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!

Axial Compression Capacity

The example column design problem from Charles Page's article in the November 2005 issue ("SpecWise: Design Examples," available at www.modernsteel.com) indicated that the 13th Edition *Manual of Steel Construction* lists an LRFD axial compression capacity of 892 kips for the W-shape. Looking up the same column shape and effective length in the LRFD third edition manual, I found that the compression capacity is 844 kips. This is a +6% of capacity. I did not know that increased capacity was incident to this specification change. Is that the case?

Question sent to AISC's Steel Solutions Center

Yes, the resistance factor (phi) and safety factor (omega) were changed for columns in the 2005 AISC specification.

In previous LRFD specifications, phi was equal to 0.85. In previous ASD specifications, the safety factor was approximately 1.76. These values were set based upon a variety of products, including columns that might be fabricated from universal mill plates. In fact, UM plates were the controlling material and dragged the phi down (the safety factor up) all by themselves.

For the 2005 AISC specification, we recognized that UM plates are no longer available and eliminated them from the determination of the resistance factor and factor of safety. As a result, phi is 0.9 and omega is 1.67 in the 2005 AISC specification; hence, the difference in strength you noted.

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

Load Tests

Does AISC have a test procedure for performing a load test on in-place steel? If yes, what publication is it in?

Question sent to AISC's Steel Solutions Center

AISC covers requirements for the evaluation of existing steel structures in Appendix 5 of the 2005 AISC specification (a free download from www.aisc.org/2005spec). You may also want to refer to Section 1713 of IBC 2003 for information pertaining to in-situ load test requirements.

Sergio Zoruba, Ph.D., P.E.
American Institute of Steel Construction

C_b Factor for Frames Braced Against Joint Translation

Why does the ASD ninth edition manual require the use of C_b = 1 when computing F_{bx} to be used in equation (H1-1) for frames braced against joint translation? Generally, the columns in frames braced against joint translation will have bending moments in-span smaller than bending moments at the ends. On the surface it would seem to be an appropriate application of the C_b term as a reflection of moment gradient.

Question sent to AISC's Steel Solutions Center

The reason for this requirement is that in Eq. H1-1 we are allowed to use $C_m = 6 - 0.4M1/M2$ to account for the effect of moment gradient when the frame is prevented from sidesway. C_m is approximately $1/C_b$. Using both in the interaction equation would be "double dipping," and it would result in an unsafe result. You can use either F_b/C_m or F_bC_b, but not both. In either case, it is F_b that is modified and not f_b.

Theodore V. Galambos, Ph.D.
University of Minnesota

Cambered Beam Connection

Is it preferable to provide short-slotted holes on at least one end of cambered floor beams? I have been told that this practice allows subtle beam end rotation to take place as the concrete is placed and the camber is relieved.

Question sent to AISC's Steel Solutions Center

The end rotation for a couple inches of camber at the middle of a 30' to 40' span is very small. But some combinations of camber and span may require consideration of the rotation and detailing to accommodate it. Short-slotted holes are permitted as a means to address this as long as the direction of load is transverse to the long dimension of the short slot for bearing type connections. The slots are permitted without regard to direction of loading in slip-critical connections. Short slots are more often used to accommodate erection tolerances and can be used as described at both ends of the member.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

1925 Steel Rivets

What grade of steel rivets was used in 1925 steel construction? I am presently checking an existing steel girder's riveted connection capacity. I would like to confirm the allowable tensions (F_t) and allowable shear (F_v) of the 1925 rivets.

Question sent to AISC's Steel Solutions Center

The grade of steel rivets used for buildings in 1925 was ASTM A9–rivet steel with a tensile strength of 46,000 to 56,000 psi and a minimum yield point of one-half tensile strength, or not less than 30,000 psi. In 1932, the designation for rivet steel used in buildings was revised to the ASTM A141 Standard.

The first AISC specification, adopted in 1923, lists the following allowable values for rivets:

- Shearing on power-driven rivets–13,500 psi
- Shearing on hand-driven rivets–10,000 psi
- Bearing on power-driven rivets–30,000 psi (DS); 24,000 psi (SS)
- Bearing on hand-driven rivets–20,000 psi (DS); 16,000 psi (SS)

(*DS and SS stand for double shear and single shear, respectively.*)

There was no allowable tensile stress given for rivets in the initial edition of the 1923 specification. The 1928 revision to that specification did institute an allowable tensile stress on the area of the nominal diameter of rivets at 13,500 psi.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

Seismic Prequalified Connections

I am designing a rigid-frame structure in California. I have heard that AISC has published the 2005 version of the *Seismic Provisions* and a new *Seismic Design Manual* with a list of prequalified connections. Is it true that these new connections will allow columns to a greater depth than the 14" limited by FEMA 350?

Question sent to AISC's Steel Solutions Center

The 2005 *Seismic Provisions* is available to download free from **www.aisc.org/2005seismic**. There is also a companion standard for the prequalification of moment connections called *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, which is about to be published and will also be available to download free from **www.aisc.org**. Both will be printed in the AISC *Seismic Design Manual*.

Columns greater than 14" will be permitted up to 36" for RBS systems and should not exceed the beam depth for end plate systems.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

Demand Critical Welds OMF

We use OMFs with bolted end-plate moment connections (neither the flanges nor the web of the beam are directly welded to the column). Section 11.2c of the 2005 *Seismic Provisions* lists three specific cases when the CJP groove weld is demand critical. However, none refers to end-plate connections. This is also consistent with the user note in 7.3b. I would appreciate further explanation, as it appears that the 2005 *Seismic Provisions* impose no additional welding requirement for the type of end-plate moment connections we use in OMFs.

Question sent to AISC's Steel Solutions Center

Section 11.2c of the 2005 *Seismic Provisions* states that all CJP groove welds to the column should be considered demand critical. Fillet welds should not. As you suggested, this should probably be consistent for end plates. If the end plates use CJP groove welds, they should be treated as demand critical.

James O. Malley, S.E.
Senior Principal
Degenkolb Engineers

HSS in 50 ksi Material

Are hollow structural sections available in F_y = 50 ksi steel strength? If yes, how do you specify the HSS?

Question sent to AISC's Steel Solutions Center

The most common grade for HSS is ASTM A500 Grade B, which has a minimum yield strength of 42 ksi in rounds and 46 ksi in rectangular and square cross-sections.

There is also an ASTM A500 Grade C designation for HSS, which may be available upon request from some producers. This grade has minimum yield requirement of 46 ksi for rounds and 50 ksi for rectangular and square cross-sections. We suggest that you contact your local fabricator or one of the HSS producers listed at **www.aisc.org/availability** to ensure that the product is available before specifying it.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

Bolt Length

What is the rule of thumb for how far bolts should extend above the nut—two threads, ½" to 1"? I saw in an old structural steel detailing book from the 1970s that this should be around ½" to 1". In looking at newer publications, I have not found any recommendation on how far these should protrude.

Question sent to AISC's Steel Solutions Center

There is no need for the bolt to extend past the nut at all. These older practices have been discontinued in current specifications. Accordingly, Section 2.3.2 of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* states, "The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed." The RCSC specification is available to download free at **www.boltcouncil.org**.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

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:

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