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steel interchange

P-Delta Effects and Second-Order Analysis

I am having difficulty applying the Direct Analysis Method. If my computer program does a P-Δ analysis using reduced stiffnesses and I have added notional loads to account for P-δ effects, are the results the required strengths? If I do this, do I need to modify the required strengths with B_1 or B_2?

AISC 360 Section C1 requires that stability, including second-order effects, must be considered. Accounting for second-order effects is just one part of the stability analysis and design requirements. The AISC *Specification* recognizes a rigorous second-order analysis (typically performed by a computer program) as one method of second-order analysis. It also provides the approximate B_1-B_2 method that is found in Appendix 8.

Note that the notional loads applied to the structure account for initial out-of-plumbness of each story, but are not directly representative of the P-δ effects as you implied in your question. The notional loads are a way of accounting for one part of Δ in the design. The other part of Δ is drift due to lateral loads. The second-order analysis is what is used to determine the second-order effects (both P-Δ and P-δ) that result from the initial out-of-plumbness and lateral drift of the structure.

So if your software is accounting for *P*-delta effects that include consideration of initial imperfections with notional loads and inelasticity using stiffness reductions per Chapter C, there is no need to also apply B_1 and B_2. The B_1-B_2 method can be used in lieu of the rigorous second-order analysis if the software doesn't do it.

The *Engineering Journal* article "A Comparison of Frame Stability Analysis Methods in ANSI/AISC 360-05" (Q3 2008) provides a good treatment of the stability analysis methods in AISC 360. It is based on AISC 360-05; however, the concepts still apply to AISC 360-10.

Erin Criste

Flow-Drilling of HSS

Page 7-14 in the 14th Ed. AISC *Manual* indicates that "shear and tension strengths of ASTM A325 bolts can be developed for certain combinations of bolt size and HSS wall thickness (see Figure 7-9)." Looking at the figure, it seems to say that a ½-in. A325 bolt may be developed in HSS with 3⁄16-in. and ¼-in. walls but not 5⁄16-in. walls or greater. Am I reading this correctly?

You are understanding this figure correctly. Flow-drill equipment manufacturers have noted that the maximum recommended material thickness is half of the nominal bolt diameter. This recommendation recognizes that the pressure needed to pierce thicker material will likely break the bit. Customized procedures can be used for thicker material, but within the range of normal procedure, half of the bolt diameter is the limit.

Erin Criste

Beam-Column Design

If a member under combined compression and bending moment is classified as "compact" for flexural compression and "slender" for pure compression, then what classification is used in design?

Because of the way the interaction equation in Chapter H works, the section is classified twice: once for axial strength calculations and once for flexural strength calculations. In the general case, AISC 360-10 Table B4.1a is used to classify the section as nonslender or slender, and the axial strength is computed accordingly using the applicable section in Chapter E. This process is the same regardless of whether the member is subject to axial load only or axial load and moment.

Then, AISC 360-10 Table B4.1b is used to classify the section as compact, noncompact or slender, and the flexural strength is computed accordingly using the applicable section in Chapter F. This process also is the same regardless of whether the member is subject to moment only or axial load and moment.

The effect of combined axial load and moment is addressed using the above results for the individual loadings in the interaction equations in Chapter H. The section classifications do not have to be the same for compression and flexure when using the interaction equation.

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

Prying Action in End-Plate Connections

I have been comparing the design procedures for Extended End-Plates (EEP) in AISC *Steel Design Guide 4* and AISC 358-10. The design procedure for prequalified EEPs in AISC 358-10 does not seem to address prying action, while the procedure in AISC *Steel Design Guide 4* does. Is this correct?

That is not correct. AISC 358-10 does consider prying in the design of moment end plates.

- Equations (6.10-3) and (6.10-4) in AISC 358-10 and Equation 2.7 in AISC *Steel Design Guide 4* are the same equation written in somewhat different forms.
- Equation (6.10-5) in AISC 358-10 and Equation 2.10 in AISC *Steel Design Guide 4* are the same equation written in somewhat different forms.
- Equation (6.10-13) in AISC 358-10 and Equation 2.10 in AISC *Steel Design Guide 4* are the same equation written in somewhat different forms.

The above equations work together to ensure that the connection designed is consistent with the thick plate model; prying action need not be considered.

Larry S. Muir, P.E.

Flange Bending

A new pipe rack will be attached to the bottom of an existing plate girder, hung from the flange tips and effectively applying a point load at the outside edges of the existing flange. Is AISC *Specification* Section J10.1 *Flange Local Bending* applicable to this detail?

No, the Section J10.1 flange bending check doesn't apply to the detail you have described. See AISC *Steel Design Guide 13* Section 2.2.2 and the Commentary to AISC *Specification* Section J10.1. The flange bending check is intended to prevent stress concentrations at tension connection plates welded across the flange.

For the case you have, here are some examples of how others have addressed the bending:

- In a contribution to the 12/1999 Steel Interchange, David Ricker proposed a simplified procedure for underhung cranes: **www.modernsteel.com/121999_SI**
- The Material Handling Industry (**www.mhi.org**) publishes CMAA 74, which provides a method for design of underhung cranes that also addresses this topic.
- Yield-line analysis could be used to compute the strength, and the localized deflection can be determined by finite element analysis or a manual approximation.
- Carden et al. provide solutions that relate to this in "Investigation of Flange Local Bending Under Flexible Patch Loading" in the Q1 2008 issue of *Engineering Journal*.

Also, don't forget to check the flange-to-web weld. Several of the methods suggested above will provide enough information to estimate the required strength of the weld.

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

Slip-Critical Joints with Fills

Can a slip-critical bolted connection be used with fillers (or shims) up to 1 in. thick? Are multiple plies of shims or fillers allowed to make up the 1-in. gap?

Yes, you can use slip-critical connections with 1-in.-thick fillers as long as you meet the requirements of the AISC *Specification*. There are two separate issues here: the use of fillers in slip-critical connections and multiple fillers in slip-critical connections.

Regarding the use of fillers in slip-critical connections in general, AISC *Specification* Section J5.2 provides an option specific to slip-critical joints. Option (d) provides that the filler can be accounted for by using SC Class B surfaces; it also allows for SC Class A surfaces if the bolts are installed by the turn-of-nut method. These two alternatives describe the cases in which the variability of slip is low enough that the presence of the filler will not affect the joint.

If you have SC Class A surfaces and the bolts are installed by a method other than the turn-of-nut method, there is enough variability in slip resistance that one of the other options (a, b or c) in Section J2.5 will be required to address the presence of the filler. As these other options are all related to bearing strength, this may seem confusing. However, the point is that this case has a greater likelihood of slip occurring and the bolt shear strength must either be reduced (J5.2a) or the number of bolts increased (J5.2b or c) to account for the presence of the filler.

The second part of your question, about using multiple fillers, also requires consideration because bolt bending can occur when multiple fillers are used. Section J3.8 in the AISC *Specification* specifies that the filler factor, h_f, is 1.0 for a single filler and 0.85 for multiple fillers.

Carlo Lini, P.E., and Charles J. Carter, S.E., P.E., Ph.D.

Diagonal Bracing Connection Design

The Section in the 14th Ed. AISC *Manual* entitled "Force Transfer in Diagonal Bracing Connections" in Part 13 discusses three methods that have been shown to yield safe gusset plate designs and gives a reference to a paper by Thornton from 1991. The *Manual* has adopted one of these—the Uniform Force Method (UFM). Is the UFM the only diagonal bracing connection design method allowed by AISC? Does AISC deem the other two methods referenced in the 1991 Thornton paper (Method 2A and 4) acceptable?

The AISC *Specification* establishes requirements that become law when they are adopted by the building code. Although some of the AISC *Manual* incorporates these requirements, the rest of the *Manual* is a compilation of recommendations, not requirements.

The Uniform Force Method is a recommendation, not a requirement. In the view of the AISC Committee on Manuals, this method best predicts the available strength and critical limit state of the connection. Note that it also allows for flexibility in application of the method, including the special cases presented and other practices, such as moving work points (with appropriate compensating analysis requirements) for convenience in design.

The AISC *Manual* does not address the other methods, and the use of these methods is a matter of engineering judgment. The 1991 paper by Thornton referenced in the *Manual* provides guidance for these alternative approaches. In general, any method that satisfies equilibrium of internal forces and uses materials and connections with sufficient ductility to redistribute those forces is an acceptable method of design.

Thornton, W.A. (1991), "On the Analysis and Design of Bracing Connections," National Steel Construction Conference *Proceedings*, pp. 26.1–26.33, AISC, Chicago, IL.

Carlo Lini, P.E.

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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:

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