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Width-Thickness Limits

When reviewing the width-thickness ratios of elements in a custom shape, I found the term "NA" under λ_p in ANSI/AISC 360-05 Table B4.1. Does that signify that for this case, the shape having this element may be considered compact until the width-thickness ratio of that element reaches the limit defined by λ_r?

No. Some of the cases in Table B4.1 are for use when computing the axial strength, P_n, per Chapter E of the AISC *Specification*, while some are for computing the flexural strength, M_n, per Chapter F.

For compressive strength calculations, the AISC *Specification* Chapter E considers two situations for a W-shape column and other types of sections are similar: those with and those without slender elements. A section with non-slender elements can achieve yielding at the flange tips (note this does not necessarily mean full yielding of the entire section, which is not required in a column) before local buckling. A section with slender elements cannot. Therefore, only one limiting *b/t* ratio is required to define the boundary between slender and nonslender elements. These are called λ_r and Case 3 in Table B4.1 is an example. The key word in the description is "uniform compression" which indicates that they are used for a column.

Contrast this to the situation for flexural strength calculations. Chapter F of the AISC *Specification* considers three situations: compact, noncompact and slender elements. Sections with compact elements are able to achieve M_p and a significant inelastic rotation before the elements locally buckle. Sections with noncompact elements can achieve some yielding, but not full yielding, before an element locally buckles. Sections with slender elements cannot achieve yielding before an element buckles locally. Because there are three classifications, two boundaries must be provided. λ_p is the boundary between compact and noncompact. λ_r is the boundary between noncompact and slender.

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

Snug-Tight TC Bolts

Is it acceptable to use twist-off type tension control bolts (TC bolts) in a connection specified as snug tight? If TC bolts are used in a connection specified as snug-tight, are the procedures for pre-installation verification, installation (snug tight joints, then pretension by starting with most rigid part of joint) and inspection required?

It is acceptable to use TC bolts in a snug-tight joint. Heavy hex head and TC bolts are treated as equivalents in the AISC *Specification* and will actually be grouped together and referred to as "Group A" or "Group B" bolts in the 2010 AISC *Specification* to clarify this. (The 2010 AISC *Specification* is now available as a free download at **www.aisc.org/2010spec.**) Group A includes ASTM A325 and F1852. Group B includes ASTM A490 and F2280.

Preinstallation verification and pretentioned installation and inspection procedures are not required for a TC bolt used in a snug-tight joint. A TC bolt used in a snug-tight joint is subject to the same requirements as a heavy hex head bolt used in a snug-tight joint—no more and no less.

Larry S. Muir, P.E.

Production Lots for Bolts

How many high-strength bolts comprise a typical lot?

There is no standard practice for lot size in the industry. There is a wide variation of lot size allowed in ASTM, and a large variation occurs in practice. The relative demand for a specific ASTM designation, a specific fastener diameter, as well as for a particular length within that diameter each plays a role. In addition, there are preferred production practices that vary by manufacturer. Some specialty fastener manufacturers may produce a dozen fasteners in a lot, whereas the lot size for other manufacturers making a more typical grade of fastener can range from the tens of thousands to the hundreds of thousands of units.

Charles E. Hundley

Fillet Weld for a Skewed Connection

What is the practical maximum angle between two pieces of steel that may be connected by a fillet weld on the "open" side? Clearly at a 180° angle, a true fillet weld is no longer feasible; but at what point between 90° and 180° does it become unreasonable to consider the use of a fillet?

This is one of those instances where a picture is worth a thousand words. The answer to your question is found in Figure 3.11 of AWS D1.1. The maximum angle is 135°, if the weld is to be applied to the face of the plate. However, detail (C) of Figure 3.11 indicates a weld applied to the edge of plate (which in some instances will have to be prepped/beveled). In this arrangement, the 180° angle you thought out of the question is possible.

Larry S. Muir, P.E.

Plastic Design

What does plastic design of steel mean?

Plastic design is a design methodology that once was fairly common for steel structures. At the present time, it is not used as frequently because it is somewhat difficult to implement using a computer program and most engineers rely on computerized analyses. Note that many of the benefits of plastic design are already captured in the basic design methods we have used in ASD and LRFD for many decades.

In plastic design, the basic idea is to utilize structural continuity to redistribute moments and find the load that would cause actual collapse of the continuous beam or frame that's being designed. A simple example is a two-span continuous beam with equal spans and equal loads. The negative moment at the middle support is larger than the positive moments near the middle of the spans. At some magnitude of load, the negative moment will reach the plastic moment, M_p, and remain constant as additional load is applied. This application of additional load will not cause the beam to collapse, however. Moment redistribution will take place and the positive moments will increase. At some point, the load will be large enough that the positive moments reach M_p and a "collapse mechanism" results. No additional load can be applied. The same idea is used in yield line analysis for plates and slabs.

Plastic design is described in some advanced steel design textbooks and specifically addressed in AISC *Specification* Appendix 1.

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

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Shear Strength of Round HSS

The 1989 AISC ASD *Specification*, Section F4 specifies the allowable shear stress for round HSS as $F_v = 0.4F_y$. The 2005 AISC *Specification*, Section G6 specifies $V_n = F_{cr}A_g/2$ and $F_{cr} \leq 0.6F_y$ for round HSS. Considering that $\Omega_v = 1.67$, the 2005 *Specification* results in a maximum allowable stress, $F_v = 0.18F_y$. Why does the current steel code penalize the shear capacity of the round HSS by a factor of 2 from the previous code?

In the 1989 ASD *Specification*, Section F4 states that the allowable shear stress is $F_v = 0.4F_y$, which is the same as $F_v = 0.6F_y/1.50$, where the 1.50 corresponds to what we now call Ω. It does not indicate how f_v is to be calculated for a pipe or other hollow round shape. Therefore, that step is left to the engineer.

The value of f_v could be determined using the mechanics of materials equation, $f_v = VQ/(It)$. If you set $f_v = F_v$, you can back out the allowable shear force, $V_a = F_v(It)/Q$. The quantity $(It)/Q$ can be thought of as the shear strength area. This area is roughly equal to the $A_g/2$ term shown in 2005 AISC *Specification* Equation G6-1. In other words, the appropriate shear stress area to use for round HSS with both the 1989 ASD *Specification* and the 2005 *Specification* is $A_g/2$.

You are correct that the 1989 *Specification* does give a higher allowable strength, but that's because the 2005 *Specification* uses Ω = 1.67 for round HSS as compared to the value of 1.50 that was used in the 1989 *Specification*. The comparable allowable shear stress based on the 2005 *Specification* is $F_v/\Omega = 0.6F_y/\Omega = 0.36F_y$. This is very close to the $0.4F_y$ allowed in the 1989 ASD *Specification*.

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

Connection Design

What is the standard of practice for connection design by the Structural Engineer of Record (SER)? Is it sufficient for the SER to provide connection loads and require an engineer working for the fabricator to provide the connection designs? Is the delegation of connection design appropriate for connections related to lateral force resisting systems? What about "special" seismic systems in moderate-high Seismic Design Categories?

Standard practice related to connection design can vary from one industry sector to another and from one region of the country to another. It is common in the East for connection design to be delegated to the fabricator. In the West this practice is less common, but it does happen in all regions.

Revisions have been made to Section 3.1.2 of the 2010 AISC *Code of Standard Practice* to clarify the requirements when the SER delegates connection design. Connection design delegation is referred to as "option 3" in the 2010 AISC *Code*. Provisions covering connection design also have been made in Section 4 to correspond with the addition of option 3 in Section 3.1.2. One of the key elements to successfully delegating connection design is proper communication of the loads and any other requirements placed on the connection design by the SER; that and many other considerations are addressed in the way the 2010 AISC *Code* has been written.

Design of connections within the lateral force resisting system can be delegated. This applies equally to structures with low and high seismic demand. Note that the connections and members often are interrelated in their designs, and care is required to communicate when connection design is delegated.

Larry S. Muir, P.E.

Static Loads on Bolts

Section 4 of the 2009 RCSC *Specification* allows snug-tight shear bearing joints with tension as long as the tension load is "static." What is the definition of a "static" load? Are wind or seismic loads considered to be static loads?

Wind and seismic loads are considered to be static loads (as opposed to loads that cause fatigue). This is stated in Section B3.9 of the 2005 AISC *Specification*. The 2010 AISC *Specification* will more clearly define "statically loaded" as "not subject to significant fatigue stresses. Gravity, wind and seismic loadings are considered to be static loadings."

ASTM A325 bolts subjected to forces resulting from wind and seismic loads are not required to be pretensioned in the general case. However, if one of the cases requiring pretension in Section 4 of the RCSC *Specification* applies, pretension may be required for another reason. Similarly, if the structure must meet the requirements of the AISC *Seismic Provisions*, AISC 341-05 Section 7.2 requires that all bolts in the seismic load resisting system must be pretensioned.

Larry S. Muir, P.E.

Prying Action

When evaluating prying action using the 13th Edition AISC *Steel Construction Manual*, the equation for t_c has changed from the previous editions. The equation in the 13th Edition uses F_u whereas the equation in the previous editions is based on F_y. Why did the 13th Edition change to base the calculation of t_c on F_u?

This calculation was changed in the 13th Edition *Steel Construction Manual* because the use of F_u in the calculation of t_c results in a better match to the available test data on prying action. It may seem less "rational" to use F_u, but its use in this equation provides a better prediction of actual strength and behavior.

Amanuel Gebremeskel, 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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