IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you!

Nominal or Tensile Stress Bolt Area?

Why does Table 7-2 in the 13th Edition *Manual* use the nominal bolt area and not the net tensile area in determining the available tensile strength of the bolts? If I were to use a 1" diameter ASTM A490 bolt, Table 7-2 indicates a nominal area of 0.785 in.^{2} and a tensile strength of 66.6 kips. But, if calculated based on the net tensile area of 0.606 in.^{2} given in Table 7-18, the strength would be 51.4 kips. What am I not seeing?

Question sent to AISC's Steel Solutions Center

For the design of bolted joints, the use of net tensile area in conjunction with F_{nt} (Table J3.2) of the *Specification* is not correct because the effect of threading is already included in the determination of F_{nt}. Accordingly, Table 7-2 uses the nominal bolt area and reflects the similar format used in the *Specification* where A_b is defined as the nominal unthreaded body area. The Specification Committee decided long ago to do this for simplicity and uniformity in bolted joint design for the various conditions like tension, shear with threads included in the shear plane, or shear with threads excluded from the shear plane. The threading reduction factor for tension is 0.75, while 0.8 is used for shear; these factors are already accounted for the the values given in Table J3.2 of the *Specification*.

Note that one must be more aware of the source of information when designing anchor rods. In the AISC specification, anchor rods are covered with bolts in Table J3.2, and the threading reduction values are included in those values. However, ACI 318 Appendix D is based upon tensile stress area.

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Limit States for HSS Flexural Members

The table User Note F1.1 lists yielding, flange local buckling, and web local buckling as the limit states for rectangular HSS in bending. Are rectangular HSS in flexure not affected by unbraced length? Is there no lateral-torsional buckling mode? I know that HSS are good for torsion, but as you get to some oblong sections (say an HSS 16×4), I would think that lateral-torsional buckling should begin to affect the strength.

Question sent to AISC's Steel Solutions Center

For rectangular HSS bent about the major axis, the limit state of lateral-torsional buckling (LTB) is not included in the *Specification*. It is essentially ignored for the reasons described in the commentary to Section F7. The high torsional resistance of closed sections such as HSS makes the critical lengths L_p and L_r very large for this type of shape. In the example given in the Commentary for an HSS 20×4×5/16, which has one of the most dramatic depth to width aspect ratios of all rectangular HSS, L_r is 137 ft. For a more practical maximum span dimension of 40 ft, the reduction is only 7 percent, and most HSS will not even have this much reduction because their aspect ratio is less severe. This reduction will be even smaller when the effects of moment gradient (C_b) are considered. Thus, to simplify the requirements, LTB is ignored.

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Hooked or Headed Anchor Rod Embedment?

I understand that changes have been made in AISC's *Steel Construction Manual*, whereby hooked anchor rods are to be replaced by headed anchor rods in tension applications. When does this become effective in the various building codes? What are the dimensional specifications for the head configuration? Has AISC published guidelines for material specifications, etc.?

Question sent to AISC's Steel Solutions Center

Neither the *Specification* nor the *Manual* stipulates what type of anchor rod is to be used on a particular project. I am not aware of a building code that makes such a stipulation either. The *Manual* simply recommends that hooked anchor rods should not be used for applications involving tensile loads. See page 14-10 in the 13th Edition *Steel Construction Manual* for the actual discussion.

Instead, the *Manual* recommends that headed or threaded and nutted anchor rods be used for embedments subjected to tension. The usual material specifications for anchor rods remain applicable. ASTM F1554 provides all you need, covering three strength grades, as well as the requirements for the headed, threaded and nutted, and hooked configurations. No special head size or nut size requirements are required, as the common head or nut dimensions are sufficient. If you have not yet seen ASTM F1554, you should give it a look. It is the only ASTM specification that covers all aspects and requirements applicable to anchor rods. It is included in *Selected ASTM Standards for Structural Steel Fabrication*, available from AISC at **www.aisc.org/bookstore**.

ACI 318 Appendix D provides methods of determining available pullout capacity for both types. This subject is discussed in Section 2.5 of AISC's *Design Guide 1, Base Plate and Anchor Rod Design*, Second Edition. Table 2.2 of the design guide lists anchor rod materials; see also Table 2-5 in the 13th Edition *Manual*.

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Use of "Actual" Tested Yield Strength for Design

Is it correct and permissible to use the "actual" tested yield strength of a square HSS (e.g., F_y = 49.3 ksi) in lieu of the "design" yield strength of ASTM A500 grade B (46 ksi) for the calculation of the flexural strength?

Question sent to AISC's Steel Solutions Center

There is no permission given in the AISC specification to use actual tested yield strength for design. F_y as defined in the *Specification* is the *specified minimum* yield stress of the type of steel being used. This value is used to account for the variability of actual yield strength of the material that can occur within a project, rolling, and even an individual piece.

One could use the provisions of Appendix 5 in the 2005 AISC specification, which cover evaluation of existing structures, to determine tensile properties. But note that this is not as simple as cutting a specimen and testing it. Section 5.2.2 specifies that the tensile testing must be done in accordance with the requirements in the ASTM specification that is applicable for the mechanical testing of the type of product being evaluated. In essence, you will then be testing the product to determine to which ASTM material specification it can be certified. Thereafter, the specified minimum yield stress is determined as permitted by the ASTM specification that is determined to apply.

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Shop Camber—Field Measurement Conflict?

We seem to have a recurring issue with camber in composite floor systems. We mechanically camber the member in the shop, and our QC staff visually verify with a string line and tape that we have induced the specified camber per the shop/contract drawings. However in a number of cases, during surveys of the in-place members, as much as half of the camber we originally induced has relaxed out of the member.

We are familiar with Section 6.4.4 and the Commentary in the AISC *Code of Standard Practice*, and frequently provide our clients with a copy of it. We frequently ask the client to survey the floor prior to placing that concrete to address any related issues so that we can shore beams, if necessary. As indicated above, that's when we find that the camber is no longer the same as it was when it left the shop.

We're looking for any type of information that would help support the commentary in section 6.4.4 that it can only be measured in the shop and how to overcome the end user's perception of expected floor performance.

Question sent to AISC's Steel Solutions Center

The same reasons that you cite for not achieving the camber in the field as in the shop, is the reason that the COSP stipulates that camber is to be measured in the shop in the *unstressed* condition. This becomes the only consistent method of evaluating the fabricator's work in providing the specified camber due to the varying factors of shipping, storage, and relaxation of the steel, as well as self-weight and any load induced deflection of the member.

You mentioned that the field measurement is taken before the concrete is placed. But, have the deflections due to self-weight, metal decking, and any construction loading been accounted for? These would likely have diminished the camber in the erected position from that measured in the shop.

After considering these probable causes of deviation, if you are still experiencing consistent difficulties, remember there is a plus tolerance in the camber measurement, but no minus tolerance allowed. You may consider a multi-pronged approach:

1. Discuss camber with the designer to determine the proper approach to calculation of required camber.
2. Discuss how much the floor slab thickness can be increased above the minimum dimension to provide a cushion for the expected variations in erected position.
3. Over-camber within the allowable tolerance in attempt to offset the losses you are experiencing.

Note that the above ideas must work together, but probably would not work if all were not simultaneously implemented. Also, remember that camber is not an exact science, and that the specified camber is only one of the factors involved in the final expected floor levelness equation.

There was an article on the subject in the June 2005 *Modern Steel Construction* titled "Tolerating Tolerances." Past issues of MSC can be accessed online at **www.modernsteel.com**.

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Floor Plate Design

Do you know of any resources or design aids for the design of solid plate floor decking such as "diamond plate"? We tend to do quick designs using ASD and $0.75F_y$ for bending; then check deflections—but this seems too conservative.

Question sent to AISC's Steel Solutions Center

Strength and deflection of floor plates is addressed in the 13th edition *Manual*. The *Manual* tables are based on an allowable flexural stress of 16 ksi for ASD and 24 ksi for LRFD. These values are used because the yield strength of floor plate material is often undefined. The deflection-controlled tables are based upon a maximum deflection of $L/100$.

From a strength perspective, the 2005 AISC specification requires $M_n = F_yZ \leq 1.6F_yS$ for a plate in weak-axis bending. Since $Z/S = 1.5$ for rectangular elements and the factor of safety for ASD is 1.67:

$$M_a = F_y(1.5S)/1.67 = 0.90F_yS$$

Thus, the 2005 AISC specification permits the use of $F_b = 0.90F_y$, which is larger than the value you have been using. However, depending upon the deflection criteria you choose, you may find that most floor plate designs are controlled by deflection.

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Steel Interchange is a 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.

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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