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Wind Connections in Seismic Areas

I am designing a large retail store with moment frames. In one direction the wind governs and in the other direction seismic governs. For the flexible moment connection (Type 2 with wind), page 4-100 of the 9th edition manual mentions that the moment connection is to be designed for wind moment only, and it is assumed to rotate enough to be considered simply supported for gravity loads. Does this assumption also apply to moment connections for seismic load?

Question sent to AISC's Steel Solutions Center

The 13th edition manual's updated information on this type of connection states that flexible moment connections are useful for low-seismic application (design in which the seismic response modification factor R is taken equal to 3). For high-seismic applications (design in which R is taken greater than 3), flexible moment connections are not a recognized structural system.

Sergio Zoruba, Ph.D., P.E.

Composite Steel Beam

What are the key advantages of choosing composite steel beam and steel deck with concrete slab instead of a non-composite system? Is it possible to specify that the shear studs be shop-welded to the beams prior to arrival on-site? How can overall quality of shear stud installation be ensured?

Question sent to AISC's Steel Solutions Center

There is a trade-off of beam weight savings associated with the composite beam system versus the additional cost of adding the studs. Generally, the cost of the installed studs would probably be equivalent to about 10 pounds per linear foot of beam weight.

Shop application of studs to steel beams used in building applications is considered a construction safety hazard, and therefore not permitted by OSHA regulations. To eliminate the tripping hazard for ironworkers, shear studs for composite beams are field-installed in buildings.

The use of composite beam construction in building applications is not unusual, and the use of shear studs to achieve this composite action is also very common. The installation of the shear studs is typically done with a specific "gun" process that is controlled and usually very reliable. Section 7 of AWS D1.1 covers the subject of stud-welding in detail, including the subjects of workmanship, techniques, stud application qualification requirements, production control, and fabrication and verification inspection requirements. Following these requirements should provide a high level of confidence in the quality of the product.

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

Base Plate Bending

I cannot find the requirements for how to determine the thickness of a base plate subjected to a weak-axis column moment in AISC's *Design Guide 1: Base Plate and Anchor Rod Design*. Could you provide some references?

Question sent to AISC's Steel Solutions Center

The thickness of the base plate is determined based on the cantilever dimension(s) and the pressure distribution developed between the base plate and foundation as a result of the applied axial load and/or moment. The only difference in procedure is the assumed pressure distribution caused by the weak-axis bending with respect to the orientation of the W-shape. The plane of bending about the weak-axis of the W-shape is shown in Figure 3.1.1(b) in *Design Guide 1* (a free download for AISC members at **www.aisc.org/epubs**). You will see the assumed bending lines about the weak-axis, designated at $0.80b_f$, as opposed to $0.95d$ for the strong axis bending. The pressure resulting from axial compression is additive in defining the pressure distribution for either case of bending.

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

Bolt Values

I noticed that the bolt values have changed for A325 N bearing bolts from the 9th edition manual (7/8" A325 N bolt = 12.6 kips) to the 13th edition (Table 7.1 gives 14.4 kips). Is that true?

Question sent to AISC's Steel Solutions Center

Yes, that is true. There are many areas in the new manual where allowable strength has increased as compared to the 9th edition. In your example, the 1989 and 2005 AISC specifications use different values and approaches to get the results you mentioned. A quick review of this example is as follows:

9th ed. manual

F_v = 21 ksi (allowable)

$P_a = F_vA = (21)(0.601) = 12.6$ kips

13th ed. manual

F_v = 48 ksi (nominal), factor of safety $\Omega = 2.0$

$P_a = F_vA/\Omega = (48)(0.601)/2.0 = 14.4$ kips

The F_v in the 1989 ASD specification was 21 ksi (allowable), whereas in the 2005 AISC specification, it is listed as 48/2.0 = 24 ksi (allowable). Hence an increase of nearly 15% in allowable strength is gained in this instance by using the latest ASD procedure found in the 2005 AISC specification.

This increase is due to a change made by RCSC in the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*. The factor implicit in the tabular values accounting for reduction due to threading was increased from 0.7 to 0.8.

Sergio Zoruba, Ph.D., P.E.

Torsional Unbraced Length

What is the torsional unbraced length? Isn't this equal to the lateral unbraced length? If a simply supported W-shape beam is laterally supported at the top flange every 5 ft and at the bottom flange every 10 ft, is the lateral unbraced length 5 ft for the top flange and 10 ft for the bottom flange? Isn't the torsional unbraced length the same?

Question sent to AISC's Steel Solutions Center

When bracing a beam flange for flexure without torsion, Appendix 6 of the 2005 AISC specification covers lateral bracing and torsional bracing. Note that for flexural members, Section F2 of the specification states, "L_b = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section."

Your question, however, does not appear to be about these two beam bracing approaches, but rather about bracing of a beam that is also subject to a torsional moment. The following answer relates to that case.

The phrases *torsional brace* and *lateral brace* are indicative of the nature of the movement that the brace is attempting to prevent. The torsional unbraced length is the distance between braces that prevent a member from rotation about its longitudinal axis. The lateral unbraced length is the distance between braces that prevent relative movement of the compression flange.

In your example the beam is simply supported, with the top flange in compression, and restrained against rotation at the end supports. The compression (top) flange is laterally braced every 5 ft, so the lateral unbraced length is 5 ft. If the same beam is connected such that twist is prevented by effective braces at the top and bottom flange only every 10 ft, the torsional unbraced length is longer at 10 ft.

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

Column Splice Design

On the contract drawings, the engineer calls for us to develop the column splice for 100% of the gross moment capacity of the upper column. Usually we use Table 14-3 from the 13th edition manual, and in this situation, Case VI, because the engineer requests bolted/welded. I am assuming this table does not incorporate the 100% requirements. If this is the case, can you lead us in the right direction to design for such requirements?

Question sent to AISC's Steel Solutions Center

Right, the table doesn't incorporate 100% requirements. The typical column splice details shown in the manual are based on transferring compressive loads through bearing and providing for shear transfer, in addition to providing for stability of the column shaft during erection. Developing a column splice for 100% of the gross moment is usually uneconomical and rarely required. Usually, column splices are avoided at locations that require full development of the cross-section. However, if this is required, it would seem that the most straightforward approach would be to use a detail with complete-joint-penetration groove welds on both the flanges and web.

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

Single-Plate Shear Connections

What are AISC's recommendations regarding the use of single-plate beam-to-girder shear connections? Typically, we do not use this type of connection, but I was wondering if there is an article discussing the pros/cons of this connection type.

Question sent to AISC's Steel Solutions Center

Single-plate shear connections are useful for both beam-to-girder and beam-to-column connections. However, past approaches have been restricted by the permitted distance from the weld line to the bolt line. The simplified and rationalized approach published in the 13th edition AISC manual allows more options and improves the usefulness of the single-plate shear connection for beam-to-girder connections. Where the limited distance to the bolt line used to require coping of the supported beam top flange, the new procedure allows an extended configuration in which the plate can be longer and the framing beam square cut. This increases the economy of the connection detail.

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