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 at:

Steel Interchange
Attn: Keith A. Grubb, S.E., P.E.
One East Wacker Dr., Suite 2406
Chicago, IL 60601
fax: 312/670-0341
email: grubb@blacksquirrel.net

Origin of K-factors for Columns

I recently looked at the theoretical K factors and the recommended design value when ideal conditions are approximated on p. 2-18 of the 1993 edition of the LRFD specification. My question is how these numbers are related; I would have expected some system was used to decide on these values, as some of the cases (e.g. pinned-pinned) have a 0% change, while others have an increase in effective length of 30% (fixed-fixed). Considering that the effective length decreases strength of the column as a square, this is a 70% strength reduction for the fixed-fixed column that is not applied to the pinned-pinned column. How were these numbers produced?

Brian Johnson
URS Corporation

The recommended design values for the effective length factor given in Table C-2 of the 1993 LRFD *Specification* are based on recommendations of the Structural Stability Research Council. There is a much more complete discussion of how some of these recommended values were determined in the Commentary (Section C2) of the latest LRFD Specification, downloadable from www.aisc.org/lrfd-spec.html.

Keith Mueller, Ph.D.
American Institute of Steel Construction
Chicago, IL

Beam Camber

Does AISC have any recommendations for sizes of beams which lend themselves well to cambering? Are there methods of cambering that are best used for light shapes such as W10x12's, W12x14's, etc.?

Robert Naples
William A. Kibbe and Associates
Saginaw, MI

The most economical way to camber beams is by use of a mechanical press. My experience has been that beams with very light webs have a tendency to cripple when forced into yield. Cambers must be limited with these light sections if they are mechanically cambered. There is also the problem that beams less then 24 ft. long may not fit in the standard cambering press that fabricators use and would require special setup or the use of heat. Heat cambering will increase the cost of cambering by a factor of three or more.

It is recomended that for very light beams and spans less then 25 ft. the designer avoid using camber. It is more economical to increase section size and eliminate the need to camber than to use special setups or heat cambering.

Larry Kloiber, P.E.
LeJeune Steel Company
Minneapolis, MN

Another response:

The capacity of the cambering device, either geometric size or load, is the limiting factor as to what shapes can be cambered.

W10x12 and W12x14 shapes are smaller than one usually sees at the cambering machine. Care should be taken to see that the thin webs do not buckle at the loading points. Lengthening the bearing areas can help in this regard. These shapes have little weak axis strength and should be well-supported laterally all during the loading cycle. Light shapes such as these are also subject to torsion deformations and the lateral support should be devised to prevent this. In other words, laterally support both top and bottom flanges at the loading points. By loading points I mean where the load piston(s) are located and where the support points are, at or near the member ends. Applied heat can be used as a "persuader" but may result in kinks in the member if poorly done. Roll cambering and gag presses are other methods but are less common because few fabricators have the required equipment.

The use of camber on such such light shapes should be reconsidered: used in floor construction, these shapes (even as composite members) will likely be bouncy and eventually annoy the building occupants. If used as roof purlins, why bother to camber? Just slope the roof.

David T. Ricker, P.E.
Javelina Explorations
Payson, AZ

Question from June 2001:

Welding Guidelines for Combination Sections

Can you suggest some guidelines for welding channels to wide-flange beams to produce the combination sections shown in the tables in the *Manual*?

Dipu Sengupta, S.E., P.E.
Sato & Associates
Honolulu, HI

Omer Blodgett (in *Design of Weldments* and *Design of Welded Structures* by the Lincoln Arc Welding Foundation) discusses the horizontal shear stresses at the joint interface, and methods to determine required weld sizes and weld patterns for such beam combinations. Shop procedures involve supporting the upside-down combination at its ends, and clamping the two pieces together with fixturing at the location of welding. By relocating the clamping fixtures at the welding location as welding progresses along the member, the quantity of fixtures required is greatly reduced. Negative cambering of the beam assembly resulting from weld shrinkage must be considered.

Dan Price
American Fabricators & Engineers
Cedar Falls, IA

Question from July 1999:

Welding Top Angles in Partially-Restrained Connections

In designing a partially restrained connection consisting of top and bottom angles and a simple shear connection, the point of inflection is between the beam flange bolt gage line and the face of the connection angle. In connections of this type, the driving of bolts is difficult and it may be impossible to tighten the bolts even if the gage lines are staggered. For this reason, instead of using bolts in the top flange of the beam, can welds that meet the strength requirements be used if the weld does not encroach on the inflection area?

Lawrence F. Kruth, P.E.
Douglas Steel Fabricating Corporation
Lansing, MI

The moment-rotation behavior of PR connections is very sensitive to even small changes in the configuration of the critical details of the connection. For example, a change in the bolt gage on the leg of the top and bottom angles attached to the supporting member would have a major impact, as would changing this leg connection from bolted to welded.

From your question, though, it seems you are concerned with the other leg of the angle—the leg that is attached to the supported beam flange. If so, you may be able to justify the change from a bolted detail to a welded detail if the flexural behavior of the angle is not affected and the resulting connection has adequate strength. As long as the intended capacity for deformation of the angle is not impaired by the change to a welded detail, a similar moment rotation behavior should be justifiable.

Note that one point of inflection will likely occur in each leg of the angle. One will probably occur near the bolt line in the angle leg attached to the supporting member. The other will likely occur in the leg attached to the supported beam flange near the toe of the leg-to-leg fillet. These will be the critical locations to ensure that the intended deformation capability exists.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL

Question from June 2001:

Steel Sheet Piling

Several readers alerted us to an additional source for sheet pile design that incorporates much of the old US Steel manual:

Pile Buck Inc.
Palm City, FL
www.pilebuck.com
561/223-1919

Thanks to Shane Mann, P.E., George M. Clendenin, P.E. and John R. Decator, Jr., P.E. for their information.