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.

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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:

Steel SolutionsCenter
One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.423.4651
solutions@aisc.org

COLUMN CONNECTION ECCENTRICITY

from October 2002

In exterior columns, should the eccentricities resulting from the beam connections be considered when the connections are not designed by the SER? For example, when a W-shape beam is framed into an exterior HSS column via a single-plate shear connection, the column could be designed for the eccentricity equal to the distance from its centerline to the bolt line. With that approach, it might make sense not to place the beams at the column lines and laterally brace the column by a light angle section. Alternatively, the beams could be assumed to extend into the column centerlines and the specialty-connection design engineer directed to design the connection for combined shear and moment. Can the AISC *ASD Manual*'s tables for single-plate shear connections or eccentric bolted connections be used for that purpose?

If the eccentricity is considered in column design, it should presumably be applied in two directions in corner columns and in columns where the exterior girders deliver vastly unequal reactions from the opposite sides. This may lead to the corner columns actually being heavier than the interior columns, which support four times the load.

Alexander Newman, P.E.
Maguire Group Inc.
Foxborough, MA

It is not a given that eccentricities such as those described need to be considered in the design. Ioannides ("Minimum Eccentricity for Simple Columns", *ASCE Structures Congress Proceedings,* Volume 1, 1995) demonstrated that normal connections also provide restraint to the column as they load it—even when connected to one side only—and mitigate the eccentric effects in normal framing configurations. If it is decided based upon engineering judgment that eccentricities must be considered, I recommend that the member be designed for the eccentricity. My reasoning is that it is much more economical to add weight to the column than to complicate the labor-intensive (and therefore more costly) connections.

Regarding what combinations of eccentricities should be used, this is a matter in which the engineer will have to use judgment. But if eccentricity is considered, it is entirely possible that the column size might increase beyond that of an interior column carrying four times the axial load. Could this be further empirical evidence justifying the historic practice of designing columns for axial load only?

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

WELD ACCESS HOLES

FEMA 350 recommends a special weld access hole configuration for use with certain moment connections for ordinary and special moment frames. It is my understanding that this weld access hole configuration will also be included in the 2002 *AISC Seismic Provisions.* As you know, there has been an allegation that use of this weld access hole configuration constitutes an infringement of the patent for a proprietary slotted-web moment connection. What is AISC's position on this issue?

C. Mark Saunders, S.E.
Rutherford and Chekene Engineers
San Francisco, CA

AISC does not agree that use of the special weld-access hole configuration constitutes a patent infringement and has published the FEMA 350-recommended weld access hole configuration in the 2002 *AISC Seismic Provisions.* We will defend both our right to publish this information and the rights of the design community and construction industry to make use of it.

Louis F. Geschwindner, P.E., Ph.D.
American Institute of Steel Construction, Inc.
Chicago, IL

CIRCULAR BASE PLATE DESIGN

from September 2002

I would like to design circular column base plates. However, there appears to be little or no information on the subject. Does anyone know of papers, articles or design guidelines for the design of circular base plates?

Question sent to AISC's Steel Solution Center

For base plates subjected to bending with tension taken by the bolts and compression by the base plate against the substrate the bearing stress diagram would be that of an "Ungula of a Right Circular Cylinder" (See 2.09 (c) of the *Engineering Mathematics Handbook* by Jan J. Tuma and Ronald A. Walsh, McGraw-Hill, 4th Edition, 1998). For this case as well as a uniformly loaded base plate I don't know why one couldn't conservatively assume a one inch wide strip of plate controls at the highest point of stress at the furthest perpendicular distance from the face of the column as the design basis for determining the minimum plate thickness.

Matthew Stuart, P.E., S.E.
Schoor DePalma Engineers and Consultants

One applicable reference is *Process Equipment Design: Vessel Design* (Lloyd E. Brownell and Edwin H. Young, John Wiley & Sons Publishing, 1959). A base plate program is also available at www.mecaconsulting.com based on this reference.

Rey Velasco
Manila, Philippines

SEISMIC FORCE REDUCTION FACTOR

Regarding the seismic force reduction factor *R*, there seems to be some discrepancies in values to use between the 1997 UBC and *Supplement No. 2* of the *AISC Seismic Provisions for Structural Steel Buildings.* For example, for ordinary moment frames (OMF), the 1997 UBC requires *R* = 4.5 while *Supplement No. 2* indicates *R* = 3.5. Which is correct?

Question sent to list server at www.seaint.org

Basically, the UBC provisions are out of date with the *AISC Seismic Provisions.* As a result, the *AISC Seismic Provisions* and UBC have different levels of detailing (energy dissipation capacity) associated with the respective OMF connections that go with the two *R* values.

SMF, IMF and OMF have evolved a bit over the past few years. During the SAC Steel Project, it was conceived that an SMF should be good for an *R* of 8 and based upon testing to achieve an inter-story drift of 4 percent (of which approximately 3 percent is inelastic). Similarly, an OMF should be good for an *R* of 4 either in prescriptive form as given in the *AISC Seismic Provisions* or based upon testing to achieve an inter-story drift of 2 percent (of which approximately 1 percent is inelastic). And to allow for an intermediate condition that didn't make 4 percent but exhibited good behavior, an IMF was included with an *R* of 6 and based upon testing to achieve an inter-story drift of 3 percent (of which approximately 2 percent is inelastic). This system was incorporated into the *AISC Seismic Provisions,* NEHRP Provisions, and IBC draft at the time.

Thereafter, it became obvious from the wealth of steel connection testing that assemblies either performed very well (i.e., achieved SMF qualification) or were only good for OMF status. Actually, the testing to date has shown that, with proper design and fabrication/erection, it's hard to configure a moment frame that would not perform acceptably as an SMF—and that OMF performance is also easily achieved with very basic connection detailing. The IMF category thus seemed like a white elephant and it was considered that the IMF should be removed.

Along the way to that conclusion, however, it came into favor that the IMF and OMF should instead be recategorized. The OMF was recast as the prescriptive form given in the *AISC Seismic Provisions* (note: this is NOT just the pre-Northridge connection; it has significant improvements to welding and configuration, backing bar treatments, web detailing, etc.). The IMF was recast as a tested assembly like the old OMF with a higher *R* and inelastic drift demand. This was incorporated into *AISC Seismic Provisions Supplement No. 2.*

On the UBC side of things, the code is just behind the times relative to what has happened already in the *AISC Seismic Provisions.* So the *R* factor you see corresponds to connection detailing requirements that were loosely consistent with older versions of the *AISC Seismic Provisions.*

Long term, the transition to the IBC or NFPA document will take place and these annoying stutter steps in code progression will disappear. For now, just make sure you properly match the selection of *R* to the corresponding detailing requirements.

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

NEW QUESTIONS

HOLLOW STRUCTURAL SECTIONS AND PIPES

Is it true that I can take advantage of the 2000 *LRFD HSS Specification* for structural designs involving ASTM A53 Grade B pipe? What, then, are the differences between HSS and pipe if both use the same specification?

EXTENDED END-PLATE CONNECTIONS

Symmetric tension bolt pitches are assumed in the published design procedures for this connection. However, due to ease of fabrication, we would like to use a different pitch above and below the top tension flange of the beam. Are there guidelines on this, or has this connection been prequalified for only symmetric pitches?

TEES UNDER FLEXURE (STEM IN COMPRESSION)

How does one design a structural WT member under flexure when the stem is in compression? Chapter F of the 1989 *ASD Specification* does not appear to address this particular case.