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Connection Strengths in the 13th Edition Manual

I have questions about the evolution of the connection strengths shown in the 9th edition manual compared to the values in the 13th edition. Why are they different?

There has been substantial research and testing on connections since the introduction of the 1989 ASD specification. The tables included in the 13th edition *Steel Construction Manual* are based on the 2005 AISC specification and reflect the culmination of that research. Yes, connection capacities per the 2005 AISC *Specification for Structural Steel Buildings* and the 13th edition *Steel Construction Manual* are higher in many cases than for the 1989 ASD specification and 9th edition manual. It depends, though, on which limit state controls, as some went up and others went down. Also, it depends upon what changes were made in design procedures given in the *Manual*.

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

PJP Weld Throat

How is the throat calculated for a partial-joint-penetration groove weld with fillet weld reinforcement? Is it the addition of the two weld throats?

You have to find the shortest distance from the PJP root to the outer edge of the weld assembly and use that as the throat thickness. This may not be the vectorial sum of the respective throats. This obviously depends on whether the fillet weld is larger or smaller than the PJP weld, and which PJP weld profile is used. Please refer to AWS D1.1 section 2.3.2.7.

Amanuel Gebremeskel, P.E.

Washers for Anchor Rod Installations

I have a project where base plate anchors were set too short to install both a washer and nut. The holes in the base plate are only 1⁄16 in. oversized for 1-in.-diameter anchors, so the contractor is requesting to eliminate the washers. How can I assess the uplift capacity of such an installation?

AISC specification requirements for steel-to-steel bolted joints are not applicable to anchor rods or the nuts on anchor rods. In the assessment of "short" anchor rods, consider what the purpose of the anchor is in the structure. In many cases, a column is always in compression in the final structure; the rods simply hold the column in place and accommodate temporary erection forces.

The hole sizes recommended for base plates in Table 14-2 of the 13th edition AISC manual are intended to facilitate the setting tolerances of other trades. The washer sizes given are such to cover the hole should the anchor be positioned to one side of the hole, and to provide a stiff surface upon which to tighten the nut and to keep the washer from dishing in the hole. When the hole diameter in the base plate is 1⁄16 in. more than the rod diameter, it is unlikely that a washer would be necessary for either of these purposes.

The other alternative is to consider the strength of the connection with the partially engaged nut. If the tensile load on the anchor rod is not required to develop the full strength of the rod, a partial nut engagement may be acceptable. The authors of an article titled "An Ounce of Prevention," which appeared in the May 2004 issue of *Modern Steel Construction*, provide suggestions on how to approach such an evaluation. Back issues of MSC can be accessed at www.modernsteel.com.

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

KL/r Modified for Single-Angle

For a single-angle compression member, I followed AISC specification section E5 to calculate the modified *KL/r*. I also calculated *KL/r*$_z$, and it turns out to be greater than *KL/r* modified. Should I use the larger of the two (*KL/r* modified, or *KL/r*$_z$) in section E3?

If you are in compliance with E5 (including attaching the angle using the longer leg, then you can use the limits on L/r_z that are provided at the ends of the both sections (a) and (b). In the first case the limit is $0.95L/r_z$ and in the second case it is $0.82L/r_z$. In essence, with your condition, you are still designing for KL/r_z but with a K value of less than 1.0 because of the higher end restraints.

Amanuel Gebremeskel, P.E.

Shoring Removal

In composite beam design and construction, if the design basis is shored construction, how long do the shores need to remain in place?

We cannot give you a definitive timeframe; however, the Commentary to Section I3.1c of the AISC specification provides guidance for assumed strength during construction. Therein it is stated that "It is usually assumed for design purposes that concrete has hardened when it attains 75% of its design strength."

It is probably worth adding that shored construction is not commonly used in steel structures.

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

Double Angle Connection - Table 10-1

The beam web strength portion of Table 10-1 in the 13th edition manual checks block shear rupture, shear yielding and shear rupture depending on the coping condition of the beam in question. When determining the connection capacity for a bolted-welded beam-to-girder connection, both Table 10-2 and the bolt and angle portion of Table 10-1 must be checked to find the limiting value. If the beam is coped at one or both flanges, there are no additional checks against block shear rupture, shear yielding, and shear rupture. Does AISC recommend checking these limit-states, or is it unnecessary for a connection with a welded beam web? If they must be checked, is there another table to expedite this check?

Table 10-1 also includes a check of bolt bearing on the beam web in the beam web strength portion.

Page 10-11 of the 13th edition AISC manual includes a discussion of why those checks are not carried out in the table.

Essentially, the thinnest web thickness is being used for the given weld size. If you do end up using web thicknesses that are less than those derived from the equations on page 10-11, then you do have to reduce the available strength given in the table by the ratio of the actual thickness to tmin.

Please note that in both tables 10-1 and 10-2 with coped beams you still have to check for flexural yielding and local buckling of the beam web as per Part 9.

Amanuel Gebremeskel, P.E.

Recycled Content

We are working on a project that is pursuing LEED certification. We are pursuing material resource credits 4.1 and 4.2 and are using structural steel as a major part of achieving this credit. We would like to bump them up as much as possible without creating a non-competitive bid environment. Where can I find information to determine the appropriate credits?

The website at **www.aisc.org/sustainability** provides documentation that the United States Green Building Council has agreed to accept in documenting the recycled content of various types of structural steel. This is contained in the article titled Steel Takes LEED with Recycled Content (LEED-NC v2.2), which can be found on the aforementioned website.

John Cross, P.E.

Butt Splicing W-shape Beams

I have a contractor who would like to butt splice two W14x22 beams together. He is proposing to use a complete joint penetration groove weld that will be done in the shop. The splice will occur 4 ft from the end on a 20-ft span. Will this weld develop the full strength of the section?

It is very difficult to provide a butt welded splice between two W-shapes that will develop the full cross section. This is because weld access holes need to be made in the web to facilitate the welding operation for the flanges. Often, only the flange area of the beam will be assumed to provide the flexural resistance in such a case. Thus if a splice is needed, it would be wise to locate it away from the point that the full section is required. If you have a simply supported, uniformly loaded beam, the described splice location at $0.2L$ would seem to accomplish this.

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

Fully Restrained or Partially Restrained

What is the ratio of end moment to beam rotation that must be achieved to consider a connection fully restrained rather than partially restrained (slope of FR line in Figure 12-1 of the 13th edition *Steel Construction Manual*)?

As per Fig. C-B3.3 (page 220) in the commentary to section B3.6 of the 2005 AISC specification, the FR slope is above $20EI/L$ at service loads. For simple connections this stiffness is below $2EI/L$. A good discussion of what load amount to use to come up with stiffness values is included on page 219. Analyzed at service levels all stiffness values between $2EI/L$ and $20EI/L$ are considered PR connections and their stiffness, strength and ductility must be considered in design.

Amanuel Gebremeskel, P.E.

Suspended Sprinkler Loads

The fire marshall is objecting to hanging fire sprinkler pipe from the bottom chords of steel bar joists. I can find no reference to disallow this in our building code. Numerous contractors are contacting our building department, wanting to know the basis of this objection. I would assume that the design professional of record has reviewed what is being hung from the joists and has performed a structural calculation of the imposed loads.

Open web joists and joist-girders are not covered by the AISC specification; rather they and covered by publications of the Steel Joist Institute (**www.steeljoist.org**). Steel joists are a manufacturer-designed component, based on load criteria supplied by the engineer of record (EOR). You may want to check with the EOR for the project to see what loads had been specified and to see what was included for sprinklers. The SJI specifications contain restrictions relative to hanger locations being at panel points unless otherwise defined, and the magnitude of hanging loads must be considered in the design. You may want to contact SJI for further information on the subject.

Kurt Gustafson, S.E., 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.

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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