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: 312.670.9032
solutions@aisc.org

BEARING STRENGTH

In the 3rd Edition *LRFD Manual*, example 10.11 computes the required supported beam web thickness based on bearing strength without eccentricity. Since the current design criteria developed by Astaneh for single-plate shear connections assigns eccentricity to the bolts for shear, this seems somewhat unconservative. Why is eccentricity not considered for the bolts in the bearing check?

Question sent to AISC's Steel Solutions Center

The AISC design criteria does not use eccentricity in bearing checks because single-plates are designed to deform at the bolt holes, thereby eliminating consideration of eccentricity. You do need to consider eccentricity for shear to ensure the bolts can plow, but not for bearing.

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

ASD AND SEISMIC PROVISIONS

I am designing a steel framed building per the 2000 IBC using ASD for steel design, so I am looking at Part III of the 1997 AISC *Seismic Provisions*. My question is specifically regarding the factor to apply to E in equations 4-a and 4-b.

The AISC *Seismic Provisions* commentary discusses the factor and from that it seems to me that I should use E/1.4 but the commentary specifically references only equations 4-1 and 4-2 regarding the factor to apply to E. E is not directly in equations 4-1 and 4-2 and it seems that the logic of using E/1.4 would apply to all ASD equations.

As I see it, the E/1.4 is simply because the IBC seismic forces are limit states based and I am designing to ASD, so I logically should be reducing the "strength level" seismic forces to "service level" seismic forces by dividing by 1.4. Am I looking at this correctly?

Question posted on the SEAINT.ORG list server

Seismic design is a two-step process.

1. The members are sized in accordance with the Applicable Building Code load combinations and applicable material specifications. Depending on the material, members may be sized by either allowable stress or strength methods. Most building codes provide load combinations for both basis. The earthquake load E is always defined on a strength basis. That is why the load factors for strength design are always 1.0 and for allowable stress design are either 0.7 or 1/1.4.
2. The members and connections are detailed in accordance with the applicable seismic material specifications, AISC *Seismic Provisions* for steel, ACI 318 Chapter 21 for concrete, etc.

For your specific situation with steel materials, the AISC *Seismic Provisions* define all detailing requirements on a strength basis, regardless of whether you use Part I or Part III. It is easy to see the strength basis in Part I because of the references to LRFD. In AISC *Seismic Provisions* Part III the strength basis is reached by multiplying allowable stresses, times 1.7, times defined resistance factors (no 1/3 stress increases are to be used.) Part III is presented as a convenience to those who have sized the members of the Seismic Load Resisting System using ASD, allowing them to calculate the strength basis for seismic detailing using allowable stresses that have already been determined.

Rick Drake, S.E.
Fluor Daniel
Aliso Viejo, CA

BOLTS IN EXTENDED-END PLATE CONNECTIONS

In Example 12.4 of the 3rd Edition *LRFD Manual* pertaining to the design of a four-bolt unstiffened extended end-plate FR moment connection, ASTM A325 slip-critical bolts were specified. Shear is transferred through slip resistance offered by surfaces of end plate and column flange. Is this the reason why full tension capacity of the bolt is used per section J3.6, instead of capacity under combined shear and tension condition per J3.7?

Is it mandatory to use a slip-critical bolts instead of pretensioned bolts for this type of connection? What about a snug-tightened arrangement for this type of connection?

Question sent to AISC's Steel Solutions Center

The example you cited is based on the design criteria established by Murray. As stated in AISC's *Design Guide 4: Extended End-Plate Moment Connections*:

"Connections of this type...in which some of the bolts lose a part of their clamping force due to applied tension suffer no overall loss of frictional resistance. The bolt tension produced by the moment is coupled with a compensating compressive force on the other side of the axis of bending. Thus, the frictional resistance of the connection remains unchanged and the shear/tension interaction can be ignored. "

End-plate moment connections may use slip-critical, pretensioned as well as snug-tightened bolted joints. Please note that these connections are currently limited to ASTM A325 bolts. Refer to the paper entitled "Use of Snug-tightened Bolts in End-Plate Connections" by Murray et al. in the publication *Connections in Steel Structures II* (1992) for additional information. Contact the AISC Steel Solutions Center (solutions@aisc.org) for a complimentary copy of the paper.

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

SINGLE-PLATE SHEAR CONNECTIONS

Table 4-8 in the AISC *HSS Connections Manual* contains support conditions characterized as rigid and flexible for single-plate shear connections. Please explain what flexible and rigid supports imply?

Question sent to AISC's Steel Solutions Center

The following is a description of flexible supports for shear tabs in the latest *LRFD Manual*, available at www.aisc.org/lrfd:

"A flexible support possesses relatively low rotational stiffness and permits the adjacent simply supported beam end rotation to be accommodated primarily through this supporting member's rotation. Such an end condition may exist with one-sided beam-to-girder-web connections or with deep beams connected to relatively light columns."

For rigid supports, it states:

"In contrast, a rigid support possesses relatively high rotational stiffness, which constrains the adjacent simply supported beam end rotation to occur primarily within the end connection, such as a beam-to-column-flange connection or two concurrent beam-to-girder-web connections."

The same concepts apply for shear tabs with HSS columns.

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

THREADS IN TENSION RODS

In the August 2002 issue of *Modern Steel Construction* issue you describe in *Steel Interchange* how anchor rods are threaded. Would the same hold true for tension rod bracing?

Question sent to AISC's Steel Solutions Center

Yes, the tension rod bracing may also be threaded by cutting or rolling threads. Unless an upset rod is used, the strength values of the rods are based on the root area, which is identical for both methods of threading.

However, there will be a difference in elongation under load as follows. In general, the rod elongation Δ for a load P on a length L is:

$$\Delta = PL/AE$$

For a 2" diameter rod with cut threads, which has a minimum 1.96" body diameter per ASTM F1554:

$$\Delta = 1.04PL/E$$

For a 2" diameter rod with rolled threads, which has a minimum 1.84" body diameter per ASTM F1554:

$$\Delta = 1.18PL/E$$

Thus, although there is identical strength, there is a decrease in axial stiffness when rods with rolled threads are used, versus rods with cut threads—a 13.5% decrease in this particular case. Therefore, it is a design consideration which type of threading is to be used. If strength controls the design of the rods, I don't think there is an issue. If drift controls the design, there may be an issue. This is an aspect that I think should be considered by the engineer of record (EOR) in the design of the rods.

With this in mind, the designer should specify in the contract documents either a design based upon rolled threads or that cut threads be provided. If cut threads are specified, the fabricator should provide the rods with cut threads. Though, a slightly larger diameter rod with rolled threads might be acceptable to the Engineer as an alternate.

If the type of threading is unspecified, ASTM F1554 section 6.2 allows that the manufacturer can provide either. If the rods are of another specification, such as ASTM A307, there are similar provisions in the ANSI/ASME B1.1 threading specification that is referenced therein.

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

SINGLE-ANGLE GEOMETRIC PROPERTIES

Please advise as to where one may find the value for I_w for single-angles for equal and unequal legs. Alternatively, if not directly given by AISC, how can one calculate this property? Is it a trigonometric function of I_z?

Question sent to AISC's Steel Solutions Center

There was a paper published in *Engineering Journal* entitled "Tables for Equal Single-Angles in Compression" by Walker (second quarter, 1991) that contained some rather nice, simple geometric identities for most single-angle geometric properties. It can be downloaded from the *Engineering Journal* section of the AISC website at www.aisc.org. By the way, the paper shows that $I_w = I_x + I_y - I_z$.

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