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steel interchange

Eccentricity at Axially Loaded Beam-End Connections

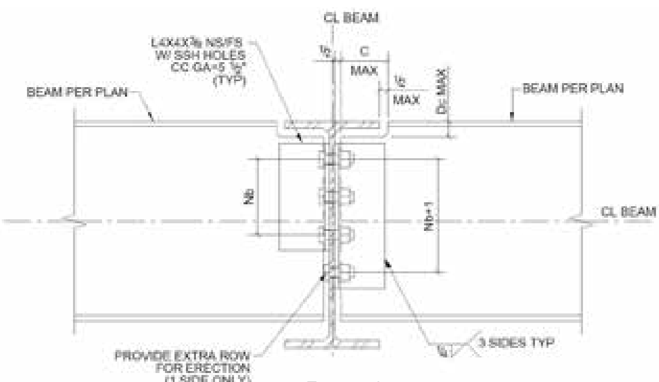

Figure 1

Shop-welded, field-bolted beam-to-beam double-angle connections must transfer both axial and shear end reactions. In Figure 1 the beams are non-composite and the entire axial force must be transferred through the connections. The connections will be designed assuming that only the top three rows of bolts, which are common to both connections, will transfer the axial force.

Does the eccentricity of the axial load from the beam centerline to the centerline of the bolt group need to be considered in the design of the connection? Does an eccentricity need to be considered when evaluating the coped section?

Axially loaded double-angle connections are typically designed without considering an eccentric moment. The rotational stiffness of the connections is typically much lower than the stiffness of the beam. Therefore, the beam will carry almost all of the moment due to the eccentricity, and it is common to assume that the beam resists the entire moment. It is also a good idea to use the maximum number of bolt rows that will fit into the web when resisting axial end reactions, as this will tend to minimize whatever eccentricity does exist.

Because floor systems are usually modeled with the beam elements at the same elevation, the eccentricity between the axial load and the beam centroid is often neglected.

I would analyze the coped section assuming an eccentricity relative to the axial load equal to the distance between the centroid of the bolt group and the centroid of the coped section. For axial loads in tension, the resulting moment opposes the moment caused by a downward vertical beam shear. I would also locate the bolt-group centroid as close to the beam centroid as practical, typically using the maximum number of bolt rows that will fit into the web. In practice, the small eccentricity that might exist is sometimes neglected based on engineering judgment.

As you stated, the bottom row of bolts on the right side of the connection should be neglected relative to the transfer of the axial load. However, all of the bolts will participate in transferring shear.

Other approaches are possible and contract-specific requirements could be imposed, but the comments above reflect what I understand to be common practice.

Bo Dowswell, PE, PhD

Cambering of Cantilevered Beam Framing Continuously Over Column

I have a beam framing continuously over a column similar to the condition shown in Figure 2-2a of the 14th Edition AISC *Manual* (available at www.aisc.org/publications); see Figure 2. In my case, the right-hand side of the beam cantilevers 17 ft beyond the column, and the left-hand side is a 10-ft back span. The beam is a W18 and the column an HSS4×4. I wish to put a camber in the cantilevered section such that its end will be ¾ in. higher than the elevation at the supported when erected.

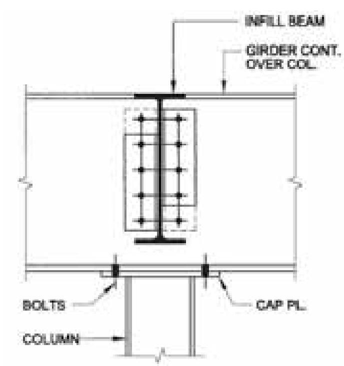

Figure 2

I am aware that there are issues with providing camber for cantilevered or moment-connected beams. However, is it feasible to camber a cantilevered beam framing continuously over a column?

No. It is generally not feasible to camber a cantilevered beam framing continuously over a column.

When designers call for a simply supported steel beam to be cambered, the steel fabricator applies a load or heat to the beam to introduce a permanent deformation in a roughly parabolic shape with the apex at mid-span. But based on the geometry you've described, this "conventional" method of introducing camber into a member does not seem like it would be appropriate for your condition. Additionally, since your member is only 27 ft long, it may not be a candidate for cambering depending on the fabricator's cambering method or equipment. Typically, it is not recommended to camber members less than about 30 ft long because most cambering equipment is not configured to accommodate shorter members.

In general, when engineers do specify camber for a cantilevered beam, it is provided in a manner that is different than the way we typically think of camber being introduced into a simply supported steel beam. The beam itself usually remains a straight element, and the beam is fabricated so the erector can simply install the beam so that the tip of the beam is higher than the beam elevation at the connection by the specified amount. Since the beam isn't being bent, you do not have the same physical constraints on how long a member needs to be before it can be "cambered."

An alternative solution might be to bevel cut and splice the member after it passes over the column, which will create a change in slope at the cantilevered portion to provide a specified top elevation at the cantilever tip. However, this is not an inexpensive approach and it should be weighed against increasing member sizes or using other methods to mitigate the effect of the anticipated deflection at the tip of the cantilever. An alternative that some fabricators prefer is to make a V-shaped cut in the member, leaving one flange intact and then bending and welding the member into the kinked geometry. If you choose to kink the beam, I would suggest you consider annotating your drawings and labeling the elevation difference as something other than "camber."

You might also consider splitting the beam and running the column through the joint. This is typically the better option for wide-flange columns, but is also a possibility for your HSS column, and in fact may be the most economical solution. If you choose this option, note that the 2016 AISC *Code of Standard Practice* (ANSI/AISC 303-16, available at **www.aisc.org/specifications**) contains new treatment of preset requirements at the ends of cantilevers that will help you with your goal; see Section 3.1.

Susan Burmeister, PE

Slip-Critical "Bolts"

What are the differences between slip-critical Class A bolts and slip-critical Class B bolts, and how should they be indicated in shop and erection drawings?

There is no such thing as a slip-critical bolt, a Class A bolt, a Class B bolt or a bearing bolt. The same bolt can be used in slip-critical joints with either Class A or Class B faying surfaces. In fact, the same bolt can be used with either slip-critical or bearing-type joints. The difference between a slip-critical *joint* and a bearing-type *joint* is that a slip-critical joint resists movement of the plies through friction, and a bearing-type joint resists movement between the plies through bolt shear and bearing at the plies. The Class A and B designations refer to the surface preparation required.

Section J3.8 of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-16, available at **www.aisc.org/specifications**) defines Class A surfaces as "unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dip galvanized and roughened surfaces" and Class B surfaces as "unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel."

The detailer must properly indicate on the shop drawings the required surface preparation at the slip-critical joints, as this affects the strength of these joints.

Slip-critical joints also need to be pretensioned, and this must be conveyed in some manner in the documents related to the bolt installation, either the shop or erection drawings. Indicating that the joints are slip-critical is sufficient to ensure pretensioning.

Carlo Lini, PE

Average Versus Peak Shear Stress

When applying Chapter G of the *Specification* to wide-flange beams, $0.6F_y$ is the shear yielding stress, and A_w is the area of the web. Some textbooks appear to indicate that a uniform distribution of shear stress can be assumed because the maximum web shear stress does not differ much from the average web shear stress.

However, for a rectangular plate the ratio of peak to average stress is 1.5, which does not seem insignificant. Should the peak stress be used when designing rectangular plates to resist shear?

You are correct that the difference between the average shear stress and the maximum shear stress in a wide-flange section is relatively small with $\tau_{peak} / \tau_{average}$ equal to about 1.15. However, this is not the reason the *Specification* is based on the average stress. The stresses above assume an elastic distribution of stress, which does not represent the true failure condition of the element. Instead, the *Specification* is based on an inelastic distribution of stress, which will be uniform.

A similar situation exists related to flexure. If an elastic distribution were used in the *Specification* for flexure, beam strength would be based on S_x. It is not. It is based on Z_x. The lower-bound ratio Z_x/S_x for rolled wide-flange beams is about 1.11, though it varies somewhat among the shapes.

Section J4.2 addresses shear in connecting elements, which are often rectangular sections, and bases the strength on the gross area—the average stress. For example, when we check a double coped beam, leaving what is essentially a rectangular "narrow beam," we base the strength on the average stress, not because we feel that 1.5 is close enough to 1.0 but rather because we are recognizing the inelastic redistribution of stress.

Larry Muir, PE

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