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!

Filling Weld Access Holes

Please provide some opinions and guidance as to filling weld access holes in beam and column splices with weld metal for appearance sake. The AWS D1.1 welding code does not appear to address the issue. In your experience, do you usually leave them open or fill them?

Question sent to AISC's Steel Solutions Center

Weld access holes should not be filled with weld metal—doing so could create the very same cracking problems the weld access hole was used to prevent. If the hole is to be filled for appearance reasons in architecturally exposed structural steel (AESS), it is possible to use a body filler such as "Bondo." Refer to Section 10.4.1 of the AISC *Code of Standard Practice* (a free download from **www.aisc.org/code**.)

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

Grouting Base Plates

We were told by a steel erector that they typically do not remove shims from column base plates after grouting. They also informed us that they do not back off leveling nuts below the base plates during the grouting process. Are these practices acceptable?

Question sent to AISC's Steel Solutions Center

Yes, shims are left in place underneath the base plates. From a construction standpoint, those shims or leveling nuts hold the load while the grout cures. Their presence after the grout is structural and does not reduce strength. And from an economic standpoint, removal would needlessly increase cost.

Axial compressive forces from the column will be almost evenly distributed as bearing forces on the shims and non-shrink grout. Even if the shims were to take the majority of the load, the assembly will deform in a self-limiting manner through localized yielding or crushing of concrete as the force-distribution model assumed in sizing the base plate is attained.

Bill Liddy
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Backing Bar Removal

We're designing an OMF using the IBC 2000 in Seismic Design Category "A". FEMA 350 states that lower flange backing bars should be removed to allow identification and correction of weld root flaws. On this project, the welding of the OMF joints is being observed full-time by inspectors. The question is now being asked, why are the backing bars specified to be removed? If the root pass is being observed by inspectors as it is installed, and is certified by those inspectors as being a good weld, why do we need to remove the backing bars to look at the weld again?

As the Engineer of Record, I find these arguments for leaving the backing bars in place to be compelling, but I'm not a weld expert. Can you offer me any guidance on this issue? Could the backing bars be left in place without causing problems?

Question sent to AISC's Steel Solutions Center

First a side note: While high-seismic design criteria (e.g., the AISC *Seismic Provisions*, FEMA 350, etc.) can be applied in Seismic Design Categories A, B and C, if you choose to use R = 3 in SDCs A, B, and C then the design only needs to meet the requirements in the AISC *Specification for Steel Buildings*. However, if you use an R greater than 3, then you would be required to meet the AISC *Seismic Provisions*' requirements (which include FEMA 350 connections) regardless of Seismic Design Category. But back to your question….

Backing-bar removal is required at the bottom flange (flange with the backing bar at the extreme fiber) because of the sensitivity of this detail to the combination of stress, flaw size and notch toughness. Since root flaws and the unfused face of the backing bar act as built-in notches—and the flange welds in an OMF are expected to withstand inelastic deformations of the connected beam—it is more reasonable to expect a greater potential for fracture. Further, even the best visual inspection cannot see under the molten weld pool to determine the flaw condition.

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Separators in Double Angle Struts

In the 3rd edition *LRFD Manual*, page 16.1-205 (or Commentary Section E4 in the 1999 *LRFD Specification*) states that for built-up compression members "the connectors must be designed to resist the shear forces which develop in the buckled shape." Can you provide some guidance on how to compute that required shear force?

Question sent to AISC's Steel Solutions Center

There is an AISC *Engineering Journal* paper that addresses the shear force calculation that you mentioned. It is entitled "Analytical Criteria for Stitch Strength of Built-Up Compression Members" (*Engineering Journal,* 3rd quarter, 1992) and can be downloaded from **www.aisc.org/ej**.

In the buckled configuration of a built-up compression member, shear force is developed between individual components due to secondary moments caused by the P-δ effect. Section E4 in the 1999 *LRFD Specification* requires that stitches be designed such that they have adequate strength to resist the shear force developed between individual components. This paper presents a derivation of analytical equations to calculate the shear force developed between individual components of built-up struts in the buckled configuration.

Equations are presented for two cases: (1) when only the first buckling load is of interest; and (2) when post-buckling bending is involved. The equations given are general enough so that they are applicable to any end condition, including the two extreme cases of pinned and fixed-end conditions. The proposed equations are verified analytically and experimentally. For analytical verification, the results from the proposed equations are examined for the extreme cases of end conditions and separation between the components. For experimental verification, test results by the authors are used. The stitch strength required for some test specimens are calculated according to the proposed equations. The results are compared with actual strength provided by the stitch welds of the corresponding specimens. It was found that specimens subjected to unsymmetrical buckling and/or post-buckling behavior did not have adequate stitch strength according to the proposed equations.

Bill Liddy
American Institute of Steel Construction

Single Angle Under Axial and Bending

We have developed a program to check single-angle members subjected to combined axial load and bi-axial bending per the LRFD 3rd Edition spec for Single-Angle Members. We would like to check the program against some example hand calculations.

Does AISC have any example calculations that we could use, or could you direct us to another source that may have such calculations?

Question sent to AISC's Steel Solutions Center

There was an *Engineering Journal* paper written by Dr. Lutz back in 1996 that gives a complete example for the case of combined axial and bending forces. It is titled "A Closer Examination of the Axial Capacity of Eccentrically Loaded Single-Angles" and can be downloaded from **www.aisc.org/ej**.

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Unbraced Length of a Cantilever

I was wondering what the laterally unbraced length value L_b is for a cantilever? My intuition tells me that I should use twice the actual length of the cantilever for L_b, but I don't see any provisions for it in Chapter F or Appendix F of the *Specification*. Does limiting the C_b value to 1.0 for cantilevers provide all that is needed, and then I would just use the actual length of the cantilever for L_b?

Question sent to AISC's Steel Solutions Center

In Section F1.2a of the 1999 *LRFD Specification* (a free download from **www.aisc.org/lrfdspec**), the coefficient C_b is taken as 1.0 for cantilevers where the free end is unbraced. When evaluating C_b for a cantilevered beam, the moment diagram will lead to a value of approximately 2.0 depending on loading conditions. You might be inclined to increase the moment capacity of the member by an equal amount, but this is unconservative and incorrect. Similar to a flagpole problem where $K = 2.0$, the effective unbraced length is twice the actual length. These two factors cancel each other since C_b would increase the moment capacity and K would decrease it. The proper calculation of the design flexural strength of a cantilever uses the actual length and a C_b coefficient of unity. For cases of restraint to the compression and/or tension flanges at the free end of the cantilever, refer to the SSRC publication *Guide to Stability Design Criteria for Metal Structures* (**www.stabilitycouncil.org**).

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Steel Interchange is a 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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