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**.

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Code of Standard Practice

It occurred to me that the title of AISC 360-05 is the *Specification for Structural Steel Buildings*, but the title of AISC 303-05 is the *Code of Standard Practice for Steel Buildings and Bridges*. I was wondering why AISC 303 has "Bridges" in the title, while AISC 360 does not.

This is just an item of history. When AISC began to work toward standardization in the early 1920s, bridge design requirements were already being developed by predecessor groups to AASHTO. The building design requirements still needed to be standardized, and AISC did that by writing the AISC *Specification*, AISC *Manual* and other design-related documents.

The "founders" of AISC realized that a standard basis for buying and selling structural steel was also needed. They developed this as the AISC *Code of Standard Practice* and recognized that this function was needed for both buildings and bridges. Hence the difference in the title.

I am always amazed that my predecessors so many generations ago were so prescient. Today, we still maintain the documents they invented, and they essentially created the lifeblood of the steel design and construction process in their first try. Very humbling!

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

Plate Flexure

How is a single cantilevered connection plate designed for flexure?

The 14th Edition AISC *Steel Construction Manual* has a section on flexural design of connection elements starting on page 9-6. You may also want to refer to the discussion of how these provisions are applied to conventional and extended single-plate connections. This discussion begins on page 10-102 of the 14th Edition AISC *Steel Construction Manual*. To quickly summarize for the general condition of a cantilevered connection plate that behaves as a connection element and not as a member:

- Strong-axis flexural effects typically will not govern the design for short elements (similar to the conventional single-plate). As the plate length increases (such as for the extended single-plates) the plate can be treated similar to a double-coped beam.
- Use Section F11 the 2010 AISC *Specification* (included with the 14th Edition *Manual*) for weak-axis flexural strength.

If the plate is long enough that it begins to behave as a member, then the requirements of AISC *Specification* Section F11 apply about both axes.

Larry Muir, P.E.

HSS Connections in OMFs

I am looking at the feasibility of using HSS for ordinary moment frames (OMFs). The AISC *Seismic Design Manual* addresses wide-flange moment frames but says that HSS could be used. Are there any design guides that address the use of HSS for moment frames?

HSS can be used in OMF. The connections used depend upon the configuration and members being framed, as well as fabricator preferences. There is information that will help you in the 14th Edition AISC *Steel Construction Manual* (**www.aisc.org/bookstore**) and AISC Steel Design Guide 24 *Hollow Structural Section Connections* (**www.aisc.org/epubs**). These documents combined have information that updates what we published in the late 1990s as the AISC *HSS Connections Manual*, which is no longer in print.

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

Use of CMTR Values in Design

AISC N690-06, *Specification for Safety-Related Steel Structures for Nuclear Facilities*, Appendix N5, *Evaluation of Existing Structures*, Section N5.2.2, Material Properties, Tensile Properties, discusses the use of actual properties from a certified mill test report (CMTR). Does this mean that AISC allows the use of yield stress from CMTRs?

AISC N690-06 Appendix N5 states that one can justify use of the properties from the CMTR when it can be shown that it properly represents the member being analyzed, and also that the value is based on a proper statistical analysis. Each of these criteria is discussed further below.

1) Properly representing the member being analyzed. This requires traceability on the project and proper record keeping of project documents over time. There can be many CMTRs on a project. It is important to make sure the CMTR used is representative of the member being investigated. For example, this means if you are analyzing a beam, the CMTR must be associated with beams, and not columns or plates or some other shape. Also, the material supplied can come from many producers and heats, so two wide-flange beams might have the same designation (such as W21×44) but they might come from two different producers or they may be manufactured by the same producer, but come from different heats of material.

2) Value based on proper statistical analysis. CMTRs represent the results of testing that a mill does in compliance with ASTM standards to label the material they produce. The CMTRs available on a given project may not create a sufficient statistical basis on their own, and coupon testing may be needed to provide the confidence for using the CMTR properties.

Thus, AISC N690 is based on the minimum specified yield and tensile strengths. Provision is made for use of values higher than the minimums but will require a basis and the use of engineering judgment.

Erin Criste

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ASTM A709 Availability

The scope of ASTM A709 states that it covers structural shapes, plates and bars. However, question 10 in the September 2011 Steel Quiz states that ASTM A709 Grades HPS 50W, HPS 70W and HPS 100W high-performance steels are available only as structural plate. Since structural shapes are within the scope of the ASTM A709, is it correct to assume that production of structural shapes in these high-performance grades is allowed, but mills currently are not producing them?

No, it depends upon the grade. ASTM A709 is a "blanket specification" that addresses both structural shapes and structural plate. The specific grade designations identify if they apply to shapes, plates or both. For example, ASTM A709 Grades 36, 50 and 50W are available as either shapes or plate (note, these are equivalent to A36, A572-50 and A588). ASTM A709 Grade 50S is available only as shapes (this is equivalent to A992). The HPS grades are available only as plates. All of these grades are produced by at least one domestic steel mill.

Bill McEleney

Stiffness Reduction Factor

In using the Alignment Charts (Figures C-A-7.1 and C-A-7.2) for the Effective Length Method in Appendix 7 of AISC 360-10, I noticed that the Stiffness Reduction Factor (Table 4-21 in the 14th Edition AISC *Steel Construction Manual*) has been increased as compared to what was given in the 13th Edition. I also notice that the notation has changed from τ_a to τ_b. In AISC 360-05, the Commentary to Chapter C provides Equation C-C2-12, but I have had a hard time finding the governing equation in AISC 360-10. Could you please let me know where the governing equations are and explain the increase in the Stiffness Reduction Factor?

In the 2010 AISC *Specification* (AISC 360-10), stability provisions were reorganized from the format found in the 2005 AISC *Specification* (AISC 360-05). The Direct Analysis Method is now in Chapter C and the Effective Length Method is in Appendix 7 (along with the first order analysis method). This means that the alignment charts are now in the Commentary to Appendix 7 (starting on page 16.1-512).

The 2005 *Specification* contained two stiffness reduction factors τ_a and τ_b. The decision was made to simply use one factor in the 2010 *Specification*. An AISC *Engineering Journal* paper by Geschwindner titled "A Case for a Single Stiffness Reduction Factor in the 2010 AISC *Specification*" (see the 1st Quarter 2010 issue) explains the differences in the various stiffness reduction factors, how they are derived and why we now only have the one reduction factor τ_b.

To quote a portion of this article:

> "There are three stiffness reduction factors available for use with the nomograph…If the intent is only to use the stiffness reduction factor to modify the column end stiffness ratios, G, for use with the nomograph, it has been shown that any of these approaches will provide satisfactory results."

"The SRF1969 and τ_a are based on strength equations that include initial out of straightness and residual stresses while τ_b is based on column strength equations that include only the effects of residual stresses…Since the intent of the stiffness reduction factor, in all cases, is to include only the influence of inelastic behavior due to residual stresses, clearly τ_b should be used. Thus, the stiffness reduction factor for both the nomograph and the direct analysis method should be taken as τ_b."

The changes made in AISC 360-10 and the 14th Edition AISC *Steel Construction Manual* followed this recommendation.

Erin Criste

Web Sidesway Buckling

AISC 360-10 Section J10.4 Web Sidesway Buckling uses the term b_f, defined as flange width. I am designing a built-up beam comprised of a monosymmetric I-shaped section with a channel cap. Which flange width am I to use for the calculations in this section: the top flange width, the cap channel width or bottom flange width?

The web sidesway buckling limit state is checking the strength of the tension flange to resist sideway buckling under a concentrated load. The strength is determined based on the lateral stiffness of the tension flange and, if the compression flange is rotationally restrained, the lateral stiffness of the web. For this reason, the flange dimension to be used is that of the tension flange.

The information in AISC 360-10 Section J10 was developed for wide-flange and similar shapes, so it can require use of judgment when applying it to other shapes. In your case, it can be directly applied if the tension flange of your built-up shape is the flange without the channel cap.

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

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Heath Mitchell is director of technical assistance, Charlie Carter is vice president and chief structural engineer, and Erin Criste is staff engineer, technical assistance at AISC. Bill McEleney is director of NSBA. Larry Muir is a consultant to AISC.
