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! Send your questions or comments to solutions@aisc.org.

Base Anchorage in Seismic Zone

Should anchor rods be designed for base shear forces only, not amplified loads, using the Ω factor and not the vertical or horizontal component of the $R_yF_yA_g$ force of the brace?

We do a lot of one-story commercial buildings where the base shear forces are quite small and we are being questioned by fabricators, contractors, architects, etc. over huge gusset plate connections that result from the $R_yF_yA_g$ of the tube braces. The footing, anchor rods, and even column sizes seem disproportionate to the brace connection. Where does the force go if you ignore it in the design of these members?

We are under the 1999 *Standard Building Code*, which does not differentiate between special and ordinary concentric braced frames (except through referring to ASCE 7). This code refers to the 1997 AISC *Seismic Provisions*, which would technically allow us to ignore the seismic provisions for one- and two-story structures. However, we have been trying to use the 2002 provisions to be more up-to-date. The 1997 provisions allowed for the brace connection for OCBF to be designed for load combinations 4-1 and 4-2, but we see that this was taken out in the 2002 provisions for OCBF, along with the one- and two-story exception. The SCBF still allows the connection to be designed for the maximum force that can be transferred by the system. If you design the foundations for the base shear and let that limit the system, then aren't you basically cancelling out the requirement for the connection to be designed for the $R_yF_yA_g$ force, which effectively puts you back at square one?

We asked Rafael Sabelli, a presenter of AISC's seminar on the seismic provisions and new seismic manual, to respond:

I believe that the stated assumptions are very good. Essentially, my view is that the engineer needs to understand how the structure will yield and design accordingly. In the case of low structures (one- and two-story), the "yielding" may be rocking—in other words, uplift of the spread footings. It is theoretically unnecessary to design elements of the structure for more force than the rocking capacity, but I would suggest that the engineer bear a few things in mind:

1. Rocking leads to large displacements, and the "stiff" CBF quickly develops very high drifts. In my own judgment, I would not to permit rocking at the design base shear for SCBF (R = 6), but I think it is OK at the base shear for OCBF (R = 3.25 in ASCE 7-05).
2. A true upper-bound rocking capacity is difficult to quantify. I would use a minimum safety factor of 1.5 on calculated footing and grade-beam weight to estimate the upper bound.
3. Taller structures (three stories and up) have been shown to yield even after rocking. Thus I would still design the steel structure to have the appropriate ductility (i.e., I would not waive the AISC 341 requirements).

So, in short, I would design the anchor rods for either the strength of the structure or the hold-down capacity of the footing (with the safety factors I feel comfortable with). For low buildings, I'd try to make sure there is enough bracing so that rocking does not occur at a very low force level.

Rafael Sabelli, S.E.
DASSE Design, San Francisco

Backing Bar Removal

We have a steel building frame where we are using ordinary moment frames (OMF) with a seismic force resisting system defined as "Structural steel systems not specifically detailed for seismic resistance," R = 3. Since this category does not require compliance with the AISC *Seismic Provisions for Structural Steel Buildings*, which requires removal of backing bars for OMFs, I would surmise that we do not need to remove the backing bars. Is this correct?

The acronym OMF is usually reserved for a seismic load resisting system with R > 3, as defined by the IBC model building code and/or ASCE 7. In your case, you really have a conventional moment frame designed to the requirements in the AISC *Specification for Structural Steel Buildings* with R = 3 and not an OMF. Hence the backing bar removal requirement found in the AISC *Seismic Provisions* does not apply.

Since you are not bound to the detailing requirements of the AISC *Seismic Provisions*, the AISC specification requirements would then be the referenced document. The AISC specification references AWS D1.1 *Structural Welding Code – Steel*, which states:

Steel backing for welds in statically loaded structures (tubular and non-tubular) need not be welded full-length and need not be removed unless specified by the Engineer.

There may be some instances where it is desirable (or necessary) to remove the backing bars to facilitate an inspection process. You may want to discuss this with the inspection agency early in the project if possible, in order to include the information in the bid documents.

Kurt Gustafson, S.E., P.E.

Moment Strength of Bolt Group

On p. 7-19 of the 13th edition *Steel Construction Manual*, an equation is given for the pure moment capacity of a bolt group when the instantaneous center is at the center of the bolt group. Where does the 1.25 in the equation come from? What is the design capacity using LRFD?

To better fit test data, it was decided to increase the bolt strength by 1/0.8, or 1.25 when checking eccentricity. This is because the Chapter J bolt values include a 20% reduction in available strength to account for uneven load distribution in long bolted connections. In most flexural connections, you will not have an extremely long connection, and this reduction is removed so that the method better reflects the available strength.

The bolt group strength is used in the design procedure for extended single plate connections to determine a maximum plate thickness. This plate thickness limit ensures that plate yielding will be more critical than bolt shear—that is, a ductile failure mode exists in the connection to accommodate the rotation required of a simple shear connection. It is used similarly in both ASD and LRFD.

Sergio Zoruba, Ph.D., P.E.

Shear Stud Placement

For design of composite steel beams, is there a good reason to consider the use of a non-uniform spacing over the span? I have heard of placing more at the ends where the shear is higher. Would that be reasonable?

Section I3.2d(6) of the 2005 AISC specification allows the placement of shear connectors as uniformly spaced on each side of the point of maximum bending moment and the adjacent points of zero moment. This is a simplification that recognizes that the shear strength is present and there is adequate capacity to deform to develop it when the studs are not concentrated toward the zones of higher shear. The phrase "unless otherwise specified" is included in that section, however, and allows for non-uniform spacing, which may be desirable in some cases, such as when uniform placement is not possible due to deck flute locations. In such cases I have seen engineers specify that more studs be placed near the high shear point.

Kurt Gustafson, S.E., P.E.

Additional Capacity Needed

I am designing a new stair that will be supported on an existing structural steel composite floor system. The beams are designed very close to their limits, so I must reinforce them. When I attempt to add steel (WT, C, L) to the bottom flange of the beam, the percentage composite drops below 25%, and I lose the ability to consider the beam composite, which means I would have to add a lot more steel. Has AISC done any research on adding studs to a composite beam to increase its composite action capability in order to accommodate additional loads?

Although the AISC *Specification* and *Manual* do not address it, additional shear studs can be installed to increase composite action. Refer to AISC FAQ 4.5.5 at **www.aisc.org/faq**, which discusses the coring of holes through the slab to allow placement of additional studs. Such a modification can be designed by the EOR, with consideration of such things as the strength of the existing concrete and the grout used to fill the holes around the added studs.

Sergio Zoruba, Ph.D., P.E.

Bearing Length

I am trying to design an unstiffened seated connection using Table 10-6 in the 13th edition manual. What is the definition of the required bearing length N_{req} (in.) shown in the table? Is this referring to the actual beam bearing length on the angle, and if so, why do the allowable load values decrease as the bearing length increases?

The N_{req} listed in Table 10-6 is the required bearing length to satisfy the *Specification* limit states rather than the actual bearing length on the angle. Thus if the required bearing length increases, so will the eccentricity of the reaction on the seat angle, resulting in a lower strength.

Kurt Gustafson, S.E., P.E.

Column Table Using r_x/r_y

I recently attended a seminar on the new manual, which included a study book outlining the seminar and including examples. I have been trying to familiarize myself with the axial compression section of this book (Chapter 2). An example uses a W14×120 column checking P_a using equation E3-2. This result is then checked against the capacity listed in Table 4-1. The results come out the same. I did a second problem using a W12×72 following the same approach and am coming up with significantly different values. I am not sure that I am using r_x/r_y correctly. Can you explain the use of the r_x/r_y factor in assessing the column capacity?

The ratio r_x/r_y is used as a convenience to permit the tables in the manual to be simplified. The tables assume that buckling about the *y*-axis controls, and the parameters must be adjusted when buckling about the *x*-axis is more critical.

Usually, the r_x/r_y ratio is used because the unbraced length is different about the *x* and *y*-axes. If you are using the specification equation, you would need to check which *KL/r* controls and determine the capacity accordingly. If you are using the Table and the KL/r_x controls, you will need to manipulate the unbraced length at which you enter the Table with the ratio r_x/r_y. This manipulation allows you to enter the Table, which is based on the *KL* with respect to the least (weak) radius of gyration r_y, and determine the correct axial strength with respect to the strong-axis radius of gyration r_x.

Kurt Gustafson, S.E., P.E.

Kurt Gustafson is the director of technical assistance, and Sergio Zoruba is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is AISC's chief structural engineer, and Lou Geschwindner is AISC's vice president of engineering and research.

Steel Interchange is a forum 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.

If you have a question or problem that your fellow readers might help you 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 Solutions Center
One East Wacker Dr., Suite 700
Chicago, IL 60601
tel: 866.ASK.AISC • fax: 312.670.9032
solutions@aisc.org