If you've ever asked yourself "why" about something related to structural steel design or construction, *Modern Steel Construction's* monthly *Steel Interchange* column is for you!

Minimum Percentage of Composite

According to Section I4 of the 1989 *ASD Specification*, the value of V'_h for shear connectors used in composite construction is not permitted to be less than ¼ the minimum of Equation (I 4-1) or (I 4-2).

If the number of connectors provided is less than required to induce a horizontal shear capacity equal to ¼ the shear capacity of the section, can V'_h still be calculated and a composite section assumed or should the assumption be zero composite action with the steel beam resisting the total load?

Question sent to AISC's Steel Solutions Center

Low percentages of composite interaction (below ¼ or 25%) are discouraged for several reasons. At levels below 50% interaction the stud failure mode tends to be sudden and catastrophic, i.e., there is little redundancy in the system and no warning of impending failure.

Another reason for the limitation is that there is very little data on the validity of the composite design equations at low levels of interaction. One should not assume composite action for interactions below 25%.

Sergio Zoruba, Ph.D.
American Institute of Steel Construction, Inc.

Design of HSS Base Plates

I know that there is a different method to use in designing the base plates of an HSS column as opposed to that for a W-shape. What equation is used for HSS bases, and how does one determine the area of the plate needed?

Question sent to AISC's Steel Solutions Center

The required base plate area of an axially loaded HSS column should not differ from that of a W-shape column with the same magnitude of load, since it is assumed that a uniform bearing pressure will size the plate plan dimension requirements based on the axial compressive load.

However, the thickness of the plates may differ for the two shapes. Typically for the closed HSS the effective bending dimension is taken as $0.95d$ in each direction, as opposed to the W-shape procedure of taking $0.80b_f$ in the direction parallel to the column flanges. For reference you may want to look at *Design Guide 1: Column Base Plates,* a free download for AISC members from www.aisc.org/epubs. The guide can also be purchased from www.aisc.org/bookstore.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction, Inc.

Constructability

We are designing a one-story strong beam/weak column moment frame. The small columns we specified are W6×25s. They work fine in terms of structural capacity, but the fabricator is having trouble making the connections. They requested that the design be changed to a strong column/weak beam moment frame. Any thoughts?

Question sent to AISC's Steel Solutions Center

Although W6 columns may perform satisfactorily on a computer analysis program, you should always consider constructability. For your case, a deeper column would not only facilitate fabrication of the connections, but may also reduce field complications during erection. A rule-of-thumb to remember is that using W10 or larger columns significantly reduces constructability issues.

Bill Liddy
American Institute of Steel Construction, Inc.

Fire Resistance of Concrete-Filled HSS Columns

I have HSS16×16 concrete-filled tubes. In AISC's *Design Guide 19*, it appears that because the section is larger than 12", fire protection is required. Is there any additional research that includes larger sizes of HSS filled with concrete that will meet fire code criteria?

Question sent to AISC's Steel Solutions Center

The following general references address fire resistance of concrete-filled HSS columns:

- American Society of Civil Engineers (www.asce.org) Standard ASCE/SFPE 29-99, *Structural Calculation Methods for Structural Fire Protection* (1999), contains relevant equations in Chapter 5, "Standard Methods for Determining the Fire Resistance of Structural Steel Construction," Article 5.2.3, "Concrete-Filled Hollow Steel Columns," and Article 5.2.4, "Concrete or Masonry Protection."
- "Design of Concrete-Filled Hollow Structural Steel Columns for Fire Endurance" by V. K. R. Kodur and D. H. MacKinnon, *Engineering Journal*, First Quarter, 2000. This paper can be downloaded (free for AISC members) from www.aisc.org/ej.
- An April 1998 AISC North American Steel Construction Conference Proceedings paper called "The Fire Endurance of Concrete-Filled Structural Steel Columns" by Kodur and MacKinnon.

Sergio Zoruba, Ph.D.
American Institute of Steel Construction, Inc.

Additional response:

These papers essentially describe the development of a formula that is contained in the ASCE 29 standard and AISC *Design Guide 19*. The column size limits associated with the formula are based on the range of sizes that were used in fire resistance tests (as the formula is essentially based on the interpolation of fire resistance test results).

I am not aware of larger concrete-filled HSS that were tested in standard fire resistance tests (column fire furnace set-ups do have practical limitations). Research on larger sections is usually done using numerical analysis.

There was a recent relevant article from the Society of Fire Protection Engineers' (SFPE) *Fire Protection Engineering* magazine (Summer 2004 issue) entitled "Case Study Using SAFIR to Predict Fire Resistance of a Unique Column Design" by Chen and Gemeny. SFPE's web site is www.sfpe.org. There is also relevant software, POTFIRE, available free of charge from CIDECT (www.cidect.org).

Farid Alfawakhiri, Ph.D.
American Institute of Steel Construction, Inc.

Continuity Plates for IMF

For the case of steel intermediate moment frames in dual systems, are continuity plates required if the requirements of FEMA 350 equations are met,

$$t_{cf} > b_f/6$$

$$t_{cf} > 0.4\sqrt{1.8 b_f t_f \left(\frac{F_{yb} R_{yb}}{F_{yc} R_{yc}} \right)}$$

or are continuity plates always required? When are stiffener plates not required in steel intermediate moment frames?

Question sent to AISC's Steel Solutions Center

An intermediate moment frame (IMF) system must satisfy the requirements of the 2002 AISC *Seismic Provisions* for demonstrated capability of meeting the requirements stipulated in Section 10.2a. The methods of conformance demonstration are given in Section 10.2b to either use a connection prequalified for IMF in accordance with Appendix P or to provide cyclic test results in accordance with Appendix S.

All FEMA 350 prequalified connections have continuity plates and the 2002 AISC *Seismic Provisions* requires that tests used to qualify the connections must be consistent with the design. However, the new seismic connection prequalification document to be published later this year has included this calculation concept for the RBS connections. Thus, for that connection you can use the FEMA 350 equations to determine if stiffener plates are not required.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction, Inc.

Punching Shear

When designing a single-plate shear connection for a beam-to-HSS column (Figure 4-4 in the *HSS Connections Manual*), the limit state of punching shear in the HSS wall is checked. In the procedure for a single-plate connection to a girder web (Example 10.12 in the 3rd Ed. *LRFD Manual*), there is no punching shear check on the web. Are the restraints sufficiently different so it is not a likely failure mode? Has there been any research on the shear-plate connection to a web?

Question sent to AISC's Steel Solutions Center

Yes, the restraints of the beam web and HSS column wall are different. Most single-plate shear connections attached to HSS columns act as rigid joints, even for columns with large b/t ratios. The beam end rotation in rigid joints occurs within the connection itself (rather than outside of it) and is due to the rather large rotational stiffness of the HSS column wall. Don Sherman, Ph.D., is attributed as having discovered this effect, which is discussed in his 1991 NASCC paper entitled "The Design of Shear Tabs with Tubular Columns."

The same rigid behavior occurs for two-sided beam-to-girder shear plate web connections, but punching cannot occur because the tabs on each side prevent this limit state from occurring. Offset tabs must be investigated for stiffness and the potential for punching shear.

At the opposite side of the spectrum, a single-sided beam-to-girder single plate shear connection will act as a flexible connection (that is, the girder rotational stiffness is small). Punching shear does not control the design because the girder rotational stiffness is too small and there is not enough rigidity for punching shear to occur. Hassan Astaneh, Ph.D., released a research report, *Design of Single Plate Framing Connections*, in 1988. Porter and Astaneh released a research report, *Design of Single Plate Shear Connections with Snug-Tight Bolts in Short Slotted Holes*, in 1990.

Sergio Zoruba, Ph.D.
American Institute of Steel Construction, Inc.

Steel Interchange is a forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

If you have a question or problem that your fellow readers might help you to 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:

Steel SolutionsCenter
Your connection to ideas + answers

One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.670.9032
solutions@aisc.org