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!

HSS S T a c

T O 2006 S I
AISC

What is the recommended width tolerance of a slot in a tube shape that is to receive a plate? $^1/_8$ in. larger? $^1/_{16}$ in. larger?

Q AISC's S S C

The answer depends on several factors. The first concern is the fabrication tolerance on the cut. Normally, the shop realizes that the slot width is a "keep" dimension, so the thermal cut is outside the line, and the kerf will tend to increase the slot width. Thermal cutting, however, tends to distort the material and can cause the slot to close slightly. This may require some heat spots to restore the slot to a parallel condition. Both the layout of the slots at each end and the orientation of the gusset plates must be in the same plane, or additional clearances will be required, because the HSS is very stiff in torsion. There can be some overrun in plate thickness, but this is seldom a concern.

Our company's practice is to detail the slot $^1/_{16}$ in. wider than the plate. On very long slots in heavy HSS we may increase the slot to $^1/_8$ in. over the plate thickness. Our fabricated dimension is usually slightly wider than the detailed dimension. We always check the slot using a plate of the proper width. This has worked well for typical HSS braces.

The question of weld sizing was addressed in the previous reply. Some engineers oversize the weld to compensate for possible gaps. This is not necessary if AWS D1.1 fillet weld requirements are followed in the field where gaps larger than $^1/_{16}$ in. automatically require an appropriate increase in leg size.

L K , P.E.
L J S C

H -S B 1956

I am analyzing a building constructed in 1956. The plans specify high-strength bolts for the lift-slab columns, but don't give an ASTM designation. Do the A325 and A490 designations go back to 1956? Do you have any other suggestions for approximating design strength of these bolts without testing?

Q AISC's S S C

The answer to the first question could be yes or no. The actual ASTM A325 Standard was in the tentative review process as early as 1949, but was not officially approved as a consensus standard until 1964.

High-strength bolts were beginning to be used in lieu of rivets in the 1950s, but may not have carried the ASTM A325 or A490 designation. The AISC specification at the time included design parameters for "turned bolts," as well as for "unfinished bolts." There was no distinction as to whether the threads were located in the shear plane or not. Allowable loads were listed in the AISC manual of the time.

Allowable working loads for ASTM A325 bolts were first listed in the 1961 AISC manual—making the distinction as to the installed thread location with regards to the shear plane. At that time, ASTM A325 bolts with threads included in the shear plane were designed based on an allowable working shear stress of 15 ksi (single shear), the same as permitted for turned bolts. If the threads were installed excluded from the shear plane in a bearing type connection, the allowable working shear stress was 22 ksi (single shear). One benefit of testing might be the use of more modern design values if the fasteners meet the more current requirements.

There is also a historical discussion on bolts in the *G D C B R J*, available as a free download at **www.boltcouncil.org**. A soft-cover copy can be purchased at **www.aisc.org/bookstore**.

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