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SMF Connection at Top of Column

AISC 358-10 shows schematic elevations of the various types of connections prequalified for use in special and intermediate moment frames. In all the details shown in this standard, the joints are shown with the column continuous, and none show a connection at a column top. I have seen some engineers extend the column above the top of the beam and use the prequalified detail; others have placed a cap plate on top of the column and connected the beam to the cap plate as illustrated in AISC 358 Commentary Figure C2.2. Are both options acceptable? If the cap plate detail is allowed, what is the recommended method of attachment to the column to comply with AISC 358-10 E3.6f(3) welding requirements? Can the cap plate weld extend into the k-area?

To use the connections in ANSI/AISC 358, continuity plate and other connection detailing requirements must be satisfied. In order to accomplish this at the column top, the column must be extended a sufficient amount to enable continuity plate welding.

Even though cap-plate detailing is not incorporated into the main section of the prequalified connection standard at this time, the Commentary figure and discussion you cite do recognize the use of cap plates. However, only very general guidance is given and the design and detailing do deviate from what is provided in the standard. Engineering judgment is required in order to properly design and detail a cap plate connection for a SMF.

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

Shear Stud Welding

I have a project where the existing concrete deck is being removed and shear studs are being added to an existing beam, after which the concrete deck is being reinstalled. The contractor is proposing to weld the shear studs to the beam with a traditional arc welding process rather than the stud welder commonly used on new projects. Is this an acceptable practice? If so, what requirements apply to the stud welding?

It is an acceptable practice to manually weld shear studs. Shear stud requirements are covered in AWS D1.1 Clause 7. See Section 7.5.5, "FCAW, GMAW, SMAW Fillet Weld Option," which states:

"At the option of the Contractor, studs may be welded using prequalified FCAW, GMAW, or SMAW processes, provided the following requirements are met..." Following this are seven different subsections, including requirements for determining the minimum fillet weld size and for making the welds.

Brad Davis, S.E., Ph.D.

NDT Responsibility

Who is responsible for performing NDT of complete-joint–penetration (CJP) groove welds made in an AISC-Certified fabrication shop?

This is a contractual issue and can vary between projects. ANSI/AISC 360 Chapter N, for example, allows that an approved fabricator can provide NDT in lieu of having it done by a third-party inspector when approved by the authority having jurisdiction. Certified fabricators provide evidence during their audit that they either have qualified NDE personnel on staff or that the service is available by subcontracting to an outside firm. This provides assurance to the auditor that the fabricator is knowledgeable and capable if NDT is included in their contract.

It is very common for specifications to require NDT services to be provided by an independent agency contracted by the owner, and for fabricators to exclude this service in their proposal. The fact that a fabricator is Certified does not imply that they are responsible for performing NDT.

Keith Landwehr

Fatigue Design of Anchor Rods

I am having difficulty understanding how to properly analyze anchor rods subject to cyclic loading for fatigue. I am using AISC 360 Appendix 3 Section 3.4 Equation A-3-6 and Equation A-3-1. If I understand properly, my maximum stress is limited to a threshold stress of 7 ksi, per Table A3.1 Case 8.5. This seems extremely low. Can I use a higher stress range if I use a material with a higher yield strength?

There are two triggers that require you to address the fatigue resistance of structural components. These are identified in ANSI/AISC 360 Appendix 3 Section 3.1. Evaluation for fatigue is required if (1) the number of cycles is greater than or equal to 20,000 and (2) the live load stress range is greater than or equal to the threshold stress range, F_{TH}. Both must be true for fatigue evaluation to be required.

In Equation A-3-1, the threshold stress range, F_{TH}, sets a minimum allowable stress range, not a maximum. F_{TH} represents the stress range below which fatigue resistance is not required to be considered, regardless of the number of cycles. Thus, there is no need to take your allowable stress range less than the threshold stress range.

The calculation of the allowable stress range, F_{SR}, is independent of F_y or F_u values. However, static loading limits addressed in the main body of ANSI/AISC 360 (typically based on F_y and F_u) still need to be investigated in conjunction with fatigue evaluation.

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

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HSS Connections

AISC *Specification* Section K1.3, Table K1.2 gives a design equation for the limit state of shear yielding (punching) when $0.85B \le B_p \le B\text{-}2t$. There is not an equation given for when $B_p \le 0.85B$. Is this correct?

The range given for HSS shear yielding (punching) in both the 2005 and 2010 versions of ANSI/AISC 360 is correct. This range, although narrow, represents the range where this limit state may control the performance of the connection. Outside of this range, other limit states will control. There is a brief discussion of this in Section 6.3.2 of CIDECT *Design Guide* #9, which is available (for AISC members only) at **www.aisc.org/hss**.

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

Seismic Design of Anchor Rods

In the design of a column base for SCBF, how does the exception in ANSI/AISC 341-05 Section 8.5 apply to ACI 318 Appendix D Section D.3.3?

The exception in AISC 341-05 Section 8.5 eliminates both the 0.75 anchor strength reduction factor and the base anchorage ductility requirements in ACI 318-05 Appendix D Sections 3.3.3 and 3.3.4, respectively. The path through the code that justifies this is as follows:

1. The exception in AISC 341-05 applies to those locations in ACI 318-05 Appendix D that contain the following text: "regions of moderate or high seismic risk or for structures assigned to intermediate or high seismic performance or design categories."
2. The portions of ACI 318-05 Appendix D that contain the quoted text are Sections 3.3.2, 3.3.3 and 3.3.4.
3. Because Section 3.3.4 is excluded, Section 3.3.5 becomes inapplicable as well. Specific to the SCBF system that you mentioned, the ductility is being designed into the brace. In general, the rest of the frame, including the column base, is being designed to support the ductile behavior of the brace, while remaining nominally elastic.

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

Prying Action and A307 Bolts

Is the prying action design procedure in Part 9 of the 14th Edition AISC *Steel Construction Manual* applicable to connections using A307 bolts?

Yes. The prying action design procedure in the *Manual* is appropriate for use with A307 bolts. AISC treatment of prying action involves a mechanics-based approach applicable to both pretensioned and non-pretensioned joints.

Heath Mitchell, S.E., P.E.
(with help from James M. Fisher, P.E., Ph.D.)

Design of Bolted Fillers

AISC 360-10 Section J5.2(d) allows the following option for the design of bolted joints with fillers greater than ¼ in. thick: "The joint shall be designed to prevent slip in accordance with Section J3.8 using either Class B surfaces or Class A surfaces with turn-of-nut tightening." Does the turn-of-nut tightening requirement apply to joints with Class A surfaces only or to joints with either Class A or Class B surfaces?

The intent is that you can use either Class B surfaces (with any pretensioning method), or Class A surfaces with turn-of-nut pretensioning. The slip resistance of Class B surfaces has a low variability. The higher variability in slip resistance of Class A surfaces is offset if you use the turn-of-nut pretensioning method, which tends to produce a higher pretension.

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

Column Splice Design

Are all column splices required to be designed as slip-critical connections?

No. Column splices are allowed to be designed as bearing connections—unless one of the specific requirements for slip resistance is applicable. In general, bolted column splice design requirements are the same as those for other bolted joints. For example, any bolted connection that uses oversized holes is required by ANSI/AISC 360 Section J3.2 to be designed as slip-critical. This applies equally to column splices and other types of bolted joints. There are some provisions that would require column splices to be pretensioned, such as those found in AISC *Specification* Section J1.10 and AISC *Seismic Provisions* Section 7.2, but none specifically require all column splices to be designed as slip-critical joints.

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

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