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steel interchange

Double-Angle Compression Members

How is the number of intermediate connectors calculated in AISC *Steel Construction Manual* Table 4-8 for the design of double-angle compression members? Using the "75% rule" found in AISC 360 Chapter E, my results do not always match those shown in the table.

AISC *Specification* Section E6.2 requires that the slenderness of the individual components of the built-up member must not exceed three-quarters of the controlling slenderness of the overall built-up member. I believe this is what you are referring to as the "75% rule." This criterion is used in Table 4-8 in the AISC *Manual*. However, as a practical matter aimed at the efficient use of materials, the AISC Committee on Manuals chooses to use an additional criterion in the creation of this table.

The tabulated values for axial strength and corresponding number of intermediate connectors given in the table are such that the available compression buckling strength about the Y-Y axis is equal to or greater than 90% of that for compression buckling of the two angles as a unit. In many cases, using only the "75% rule" in AISC *Specification* Section E6.2 would require fewer connectors than the number tabulated in the table. However, if this were done, then the tabulated values cannot be used and the compression strength must be recalculated using the corresponding modified slenderness from Section E6.1.

This information is outlined in the description for Table 4-8 found on page 4-7 of the 14th Edition *Manual*.

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

Specifying Clevises and Pins

The 14th Edition AISC *Steel Construction Manual* Table 15-4 provides the maximum diameter of the connecting rod (D) for various clevis sizes, and Table 15-5 provides a range of clevis sizes to match possible rod and pin sizes. It seems to me that this implies, on the contract documents, that we should not only specify the clevis size but also the pin size to match the connecting rod size. Is this correct or is it sufficient to just specify the clevis size?

Not exactly. The size of the clevis does not set the size of the pin. Both the clevis and pin must be sized for the required strength. If you are providing the design of the connection, then you should provide both the pin and the clevis sizes on the structural drawings. If you are delegating the design of the connection, then you should provide the required strength on the structural drawings.

Larry S. Muir, P.E.

OCBF Work-Point Eccentricities

I am working on a project using braced frames for the lateral force resisting system. Originally, the frames were configured as truly concentrically braced; the member centerlines all intersected coincident with the work points. A recent change has led to the work points at the base of the columns being raised up 18 in. to 24 in. Does this system still qualify as an ordinary concentrically braced frame (OCFB)? If not, is my only option to use an *R*=3 system? (I am in a low-seismic area.)

The 2010 AISC *Seismic Provisions* have a basis-of-design section that specifically addresses this issue for OCBFs and SCBFs (special concentrically braced frames). Section F1.2 for OCBFs states:

"This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if they are accounted for in the member design by determination of eccentric moments using the amplified seismic load."

This is obviously not aimed specifically at base connections, but I think the intent is the same. Small eccentricities are allowed if they are accounted for in design. It is a matter of engineering judgment how column base offsets are dealt with and the acceptable magnitude of such eccentricities. This same judgment should be exercised when permitting eccentricities in *R*=3 systems.

However, this may be more of an academic discussion if you are in Seismic Design Category C or less. Using an OCBF instead of an *R*=3 braced frame only results in a slight decrease in the design loads for the brace (*R*=3¼ vs. *R*=3), but that is coupled with a dramatic increase in design loads for the columns, beams and connections. Typically an *R*=3 system will be the more cost-effective choice, when it is permitted.

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

Welding Machine Calibration

Could you direct me to the code that addresses how often I need to calibrate a welding machine?

AWS D1.1 Clause 5.11 states that welding equipment "shall be in such condition as to enable personnel to follow the procedures and attain the results." Common industry practice is to calibrate welding machines on an annual basis, though this frequency is not specifically mandated anywhere. If you are welding to the AWS D1.5 *Bridge Code*, then Clause 4.31.1 requires that welding machine calibration be performed every three months.

Keith Landwehr

Erection Marks

Do AISC specifications or codes contain requirements for how beam erection marks are placed?

The system used for erection marks is a contractual matter and is not specifically addressed in the AISC *Code of Standard Practice*. However, typical industry practice related to erection marks is discussed in the 3rd Edition of AISC's *Detailing for Steel Construction*. The section on "Locating Marks" in Chapter 6 describes common practice as follows:

"The shop places erection marks on the left end of pieces detailed in horizontal or diagonal positions and at the bottom of pieces detailed in the vertical position. Therefore, placement of these marks on the erection drawings must follow the same system. This marking system, along with the fact that the marks are placed on steel to read right-side up, enables the erector to position most of the members in a structure by referring to the location of marks on the drawings.

Some fabricators prefer to use variations of this system. For example, the compass direction is noted on some members, notably columns. Thus: "Mark Face A North." Likewise, members such as long girders or trusses, which cannot be turned at a job site, will require a compass direction on the appropriate end so it will be shipped that way (i.e., with the end pointed in the proper direction upon its arrival at the job site).

Although there are no requirements in the AISC *Specification* or *Code*, this guidance reflects what is common in the industry.

Erin Criste

Free Edge Buckling of Gusset Plates

When designing connecting gusset plates for braces in an inverted-V braced frame to the beam above, what are the requirements to determine the thickness and the width of the stiffeners that are placed between two braces on the gusset plate to limit the free edge buckling length?

That is a very good question, and it is not well-known that these stiffeners are not necessarily required.

The use of intermediate stiffeners in the gusset plate and the corresponding stiffeners at the gusset edges is based on a 1998 publication in the *Steel Tips* series. It recommends a maximum free edge buckling length, but if this maximum length was exceeded, there were not any recommendations on how to size or connect these stiffeners. The result is that "nominal" stiffeners and welds were used with little research or design guidance to justify the design. Example 3.10 in the 1st Edition of the AISC *Seismic Design Manual* includes these stiffeners because the free edge criterion is exceeded, and the stiffeners and welds are sized and shown on the final figure (Figure 3-14). However, these sizes are simply stated as nominal values using normal plates and corresponding weld sizes.

Later research and investigation into the requirements for these stiffeners, intended to improve the guidance provided, has actually changed the current thinking. It is now thought that these stiffeners do not serve the purpose they were proposed to serve and that the gussets do not have free edge buckling problems. Furthermore, use of stiffeners to limit the free edge length may actually be detrimental to the performance of the connection as they introduce a point of high local stiffness and can have the tendency to increase and concentrate deformational demands at the stiffener location. As a result, the free edge buckling check has been removed in the 2nd Edition AISC *Seismic Design Manual* examples. The technical justification for this is provided in the Commentary to Section F2.6c in AISC 341-10:

"Certain references suggest limiting the free edge length of gusset plates, including SCBF brace-to-beam connection design examples in the *Seismic Design Manual*, (AISC, 2006), and other references (Astaneh-Asl et al., 2006; ICC, 2006). However, the committee has reviewed the testing cited and has concluded that such edge stiffeners do not offer any advantages in gusset plate behavior. There is therefore no limitation on edge dimensions in these provisions."

Therefore, it is AISC's recommendation that stiffeners not be used to limit the free edge length of gusset plates in OCBFs or SCBFs.

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

Preheat Requirements for Heavy Shapes

The 1999 LRFD *Specification* Section J2.8 requires a minimum preheat of 350 °F for welded splices in Group 4 and 5 shapes. I cannot find this requirement in AISC 360-05 or AISC 360-10. Has this requirement been removed?

Yes, this requirement has been removed. Regarding terminology, a significant change was made in the 2005 *Specification*. Following a similar change in ASTM A6, AISC 360 dropped the group designations and now refers to "heavy sections." Generally, these are what used to be called Group 4 and 5 shapes. Essentially, these are W-shapes with flanges greater than 2 in. thick. If you review the beam tables in the 13th or 14th Editions of the AISC *Steel Construction Manual*, you'll see that the heavy shapes are annotated with footnote "h."

As to preheat requirements for splices in heavy sections, you'll now want to review AISC 360-10 Section J1.5. The 350 °F preheat requirement was deleted primarily because it was determined that the AWS D1.1 preheat requirements were sufficient in this application. Therefore, AWS D1.1 should be consulted for specific preheat requirements. In addition, if you follow the references made in AISC 360 Section J1.5, you will find that the 150 °F preheat is still required by Section M2.2 for thermal cutting of copes and weld access holes.

Keith Landwehr

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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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