STEEL INTERCHANGE

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. If you have a question or problem that your fellow readers might help you to solve, please forward it to *Modern Steel Construction*. At the same time, feel free to respond to any of the questions that you have read here. Please send them to:

Steel Interchange
Modern Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

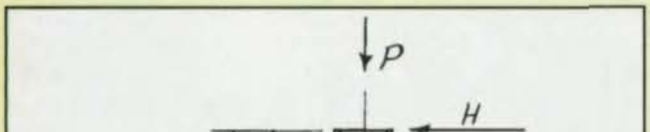
Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

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 principals to a particular structure.

Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 800/644-2400.

The following responses from previous Steel Interchange columns have been received:

In a partially cover-plated column, how would you analyze the column for governing l/r ratio to calculate F_a?



Since the r's (radius of gyration) are known, the key to the calculation of F_a is to find the K in the allowable stress equation for axially loaded compression members.

Per AISC Design Guide, "Industrial Buildings, Roofs to Column Anchorage" Appendix B: Calculation of Effective Lengths of Stepped Columns, the values of K_1 and K_2, representing the effective length factors for upper and lower segments of column, respectively, can be attained by solving the equations proposed by J.P. Anderson and J.H. Wodward (AISC Engineering Journal, October 1972, pp. 157 - 166) or by entering Table 1 (included in Appendix B) with the values of I_1/I_2, L_2/L_T, and P_2/P_T under the appropriate end condition category.

In this case, by using Table 1, L_2 = 10', L_T = 30', P_2 = 0.0, P_T = p, then L_2/L_T = 0.33, P_2/P_T = 0.0 and the end condition is "Fix-Free"; therefore, by entering the value of I_1/I_2 (the ratios of moments of inertia of upper and lower segments, respectively) the values of K_1 and K_2 can be determined by interpolation.

Kunming Gwo, P.E.
HCI Steel Building Systems, Inc.
Arlington, WA

Another answer:

The L/r ratios do not work well for this type of column. Instead, calculate the theoretical Euler buckling load, and then check both the elastic and inelastic allowable buckling loads.

There are several procedures to calculate the Euler buckling load. Simple hand calculation methods include Finite Differences, integration, or Newmarks Method. Example calculations for the first two methods are available in " Principles of Structural Stability Theory" by Alexander Chajes, and the last in "Theoretical and Applied Mechanics" edited by N.M. Newmark.

After finding the theoretical Euler buckling load (P_{cr}), determine whether elastic or inelastic buckling controls. If the Euler buckling load (P_{cr}) is less than $P_y/2$ (where $P_y = AF_y$) then elastic (Euler) buckling controls. This is the basis for the AISC ASD equation for C_c, see the AISC ASD Commentary. If elastic buckling controls, simply multiply the theoretical buckling load by $^{12}/_{23}$ and you have the allowable axial load.

If inelastic buckling controls (i.e. $P_{cr}>P_y/2$) it becomes a little more complicated. Lets dissect AISC ASD Equation E2-1. The denominator is simply a sliding factor of safety that results in $^{23}/_{12}$ where $Kl/r = C_c$, and results in $^5/_3$ when $Kl/r = 0$. For the moment, call the denominator the safety factor, SF. Now we can rewrite equation E2-1 as:

$F_a = [1 - ((Kl/r)^2/(2C_c^2))]F_y/SF$

This equation can be rewritten in terms of force s follows:

$$P_a = [(1 - AF_y) / 4P_{cr}]AF_y / SF$$
$$\text{since} P_{cr} = \pi^2 E / (Kl / r)^2$$
$$\text{and} C_c = \sqrt{2\pi^2 E / F_y}$$

Now, having calculated P_{cr} from one of the methods noted above, and knowing all the other terms, you have only to determine the appropriate safety factor, SF. The term $(Kl/r)/C_c$ can be rewritten in terms of force if one remembers that:

$$C_c = \sqrt{2\pi^2 E / F_y}$$

Substitute this into the equation for $(Kl/r)/C_c$ and you will finally get:

$$(Kl/ r)C_c = \sqrt{[(F_y A) / 2P_{cr}]}$$

Substitute this into the denominator in place of $(Kl/r)/C_c$ and you have an equation rewritten in terms of force. Use the smaller section to determine steel area, A, since inelastic buckling will occur there first.

Remember that K is already included in P_{cr}, since you determined the buckling load P_{cr} directly with its actual boundary conditions.

Duane L. Siegfried, P.E.,S.E.
Ralph Hahn and Associates, Inc.
Springfield, IL

Another response:

The most common approach is to use the formula $P_{cr}=K_1\pi^2E_1I_1/L_2$, where E_1/I_1 is the stiffness of the smaller section, L the total length of the column, and K_1 depends not only on end constraints, but also on the ratio of stiffness and lengths for the two sections.

Values of K1 are tabulated for different end constraints and ratios of stiffness and lengths in "Formulas for Stress and Strain", by Raymond J. Roark and Warren C. Young, McGraw-Hill, New York (various editions).

For the case in question (cantilever column), K_1 varies between and 0.5, with the larger section up to twice as stiff as the smaller one.

Behrouz Jazayery, P.E.
Mueser Rutledge Consulting Engineers
New York, NY

What is the flatness tolerance for webs of welded plate girders?

Since for statically loaded (buildings) structures,, web flatness does not affect the structural integrity of a girder, neither the LRFD Specification not the AISC Code of Standard Practice provides a limitation on the maximum out-of-flatness of girder webs. AWS D1.1 Section 8.13.2 does, however, provide such requirements for welded plate girders. Problems arise, however, when these tolerances are applied to girders with thin webs. Specifically, in girder webs less than $^5/_{16}$-in. Thick, they do not account for operational difficulties caused by shrinkage resulting from web-to-flange welds and/or welds that attach stiffeners to the web. Because of this, in some cases, flatness within AWS tolerances cannot be practically provided.

AISC recommends that, for statically loaded (building) structures, the dimensional tolerance for deviation from flatness of a girder web less than $^5/_{16}$-in. Thick, without stiffeners or with stiffeners on one or both sides, be determined by the larger of ½-in or AWS Section 8.13.2. If architectural considerations require special flatness tolerances, such special requirements must be identified on the engineering drawings and stipulated in the bid documents.

American Institute of Steel Construction
Chicago, IL

New Questions

Are there special requirements for the design of High-strength A325 or A490 bolts that are going to be in a high temperature area?

Alice Leich
S.E.S Environmental, Inc.
Knoxville, TN

If a W-shaped column is made up of three welded plates, how does one design the welds connecting the plates together?

Correction: *The answer by James McCarthy in the June issue referred to the wrong question. The correct question is: In a partially cover-plated column, how would you analyze the column for governing l/r ratio? We regret the error.*