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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.

Bolt Head and Nut Angles

Must bolt heads and nuts be oriented at the same angle, as illustrated in Figure a? Or is the arrangement illustrated in Figure b acceptable?

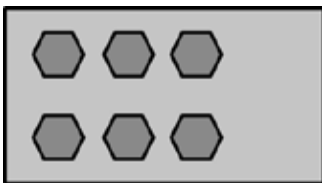


Figure a

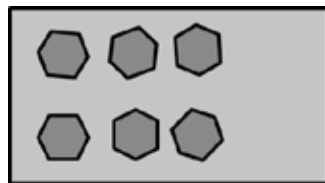


Figure b

Generally, there is no requirement related to the orientation of bolt heads and nuts. The bolts can be entered from either side, and all bolts in a joint need not be placed from the same side. Bolt heads can be placed on either or both sides of the joint.

However, when addressing fabrication and erection requirements for architecturally exposed structural steel (AESS), Sections 10.4.1 and 10.6 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303) state: "(f) All bolt heads in connections shall be on the same side, as specified, and consistent from one connection to another." These requirements address only the location of the bolt head, not the rotation of the head or the nut.

Section 10.2 of the *Code* states: "The following additional information shall be provided in the contract documents when AESS is specified... (e) Any other special requirements for AESS members and components, such as the orientation of HSS weld seams and bolt heads." Section 1.1 of the *Code* also states: "In the absence of specific instructions to the contrary in the contract documents, the trade practices that are defined in this *Code* shall govern the fabrication and erection of structural steel."

It would be at least theoretically possible for the contract documents to require that all the bolt heads and nuts be installed at the same angle. However, this requirement would have to be clearly stated in the contract documents, and it could add considerably to the cost of the fabrication and erection. In some instances, it might also be impractical to try to align the heads and the nuts.

Bolts must be installed at least snug-tight. This means the plies must be in firm contact. Especially for thicker plies, considerable effort may be required to pull the plies into firm contact. Given the different levels of stiffness and gaps that might exist throughout the joint, it is unlikely that the same rotation would need to be applied at each bolt to provide firm contact. For a

snug-tight condition, it might be possible to apply additional rotation to end up with a condition where all of the nuts and heads are at the same angle. However, in extreme cases, the possibility exists that the additional rotation could exceed the capacity of the installation tools or could even lead to bolt fracture.

For joints that must be pretensioned, the process becomes even more complicated. The bolts must first be installed to a snug-tight condition, so all of the issues above still apply. However, for pretensioned joints and slip-critical joints, the bolts will already be installed to at least 70% of their nominal tensile strength. Additional rotation in the tightening direction is even more likely to result in bolt fracture. Rotation in the loosening direction that causes a reduction in the required tension is absolutely unacceptable.

If an attempt is made to install bolt heads and nuts at the same angle, then it is advisable to recognize that bolts may break and to develop procedures to address the situation at the outset of the project.

Larry S. Muir, PE

Bolting Inspections

I am told that the quality assurance inspector does not need to be present during bolt installation on my project. Is this assertion correct?

Yes. Section N6.(a) of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) states: "The QCI and QAI need not be present during the installation of fasteners in snug-tight joints." Section N6.(b) states: "For pretensioned joints and slip-critical joints, when the installer is using the turn-of-nut method with matchmarking techniques, the direct-tension-indicator method or the twist-off-type tension control bolt method, monitoring of bolt pretensioning procedures shall be as specified in Table N5.6-2. The QCI and QAI need not be present during the installation of fasteners when these methods are used by the installer."

The inspector must perform the tasks listed in Chapter N. In order to adequately perform some of the tasks, the inspector must be present to adequately observe the tasks. There are some operations where the inspector need not be present during the operation because the observations associated with tasks can be adequately performed before or after the actual operation.

In the case of bolting for pretensioned joints and slip-critical joints, a pre-installation verification is performed before work starts. This will ensure that the crews understand the installation procedure and know how to execute it properly. The turn-of-nut method with matchmarking techniques, the direct-tension-indicator method or the twist-off-type tension control bolt method all leave evidence that the method has been properly performed for each joint. Therefore, the *Specification* does not require the inspector to be present during the actual installation.

Larry S. Muir, PE



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Nondestructive Testing of Weld Access Holes

Section N5.5c of the 2010 *Specification* states: "Thermally cut surfaces of access holes shall be tested by QA using MT or PT when the flange thickness exceeds 2 in. (50 mm) for rolled shapes or when the web thickness exceeds 2 in. (50 mm) for built-up shapes. Any crack shall be deemed unacceptable regardless of size or location." A similar requirement does not appear in the 2016 *Specification*. Can you provide some background for the change?

Yes. This requirement was removed in the 2016 *Specification* based on reports that the required nondestructive testing (NDT) did not reveal cracks in weld access holes in practice. Either the cracks are simply not present or if they are, then the grinding required in Section M2.2 is sufficient to remove them. Therefore, it was decided that there was no reason to continue a practice that apparently served no purpose.

Carlo Lini, PE

Overstrength and Headed Stud Anchors

There is a discussion about collector elements and their connections on page 8-14 of the 2nd Edition AISC *Seismic Design Manual*. My interpretation of this discussion is that if steel headed-stud anchors are used to transfer horizontal shear to the steel wide-flange beams, then the connection must be designed for the amplified seismic load (Ω_o level loads). This is not common practice in my experience. Is my interpretation correct?

Your interpretation of what is done in the example is correct. The example reflects the interpretation of requirements in other codes (ASCE 7) by the AISC Committee on Manuals. If you have questions relative to the intent of other codes, you will need to contact the organization that produces the code in question—in this case, ASCE. I will provide some additional thoughts.

The Commentary to Section B5.1 of the *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) states: "For recommendations on the design of diaphragms, see Sabelli et al. (2011)." The NEHRP document referenced is freely available online (tinyurl.com/nehpsdtb5) and provides some further insight into the various approaches that can be used and further considerations. It states: "To complete the diaphragm analysis, forces on individual components must be determined. The unit shear in the deck and chord, and the collector forces, must be calculated so that those components may be designed. The deck shear may be uniform or non-uniform; chord and collector forces may be considered to be concentrated or distributed. There is relatively little guidance in design standards and other publications for the determination or selection of appropriate distributions of shear forces along chords and collectors. At a minimum, the forces calculated in the chords and collectors should be consistent with the assumed shear distribution, as discussed below. In the absence of a rigorous analysis that includes both the nonlinear diaphragm properties and the nonlinear behavior of the system (as well as the full range of possible ground-motion characteristics) the design in effect relies on some limited ductility in the diaphragm to permit redistribution of forces to account for the simplifications in the assumed distribution."

Though not directly stating what is stated in the *Seismic Design Manual*, it seems to lead to the same conclusion: We are making a lot of assumptions, we are not performing rigorous analysis, there may be limited ductility in the diaphragm and there is relatively little guidance provided—so at a minimum, the forces calculated in the chords and collectors should be consistent with the assumed shear distribution.

The May 2018 SteelWise article "Developing Diaphragm Analysis," available at www.modernsteel.com, provides similar guidance and states: "Elements like shear studs and puddle weld deck attachments to SFRS members may even be treated as collectors and designed for the required collector forces per ASCE 7-10. Such a distinction enables an assurance that the forces get to the system as designed, keeping the expected inelastic behavior of the SFRS contained within the vertical resisting elements."

Larry S. Muir, PE