

24 Tips for Simplifying Braced Frame Connections

Paying attention to braced frame connections early in the design process can minimize detailing, fabrication, and erection problems.

BY VICTOR SHNEUR, P.E.

ALMOST EVERY BUILDING HAS BRACED FRAMES. And even though they are often isolated within buildings and don't encompass the majority of the steel members, design and construction teams pay a lot of attention to them because they are unique and serve a critical function.

Here are a few suggestions for making braced frame connections easier to design, detail, fabricate, and erect, along with a few recommendations for avoiding field problems.

1 Select an R of 3 or less whenever possible. When $R > 3$ the AISC *Seismic Provisions* must be applied, which has a significant associated cost implication.

2 Specify ASTM F1554 Grade 36 and Grade 55 anchor rods with S1 (Supplementary Requirement for Weldability). Use larger diameter rods whenever possible instead of specifying F1554 Grade 105 anchor rods. Welding repair is not an option at these anchor rods, and more sophisticated (and expensive) repairs for mislocated or short anchor rods may be required.

3 Consider using shear keys or embedded plates with field-welded side plates to transfer substantial horizontal shear forces from the column base to the foundation. Concrete floors can also transmit shear force. Do not try to develop substantial shear with anchor rods—the large holes required for anchor rods make it difficult to transmit shear. See the AISC *Design Guide 1—Column Base Plates* for more information, available to purchase at www.aisc.org/bookstore.

4 Consider a detail with fillet welds for column-to-base plate connections. Fillet welds up to $\frac{3}{4}$ " (on both sides) are typically less expensive than complete joint penetration welds. Also, remember to apply a directional strength increase

factor permitted by AISC and AWS.

5 Don't specify "all-around" fillet welds for column base plates. Fillet welds that wrap around the flange ends (flange toes) and web-to-flange fillets take additional time because of changing welding positions and may lead to cracks due to high residual stresses in the welds. Also, these welds add very little to the strength. Try to stay with welds on both sides of the web and flanges, if possible.

6 Consider using thicker base plates rather than adding brackets or stiffener plates at base plates when the base plate connection needs to resist large uplift forces. Brackets and stiffeners add cost and complicate connections, making field fixes for anchor rods extremely expensive.

7 Try diagonal bracing, V-bracing, or chevron bracing, if possible. These configurations need fewer connections and members than X-bracing. Also, architects may prefer diagonal or chevron bracing because they offer greater freedom in locating doors.

8 Use W12 and W14 columns whenever possible. Unless architecturally required, avoid W10 or W8 columns because they have very limited space between flanges, which makes connections more difficult.

9 Don't use HSS columns when brace forces are substantial. HSS columns are very "sensitive" to horizontal loads, and difficult connections with through plates may be required. Instead, try to use wide-flange columns or move brace work points to the face of the HSS column. In the latter case, horizontal components of brace forces don't need to be resisted by gusset plates and girder-to-column connections (uniform force method).



Victor Shneur is Chief Engineer of LeJeune Steel Company in Minneapolis.

10 Use a minimum of W18 braced frame girders if loads and other framing considerations allow. Avoid shallow (and heavy) braced frame girders, even if they meet the design criteria, because they make it difficult to frame deeper floor beams. In many cases the brace girder-to-column connection also needs to resist large horizontal forces, and it is much easier to do so in the case of a deep girder.

11 Don't specify camber for braced frame girders. This complicates the connections and adds cost. On the other hand, large end connections for girders and gusset plates provide more end restraint and reduce deflection.

12 In the case of tensile brace forces, don't size members based on only the gross section. The effective net section may control and require costly reinforcement. Size tension braces based on the gross and the effective net section, staying on the conservative side when the actual connection design is unknown.

13 Always consider fabrication and erection factors when sizing braces. Don't size the braces based only on the design loads. For example, using fewer brace sizes may lead to extra material weight, but it will simplify detailing, fabrication, and erection. It will also reduce the amount of possible mistakes. Contact your local fabricator for his or her opinion in special cases (very large forces, sophisticated paint systems, unique geometry, etc.).

14 Specify actual tensile forces for column splices. Don't specify that column splices should develop the full tensile or bending strength of the upper shaft. This requires very expensive field splices, often with complete joint penetration welds. Due to typical splice locations and the presence of compressive forces, it is an extremely rare case when full bending or tensile strength needs to be developed.

15 Don't specify that column splices should resist horizontal shear forces. Because the braces resist horizontal shear, column splices don't need to be sized for shear forces.

16 Always provide actual brace forces for economical connections. Don't require that brace connections be sized for the full tensile or compressive strength of the member. In a case when the brace is sized for tensile force, there's always the question, "Should it be sized for the full strength of the gross or effective net section strength?" Such design requirements lead to confusion and more expensive connections.

17 For economical connections, always specify actual gravity end reactions. Don't specify vertical end reactions for braced frame girders based on the percentage of UDL.

18 If transfer forces need to be developed, always specify them. Don't assume that the connection engineer will figure out the transfer forces. (The transfer force is the axial force that must be transferred to the opposite side of the column. It is not necessarily the beam axial force that is obtained from a computer analysis of the structure. Thornton, 1991)

19 If calculations show that stiffeners or doublers are required at columns, first try to use heavier members or gusset plates. Upgrading gusset plate material is also a good option. Don't automatically specify stiffeners and doublers at columns, girders, and gusset plates. They make connections much more complex and difficult to fabricate. Refer to Chapter 3 "Economic Selection of Columns" in AISC *Design Guide 13—Wide-Flange Column Stiffening at Moment Connections* for suggestions and cost comparisons.

20 When a brace and girder frame into the column flange, the work point doesn't always need to be at the column and/or girder center line. If the column or girder can take additional bending moment due to eccentricity (see the Uniform Force Method), the work point can be moved to the face of the column and to the top or bottom of the girder. This makes braced frame connections much simpler because connection eccentricity doesn't need to be considered for gusset and girder to column connections, and brace connections become more compact because of improved geometry.

21 If typical connections (end plate or double-angle) work, use them. Don't create any special connections—they drive up cost.

22 Remember clearances and tolerances. At just one brace connection, there are at least three members coming from different directions and framing with heavy connections. Disregarding clearances and tolerances may make connections unworkable and lead to costly field modifications. Refer to the AISC *Steel Construction Manual* for required clearances for welding and bolting, and provide adequate room. Depending on actual connections, there are a number of different ways to provide for tolerances, such as; using slip-critical bolts in oversized holes for brace connections; detailing braced frame girders that have large end connections a little shorter in case columns overrun in depth; and using field shims, etc.

23 Ask for advice. In case of a special situation, contact a local fabricator or erector for his or her opinion. This can save time and money down the road.

24 The perfect design will not eliminate all mistakes. Good connections substantially reduce the number of shop and field problems, but people make mistakes, actual tolerances may be larger than expected, etc. As the engineer of record:

- **Request an as-built report.** It clearly shows the problem and eliminates misunderstanding or misinterpretation.
- **Discuss possible solutions with a fabricator or erector—** chances are they already have a suggestion.
- **Consider the cost—**labor is expensive, material is cheap.
- **Try to proceed with fast decisions and approvals** to continue the work.
- **Remember that not every error requires correction.**

Each project has different forces, member sizes, and braced frame elevations, but one fact remains the same: bracing connections should be efficient. They need to be efficient from a fabrication and erection standpoint, as well as from a strength standpoint. This dual purpose is best achieved when designers combine their expertise with the expertise of the rest of the design and construction team.