. A master of appropriate, helpful criticism, he also was your strongest supporter as you adapted to resolve it. His vision was always of what could be. His talent was orchestrating it to happen."

Roger Ferch, former president of AISC, said, "I first met Terry 20 years ago when I joined the AISC Board. From the beginning, Terry was a mentor to me and very welcoming to new Board members. He was always very informed on topics of discussion and certainly never bashful about making his opinions known. But perhaps his greatest strength was that of being a team player and supporting the organization once a decision was made. As strong as he was working behind the scenes and voicing his thoughts during Board meetings, once a vote was taken, Terry transferred his tireless volunteerism into executing the plan. I have many fond memories of Terry and will remember him as both a great friend and a mentor. He was a genuine leader and I listened closely to his sage advice. Terry was also the most influential in recruiting me to move from sunny California to Illinois to become AISC's president in 2006."

Larry Kruth, AISC's vice president of engineering, added, "Terry has been a major influence to many people in the steel industry, not only through his leadership at AISC but also his many years as a member and leader of CFA, and he will be greatly missed. He was a great inspiration to me personally by getting me actively involved in both organizations, which eventually led to the position I now hold at AISC after working at Douglas Steel [an AISC member and certified fabricator] for over 30 years. I will truly miss his guidance and in-depth knowledge of the industry, and will always treasure his friendship and the help he has offered to me over the years. One of the most memorable experiences I had with Terry was the time he had the CFA Dinner hosted at the Auburn Cord Duesenberg Automotive Museum. In addition to the great experience, the most amazing part was to have Terry show us his 1932 Auburn Boattail Speedster."

An avid car collector, Peshia was a member of the Auburn Cord Duesenberg Club and lifetime member of the ACD Museum in Auburn, Ind. He was also a freemason and a member of the Jerusalem Temple Lodge in Aurora, Ill. He was very involved in many county and community organizations, including the Kendall County Board, the Oswego Village Board and the Kendall County Historical Society, where he was a past president, and he was also a former Oswego police commissioner.

"In every organization, there is a committed individual that has a major impact on the success of the group," said Lou Gurthet, former president of AISC. "Terry Peshia was that person with his family, with Garbe Iron Works and with AISC. Quiet, thoughtful and determined—even downright stubborn—he played that role in support of AISC. He had the ability to articulate what was needed, how it could be achieved and the strength to hold to his conviction. He was the person you wanted as a friend and on your team."

Tom Schlafly, chief of engineering at AISC, added, "I worked with Terry over 20 years ago and for many years since. He impressed me with his attention and success with some of the more arcane issues of running his business that many others did not show interest in."

TARIFFS

U.S. Imposes Tariffs on Chinese Steel

In August, the Office of the United States Trade Representative (USTR) released its list of approximately $16 billion worth of imports from China that will be subject to a 25% tariff as part of the U.S. response to China's unfair trade practices related to the forced transfer of American technology and intellectual property. The list contains 279 of the original 284 tariff lines proposed, **including those AISC requested for inclusion in its two separate testimonies in front of the USTR Section 301 Committee**. This is the first time that tariffs have been extended beyond mill products to include fabricated structural steel, and they went into effect on August 23.

To read AISC's most recent testimony given by David Zalesne, chairman of AISC and president of Owen Steel Company (an AISC member and certified fabricator) visit **www.aisc.org/tariffs**.

Charlie Carter, president of AISC, said, "On behalf of AISC and the American structural steel industry, we appreciate USTR's willingness to listen, and commend the USTR for including fabricated steel tariff codes on the current list for action under Section 301. This is a very positive step toward protecting downstream users that have been left exposed to foreign fabricators circumventing current 232 trade actions."

BRIDGES

SSSBA Publishes Research on New Steel Tub Girder Bridge Technology

A new report from the Short Span Steel Bridge Alliance (SSSBA), *Development and Experimental Testing of Press-Brake-Formed Steel Tub Girders for Short Span Bridge Applications*, reveals new technology that allows for accelerated construction and reduced traffic interruptions.

The five volumes, compiled by Karl Barth, PhD, and Greg Michaelson, PhD, PE, can be downloaded for free at **www.shortspansteelbridges.org**.

- Volume I – "Development and Feasibility Assessment of Shallow Press-Brake-Formed Steel Tub Girders for Short Span Bridge Applications"
- Volume II – "Experimental Evaluation of Non-Composite Shallow Press-Brake-Formed Steel Tub Girders"
- Volume III – "Evaluation of Modular Press-Brake-Formed Tub Girders With UHPC Joints"
- Volume IV – "Field Performance Assessment of Press-Brake-Formed Steel Tub Girder Superstructures"
- Volume V – "Fatigue Performance of Uncoated and Galvanized Composite Press-Brake-Formed Tub Girders"

STEEL SYSTEMS

Connection Testing Leads to More Testing—and New Connections

Sometimes testing yields further, unanticipated testing.

Earlier this year, in testing its bolted special moment frame connection at the University of California San Diego test lab, seismic connection manufacturer SidePlate successfully deformed a W27×102 beam on an HSS20×20×⅞ column with no damage to the connection after full cycle testing at 1.0%, 2.0%, 3.0%, 4.0%, 5.0%, 6.0% and 7%. Realizing the connection was fully intact, the team decided it could be tested again.

So an identical beam was installed (all tests were done with a story height of 14 ft and a span of 22 ft) but to make it more interesting this time around, the research team removed the VSE (vertical shear element) bolts and tested again, resulting in a full test cycle up to 6%—far exceeding the code required limit of 4%—and the connection was still in undamaged. The test team met and decided that the connection offered significant versatility and, based on finite element analysis performed in-house, the company's engineers developed a new tuck connection that minimizes the spatial impact of the joint. This configuration, tested with a W33×106 "built-up" beam, saw 1.0%, 2.0% and 3.0% cycles and two full cycles at 4%. In other words, one connection and column had now been through 18 cycles and was still ready for further testing.

"We didn't set out on a resiliency test path in developing this new connection, but things happen in engineering," exclaimed SidePlate's president, Henry Gallart.

Test four used a duplicate "built-up" beam and after achieving a successful 4% test, the plate finally showed some deformation but held up through another 4% test, which was performed for good measure. In all, the connection completed 23 test cycles and resulted in two new connections configurations.

Additionally, the column used in the testing allowed a *b/t* ratio of up to 21, far exceeding the standard of 14. By pushing the limits, the team was able to turn the costs associated with testing into certification testing, validating a new design and opening up a new era of design with the larger HSS columns.

"Conducting four successful full-scale tests with the same column and side plates means that a building using our connections can be subjected to a code-level earthquake four times and still be operational after replacing the beams," noted SidePlate's director of research and development, Bezhad Rafezy. "It validates our philosophy for our field-bolted connection design: life-safety performance that gives a building owner reduced downtime after a major earthquake."

STUDENT STEEL BRIDGE COMPETITION

2019 Student Steel Bridge Competition Rules Released

Each year, the Rules Committee creates a new mock scenario and set of rules for the Student Steel Bridge Competition (SSBC), keeping the competition challenging and exciting for the student teams—and the 2019 rules are now available.

This year's problem statement involves historic steel railroad bridges in Hawaii that must be rebuilt due to volcanic flows and earthquake activity. But there's a twist: Each bridge must include a cantilever supported by offset footings on one end.

As in past competitions, the rules include design, construction, equipment and safety requirements. After constructing their bridges, teams will perform vertical and lateral load tests.

Starting in early spring of 2019, 18 regional competitions will be hosted by universities nationwide. AISC will offer funding and assistance in finding sponsors for bridge teams and host schools. Depending on the number of teams at each competition, between one and four teams will move on to the 2019 Student Steel Bridge Competition: National Finals at the University of Texas El Paso, May 24-25.

For the complete problem statement and rules, visit **www.aisc.org/ssbc**.

In addition, AISC, in coordination with its many professional, industry and academic volunteers, will conduct all aspects of the 2019 SSBC as the sole organizing sponsor, including regional competitions. For more on the new organization of the SSBC, see the related August 16 news item at **www.modernsteel.com**.

correction

In the July article "Intensified Care" (available at **www.modernsteel.com**) the bender-roller for the CHI Health project was erroneously left off the project team list. Curved steel was provided by AISC associate member Chicago Metal Rolled Products.

This seminar focuses on the updates in these new publications and will be presented in the following cities in Fall 2018:

10/4 Seattle, WA
10/18 Minneapolis, MN
10/19 Anchorage, AK
10/25 Honolulu, HI
11/8 Washington, DC

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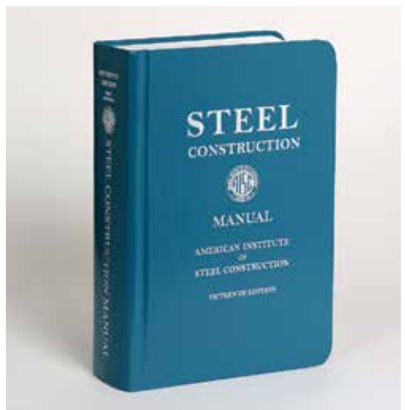

More information can be found at **www.aisc.org/seminars**

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Contract auditors must have knowledge of quality management systems, audit principles and techniques. Knowledge of the structural steel construction industry quality management systems is preferred but not required as is certifications for CWI, CQA or NDT. Prior or current auditing experience or auditing certifications are preferred but not required. Interested contractors should submit a statement of interest and resume to **contractor@qmconline.org**.

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Peddinghaus FPDB-2500 CNC Heavy Plate Processor, 96"Width, (3) Drill Spindles, HPR260 Plasma, (1) Oxy Torch, (1) Plasma Head, Siemens 840D CNC, 2008 **#27974**
Controlled Automation BT1-1433 CNC Oxy/Plasma Cutting System, 14' x 33', Oxy, (2) Hy-Def 200 Amp Plasma, 2002 **#20654**
Controlled Automation ABL-100-B CNC Flat Bar Detail Line, 143 Ton Punch, 400 Ton Single Cut Shear, 40' Infeed, 1999 **#24216**
Controlled Automation 2AT-175 CNC Plate Punch, 175 Ton, 30" x 60"Travel, 1-1/2" Max. Plate, PC CNC, 1996 **#23503**
Controlled Automation DRL344 CNC Beam Drill Line, Hem WF140 Saw, Tandem Line, 2008 **#24937**
Ficep Gemini 324PG Plate Processor, 10' x 40', 15 HP Drill, HPR260XD Plasma Bevel Head, (1) Oxy, 2014 **#28489**

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STEEL BALLOONS

STEEL IS A LOT of things, though inflatable probably isn't one of the first adjectives that comes to mind when describing it.

But inflatable steel is the star of Balloonité, a multistory residential design concept created by Austin Vandepoll and Nathalie Altamirano, both architecture students at the University of North Carolina at Charlotte.

The project is a nod to Unité d'habitation, a mid-century housing design principle focused on communal living, and reimagines the concept using inflatable steel technology.

How do you inflate steel? Simple. First, cut two 18-gauge steel sheets into the desired shape. Next, weld the edges and seams together, making sure to keep the blowhole open. Finally, pump 90-psi air into the cavity. Repeat until you have the desired number of modules for your building.

The concept was created for the 2017-2018 Steel Design Student Competition. Administered by the Association of Collegiate Schools of Architecture (ACSA) and sponsored by AISC, the competition encourages architecture students from across North America to explore the many functional and aesthetic uses for steel in design and construction. This year's competition included two categories. Category I—for which Balloonité was the winner—focused on affordable housing, and Category II was an open competition. You can learn more about Balloonité, as well as of this year's winners, in next month's issue (you can also view the winners at **www.acsa-arch.org**).

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