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

steel interchange

Note: Unless specifically noted, all AISC publications mentioned in the questions and/or answers are independent of the edition and can be found at www.aisc.org/specifications.

Field-Modified Base Plates

Due to misplaced anchor rods, the holes in a column base must be enlarged. The contractor is proposing to turn the round, oversized holes into oversized slots by thermally cutting the base plate. Is this acceptable? If so, what is the minimum edge distance for the enlarged hole?

Yes. Section M2.9 of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) addresses holes for anchor rods and indicates that they are permitted to be thermally cut in accordance with the provisions of Section M2.2. As indicated in Section M2.2, thermally cut edges shall meet the requirements in clauses 5.14.5.2, 5.14.8.3 and 5.14.8.4 of AWS D1.1.

The *Specification* contains no minimum edge distance requirements for base plate holes. AISC Design Guide 1: *Base Plate and Anchor Rod Design* (a free download for members at **www.aisc.org/dg**) states: "When the hole edge is not subject to a lateral force, even an edge distance that provides a clear dimension as small as ½ in. of material from the edge of the hole to the edge of the plate will normally suffice, though field issues with anchor rod placement may necessitate a larger dimension to allow some slotting of the base plate holes. When the hole edge is subject to a lateral force, the edge distance provided must be large enough for the necessary transfer."

Carlo Lini, PE

Erection Bracing

I am an erector and have a contract to erect a series of steel frames along three separate column lines. The three lines are not interconnected. None of the frames appear to contain lateral force-resisting elements in that there are no designated moment connections and no vertical bracing. The beams are also very deep, with long spans. The steel supports a floor that is part of a building otherwise constructed of concrete, and I suspect that the concrete slab and the shear walls provide stability.

I believe it will be difficult to erect these frames in a safe manner, and that even once erected the structure will be inherently unstable without whatever temporary bracing I provide. It now seems that my bracing will have to remain throughout the duration of construction and will also have to support loads due to the performance of work by other trades.

1. **Am I allowed to simply remove my bracing at the completion of my work?**
2. **If not, should I be compensated for the use of my bracing during the time it remains in place after erection?**
3. **Should my bracing be removed and returned to me when the structure is finally stable?**

I have addressed your three questions below:

1. No. From your description, it would not seem reasonable or safe to simply remove the bracing when your work is complete. However, it seems that some important elements of the contract may have been neglected.

   Section 3.1.4 of AISC's *Code of Standard Practice for Buildings and Bridges* (ANSI/AISC 303) states: "When the structural steel frame, in the completely erected and fully connected state, requires interaction with non-structural steel elements (see Section 2) for strength and/or stability, those non-structural steel elements shall be identified in the contract documents as required in Section 7.10."

   Section 7.10.4 states: "Temporary supports provided by the erector shall remain in place until the portion of the structural steel frame that they brace is complete and the lateral force-resisting system and connecting diaphragm elements identified by the owner's designated representative for design in accordance with Section 7.10.1 are installed." You have stated that you suspect the concrete slab and the shear walls provide stability. Based on the uncertainty, I will assume that the contract documents are silent relative to the lateral force-resisting system. They should not be.

   Section 7.10.1 requires the engineer to identify "the lateral force-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure."

   Since no lateral force-resisting system or connecting diaphragm elements are identified, you will need to request information from the owner's representatives. The information provided will hopefully clarify the engineer's intent.
2. Your question reflects a contractual issue and I cannot arbitrate. The parties will have to find a way to resolve the issue. However, I will provide some thoughts.

   Though seemingly not clear in the contract documents, common sense dictates that you cannot simply remove your bracing when the bare steel is erected. However, leaving your bracing until the other trades complete their work is not the only option. Others could provide temporary bracing necessary to safely complete the project, leaving you free to remove your bracing and be done with your portion of the project.

It may be that leaving your bracing in place will be the best option for the project. However, Section 7.10.2 states: "The owner's designated representative for construction shall indicate to the erector, prior to bidding, the installation schedule for non-structural steel elements of the lateral force-resisting system and connecting diaphragm elements identified by the owner's designated representative for design in the contract documents."

Not only should the non-structural steel elements of the lateral force-resisting system have been identified, but you also should have been provided with a schedule that would have indicated how long your bracing would likely be required after the completion of your work.

Providing this information after award is likely a revision to the contract. Such revisions are addressed in Section 9.3.

You also mention loads produced by the work of other trades. Section 7.10.3 states: "The erector need not consider loads during erection that result from the performance of work by, or the acts of, others, except as specifically identified by the owner's designated representatives for design and construction…" Again, a request for information would seem to be in order.

3. Yes. Section 7.10.4 states: "Temporary supports that are required to be left in place after the completion of structural steel erection shall be removed when no longer needed by the owner's designated representative for construction and returned to the erector in good condition."

Larry S. Muir, PE

Shear on Round HSS and their Welds

Section G5 of the *Specification* limits the effective area of a round hollow structural section (HSS) subjected to shear to half of the gross area. However, when evaluating welded connections, many textbooks and handbooks indicate that the entire circumference is the length of the weld. Can you explain this discrepancy?

The welds should be designed based on the stiffness of the connected element. For solid round bars, 100% of the weld length can be used. However, because thin-walled circular structures such as stacks have negligible strength and stiffness perpendicular to the wall, weld components perpendicular to the wall are ineffective. In this case, only about 50% of the weld length is effective. The stiffness of round HSS is between these two extremes; therefore, 50% of the weld length can be used as a conservative approximation. Ultimately, you must use your own judgment to determine what is appropriate for your situation.

Bo Dowswell, PE, PhD

Weld Access Holes in Seismic Base Plates

Per Sections E3.6a and F2.6a of the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) welds at column-to-base plate connections are demand-critical, though some exceptions can apply. Does this mean that the weld access hole geometry needs to conform to the alternate geometry of AWS D1.8 Clause 6.10.1.2? What are the impacts, if any, on structural performance of demand-critical welds at the column bases if we allow either weld access hole geometry?

AWS D1.8 provides welding requirements for demand-critical welds.

Clause 6.10.1.1 of AWS D1.8 permits weld access holes meeting the dimensions and tolerances of AWS D1.1 or the AISC *Specification*. At the option of the contractor, the geometry specified in AWS D1.8 clause 6.10.1.2 may be substituted for the clause 6.10.1.1 geometry.

The Commentary to Section 8.5 of AISC's *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358) speaks to your second question on structural performance related to differing weld access hole geometries. It states: "A key feature of the WUF-W moment connection is the use of a special weld access hole. The special seismic weld access hole has specific requirements on the size, shape and finish of the access hole. This special access hole… is intended to reduce stress concentrations introduced by the presence of the weld access hole." It should be noted that the inelastic demand at a column base will likely be much lower than that at a WUF-W moment connection. The alternate geometry is also not required for reduced beam section moment connections. Though there are benefits to the alternate geometry, it should only be required where these benefits are likely to be realized. The only conditions for which it is required are the WUF-W connections and the prescriptive OMF (ordinary moment frame) moment connection described in Section E1.6b(c) of the *Seismic Provisions*.

Jonathan Tavarez

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

Larry Muir is director of technical assistance, Carlo Lini is a senior staff engineer and Johnathan Tavarez is a staff engineer in the Steel Solutions Center, all with AISC. Bo Dowswell is a consultant to AISC.