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Stiffeners Required?

When a moment connection is required on either side of a W-shape column, typically stiffeners are welded to the column web and flanges to transmit the beam flange forces through the column web. What is required when the column is a tube section and the beams are typical W-sections? Is any stiffening of the tube required?

It may be typical for some engineers to arbitrarily place stiffeners in W-shape columns at all rigid moment connection joints, but this may not always be the economical approach. It is sometimes more economical to revise the size of the column in order to eliminate the need for stiffeners (continuity plates) in such regions. AISC has developed a SteelTool (available at **www.aisc.org/steeltools**) called Clean Columns. This SteelTool can be used as an aide in determining what W-shape column size is required in order to eliminate the need for stiffeners.

The use of moment connections involving HSS columns may be more difficult if the tube wall is inadequate to accommodate the moment forces from the rigid beam connection. One would first want to check if the HSS can accommodate the concentrated forces using the specification's Chapter K provisions (a free download at **www.aisc.org/2005spec**). If the tube cannot accommodate those forces, then one may want to consider adding flange connection plates that are connected around the outside of the HSS, as adding internal stiffeners generally isn't practical (or possible).

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

Standard Bolt Hole Size

The AISC standard hole is 1/16 in. larger than the bolt. I am designing bearing-type connections, but the steel fabricator is asking to provide holes larger than the standard holes for constructability. He is proposing to use holes that are 1/8 in. larger than the bolt diameter, rather than 1/16 in. Since this is a bearing-type connection rather than a slip-critical connection, I am not quite sure if this will affect the shear strength of the bolt for the connections. I know that the oversized holes should not be used for a bearing-type connection. However, the holes that the fabricator is proposing are between the standard and oversized. Could you give your opinion on this issue?

If a bolt hole diameter is 1/8 in. over the nominal bolt diameter, it must be considered oversized. The hole sizes specified in the AISC specification are maximums; intermediate values must be treated as the next larger size.

Amanuel Gebremeskel, P.E.

Fillet Weld/Base Metal Thickness Correlation

On page 13-27 of the 3rd edition LRFD manual, the minimum gusset plate thickness is calculated as t_{min} = 6.19D/F_u. The explanation is that gusset plate thickness is checked against weld size required for strength. In the 2nd edition LRFD manual, this equation is 5.16D/F_y (page 11-37 of Volume II, Connections). Can you please explain where the 6.19 is coming from?

At the time the 2nd edition LRFD manual was in use, the shear yield strength of the base metal was compared to the shear rupture strength of the weld. Now the shear rupture strength of the base metal is compared to the shear rupture strength of the weld. You can also find the current procedure, using the 6.19 coefficient, defined in Part 9 of the 13th edition manual as well.

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

Slip-Critical Faying Surfaces

I have noticed a discrepancy between the 2005 AISC specification and the *RCSC Specification for Structural Joints Using A325 or A490 Bolts*. The Class A coefficient is 0.35 in the AISC specification and 0.33 in the RCSC specification. Which coefficient is correct?

The 2005 AISC specification was published after the 2004 RCSC specification and reflects later revisions made for simplicity in the area of slip-critical faying surface requirements. Note that besides the slight variation made in the coefficient for the Class A surface, the current AISC specification now only includes two classes (A and B), unlike the previous AISC specifications and the 2004 RCSC specification, which had three classes (A, B, and C). This slight change was made for simplicity.

As discussed in the Commentary to Section J3.8 of the 2005 AISC specification, "This Specification has combined the previous Class A and Class C surfaces into a single Class A surface category that includes unpainted clean mill scale surfaces or surfaces with Class A coatings on a blast cleaned surface, and hot-dip galvanizied and roughened surfaces with a coefficient of friction of μ = 0.35. This is a slight increase in value from the previous Class A coefficient. Class B surfaces, unpainted blast-cleaned surfaces, or surfaces with Class B coatings on blast-cleaned steel remains the same at μ = 0.50."

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

V and Inverted-V Lateral Systems

For chevron braced frames, the AISC *Seismic Provisions* state that we must design the beam for the resultant force caused by one of the braces buckling in compression and the other yielding in tension. The beam design is generally an uncoupled design problem. However, it becomes coupled if using two-story X-braces or zipper columns (see Fig C-1-13.3 in 2005 *Seismic Provisions*).

It would seem to me that the design should proceed on a story-by-story basis. That is, you would assume that the brace buckling/yielding occurs in one story and then design the zipper/opposing V for the resultant. Is that true? Or do you assume that the buckling/yielding occurs in multiple stories simultaneously?

If you have a zipper column or a two-story X-braced system, it is not necessary to design for unbalanced load on the beam. The two-story X and zipper configurations provide a load path that does not involve the beam.

Amanuel Gebremeskel, P.E.

Column Splice Locations

We are designing a nine-story building, which makes it logical to have two column splices, with three segments of three-story columns. However, in some AISC articles on economy in steel design, it is noted that "three-floor columns are to be avoided due to erection difficulties." I found one article that suggests that erection typically occurs in two-story increments, and mentions the resulting problem of another long column dangling in the air if you use three-story columns. Is that the reason for the recommendation for either two- or four-story columns? With nine stories, we would end up with either 4-4-1 or 2-2-2-3. What are your thoughts?

The suggestion of two- or four-story lifts arises from the thought that three-floor column tiers are somewhat out of sync with the OSHA requirement for providing decking, planking, or netting within the lesser of two floors or 30 ft under steel erection.

In the two-story lift erection scheme, the raising gang will erect two levels of framing, and the decking crew will deck the top level first. That permits the raising gang to erect the next tier while the decking crew decks the intermediate floor.

The four-story lift erection scheme is slightly different. The raising gang will erect the first two levels of framing, and the decking crew will then deck the second level. The raising crew then continues with the erection of the third and fourth levels as the decking crew decks the first level. After that, the decking crew decks the fourth level, and the process is repeated up the building.

In a three-story erection scheme, some efficiency is lost: The OSHA height limit means that you can't nest two floors of decking in each cycle.

Kurt Gustafson, S.E., P.E

Direct Analysis Method

Section 7.3 of Appendix 7 of the 2005 AISC specification states, "For ASD, the second-order analysis shall be carried out under 1.6 times the ASD load combinations and the results shall be divided by 1.6 to obtain the required strength." Can you explain what this means? How do you incorporate this when using a computer program?

The application is actually quite simple. If the ASD load is 30 kip-ft, then the load that should be used for the DAM is 30 × 1.6 = 48 kip-ft. The resulting second-order moment (say, from the computer program) may then be 62 kip-ft, including P-Delta effects. The ASD load to use for design would then be 62/1.6 = 38.8 kip-ft. Note that the reason for this manipulation is because the relationship of forces when doing second-order analysis is not linear, and direct analysis and design for the service level forces underestimates the true impact of second-order effects on frame deformations.

In other words, if you simply apply the 30 kip-ft ASD load in the analysis, you would get a result that is less than 38.8 kip-ft, because the lateral deflections are lower.

Amanuel Gebremeskel, P.E.

Singly Symmetric Shapes

I have questions pertaining to Equations (C-F4-3) and (C-F4-4) shown in the Commentary to the 2005 AISC specification. What is the significance of $\alpha - \beta_x$? What is the F_{yr} being used in the calculation of L_r? It is not defined.

As indicated in the Section F4 Commentary, these Equations are suggested as possible alternative equations that can be used in lieu of the Specifications (F4-5) and (F4-8). Professor Don White of Georgia Tech is the author of the supporting references and has kindly responded as follows:

"Regarding β_x, this is the nature of the problem for singly symmetric girders. The shear center is closer to the larger flange; this leads to a decrease in the buckling capacity captured by the negative β_x.

F_{yr} should read F_L. We use F_{yr} for this term in AASHTO. It was that way in the AISC 2005 specification draft up until very late, when it was pointed out that this was in conflict with a symbol used by the composite committee."

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

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Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is AISC's vice president of engineering and research.

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Steel Solutions Center
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.803.4709
solutions@aisc.org