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:

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OCBFS IN LOW BUILDINGS

I have a low building in Seismic Design Category D and have run into a problem designing the bracing connection in an Ordinary Concentrically Braced Frame (OCBF). There used to be an exemption for low buildings (Section 14.5 of the 1997 Seismic Provisions) in OCBFs. It is unclear to me whether or not these exemptions still exist in AISC *Seismic Provisions* Supplement No. 2. The new provisions make it look as if the engineer has to design the brace connection in this low building to develop the full tensile capacity of the brace...for serviceability reasons, my brace is a 6×6 HSS (with a tensile capacity of about 300 kips). However, the greatest design load on the brace is only about 30 kips. Does the bracing connection have to be designed for the tensile capacity of the brace? Is there a good way out of this?

Question sent to AISC Steel Solutions Center

The first question that needs to be asked is "What is the Applicable Building Code?" The 2000 IBC refers to the 1997 AISC *Seismic Provisions*, including Supplement No. 1. The 1997 UBC doesn't refer to the AISC *Seismic Provisions*. Supplement No. 2 may not be applicable.

If the 1997 AISC *Seismic Provisions*, including Supplement No. 2 (or later) is applicable for his project, the brace connection needs to be designed for the tensile capacity of the brace. Supplement No. 2 reflects a definite change in philosophy, reflecting the potentially non-ductile performance of OCBFs. The revised *Provisions* are clear, simple to use, and definitely encourage the use of the highly ductile SCBFs. The same holds true for the 2002 *Seismic Provisions*.

The OCBF strength requirements are not serviceability related, they are to assure that the connections have a strength to match the maximum load the member can deliver. From a practical standpoint, if the bracing members are so long that slenderness requirements govern the cross-section, it may not be economical to design the frame as an OCBF.

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ORDINARY AND INTERMEDIATE MOMENT FRAMES

The AISC *Seismic Provision*s Supplement No. 2 Commentary to Sections 10 and 11 regarding IMF and OMF, respectively, states "As a result of the SAC program (FEMA 2000a), the IMF as defined in the 1997 AISC *Seismic Provisions for Structural Steel Buildings* is no longer applicable. This system has been eliminated and the OMF as given in AISC (1997) has been spit into two systems: the IMF based on a tested design procedure and the OMF based on a prescriptive design procedure." My question is that IBC 2000 currently prescribes an *R* value of 4 for an OMF and 6 for an IMF. This commentary seems to suggest that the level of ductility provided by an OMF and IMF are comparable and, therefore, the *R* values should be approximately the same. What are the thoughts of the AISC *Seismic Provisions* Committee?

I have read the summary of the Lehigh test data for the prescriptive connection and, if I recall correctly, the level of ductility available within this connection if the current requirements are followed (weld metal meeting CVN toughness requirements, backer bar removal and welding requirements, welded beam web, good access hole geometry, and continuity stiffener requirements) is pretty high.

Question sent to AISC Steel Solutions Center

The "modern" AISC *Seismic Provisions* started with:

Type	R	Min. total story drift (corresponding radians inelastic)
SMF	8	4% (0.03 rad)
IMF	6	3% (0.02 rad)
OMF	4	2% (0.01 rad)
Normal	3	No requirement (this is the LRFD or ASD type moment frame, which would generally have numbers like 1% and nominally zero rad — elastic)

IMF were included at that time because it was postulated that some tests in SAC would be good, but not make 4%. A hedge category of sorts.

After SAC, we now know that connections either work or they don't, and IMF was an unnecessary category as conceived. It was proposed to eliminate it. Instead, IMF morphed into a tested version of the circa-1997 OMF (with a reduced *R*) and OMF morphed into a prescriptive version of the circa-1997 OMF (with a reduced *R*). The levels of ductility are anticipated to be similar. You do get to take advantage of more with the current (2002 AISC *Seismic Provisions*) IMF, though, because there is testing involved.

You are also correct that the improved version of the directly welded-flange moment connection with backing bars removed, notch-tough weld metal, improved weld access hole geometry, improved web connection, etc. does often demonstrate SMF performance. There are some tests with less-than-SMF performance, so the AISC *Seismic Provisions* Task Committee (TC 9) has not seen fit to restore the improved detail to the "glory" of use as an SMF.

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TEES UNDER FLEXURE (STEM IN COMPRESSION)

from October 2002

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.

In Section 9.12, which is "Lateral Buckling of Channels, Zees, Monosymmetric I-Shaped Sections, and Tees," of the reference *Steel Structures: Design and Behavior*, fourth edition, by C.G. Salmon, and J.E. Johnson, the authors provide a discussion and also an example under the heading "Tee Sections."

In the example provided (Example 9.12.2), the moment strength of a structural tee section (a WT7×19) is investigated when the flange is in compression (this being Case 1) and when the stem is in compression (this being Case 2). The example also shows how the strength of the tee is affected by lateral bracing.

While this design procedure is based on the *LRFD Specification*, it is also applicable for structural tee sections using the *ASD Specification* with an appropriate factor of safety applied.

Timothy M. Young
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ASTM A36, A572, AND A992

An inspector on our job insists that A36 is no longer manufactured. I have recently taken an advanced steel course and heard no mention of such a fact. I understand that the A572 Specification has been refined, but I find it difficult to believe that A36 is gone altogether. If you could direct me to a reference I would greatly appreciate it.

Question sent to AISC Steel Solutions Center

ASTM A36 is still commonly used in steel plate, angles, channels, S-shapes, and HP-shapes. Your inspector has probably heard that A36 is not available in wide-flange shapes. And this is almost true. The preferred (and most common) material specification for W-shapes is now ASTM A992, which is the official title of the "refined A572" that you made reference to. A992 is a grade 50 material. Engineers can still specify A36 for W-shapes. However, A36 costs more than A992 and the steel that you get probably will also meet a specification with a higher yield strength anyway.

There are a couple of articles that may be of interest to you on AISC's web site. If you search "A992", you will get links to *Modern Steel Construction* articles such as "Are You Properly Specifying Materials", "Steel Industry Embraces A992" and "Do you 992?"

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NEW QUESTIONS

HEIGHT LIMITATIONS IN OMFS

Why has the height limitation of 160 ft for OMFs in UBC 97 been reduced to 35 feet in the IBC 2000, for structural steelwork buildings in Seismic Design Category (SDC) D? I can't point to an exact reason, but commentary from some of the steel seismic seminars leads me to believe that AISC wants people use special frames of all types for almost everything (except maybe SDC A and B). I would expect the penalties to keep going for using ordinary frames in zone with moderate seismicity as well.

HSS MEMBERS IN SEISMIC DESIGN

Are there any types of moment resistant HSS (beam) to HSS (column) connections that could satisfy the requirements for Special Moment Frames? Are there any types of HSS (beam) or HSS (column) to HSS (brace) connections that could satisfy the requirements of Concentrically Braced Frames?

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SIZING WASHERS

Given the correct loading conditions, it is possible for nuts on anchor rods to pull through anchor rod holes or, when leveling nuts are used and the column is not grouted, for the base plate to push through the leveling nuts. Both failure modes may occur when a washer of insufficient size (diameter and thickness) is used. What procedures are available for properly sizing the washer to help prevent such failures?