Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

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

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

Steel Solutions Center
One East Wacker Dr., Suite 3100
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solutions@aisc.org

PRETENSIONING ANCHOR RODS

from September 2003

I have a project where there is a need to pretension the column anchor rods to 110 kips. I am using 1.75″-diameter ASTM F 1554 Grade 105 rods. The contractor proposes using a hydraulic torque wrench in lieu of a hydraulic bolt-pretensioning device due to technical, safety, schedule and cost considerations. If we were to allow him to use the hydraulic torquing, what is the necessary torque (ft-lb) required to obtain the 110 kip pretension force? In your opinion, at this 110 kip tension, how much of a risk is there for the anchor rod to fail as a result of the combined tension and torsion stresses? If this was a standard slip-critical bolted connection and the bolt failed during tensioning it would not be a big deal, a new bolt could easily be inserted. However with a concrete-embedded anchor rod it would be an issue if the rod were to fail.

Question sent to AISC's Steel Solutions Center

Pretensioning anchor rods is good requirement for high-performance buildings. Tensioning anchor rods with a hydraulic tensioner is a common requirement in the nuclear industry, and the petro-chemical industry. The best solution is undoubtedly the use of a calibrated hydraulic bolt pretensioner such as those manufactured by Biach, Hydraulics Technology Inc., Boltech, and Tentec. Bolt pretensioning mitigates problems with low cycle fatigue and helps reduce lateral drift.

Torque is a very poor indicator of bolt tension. That is why it is not allowed for tensioning high-strength bolts. If the bolts are less than or equal to 1½″ diameter, a viable alternative is the load-indicating washer. Some manufacturers of load indicating washers can provide a load chart that will indicate the gap for bolt pretension forces less than the 70% minimum tensile strength required for high-strength slip-critical bolts.

That said, there is a procedure that can convert bolt torque to bolt tension. The procedure is contained in the *Fastener Design Manual*, by Richard Barrett, March 1990, NASA Reference Publication 1228. The formula is $T = KFd$ where T is the torque, K is a torque coefficient, F is the axial load, and d is the bolt diameter. The K is often assumed to be 0.2, but this is a gross approximation and varies markedly depending on coefficient of friction of the bolt to rod and bolt head to clamping surface. Calibrating the tension, if required, would be a very challenging process.

Again, I would strongly advise to use a hydraulic bolt-tensioning device if for no other reason then to make the calibration process easier.

Harold O. Sprague, P.E.
Black & Veatch Special Projects Corp.
Overland Park, KS

INDUSTRY STANDARDS

Please indicate industry standards for each of the following:

1. **Who is responsible for the design of the moment connections once the design criteria is provided on design drawings (reactions, moments, etc.)?**
2. **Should we request in our Specifications that structural calculations be provided as part of the shop drawings from the fabricator?**
3. **If the fabricator does not provide standard AISC details for moment connections, who is responsible for showing that they meet the design criteria?**

Question sent to Editor at **Structural Engineer** *magazine*

Most of your questions relate to the AISC *Code of Standard Practice, March 7, 2000*, which is available as a free download at **www.aisc.org/code**.

1. Section 3.1.2 and the commentary discuss it. We suggest that the contract documents should clearly indicate when connection design is delegated, with the specific requirements for what must be submitted in the review and approval process. That way, all bidders will be looking at the same requirements and the bids will be suitable for direct comparison.
2. There are a variety of practices used, some more than others. Standard practice seems to vary by region as well. We have seen the one you describe. If you require in the contract documents that the fabricator provide calculations for the moment connections, the fabricator should include the associated cost, whether they have the necessary engineering capability in-house or must contract separately for this service.
3. At a minimum, the engineer's preferences for connections should be shown on the structural design drawings with details. If the fabricator wants to use

alternative connections, Section 4.2 of the AISC *Code of Standard Practice* requires that a request be submitted before submission of shop and erection drawings. The details AISC shows in the *Manual* and other technical literature offer many good alternatives. We have seen others used that are suitable as well. The ultimate decision regarding suitability rests with the Engineer of Record. If I were submitting connection details for the review and approval of the EOR, I'd think it is my job to show they are suitable to the EOR. Presumably this will involve more work for me if I want to do something that is not in the *Manual*, but it is not impossible.

Bill Liddy
AISC Steel Solutions Center
Chicago, IL

SPRAY-APPLIED FIRE PROTECTION

Reprint of FAQ 11.1.7 at www.aisc.org/faq

Can spray-applied fire protection be applied to painted or galvanized steel?

In most cases, steel that is to be fire-protected should not be painted or galvanized. However, when such steel must be painted, additional measures can be taken to ensure adhesion. It is always prudent to consult your fire-protection contractor/supplier and your authority having jurisdiction (AHJ) in the early stages of the project on this issue, if you expect any of your painted or galvanized steel to be fire-protected by spray-applied materials. Spray-applied fire-protection materials can be applied to primed or painted steel shapes provided they have passed the bond tests in accordance with ASTM E736 *Standard Test Method for Cohesion/Adhesion of Sprayed Fire Resistive Materials Applied to Structural Members.* These tests should indicate a minimum average bond strength of 80% and a minimum individual bond strength of 50%, compared to the bond strength of the same fire protection material when applied to clean unpainted/ungalvanized steel plate surface. Some AHJs will enforce similar requirements for galvanized surfaces. Producers of fire protection materials usually maintain a list of "pre-approved" paints that have already passed the ASTM E736 tests. They will also advise you on the applicability of their product to galvanized surfaces. Additional tests will be needed if the coated steel surface has not been pre-approved. Where the bond strength is found unacceptable, a mechanical bond may be obtained by wrapping the structural member with expanded metal lath (min. 1.7 lbs/sq. yd).

STEEL AFTER FIRE

Reprint of FAQ 11.2.4 at www.aisc.org/faq

Can steel continue to be used in a building after it has been in a fire? How can you assess the capacity of steel that has been exposed to fire? Are there concerns about internal or residual stress effects that have to be considered?

It should be kept in mind that steel is born in a melting process that is significantly hotter than any building fire. Significant residual stresses are therefore present in all newly manufactured steel members. A detailed discussion of post-fire steel assessment issues is provided in

R. H. R. Tide, "Integrity of Structural Steel After Exposure to Fire," *Engineering Journal*, First Quarter, 1998, pp. 26-38.

A general rule of thumb reads: "If it is still straight after exposure to fire—the steel is OK". Straightening techniques are also available for steel members that have been misaligned after fire exposure. See also FAQ 11.2.1.

FIRE TEMPERATURES

Reprint of FAQ 11.3.3 at www.aisc.org/faq

At what temperature does a typical fire burn?

The duration and the maximum temperature of a fire in a building compartment depends on several factors, including the amount and configuration of available combustibles, ventilation conditions, properties of the compartment enclosure, weather conditions, etc. In common circumstances, the maximum temperature of a fully developed building fire will rarely exceed 1800°F. The average gas temperature in a fully developed fire is not likely to reach 1500°F. Temperatures of fires that have not developed to post-flashover stage will not exceed 1000°F.

NEW QUESTIONS

CHANGE IN *R* VALUES

There are several changes related to *R* (response modification coefficient) values, for example, the *R* value for Ordinary Steel Moment Frames changed from 4.0 (IBC 2000) to 3.5 (IBC 2003). What are the major reasons for these changes of *R* values?

Uksun Kim, Ph.D.
Georgia Institute of Technology

Hodgson Custom Rolling Inc.

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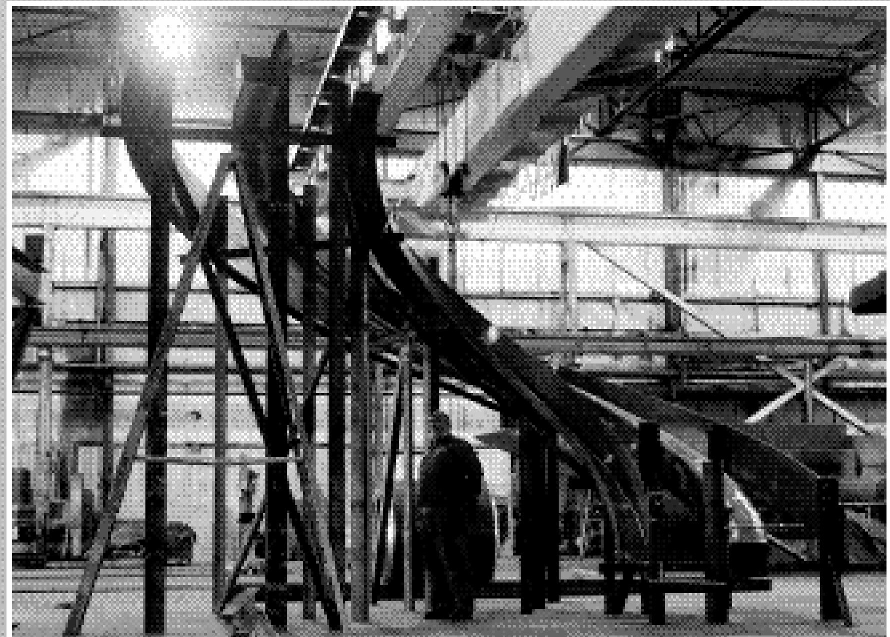

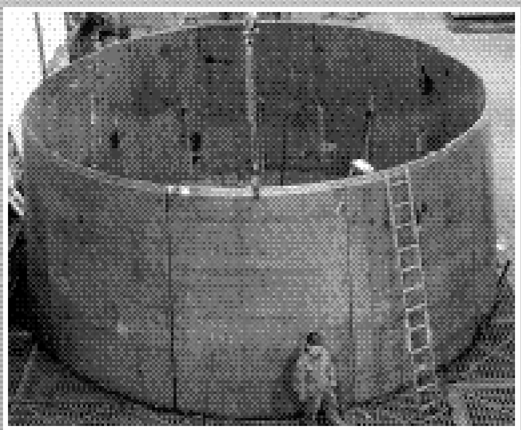

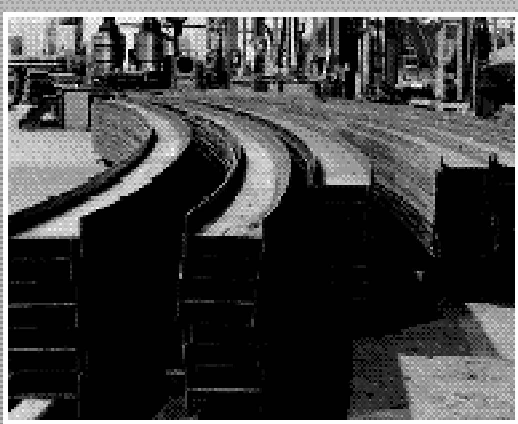

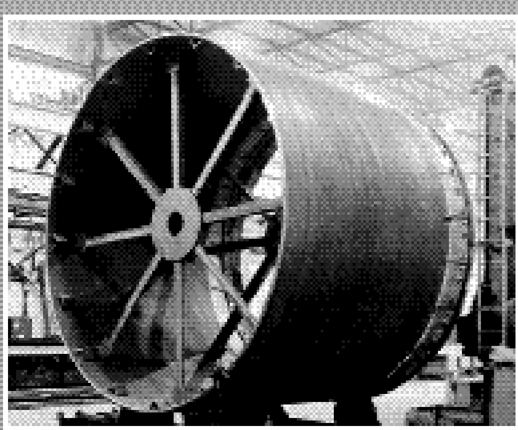

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HODGSON CAN HELP SOLVE YOUR PROBLEMS

Steel Quiz, a monthly feature in *Modern Steel Construction*, allows you to test your knowledge of steel design and construction. All references to LRFD specifications pertain to the 1999 *LRFD Specification for Structural Steel Buildings*, available as a free download at

www.aisc.org/lrfdspec

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

If you or your firm are interested in submitting a *Steel Quiz* question or column, contact:

Steel Solutions Center

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fax: 312.670.9032
solutions@aisc.org

This month's *Steel Quiz* was prepared by the staff of the AISC Steel Solutions Center.

1. Can ASTM A325 or A490 high-strength bolts be torch cut?

2. What is the maximum length for ASTM A325 high-strength bolts?

3. Can ASTM A325 or A490 high-strength bolts be used as anchor rods?

4. Why must a cambered beam be measured in the Fabricator's shop in the unstressed condition?

5. For connections whose design requires flexibility of the outstanding legs, what is the maximum length of fillet welded end returns?

6. Is there a comprehensive Specification available for designing with ASTM A53 Grade B pipe?

7. Where can I obtain electronic copies of past NASCC Proceedings?

8. Can the new AISC Bookstore product codes can be used to identify contract document specifications?

9. What was one of the first high-strength steels used in 1915 in the Metropolis Bridge (Illinois) and later in portions of the Golden Gate Bridge?

10. What is the maximum spacing to anchor steel deck to supporting members in order to resist uplift?

TURN PAGE FOR ANSWERS

STEEL QUIZ

ANSWERS

1. No. ASTM A325 and A490 bolts are heat treated, quenched and tempered according to the ASTM A325-02 and A490-02 Standards. Heat, such as from torch cutting, can alter the mechanical properties of the bolt.

2. Although there is no maximum length specified in the ASTM A325-02 Standard, some suppliers can stock up to 10-inch lengths, which is near the practical limit for high-speed cold heading machines. Longer lengths may be available through special orders.

3. No. ASTM A325 and A490 are high-strength structural bolts developed for use in steel-to-steel bolted assemblies based on the design provisions found in the AISC Specification (Chapter J) and/or the *RCSC Specification for Structural Joints using ASTM A325 or A490 Bolts*. Also, the length limitations discussed in Answer No. 2 hamper their use as anchorages. Proper materials for use as anchor rods are discussed in AISC FAQ 7.1.2 at **www.aisc.org/faq** or Table 2-3 of the 3rd Edition *LRFD Manual*.

4. According to Section 6.4.4. of the 2000 *Code of Standard Practice for Steel Buildings and Bridges*, there is no known way to inspect beam camber after the beam is received in the field because of factors that include:
   a. The release of stresses in members over time and in varying applications;
   b. The effects of the dead weight on the member;
   c. The restraint caused by the end connections in the erected state; and,
   d. The effects of additional deal load that may ultimately be intended to be applied, if any.

5. The length of weld returns cannot exceed four times the nominal size of the fillet weld. Refer to Section J2.2b(3) of the 1999 *LRFD Specification* (a free download from **www.aisc.org/lrfdspec**).

6. Yes, the 2000 *LRFD Specification for Steel Hollow Structural Sections* was developed explicitly for HSS (ASTM A500, A501, A618 and A847) and Steel Pipe (ASTM A53 Grade B) members. It is available as a free download from **www.aisc.org/freedownloads**.

7. The most popular NASCC Proceedings papers, as well all AISC Design Guides and a special version of the AISC Shapes Database are now available for immediate download from the ePubs section of the AISC website, **www.aisc.org/epubs**. If you are a Professional Member, the service is free-of-charge.

8. Yes. The new AISC product codes serve a dual-purpose. They are used to place orders with the AISC Bookstore (**www.aisc.org/bookstore**) or can be used in specification contract documents. As an example, the 1999 *LRFD Specification* is product code AISC 350-99. As one can readily notice from this product code, this specification was released in 1999.

9. ASTM A94, also historically known as silicon steel. During retrofits, one may find mention of silicon steel in old structural design drawings. One of the first high-strength steels, silicon steel had a yield strength of 45 ksi and a tensile strength of 80-95 ksi. It typically was used in steel bridges and incorporated into the lower portions of built-up columns in buildings back in the 1910s and 1920s. For information regarding historical materials, shapes and specification design values, refer to *Design Guide 15: AISC Rehabilitation and Retrofit Guide*.

10. 18 inches. Section I3.5b of the 1999 *LRFD Specification* contains this information, and states that anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

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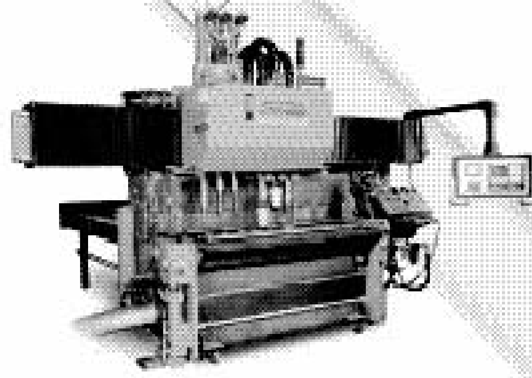

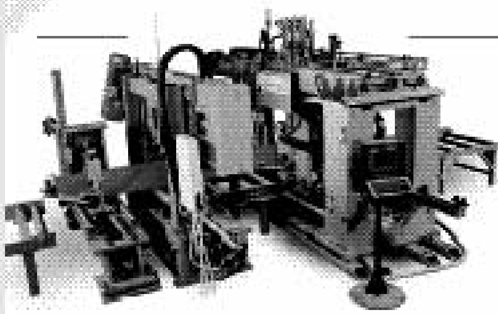

Adapting new technologies into productive machine designs has enabled Peddinghaus to provide cost efficient fabrication methods for all structural steel sections. Five automated systems are available for drilling, punching, coping, and flame cutting with capacities up to 50" wide for fabricated beams and columns. Peddinghaus has pioneered the development of multiple system CNC operation, which saves floor space, labor, and related costs.

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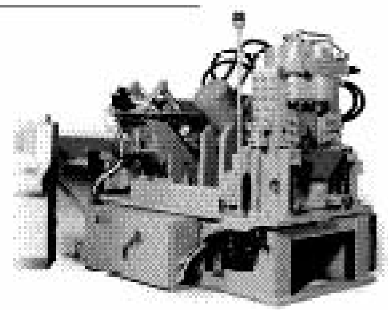

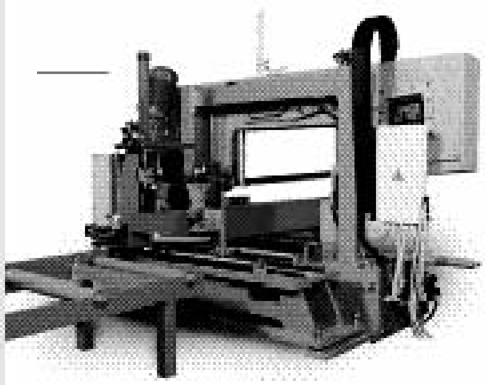

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