steel interchange

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Nut Tightening on Anchor Rods

When considering nut installation on a threaded anchor rod, how is the pretension specified on the drawings? The desire is to prohibit loosening of the nut under load reversal. I see the recommendations for anchor-rod nut installation on pages 14-10 and 14-11 of the 13th edition AISC manual; however, the minimum bolt pretension force shown in Table J3.1 does not seem to apply. Is the required tensile force still $0.7F_u$ of the anchor rod?

Steel-to-concrete anchor-rod installations are a completely different subject than high-strength bolt installations used in steel-to-steel connections. The AISC specification does not require pretensioned installation for an anchor rod. The suggestions given in Part 14 of the *Manual* are recommendations as to how to achieve a tightened connection but are not representative of a pretensioned condition.

In the majority of cases, and particularly for axial compression loaded column bases, anchor rods primarily are present for erection and serve no calculated function in the final structure. In other cases, such as moment bases or bases subject to uplift, the rods are necessary for force transfer. But these details also usually do not require pretension.

If you have a condition where loosening of the nut might be a consideration, such as in vibrating machinery, you may want to consider double-nutting or proprietary thread-locking methods.

If you choose to require pretensioned anchor rods, you would need to define how much pretension is required, how that pretension is to be achieved, and how that pretension is to be maintained against such factors as creep in the concrete and the variations in bond along the length of the rod over time.

Kurt Gustafson, S.E., P.E.

Combined Forces

The title of Section H1.3 of the 2005 AISC specification is "Doubly Symmetric Members in Single Axis Flexure and Compression." This title indicates that this section can be used for doubly symmetric I-sections and rectangular (or round) HSS as well. Bending can be either in the major axis or the minor axis. After reading the Commentary, my understanding is that this section is intended for doubly symmetric I-sections subjected to *major* axis bending only. Is section H1.3 applicable to HSS?

Yes, this Section applies to HSS, and the application is not limited to wide-flange shapes. LTB limit states are rare for HSS, but it is conceivable for this to occur in highly rectangular box shapes. At the top of page 73 in the 2005 AISC specification (a free download at **www.aisc.org/2005spec**), it states that in cases where the single-axis flexure is in the weak axis, it is permitted to neglect the moment ratio in Equation H1-2. This is because weak-axis bending cannot result in LTB. Thus, in this case, in-plane stability is addressed with Equations H1-1 as indicated in Section

H1.3(a), and out-of-plane buckling is then simply a function of the axial load ratio.

The parabolic equation is meant to address the additional capacity that you can get out of a beam-column that has a limit state of flexural buckling (axial) and LTB (flexure) in the same direction. This additional capacity comes from the use of the parabolic formulation, whereas the Equations in Section H1-1 have a straight-line formulation.

Amanuel Gebremeskel, P.E.

Determining r_t

I am searching for information pertaining to r_t. I am looking to see if there are any equations that can be used to calculate this value. I have looked through a couple of text books but have not found an equation for this. Can you point me in the right direction?

The radius of gyration r_t is defined as the radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to the application of major axis bending moment alone. Equation (F4-10) in the 2005 AISC specification (a free download at **www.aisc.org/2005spec**) defines r_t. The User Note also provides a simplification that you can use if you so prefer.

Kurt Gustafson, S.E., P.E.

LRFD or ASD

What is the difference between LRFD and ASD design?

Pages 2-6 and 2-7 of the 13th edition AISC manual (available at **www.aisc.org/bookstore**) provide detailed statements on the two approaches. The 2005 AISC specification supports both approaches with no preference for either one. Section B3, particularly subsections 3 and 4, of the AISC specification (a free download at **www.aisc.org/2005spec**) addresses the difference between these two approaches.

Although more significant differences used to exist between previous LRFD and ASD specifications, we have intentionally brought ASD and LRFD into essential equivalency in the 2005 AISC specification. The difference between them now amounts to whether you calculate your loads using LRFD load combinations from ASCE 7 or ASD load combinations from ASCE 7.

Amanuel Gebremeskel, P.E.

Steel for High-temperature Applications

The Brockenbrough and Merritt text referenced in Part 2 of the 13th edition AISC manual indicates that "For special elevated-temperature applications in which structural steels do not provide adequate properties, special alloy and stainless steels with excellent high-temperature properties are available." Can you direct me to publications on the properties of these special alloy and stainless steels?

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Such steels are not commonly used in building construction and are not covered by the AISC specification. However, *The Making, Shaping and Treating of Steel*, published by U.S. Steel, includes a Chapter on "Steels for Elevated-Temperature Service." This text is still published (I found it through **amazon.com**).

Kurt Gustafson, S.E., P.E.

Diagonal Brace Connection

In considering a brace-to-gusset connection for a SCBF, it is my understanding that the Uniform Force Method is generally the most economical method for determining gusset size and welds (13th edition page 13-3). However, I noticed that most of the examples in the new *Seismic Design Manual* use the Whitmore section (pages 3-58, 3-66, etc.). Are there any advantages and/or restrictions in using the Whitmore section versus the Uniform Force Method? Is there any reason the Whitmore section was used in the *Seismic Design Manual* versus the Uniform Force Method?

These are not two different methods of bracing connection design. Rather, they are both used in the design of bracing connections.

The Uniform Force Method is a bracing connection design method that entails selecting the geometry of the connection so that moments do not exist (or at least are minimized) on the three connection interfaces between the gusset plate, beam, and column. This way one can design these connections for shear and tension only.

The Whitmore section on the other hand allows the calculation of the effective width of the gusset plate to resist the load from the brace in the connection of the brace to the gusset. Please see page 13-3 for examples of the former and 9-3 for examples of the latter.

Therefore, the two checks are applied to solve different problems, and a direct comparison of the two methods is not appropriate. For a gusset plate in a compression brace connection, for instance, the uniform force method would be used to design the connection of the plate to the beam and column, whereas the Whitmore method would be applied to check the plate itself for buckling. Both checks could influence the thickness of the gusset plate, but these address different limit states.

Amanuel Gebremeskel, P.E.

Section Properties of Historic Shape

I recently attended the AISC seminar *Design Steel Your Way* (www.aisc.org/seminars) and received the AISC manual and companion CD. In a project I am working on we have a shape that we cannot locate in the historical shapes database that was included on the CD, nor in any texts that we currently have. I was wondering if you might be able to shed some light on some of its dimensions and properties.

The building was built by the government in 1951. On the drawings the shape is called out as a 12WF19. We are looking to do some analysis on this structure and wondering if you could direct us to where we can find the properties?

It is likely that the shape was incorrectly designated on the drawings as a WF shape. Rather, I think the designer or drafter did not write the correct shape designation. Many of the lighter wide-flange shapes at the time were officially designated as B or BL shapes. Look on the CD under the ASD5 (the 5th edition manual was in effect in 1951) using the designation BLB. You will find a 12-in. beam at 19 lb per foot.

Kurt Gustafson, S.E., P.E.

Strain Hardening

AISC 341-05 requires $1.1R_y$ for the design of some connections and R_y in other places. Why is this so? Should it not be only $1.1R_y$, since I understand the 1.1 is to account for the increase in strength due to strain hardening under cyclic load? If this is the case, all elements would be subject to cyclic loading in a seismic event, and hence $1.1R_y$ should be applicable everywhere.

You are correct that the 1.1 factor is used to take strain hardening into consideration. Note that the flexural checks tend to include the 1.1 factor while the axial checks do not. This is because the strain in flexural members is much greater than that for axial members. In order to understand this, imagine a brace that yields in tension. Before the yielded part can experience strain hardening the entire brace would have to yield, thus elongating the entire brace considerably, by which point the cycle is reversed. This is not the case when a wide-flange yields in flexure where the hinge location is concentrated and the section in tension is forced to go into strain hardening before the cycle reversal.

Amanuel Gebremeskel, P.E.

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Steel Interchange is a forum to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

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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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