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Built-Up Compression Member

How do I determine the required strength in shear for an intermediate connector in a built-up double-angle compression member?

An article by Aslani and Goel published in the Third Quarter 1992 AISC *Engineering Journal* derives the equation to determine the shear force that develops between the individual components due to buckling as:

$$F_s = \frac{\alpha^2}{\alpha^2+1} P_y \left(1 - \frac{P_{cr}}{P_y}\right)$$

Where

$\alpha = h / 2r_{ib}$, separation ratio for section
h = distance between the centroids of the two components of the built-up member
r_{ib} = z-axis radius of gyration of each component of the built-up member
$P_y = F_y A$
P_{cr} = critical buckling load of built-up member

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

Historic Shape

I want to check the strength of a shape called out on the plans as a "12B19" The plans are dated 1971, but have many revisions and an as-built date of 1976. I've looked in AISC *Design Guide 15*, but I can't find this particular shape.

The 1971 era is just when a change in terminology was being made to phase out the light beam (B) designation. The 7th edition AISC *Manual* reclassified many of the previous light beam shapes as W-shapes, and changed the rolling properties slightly.

The 12B19 had been the designation used in the 6th edition AISC *Manual*, and represented a 12×4 light beam in manuals before the 1971 era. *Design Guide 15* does not list the shape in the 1971 to 2000 section, but rather on page 86. You will see two classifications listed in the 12-in. depth at 19 plf.

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

S-shapes in Grade 50

Are S-shapes available in both ASTM A36 and ASTM A992 material? Is one grade more common?

Table 2-3 in the 13th edition AISC *Steel Construction Manual* lists applicable ASTM specifications for various structural shapes. That table lists S-shapes as being most commonly available in ASTM A36. Similarly, the Shapes Availability link at **www.aisc.org/availability** lists the base grade for S-shapes as ASTM A36. Thus, ASTM A36 remains the default material grade.

However, it is increasingly common that mills are producing S- and similar shapes in other material grades, and especially ASTM A992 (or ASTM A572 Grade 50). You may want to check with some of the mills or service centers listed on that same web link to see what grades they may currently produce or stock.

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

Beam Framing Over Column

What are the requirements for splicing a beam when it is framing over a column; rather than into the side of the column?

Typically when beams frame over a column rather than into the side, it is because the beam is required to be continuous. This continuity could reflect either an interior condition where the beam frames over a column into the next span in one or both directions, or an exterior condition where a spanning beam cantilevers over an exterior column. I am not sure if your condition is one of these, or if you are describing two simply supported beams, each bearing on top of the same column. In any case, whether the beam is continuous or simply supported on top of the column, rotation about the longitudinal axis of the beam must be prevented to provide for stability of the column top. This framing condition is discussed in Part 2 of the 13th edition AISC *Steel Construction Manual*, including common solutions with framing beams, kickers, and stiffeners providing for the stability.

If the beam is continuous over the top of the column, and a splice is required somewhere in the span, it would *not* be desirable to place a beam splice at or close to the point of maximum negative moment over the column. For the continuous cantilever condition, any splice should be located in the back-span, and preferably near the inflection point. As an alternative to a continuous splice, it is common to use cantilevered framing with drop-in beams and shear splices.

For the case of two simply-supported beams bearing on top of the column, the member ends should not be spliced, as this would tend to restrict the rotation that is supposed to occur in the plane of the member.

Part 10 of the 13th edition *Steel Construction Manual* provides guidance for seated shear connections, and Part 12 provides guidance for moment splices.

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

Weld Electrodes

I am doing work on a building constructed in 1963, for which many of the connections are welded. I understand that ASTM A36 steel was used on the project, and need to determine what electrode strength level was used. Did the AISC *Specification* of that era permit the use of E70 electrodes?

The historic specifications are available on the AISC website at **www.aisc.org/epubs**. However, these will only tell you what types of electrodes were permitted to be used; not what type was actually used in the construction.

The AISC *Specification* in effect in 1961 was the 1949 edition, which permitted use of the E60 series electrodes. The *Supplementary Provisions Governing Use of ASTM-A36 Steel* issued in 1960 did not define alternatives for other weld materials.

The AISC *Specification* dated 1963 did permit the use of both E60 and E70 weld types.

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

steel interchange

Drift Index – *H*/400

Would you please explain the background of the origin of the commonly used criterion of a building drift index of *H*/400?

Engineers that have been practicing structural engineering for up to half a century probably remember that the drift index criterion of *H*/400 (0.0025) was a commonly accepted practice when they first started. I remember reading that in the 1960s, when light curtain wall systems were beginning to be used in high-rise framed buildings in lieu of the heavier masonry infill wall systems, a significant reduction in lateral stiffness of the structure was noticed.

Doing a little more research to refresh my memory, I found the subject discussed in the "classic" 1960s steel design textbook *Structural Steel Design*, which often is referred to as "We the People" in reference to the long list of distinguished authors from Lehigh University at the time. The discussion in this text on wind drift stated:

> "The results on measurements of wind deflection of the Empire State Building and a model study of that building show that the masonry walls increased the rigidity of the building about 350 percent above the rigidity of the steel frame. With the trend toward lightweight construction and cladding, much of this restraint may be reduced in the future. An ASCE Committee(1) study of wind bracing in tall buildings recommended limiting deflections to 0.002 times the height of a building although buildings with drift indexes of upwards of 0.004 or 0.005 have behaved satisfactorily."

(1) ASCE Subcommittee No. 31, "Wind Bracing in Steel Buildings (Final Report), *Transactions American Society Civil Engineers*, 105, pp. 1713-1738 (1940).

In discussions with Bob Disque, he indicated that "when I started out as a designer in 1950, the *H*/400 limit was common. I had the feeling at the time that there was no analytical basis, but that it was a result of the experience of engineers who designed high-rise buildings." A review of the referenced ASCE Subcommittee No. 31 Report from 1940 reflects this same concept of being primarily based on experience.

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With consultation from:
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R_y for Seismic Design

The R_y factor is applied to the loads for which to design connections in many seismic applications. However, the R_y factor is not considered in the capacity of the connection material. Is this not an overly conservative approach?

No. R_y is applied to F_y for the yielding element to ensure that the elements intended to remain elastic (like connection elements) can develop the expected strength of the yielding element. We apply R_y because we want to account for material that might be overstrength. However, if R_y were applied in the design of the elements intended to remain elastic, you would be counting on material that might be overstrength, but also might not.

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

Second-Order Analysis for Braced Frames

When accounting for second order effects using the B_1-B_2 method, Section C2.1b of the AISC *Specification* lists two options for R_m under Equation C2-6b. The first option lists R_m as 1.0 for braced-frame systems and the second option lists R_m as 0.85 for moment-frame systems. Why does the *Specification* include a condition for braced-frame systems when determining B_2? My understanding is that braced-frame systems are not supposed to experience lateral translation and, henceforth, no lateral-translational moments or forces need be amplified.

It is incorrect to state that braced frames do not experience lateral translation. Some lateral drift will occur in all braced frames subject to lateral loads, and the amount could be significant, particularly if the frame is tall or if the lateral loads are large. Therefore, the axial loads in the braced frame columns and braces will be higher from a second-order analysis than from a first-order analysis. There are common cases when the amplification is small for a braced frame, but it does exist and its consideration is required by *Specification* Section C1.1. The requirement also shows up in Equation. C2-1b.

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

CJP Groove Weld for HSS

When installing a backing bar on HSS that require an all-around complete-joint-penetration groove weld, should the backing bar be curved to follow the inside corner profile of the shape, or can four individual straight pieces be used?

Section 5.10.2 of AWS D1.1 requires that steel backing shall be made continuous for the full length of the weld, and that all joints in the steel backing shall be a CJP groove weld butt joint. This would preclude the use of segmented backer bars for the all around CJP groove weld.

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

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