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Extended End-Plate Stiffener

The stiffeners in the extended end-plate moment connection as shown in Design Guide 4 are detailed at a 30° angle. How is this angle selected? In many gusset connections the angles are set at 30°. What is the significance of the angle?

There is a discussion of the 30° stiffener distribution angle on page 16 of Design Guide 4 as follows:

"Analytical and experimental studies have shown that a concentrated stress applied to an unsupported edge of a gusset plate is distributed out from that point towards the supported edge at an angle of approximately 30°. This force distribution model is commonly referred to as the 'Whitmore Section.'"

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

Anchor Rod Tensile Strength

When calculating the strength of a threaded anchor rod per Section J of the AISC *Specification*, is the strength based on failure in the threads or in the gross area?

An anchor rod subjected to tension will always fail in the threaded section, unless an upset rod is used. The tensile strength is based on tensile rupture through the threaded area. A factor of 0.75 is incorporated into the design values provided in the AISC *Specification* as a simple means of accounting for the reduction in tensile stress area due to threading. As a result, the designer can use the nominal bolt diameter in the calculations.

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

Tension Calibrator

Is a torque wrench considered a tension calibrator?

No. A torque wrench is an installation tool that measures the torque applied when turning a nut on a bolt. A tension calibrator is a device that measures the tension induced in a bolt as the nut is tightened—it is used to determine what torque should be used when the torque wrench is used. The RCSC *Specification* (a free download at www.boltcouncil.org) provides further information on both of these. See Sections 7 and 8.2.2 in particular.

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

Flexure of an Unequal Leg Angle

For bending of an unequal-leg angle about the major principal axis, M_n is found with Equation F10-2 or F10-3 using the M_e value in Equation F10-6. What should be used to find the minor principal axis bending of an unequal-leg angle?

When evaluating angle bending on the principal axes, the minor axis is the axis of least strength. Therefore, lateral-torsional buckling does not occur for bending about the minor principal axis of the angle. If local buckling is not an issue, yielding controls for bending about the minor principal axis.

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

Torsional and Flexural-Torsional Buckling

Could you help me understand when torsional or flexural-torsional buckling needs to be checked for a column?

It is possible for both symmetric and non-symmetric shapes to buckle in a torsional mode, and in theory, all members must be checked for this failure by evaluating the torsional unbraced lengths. However in common construction practice, most columns are connected to the floor framing in such a way that their lateral braces also happen to restrain the member in torsion. In those cases it is highly unlikely that the limit state in Section E4 will control over those in E3.

A case that might have a torsional mode that controls is a column with a mid-height lateral brace from a bare beam framing to its weak axis. In this case, the torsional unbraced length might be twice the weak-axis unbraced length.

Amanuel Gebremeskel, P.E.

Minimum Lateral Load

I'm working on the design of a large maintenance platform, and the construction is relatively light. The project is in one of the lowest seismic risk areas and the platform will be inside a building, so the lateral loads prescribed by ASCE 7-05 are quite low—no wind and the seismic loads are simply 1% of the seismic weight, *W*, of the platform. Isn't there more that might need to be considered here?

One additional consideration is the stability of the platform. The Direct Analysis Method of Appendix 7 in the AISC *Specification* prescribes a notional load to account for the destabilizing moment caused by the vertical loads acting through the initial out-of-plumbness. However, that load is also quite small: $0.002Y_i$, where Y_i is the vertical load on the entire story.

It's hard to know what other considerations might apply. Is there any usage of equipment on the platform? Forklifts, for example, will apply horizontal forces to the platform due to acceleration or deceleration, and those forces can be determined from manufacturer's literature. Can a vehicle hit the platform from the side? That would be another source of load to consider.

Other than special considerations like these that may or may not apply, the basic structural integrity requirements (like $0.01W$) and stability requirements (like $0.002Y_i$) are sufficient.

Brad Davis, Ph.D., S.E.

Field Cutting of Steel

Is flame cutting or air-carbon arc cutting the preferred process for cutting steel in the field?

Flame cutting [along with plasma cutting for thinner material] is by far the preferred method for cutting steel. Air-carbon arc cutting is more typically used to remove material such as weld filler metal and steel; it is a gouging process rather than a cutting process. Such practice is common in welding where the material may have to be back gouged in order to repair the finish.

Amanuel Gebremeskel, P.E.

C-Coefficients

Why are the *Coefficients C for Eccentrically Loaded Weld Groups* listed in Table 8-4 of the 13th edition *Manual* for some weld groups different than those listed in the tables of the 9th edition *Manual* for the same weld groups?

As reflected in Section J2.4(b) of the 2005 AISC *Specification*, welds loaded in a direction other than longitudinal to their axis have increased strength. The higher value in the 13th edition *Manual* tables takes this directional strength increase into account. The 9th edition *Manual* did not, because the strength increase was not allowed in the 1989 AISC *Specification*.

Larry S. Muir, P.E.

Temperature at Time of Erection

Are the erection tolerances in the *Code of Standard Practice*, or COSP, to be considered based on a specific ambient temperature, say 70 °F? For example, if a column stack is within plumb tolerance at 70 °F and out of tolerance at 30 °F (due to contraction of the building), is this OK?

I do not believe the COSP gives specific guidance, but buildings should be erected such that the center of rigidity is plumb when environmental conditions are not deflecting the structure. There are many old stories of high rises being checked for plumb at midnight to avoid the effect of sun on one side of the building.

The places where I have had to adjust for temperature were at expansion joints in both bridges and buildings. This was quite difficult when erecting buildings, because the construction tolerances and movement of the building were as large, or larger, than the expansion joint planned movement.

Presuming the structure has been built reasonably within plumb tolerances, it is my thought that forces due to temperature deformations are a load to be considered in the analysis. I believe the AISC *Specification* would accommodate a 1:500 out of plumb, but I have seen engineers account for that also.

Tom Schlafly

Plug Welds for Doubler Plates

Section 9.3b in the 2005 AISC *Seismic Provisions* indicates that a doubler plate can be restrained by plug welds to prevent buckling. Section J2.3b of the 2005 AISC *Specification* defines the diameter of the holes for plug welds. However, no requirement for the depth of the weld is stipulated. Since such plug welds are only there to prevent buckling of the doubler plate, what is the minimum depth of weld required to achieve this?

The required depth of plug welds is a function of the thickness of the doubler plate as defined in Section 2.3.5.4 of AWS D1.1. Up to ⅝-in. plate thickness, the full thickness must be filled. Over ⅝-in. plate thickness, half the thickness or ⅝ in. must be filled, whichever is greater. In all cases, the filled thickness need not exceed the thickness of the thinner part joined.

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

Bearing Length – N_{req}

In Table 10-6 in the 13th edition AISC *Steel Construction Manual*, to what does the first column "Required Bearing Length N_{req}" refer? What is this measured from?

The required N values are the amount of bearing area required to develop the strengths listed in the table based on the limit states of local web yielding and crippling; as a minimum, N_{req} is taken not less than the value of k_{des} for the beam. This is discussed briefly on page 10-87 in the *Manual*, and in greater detail in a paper in the 4th Quarter 1997 issue of the AISC *Engineering Journal* (Discussion: "The Behavior and Load-Carrying Capacity of Unstiffened Seated Beam Connections").

The reason it is shown this way is because the beam does not necessarily bear over the full length of the seat as one might assume. Instead, the angle bends away from the bottom flange of the beam as the load is applied. In order to carry more load, the beam web must yield locally to sit back down on the angle as the angle bends away. At lower load, a smaller bearing area is required and the angle experiences a smaller eccentricity. The opposite is true for a more heavily loaded seat.

N_{req} is measured from the end of the beam in the direction toward the toe of the outstanding angle leg.

Larry S. Muir, P.E.

Reduction in Area for Bolt Holes

Table 9-1 in the 13th edition AISC *Steel Construction Manual* lists Reduction in Area for Holes. For a standard hole, why is the reduced area calculated assuming that the hole diameter is equal to d_{bolt} + ⅛ in.? Shouldn't this be d_{bolt} + 1⁄16 in.? Table J3.3 shows that the standard holes are typically 1⁄16 in. oversize compared to bolt diameter.

Per Section B3.13.a of the 2005 AISC *Specification*, "In computing the net area for tension and shear, the width of the bolt hole shall be taken as 1⁄16 in. (2mm) greater than the nominal dimension of the hole." Thus, there is the 1⁄16 in. hole clearance and the additional 1⁄16 in. required to be deducted above.

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

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