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steel interchange

*Note: Unless specifically noted, all AISC publications mentioned in the questions and/or answers reference the current edition and can be found at **www.aisc.org/specifications**.*

Rolled vs. Cut Threads in Threaded Parts

I am designing tension rod bracing and the topic of rolled versus cut threads has come up. As the engineer of record (EOR) do I need to care whether the threads are rolled or cut?

You probably don't need to be concerned about the process used to make the threads. However, this may be of some concern if the anchor rod is subject to bending. I will provide a little background. Note that the following discussion is simplified to suit the purpose at hand. Whether the threads are rolled or cut, the same hardware (nut, clevis, turnbuckle, etc.) will be used. This means the geometry of the threads produced must have essentially equivalent geometry.

For cutting, material is removed to form the threads. To produce the proper fit for the threads, the manufacturer starts with a rod that is equal to the nominal diameter of the rod being purchased. This is probably what most people would expect, so there is generally no concern relative to cut threads.

For rolling, material is displaced to form the threads. Some of the material is moved out of the valley of the thread and forms the crest of the thread. To produce the proper fit for the threads, the manufacturer starts with a rod that is smaller than the nominal diameter of the rod being purchased. The diameter of the rod is roughly the average of the diameters measured at the crests and valleys of the threads, which for common sizes is about 10% smaller than the nominal diameter. This will result in a gross area about 81% of the nominal area of the rod.

The fact that the diameter of the rod is smaller than the nominal diameter can cause concern. Usually, this concern is unwarranted. It does not mean that a rod with rolled threads will have a design strength that is only 81% of the design strength of a rod with cut threads. The following calculations illustrate the effect.

The LRFD design strength of a threaded rod (F_y = 50 ksi, F_u = 65 ksi) with rolled threads is:

- Gross area yielding per the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) Section D2:
  (0.9)(0.81)(50)Ab = 36.5Ab. (D2-1)
- Net tensile rupture per *Specification* Section J3.6:
  (0.75)(0.75)(65)Ab = 36.6Ab. (J3-1)

The LRFD design strength of a threaded rod (F_y = 50 ksi, F_u = 65 ksi) with cut threads is:

- Gross area yielding per *Specification* Section D2:
  (0.9)(50)Ab = 45Ab.
- Net tensile rupture per *Specification* Section J3.6:
  (0.75)(0.75)(65)Ab = 36.6Ab.

The design strength for the rods with rolled and cut threads is essentially the same.

The difference between the actual diameter of the rod and the nominal diameter of the rod can have a greater impact when the rod is subjected to bending. The section modulus of a rod is dependent on its diameter cubed. The actual section modulus can be 30% smaller than the section modulus based on the nominal diameter. The only application for which a threaded rod might commonly be subjected to bending is where anchor rods are relied upon to transfer shear at a base plate. Resisting shear through anchor rods is generally discouraged, but it is not an uncommon practice.

A couple of further points should be addressed. First, the process described above generally applies to threaded parts but does not apply to bolts satisfying ASTM F3125. Though ASTM F3125 permits the threads to be either cut or rolled, the actual body diameter of the bolt must be equal to the nominal bolt diameter.

The calculation of the net tensile rupture per *Specification* Section J3.6 (shown above) assumes that the ratio of the effective tension area of the threaded portion of the bolt to the area of the shank of the bolt is 0.75. This is consistent with the value given for F_{nt} in Table J3.2 of the *Specification*, which is calculated as $0.75F_u$. It is also possible to directly calculate the net tensile area of the threaded rod and apply the net section tensile rupture check from Section D2 of the *Specification*. This will produce slightly different results. It is common practice to apply *Specification* Section J3.6 to threaded parts.

Larry S. Muir, PE

Unframed Ends

Section J10.7 of the *Specification* states: "At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided." What loads should the transverse stiffeners be designed for?

Providing stiffeners at an unframed end is a way of providing sufficient restraint against rotation about the longitudinal axis. Both sufficient strength and stiffness must be present. I suspect that the demand is relatively modest such that a reasonably sized stiffener and minimum fillet weld sizes would suffice. It is likely something that would be judged by inspection to be sufficient. You could also quantify the required strength and stiffness using Appendix 6. Part 2 provides similar guidance in the section on beam ends supported on bearing plates, stating: "In atypical framing situations, such as when very deep beams are used, the strength and stiffness requirements in AISC *Specification* Appendix 6 can be applied to ensure the stability of the assembly."

Carlo Lini, PE

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Eccentricity on Columns

We recently noticed that our structural design software accounts for an eccentricity from the center of the column to some distance beyond the cross-sectional dimensions of the column when a beam is simply connected to the column. Is this a requirement of the *Specification* or an error in the software?

It is neither a requirement of the *Specification* nor an error in the software. The eccentricity from the center of the column to the face of the column is generally neglected by many engineers. This is addressed in the February 2016 Steel Interchange question and answer "Eccentricity on Columns" (available at **www.modernsteel.com**).

As stated in that issue, some software programs account for an assumed amount of connection eccentricity as a default when sizing the columns. Ultimately, the design decisions, whether implemented through manual calculations or through software, should reflect the judgment on the EOR. In my experience, the eccentricity you have described is often defined in the program settings, so you can likely change the assumed eccentricity to match your own preferred design practices.

Carlo Lini, PE

Constrained-Axis Torsional Buckling

I have seen some engineers check collector beams attached to composite deck for torsional buckling. I have been told that this check comes from a paper, but I do not know which paper. This equation accounts for the effect of the top flange being braced by the deck while the bottom is flange free. I cannot find such a limit state in the *Specification*. Is this a valid check?

Yes. I believe the paper you are referring to is "Torsional and Constrained-Axis Flexural-Torsional Buckling Tables for Steel W-Shapes in Compression" in the 4th quarter 2013 issue of *Engineering Journal* (available at **www.aisc.org/ej**). The paper states: "The AISC *Specification* (2010) does not provide an equation for determining the CAFTB [constrained-axis flexural-torsional buckling] available compressive strength, nor does the AISC *Manual* (2011) include a design aid with CAFTB available strengths. Thus, when designing a member with a potential CAFTB mode, designers usually resort to a conservative approach such as evaluating the aforementioned beam for weak-axis FB (a mode that does not apply in this example) with $(KL)y$ equal to the member length in lieu of computing the CAFTB strength."

User notes in Sections E4 and 6.2 of the 2016 *Specification* also point to the Commentary to Section E4, which provides further information.

Carlo Lini, PE

Bolting Requirements

I am working on a project where neither the project specifications nor drawings specify the types of bolted connections (snug-tight, pretensioned or slip-critical). It is my understanding that the vast majority of bolted connections are snug-tight connections. Are there any provisions in the *Specification* stating that if this information is not provided, then the connections shall be snug-tight?

Yes. Section J3.1 of the *Specification* states: "The snug-tight condition is defined in the RCSC *Specification*. Bolts to be tightened to a condition other than snug tight shall be clearly identified on the design drawings." This indicates that, unless noted otherwise in the contract documents, bolts are to be installed snug-tight.

Jonathan Tavarez

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