Steel Interchange is an open 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
One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.423.4651
solutions@aisc.org

EFFECTIVE LENGTH FACTOR

Is there any documentation regarding the assumption an effective length factor of $K = 1.0$ if a P-delta analysis is performed?

Question sent to www.seaint.org

Your question is one with which engineers regularly struggle and I hope I can provide a useful answer. The first, and perhaps most important, issue has to do with looking at and combining provisions from two different specifications. The AISC, CISC and other steel design specifications will all accommodate stability issues differently and it is imperative that the provisions within a single specification are used consistently. One approach taken in the AISC *Specification* is to use effective length for column axial capacity and a second-order moment magnification for checking bending capacity. Alternatively, the engineer could choose to do a second-order analysis but it must be done correctly and that is usually the place where misunderstandings occur.

The Canadian code uses a different approach to stability and second-order effects (the notional load approach). Thus, their statement that the effective length factor may be taken as 1.0 when a second-order analysis is used is inappropriate for use with the AISC *Specification* at this time.

When carrying out a second-order analysis one must be sure to address both system buckling and moment amplification. The second-order analysis procedures in a typical computer program will determine the magnitude of the moments in the displaced equilibrium configuration corresponding to the applied loads. They will not determine the buckling capacity of the frame.

However, frame buckling capacity can be determined if the second-order analysis is taken incrementally to the limit. This requires more than just checking the box in the software program for a second-order analysis.

It should also be noted that a second-order incremental approach can have some potential problems. If the frame to be analyzed has no lateral load and no sway under gravity load, a second-order analysis will yield the same results as a first-order analysis, thus saying nothing about column capacity or any second-order moments.

So, using a typical second-order analysis at ultimate load will permit the elimination of the code-specified moment amplification factor from the interaction equation but will not permit the use of $K = 1.0$. A second-order analysis under ultimate load taken to the limit will determine the buckling capacity of the system and that will negate the need to use a K-factor to determine that capacity.

A paper I published in the *Proceedings* of the 2000 North American Steel Construction Conference, "A Practical Look at Frame Analysis, Stability and Leaning Columns" might be of help to you. A paper of the same title that should be published soon in the AISC *Engineering Journal* will provide some additional discussion and should also be of assistance.

Louis F. Geschwindner, P.E., Ph.D.
American Institute of Steel Construction
Chicago, IL

HSS SLOTTED GUSSET PLATE CONNECTION

On page 6-17 of the *HSS Connections Manual*, a "rule of thumb" for L_w is that it should be at least equal to the HSS depth. Is this a requirement?

The 1999 *LRFD Specification* Section J2.2b (last paragraph on page 16.1-54 of the 3rd edition *LRFD Manual*) requires a similar minimum distance for longitudinal welds at the end of a flat bar member in tension. However, I don't consider a tube to be a flat bar, but obviously from the wording in the *HSS Manual*, it seems that I should be still adhering to this rule of thumb. Using the example I described above, is it allowed to use a length of weld less than the rule of thumb as long as the connection checks?

Question sent to AISC's Steel Solutions Center

Section J2.2b applies only to flat-bar tension members in order to minimize the effect of shear lag. The weld length can be less than the HSS width as long as it satisfies the shear lag fracture limit state checks. However, the rule of thumb is a good one to follow and will result in reasonably proportional connections.

There is a recent paper that may be helpful to you: "Gusset Plate Connections to Round HSS Tension Members" by Cheng, et al. published in *Engineering Journal* during the 4th quarter of 2000. This paper establishes that shear lag effects are minimized ($U = 1$) for round HSS with slotted/gusset plate connections, as long as the fillet weld lengths are at least 1.3 of the HSS diameter.

Sergio Zoruba, Ph.D.
American Institute of Steel Construction
Chicago, IL

PRETENSIONING ANCHOR RODS

from October 2002

ASTM F1554 Grade 105 anchor rods will be used in a coastal area with wind gusts of 130 mph located in a very high seismic zone. We are planning to pretension the anchor rods to avoid the risk of inducing tensile fatigue from loading cycles resulting from wind loads. Do you agree with this as being a valid reason to pretension these rods?

Question sent to AISC's Steel Solutions Center

As an addition to Charlie Carter's reply concerning pretensioned anchor rods, such rods must have a bond breaker. PVC pipe is good, for a distance to break the bond with the concrete. Otherwise, the elongation takes place in a short distance, and under fatigue, the anchor rods will suffer a fatigue fracture. This has happened too many times on pedestal cranes used in paper mills, lumber yards, etc. To verify the anchor rod tension, measure the elongation required to give the tension as in $\Delta = PL/AE$. It's quite a bit more accurate than torque. The length of the unbonded rod is based on engaging enough foundation vertical steel to transfer the rod load to the concrete reinforcing, embedment length of the vertical rebar plus distance from the rod to rebar. Since we began using this detail, we haven't broken an anchor rod.

John Stiles, P.E.
CH2M HILL
Milwaukee, WI

CONSIDERATION OF CONNECTION ECCENTRICITY

from October 2002

In exterior columns, should the eccentricities resulting from the beam connections be considered when the connections are not designed by the SER? For example, when a W-shape is framed into an exterior HSS column via a single shear plate, the column could be designed for the eccentricity equal to the distance from its centerline to the bolt line. With that approach, it might make sense not to place the beams at the column lines and laterally brace the column by a light angle section.... Alternatively, the beams could be assumed to extend into the column centerlines and the fabricator directed to design the connection for combined shear and moment. (Can the AISC *ASD Manual's* tables for single-plate shear connections or eccentric bolted connections be used for that purpose?)

If the eccentricity is considered in column design, it should presumably be applied in two directions in corner columns and in columns where the exterior girders deliver vastly unequal reactions from the opposite sides. This might lead to the corner columns actually being heavier than the interior columns that support four times the load.

Alexander Newman, P.E.
Maguire Group Inc.
Foxborough, MA

The effect of connection eccentricities must be considered when designing columns. In the case of beam-to-column connections using standard simple shear connections, the effects of connection eccentricities will usually be negligible. There are situations however, such as at corner columns, where effects of connection eccentricities might be substantial enough to affect the column size. This is especially true when corner columns are slender and lightly loaded.

According to Section 3.1.2 in the *Code of Standard Practice for Steel Buildings and Bridges* (March 7, 2000), when the structural engineer allows a Fabricator to select shear connections, the structural drawings must indicate any restrictions and limitations to which the Fabricator must adhere when selecting and detailing the connections.

At a minimum, the structural drawings should show all beam and girder reactions and should indicate the maximum permitted connection eccentricity for which the simple shear connections could be detailed.

Most structural steel design software programs permit designers to input connection eccentricities into the computer model at beam-to-column shear connections. An eccentricity of 3" is commonly used for this variable. This value is accurate for some types of shear connections (i.e., single-plate shear connections and seated beam connections) and conservative for others (such as end-plate shear connections).

Since the cost of designing and fabricating connections is a substantial percentage of the total cost of structural steel construction, and since the majority of connections on building structures are simple shear connections, structural engineers seeking to produce cost-efficient designs should permit steel fabricators to bid projects with the understanding that they will be able to use their preferred shear connection.

While the use of a standardized connection eccentricity could affect the size of some corner columns, a fabricator's ability to use standardized shear connections will most likely result in cost savings that will more than offset the increased material costs due to additional weight on some of the columns.

Clifford Schwinger, P.E.
Cagley, Harman & Associates
King of Prussia, PA