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.

C_b Factor

I am comparing the 2010 AISC *S c ca* to the 1989 ASD *S c ca* . 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 *S c ca* 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 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 , although it dates back to Kirby and Nethercot's 1979 publication *D* . 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 *E* 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 *G D C* , 6th Ed.

B D .E., .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 / > 200, Equation E5-2 does not apply and Section E3 or E7 applies instead. In these latter sections,

steel interchange

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.

HSS Workable Flat

What is the definition of the "workable flat" dimension given in the HSS tables in the 14th Edition AISC *S C c Ma a*?

The workable flat dimension is defined in Part 1 of the 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 2.25 . The term 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 3 ."

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 (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 also states, "Because of possible damage around a hole during drilling or punching operations, 1/16