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steel interchange

Chevron Brace with Both Braces in Compression

I am designing the gusset for a chevron brace connection with both braces in compression. The Whitmore sections of the individual braces overlap. How should this condition be treated?

The condition with both braces in compression is addressed in the Example 5.9 of AISC Design Guide 29: *Vertical Bracing Connections—Analysis and Design* (a free download for members at **www.aisc.org/dg**), though the Whitmore sections in the example do not overlap. It should be noted that a few different approaches are proposed for checking the stability of the gusset. Each is only a model considered to be reasonable by the authors. In your case, you must determine a reasonable model based on your own engineering judgment.

I imagine there could be many approaches one could take. You could simply ignore the portion of the Whitmore section that overlaps. You could perform some type of stress interaction check. You could run a fine-element analysis. Personally, I would likely be okay with the overlap in many instances for a few reasons. First, when we check the Whitmore section, we assume an even stress distribution along the Whitmore section area which is established using the 30° angle. This was found to give a good prediction of the peak stresses measured from aluminum joint testing performed by Whitmore [Whitmore, R.E. (1952), "Experimental Investigation of Stresses in Gusset Plates," Bulletin No. 16, Civil Engineering, The University of Tennessee Engineering Experiment Station, Knoxville, TN.]. Stress trajectories were plotted from the test data, and they vary greatly along the Whitmore section. The stresses were lower near the ends of the Whitmore section where the overlap occurs in your situation, although connection configurations could impact the stress distributions. Also, with the braces both being in compression, I imagine the stress level will be quite a bit lower than the yield strength of the plate.

Carlo Lini, PE

NDT and Special Inspection Waivers

Please confirm that when third party special inspections are waived by the authority having jurisdiction over the project, the NDT requirements in Chapter N of the *Specification* are also waived.

This is not correct. Section N7 clarifies the intent, stating: "Quality assurance (QA) inspections, except nondestructive testing (NDT), may be waived when the work is performed in a fabricating shop or by an erector approved by the authority having jurisdiction (AHJ) to perform the work without QA." NDT must be performed even when the QA inspections are waived.

Larry S. Muir, PE

Channels Warped During Galvanizing

Several channels we are using on a current project have warped significantly during galvanizing. It has been suggested that the channels may have been a poor choice for these members. Is there are validity to this suggestion?

Yes. ASTM A384: *Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing of Steel Assemblies* recommends the use of symmetrical shapes and singles out channels as a member type that typically requires straightening after galvanizing. The April 2004 article "Galvanizing Tips" (available at **www.modernsteel.com**) reinforces this point and provides other useful tips related to galvanizing.

In addition, the American Galvanizers' Association suggests a collaborative effort should be used to achieve the best results: "The design of parts to be hot-dip galvanized is the responsibility of the design engineer and the architect; however, when there is a part that has an asymmetric design the galvanizer should let his customer know the part is very likely to distort during the galvanizing process."

Larry S. Muir, PE

Removal of Shim Stacks

For base plates that are shimmed and grouted, does AISC consider it necessary to remove the shim plates and pack grout in the voids left by the shims?

No, AISC standards do not require removal of the shim stacks. Leaving the shims in place under the base plate is common practice.

Ideally, column bases should be grouted as soon as possible in construction when the axial load to the column is only a small fraction of what the total anticipated final load could be. Done properly, the base plates should be grouted before any concrete is cast for the elevated floors when the only load delivered to shim stacks is the weight of the bare steel frame and some construction live loads. As additional load is added to the column, the grout will then distribute the load to the foundation.

Axial compressive forces from the column can be assumed to be evenly distributed as bearing forces on the shims and non-shrink grout. Even if the shims were to start out taking the majority of the load, the assembly will deform in a self-limiting manner through localized yielding of the steel as the force-distribution model assumed in sizing the base plate is attained.

Susan Burmeister, PE

A More Efficient Approach to Uplift

I have designed a 30-ft-long W14×22 roof beam to resist gravity loads. However, when we check the beam for wind uplift, bottom flange bracing is required at the mid-span. The W14×22 seems like a reasonable size for this application, and I have seen it called out on other similar projects without bottom flange bracing. Is there a method that might permit me to omit the bracing?

Based on the scenario you have described, bracing of the bottom flange might be needed. I have personally used bottom-flange bracing on numerous projects where the wind uplift pressures exceed the roof dead loads. In my experience, this is not an uncommon practice.

However, you may be able to calculate enough extra capacity for your beam if you take a closer look at the value of C_b used in your analysis. The Commentary to Section F1 of the AISC *Specification* provides some additional formulas that can be used to calculate C_b for a roof beam subject to uplift loads, as shown in Figure C-F1.5. This may increase the available strength enough to eliminate the need for bottom flange bracing. It is certainly worth investigating, especially for repetitive beam conditions.

Susan Burmeister, PE

Cambering Plate Girders and Heavy Beams

Can very large and very long beams, such as a 56-ft-long W40×593, be cambered? Likewise, can a 56-ft-long, 50-in.-deep plate girder be cambered?

Many fabrication shops have the capability to camber typical floor beams using a cold-bending operation (cold cambering). If the machine capacity is exceeded, heat can be applied to the member to reduce the yield stress. Because many bender-roller companies have specialized, high-capacity equipment, it is often more economical for the fabricator to sublet the cambering of large beams. However, it is doubtful that a 56-ft-long W40×593 could be cambered by cold-bending or heat-assisted bending.

Another potential option is heat curving, which is a bending process that relies only on the application of heat in specific patterns to induce curvature. This method is used primarily used by fabricators for cambering and curving to very large radii and for repairing damaged members. You should contact a fabricator to get their advice on this method.

Generally, plate girders cannot be efficiently cold-bent about the strong axis due to the high depth-to-thickness ratio of the web. In most cases, cold bending would cause local web buckling during the bending operation. The welding of the section also would be a challenge, since curving means plastic deformation and shear in the welds probably much greater than the design anticipated for loads in service. It's likely too that plate girders would usually exceed the capacity of the available cambering machine. Fortunately, there is another way. Plate girders are often cambered by cutting the web to the desired curvature, and then welding the flanges in place. This may be the best option.

Bo Dowswell, PE, PhD

Demand-Critical Welds on Seismic Projects

Must all welds on a seismic project meet AWS D1.8, making them all demand-critical welds?

No. Your question indicates quite a bit of confusion about the requirements and the terms used in AISC 341: *Seismic Provisions for Structural Steel Buildings*. I will try to clarify the requirements for you.

First, seismic effects must be considered for all projects. By seismic project, I assume you mean a project that must meet the *Seismic Provisions*.

The *Seismic Provisions* make several references to AWS D1.8. Each of these applies only to welds within the seismic force resisting system (SFRS). For example, Section A3.4a states: "All welds used in members and connections in the SFRS shall be made with filler metals meeting the requirements specified in clause 6.3 of *Structural Welding Code—Seismic Supplement* (AWS D1.8/D1.8M)." Welds outside the SFRS need not satisfy AWS D1.8.

Additionally, welds required to satisfy AWS D1.8 are not necessarily demand-critical welds. Demand-critical welds are a subset of the welds addressed in AWS D1.8. This is can be seen in the User Note that accompanies Section A3.4a, which states: "AWS D1.8/D1.8M subclauses 6.3.5, 6.3.6, 6.3.7 and 6.3.8 apply only to demand-critical welds."

It should be noted that per Section A4, the engineer is responsible for identifying the welds subject to requirements beyond those in AWS D1.1 through "Designation of the SFRS," "Identification of the members and connections that are part of the SFRS" and providing the "Locations of demand critical welds."

Larry S. Muir, PE

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