New HSS standards and changes to slender elements, shear strength, angles, connections, advanced analysis and fire design are just a few of the updates in the 2016 AISC *Specification*.

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# WHAT'S NEW IN THE SPEC?

BY SAM BAER AND MATTHEW TROEMNER

**AISC IS SET** to release the 2016 edition of the *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) in the near future.

Changes from the 2010 edition reflect the Committee on Specifications' desire to implement only essential changes that reflect new research, provide for more efficient designs or broaden its scope. Many of these changes were technical in nature, though edits were also made that focused on improving usability, transparency and editorial content.

The following is a brief overview of the most significant changes, some of which have the potential to substantially affect design procedures. A complete list of differences between the 2010 and 2016 AISC *Specification* will soon be available as a free download at **www.aisc.org/manualresources**.

## HSS Standards

ASTM A1085 and ASTM A1065 have been added to the 2016 *Specification* as approved hollow structural section (HSS) material standards. ASTM A1085 is a newly developed material standard that has more stringent requirements than other already approved HSS standards (such as ASTM A500), including a mass tolerance and a stricter wall thickness tolerance. As a result of these requirements, the design wall thickness can be taken as the full nominal wall thickness.

ASTM A1065 is an HSS standard based on the use of already approved ASTM plate material standards. Due to the similarly reduced tolerances of these existing plate standards, design wall thickness for A1065 can also be taken as the full nominal wall thickness. It is important to note, however, that the 2016 *Specification* still requires that the design thickness of other HSS materials, including the more common ASTM A500, be taken as 0.93 times the nominal wall thickness.

Additionally, ASTM A1085 and A1065 specify a minimum yield stress of 50 ksi regardless of shape. Further information on the benefits and impact of A1085 is detailed in the September 2013 article "Hollow Product, Solid Benefit," available at **www.modernsteel.com**.

## Slender Elements in Compression

The method for determining compressive strength of members with slender elements has been revised in the 2016 *Specification*. Since 1969, the *Specification* used an approach centered on a reduction factor, $Q$, which modified the column critical stress. For slender unstiffened elements, $Q$ was given by equations that included the width-to-thickness ratio of the element and was a constant for a particular shape, regardless of the load on the column; these $Q$ values were tabulated in the *Manual*. For stiffened elements, $Q$ was based on the ratio of a reduced effective area to the gross area of the member and was a function of the magnitude of the column stress.

While the 2010 *Specification* used a reduced effective area approach for stiffened elements, this methodology has now been refined and expanded to include both stiffened and unstiffened elements. The new provisions determine the reduced effective area and use that along with the unmodified column critical stress to determine compressive strength. As in the past, classification of members as slender element members, as determined in Table B4.1a, is based on the assumption that the column stress has reached the yield stress, $F_y$. Thus, members that were considered slender in prior editions of the *Specification* continue with that same designation. However, because the magnitude of the stress on the column influences the local buckling of the member elements (for all but round HSS), members that have been designated as slender element members may not actually experience a reduction in strength due to that slender element. For round HSS, the effective area is based on the diameter-to-thickness ratio and the yield stress of the material.

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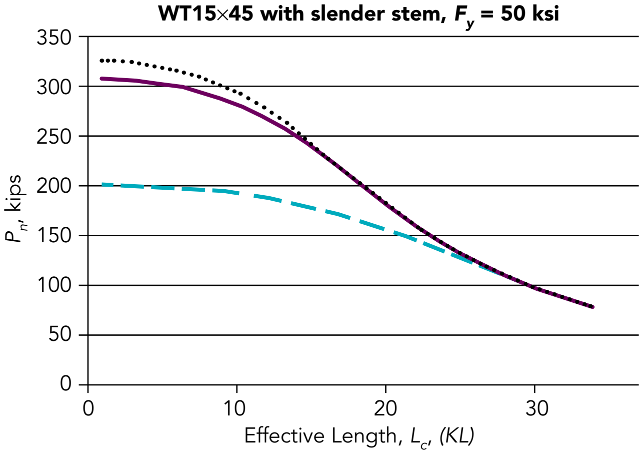
▲ The provisions for double angles and tees in flexure have been reorganized for clarity.

▲ ASTM A1085 and ASTM A1065 have been added to the 2016 *Specification* as approved HSS material standards.

The changes in determining compressive strength for members with slender elements have significantly altered the nominal compressive strength of select steel shapes (the difference for one such case is shown in Figure 1). Further information on the impact of this change and other affected shapes can be found in the third-quarter 2016 *Engineering Journal* article "Notes on the AISC 360-16 Provisions for Slender Compression Elements in Compression Members."




▲ Figure 1. Comparison of available compressive strength to effective length from the 2010 to the 2016 *Specification*.

## Shear Strength for I-Shaped Members and Channels

Chapter G of the 2016 *Specification* includes two significant changes in the provisions for I-shaped members and channels. The provisions for determining shear strength without consideration of tension field action have been revised to allow for inclusion of some post-buckling strength. This leads to an increase in available shear strength for certain built-up girders. In addition, the web plate shear buckling coefficient has been increased from 5 to 5.34 to better reflect its theoretical derivation. Since all W-shapes have webs that will be controlled for shear by the limit state of yielding, these changes will not impact the shear strength of these members.

For tension field action, several of the restrictions found in the 2010 *Specification* have been relaxed—designers may see some increased shear strength for interior panels of beams with stiffener spacing less than or equal to the height of the beam web in cases that previously could not have taken advantage of tension field action.

With the increase in available shear strength, the requirements for stiffeners have been increased. If the increase in available shear strength is to be used, larger transverse stiffeners than were required by the 2010 *Specification* may be necessary.

## Angles, Double Angles and Tees in Flexure

The provisions for double angles and tees in flexure have been reorganized so that the distinction between their provisions is clearer. In addition, the lateral-torsional buckling provisions for stems and legs in tension are given separately from the provisions for stems and legs in compression. Flange local buckling provisions for tees have remained unchanged but for double angles it is now clear that strength should be determined as that of two single angles. Stem local buckling of tees was added to the 2010 *Specification* for the first time and these provisions have been revised in 2016 to more correctly reflect the strength for this limit state. As with flange local buckling, stem local buckling of double angles is to be assessed as for two single angles.

The 2016 *Specification* contains revised provisions for the lateral-torsional buckling limit state of single angles that are simply a reorganization of the previous editions equations. The nominal moment strength is a function of the elastic lateral-torsional buckling moment, formerly defined as $M_e$ and now as $M_{cr}$, which is given for bending about the major principal axis of all single angles or the special case of bending about the geometric axis of equal leg angles.

## Connections

The requirement for length of longitudinal fillet welds when used alone in end connections of tension members has been revised. The requirement from Section J2.2b—that weld length be at least equal to the distance between the parallel welds—has been replaced by a revised approach for calculation of the shear lag factor in Section D3 for longitudinal welds in end connec-

Welding requirements have seen some changes in the new spec.

tions of tension members. Thus, shorter welds, as well as different length parallel welds, are permitted, but the resulting shear lag factor will be small and the strength of the member will be reduced accordingly. In addition, the specific situational limitations for fillet weld terminations, stopped short or extended, have been removed and replaced with a performance-based requirement that the termination does not result in a notch subject to applied tension loads. The weld also must not prevent deformation required to provide assumed design conditions.

Two new ASTM bolt standards originally approved in 2014, that address bolts with a minimum tensile strength of 200 ksi, have been approved for use in the 2016 *Specification*. These bolts have been incorporated into the *Specification* via the designation Group C bolts and cover ASTM F3043 and F3111 material. To date, these are only available as proprietary products, but ASTM encourages other producers to propose alternatives. Another change implemented by ASTM that impacts the *Specification* is the development of a summary bolt standard, F3125 which includes the former A325, A490 and similar standards as grades. For Group A bolts of diameters greater than 1 in., the specified minimum bolt tensile strength had previously been 105 ksi and is now the same as for smaller bolt diameters: 120 ksi. This increase results in a higher available slip resistance for larger Group A bolts.

The requirement of pretensioned bolted connections in multistory structures over 125 ft for column splices and connections of beams and girders bracing columns has been removed, since it was arbitrary and could not be supported by any technical rationale. Additionally, for bolts of diameter 1 in. and greater, the maximum nominal diameter of standard size holes and the maximum nominal width of short-slots and long-slots has been increased by 1⁄16 in. The increase in maximum allowable hole size provides for greater ease of erection when connections make use of large-diameter bolts.

Chapter K in the *Specification* was changed from "Design of HSS and Box Member Connections" to "Additional Requirements for HSS and Box Section Connections" to reflect a change in approach that uses the requirements of Chapter J when applicable to HSS connections and only uses Chapter K for specific requirements pertinent to HSS and box-section connections. This resulted in the Chapter K tables being revised to reflect a reduction in the types of connections covered in that chapter, with the other cases treated according to the more fundamental approaches provided in Chapter J.

## Analysis and Fire Conditions

Appendix 1 permits the use of analysis methods that are more sophisticated than those normally used in design. Section 1.2 has been added to Appendix 1 in the 2016 *Specification* to permit elastic analysis that includes direct modeling of system and member imperfections. The advantage of this analysis approach is in the determination of compressive strength using only the member cross-section strength without the need to consider member length effects.

For fire design, the 2016 version of Appendix 4: Structural Design for Fire Conditions includes two additions. A new table relating bolt temperature to available strength is provided, and a simplified method is presented for calculating the nominal flexural strength of a composite beam using the bottom flange temperature. This new method incorporates the use of a tabulated retention factor dependent on the bottom flange temperature and the nominal flexural strength of the composite beam at ambient temperature, calculated according to the provisions of Chapter I.

These are just some of the changes to the *Specification* and should not be thought of as the only ones that might impact a particular project. A complete review is highly recommended so that one is familiar with the new version when building codes that incorporate it are adopted. For additional information on the major differences between the 2010 and 2016 *Specification*, take a look at the 2016 NASCC: The Steel Conference presentation "An Overview: The 2016 AISC Specification for Structural Steel Building," available at **www.aisc.org/2016nascconline**. ■

The new version of the specification includes a handful of significant updates to bolted connections.