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steel interchange

Tension-Only OCBF

I am designing a tension-only OCBF using cables for braces. The 2005 and 2010 AISC *Seismic Provisions* require braces to comply with specific width-to-thickness limits in Sections 14.2 and Section F1.5a, respectively. How is *b/t* for a cable calculated? I also noticed that the user note discussing tension-only OCBFs in 2005 AISC *Seismic Provisions* is absent from the 2010 version of this document. Does this mean that tension-only OCBFs are no longer allowed?

Tension-only systems are a special application of OCBFs. Even though they are not specifically addressed in the main body of AISC 341, they are allowed to be used under both the 2005 and 2010 AISC *Seismic Provisions*.

The width-thickness limits are for "compression" elements. By design, there are not any compression elements in a tension-only system, so they do not apply. Technically, the braces may see some compression, but they will be so slender that they buckle elastically. So even if their very small compression capacity were accounted for, the width-thickness limits would make little difference in the performance of the system.

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

Single-Sided Fillet Welds

Can the web and flange of a built-up girder be joined using a single-sided fillet weld or is a double-sided fillet weld required?

It is relatively common for built-up girders to have the web welded to the flange with a single-sided fillet weld, especially in the metal building industry. This is generally acceptable because the weld transfers only shear.

There are some instances for which the single-sided weld is not appropriate, including the transfer of certain concentrated loads and some cases related to seismic lateral force resisting systems. Some examples are provided in Section F3 of the AISC *Seismic Provisions* and Section 2.3 of AISC 358 (both documents are free downloads from **www.aisc.org/epubs**).

Larry S. Muir, P.E.

A325 Bolt Strength

I noticed a conflict between AISC *Specification* Table J3.2 and AISC *Manual* Table 2-6. *Specification* Table J3.2 gives the nominal tensile strength of ASTM A325 bolts as a constant value for all bolt diameters. However, *Manual* Table 2-6 shows that there is a reduction in the tensile strength of A325 bolts over 1 in. in diameter. Why does the AISC *Specification* not reflect this strength reduction?

The AISC *Specification* (a free download from **www.aisc.org/2010spec**) intentionally neglects the strength differences between the larger and smaller diameter A325 bolts. The Commentary to Section J3.6 states:

"For ASTM A325 or A325M bolts, no distinction is made between small and large diameters, even though the minimum tensile strength, F_u, is lower for bolts with diameters in excess of 1 in. (25 mm). Such a refinement is not justified, particularly in view of the conservative resistance factor, ϕ, and safety factor, Ω, the increasing ratio of tensile area to gross area and other compensating factors."

The origin of this difference goes back to the early days when hardening practice was less advanced and A325 bolts in larger diameters were case hardened. As other standards came in—like A490, A325M, A490M, F1852 and F2280—hardening practices had improved so that the larger diameters were through hardened just like the smaller diameters. Thanks to tendencies for things to live on in standards once they are written, ASTM A325 still shows the early history, even though hardening practices haven't needed the reduction for several decades.

Testing of thousands of A325 bolts has also not shown any significant difference in strength between the larger and smaller diameter bolts, despite the allowance for a lower strength in the larger diameter bolts by ASTM. Committees of AISC, the Research Council on Structural Connections (RCSC) and the American Society for Testing and Materials (ASTM) have been discussing how to accomplish revising ASTM A325 to be consistent with practice.

Larry S. Muir, P.E.

Significant Load Reversal

RCSC *Specification* Section 4.2 Item (2) states that a pretensioned joint is required when the joint is subject to significant load reversal. Is wind loading considered a significant load reversal?

Neither AISC nor RCSC define the degree of load reversal in terms of either magnitude or frequency, so some engineering judgment must be exercised. I would argue that wind loading on the main wind force resisting system in typical buildings does not produce significant load reversal. Significant load reversal implies the full design load (or close to it) in both directions. With the mean recurrence interval for wind loads in ASCE 7 being 50 years, a brace may see its full design wind loading in one direction, but it is unlikely that it will also undergo the full design loading in the other direction next in the loading sequence.

Charles J. Carter, S.E., P.E., Ph.D.

Shear Stud Strength

In AISC *Manual* Table 3-21, for the case of no-deck, normal-weight concrete and f'_c=4 ksi, the values in the 14th Edition *Manual* have changed in comparison to the 13th Edition *Manual*. Why have these values changed?

The change is a result of a reduction in the R_p factor in 2010 AISC *Specification* Section I8.2a. The Commentary provides the following discussion relative to this change:

> "The reduction factor, R_p, for headed stud anchors used in composite beams with no decking has been reduced from 1.0 to 0.75 in the 2010 *Specification*. The methodology used for headed stud anchors that incorporates R_g and R_p was implemented in the 2005 *Specification*. The research (Roddenberry et al., 2002a) in which the factors (R_g and R_p) were developed focused almost exclusively on cases involving the use of headed stud anchors welded through steel deck. The research pointed to the likelihood that the solid slab case should use R_p = 0.75; however, the body of test data had not been established to support the change. More recent research has shown that the 0.75 factor is appropriate (Pallares and Hajjar, 2010a)."

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

SCBF Brace Slenderness

The SCBF brace design example in the 1st Edition AISC *Seismic Design Manual* used a maximum slenderness for the brace of $4\sqrt{(E/F_y)}$ = 115. In the 2nd Edition this ratio has been increased to 200 with a specific calculation. Is the 2nd Edition slenderness limit correct?

The 2005 *Seismic Provisions* Section 13.2a had a maximum slenderness limit of $4\sqrt{(E/F_y)}$. However, there was an exception that allowed this limit to increase to 200 if the columns were designed for the loads resulting from the braces reaching their expected strength. The 1st Edition *Seismic Manual* used the $4\sqrt{(E/F_y)}$ limit.

The 2010 *Seismic Provisions* Section F2.3 requires that load effects on the lateral system due to braces reaching their expected strength must be directly analyzed as part of the determination of required strengths. Due to this change, the 2005 exception will be satisfied on all designs and the lower slenderness limit equal to $4\sqrt{(E/F_y)}$ was removed. The maximum of 200 is now the only limit specified for SCBF.

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

Piece Mark Appearance

Is there an AISC requirement for field priming of structural steel piece marks, or are piece marks left unpainted?

The appearance of piece marks in a finished structure varies widely. It is a contractual item and is dependent on the exposure of the steel once the structure is completed. There is not an AISC standard procedure that governs removal, painting or other treatment of piece marks, except that Section 7.17 in the AISC *Code of Standard Practice* relieves the fabricator and erector of any field painting responsibilities.

In many cases, the steel is fire protected and removal or treatment of shop-applied piece marks is typically not required. In the case of exposed or painted structures, aesthetics usually govern the handling of piece marks. Assuming painted steel, the options are to simply paint over piece marks as a part of the paint touch-up process or, in some rare cases, it may be determined that piece marks must be removed because they pose a bleed-through risk or are not compatible with the paint chosen as a finish coat.

Where a consideration like architectural exposure means special treatment is required, the required treatment should be specified in the contract documents.

Keith Landwehr

Single-Angle Connection

Can eccentricity be ignored when checking the limit state of bolt bearing on the support (girder web) for a single-angle beam-to-girder connection?

No. As shown in Figure 10-14 of the 14th Edition AISC *Manual*, eccentricity is always considered for the bolts on the support side. Consideration of the eccentricity on the bolts at least implies that some consideration of eccentricity should be made relative to bearing. In this instance it is very unlikely that the portion of the check governed by edge distance (the tear-out portion) of the bearing check will govern, so the bearing strength can be calculate as the bearing strength of a single bolt (it should be the same for all the bolts) multiplied by the C-value.

Generally, bearing is not critical, though it can be when attaching to a thin-webbed girder with connections to both sides using common bolts. For some reason this is a check that is commonly overlooked. I am not sure why this occurs. Table 10-11 defines "ϕ_{rn} = design strength of one bolt in shear or bearing, kips/bolt" and "r_n/Ω = allowable strength of one bolt in shear or bearing, kips/bolt."

Larry S. Muir, P.E.

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