

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!

Specifying Connection Design Forces

In preparing specifications for structural steel, is it common practice to have connections designed for 50% of member capacity? If so, is this a service load or factored load? What are your recommendations on how to properly specify connection design forces?

Question sent to AISC's Steel Solutions Center

The practice you mentioned is an old one and does not follow the requirements of the AISC *Code of Standard Practice*. It can work, but is not ideal. Problems associated with specifying a percentage of the uniform load capacity in designing connections can be seen in long, shallow beams. The connection in such beams could be difficult to detail and could result in constructability (clearance) issues. The connections in shorter beams become penalized. These are a few of the issues.

It is therefore ideal to have the forces indicated on the structural design drawings to eliminate these potential pitfalls. According to the 2000 AISC *Code of Standard Practice* (www.aisc.org/code), Section 3.1.2, all loads and forces must be indicated on the structural design drawings. In addition, the drawings must state whether LRFD or ASD is to be used in the selection or completion of connection details. Alternatively, connections can be designed by the engineer of record.

Sergio Zoruba, Ph.D.
AISC's Steel Solutions Center

Column Splice Gaps

We have gaps at our column splices and are aware that a gap of $\frac{1}{16}$ " or less can be ignored. Is there something in writing that discusses the philosophy behind this statement?

Question sent to AISC's Steel Solutions Center

Gaps in column splices $\frac{1}{16}$ " or less are specifically allowed by the AISC *Specification*. Referring to Section M4.4 of the 1999 AISC *LRFD Specification* (a free download from www.aisc.org/lrfdspec), it states:

Lack of contact bearing not exceeding a gap of $\frac{1}{16}$ -inch (2mm), regardless of the type of splice used, is permitted.

Also refer to AISC FAQ 3.3.3 at www.aisc.org/faq.

The Commentary to Section M4.4 mentions that the $\frac{1}{16}$ " spliced configuration was full-scale tested at the University of California-Berkeley by Popov and Stephen in the mid-1970s. Columns with splice gaps of $\frac{1}{16}$ " resulted in no reduction in load-carrying capacity compared to unspliced columns.

A paper published in AISC's *Engineering Journal*, "Capacity of Columns with Splice Imperfections" by Popov and Stephen, can be downloaded from www.aisc.org/ej.

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Panel-Zone Web Shear

For the 1997 AISC Seismic Provisions, can an OMF connection be designed with panel-zone web shear as the primary source of inelastic deformation or as a way to utilize the exemption for the maximum force that can be transferred by the system?

Question on SEAJNT list server

The goal of an OMF connection in the 1997 AISC Seismic Provisions is to provide for 1 percent plastic rotation through controlled inelastic deformations. A prescribed detail is provided that can be used, but it is not normally going to have panel-zone web shear as the primary source of inelastic deformation. The other option given is for the use of a tested connection, which means tested in accordance with the requirements in Appendix S.

Let's assume the detail proposed is so qualified as a tested connection. If the testing upon which it is based shows that the required inelastic rotation is reliably achieved through a mechanism such as shear yielding of the panel zone, it seems to me the connection is acceptable.

To me, the exception for the maximum force that can be transferred is something entirely different. This approach generally is not supposed to be used by looking at elements that are a part of the framing that is a part of the seismic-force-resisting system. Rather, the maximum force exception provides a way to examine parts of the properly designed system other than those in the seismic-force-resisting system and find maximums that can't be exceeded credibly without first failing them. Theoretically speaking, that is something that cannot happen if they are properly designed unless the ground motion exceeds the design value.

The most common example I've given to illustrate an appropriate maximum force exception is a building on spread footings having zero capacity to resist uplift but properly designed to resist the effects of overturning. The forces and moment in the seismic-force-resisting system would never see a force greater than that corresponding to the overturning of the building (foundation uplift) unless the design ground motion were exceeded. The same building on a pile system with tension capacity would not benefit from such a system limit.

Charlie Carter, P.E., S.E.
American Institute of Steel Construction

Cracks over Composite Beams

Do concrete cracks over composite beams affect the composite action of the beam? How can such cracking be prevented?

Question sent to AISC's Steel Solutions Center

The cracks you see are shrinkage cracks and do not affect the strength of the composite beam because the concrete area

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in compression is still engaged by the embedded shear connector despite the crack. Essentially, cracking is a serviceability issue as discussed in the last paragraph of Commentary Section I3.2 in the 1999 AISC LRFD Specification.

One way to minimize cracking is to place supplemental reinforcing over the girder. The recommendation in the Commentary cited above is to provide uniformly distributed transverse reinforcement of at least 0.002 times the concrete area in the longitudinal direction.

Bill Liddy
AISC's Steel Solutions Center

Weak-Axis Lateral-Torsional Buckling

Question sent to AISC's Steel Solutions Center

In the April 2004 issue of *Modern Steel Construction*, the following Q/A was published:

True or False: A shape bent about its weak axis must be checked for lateral-torsional buckling.

The answer given was:

False: Lateral-torsional buckling is a phenomenon that occurs only when rotation would produce a lower energy position for that shape. A shape bent about its weak axis is already in its lowest energy position.

While this answer is most often correct, there is a special situation where it is not—that of a load acting above the shear center. In this case, lateral-torsional buckling of a shape bent about its weak-axis can cause a significant decrease in the potential energy of the loads and a reduction in capacity.

A possible example is that of lipped trough girders, with loads at the level of the lips which are well above the shear center below the bottom flange. Trough girders often are used with composite reinforced-concrete decks. Such girders have been said to have failed during construction during the pouring of the concrete deck.

Other similar examples include circular or square hollow section beams which are loaded high above their shear centers.

Information for predicting the elastic lateral-torsional buckling for these examples is given in a number of publications, including Section 7.6 of *Flexural-Torsional Buckling of Structures* by N.S. Trahair (E&FN Spon, London, 1993).

Nick Trahair, Emeritus Professor
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Sloped Column Continuity Plates

If two moment girders frame into a column joint at offset elevations (3" or so), should skewed continuity plates in the column web adjoin the adjacent girder flanges, top and bottom and each side of the web (4 total), or is it better to add flat/level continuity plates at each girder flange, (8 total)?

Question sent to AISC's Steel Solutions Center

The best approach is to eliminate offsets in the design phase. Thinking the connection details through before select-

ing members is always a good idea and could help eliminate these kinds of connection difficulties later. But, it's not always possible to avoid such a detail, so...

For a small offset like this, sloped continuity plates will reduce the cost by about 50 percent over twice as many flat/level stiffeners. For larger offsets, a set of flat/level partial-depth stiffeners could be viable and more cost effective.

Additional information is found in AISC *Design Guide 13: Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (available from www.aisc.org/bookstore, or if you are an AISC Professional Member, download your free copy from www.aisc.org/epubs.) Refer to Chapter 5 in the design guide for design recommendations for both approaches.

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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.

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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:

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