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.

KL/r Modified for Single-Angle

A question and answer on this subject appeared in the January 2008 Steel Interchange (reprinted below). LeRoy Lutz, a member of the AISC Specification Task Committee covering the design of members, was kind enough to provide the following supplementary discussion pertaining to the design of single-angles: Single-Leg Angles per E5.

This is a response to the comment made with regard to single-angle slenderness in section E5 of the AISC *Specification* in the January 2008 issue of MSC.

a) First, I was unsure what the designer meant when he indicated calculating KL/r_z (I presume that he meant L/r_z). One considers the KL/r to be an equivalent L/r_z when designing.
b) The 0.95 and $0.82L/r_z$ limits given in E5 (a) and (b) are in the paragraph that addresses unequal-leg angles with the long leg projecting and does not apply to equal-leg angles.
c) The limits of 0.95 and $0.82L/r_z$ for equal-leg angles would occur when the KL/r is at about the upper limit of 200, so that there would be no need to check those limits for equal-leg angles. For unequal-leg angles with the short leg projecting, those limits would occur well beyond an L/r of 200.

Here is a short summary for stocky and slender equal-leg angles:

1) For an L3×3×¼ equal-leg angle with L/r_z of 30 (and L/r_x of 19), the KL/r (i.e. L/r_z) calculated from (a) would be 86.2 and from (b) would be 75.2. The axial load design at these values of slenderness would account for strength reduction based on the load's eccentricity reduced some by the end restraint (as compared to a pinned-end member).
2) For an L3×3×¼ equal-leg angle with L/r_z of 160 (and L/r_x is 101), KL/r (i.e. L/r_z) calculated from (a) would be 158 and from (b) would be 146. The axial load design at these values of slenderness account for a slight strength increase due to the end restraint and reduction based on the load's eccentricity (as compared to a pinned-end member).

LeRoy Lutz

Original question/answer from the January 2008 Steel Interchange:

For a single-angle compression member, I followed AISC specification section E5 to calculate the modified *KL/r*. I also calculated KL/r_z, and it turns out to be greater than *KL/r* modified. Should I use the larger of the two (*KL/r* modified, or KL/r_z) in section E3?

If you are in compliance with E5 (including attaching the angle using the longer leg) then you can use the limits on L/r_z that are provided at the ends of the both sections (a) and (b). In the first case the limit is $0.95L/r_z$ and in the second case it is $0.82L/r_z$. In essence, with your condition, you are still designing for KL/r_z but with a K value of less than 1.0 because of the higher end restraints.

Amanuel Gebremeskel, P.E.

Flexural Capacity of Channels

Why are the maximum strong and weak axis bending stress values for channels limited to $0.6F_y$ and $0.66F_y$ respectively? The weak axis limit seems particularly conservative given that compact, doubly symmetric sections and plate have a $0.75F_y$ limit.

I am not sure which document you are looking at, but in the 2005 AISC *Specification* (available at www.aisc.org/2005spec) channels in strong axis bending have an ASD limit of $0.66F_y$ and $0.9F_y$ for weak axis bending if you approximate a lower bound shape factor and use S to make the comparison to older versions of ASD. The derivation for these is as follows:

For the weak axis, $M_n / \Omega = 0.9F_yZ_y/1.5 = 0.9F_y(1.5S_y)/1.5 = 0.9F_yS_y$. This derivation assumes that $Z_y/S_y = 1.5$, which is reasonable for weak axis bending. For strong axis bending a similar derivation using $Z_x/S_x = 1.1$ results in $M_n / \Omega = 0.66\ F_yS_x$. These approximations assume a wide-flange cross-section.

Amanuel Gebremeskel, P.E.

Fillet Weld Strength

I am familiar with the method of determining the fillet weld strength using the ASD load approach, but I am having difficulty determining this strength when using the LRFD load approach.

For a simple ASD fillet weld (load at 90° to the fillet) the magic number is 0.9 kips/in. of weld, which is based upon 0.3*70 ksi electrode per 16th of weld.

I noticed in the 13th Edition Manual that the weld strength is increased when the load is at 90° to the fillet. I always thought a weld had the same strength whether a load was in the same direction, along the weld or perpendicular to the weld. The 13th Edition Manual seems to indicate that it is 50% stronger when loaded at 90°.

The increase for direction of load is covered in Section J2.4(a) of the *Specification*. Where the load is oriented at 90°, this amounts to a 50% increase and applies to both ASD and LRFD load approaches.

The weld value in the old versions of ASD was 0.928 kips per 1/16 in. of leg. This comes out to the same Allowable Strength in the ASD approach of the 2005 *Specification*. The Available Strength of Fillet Welded Joints is $0.60F_{EXX}$, regardless of whether the ASD or LRFD load approach is used. (See Table J2.5 of the 2005 *Specification*.)

If the ASD load approach is used, $\Omega = 2.00$, resulting in an Allowable Strength of $0.60F_{EXX}/2.00 = 0.30F_{EXX}$, then this resolves as follows: (0.30)(70)(0.707)/(16) = 0.928 kips per 1/16 in.

If the LRFD load approach is used, $\varphi = 0.75$, resulting in a Design Strength of $(0.60F_{EXX})(0.75) = 0.45F_{EXX}$, then this resolves as follows: (0.45)(70)(0.707)/(16) = 1.392 kips per 1/16 in.

Note that the LRFD Available Strength is always 1.5 times the ASD Available Strength.

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

Anchor Rod Push-Out

Section 2.9.1 of Design Guide 1 contains the following statement:

"When designing anchor rods using setting nuts and washers, it is important to remember these rods are also loaded in compression and their strength should be checked for push-out at the bottom of the footing."

How does one go about calculating the push-out of the anchor through the bottom of the footing?

Another AISC source of information on the subject, Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Buildings*, provides a discussion of anchor-rod push-out. See Section 4.2.7 in that publication.

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

Fillet Weld for Single-Plate Shear Connection

My question has to do with single-plate connections to supports. Chapter 10 of the 13th Edition (page 10-101) states "the weld between the single plate and the support should be sized as ⅝ t_p, which will develop the strength of the plate." Is this a minimum or maximum limit? Should we design the required weld size needed, and then compare to this value? This question stems from the 9th Edition Commentary on single-plate connections (page 4-53) where it stated the weld size need not exceed 0.75t. Are these (2) requirements discussing the same subject? It seems that the 9th Edition is trying to make sure we have more web thickness than weld, but the 13th Edition Commentary is stating to use an exact amount of weld. Can you shed light on this?

You are correct that the requirement on page 10-101 in the 13th Edition Manual is intended to develop the plate. This means it is a minimum weld recommendation if the designer wishes to develop the strength of the plate, and is based on weld shear rupture. You do not need to calculate a weld size for load and compare. Rather, if the plate is adequate in shear, the weld size is then selected as ⅝ t_p and will have adequate strength. The idea behind the older ¾ *t* requirement in the 9th Edition Manual was also the same, but it is an older approach based on weld yield that has been dropped in favor of the new approach.

Amanuel Gebremeskel, P.E.

L_p for Non-Compact Shapes

I have a question regarding a value in Table 3-2 of the 13th Edition Manual pertaining to the W21×48 (page 3-17). The listed value for L_p is 6.09 ft, whereas when I calculate the value myself (for a 50 ksi beam) I get 5.86 ft. Can you please review this value and let me know if this is an error?

The number listed in the table for the W21×48 is not an error. A W21×48 is non-compact, and the L_p for non-compact shapes, L'_p, is not calculated per Equation (F2-5). Remember that non-compact shapes are not capable of achieving the full plastic moment. Therefore, a point on the moment versus unbraced length curve is used to define the value M'_p for the shape. See page 3-4 in the 13th Edition Manual for discussion, and the Equation upon which the the value of L'_p is based.

Amanuel Gebremeskel, P.E.

ASD or LRFD?

What is the AISC position on use of LRFD or ASD design? It appears that the equations in the 2005 AISC *Specification* can be utilized in the LRFD or ASD method by multiplying by the φ factor or dividing by the Ω factor. Is this correct, because people are telling me one cannot use the ASD method anymore? What about the IBC—do you know if they specifically require the use of LRFD?

The governing building code typically specifies load combinations. IBC (and ASCE 7) provide load combinations that can be used with either ASD or LRFD design approaches. IBC 2006 also references AISC 360-05 (the 2005 AISC *Specification*), which provides both ASD and LRFD methods of design.

Therefore, the answer to your question is that you can use either method when using the 2005 AISC *Specification*. Since AISC 360 uses the same equations for both methods, the only differences between LRFD and ASD will be due to variations in the load combinations from ASCE 7 or IBC. Just be sure to use the corresponding set of load combinations.

Amanuel Gebremeskel, P.E.

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Charlie Carter is an AISC vice president and the chief structural engineer.

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: 312.803.4709
solutions@aisc.org