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.

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:

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Tensile Area for Threaded Fasteners

Why is the tensile stress area larger than the minimum root area for threaded fasteners? (See p. 4-147 of ASD 9th Edition) Shouldn't I be using the smallest cross section for my design?

Bolt design is based upon the nominal area. The reductions to account for threading are already taken in the design strengths (LRFD) and allowable stresses (ASD).

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Steel Sheet Piling

I would like to obtain a recent copy of the *US Steel Sheet Piling Design Manual*. I have an old copy (sixties vintage) but would like to obtain a more current edition Can you direct me to a source?

Roger Hove, P.E.

US Steel is currently part of the USX Corporation, www.usx.com. You can find the US Steel site and related contact information as well as product information at

www.usx.com/corp/ussteel/index.htm

It is my understanding that another big player in sheet piling (among others) is Nucor-Yamato Steel. They can be reached at 870/762-5500 or 800/289-6977 or www.nucoryamato.com.

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Semi-Rigid Connections

In the design of semi-rigid connections in steel frame buildings I have typically used moment magnitudes generated by wind or seismic loading, based on chapter four of AISC's *Engineering for Steel Construction*.

In the paper "Design Tables for Top- and Seat-Angle with Double Web-Angle Connections" (Kim and Chen, *Engineering Journal*, 2nd quarter 1998). they used ½ the plastic moment of the beam as the design moment for connection (six times my wind moments).

Is this a limit state criteria? Should I redesign my connections based on this? Is this approach overkill for buildings less then 35′ high and in low seismic regions?

Your approach is consistent with usual practice for flexible wind connection design. The Kim and Chen paper just used 50% of the fixed end moment as a way to get an end moment for their design example. There's no requirement to use that as a minimum, though.

The design approach for flexible wind connection systems is the same in LRFD and ASD, so it's not a limit-state criterion. It's just a convenience used by the authors in writing their paper. Given that wind moments may be much lower than 50% of the fixed-end moment, particularly for the beams in flexible moment connections or frames, which are sized for gravity loads as simple beams, it may be overkill as you mentioned. Personally, I'd use the analysis value as is customary in historic practice.

Flexible wind connections are a simplified approach to PR connections and were popularized by Robert Disque (an AISC alumnus) in the early 1960s. Disque recommended that the beams and beam end shear connections be designed for gravity as simple beams, the flange connections be designed for the wind moments from a portal (or similar) analysis, the columns be designed for gravity plus wind moments, and the details be such that there is enough inelastic deformation capacity to avoid over-stressing fasteners or welds. There are two papers he wrote on it in the AISC *Engineering Journal* (3rd quarter 1964 and 1st quarter 1975). The overall frame stability of flexible wind connection system was addressed by Gertsle and Ackroyd, also in the AISC *Engineering Journal* (1st quarter 1990). Their main conclusion was that the system was OK up to 10 stories, within other limits as described in that paper. It may be interesting to note that the steel frame of the Empire State Building was designed this way. However, there is also significant masonry infill and core bracing in that building, so it's not really just a flexible wind connection system. Bob Disque told me he thinks flexible wind connections are best suited for two stories or less.

Although these kinds of systems have always been talked about as flexible *wind* connection systems, I personally think they could be applied for seismic as well, provided the *R* factor is not taken greater than three. Perhaps we should call them flexible moment connections to be more generally applicable.

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question from February 2001:

More on Jam Nuts

Several readers have written to let us know that most references and manufacturers recommend that jam nuts be placed under the structural nut. This practice is supported by references and recommendations from the Industrial Fasteners Insititute (IFI).

We must clarify, however, that jam nuts are *not* needed for structural connections with ASTM A325 or A490 bolts and therefore are not covered in the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (2000), available for free downloading at www.boltcouncil.org. Jam nuts are of interest, however, with some ASTM A307 bolts where it is necessary to prevent nuts from backing off in vibratory or similar applications.

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Bolt Shear/Tension Interaction

The equations for bolt shear and tension interaction in table J3.3 (ASD specification) for A325 and A490 bolts give an elliptical interaction curve. I am wondering if I can apply the same type of modification to these equations as was done in the LRFD specification, in which the elliptical interaction line is simplified into 3 straight lines giving "minor" deviations. I would like to do the same to the ASD equations to be able to graph them easier. What are the implications in making such a conversion?

Zachary Goswick, EIT

The elliptical interaction curve and three-straight-line approximation of the curve are equally valid approaches to the design of bolts for combined shear and tension. The advantage of the elliptical equation is that it is a continuous fuction—good for computer programming. The advantage of the three-straight-line approach is that you don't have to drop the bolt shear strength when you have a very small tension (and vice versa). In the end, the joint design will be very similar with either approach.

The development and basis for both approaches is given in a paper I wrote with Ray Tide and Joe Yura. You can find it in the 3rd quarter 1997 AISC *Engineering Journal*. It is titled "A Summary of Changes and Derivation of LRFD Bolt Design Provisions."

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New Questions

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

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