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steel interchange

Application of Q_f

When the variable Q_f is used in AISC 360-10 Section K1, it is typically applied as a multiplier outside of the bracketed portion of the equation. As such, it has an effect on the entire nominal strength calculation. However, Equations K1-12 and K1-13 for the limit state of wall plastification have Q_f inside the bracketed portion of the equation. Is it correct for these two limit states that Q_f only applies to a portion of the nominal strength equation?

I actually had the same question when I was reviewing this section of the AISC *Specification*. Q_f is in the correct location (inside the brackets) in 2010 AISC *Specification* Equations K1-12 and K1-13. The equation is based on a yield line approach. Because the force in the member has a greater effect on the strength of the yield lines transverse to the chord axis and little effect on the yield lines parallel to it, Q_f is applied to only those portions where it has a significant effect. This approach predicts a strength consistent with test results.

Larry S. Muir, P.E.

Weld Access Holes

It seems that the industry standard for end-plate moment connections is to not have weld access holes at the beam flange to end-plate CJP groove welds. This is different from the directly welded flange moment connection, which requires a weld access hole for the beam flange CJP groove welds. Why are weld access holes not used in end-plate moment connections?

The behavior of bolted end-plate connections has been shown to differ from directly welded flange connections in physical tests and in finite element models. In end-plate connections, the flange force is partially resisted by the bolts inside the flange. The weld access hole interrupts the stress flow to these bolts and causes a stress-riser that promotes flange fracture early in the inelastic range. Accordingly, weld access holes are not recommended in these types of connections.

This is briefly mentioned in the AISC *Manual* in Part 12 on end-plate moment connections. It states, "As reported by Meng and Murray (1997), use of weld access holes can result in beam flange cracking. If CJP welds are used, the weld cannot be inspected over the web; however, because this location is a relative 'soft' spot in the connection, it is of no concern." To quickly find this reference online, or any of the others in the 14th Edition *Manual*, go to **www.aisc.org/manual14** and follow the link to the Interactive Reference List.

Heath Mitchell, S.E., P.E.

Single Angles in Compression

Do the effective slenderness ratio provisions of AISC 360-10 Section E5 apply to the design of a concentrically loaded, compact single angle?

No, the slenderness modifications only apply to eccentrically loaded single angles that meet the specific criteria outlined in Section E5. The charging language of AISC 360 Section E5, states, "The *nominal compressive strength*, P_n, of single angle members shall be determined in accordance with Section E3 or Section E7, as appropriate, for axially loaded members. For single angles with $b/t > 20$, Section E4 shall be used. Members meeting the criteria imposed in Section E5(a) or E5(b) are permitted to be designed as axially loaded members using the specified effective slenderness ratio, KL/r." In other words, the slenderness modifications in Section E5 allow the eccentricity to be neglected in some eccentrically loaded single angles by designing them as axially loaded members with an effective slenderness ratio.

Heath Mitchell, S.E., P.E.

Extended Shear Tab Design

Consider a beam-to-girder extended single-plate connection that has a depth equal to the full-depth between, and is welded to, the flanges of the supporting girder. Is the eccentricity used in designing the bolt group taken as the distance from the bolt line to the beam web or can it be taken from the bolt line to where the single plate has full support (supporting beam flange tips)?

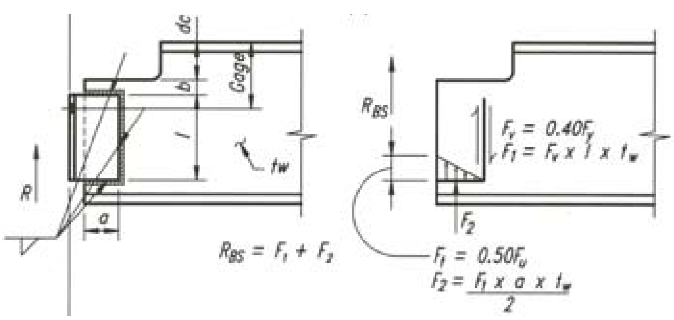
The procedure provided in the 14th Edition AISC *Steel Construction Manual* assumes an eccentricity on the bolt group from the face of the support to the center of the bolt group. The *Manual* also specifically allows the use of other rational methods. One of these might be to consider the "balancing" effect of having connections to both sides of the girder web. However, if this is done only the persistent dead loads should be considered when evaluating the countering effects of the additional connection.

If there is a connection to only one side of the girder, this is a difficult matter. Because wide-flange sections are generally weak in torsion, it would not be advisable to assume that the beam itself resists any of the moment caused by the eccentricity. This is essentially what you would be doing if you were to take the eccentricity from the toe of the support beam flange to the bolt group.

Larry S. Muir, P.E.

Block Shear of Welded Single-Angle Connection

What shear area should be used when checking the block shear strength of a top coped beam with a welded clip angle connection? The most recent design guidance I have found is in the AISC *Manual – Volume II Connections, ASD 9th Edition and LRFD 1st Edition*, which includes Example 9 and Figure 3-23 (see below). This shows that the shear area is the length of the connection angle multiplied by the beam web thickness and the portion of exposed beam web above the connection is ignored. Is it an AISC *Specification* requirement that the portion of exposed beam web beyond the connection angle be ignored in the block shear strength calculation?



I do not think it was ever AISC's intent to limit the length of the shear area to the length of the welded clip. This certainly does not need to be done to satisfy the AISC *Specification*.

It is sometimes convenient to make this kind of conservative assumption when the vertical location of the clip angle relative to the cope is not known or could vary, such as when the cope depth varies but the punch down remains constant. In practice, assuming the length of the shear area equal to the length of the clip would allow a general calculation to be conservatively applied over a range of connections, only performing a more exact calculation where the strength predicted from the conservative length does not satisfy the contract loads.

The full available dimension to the cope can be used in the calculation.

Larry S. Muir, P.E.

Correction of Errors

What is the intent of "moderate amounts of reaming" as used in 2010 AISC *Code of Standard Practice* Section 7.14? Does it mean that the erector should expect to ream at every connection?

The Commentary to AISC *Code* Section 7.14 states, "As used in this Section, the term 'moderate' refers to the amount of reaming, grinding, welding or cutting that must be done on the project as a whole, not the amount that is required at an individual location. It is not intended to address limitations on the amount of material that is removed by reaming at an individual bolt hole, for example, which is limited by the bolt-hole size and tolerance requirements in the AISC and RCSC *Specifications*."

In other words, it is not intended to mean that the erector should expect a moderate amount of reaming on each hole or connection. Rather, the erector should expect that some reaming may be necessary on the project. The need to correct minor misfits should be the exception, not the norm. In addition, the RCSC *Specification* tolerances on bolt holes still apply to the final, reamed hole.

Heath Mitchell, S.E., P.E.

T-1 Steel

I am doing research on bridges built prior to 1960. Does "T-1" steel fall under the ASTM A7 standard?

No. T-1 was the trademarked name used by U.S. Steel for high-strength, quenched and tempered 100 ksi steel, which is not the same material as ASTM A7. The product that U.S. Steel called T-1 is currently covered by a variety of similar standards: AASHTO M270 Grade 100, ASTM A514 or A517, and ASTM A709 HPS 100.

AISC *Steel Design Guide No. 15, AISC Rehabilitation and Retrofit Guide*, is a reference for historic shapes and specifications. It is available as a free download for AISC members, and for purchase by others, at **www.aisc.org/dg**. Table 1.1a lists the historic specifications. You will see the history of A7 and other steels as well as their yield and tensile properties.

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