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Doubler or Stiffener Plate?

My question pertains to the design of column web reinforcement for directly welded flange moment connections.

When local yielding of the column web occurs, is it acceptable to use web doubler plates in lieu of a pair of transverse stiffeners to provide for the additional material necessary to exceed the design strength requirements? The reason I ask is because all of the AISC design examples, and also the software that we use for connection design, always provide for transverse stiffeners instead of doubler plates when local yielding of the column web is an issue.

If web local yielding is the limit state being checked, the use of a doubler plate is an option. This is covered in Section J10.2 of the 2005 AISC *Specification*.

Design examples often use the transverse stiffener option because it is generally considered easier to fit and fillet weld the stiffener than a doubler plate where the weld to the fillet region becomes somewhat more tedious—and expensive. But if you are providing a doubler plate already for another reason, like web shear strength, it may not be an additional cost.

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

Thermally Cut Holes

On one of my projects, the fabricator is asking to use thermally cut bolt holes. He is citing Section M2.5 of AISC 13th edition *Steel Construction Manual*. That section states that thermally cut holes shall be permitted with a surface roughness not exceeding 1,000 μin.

What are the advantages and disadvantages of thermally cut holes?

How does roughness of surface come into the picture for cutting the holes?

The primary advantage of thermal cutting for the making of holes is that the shaping of the plate and the burning of the holes can often be done on a single piece of equipment. This saves time in handling. Also, thermally cut holes can be cut to different diameters, if necessary, without changing bits. The advantages are primarily economic, though it also could be argued that the thermally cut holes generally will be cleaner as produced. The presence or absence of small burrs is not a significant issue, but the absence of burrs in thermally cut holes often is touted as an advantage in the literature.

Provided that the requirements for surface condition are met, there is no disadvantage to a properly made thermally cut hole. The profile of the hole is also important, and I would not allow thermal cutting of holes by hand, such as for repairs, unless approved by the EOR. Hand-guided thermal cutting usually would produce holes of questionable quality.

Larry S. Muir, P.E.

ASD Flexural Capacity in the 2005 *Specification*

When designing a channel for flexure, I am somewhat confused regarding the allowable/available moments that are published in the 13th edition *Manual*.

I came to the conclusion that all of the channels now have an allowable bending stress of 0.75F_y as opposed to the older 0.66F_y. Am I correct in assuming that this is what the new allowable stress is for channel beams? Can I also get an explanation as to why the sudden increase in allowable stress has been made to be 0.75F_y?

That's not quite correct. The comparison can't be made only on the basis of F_y, because there also is a difference between them on the material property used in the calculation.

In the 2005 AISC *Specification*, the nominal moment capacity based on the limit state of yielding for compact shapes is $M_n = F_y Z_x$. The old ASD *Specification* assumed a lower bound shape factor of 1.1 for rolled shapes and $M_n = F_y (1.1S_x)$, which resulted in allowing a 10% increase in flexural capacity for compact shapes when LTB did not control. Once the factor of safety of ⁵⁄₃ was applied, this resulted in the allowable stress of 0.66F_y in lieu of 0.60F_y.

To make the comparison you're making, then, you also need to account for the ratio of Z_x to S_x. When converting the moment to a stress, the base stress is still 0.60F_y, but the shape-property ratio likely will be greater than 1.1 because it is dependent on the particular shape factor (Z_x/S_x) for the shape.

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

Steel Types

When were A36 and A6 steels in general use for building construction?

ASTM A36 was the common structural steel used in building construction from the early 1960s to the late 1990s. The ASTM A6 Standard is not a specific material type per se, but rather a Standard Specification for General Requirements for Rolling Structural Steel Bars, Plates, Shapes, and Sheet Piling, which defines the cross sections and tolerances for hot-rolled shapes.

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

Knee-Braced Frame

Is a frame with a kick brace considered a special braced frame?

I am assuming that when you refer to a "kick brace" that this is the same as a "knee brace," which is covered in the *Seismic Provisions*. It is treated as an Ordinary Moment Frame, not a braced frame, since the primary response of such a frame is through flexure of the beams and columns between the knee braces and not axial effects in the braces. Please see the Commentary to Section 11 for more detailed information.

Larry S. Muir, P.E.

Dynamic Analysis

What type of structure requires a dynamic analysis?

The subject of what type of analysis is appropriate for a particular structure is not covered by material standards such as the AISC *Specification*, but rather in ASCE 7. It could be argued that all structures behave dynamically to some extent when subjected to loads or displacements. Typically, however, the loads and displacements we address in building design are applied slowly enough that a static analysis is justified. Blast loading is one exception to this. The decision as to what type of analysis is appropriate for a specific application is left to the responsible design professional.

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

Skewed Single-Plate Connection

Per the discussion on page 10-151 of the 13th edition AISC *Steel Construction Manual*, the maximum beam-web thickness is a function of the maximum root opening and the angle of skew in a skewed single-plate connection. Why?

If the beam web becomes too thick relative to the skew angle, the root opening will begin to exceed 3⁄16 in. This limit is the maximum root opening allowed by AWS for a fillet weld. If the opening is 3⁄16 in. or less, the fillet weld size must be increased by the root opening dimension. If the root opening provided by the unbeveled web or plate exceeds 3⁄16 in., the plate or web must be beveled.

Larry S. Muir, P.E.

Prequalified 4ES Connection

Table 6.1 of AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, lists 10¾ in. as the only width allowed for the end plate of a 4ES connection. Is this correct that the identical max. and min. values are the same?

In the first release of the standard, the width of 10¾ in. was included intentionally, as all of the tests were conducted with that plate width. Subsequent testing since the release of that standard has shown that the limits can be expanded to cover plate widths between 7 in. and 10¾ in., inclusive. This is proposed for inclusion in the Supplement to the standard, currently available at **www.aisc.org/358s2**. Note that this document is still under development, and will not be officially released until next year.

Chris Hewitt, S.E.

Prequalified Connection Standards

I am attempting to design my first SMF using a prequalified connection from AISC 358-05. The only prequalified connections listed are the RBS and the unstiffened and stiffened end plates. Does this document supersede FEMA 350, which lists such connections as the WUF-W as valid for SMF framing systems? Do the FEMA 350 connections not listed in AISC 358-05 lack the proper testing for the AISC 358-05 prequalification?

I have also heard that some proprietary connections have been submitted for prequalification. Is there a location where I can find a list of the connections that have been approved since the publication of AISC 358-05?

AISC continues to develop the AISC 358 standard. Because this is a relatively new standard, there are other connection types in FEMA 350 and other testing reports that have not yet been reviewed for possible inclusion in the standard. The RBS and end-plate connections were included in the first version (2005) of the standard because these had the broadest range of testing and therefore were the easiest to prequalify.

For the other connections in FEMA 350, many code authorities will allow you to use the criteria in FEMA 350 while the connections are being considered by the AISC CPRP for inclusion in the standard. In essence, you are then using qualified connections as permitted in AISC 341, and this is not uncommon. A supplement to the standard, which will address several other connection types—including WUF-W connections—is available from the AISC website at **www.aisc.org/358s2**. As stated in the previous answer, this document is still under development, and will not be officially released until next year.

Chris Hewitt, S.E.

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