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Single-Angle Connection Tables

I do not understand the reason behind one of the notes for Table 10-11 in the 13th edition *Manual*. I'm unclear why a "smaller half web will result in these values being conservative."

As shown on page 10-123 for this case, the eccentricity is considered on the leg attached to the supporting member. The eccentricity is measured from the center of the web of the supported member to the center of the bolt or weld group. The values in the tables assume a ½-in. web (or ¼-in. half web) thickness dimension. The strength of the bolts (or weld for that matter) is calculated using the instantaneous center of rotation method discussed in Parts 7 and 8 of the *Manual*. If the web thickness is less than ½ in., then the assumed eccentricity will be larger than the actual eccentricity, and the strength given in the table will be conservative. If the web thickness is greater than ½ in., then the assumed eccentricity will be less than the actual eccentricity, and the strength given in the table will be unconservative and must be either reduced by the rule of thumb given in the footnote or recalculated.

Larry S. Muir, P.E.

Referenced Design Standards

I have several design spreadsheets that are written around the ASD 9th edition *Manual*, and I was wondering if any of the new IBC or ASCE codes reference a certain manual, like the new 13th edition, or if the older edition is still acceptable to use for design purposes.

The IBC and ASCE 7 both reference a particular release of the AISC *Specification*; not an edition of the AISC *Steel Construction Manual*. For example, the 2006 IBC references AISC 360-05, which is the 2005 AISC *Specification*. The 13th edition AISC *Steel Construction Manual* is based upon the provisions of the 2005 AISC *Specification*, but is not the document referenced in the building code.

AISC recommends that the latest revision of specifications, manuals, and other documents should be used. However, AISC does not define which edition of the *Specification* must be followed in the execution of a project. Rather, that is stipulated by the applicable building code. That stipulation defines what is acceptable.

Most jurisdictions now use the IBC, and this means that the 2005 AISC *Specification* is probably the referenced standard. If the applicable building code does require use of an outdated specification, we recommend a discussion with the building official to see if they will accept the latest specification for use on the project.

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

Brace Stiffness

I have been questioned about calculations for a stability bracing member per AISC 360-05 Appendix 6, Equations A-6-7 and A-6-8. I can calculate the required brace stiffness, but how do I calculate the actual brace stiffness provided?

The required brace stiffness from Equation A-6-8 in the AISC *Specification* represents the required axial stiffness of the brace. The actual brace stiffness provided can be calculated using the relationship $\Delta = PL/AE$.

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

ASTM F1554 versus ASTM A449 Anchor Rods

I am trying to better understand when to specify F1554 vs. A449 for anchor rods. Table 2-5 in the *Manual* does not indicate a preferred material specification for high-strength anchor rods. Is there a reason for this? Is there a preferred material for anchor rods?

ASTM F1554 and ASTM A449 refer to specific material types that meet specific ASTM Standards. Both of these material types are permitted for use as anchor rods under the auspices of the AISC *Specification*. Table 2-5 in the 13th edition *Manual* shows ASTM F1554 Grade 36 as the usual grade for the general case. If you are specifically going to use a high-strength anchor rod, ASTM F1554 is the preferred type, since this is a standard specifically developed for anchor rods.

The ASTM F1554 anchor rods are available in three grades of 36, 55, and 105 ksi minimum yield stress material, and are available in specified lengths, with threading lengths as specified. ASTM A449 is a general material standard that is applicable to other applications of bolts, screws, and studs, as well as for anchor rods. ASTM A449 does not have stipulated minimum yield strength; however, the material exhibits tensile strengths similar to some of the ASTM F1554 grades. Since the nominal tensile stress listed in Table J3.2 is determined based on the F_u of the material, the EOR is able to assess the tensile capacity of the ASTM A449 rods.

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

Calculating C_b

I'm wondering how to analyze a W-shape beam at an inflection point in terms of the C_b value. I understand that the inflection point cannot be considered a braced point, so when you calculate the C_b value some moment values will be negative and some moment values will be positive. The *Specification* says that C_b is permitted to conservatively be taken as 1.0. Is this a requirement? If not, then which one is true in order to get a C_b value?

Case A:

Use the absolute values for M_a, M_b, M_c and M_{max} across the entire unbraced section. (This would include positive and negative moments.)

Case B:

Use the absolute values for M_a, M_b, M_c and M_{max} from one brace point to the inflection point. (This includes only negative or positive moments.)

It is not mandatory to use a C_b of 1 in all cases. Rather, the *Specification* states that is a conservative assumption that can be used in all cases if you choose. The C_b factor can be used to permit adjustments in cases where the moment diagram is non-uniform.

Your Case A describes the stated procedure, which is applicable to the unbraced segment from braced point to braced point.

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

steel interchange

The Richards Factor

Design examples 3.10 and 3.11 in the *Seismic Design Manual*, take the stress for the welds as the greater of f_{peak} or $1.25f_{avg}$. Is this design practice specified somewhere in the *Manual* or some other publication?

The 1.25 adjustment factor accounts for the potential for uneven distribution of stress in a welded gusset plate edge connection that may have a local hot spot due to the proximity of the brace-to-gusset connection. It is called the Richards Factor and represents a measure to provide for ductility in the case of a uniform (or nearly uniform) distribution on the weld.

There was a paper that appeared in the First Quarter 2004 AISC *Engineering Journal* titled "Rationale Behind and Proper Application of the Ductility Factor for Bracing Connections." AISC *Engineering Journal* articles are available for free to AISC members at **www.aisc.org/epubs**.

Note that the Richards Factor is discussed in the AISC *Manual* as a part of the design procedure recommended for bracing connections. It is not a requirement stipulated in the AISC *Specification*.

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

C_b for HSS Beams?

Are C_b values permitted in the design of HSS beams? Are C_b values greater than 2.3 permitted, in any case, in ASD? Is there an instance where C_b = 4.7 for an unbraced square HSS cantilever with a concentrated load at the end is justifiable?

The amplification of beam strength by C_b cannot result in a value that is larger than the full yield strength of the member ($F_y Z$)—that is, C_b can only be applied to the lateral-torsional buckling portions of the beam curve up to the value of full yield of the section. This is demonstrated graphically on page 3-4 of the 13th edition AISC *Manual*. Since HSS beams are not subjected to lateral-torsional bucking, and are always controlled by the yield or local buckling strength of the member, C_b does not apply.

Speaking more generally, the upper limit on C_b is 3.0, as given by formula F1-1 of the 2005 *Specification*. So yes, a value greater than 2.3 is permitted. However, there is no case where C_b = 4.7 can be used.

Chris Hewitt, S.E.

Shear Connectors Used in Multi-Story Construction

I have heard that rigid-frame beams in multi-story construction should not have welded studs applied to make them composite beams. Can you please give me a reference where this is stated?

I am not aware of any document that prohibits the use of shear connectors to provide composite action for moment-frame beams. OSHA Section 1926.754 (c) (1) (i), which deals with tripping hazards during erection, restricts the use of shop-applied studs in such cases. However, field-applied shear connectors are used very commonly in all types of structural steel construction to provide composite beam construction. Perhaps you heard that AISC recommends that the use of camber should be avoided in moment frame beams?

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

Double-Angle Connection Capacity

Table 10-1 Page 10-23 of the *Steel Construction Manual* for All-Bolted Double-Angle Connections lists the available ASD capacity at 32.6 kips for ¼-in.-thick A325N bolts. However, my calculation for angle shear rupture shows a value of 33.7 kips. If the hole diameter is changed from 13⁄16 in. to ⅞ in., the value is 32.6 kips, per your table. Is a ⅞-in. hole size assumed for a ¾-in.-diameter bolt, and if so, where is this stated? Otherwise, how is the 32.6 kip value obtained?

Yes, an extra 1⁄16 in. is added. Bolted connection limit state checks for net area that involve tension or shear both require the bolt hole size + 1⁄16 in. be deducted. See Section B3.13.b of the 2005 AISC *Specification* for this requirement. This same requirement was given in the 1989 ASD *Specification* as well.

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

Basic Design Value Cards

AISC "Basic Design Values 1" (laminate card, copyright 2005) shows bending about weak axis = $0.9F_yS_y$ (ASD). Please verify the coefficient is 0.9 and not 0.75.

Yes, 0.9 is the correct coefficient. When using the 2005 AISC *Specification*, Section F6.1 provides the basis for weak-axis bending strength. Equation F6-1 provides the flexural strength for a compact cross-section as $F_yZ_y/\Omega \leq 1.6F_yS_y/\Omega$. We know the shape factor will be at least 1.5 for a W-shape in weak-axis bending but not greater than 1.6. Thus, the flexural strength can be stated as at least $1.5F_yS_y/\Omega$. With $\Omega = 5/3$, it thus is permitted to use $0.9F_yS_y$ for ASD design, as long as all elements are compact.

Amanuel Gebremeskel, P.E.

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