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Width-Thickness Limits for S-Shape with Cap-Channel

I am designing a monorail beam, which is an S-shape with a cap channel. I'm having trouble determining the limiting width-thickness ratios for strong-axis bending per Table B4.1 of the AISC 13th Edition *Manual*. For strong-axis bending I am checking three components:

- **channel web between two fillet welds per Table B4.1 Case 12**
- **channel web between channel top flange and fillet weld as stiffened elements per Table B4.1 Case 12**
- **S-shape beam flange as unstiffened element per Table B4.1 Case 2**

Am I doing this correctly?

From your description, it is assumed that your beam has the top flange in compression and that the cap channel is connected to the top flange.

For the first two checks, you are correct that the channel web between the two fillets and between the fillet and channel flange should be checked using Table B4.1 Case 12. This situation is similar to "flange cover plates between lines of fasteners or welds."

For the last check, it is conservative to check the S-shape flanges as unstiffened elements, per Case 2. However, it is justified to consider the S-shape flange to be a stiffened element and check it per Case 12 with "*b*" equal to half the flange width.

There is another element to check: the S-shape web, although it will be compact for standard North American S-shapes with F_y not exceeding 65 ksi. Case 11 is the correct case for the S-shape web, because the presence of the cap channel moves the elastic and plastic neutral axes toward the top of the section as shown in the figure for Table B4.1 Case 11. Please note that if the crane beam is subjected to axial "tractive" forces, uniform compression width-to-thickness ratios should be checked as well.

Brad Davis, Ph.D., S.E.

Column Buckling

I am reviewing an existing built-up column. The section is singly symmetric (symmetrical about the weak axis). The column is subject to combined axial force and flexure about the strong axis. Does the web element for uniform loading fall under Table B4.1, Case 14 of the 2005 AISC *Specification*? While checking the limit states of flexural-torsional and torsional buckling, I am using Equation E4-5 for singly symmetric members. Is this the correct equation when the axis of symmetry is the weak axis?

Yes, either Case 10 or Case 14 in Table B4.1 works for this case. I am assuming that the shape is not tapered.

When checking for axial strength, either flexural bucking (E3) or flexural torsional buckling (E4) can control the design, depending on the bracing details. In those cases, buckling about the weak axis typically controls. If the web is slender as per Table B4.1, the provisions of Section E7 must be applied.

After checking the flexural strength about the strong axis as per Chapter F, the interaction equations in Chapter H then can be used to determine the strength of the member for the combined effects.

Amanuel Gebremeskel, P.E.

Countersunk Bolts

I am trying to find the preferred material specification for countersunk high-strength bolts. Building codes are virtually silent on the subject of countersunk bolts for structural applications, yet there are occasions where, because of interference, a regular hex-head A325 or A490 bolt will not work and a countersunk bolt is needed. Is this addressed anywhere in the AISC *Steel Construction Manual*?

The AISC *Specification* does not address the use of countersunk bolts. These are ASTM A307 (or similar soft material) bolts and used only in bearing connections. These are not generally used as primary structural connections. There is a short discussion in Part 7 of the 13th Edition AISC *Steel Construction Manual* pertaining to checking the available bearing strength at such bolt holes.

Kurt Gustafson, S.E., P.E.

Prequalified and Qualified High-Seismic Moment Connections

Table 2-2 of FEMA 350 allows bolted flange plate (BFP) moment connections as prequalified moment connections for OMF and SMF in high-seismic applications. ANSI/AISC 358 makes no mention of this type of connection. Is the use of BFP moment connections still permissible in high-seismic applications?

The AISC *Seismic Provisions* (ANSI/AISC 341-05) does not limit the special moment frame connection types to those shown in ANSI/AISC 358-05. As covered in Section 9.2b of AISC 341-05, using ANSI/AISC 358-05 is one of three methods permitted to provide conformance demonstration. The three options are: use of a connection that is prequalified, like those in ANSI/AISC 358; use of a connection that is qualified based upon available test results; and use of a connection that is qualified based upon project-specific testing.

Although not all FEMA 350 connections have yet been adopted into AISC 358, more types of moment connections are being added, as the necessary testing and review is performed. It often is possible to use the testing behind the connections that are included in FEMA 350 to justify their use. The Commentary to Section 9.2b of the AISC *Seismic Provisions* discusses the published testing, such as that conducted as part of the SAC project, and reported in FEMA 350 and 355 may be used to satisfy this provision.

Kurt Gustafson, S.E., P.E.

HSS Seismic Connections

Prequalified seismic moment connections only include W-shape beams. Can HSS beams be used for IMF? How can the seismic requirements for this type of connection be met?

The prequalified connections do not provide for the use of HSS beams. In order for these to be used, the connections must be qualified in accordance with Appendix S of the AISC *Seismic Provisions*. Alternatively, an OMF can be used.

Larry S. Muir, P.E.

steel interchange

Design Using the 2005 *Specification*

I have been using the ASD 9th Edition *Manual*. I am trying to learn how to use the 13th Edition. I am having a hard time finding the allowable stresses for different members, such as tension members, compression members, and members in flexure just to name a few. Is the bending stress for flexural members still $0.66F_y$ and $0.6F_y$, depending on my unbraced length? Where are these located?

The 2005 AISC *Specification* is based on a strength format rather than stress, but strength equations can always be formatted as stress by dividing out the appropriate section property. While many of the limit states are similar to those used in the old ASD specifications, there may be slight variations. You will find the nominal limit state capacities for tension in Chapter D, for compression in Chapter E, for flexure in Chapter F, and so on.

For flexure, a compact shape is handled somewhat differently in the 2005 *Specification* than in the older ASD specifications. It is now permitted to use the actual shape factor for the section—instead of the lower bound shape factor of 1.1 for W-shapes, which was incorporated in the older ASD specification provisions:

$$M_n = F_y Z_x$$

Using ASD: $\Omega = 1.67$, therefore $M_n / \Omega = 0.60F_y Z_x$

The shape factor = Z_x / S_x, which ranges from 1.1 to 1.3 for W-shapes. If one uses the shape factor = 1.1 as assumed in the old ASD specifications:

$$M_n / \Omega = 0.60F_y Z_x = 0.60F_y (1.1S_x) = 0.66F_y S_x$$

Most other cases are more straightforward in that they do not require mathematical manipulation to compare the new to old. For example, tension yielding has $F_y/\Omega = 0.6F_y$.

Do these look familiar?

Kurt Gustafson, S.E., P.E.

Rivet Head/Shaft Diameter Relationship

We are doing a project involving inspections of truss bridges, most of which were built in the early 1900s and are connected together with gusset plates attached with rivets. We have not been able to locate any literature relating the diameter of the head of the rivets to the shaft diameter. Is there any reference material that denotes the relationship of the diameter of the head to the actual shaft diameter?

There was a general relationship for driven rivet heads as a function of the diameter of the rivet published in the Fifth Edition AISC *Steel Construction Manual:*

Diameter of Head = 1.5 × Diameter of Rivet + ⅛ in.

There was also a general published relationship for manufactured heads as a function of the diameter of the rivet of:

Diameter of Head = 1.5 × Diameter of Rivet + $\frac{1}{32}$ in.

I am not sure if this applied to both hand- and power-driven rivets, but I would surmise that it was fairly standard. The manufactured head equation was published in the manuals of the 1950s but not in earlier ones of the 1920s. Therefore, there could have been a change in this standardization depending on the era of the rivets. You may want to sample a few rivets to see if this relationship is accurate for the specific project.

Kurt Gustafson, S.E., P.E.

Single-Plate Shear Connections to HSS

Is there, or will there be, an update of the 1997 *Hollow Structural Sections Connections Manual*? I am particularly interested in finding information pertaining to single-plate shear connections to HSS.

There are no plans to develop another HSS connections manual, but there is an HSS connections design guide that is soon to be printed (see SteelWise on page 53 for more on this design guide). The HSS Connections Manual was based on the stand-alone AISC *LRFD HSS Specification*. Much of that information has now been included in the 2005 AISC *Specification*, with Chapter K covering HSS connections. See the User Note to Equation (K1-10) in the 2005 *Specification* for discussion of the yielding (punching) check on the wall of the HSS tube. Also, there is information on shear connections to HSS beginning on page 10-156 in the 13th Edition AISC *Steel Construction Manual*.

Kurt Gustafson, S.E., P.E.

Shear Lag

Could you please explain the term "shear lag?"

Shear lag is the phenomenon discussed in Section D3.3 of the 2005 AISC *Specification* (a free download at **www.aisc.org/2005spec**). The bottom figure on page 16.1-252 of the *Specification* Commentary provides a good example. Using that example, away from the connection, the stress is uniform across the entire angle. However, because the horizontal leg is bolted to the support, and the vertical leg is not, the total load must transition to being only in the horizontal leg and transferred to the support along the length of the connection. With enough distance to accomplish this transition, the tension rupture strength will not interrupt this transition. But the shorter the connection is, the more abrupt the transition is. The effective net area concept is how this phenomenon is addressed, and this is accounted for by using the *U*-factor of Section D3.3.

Brad Davis, Ph.D., S.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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