Steel Interchange is an open forum for *Modern Steel Construction* readers 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.

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

If you have a question or problem that your fellow readers might help you to 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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BUILT-UP SHAPES

With regards to built-up shapes, what are the minimum and maximum web thickness and flange thickness? Is there a minimum or maximum relationship between t_f and t_w?

Question sent to AISC Steel Solutions Center

There is no minimum, maximum or relationship required between the flanges and webs. Rolled shapes do have relationships that must be maintained for production purposes (cooling rates create issues when very thick and very thin elements adjoin each other). But for built-up cross-sections, there is no such limitation. AWS D1.1 (Section 5.23) does have dimensional tolerances on built-up members for such items as flatness, straightness, and camber, but web and flange thickness is not specifically addressed.

Although there may not be any specified limits, some fabricators may have some in-house "rules-of-thumb" developed to help reduce distortion. For example, some bridge fabricators may not want to build up a girder with a web less than 5/16" to 3/8" thick. Consult your fabricator for limitations that they may follow in their shop.

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SHEAR CAPACITY OF BOLTS

A rigid support forces any beam-end rotation to take place within the connection. This leads to horizontal forces in the bolts, creating a couple. It seems to me that this imposed horizontal force in the bolts and plate will reduce the available capacity in a vertical direction when vectorially subtracted from the bolt's capacity (or plate's bolt-bearing strength, or weld capacity). Yet the *HSS Connections Manual* Table 4-8, and the *LRFD Manual* Table 9-10 (identical tables) show rigid connections having more shear capacity than flexible ones. Why is this?

Question posted at SEAINT list server (www.seaint.org)

The Astaneh approach is empirical in that it relates observed behavior to some prescriptive predictive equations for connection strength. The suitable performance is inherent in the connection due to the detailing requirements. So in a sense, your strength of materials and vector mechanics analogies, which are quite correct, are already internal to the Astaneh solution that is tabulated in the *Manual*. Rigid supports (those that constrain the end rotation to occur in the connection) have lesser eccentricities according to the Astaneh prescriptive predictive equations in general than do flexible supports (those that allow the end rotation to occur in the supporting element. Also, a rigid support has rotation (no or at least less moment) at the bolt line. Thus, the connection strength will generally be higher for rigid supports.

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WORKABLE GAGES

Please explain how workable gages are calculated in the 3rd edition *LRFD Manual*. The dimension tables in the new manual sometimes use the same workable gage for flange widths that differ considerably for a given beam depth, such as W14. Why is this?

Question sent to AISC Steel Solutions Center

Workable gages are derived from past recommendations from fabricators that effectively fulfilled two purposes. The first was to satisfy minimum edge distance and spacing requirements, as well as providing for entering/tightening clearances. The second purpose was that it made punching or drilling holes in the old days easier, because standardizing gages reduced the need to adjust the gage distance in the punching/drilling line, which increased fabrication efficiency.

The standard gages were dropped after the 7th edition *Manual* because people were unnecessarily insisting that only the 7th edition gages could be used, while at the same time new fabrication equipment was available that allowed fabricators to conveniently change gages on the beam line. They were reintroduced in the 3rd edition *LRFD Manual* as workable gages with clarification that any other gage that satisfies edge distance and entering clearances can also be used. Using these workable gages will often save the fabricator time and the owner money.

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GALVANIZING

Could anybody advise me on how to check the adequacy of galvanization of members of steel towers used for power transmission lines? What are the specifications that apply, and is the thickness of galvanization proportional to the thickness of the member?

Question sent to AISC Steel Solutions Center

Hot-dip galvanizing is performed in accordance with:

- ASTM A123 for members
- ASTM A153 for connection elements

You can read the scopes of these documents with the links here:

www.engr.psu.edu/ae/steelstuff/matls.htm

See Section 6 of ASTM A123 for coating thickness requirements. The minimums therein are a function of element thickness. You can get further information about galvanizing from the American Galvanizers Association (www.galvanizeit.org)

There are also a few other resources linked here:

www.engr.psu.edu/ae/steelstuff/rust.htm

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ANCHOR RODS

A supplier on one of our projects is claiming that when we specify ASTM A307 anchor rods, he can supply anchor rods with rolled threads. The motivation is that these rods are slightly cheaper. His argument is based on the claim that ANSI B1.1, which is referenced by A307, reportedly lists rolled threads as an option. This causes problems since the bolt shank is slightly smaller which results in a weaker bolt and results in a larger space between rod and hole. I believe that the supplier is wrong since A307 specifies a minimum area and a tensile capacity that "trumps" B1.1. I may be wrong if the minimum area is measured at the threaded region. Is the supplier blowing smoke or do we need to modify our specifications to exclude rolled threads?

Question posted at SEAINT list server (www.seaint.org)

Threads can be produced by cutting or rolling. The threading profile is identical for both—it has to be, since the product has to mate properly with a standard nut. Strength values are based upon the root area so the strength is also identical for both. The nominal area is used as a convenience in the calculation process. The design values include a reduction for threading to get from that nominal area down to the threaded area.

To cut threads, you have to start with the shank diameter equal to the outside thread diameter. To roll threads, you have to start with a shank diameter that is intermediate between the outside thread diameter and root diameter so that the material pressed *in* properly deforms *out* to match the threading requirements. If it is desired to have rod tension yielding (i.e., stretching of the unthreaded length) control the strength, the only way to do it is to upset the ends. Otherwise, essentially all deformation occurs in the threaded portion whether the threads are rolled or cut.

The rod is not weaker because the critical section is always through the threads. Unless you have a shear load at the column base, the slight increase in clearance between the rod and hole is not an issue. If you have shear, bearing on the anchor rods to transfer the load is not recommended anyway. Shear lugs should be used for significant shear. Smaller values can be handled in a shear-friction model without a shear lug, though.

You should consider using ASTM F1554 grade 36 whenever you would have used ASTM A307 before. ASTM F1554 is a single-source document for anchor rods that gets you everything you need in one place for threading (and heading, threading/nutting, bending—along with two additional strength grades as well—55 and 105). In essence, ASTM F1554 grade 36 was written to replace and simplify anchor rods where ASTM A307 material was being used—ASTM A307 is a bolt specification, primarily. Grade 105 similarly improves the situation where we used to use ASTM A193 grade B7. It's a really, really good specification. You can find more on it (and a few other new ones) here:

www.engr.psu.edu/ae/steelstuff/astmspcs.htm

The supplier is not blowing smoke. In fact, he's probably given you some pretty good insight into how anchor rods are made. Sometimes, you'll find you can learn a lot from these folks, just as they can learn a lot from you. I would not recommend excluding rolled threads for your projects. If you did, I think you'd be doing something that is unnecessary and costing your owner money in doing so.

AISC has a related FAQ on it's web site:

> *Are rolled and cut threads equally acceptable for anchor rods?* Yes. The use of either rolled or cut threads is permitted in ASTM F1554 Section 6.2. Rolled threads are formed by pressing threading dies into the shank to displace the surplus of the metal outward. The original rod diameter must be slightly less than the nominal diameter, although the root area will still be critical, unless the rod end is upset. The steel is cold-worked, compressing its grain and increasing the yield and tensile strength, generally from 10 to 30 percent. Cut threads are made with a thread-cutting die or by lathe cutting. The original rod diameter is approximately equal to the nominal diameter; again, the root area will be critical as is normal in design.

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