If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

Thermal Cutting

We are purchasing a plasma table for our fabrication facility. Is plasma permitted to be used to create holes for bolts and anchor rods?

Thermally cut holes for bolted connections in buildings are explicitly allowed in the AISC *Specification*. See Section M2.5, which states:

"Bolt holes shall comply with the provisions of the RCSC *Specification for Structural Joints Using High-Strength Bolts*, hereafter referred to as the RCSC *Specification*, Section 3.3 except that thermally cut holes are permitted with a surface roughness profile not exceeding 1,000 µin. (25 µm) as defined in ASME B46.1. *Gouges* shall not exceed a depth of 1/16 in. (2 mm). Water jet cut holes are also permitted."

The glossary to the *Specification* then defines "thermally cut" as being "cut with gas, plasma or laser" (see page 16.1-liv of the 2010 AISC *Specification*).

So, assuming that the plasma equipment in question can produce holes of the necessary quality, it is permitted—and indeed plasma equipment is becoming extremely common due to the efficiencies they can provide.

You may also find Section M2.2 of the AISC *Specification* to be useful, as it discusses thermal cutting for purposes other than bolt holes (the Commentary to Chapter M is also useful in a general sense). The above applies to buildings and building-like structures.

Note: If you are working on bridges, then thermally cut holes may be prohibited by the owner.

Martin Anderson

Group A & B Bolts

The tables in the 14th Edition of the AISC *Steel Construction Manual* refer to Group A and Group B bolts. What is the definition of Group A and Group B bolts?

This terminology is pursuant to Section J3.1 of the 2010 AISC *Specification*, and the groups correspond to material strength.

Per J3.1, Group A is composed of those materials that have a tensile strength similar to ASTM A325, and includes ASTM A325/A325M, F1852, A354 Grade BC and A449. Group B is composed of those materials that have a tensile strength similar to ASTM A490, and is composed of ASTM A490/A490M, F2280 and A354 Grade BD.

Section J3.1 itself can be found on page 16.1-118, with some relevant Commentary on the matter starting on page 16.1-400.

This change was made to simplify references to those strength groups (for example, when discussing connections it is convenient to distinguish between Group A and Group B as they have different strengths; it similarly simplifies discussions of minimum bolt pretension).

Martin Anderson

Stability Design and the ELM

In AISC 360 Table C-C1.1 "Comparison of Basic Stability Requirements with Specific Provisions," in reference to the effective length method, it states, regarding basic requirements (3), (4) and (5): "All these effects are considered by using *KL* from a sidesway buckling analysis in the member strength." Can you explain how using *KL* addresses each of these items? In addition, on a current project, I noticed that the *K*-factor from an eigenvalue buckling analysis is almost equal to that given by the alignment charts. Does this mean that an eigenvalue analysis considers basic requirements (3), (4) and (5)?

First, the *K*-factor you get from an eigenvalue analysis is an elastic *K*-factor, so it does not account for any inelastic effects (4), it is not able to account for out-of-straightness (3) and it does not address uncertainty (5). Now, let me go through the six items that Table C-C1.1 says are addressed by using *KL* from a sidesway buckling analysis:

1) **Member imperfections on structure response.** You must do a second-order analysis in each method. This is the P-δ or "member effect" and its influence on the sway effect. Eigenvalue does not do this.
2) **Member imperfections on structure strength.** The column strength equations in AISC 360 Chapter E are based on initial out-of-straightness of the member, thus there is nothing more for the engineer to do in either method.
3) **Effect of stiffness reduction on response.** In determining the effective length factor you must take stiffness reduction into account. You can do this with the stiffness reduction factor when using the alignment chart. This has been in the AISC *Manual* for a very long time. Eigenvalue does not do this.
4) **Effect of stiffness reduction on strength.** In determining strength, inelastic buckling is already taken into account in the column strength equations in AISC 360 Chapter E. There is nothing more for the engineer to do.
5) **Effect of uncertainty on response.** This already is taken into consideration in the stiffness reduction factor for effective length. Eigenvalue does not do this.
6) **Effect of uncertainty on strength.** This is already accounted for the resistance or safety factors.

The fact that your eigenvalue solutions closely match the alignment chart is likely because your model matches the assumptions used in developing the chart. I find it hard to believe this is always the case for your structures as we almost always violate some of these assumptions—gravity-only columns, inelastic behavior, all columns buckling at same time, etc.

Heath Mitchell, S.E., P.E.
(with assistance from Louis F. Geschwindner, P.E., Ph.D.)

Special Inspection

I cannot seem to find the Special Inspection tables for structural steel in the 2012 International Building Code. Where are they located?

Those tables are no longer in the IBC. They are now located as chapters within the relevant AISC standards. For special inspection of structural steel other than seismic lateral force resisting systems, 2012 IBC Section 1705.2.1 states:

> "**1705.2.1 Structural steel.** Special inspection for structural steel shall be in accordance with the quality assurance inspection requirements of AISC 360."

You will find these special inspection (QA) requirements in AISC 360 Chapter N. For special inspection of seismic lateral force resisting systems, 2012 IBC Section 1705.11.1 states:

> "**1705.11.1 Structural steel.** *Special inspection* for structural steel shall be in accordance with the quality assurance requirements of AISC 341.

Exception: *Special inspections* of structural steel in structures assigned to *Seismic Design Category C* that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems."

You will find these special inspection requirements in AISC 341 Chapter J. These are in addition to the special inspection requirements in AISC 360-10 Chapter N.

All AISC specifications noted above are available as free downloads at **www.aisc.org/epubs**.

Heath Mitchell, S.E., P.E.

Capacity of Existing Welds

I am trying to determine the capacity of existing welds. Can I do this using NDT methods?

No. There are no nondestructive testing methods that can be used to determine the strength of the weld metal or the base metal. To determine the strength you generally have to break something.

NDT is used to determine the quality and geometric characteristics of welds. If the weld is a CJP groove weld, then ultrasonic testing or radiographic testing could be used to investigate the quality of the weld. These methods could also be used to determine if a groove weld is a PJP groove weld rather than a CJP groove weld. However, the quality of a PJP groove weld or fillet weld generally cannot be easily or accurately determined through these tests. The size of a fillet weld can be easily determined through visual inspection. Visual inspection can also reveal any issues at or near the surface of the weld.

Appendix 5 Section 5.2.5 of the AISC *Specification* (a free download at **www.aisc.org/2010spec**) states:

"Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of AWS D1.1/D1.1M are not met, the engineer of record shall determine if remedial actions are required."

The tests described are destructive tests. You must take "representative samples of weld metal" and physically test them.

Larry S. Muir, P.E.

Filling Weld Access Holes

If weld access holes are required to be filled, how should this be accomplished? Is filling them with weld metal appropriate?

In the June 2009 issue of *MSC* (**www.modernsteel.com**), the article "In the Moment" by Victor Shneur offers the following advice:

> "Do not fill weld access holes with weld material for cosmetic or corrosion-protection reasons. In addition to the cost, it creates undesirable triaxial stresses. Using mastic materials is preferable to welding."

Weld access holes exist not only to facilitate welding, but also to limit the "undesirable triaxial stresses," Shneur explains. The only pros to filling weld access holes are likely to be based in cosmetic or aesthetic reasons. The cons to filling them with weld metal include changes in the assumed stress distribution, increased cost and the cracking that weld access holes are used, in some applications, to prevent. Also, when filling by welding, unless done using a qualified procedure shown to develop the strength of the base metal, the resulting strength and behavior of the material within the filled hole may be dubious.

Larry S. Muir, P.E.

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

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