steelwise

REINFORCING THE POINT

BY BO DOWSWELL, P.E., PH.D.

It's important to know your options, as well as your existing structure, before moving forward with a reinforcement scheme.

STRENGTH, STABILITY AND STIFFNESS are, of course, all factored into the design of a structural framing system. But as buildings age, take on new uses or are expanded, sometimes they a need a structural boost.

There are several ways to incorporate structural reinforcement into a building or other structure. Sometimes the simplest method is to change the load path by adding new members. If a concrete floor is present, shear connections can be added to engage the concrete in composite action. Another method is the addition of prestressing. Finally, one can employ member reinforcement, which is accomplished by enlarging the member to increase the section properties. Whichever method is used should consider safety and any potential negative effects on the structure during erection.

Here, we'll look at key factors to keep in mind when reinforcing beams and columns, as well as welding and tolerance considerations.

Beams

Let's start with beams. Floor systems, roofing or other obstructions may limit access and prevent welding to both flanges of a beam. This limited access, combined with the desire to eliminate overhead welding, usually leads to built-up shapes that are singly symmetric. Section F4 in the AISC *Specification* applies to singly symmetric I-shaped members bent about their major axis. Calculation methods in other publications, such as the *Guide to Stability Design Criteria for Metal Structures* (by R.D. Ziemian), can be used to determine lateral-torsional buckling loads for non-I-shaped members.

Common reinforcement schemes for beams are shown in Figure 1. When the bottom flange is easily accessible, the most economical option is a single plate welded to the bottom flange, as shown in Figure 1a. Because the plate is wider than the flange, welding is done in the horizontal position, which has about four times the production speed of overhead welding. Also, the camber due to weld shrinkage is upward and the plate can be easily clamped in place for fit-up. If more strength or stiffness is required, an additional plate can be welded to the top flange as shown in Figure 1b; however, the top flange is usually not accessible in commercial buildings. The top plate is usually narrower than the beam flange to allow horizontal-position welding.

Another option is tee reinforcement, shown in Figure 1c, which provides large increases in strength and stiffness but requires overhead welding. Web plates and cap channels—shown in Figures 1d and 1e, respectively—are effective in resisting weak-axis bending. Closed sections are extremely efficient in resisting torsion. The web plate and double angle reinforcement schemes, shown in Figures 1d and 1f, should be considered for these cases. If lateral-torsional buckling is a concern, the web plate and double angle reinforcement, as well as the channel cap, are efficient options.

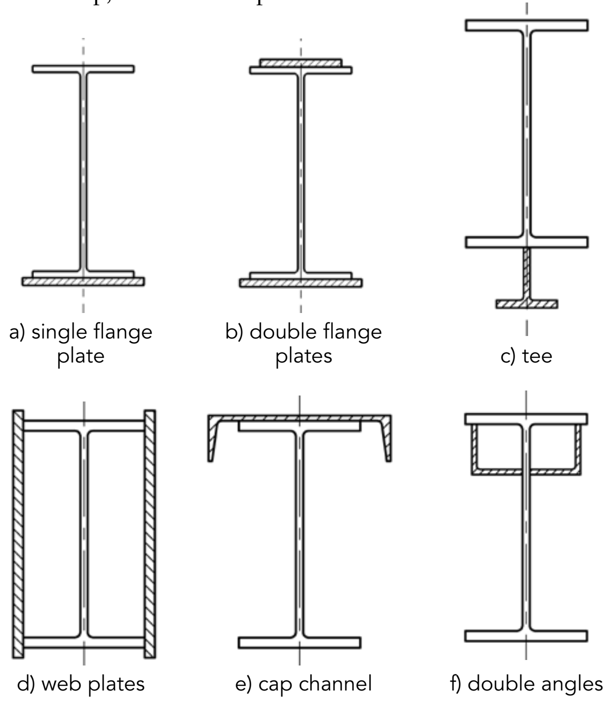

Fig. 1. Common reinforcement schemes for beams.

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In determining the reinforcement scheme, an important practical consideration is the types of obstructions that will be encountered. For example, the bottom flange reinforcement in Figure 2a must be stopped short of the end of the beam. If the reinforcement must extend to the end of the beam, another type of reinforcement may be more economical. A similar case is shown in Figure 2b for web plate reinforcement obstructed by a secondary beam framing to the web of the reinforced beam.

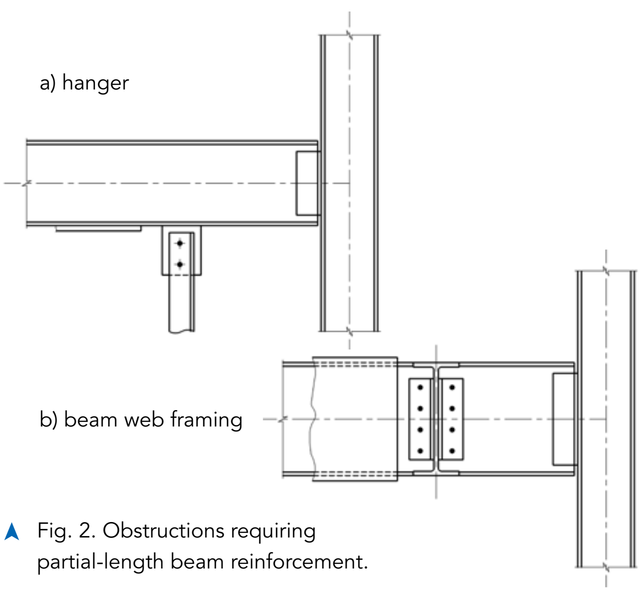


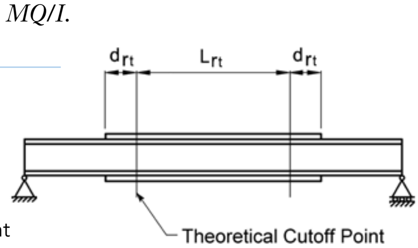
Fig. 2. Obstructions requiring partial-length beam reinforcement.

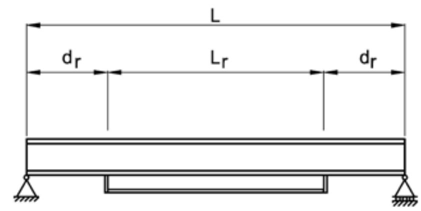
Due to obstructions and economic considerations, reinforcement is often placed on only part of the member length. Because reinforced members have an abrupt change in stiffness at the reinforcement cutoff location, a stepped member approach must be used to determine member stability. In their 1971 article "Elastic Lateral Buckling of Stepped I-Beams" (in the *Journal of the Structural Division*) Trahair and Kitipornchai developed a simple equation to calculate the lateral-torsional buckling moment of stepped beams with equal setbacks at each end, as shown in Figure 3a. Partial-length tee reinforcement often requires end plates to provide lateral stability to the tee, as shown in Figure 3b.

Section F13.3 of the AISC *Specification* requires partial-length reinforcement to extend beyond the theoretical cutoff point (see Figure 3a). The force that must be developed over length, d_{rt}, can be calculated using an elastic distribution as discussed in the Commentary to AISC *Specification* Section F13.3. This gives a force, $F = MQ/I$.

Fig. 3. Partial-length beam reinforcement.

a) load development





b) stabilizer plates

Fig. 3. Partial-length beam reinforcement.

An important serviceability consideration is the deflection of beams reinforced under load. The total deflection is the sum of the pre-reinforcement deflection and the post-reinforcement deflection. If the structural analysis model is built with the reinforced section properties, only the deflection of the reinforced member under the total load is provided in the output. Because the initial deflection should be calculated with the non-reinforced member properties, the actual deflection will be higher than the computer output value. An additional consideration is the weld shrinkage deformation that can cause upward or downward deformation, depending on the weld sequence, member properties, heat input, reinforcement geometry and initial load at the time of welding.

Columns

Switching gears to columns, several reinforcement schemes are shown in Figure 4. Because eccentricities between the axial load and the centroid of the reinforced cross section must be accounted for in the design, reinforcement is most efficient when placed symmetrically about the centroid of the member. However, girts, walls and other obstructions may limit access and prevent welding to both flanges of a member. Double-flange plates, shown in Figure 4a, are usually the most economical scheme.

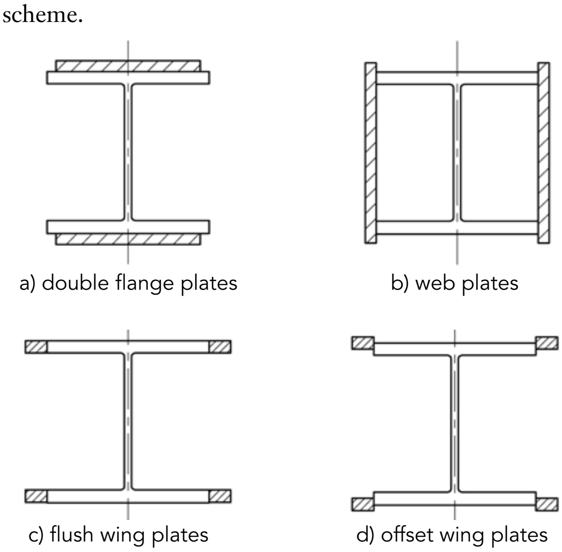


Fig. 4. Common reinforcement schemes for columns.

Obstructions requiring partial-length column reinforcement are shown in Figure 5. Under most conditions, the reinforcement can be discontinuous at lateral brace locations. In these cases, the column should be designed as a stepped member. Additionally, the non-reinforced part of the column must be checked for yielding.

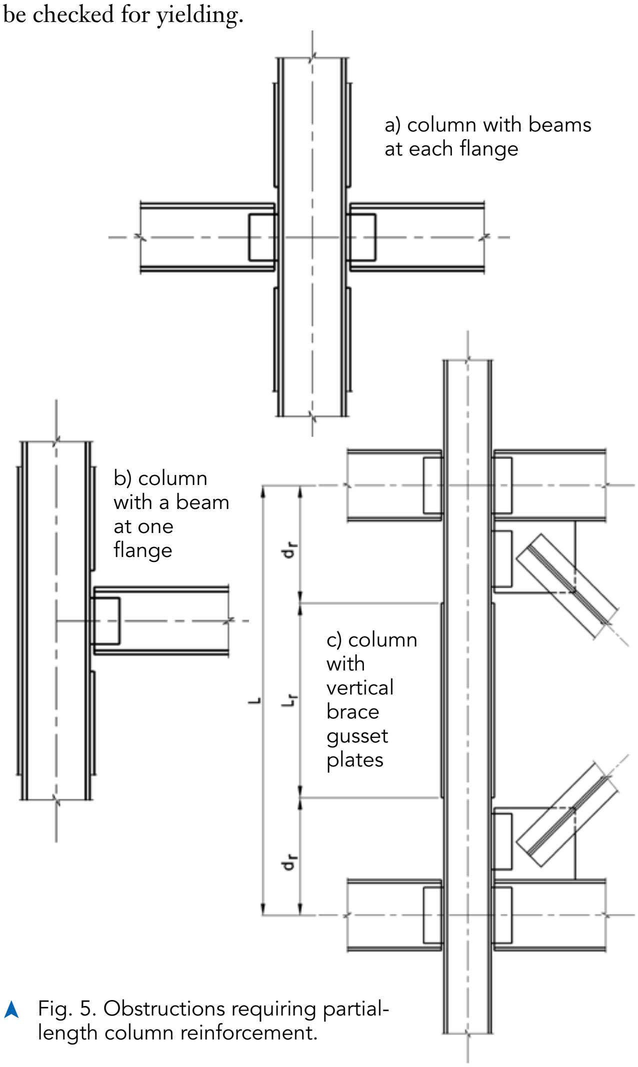


Fig. 5. Obstructions requiring partial-length column reinforcement.

In the rare case where the non-reinforced part of the column is overstressed, cross-sectional area can be added with wing plates welded between the cover plates. The flush wing plates in Figure 4c require groove welds; therefore, the more economical choice is usually the offset wing plates shown in Figure 4d, which can be fillet welded to the column. The wing plates must extend beyond the end of the cover plate for a length adequate to develop the load into the column.

In a 1969 *Engineering Journal* article, "Some Non-Conventional Cases of Column Design," S.T. Dalal tabulated effective length factors for the stepped column configurations shown in Figure 6. The effective length factors can be used with the lateral buckling provisions in *Specification* Section E3.

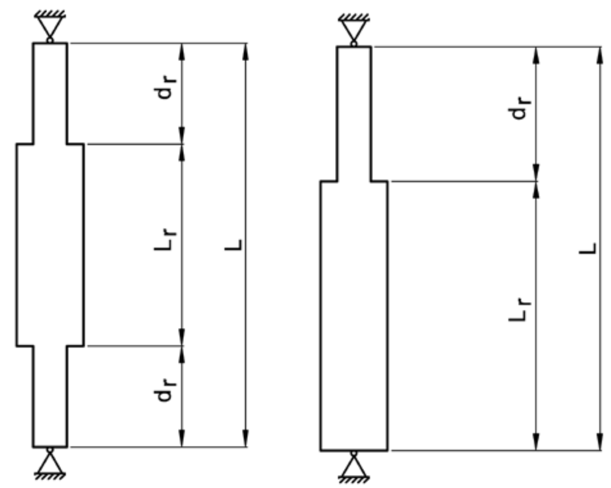


Fig. 6. Partial-length column reinforcement.

Welding

There are several factors to consider when it comes to welding reinforcement members.

Coating removal. Section M3.5 in the AISC *Specification* requires surfaces within 2 in. of a field weld to be "free of materials that would prevent proper welding or produce objectionable fumes during welding." Although many coating systems are not detrimental to the strength of the finished weld, most should be removed due to health concerns for the welder. Corrosion and other materials should also be removed prior to welding.

Stitch welds. To reduce welding distortion and cost, reinforcing plates are usually stitch welded to the member as shown in Figure 7. If the structure is exposed to extremely corrosive conditions, continuous welds may be required. However, if an adequate paint system is applied after welding, properly-spaced stitch welds can be used for most structures exposed to normal weather conditions.

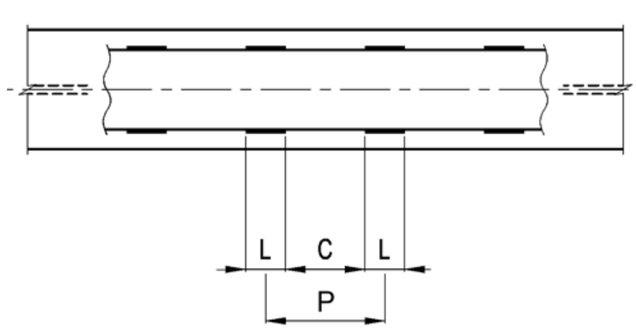


Fig. 7. Stitch welding of reinforcement plate.

Various parts of the AISC *Specification* limit the longitudinal distance between welds. The provisions in Section J3.6 are intended to ensure close fit-up over the entire faying surface and to prevent corrosion between the connected elements. The dimensional limitations in Section J2.2b are to ensure proper welding techniques. For members loaded in axial compression, the limits in Section E6.2 are to prevent longitudinal buckling between the welds.

Weldability. To minimize the risk of cracking of the weld and base metal, the weldability of the existing steel must be analyzed. As discussed in AWS D1.7 *Guide for Strengthening and Repairing Existing Structures*, several different carbon equivalent (CE) equations have been developed to estimate weldability based on the chemical composition of a steel. The CE value indicates the level of brittleness of the heat affected zone upon weld cooling; therefore, as the CE value increases, weldability decreases.

The chemical content of the steel can be found in mill test reports or chemical tests of samples cut from redundant parts of existing structures. Weldability can also be ensured if the structure has been successfully welded in the past. A bend test to determine weldability is also described by D. Ricker in a 1988 *Engineering Journal* article, "Field Welding to Existing Structures."

Welding to loaded members. In addition to the final as-built design, member strength during erection must be considered. Welding has a detrimental effect on loaded members due to a reduction in material properties at high temperatures near the arc. In some cases, the load can be removed from the reinforced member until welding is complete; however, this can be impractical and is usually not required because the effect of welding heat is highly localized.

Member strength can be evaluated based on a reduced cross section, where the high-temperature area near the arc is inactive in resisting load. The width of inactive material is proportional to the heat input, which is dependent on the current, voltage and arc travel speed. For welds with low heat input, the inactive width is less than 3 in. parallel to arc travel and 2 in. perpendicular to arc travel. However, the inactive width can be much larger for high heat input processes, such as flux-cored arc welding (FCAW). The welding procedure specification (WPS) provided by the erector will include the required information for heat input calculations.

General guidelines for low heat input are:

- Low welding current
- Small diameter electrodes
- Allow time for welds to cool between successive passes
- Use stringer beads only (in lieu of optional stringer or weave beads)
- Intermittent welding in short lengths
- Temperature crayons or other suitable means should be used to monitor the temperature of the base metal near the weld

Member Tolerances

All members have initial imperfections. The initial out-of-straightness has a critical effect on the lateral buckling strength of columns. Figure 8 shows this effect using three magnitudes of initial out-of-straightness. The column that is in tolerance, represented by the solid line, has typical column behavior where the P-δ curve is almost linear until the maximum load is approached. The middle curve, representing the column with δ_0 = 0.48 in., is out of tolerance by a factor of three. This column behavior is characterized by a more nonlinear P-δ curve, which results in a higher second order moment and lower strength. The bottom curve represents a damaged column.

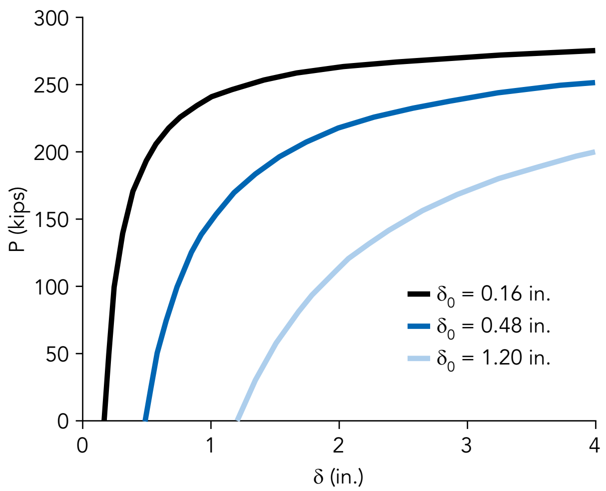


Fig. 8. Load versus lateral deformation for columns with various initial out-of-straightness.

For reinforced members, geometric imperfections are caused by a combination of:

- Imperfections of the non-reinforced member resulting from rolling, fabrication and erection
- P-δ deformations from the initial load in the member
- Shrinkage deformation from welding of reinforcement (see Figure 9, next page)

All imperfections except weld shrinkage deformation can be measured prior to member design. Weld shrinkage deformation for non-loaded members can be estimated using empirical equations (for example, see Blodgett, 1966). The values should be adjusted to account for any load in the member at the time of welding.

According to Section 6.4.2 in the AISC *Code of Standard Practice*, the maximum variation in straightness for a built-up shape is $1/1000$ of the length between points of lateral support. The total post-weld out-of-straightness for members with significant initial load is likely to exceed this value—especially where singly symmetric reinforcement is used. Because the column curve in the AISC *Specification* is based on the maximum variation in

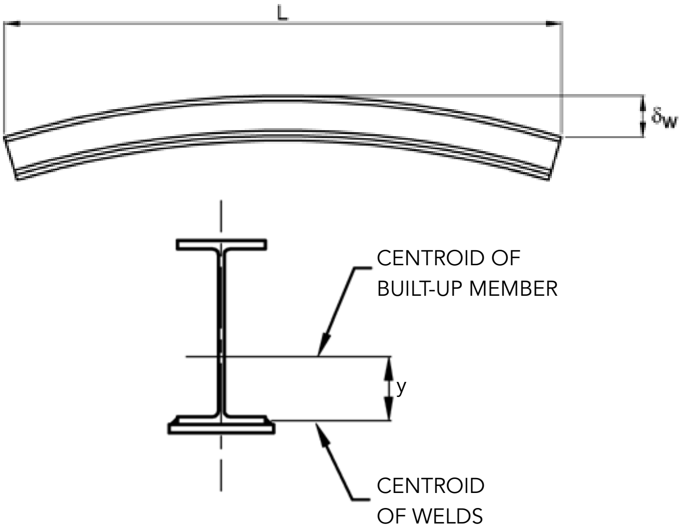


Fig. 9. Camber due to weld shrinkage.

straightness allowed by the AISC *Code of Standard Practice*, any out-of-straightness in excess of 1/1000 of the length of the member must be accounted for in member design. This can be accomplished using any of the methods in Chapter C of the *Specification*.

Contract documents must convey the importance of minimizing weld distortion. As a minimum, a simple drawing note should be provided, stating, "Reinforcement shall be welded by qualified welders using techniques and sequences that minimize post-weld distortion of the member. Welding procedure specifications and welding sequences shall be submitted to the engineer of record for review." In the design stage, distortion can be minimized by selecting intermittent welds and other welds with low heat input. Post-weld member tolerances should also be included in the contract documents.