If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

Note: Unless specifically noted, all AISC codes and standards mentioned in the questions and/or answers are independent of the edition (2010 vs. 2016)—and can be found at www.aisc.org/publications.

Flexural Yield Strength of Single Angles

Section F10.1 of the 2010 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-10) addresses flexural yielding of single angles. The description preceding this section describes this limit state as "the limit state of yielding (plastic moment)." This is confusing me because previous sections differentiate between the plastic moment, F_yZ_x, and the yield moment, F_yS_x, but here they seem to be combined. What is the yield moment for a single angle?

I understand your confusion, and hopefully I can provide an explanation as to what is intended.

M_y, which is generically represented by F_yS, provides the point of "first" yield. It is defined in the symbols list as "moment at yielding of the extreme fiber." That is to say, yielding is only just starting to occur at the extreme fibers of the member, but the majority of the cross section (nominally 100%) is still elastic. The plastic moment, M_p, which is generally represented as F_yZ, provides us the point of "last" yield. That is to say, yielding has now occurred across the full cross section of the member, reaching the point of full plasticity, after which the strength remains constant as deformations continue.

In general, throughout Section F, when we look at the member bending capacity associated with the limit state of yielding, we are looking to capture the member capacity where full yield is occurring and which corresponds to M_p. However, with some of the irregular geometries, such as single angles in F10, AISC restricts the nominal flexural strength (the moment that can be considered) to something less than F_yZ because the shape introduces some level of uncertainty that full plastic yield is achievable. In Section F10.1, the "limit state of yielding (plastic moment)" phrase is intended to convey that what we are interested in, as a limit state, is where we are yielding most of the member cross section (plastic behavior) and not first yield (elastic behavior). In Equation F10-1, the *Specification* limits the usable plastic capacity to $1.5M_y$, which for angles is less than F_yZ but still far beyond the point of first yield. The commentary to F10.1 provides some additional discussion as to the origin of the 1.5 factor, if you are curious.

Susan Burmeister, PE

Minimum Bend Radius

I am working on a project that involves a staircase with a roughly helical geometry. We plan on using HSS for the stringers. I am concerned about the effect of the bending process on the strength of the section. Is there a minimum radius to which HSS can be bent?

Rigid guidelines for a minimum bending radius are not available because it is dependent on several variables, including:

- Axis of curvature
- Cross-sectional shape of the member
- Wall thickness of the member
- Bending method used by the bender-roller
- The equipment limitations of the bender-roller
- Level of acceptable cross-sectional distortion
- Level of acceptable cold-working of the material

These limitations should be discussed with the bender-roller providing the service. There is a list of bender-roller companies at **www.aisc.org/membership/bender-roller-committee**. You can also find information related to curved members at **www.aisc.org/technical-resources** (click "Curved Steel").

If the stringer geometry is formed by cold bending, the primary structural concern is the cross-sectional distortion that occurs during the bending operation and the effects of cold work on the material properties.

Tolerances on the cross-sectional distortion should be developed with significant input from the bender-roller.

Cold bending of members involves plastic straining of the member to induce a permanent curvature. Any significant cold working of steel will have an effect on the material properties. These changes are primarily attributed to the effects of strain hardening and strain aging, which cause an increase in yield stress, ultimate stress and hardness and a decrease in ductility and toughness. In the November 1984 issue of *Steel Construction* (a publication of the Australian Institute of Steel Construction) article "Curving Structural Steel," G. Riviezzi recommended that cold working strains up to 5% can be neglected for statically loaded members with no welded attachments, holes or stress concentrations. For statically loaded members with welded attachments, holes or stress concentrations, he recommended a maximum strain of 4%. For members that are symmetric about the axis of curvature, the maximum strain induced by the cold bending operation is $\varepsilon_{max} = d/(2R)$, where R is the radius of curvature of the neutral axis and d is the member depth.

Bo Dowswell, PE, PhD

"Old" Steel

I am reviewing material test reports from a fabricator. Some of the steel was rolled as far back as 2012. Is it common to see rolling dates this old?

It is not unusual for fabricators to maintain lists of materials in their stock that may have been purchased for projects that never went forward or that were never used due to changes or other reasons. They will use this stock material when the opportunity arises. Steel is as good every day in its future as it was the day it was rolled.

Section 5.2 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303) addresses the use of stock material. Section 5.2.2 addresses the material test reports you are seeing and states: "Material test reports shall be accepted as sufficient record of the quality of materials taken from stock by the fabricator. The fabricator shall review and retain the material test reports that cover such stock materials. However, the fabricator need not maintain records that identify individual pieces of stock material against individual material test reports, provided the fabricator purchases stock materials that meet the requirements for material grade and quality in the applicable ASTM specifications."

Carlo Lini, PE

Welds in Holes

I understand that a plug weld cannot be used to resist tension. I have several questions related to welds in holes:

1. **Is it acceptable to place a fillet weld around the circumference of a hole to resist combined tension and shear?**
2. **The diameter of the hole is ½ in. and the thickness of the material is 3⁄16 in. Is it possible to make a fillet weld in such a small space?**
3. **When the welding is complete, would the fillet weld around the edge of the hole be significantly different than a plug weld?**

Here are answers to your questions:

1. Yes. The Commentary to Section J2.3 of the *Specification* states: "A fillet weld inside a hole or slot is not a plug weld." Section J2.2b(h) states: "Fillet welds in holes or slots are permitted to be used to transmit shear and resist loads perpendicular to the faying surface in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds are permitted to overlap, subject to the provisions of Section J2. Fillet welds in holes or slots are not to be considered plug or slot welds."
2. In my experience, it is very difficult (nearly impossible) for fabricators to make welds as small as 3⁄16 in. with equipment used to fabricate typical structural steel. Most 3⁄16-in. fillets welds usually end up being closer to a ¼-in. fillet weld when completed.

   Another issue is the effectiveness of the weld, if indeed it can be made. The Commentary states: "Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Figure C-J2.3(b), unless restrained by a force, *F*, as shown in Figure C-J2.3(a)... The use of single-sided fillet welds in joints subject to rotation around the toe of the weld is discouraged." Though fillet welds like what you describe may be effective in resisting the relatively small forces required to "prevent the buckling or separation of lapped parts," they may not be fully effective in resisting significant tension. This may be especially true for the condition you have described. The 3⁄16-in. plate is thin and flexible, and tension applied to the plate away from the weld will result in bending and a condition similar to C-J2.3(b).
3. You could potentially fill the entire hole with weld metal as you would with a plug weld. The physical weld then would be identical whether it was treated as a fillet weld or a plug weld. The difference is in the design approach for the weld. The effective area would be different. The effective area of the plug weld would be the nominal cross-sectional area of the hole in the plane of the faying surface. The effective area of the fillet weld would be the effective throat times the circumference of a circle.

Larry S. Muir, PE

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