

Steel Interchange

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.

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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 at:

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What is supposed to happen when you need through plates in each direction for a tube or pipe connection? It seems to me that you wouldn't get a lot of benefit out of just running the plate with the lesser load back into the other plate and butting against it.

It's an amusing detail to say the least. Unless you're at the end of the HSS, I suppose you need a really short welder to crawl down into the HSS!

Seriously, though, the AISC *HSS Connections Manual* covers both single-plate (shear tab) connections, which are made with a plate welded directly to the face of HSS (i.e., without a slot), and their more expensive cousin, through-plate connections, which are made with a plate that penetrates the face of the HSS. It is expressed in that publication that the conventional single-plate connection should be used whenever possible. The design procedure covers all limit states, including those to ensure that the column face is adequate without the need to extend the plate through the wall and make it a through-plate connection.

Through plates are expensive because they are difficult to fabricate, so avoid them if you can. And you almost always can. Avoid them especially when opposite through plates would intersect each other inside the HSS. If the plate isn't continuous to the other HSS wall, I can't see what good it does to extend it inside. It may actually do harm since you would have to hack up the wall of the HSS to make that kind of connection.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL

On one of my projects, the steel fabricator/erector has chosen to use twist-off-type tension-control bolts. In a few locations at beam to column connections, they are requesting to remove the twist off extension and use the turn-of-the

nut method or a tension-calibrated device because of clearances.

The question becomes how to remove the extension. The erector wants to flame cut the extension. Is this detrimental to the chemical composition of the bolt? Should this be permitted?

S. Craig Brown, P.E.
Brown and Kubican, PSC
Lexington, KY

Flame-cutting of the twist-off tip should not be allowed. The heat can affect the material properties of the bolt. Usually, the engaged threads are at (or very close to) the end of the bolt. So, changing the material characteristics where the pretension is transferred from the bolt to the nut is not advisable. There are a few possible alternatives, though.

1. The bolts may not be required to be pretensioned. Just because pretension is a characteristic of TC bolts, doesn't mean pretension is required in the joint. If it is not required, just install the bolts in the joint to the snug-tight condition instead.
2. Consider if the tip has to be removed at all.
3. The current (1994) RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* allows A325 black (non-galvanized) bolts to be reused with the approval of the EOR. This is based on research presented in *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd ed. (Kulak, Fisher, and Struik). So, if they are A325 black bolts and he approves reuse they can install the bolts in a "blank" hole. Once the bolt is pretensioned and the tip is twisted off, the nut can be loosened, the bolt removed and installed in its intended joint. The turn of nut, calibrated wrench or DTI method can be used to induce the required pretension. If the bolts are button heads, it may be hard to calibrate a wrench in a Skidmore due to the polished bushings, but installation in the steel can usually be accomplished without rotation of the assembly. Typically, there

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is enough friction between the bolt head and the steel to prevent rotation. If DTT's are used, they should not be placed under the head in this specific application.

4. There are a variety of specialty TC wrenches and extension sockets available and one may suit this application to allow the TC installation.
5. Substitute standard hex-head bolts of the same nominal length for those holes where clearance is an issue. This may look strange, but it doesn't affect the behavior of the joint.

Heath Mitchell

American Institute of Steel Construction
Chicago, IL

I recently detailed a job and used the engineer's drawings to place the shop marks for the structural steel, with my title block. I got a phone call from the engineer who chewed me out for using his drawings. In short, I had to apologize until I was blue in the face. Now I have to redraw the exact same thing, but with my handwriting on it.

In all my years in the business, this was the first time I have had to apologize for something like this. I guess I have been lucky to work with people who didn't mind sharing their drawings. Is this client being overly picky, or am I doing something wrong?

This issue is one that has long been a concern of both the design community and the construction industry. When drawings were only in paper form, the stickyback was as useful as the cut and paste is in an electronic file today.

In either case (or any similar derivation), however, such use without permission is essentially not legal. Drawings in paper and/or electronic form are intellectual property owned by the producer (engineer, architect, etc.). And if that producer is not agreeable, no one else may legally use them, no matter how common such use has been or may be.

The exact extent of the problem varies significantly. I know of one extreme example where the engineer's stamp was transferred to the shop and erection drawings through copying of the plans. Just try to get that guy to give permission today!

More likely, though, the designer knows how important it is to the project schedule (and to project simplification and cost reduction) to grant permission to use this kind of information in the preparation of shop and erection drawings. However, it is

quite important to realize that permission is required for the use of the designer's drawings.

To ease the process of obtaining such permission, the new AISC *Code of Standard Practice* specifically addresses a default arrangement for permission for the use of designer's drawings in the preparation of shop and erection drawings. You can see the new AISC *Code of Standard Practice* here:

www.aisc.org/code.html

See Section 4.3, which covers the use of CAD files or copies of the design drawings.

Charles J. Carter, S.E., P.E.

American Institute of Steel Construction
Chicago, IL

I have two questions. First, what is the physical meaning of the \bar{x} function shown on page 6-19 of the AISC *Hollow Structural Sections Connection Manual*?

Second, it appears that the section properties for HSS indicated in the HSS *Connections Manual* are different than those shown in the *ASD Manual of Steel Construction*, 9th ed. and the *LRFD Manual of Steel Construction*, 2nd ed. Why is this?

Jay Kleven

EAPC Architects and Engineers
Grand Forks, ND

Regarding your first question, section 2.1 of the RHSS specification (in the back of the black HSS *Manual*) describes \bar{x} . It is essentially the distance from the plane of a connection to the centroid of the steel carrying forces being attributed to that weld. It's used to account for shear lag in tension rupture strength calculations.

In response to your second question, the section properties in the HSS *Manual* are slightly smaller than those in the other manuals. HSS producers tend to produce HSS at the minimum thickness tolerance specified in ASTM A500 which allows the actual wall thickness to be 10% less than the nominal wall thickness. After an attempt to change the ASTM tolerances failed, AISC and the Steel Tube Institute recalculated their tables based on a 7% reduction in wall thickness, resulting in slightly smaller section properties.

Keith M. Mueller, Ph.D.

American Institute of Steel Construction
Chicago, IL

I have a question regarding the Gas Metal Arc Welding (GMAW) process. For as long as I can remember, our steel specifications prohibit the use of GMAW. Despite this, we are finding fabricators using this method in their shop. It is my understanding that the prohibition of the GMAW process is due to the fact that the weld quality is dependent on maintaining controlled conditions, which cannot always be assured.

Does AISC maintain a position on this welding process? Are there some types or locations of welds that should or should not use this process?

James A. Boje, P.E.
Structural Engineer

Regarding the use of Gas Metal Arc Welding (GMAW), AISC does not prohibit its use. The AISC LRFD specification references the AWS Structural Welding Code (AWS D1.1), which has provisions for its use. GMAW is listed as a prequalified welding process (AWS D1.1-2000, para. 3.2.1), as long as the short circuiting transfer mode is not used. In order to use the short circuiting mode, the Welding Procedure Specification (WPS) must be qualified by test (AWS D1.1-2000, para. 3.2.2). Additionally, the AWS Bridge Welding Code (AWS D1.5-95, para. 1.3.1) allows GMAW in qualified WPSs.

The quality of any weld is dependent on the conditions. For example, the quality may be effected by the ambient atmosphere, the base metal, the welding procedures, the electrode storage and handling, and other conditions. However, the welding code provides guidance to minimize the influence of these variables.

(continued)

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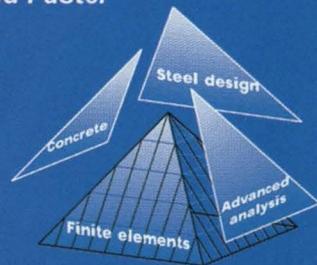
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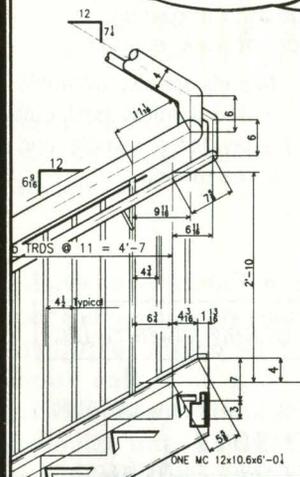
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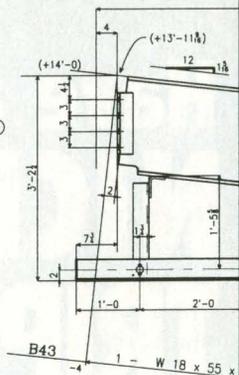
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GMAW should not be used in windy conditions due to potential loss of shielding gas. The welding code states that "GMAW, GTAW, EGW, or FCAW-G shall not be done in a draft or wind unless the weld is protected by a shelter. Such shelter shall be of material and shape appropriate to reduce wind velocity in the vicinity of the weld to a maximum of five miles per hour" (AWS D1.1-2000, para. 5.12.1).

Scott Funderburk
Lincoln Electric
Cleveland, OH

New Questions

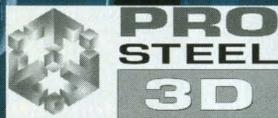
Are there any standards or guidelines for structural steel drafting, particularly for the presentation and content of design drawings? I am interested in finding "rules" about showing the weights of beams on plans and column weights on elevations, etc. Each company seems to have developed its own drafting convention, but I would like to know if there are industry-wide standards.

Brian W. Bersch, P.E.

The *LRFD Manual*, Vol. II, part 11, includes a discussion on holes for anchor rods and grouting in column base plates. Table 11-3 recommends base plate hole sizes to accommodate anchor rods. The discussion indicates that "An adequate washer should be provided for each anchor rod." Why are there recommended holes sizes so much larger than those in the *ASD Manual*, Part 4? What washer materials and thicknesses would be considered adequate?

Rick Drake, S.E.
Fluor Daniel, Inc.
Aliso Viejo, CA

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