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Erection Plan and Safety Plan

Is it OK to combine our site-specific erection plan with our site-specific safety plan?

I assume that you are asking from the perspective of what is required for AISC Certification and if that is so, AISC Certification guidelines do not require that the two plans be separated, nor do they prohibit the combining of the two. One word of caution is that the general contractor and/or contract documents for the project may require separate plan documentation, as they are likely sent to different sources for review. My advice is to check your contract documents and comply with any requirements that may exist there; naturally, these requirements would take precedence.

Keith Landwehr

C_b Factor

I am comparing the 2010 AISC *Specification* to the 1989 ASD *Specification*. Is it correct that a uniformly loaded simply supported beam will have a C_b=1.14 and that a simply supported beam with concentrated load at center will have C_b=1.32 when using AISC 360 Equation F1.1? It seems like the equation found in the 1989 ASD *Specification* would result in C_b=1.75 for each case. Why is there a difference between the two specifications?

Yes, for a simply supported beam unbraced except at the ends, C_b=1.14 or 1.32 for uniform load or midspan point load, respectively, per AISC 360 Equation F1-1.

However, C_b is not equal to 1.75 using the 1989 ASD *Specification* for these examples. The following statement appears in the paragraph defining C_b:

> "When the bending moment at any point within an unbraced length is larger than that at both ends of this length, the value of C_b shall be taken as unity."

Moreover, it is important to remember that the 1989 ASD C_b equation only applied to unbraced segments with linearly varying moment diagrams. Thus, that equation is not applicable to your example.

The current C_b equation was first used in the 1993 LRFD *Specification*, although it dates back to Kirby and Nethercot's 1979 publication *Design for Structural Stability*. It applies to a much wider variety of situations, including your example. Other good discussions of the C_b factor include Zoruba and Dekker's *Engineering Journal* article "A Historical and Technical Overview of the C_b Coefficient in the AISC Specifications" (3rd Quarter 2005) and R.D. Ziemian's (ed.) 2010 publication *Guide to Stability Design Criteria for Metal Structures*, 6th Ed.

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

Single-Angle Design

I am designing a single angle for axial compression following AISC 360 Section E5. The slenderness ratio calculated according to Equation E5-2 for this particular angle results in a value larger than 200. The equation has a limit equal to or less than 200. Does this mean that I consider the member slenderness to be 200, or does this mean that E5 is no longer applicable for this member?

If KL/r > 200, Equation E5-2 does not apply and Section E3 or E7 applies instead. In these latter sections, $KL/r \leq$ 200 is a guideline, not a *Specification* requirement. See the Commentary to Section E2. In cases where Section E5 does not apply and the angle is connected such that eccentricity exists, the effects of combined axial force and moment must be evaluated using Chapter H.

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

Connection Eccentricity

The single-plate shear connection design procedure in the 13th Edition AISC *Manual* allows eccentricity to be neglected when "a" is less than 3½ in. Do we need to check for eccentricity in a welded connection with a similar configuration?

Yes, connection eccentricity would need to be included in the design of all-welded connections. The 13th Edition AISC *Manual* design procedure allows eccentricity to be neglected when designing conventional shear tabs—not because we believe no moment exists, but rather because the design based on neglecting the eccentricity correlates better with test results.

There is no similar data to point to in the design of welded connections, so the eccentricity should be considered for the welded connections.

I will also mention that one reason that neglecting the eccentricity on a shear tab produces good correlation with the tests is because the bolt shear strengths listed in the AISC *Specification* are reduced from the actual shear strength of a single bolt to account for the uneven distribution that occurs in end-loaded connections, such as lap splices. There is no end-loading effect for simple shear connections. You may have noticed that the bolt values have increased somewhat from the 2005 *Specification* to the 2010 *Specification*. This is because we have reduced the reduction taken for end-loading (and also reduce the length at which a subsequent drop is required). Since the bolt values have increased, some eccentricity is now considered in the design of conventional shear tabs in the 14th Edition of the *Manual*.

Larry S. Muir, P.E.

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Seismic Moment Frame Continuity Plates

The contact documents direct us to size all continuity plates in seismic moment frames as the minimum thickness required by AISC 341. It is my understanding that continuity plates in a one-sided moment connection must be a minimum of half of the flange thickness of the moment-connected beam. At a two-sided condition the continuity plates must be no less than the largest flange thickness of the two moment-connected beams. Is my understanding correct?

It depends on the type of moment frame. The thickness requirements you mention are specific to special moment frame (SMF) and intermediate moment frame (IMF) connections, as well as the prescriptive option for ordinary moment frame (OMF) connections. These requirements can be found in AISC 341-10 Sections E2.6f, E3.6f and E1.6b(c), respectively. For other applications, including non-prescriptive OMF options in AISC 341 Sections E1.6b(1) and (2), those limits do not apply; rather, the thickness requirements in AISC 360 Section J10 apply.

Heath Mitchell, S.E., P.E.

HSS Workable Flat

What is the definition of the "workable flat" dimension given in the HSS tables in the 14th Edition AISC *Steel Construction Manual*?

The workable flat dimension is defined in Part 1 of the *Manual* beginning on page 1-5. It states:

> "In the tabulated workable flat dimensions of rectangular (and square) HSS, the outside corner radii are taken as $2.25t_{nom}$. The term *workable flat* refers to a reasonable flat width or depth of material for use in making connections to HSS. The workable flat dimension is provided as a reflection of current industry practice, although the tolerances of ASTM A500 allow a greater maximum corner radius of $3t_{nom}$."

Erin Criste

Bolt Hole Size Tolerance

Is there a size tolerance with which bolt holes must comply?

Yes, there is a tolerance on the size of bolt holes. Table 3.1 of the RCSC *Specification* (a free download from **www.boltcouncil.org**) states, "The upper tolerance on the tabulated nominal dimensions shall not exceed 1⁄32 in. Exception: In the width of slotted holes, gouges not more than 1⁄16 in. deep are permitted."

There is sometimes damage that occurs around the hole during hole-making. The Commentary to the AISC *Specification* also states, "Because of possible damage around a hole during drilling or punching operations, 1⁄16 in. (1.5 mm) is added to the nominal hole diameter when computing the net area." Also note that this tolerance does not preclude the presence of taper, which will be present in some cases, such as when holes are punched.

Larry S. Muir, P.E.

Section Properties

Where can I find a document that contains the weights, geometries and geometric properties of the different types of beams?

The primary source for that information is the AISC *Steel Construction Manual*, which is presently in its 14th Edition (**www.aisc.org/manual**). Alternatively, there is the AISC Shapes Database, which is an Excel file that lists the various dimensions and properties of structural steel shapes (**www.aisc.org/shapesdatabase**).

The database is a free download, but it assumes that you are already familiar with the various shapes as well as what the various dimensions and properties mean. There is a "read me" file linked from the above page that may help, but if you are not familiar with structural steel shapes you may find it easier to simply use the *Manual*, as it contains useful diagrams and charts.

Lastly, a number of useful utilities have been made by others using our Shapes Database file, and you may find that some of them meet your needs. One website where these sorts of utilities are posted is **www.steeltools.org**.

Martin Anderson

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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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