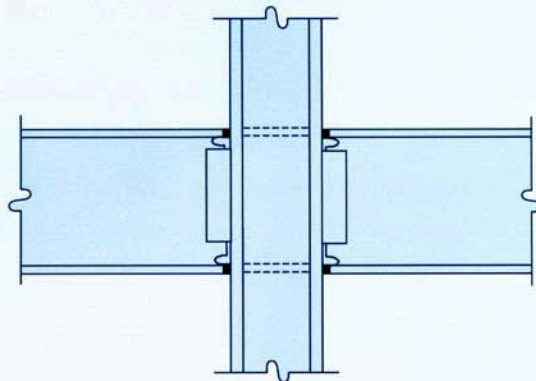




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Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications



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Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications

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American Institute of Steel Construction, Inc.
Chicago, IL

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Chapter 1

INTRODUCTION

1.1 Scope

The design of columns for axial load, concurrent axial load and flexure, and drift considerations is well established. However, the consideration of stiffening requirements for wide-flange columns at moment connections as a routine criterion in the selection of the components of the structural frame is not as well established. Thus, the economic benefit of selecting columns with flange and web thicknesses that do not require stiffening is not widely pursued, in spite of the efforts of other authors who have addressed this topic previously (Thornton, 1991; Thornton, 1992; Barger, 1992; Dyker, 1992; and Ricker, 1992). This Design Guide is written with the intent of changing that trend and its contents are focused in two areas:

1. The determination of design strength and stiffness for unreinforced wide-flange columns at locations of strong-axis beam-to-column moment connections; and,
2. The design of column stiffening elements, such as transverse stiffeners (also known as continuity plates) and web doubler plates, when the unreinforced column strength and/or stiffness is inadequate.

Recommendations for economy are included in both cases.

Force transfer and design strength of unreinforced columns with strong-axis moment connections are covered

in Chapter 2. Economical considerations for unreinforced columns and columns with reinforcement are given in Chapter 3. Force transfer and design strength of reinforced columns with strong-axis moment connections, as well as the design of transverse stiffeners and web doubler plates, is covered in Chapter 4. Special considerations in column stiffening, such as stiffening for weak-axis moment connections and framing arrangements with offsets, are covered in Chapter 5. Design examples that illustrate the application of these provisions are provided in Chapter 6, with design aids for wind and low-seismic applications in Appendices A, B, and C.

1.2 Column Stiffening

Transverse stiffeners are used to increase the strength and/or stiffness of the column flange and/or web at the location of a concentrated force, such as the flange force induced by the flange or flange-plate of a moment-connected beam. Web doubler plates are used to increase the shear strength and stiffness of the column panel-zone between the pair of flange forces from a moment-connected beam. The panel-zone is the area of the column that is bounded by the column flanges and the projections of the beam flanges as illustrated in Figure 1-1.

If transverse stiffeners and/or web doubler plates carry loads from members that frame to the weak-axis of the

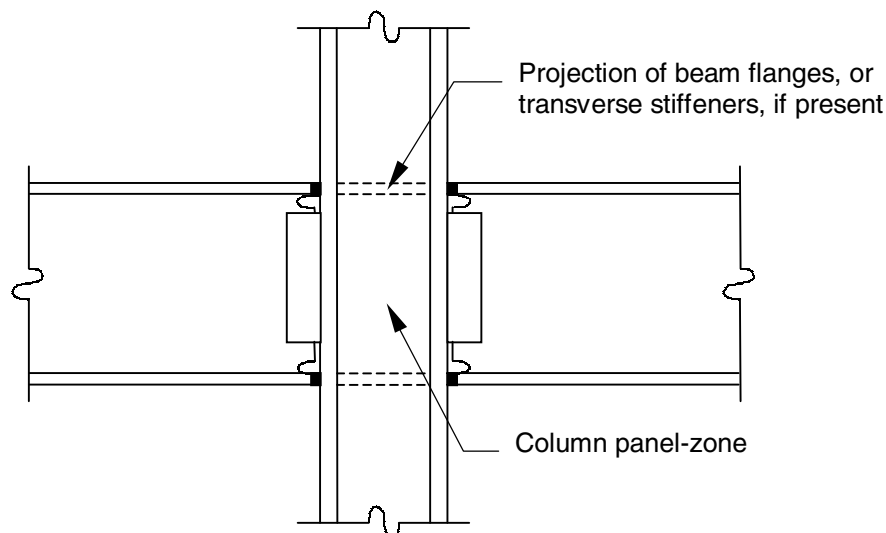


Figure 1-1 Illustration of column panel-zone.

column, the recommendations herein must be adjusted as discussed in Sections 5.2, 5.3, and 5.5. As discussed in Section 5.4, if web doubler plates are required to increase the panel-zone shear strength, they can also be used to resist local web yielding, web crippling, and/or compression buckling of the web per LRFD Specification Section K1. As discussed in Section 5.6, diagonal stiffening can be used in lieu of web doubler plates if it does not interfere with the weak-axis framing.

1.3 References Specifications

This Design Guide is generally based upon the requirements in the AISC LRFD *Specification for Structural Steel Buildings* (AISC, 1993), hereinafter referred to as the LRFD Specification, and the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997a), hereinafter referred to as the AISC Seismic Provisions. Although direct reference to the AISC *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* (AISC, 1989) is not included, the principles herein remain generally applicable.

1.4 Definitions of Wind, Low-Seismic, and High-Seismic Applications

For the purposes of this Design Guide, wind, low-seismic and high-seismic applications are defined as follows. Wind and low-seismic applications are those for which the structure is designed to meet the requirements in the LRFD Specification with no special seismic detailing. This includes all applications for which the structural response is intended to remain in the nominally elastic range and the response modification factor R used in the determination of seismic forces, if any, is not taken greater than 3.

High-seismic applications are those for which inelastic behavior is expected in the beams or panel-zones as a means of dissipating the energy induced during strong ground motions. Such buildings are designed to meet the requirements in both the LRFD Specification and the AISC Seismic Provisions and a response modification factor R that is appropriate for the level of detailing required for the moment-frame system selected is used in the determination of seismic forces.¹ Additionally, the moment connections used in high-seismic applications have special seismic detailing that is appropriate for the moment-frame system selected.

1.5 Acknowledgements

This Design Guide resulted partially from work that was done as part of the Design Office Problems activity of the ASCE Committee on Design of Steel Building Structures. Chapter 3 is based in large part upon this previous work. Additionally, the AISC Committee on Manuals and Textbooks has enhanced this Design Guide through careful scrutiny, discussion, and suggestions for improvement. The author thanks the members of these AISC and ASCE Committees for their invaluable input and guidance. In particular, Lawrence A. Kloiber, James O. Malley, and David T. Ricker contributed significantly to the development of Chapters 3 and 4 and William C. Minchin and Thomas M. Murray provided helpful comments and suggestions throughout the text of this Design Guide.

¹From AISC Seismic Provisions Commentary Table I-C4-1, R -values of 8, 6, and 4 are commonly used for Special Moment Frames (SMF), Intermediate Moment Frames (IMF), and Ordinary Moment Frames (OMF), respectively.

Chapter 2

STRONG-AXIS MOMENT CONNECTIONS TO UNREINFORCED COLUMNS

In wind and low-seismic applications, it is often possible to use wide-flange columns without transverse stiffeners and web doubler plates at moment-connected beams. To use an unreinforced column, the following criteria must be met:

1. The required strength (Section 2.1) must be less than or equal to the design strength (Section 2.2); and,
2. The stiffness of the column cross-section must be adequate to resist the bending deformations in the column flange (Section 2.3).

If these criteria cannot be met, column stiffening is required.

In high-seismic applications, transverse stiffeners are normally required, as discussed in Section 2.3. However, it remains possible in many cases to use wide-flange columns in high-seismic applications without web doubler plates at moment-connected beams.

2.1 Force Transfer in Unreinforced Columns

In an unreinforced column, concentrated forces from the beam flanges or flange plates are transferred locally into the column flanges. These concentrated forces spread through the column flange and flange-to-web fillet region into the web as illustrated in Figure 2-1a. Shear is dispersed between them in the column web (panel-zone) as illustrated in Figure 2-1b. Ultimately, axial forces in the column flanges balance this shear as illustrated in Figure 2-1c.

2.1.1 Required Strength for Local Flange and Web Limit States

In wind and low-seismic applications, beam end moments, shears, and axial forces are determined by analysis for the loads and load combinations in LRFD Specification Section A4.1. Note that the total design moment is seldom equal to the flexural strength of the beam(s). A rational approach such as that illustrated in Example 6-4 or similar to that proposed by Disque (1975) can be used in conjunction with these loads and load combinations. Different load combinations may be critical for different local-strength limit states.

For the general case, the beam end moment is resolved at the column face into an effective tension-compression

couple in the beam flanges or flange plates. The corresponding flange force P_{uf} is calculated as:

$$P_{uf} = \frac{M_u}{d_m} \pm \frac{P_u}{2} \quad (2.1-1)$$

where

P_{uf} = factored beam flange force, tensile or compressive, kips

M_u = factored beam end moment, kip-in.

d_m = moment arm between the flange forces,² in.

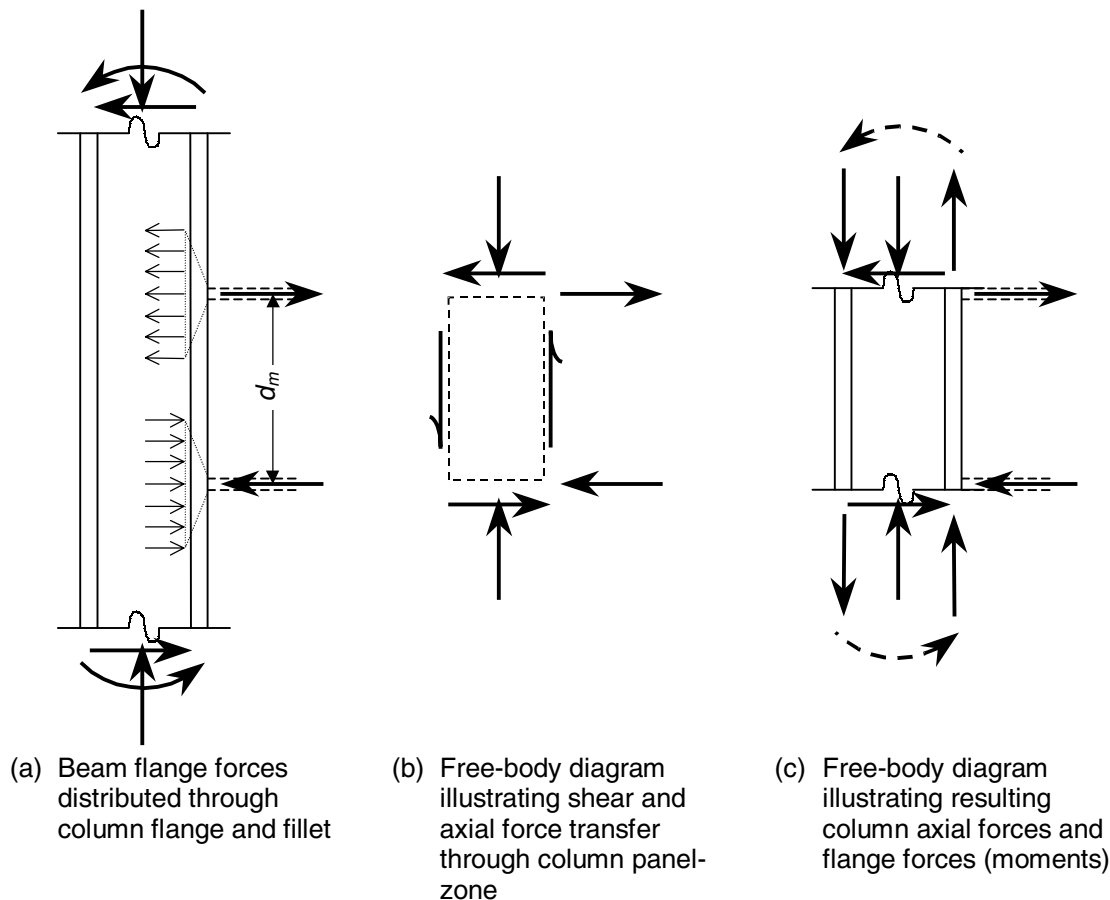
P_u = factored beam axial force, kips

The formulation of Equation 2.1-1 is such that the combined effect of the moment and axial force is transmitted through the flange connections, ignoring any strength contribution from the web connection, which is usually more flexible.

When the moment to be developed is less than the full flexural strength of the beam, as is commonly the case when a drift criterion governs the design, and the axial force is relatively small, this calculation is fairly straightforward. However, when the full flexural strength of the beam must be developed, or when the axial force is large, such a model seems to guarantee an overstress in the beam flange, particularly for a directly welded flange moment connection. Nonetheless, the above force transfer model remains acceptable because inelastic action into the range of strain hardening allows the development of the design flexural strength of the beam in the connection (Huang et al., 1973). Such self-limiting inelastic action is permitted in LRFD Specification Section B9. Alternatively, a web connection with a stiffness that is compatible with that of the connections of the beam flanges can be used to activate the full beam cross-section and reduce the portion carried by the flanges.

Note that, if a composite moment connection is used between the beam and column, the calculations in Equations 2.1-1 and 2.1-2 must be adjusted based upon the appropriate

²The actual moment arm can be readily calculated as the distance between the centers of the flanges or flange plates as illustrated in Figure 2-1a. Alternatively, as stated in LRFD Specification Commentary Section K1.7, 0.95 times the beam depth has been conservatively used for d_m in the past.



Note: beam shear and axial force (if any) omitted for clarity.

Figure 2-1 Force transfer in unreinforced columns.

detailing and force transfer model. Some possible composite connections are illustrated in AISC (1997a), Leon et al. (1996), and Viest et al. (1998).

In high-seismic applications, the moments, shears, and axial forces are determined by analysis for the loads and load combinations in LRFD Specification Section A4.1 and AISC Seismic Provisions Section 4.1. The resulting flange force P_{uf} is then determined using Equation 2.1-1. Note that the corresponding connection details have special seismic detailing to provide for controlled inelastic deformations during strong ground motion as a means of dissipating the input energy from an earthquake.³

For Ordinary Moment Frames (OMF), a cyclic inelastic rotation capability of 1 percent is required. Moment connections such as those discussed in AISC Seismic Provisions Commentary Section C11.2 and illustrated in

³With strong panel-zones and fully restrained (FR) construction, the primary source of inelasticity is commonly hinging in the beam itself. If the panel-zone is a significant source of inelasticity, or if partially restrained (PR) construction is used, the flange-force calculation in Equation 2.1-2 should be adjusted based upon the actual force transfer model.

Figure C-11.1 can be used. From AISC Seismic Provisions Section 11.2a, the flange forces in Ordinary Moment Frames (OMF) need not be taken greater than those that correspond to a moment M_u equal to $1.1R_yF_yZ_x$ or the maximum moment that can be delivered by the system, whichever is less.

For Special Moment Frames (SMF) and Intermediate Moment Frames (IMF), a cyclic inelastic rotation capability of 3 and 2 percent, respectively, is required. Several alternative connection details using reinforcement, such as coverplates, ribs, or haunches, or using reduced beam sections (dogbones), have been successfully tested and used. Such connections shift the location of the plastic hinge into the beam by a distance a from the column face as illustrated in Figure 2-2. From AISC Seismic Provisions Section 9.3a, the flange forces in Special Moment Frames (SMF) and Intermediate Moment Frames (IMF) need not be taken greater than:

$$P_{uf} = \frac{M_u}{d_m} = \frac{1.1R_yF_yZ + V_u a}{d_m} \quad (2.1-2)$$

where 1.1 is an adjustment factor that nominally accounts for the effects of strain hardening, and

R_y = an adjustment factor that nominally accounts for material yield overstrength per AISC Seismic Provisions Section 6.2

= 1.5 for ASTM A36 wide-flange beams

= 1.3 for ASTM A572 grade 42 wide-flange beams

= 1.1 for wide-flange beams in other material grades (e.g., ASTM A992 or A572 grade 50)

F_y = beam specified minimum yield strength, ksi

Z = plastic section modulus of beam cross-section at hinge location (distance a from column face), in.³

V_u = shear in beam at hinge location (distance a from column face), kips

a = distance from face of column flange to plastic hinge location, in.

The axial force effect is neglected in Equation 2.1-2, since the model is already based conservatively upon the fully yielded and strain-hardened beam flange at the critical section.

2.1.2 Required Strength for Panel-Zone Shear

As illustrated in Figure 2-3, the total panel-zone shear force V_u at an interior column results from the combined effects of two moment-connected beams and the story shear V_{us} . In wind and low-seismic applications, the total panel-zone shear force V_u is calculated as:

$$V_u = (P_{uf})_1 + (P_{uf})_2 - V_{us} \quad (2.1-3)$$

In high-seismic applications, when the flange forces have been calculated using the moment resulting from AISC

Seismic Provisions Load Combinations 4-1 and 4-2 and Equation 2.1-1, the total panel-zone shear force is calculated with Equation 2.1-3. As a worst case, however, the total panel-zone shear force V_u need not be taken greater than:

$$V_u = 0.8[(P_{uf})_1 + (P_{uf})_2] - V_{us} \quad (2.1-4)$$

The factor 0.8 in Equation 2.1-4 is from AISC Seismic Provisions Section 9.3a. It recognizes that the effect of the gravity loads will counteract some portion of the effect of the lateral loads on one side of an interior column and thereby inhibit the development of the full plastic moment in the beam on that side.

In wind, low-seismic, and high-seismic applications, for a column with only one moment-connected beam, Equation 2.1-3 can be reduced to:

$$V_u = P_{uf} - V_{us} \quad (2.1-5)$$

Note that gravity-load reduction, as used for high-seismic applications in Equation 2.1-4, is not appropriate in Equation 2.1-5 for a column with only one moment-connected beam.

2.2 Determining the Design Strength of an Unreinforced Column

An unreinforced column must have sufficient strength locally in the flange(s) and web to resist the resulting flange-force couple(s). Moment connections are termed “double concentrated forces” in LRFD Specification Section K1 because there is one tensile flange force and one compressive flange force acting on the same side of the column as illustrated in Figure 2-4a. When opposing moment-

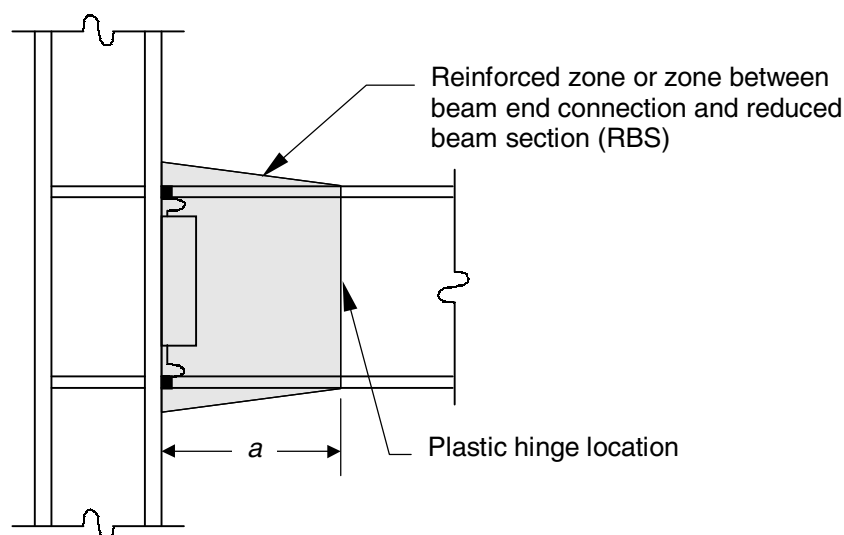


Figure 2-2 Schematic illustration of moment connection for high-seismic applications.

connected beams coincide, a pair of double concentrated forces results as illustrated in Figures 2-4b (the gravity load case) and 2-4c (the lateral load case).

The design strength of the panel-zone in shear must be checked for all columns with moment connected beams. For a tensile flange force, the design strength of the flange in local flange bending and the design strength of the web in local yielding must also be checked. For a compressive flange force, the design strength of the web in local yielding, crippling, and compression buckling must be checked. Note that the compression buckling limit state is applicable only when the compressive components of a pair of double concentrated forces coincide as illustrated in Figure 2-4b (i.e., at the bottom flanges). If the magnitudes of these opposing flange forces are not equal, the compression buckling limit state is checked for the smaller flange force, since only this portion of the larger flange force must be resisted. Each of these limit states is discussed below.

2.2.1 Panel-Zone Shear Strength

In wind and low-seismic applications and high-seismic applications involving Ordinary Moment Frames (OMF), the design shear strength of the panel-zone ϕR_v is determined with the provisions of LRFD Specification Section K1.7, which allows two alternative assumptions.

The first assumption is that, for calculation purposes, the behavior of the panel-zone remains nominally within the elastic range. The resulting design strength given in Equations 2.2-1 and 2.2-2 is then determined from LRFD Specification Equations K1-9 or K1-10 with consideration of the magnitude of the axial load P_u in the column:

$$\text{For } P_u \leq 0.4P_y, \quad \phi R_v = 0.9 \times 0.6F_y d_c t_w \quad (2.2-1)$$

$$\text{For } P_u > 0.4P_y, \quad \phi R_v = 0.9 \times 0.6F_y d_c t_w \left(1.4 - \frac{P_u}{P_y} \right) \quad (2.2-2)$$

In the second assumption, it is recognized that significant post-yield panel-zone strength is ignored by limiting the calculated panel-zone shear strength to that in the nominally elastic range. At the same time, it must be realized that inelastic deformations of the panel-zone can significantly impact the strength and stability of the frame. Accordingly, a higher strength can generally be utilized as long as the effect of inelastic panel-zone deformation on frame stability is considered in the analysis. When this option is selected, the resulting design strength given in Equations 2.2-3 and 2.2-4 is determined from LRFD Specification Equations K1-11 and K1-12 with consideration of the magnitude of the axial load P_u in the column:

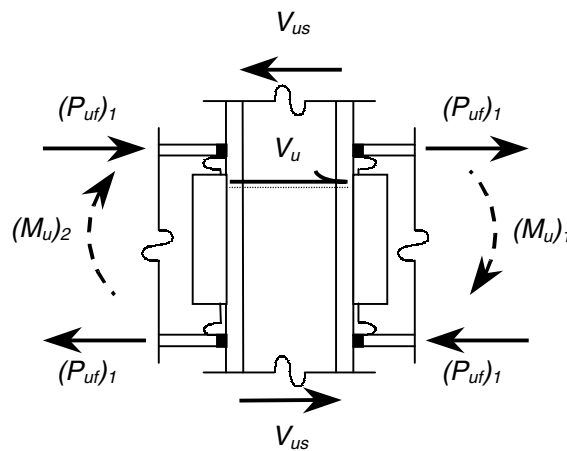
$$\text{For } P_u \leq 0.75P_y,$$

$$\phi R_v = 0.9 \times 0.6F_y d_c t_w \left(1 + \frac{3b_f t_f^2}{d_b d_c t_w} \right) \quad (2.2-3)$$

$$\text{For } P_u > 0.75P_y,$$

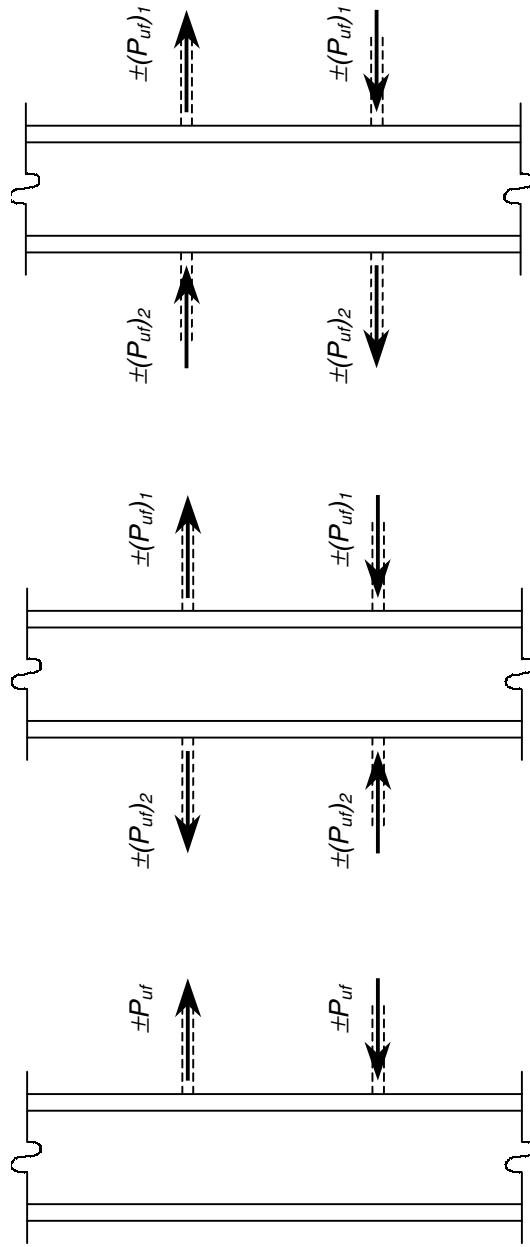
$$\phi R_v = 0.9 \times 0.6F_y d_c t_w \left(1 + \frac{3b_f t_f^2}{d_b d_c t_w} \right) \left(1.9 - \frac{1.2P_u}{P_y} \right) \quad (2.2-4)$$

For F_y equal to or less than 50 ksi, all W-shapes listed in ASTM A6 except a W30 $\times$ 90 and a W16 $\times$ 31 have a web thickness that is adequate to prevent buckling



Note: shear forces in beams and moments and axial forces in column omitted for clarity.

Figure 2-3 Panel-zone web shear at an interior column (with moment-connected beams bending in reverse curvature).



(a) Double concentrated forces (b) A pair of double concentrated forces, gravity load case (c) A pair of double concentrated forces, lateral load case

Figure 2-4 Moment connection flange force terminology.

under panel-zone web shear per LRFD Specification Section F2. For $F_y = 50$ ksi, these two shapes exceed the limit on h/t_w by 1.9 and 1.5 percent, respectively. Thus, for all practical purposes, in wind and low-seismic applications, shear buckling of the column web need not be checked for columns with F_y equal to or less than 50 ksi.⁴

In high-seismic applications involving Special Moment Frames (SMF) or Intermediate Moment Frames (IMF), the effect of inelastic panel-zone deformation on frame stability must be considered in the analysis. The design shear strength of the panel-zone ϕR_v given in Equations 2.2-5 and 2.2-6 is determined from AISC Seismic Provisions Section 9.3a:

For $P_u \leq 0.75P_y$,

$$\phi R_v = 0.75 \times 0.6F_y d_c t_w \left(1 + \frac{3b_f t_f^2}{d_b d_c t_w} \right) \quad (2.2-5)$$

For $P_u > 0.75P_y$,

$$\phi R_v = 0.75 \times 0.6F_y d_c t_w \left(1 + \frac{3b_f t_f^2}{d_b d_c t_w} \right) \left(1.9 - \frac{1.2P_u}{P_y} \right) \quad (2.2-6)$$

These provisions are identical to those in LRFD Specification Equations K1-11 and K1-12, except that a lower resistance factor is used to provide an added margin against excessive panel-zone yielding. Additionally, to prevent shear buckling under the higher inelastic demand associated with high-seismic loading, the minimum thickness of the unreinforced column web given in Equation 2.2-7 is determined from AISC Seismic Provisions Section 9.3b:

$$t_{w \min} = \frac{d_m + d_c - 2t_f}{90} \quad (2.2-7)$$

where

- t_w = column web thickness, in.
- b_f = column flange width, in.
- t_f = column flange thickness, in.
- d_b = beam depth, in.
- d_c = column depth, in.
- F_y = column minimum specified yield strength, ksi
- P_u = column required axial strength, in.
- $P_y = F_y A$, column axial yield strength, in.
- A = column cross-sectional area, in.²
- d_m = moment arm between concentrated flange forces, in.

⁴If using allowable stress design, the shear buckling limit is slightly more conservative and the following W-shapes must be checked for shear buckling: W44×230, W40×215, W40×199, W40×183, W40×174, W40×167, W40×149, W36×150, W36×135, W33×130, W33×118, W30×99, W30×90, W27×84, W24×68, W24×55, W21×44, W18×35, W16×31, W16×26, W14×22, and W12×14.

Note that Equation 2.2-7 is in a form that has been adapted from that which appears in the AISC Seismic Provisions.

2.2.2 Local Flange Bending

When a directly welded flange or flange-plated moment connection is used, differential stiffness across the width of an unstiffened column flange results in a stress concentration in the weld adjacent to the column web as illustrated in Figure 2-5 that must be limited for tensile flange forces. The design local flange bending strength ϕR_n given in Equation 2.2-8 is determined from LRFD Specification Equation K1.1 with consideration of the proximity of the concentrated flange force to the end of the column:

$$\phi R_n = 0.9 \times 6.25t_f^2 F_y \times C_t \quad (2.2-8)$$

When an extended end-plate moment connection is used, flange bending must be limited to prevent yielding of the column flange under tensile flange forces. The design local flange bending strength ϕR_n given in Equation 2.2-9 is determined from Murray (1990) with consideration of the proximity of the concentrated flange force to the end of the column as:

$$\phi R_n = 0.9 \times \left(\frac{b_s}{\alpha_m p_e} \right) t_f^2 F_y \times C_t \quad (2.2-9)$$

where

- t_f = column flange thickness, in.
 - F_y = column specified minimum yield strength, ksi.
- Note that Equation 2.2-9 was developed from research that considered only ASTM A36 material (Curtis and Murray, 1989). If column material with higher yield strength is used, it is recommended that F_y be taken conservatively as 36 ksi in Equation 2.2-9.

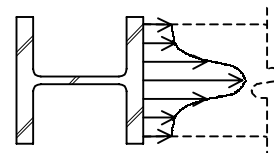


Figure 2-5 Concentration of stress in flange or flange-plate weld for a column with thin flanges and no transverse stiffeners.

$C_t = 0.5$ if the distance from the column end to the closer face of the beam tension flange is less than $10t_f$
 $= 1$ otherwise
 $b_s = 2.5(2p_f + t_{fb})$, in., for a four-bolt unstiffened extended end plate; see Figure 2-6a
 $= 2p_f + t_{fb} + 3.5p_b$, in., for an eight-bolt stiffened extended end plate; see Figure 2-6b
 p_f = distance from centerline of bolt to nearer surface of the tension flange, in.; d_b plus $1/2$ in. is generally enough to provide wrench clearance; 2 in. is a common fabricator standard
 t_{fb} = beam flange thickness, in.
 p_b = vertical pitch of bolt group above and bolt group below tension flange, in.
 $\alpha_m = 1.36 \left(\frac{p_e}{d_b} \right)^{1/4}$ for a four-bolt unstiffened extended end plate
 $= 1.13 \left(\frac{p_e}{d_b} \right)^{1/4}$ for an eight-bolt stiffened extended end plate
 $p_e = \frac{g}{2} - \frac{d_b}{4} - k_1$
 g = bolt gage, in.
 d_b = bolt diameter, in.
 k_1 = distance from beam web centerline to flange toe of flange-to-web fillet, in.

2.2.3 Local Web Yielding

When a directly welded flange or flange-plated moment connection is used, the concentrated force is distributed to the column web as illustrated in Figure 2-7a. The design local web yielding strength ϕR_n given in Equation 2.2-10 is determined from LRFD Specification Equations K1-2 or K1-3 with consideration of the proximity of the concentrated flange force to the end of the column:

$$\phi R_n = 1.0 \times [C_t(5k) + N]F_y t_w \quad (2.2-10)$$

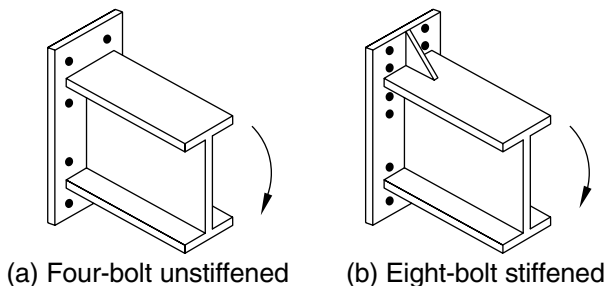


Figure 2-6 Configuration of extended end-plate moment connections.

When an extended end-plate moment connection is used, the concentrated force is distributed to the column web as illustrated in Figure 2-7b. The design local web yielding strength ϕR_n given in Equation 2.2-11 is determined from Murray (1990) with consideration of the proximity of the concentrated flange force to the end of the column:

$$\phi R_n = 1.0 \times [C_t(6k + 2t_p) + N]F_y t_w \quad (2.2-11)$$

where

t_w = column web thickness, in.
 F_y = column specified minimum yield strength, ksi
 k = distance from outside face of column flange to the web toe of the flange-to-web fillet, in.
 N = beam flange or flange plate thickness plus $2w$, in.
 $C_t = 0.5$ if the distance from the column end to the closer face of the beam flange is less than d_c
 $= 1$ otherwise
 w = leg size of fillet weld or groove weld reinforcement, if used, in.
 t_p = end-plate thickness, in.
 d_c = column depth, in.

2.2.4 Web Crippling

The design local web crippling strength ϕR_n given in Equation 2.2-12 is determined from LRFD Specification Equations K1-4, K1-5, or K1-6 with consideration of the proximity of the concentrated flange force to the end of the column:

$$\phi R_n = 0.75 \times 135 C_t t_w^2 \left[1 + N_d \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_y t_f}{t_w}} \quad (2.2-12)$$

where

$C_t = 0.5$ if the distance from the column end to the closer face of the beam compression flange is less than $d_c/2$
 $= 1$ otherwise
 t_w = column web thickness, in.
 $N_d = 3N/d_c$ if the distance from the column end to the closer face of the beam tension flange is either:
 (1) greater than or equal to $d_c/2$; or,
 (2) less than $d_c/2$ and N/d_c is less than or equal to 0.2.
 $= \left(\frac{4N}{d_c} - 0.2 \right)$ otherwise
 t_f = column flange thickness, in.
 F_y = column specified minimum yield strength, ksi
 N = beam flange or flange plate thickness plus $2w$ for directly welded flange or flange-plated moment connection, in.
 $=$ beam flange thickness plus $(2w + 2t_p)$ for extended end-plate moment connections, in.

w = leg size of fillet weld or groove weld reinforcement, if used, in.
 t_p = end-plate thickness, in.
 d_c = column depth, in.

Note that, from LRFD Specification Commentary Section K1.4, for the rolled shapes listed in ASTM A6, the limit state of web crippling will not govern the design of transverse stiffening for a moment connection, except to a W12×50 or W10×33 column. That is, if transverse stiffening is required, another limit state, such as local web yielding or local flange bending, will be more critical in all except the aforementioned two cases.

2.2.5 Compression Buckling of the Web

When a pair of compressive flange forces coincide as illustrated in Figure 2-4b, the column web is subject to out-of-plane buckling as illustrated in Figure 2-8. The design

web compression buckling strength ϕR_n given in Equation 2.2-13 is determined from LRFD Specification Equations K1-8 with consideration of the proximity of the concentrated flange force to the end of the column:

$$\phi R_n = 0.90 \times \frac{4,100 C_t t_w^3 \sqrt{F_y}}{h} \quad (2.2-13)$$

where

$C_t = 0.5$ if the distance from the column end to the closer face of the compression flanges is less than $d_c/2$

$$= 1 \text{ otherwise}$$
$$t_w = \text{column web thickness, in.}$$
 F_y = column specified minimum yield strength, ksi
$$h = d_c - 2k, \text{ in.}$$
 d_c = column depth, in.

k = distance from outside face of column flange to the web toe of the flange-to-web fillet, in.

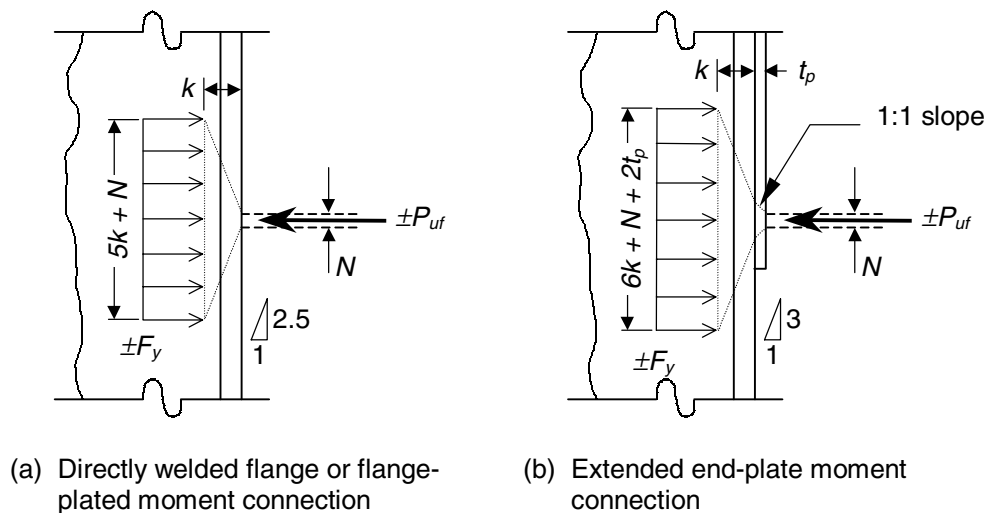


Figure 2-7 Local force transfer for local web yielding limit state.

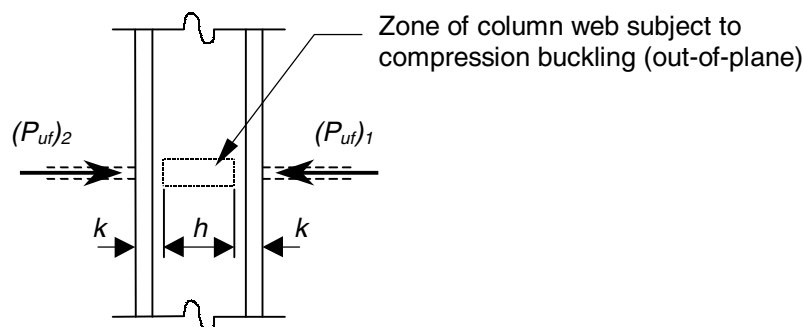


Figure 2-8 Compression buckling of the column web.

2.3 Column Cross-Sectional Stiffness Considerations

In addition to satisfying the strength requirements given in Section 2.2, the supporting column must also have sufficient stiffness to resist local deformations of the cross-section under the tensile and compressive flange forces. In wind and low-seismic applications, design for the strength criteria in Section 2.2 has historically resulted in columns with suitable stiffness as well as strength. In high-seismic applications, however, the associated higher inelastic demand necessitates a more explicit consideration of flange stiffness to limit the variation in stress distribution across the width of the connected flange or flange plate. AISC Seismic Provisions Sections 9.5 and 11.3 indicate that transverse stiffeners that match the configuration of those used in the qualifying cyclic tests (see AISC Seismic Provisions Appendix S) for the moment connection to be used are required. Note that transverse stiffeners are not required

if testing demonstrates that the intended inelastic rotation can be achieved without their use.

2.4 Design Aids

For wind and low-seismic applications, the determination of the design strength of unreinforced wide-flange shapes used as columns is simplified with the tables in Appendices A, B, and C. In Appendix A, the design column panel-zone shear strength is tabulated. In Appendix B, the design local column strength at locations of concentrated flange forces is tabulated assuming that the concentrated force is not at a column-end location. In Appendix C, the design local column strength at locations of concentrated flange forces is tabulated assuming that the concentrated force is at a column-end location. The use of these tables is illustrated in several of the example problems in Chapter 6.

Chapter 3

ECONOMICAL SELECTION OF COLUMNS

Transverse stiffeners and web doubler plates are extremely labor-intensive detail materials due primarily to the fit-up and welding that is associated with their use. Additionally, issues such as restraint, lamellar tearing and welding sequence must be addressed when transverse stiffeners and/or web doubler plates are used. As such, they add considerable cost in spite of their disproportionately low material cost. If transverse stiffeners and web doubler plates can be eliminated and an unreinforced column can be used, significant cost savings can often be realized. Additionally, the elimination of column stiffening will simplify (and thereby economize) connections that are made to the weak axis of the column.

In wind and low-seismic applications, the specification of column sizes that eliminate transverse stiffeners is encouraged. In high-seismic applications, however, transverse stiffeners will normally be required, as discussed previously in Section 2.3.

In wind, low-seismic, and high-seismic applications, the specification of column sizes that eliminate web doubler plates is encouraged. Web doubler plates require significant welding into the column flange-to-web fillet region (k-area), which is an area of potentially lower notch toughness (AISC, 1997b). The shrinkage that accompanies the cooling of these welds typically can distort the cross-section and overwelding in this region carries the potential for cracking. Additionally, the weld joint may require the use of a non-prequalified detail as discussed in Section 4.4.3.

3.1 Achieving Balance Between Increases in Material Cost and Reductions in Labor Cost

In Table 3.1, estimated costs are given for some arbitrarily selected transverse stiffener and web doubler plate details as illustrated in Figure 3-1. These estimated costs were determined by averaging the cost estimates⁵ provided by several fabricators and rounding the result to the nearest five-dollar increment. When comparing these typical details to actual details, it should be noted that the comparative weld types and sizes are of much greater significance than the thicknesses or overall dimensions of the plate materials. It is the labor involved in cutting, profiling, and

⁵The estimated costs are predicated upon the material and labor costs that existed at the time this Design Guide was written (circa early 1999). Because it is anticipated that labor costs will continue to rise at a faster rate than material costs, the user may find it advantageous to periodically inquire with local fabricators to determine a more current estimate of these costs.

welding transverse stiffeners and web doubler plates that is predominant in their cost.

An equivalent column weight change is tabulated from these estimated costs based upon a mill price of \$425 per ton, which is a median value in the common range of from \$400 to \$450 per ton FOB,^{6,7} and a 14-ft floor-to-floor height. The tabulated values are calculated as the estimated cost times 2000 lb per ton divided by \$425 per ton divided by the 14-ft length. The resulting value is the estimated maximum per-foot column-weight increase that could be made to eliminate that element of the column stiffening without increasing cost. In fact, because the tabulated values do not consider other intangible economic benefits, such as the simplification of connections that are made to the weak axis of the column, the tabulated value should be considered conservative.

As an example, consider a W14×90 column with full-depth transverse stiffeners (Case 5, Table 3.1) at each beam flange (2 pairs total) and one web doubler plate (Case 8, Table 3.1). The total of the tabulated column-weight-change values for this column stiffening arrangement is 40 lb/ft + 82 lb/ft = 122 lb/ft. Thus, if any heavier W14 up to and including a W14×211 column could be used without transverse stiffeners and a web doubler plate, it would likely be more economical than the W14×90. In most cases, the actual increase in column weight required to eliminate column stiffening will be much less than the maximum calculated and a significant economic benefit can be realized.

When the required column-weight change exceeds the sum of the tabulated values, some engineering judgment must be used. If the comparison is unfavorable, but still close, the use of a heavier column might still be justified by the aforementioned intangibles. Alternatively, the designer may still find it advantageous to investigate the possibility of eliminating the web doubler plate only (or transverse stiffeners only in some cases).

As an example, consider again the W14×90 column with full-depth transverse stiffeners (Case 5, Table 3.1) at each beam flange (two pairs total) and one web doubler plate (Case 8, Table 3.1). If any heavier W14 up to

⁶FOB stands for “free on board,” which indicates that the quoted price assumes delivery to the indicated location. In the above case, the indicated location is the mill itself; subsequent shipping would incur additional cost.

⁷Because mill prices fluctuate, the designer may find it advantageous to periodically inquire with fabricators, steel mills, or other shape suppliers to determine the current range of mill prices.

and including a W14×159 column could be used without a web doubler plate, but with the transverse stiffeners, it would be more economical than the W14×90. Similarly, if any heavier W14 up to and including a W14×120 column could be used without transverse stiffeners, but with a web doubler plate, it would be more economical than the W14×90.

3.2 Eliminating Column Stiffening

From Section 3.1, it is clear that there is significant potential for economic benefit when transverse stiffeners and web doubler plates can be eliminated. Therefore, the designer should consider alternatives that eliminate the need for column stiffening, when possible. The design aids in Appendices A, B, and C provide for the rapid identification of column strength and stiffening requirements in wind and low-seismic applications. Some additional suggestions follow.

1. Specify column material with yield strength of 50 ksi, such as ASTM A992 or A572 grade 50 steel. The increased minimum yield strength will increase

the design strength of the column, yet there will be little or no impact on the material cost. Mill grade extras for 50-ksi wide-flange material are largely nonexistent in shapes that weigh as much as 150 lb per ft of length.⁸ Even for W-shapes in weight ranges that have grade extras, these nominal cost differences of two or three pennies per pound are negligible when compared to the advantage gained in detail material savings. Column material with even higher yield strength, such as ASTM A913 grade 65 material, is also available; however, the associated material cost differential is greater.

2. Consider a different column section that has a thicker flange and/or web, as appropriate. This increase in material cost, given today's typical FOB mill price for common grades⁹ of steel of approximately \$400 to \$450 per ton, is in most cases

⁸Inquire with steel mills to determine the current range of shapes for which a grade extra applies.

⁹Common grades include ASTM A992, ASTM A572 grade 50, and A36.

Table 3.1
Estimated Cost of Various Column Stiffening Details (as illustrated in Figure 3-1)

Case	Thickness	Attachment to Column Flange	Attachment to Column Web	Estimated Cost	Equivalent Column Weight (lb/ft) if Wide-Flange Steel Costs \$425 per Ton from Rolling Mill ³
Partial-Depth Transverse Stiffeners (Two Pairs) 4 PL 4½ × 0'-10 (ASTM A36) with one ¾ × ¾ corner clip each					
1	½ in.	fitted to bear	¾-in. fillet welds	\$80	27
2	1 in.	fitted to bear	¾-in. fillet welds	\$120	40
3	½ in.	¼-in. fillet welds	¾-in. fillet welds	\$90	30
4	1 in.	½-in. fillet welds ¹	¾-in. fillet welds	\$140	47
Full-Depth Transverse Stiffeners (Two Pairs) 4 PL 4½ × 1'-0 (ASTM A36) with two ¾ × ¾ corner clips each					
5	½ in.	¼-in. fillet welds	¾-in. fillet welds	\$120	40
6	1 in.	½-in. fillet welds	¾-in. fillet welds	\$210	71
7	1½ in.	CJP groove weld	½-in. fillet welds ¹	\$470	158
Web Doubler Plate (One) 1 PL 12½ × 2'-0 (ASTM A36)					
8	½ in.	CJP groove weld	¾-in. fillet welds	\$245	82
9	¾ in.	CJP groove weld	¾-in. fillet welds	\$370	124
10	¾ in.	⅝-in. fillet weld ²	¾-in. fillet welds	\$215	72
11	1 in.	⅞-in. fillet weld ²	¾-in. fillet welds	\$305	103

¹The consulted fabricators were asked if they would instead prefer a CJP-groove-welded detail in place of this larger-size fillet-welded detail. In all cases, the answer was no.

²A ¾-in. by ¾-in. bevel on the column-flange edges of the web doubler plate is used to clear the column flange-to-web fillet. It should be noted that the fillet-welded web doubler plate detail in Case 10 is not suitable for high seismic applications because the weld size does not develop the strength of the full thickness of the web doubler plate.

³A floor-to-floor height of 14 ft has been used in this tabulation.

easily offset by the savings in labor costs, as illustrated previously in Section 3.1.

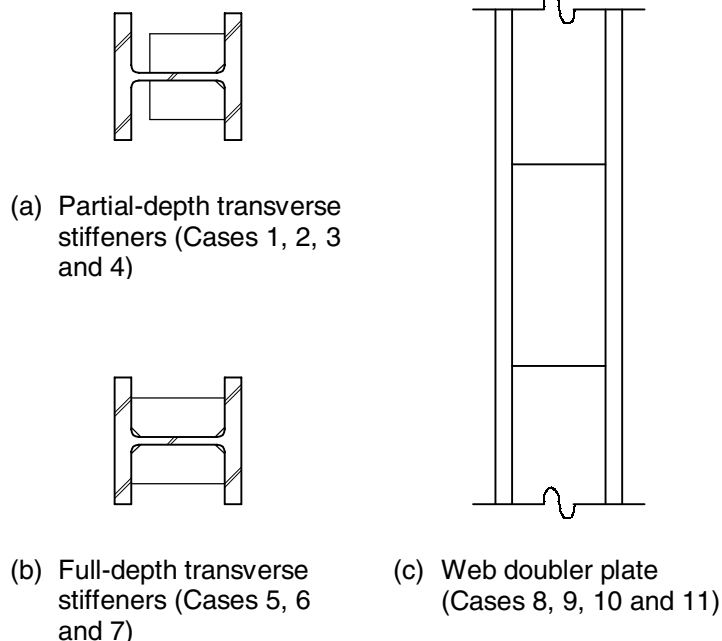
3. Consider a deeper cross-section for the beam that is connected to the column. Increasing the depth of the beam decreases the flange force delivered due to the increase in moment arm between the flange-force couple. If it were possible to replace a W16×50 with a W18×50, the material cost would not be increased; if a lighter, deeper shape were suitable, the material cost would in fact be decreased. Even if there were an increase in material cost, it would in most cases be easily offset by the savings in labor costs. Note that this suggestion may instead be punitive when the moment connection is designed to develop the strength of the beam.
4. Increase the number of moment-resisting connections and/or frames to reduce the magnitude of the moment delivered to a given connection to a level that is within the local design strength of the column section.

3.3 Minimizing the Economic Impact of Column Stiffening Requirements in Wind and Low-Seismic Applications

In some cases, the need for column stiffening may not be avoidable. When this is the case, the following suggestions may help minimize the cost impact for building structures

in wind and low-seismic applications:

1. Where allowed by governing building codes, design column stiffening in response to the actual moments and resulting flange forces rather than the full flexural strength of the cross-section; the latter simply wastes money in the majority of cases. When the Engineer of Record (EOR) delegates the determination of the column stiffening requirements, the design forces and moments should also be provided.
2. If designing in allowable stress design, take advantage of the allowable stress increase in wind-load applications (load combinations in LRFD inherently account for such concurrent occurrence of transient loads).
3. Properly address reduced design strength at column-end applications. The typical beam depth is usually such that the reduced design strength provisions for column-end applications apply only at the nearer flange force.
4. Increase the number of moment-resisting connections and/or frames to reduce the magnitude of the moment delivered to a given connection to a level that allows a more economical stiffening detail.
5. Give preference to the use of fillet welds instead of groove welds when their strength is adequate and the application is appropriate (see Chapter 4).



Note: dimensions and edge connections for the above column stiffening elements are as given in Table 3.1, based upon a W14 column.

Figure 3-1 Column stiffening arrangements for cost estimates in Table 3.1.

This is particularly true for the welds connecting transverse stiffeners to the column.

6. When possible, use a partial-depth transverse stiffener, which is more economical than a full-depth transverse stiffener because it need not be fitted between the column flanges. Select the partial-depth transverse stiffener length to minimize the required fillet-weld size for the transverse-stiffener-to-column-web weld.
7. While transverse stiffeners are required in pairs when the limit states of local flange bending or local web yielding are less than the required strength, a single transverse stiffener is permitted and should be considered when the limit states of web crippling and/or compression buckling of the web only are/is less than the required strength.
8. In cases when the flange force is only compressive, allow the option to weld the transverse stiffener end or to finish it to bear on the inside flange. In most lateral load resisting frames, however, moments are reversible and the design flange force may be either tensile or compressive.
9. Use a single web doubler plate up to a required thickness of $\frac{1}{2}$ in. If thicker web reinforcement is required, consider the use of two plates, one on each side of the column web. This practice may be more economical and is likely to reduce heat input, weld shrinkage, and member distortion.
10. Select the web doubler plate thickness so that plug welding between the column web and web doubler plate is not required.

11. Recognize that, in the concentrated-flange-force design provisions in LRFD Specification Section K1, it is assumed that the connection is a directly welded flange or flange-plated moment connection, not an extended end-plate moment connection. Appropriate design strength equations are given in Chapter 2 based upon the recommendations in Murray (1990).
12. Limit the number of different thicknesses that are used throughout a given project for transverse stiffeners and web doubler plates. Production economy is achieved when many repetitive elements can be used.

3.4 Minimizing the Economic Impact of Column Stiffening Requirements in High-Seismic Applications

In high-seismic applications, economy suggestions 4, 5, 6,¹⁰ 9, 10,¹¹ 11, and 12 in Section 3.3 remain applicable. Additionally, economy suggestion 1 remains applicable for web doubler plates, when the flange force(s) are determined from LRFD Specification Section A4.1, AISC Seismic Provisions Section 4.1, and Equation 2.1-1.

¹⁰Applicable when a moment connection is made to one flange only.

¹¹Note that this may not be possible in high-seismic applications if the column web thickness itself does not meet the seismic shear buckling criteria given in Equation 4.4-6.

Chapter 4

STRONG-AXIS MOMENT CONNECTIONS TO STIFFENED COLUMNS

When the required strength (Section 2.1) exceeds the design strength of the column for the concentrated forces (Section 2.2), or when the stiffness of the column cross-section is inadequate to resist the bending deformations in the column flange (Section 2.3), column stiffening is required. Several common stiffening arrangements are illustrated in Figures 4-1 through 4-6 with common welding options for the attachments of the stiffening elements to the column.

In Figures 4-1 and 4-2, a column with partial-depth transverse stiffeners only and a column with full-depth transverse stiffeners only are illustrated, respectively. In Figure 4-3, a column with web doubler plate(s) only is illustrated. In Figures 4-4, 4-5, and 4-6, columns with both transverse stiffeners and web doubler plates(s) are illustrated. In Figures 4-4 and 4-5, the web doubler plate(s)

extend past the partial-depth and full-depth transverse stiffeners, respectively. In Figure 4-6, the web doubler plate(s) extend to but not past the full-depth transverse stiffeners.

As illustrated in Figures 4-4, 4-5 and 4-6 the web doubler plates that are fillet welded to the column flanges are shown thicker than those that are groove welded to the column flanges are. This is intended to visually highlight the increased thickness that is often required to facilitate the use of a fillet-welded edge detail (see Section 4.4.2).

Fillet-welded and groove-welded details are illustrated generally in all cases. Fillet-welded details will be preferable in the majority of cases although partial-joint-penetration or complete-joint-penetration groove welds may be the best choice in some cases. Ultimately, preference should be given to the use of details that require the

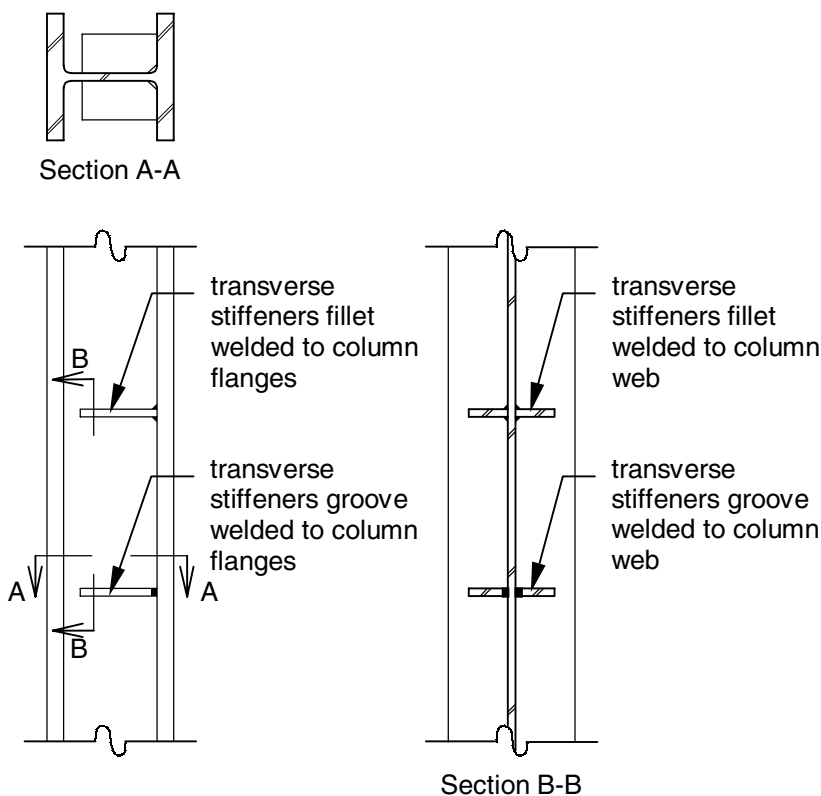


Figure 4-1 Column with partial-depth transverse stiffeners.

least amount of weld metal with due consideration of the material preparation requirements.

4.1 Determining the Column Stiffening Requirements

In wind and low-seismic applications, various alternative stiffening details utilizing transverse stiffeners, web doubler plates, or a combination thereof, are permitted in LRFD Specification Section K1, depending upon the limit state(s) for which column stiffening is required. The welding requirements are also specified for each case therein. In high-seismic applications, the required placement and welding of transverse stiffeners and web doubler plates is given in LRFD Specification Section K1 and AISC Seismic Provisions Sections 9.3c, 9.5 and 11.3. These column-stiffening requirements and alternatives are summarized in Sections 4.1.1 through 4.1.6.

4.1.1 Panel-Zone Web Shear

When the column web thickness is inadequate to resist the required panel-zone shear strength, a web doubler plate is required.¹² The welding requirements for web doubler plates are as summarized in Section 4.4.3 and 4.4.4.

4.1.2 Local Flange Bending

When the column flange thickness is inadequate to resist the tensile flange force, a pair of transverse stiffeners extending at least one-half the depth of the column web is required. They must be welded to the loaded column flange to develop the strength of the welded portion of the transverse stiffener. The weld to the column web must be sized to develop the unbalanced force in the transverse stiffener to the web.

4.1.3 Local Web Yielding

When the column web thickness is inadequate to resist the tensile or compressive flange force, either a pair of transverse stiffeners or a web doubler plate,¹³ extending at least one-half the depth of the column web is required.

In wind and low-seismic applications, when required for a tensile flange force, and in high-seismic applications, the transverse stiffener must be welded to the loaded

¹²Alternatively, diagonal stiffening can be used if it does not interfere with the weak-axis framing; see Section 5.6.

¹³See Section 5.4.

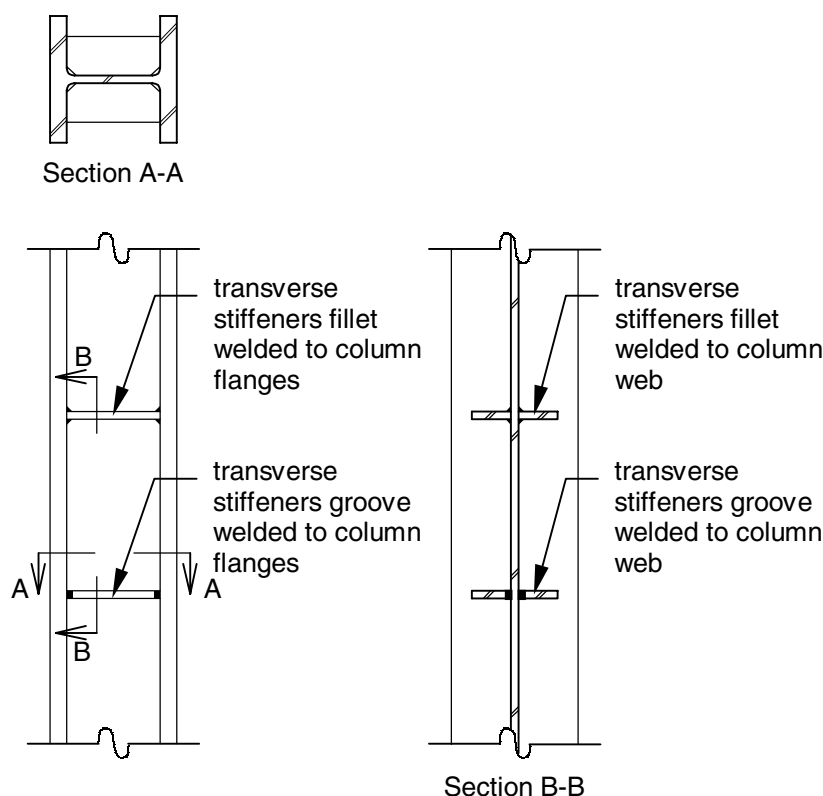


Figure 4-2 Column with full-depth transverse stiffeners.

column flange to develop the strength of the welded portion of the transverse stiffener. In wind and low-seismic applications when required for a compressive flange force, the transverse stiffener must either bear on or be welded to the loaded flange to develop the force transmitted to the transverse stiffener.

The weld to the column web must be sized to develop the unbalanced force in the transverse stiffener into the column panel-zone.

4.1.4 Web Crippling

When the column web thickness is inadequate to resist the compressive flange force, either a transverse stiffener, a pair of transverse stiffeners or a web doubler plate,¹⁴ extending at least one-half the depth of the column web, is required.

In wind and low-seismic applications, the transverse stiffener must either bear on or be welded to the loaded

flange to develop the force transmitted to the transverse stiffener. In high-seismic applications, the transverse stiffener must be welded to the loaded flange to develop the strength of the welded portion of the transverse stiffener.

The weld to the column web must be sized to develop the unbalanced force in the transverse stiffener into the column panel-zone.

4.1.5 Compression Buckling of the Web

When the column web thickness is inadequate to resist the opposing compressive flange forces, either a transverse stiffener, a pair of transverse stiffeners or a web doubler plate,¹⁵ extending the full depth of the column web, is required.

In wind and low-seismic applications, the transverse stiffener must either bear on or be welded to the loaded flange to develop the force transmitted to the transverse

¹⁴See Section 5.4.

¹⁵See Section 5.4.

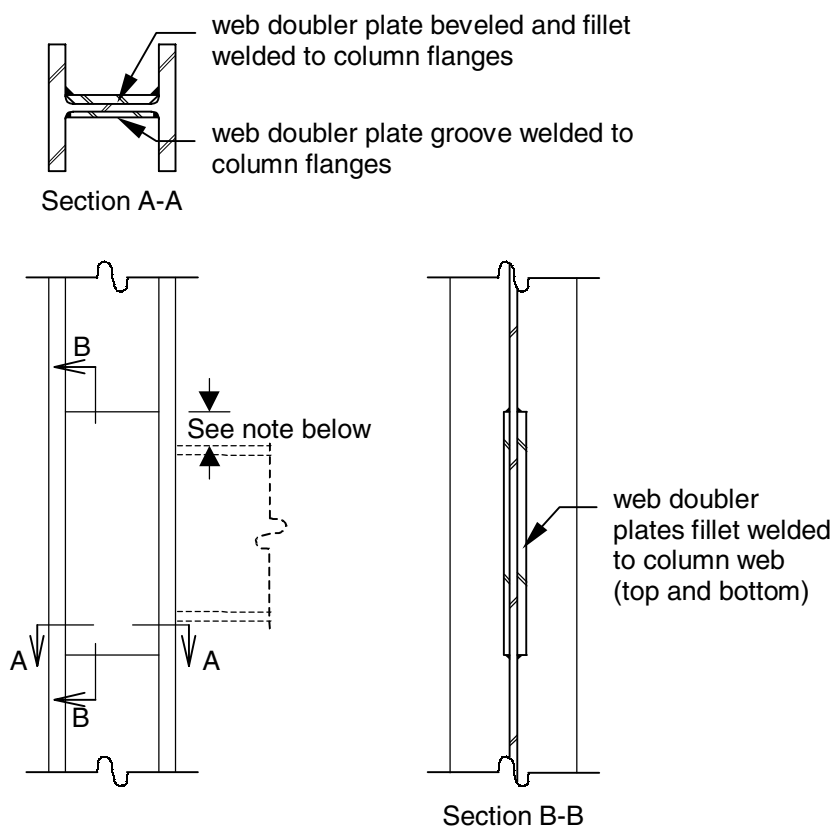


Figure 4-3 Column with web doubler plate(s).

stiffener. In high-seismic applications, the transverse stiffener must be welded to the loaded flange to develop the strength of the welded portion of the transverse stiffener.

The weld to the column web must be sized to develop the unbalanced force in the transverse stiffener into the column panel-zone.

4.1.6 Flange Stiffness

In wind and low-seismic applications, flange stiffness is addressed by the local flange bending limit state (Section 4.1.2). In high-seismic applications, transverse stiffeners will normally be required (see Section 2.3) in pairs with welding as described in Sections 4.3.4 and 4.3.5.

4.2 Force Transfer in Stiffened Columns

In a stiffened column, the load path is similar to that described in Section 2.1, except that the added stiffening elements share in a portion of the force transfer. Concentrated forces from the beam flanges or flange plates are

transferred locally into the column flanges. These concentrated forces spread through the column flange and flange-to-web fillet region into the web, transverse stiffener(s), if used, and web doubler plate(s), if used. Shear is dispersed between them in the column panel-zone. Ultimately, axial forces in the column flanges balance this shear.

4.2.1 Required Strength for Transverse Stiffeners

The following discussion is applicable to the required strength of the ends of the transverse stiffener in tension and/or compression. The required strength of the transverse stiffener in shear to transmit an unbalanced load to the column panel-zone is covered in Section 4.3.2.

In wind and low-seismic applications, transverse stiffeners are required only when the concentrated flange force (Section 2.1.1) exceeds the design strength of the column flange or web (Sections 2.2.2 through 2.2.5). In an exact solution, this force would be apportioned between the web and transverse stiffeners on the basis of relative

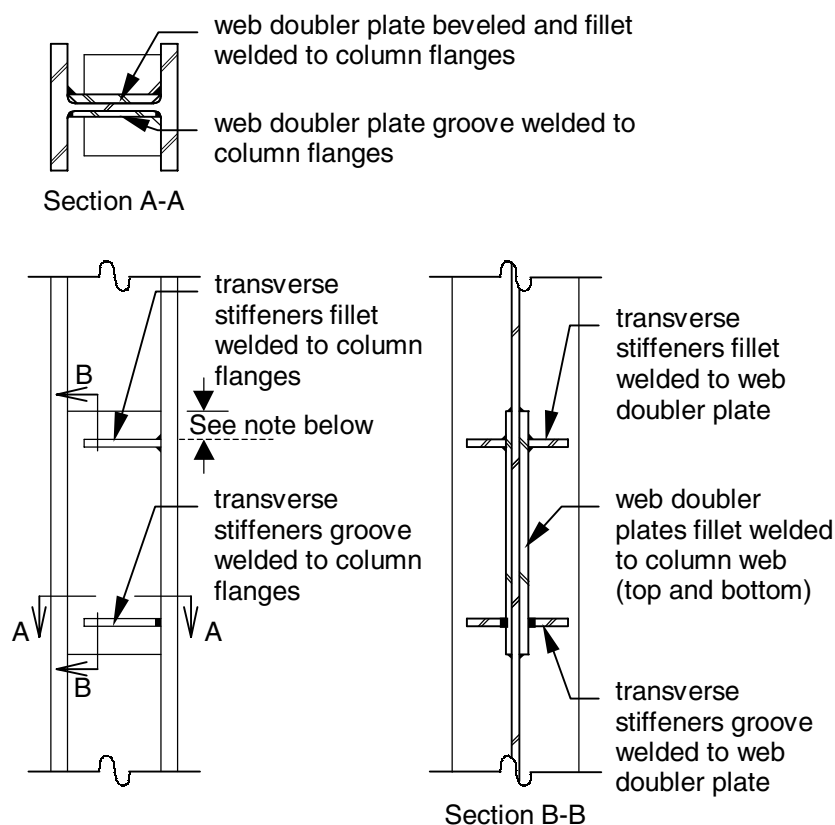


Figure 4-4 Column with partial-depth transverse stiffeners and web doubler plate(s) (extended).

stiffness and effective area. However, AISC has long allowed a simplified approach whereby only the force in excess of the governing column flange or web limit-state is assumed to be transmitted to the transverse stiffener end in tension or compression. Because minimum transverse stiffener width and thickness provisions are also included (see Sections 4.3.1 and 4.3.2), this rational method has historically provided a safe result. Accordingly, the required strength of the transverse stiffener(s) in tension and/or compression is:

$$R_{ust} = P_{uf} - \phi R_{n\min} \quad (4.2-1)$$

where

P_{uf} = factored beam flange force, tensile or compressive (Section 2.1), kips

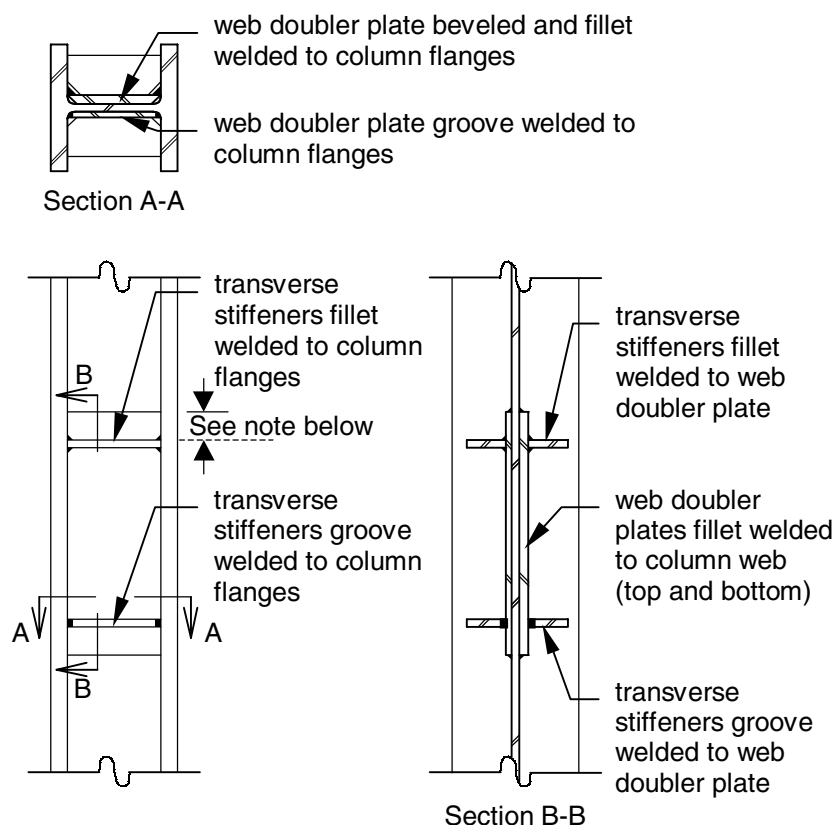
$\phi R_{n\min}$ = the lesser of the design strengths in flange bending and web yielding at locations of tensile flange forces, or the lesser of the design strengths in local web yielding, web

crippling, and compression buckling (if applicable) at locations of compressive flange forces, kips

If R_{ust} is negative, transverse stiffening is not required and its value is set equal to zero in subsequent calculations.

Note that the flange force against which each limit state must be checked may vary. For example, the compression buckling limit-state will usually be applicable for a pair of opposing compressive flange forces induced by maximum concurrent negative moments due to gravity load at a column with beams that are moment connected to both flanges. At the same time, the tensile or compressive flange forces induced by the maximum moments due to lateral loads may be more critical for the other limit-states.

In high-seismic applications, transverse stiffeners that match the configuration of those used in the qualifying cyclic tests (AISC Seismic Provisions Appendix S) for the moment connection to be used are required as discussed previously in Section 2.3.



Note: 2.5k minimum for directly welded flange and flange-plated moment connections, $3k + t_p$ minimum for extended end-plate moment connections (top and bottom)

Figure 4-5 Column with full-depth transverse stiffeners and web doubler plate(s) (extended).

4.2.2 Required Strength for Web Doubler Plates

Web doubler plate(s) are required only when the column web shear (Section 2.1.2) exceeds the design strength of the column web (Section 2.2.1). The required strength of the web doubler plate(s) is:

$$V_{udp} = V_u - \phi R_{vcw} \quad (4.2-2)$$

where

V_u = factored panel-zone shear force (Section 2.1.2), kips

ϕR_{vcw} = column web design shear strength (Section 2.2.1), kips

If V_{udp} is negative, web doubler plating is not required.

4.3 Design of Transverse Stiffeners

Transverse stiffeners are sized to provide a cross-sectional area A_{st} , where

$$A_{st \min} = \frac{R_{ust}}{\phi F_{yst}} \quad (4.3-1)$$

where

R_{ust} = transverse stiffener required strength (Section 4.2.1), kips

F_{yst} = transverse stiffener specified minimum yield strength, ksi

ϕ = 0.9

When beams are moment connected to both column flanges and share transverse stiffeners, the transverse stiffener end area is selected for the maximum individual flange force, not the combined force from both transverse stiffener ends. The combined force from both transverse stiffener ends is of interest, however, for the design of the column-web edge of the transverse stiffener and may impact the required thickness; see Section 4.3.2.

4.3.1 Width of Transverse Stiffeners

In wind and low-seismic applications, from LRFD Specification Section K1.9, the minimum width of each transverse stiffener $b_{s \min}$, as illustrated in Figure 4-7, is

$$b_{s \min} = \frac{b}{3} - \frac{t_{pz}}{2} \quad (4.3-2)$$

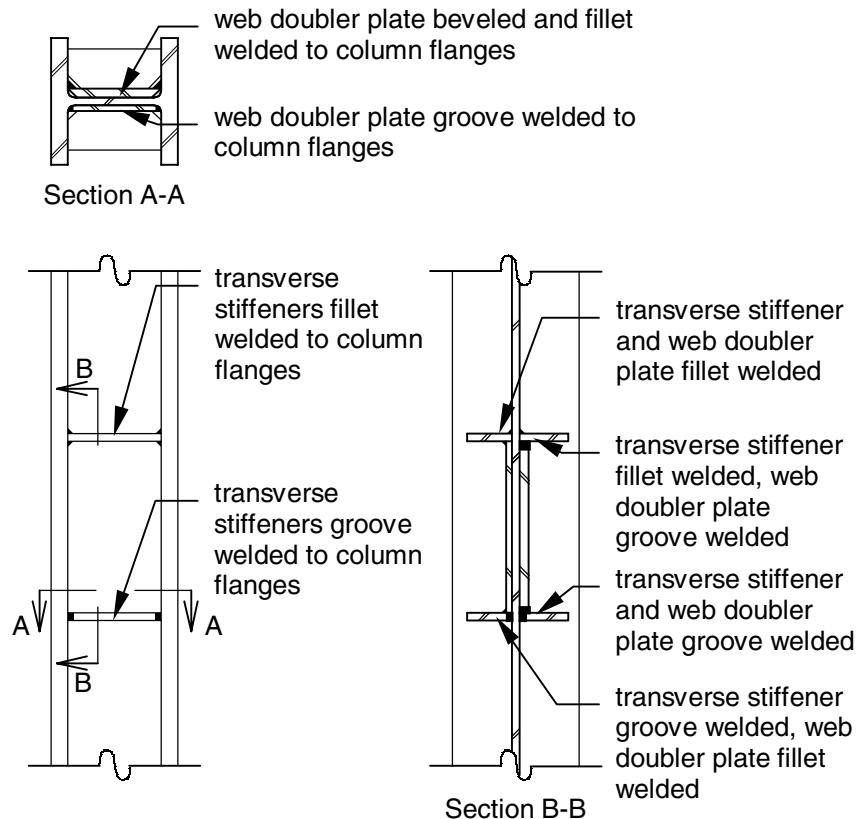


Figure 4-6 Column with full-depth transverse stiffeners and web doubler plate(s) (flush).

where

- b = width of beam flange ($= b_f$) or flange plate, in.
- t_{pz} = column web thickness, in., if a web doubler plate is not used or if the web doubler plate extends to (but not past) the transverse stiffeners; total panel-zone thickness, in., if the web doubler plate extends past the transverse stiffeners.

The specified width should be selected with consideration of the thickness requirements in Section 4.3.2, to satisfy the minimum area $A_{st \min}$ (Section 4.3). Area reduction due to corner clips that are required to clear the column flange-to-web fillets should be considered when sizing the transverse stiffener and its welds. As discussed in the AISC LRFD Manual (page 8-117) a $3/4$ -in. diagonal corner clip will generally be dimensionally adequate to clear most column flange-to-web fillets, but the clip dimension can be adjusted up or down as required to suit the various conditions.

In high-seismic applications, the width of each transverse stiffener should be consistent with that used in the tested assemblies (see Section 2.3). To date, qualifying cyclic tests have utilized transverse stiffeners of width such that the total stiffened width equals or slightly exceeds the beam flange or flange-plate width or such that

the transverse stiffeners extend to the full width of the column flange.

4.3.2 Thickness of Transverse Stiffeners

In wind and low-seismic applications, from LRFD Specification Section K1.9, the minimum thickness of each transverse stiffener $t_{s \min}$ when transverse stiffeners are required is:

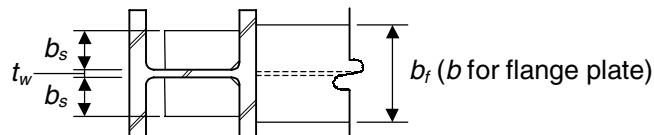
$$t_{s \min} = \frac{t}{2} \geq \frac{b_s \sqrt{F_{yst}}}{95} \quad (4.3-3)$$

where

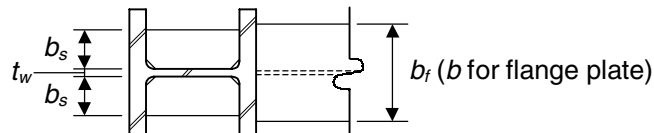
- t = beam flange or flange plate thickness, in.
- b_s = actual transverse stiffener width, in.

The specified thickness should be selected with consideration of the length requirements in Section 4.3.3, to satisfy the shear strength that is required to transmit the unbalanced force in the transverse stiffener to the column panel-zone. For a pair of partial-depth transverse stiffeners, the thickness required for shear strength is:

$$t_s \geq \frac{R_{ust}}{0.9 \times 0.6 F_{yst} (l - clip) \times 2} \quad (4.3-4)$$



(a) Partial-depth transverse stiffeners



(b) Full-depth transverse stiffeners

Note: for flange-plated moment connections, use the flange-plate width b in place of the beam-flange width b_f

Figure 4-7 Illustration of transverse stiffener width b_s (wind and low-seismic applications).

For a pair of full-depth transverse stiffeners, the thickness required for shear strength is:

$$t_s \geq \frac{(R_{ust})_1 + (R_{ust})_2}{0.9 \times 0.6F_{yst}(l - 2 \times clip) \times 2} \quad (4.3-5)$$

where

R_{ust} = required strength of the transverse stiffener (see Section 4.2.1), kips; the subscripts 1 and 2 in Equation 4.3-5 indicate the forces at each end of the transverse stiffener

F_{yst} = transverse stiffener specified minimum yield strength, ksi

l = transverse stiffener length, in.

$clip$ = transverse stiffener corner clip dimension, in.

In Equation 4.3-5, $(R_{ust})_1$ and $(R_{ust})_2$ can add, as for lateral moments, or subtract, as for gravity moments. The most critical case for transverse stiffener thickness will usually result for the case wherein they add.

In high-seismic applications, the thickness of each transverse stiffener should be consistent with that used in the tested assemblies. To date, most qualifying cyclic tests have utilized transverse stiffeners of thickness equal to that of the beam flange or flange plate to meet the recommendation of FEMA (1995).¹⁶

4.3.3 Length of Transverse Stiffeners

When full-depth transverse stiffeners are used, the length is selected for the distance between the column flanges, with due consideration of column cross-sectional tolerances and the welded joint that is to be used. When partial-depth transverse stiffeners are used, the length is selected to minimize the transverse stiffener thickness and, more importantly, the size of double-sided fillet weld that is required for the connection of the transverse stiffener to the column web; see Sections 4.3.2 and 4.3.5. Note that the minimum length for partial-depth transverse stiffeners is one-half the column depth.

4.3.4 Connecting Transverse Stiffeners to Column Flanges

In wind and low-seismic applications, when the transverse stiffener is required for a tensile flange force (due to local web yielding or local flange bending), it must be welded to develop the strength of the welded portion of the transverse stiffener. As illustrated in Figure 4-8, this can be done with double-sided fillet welds, double-sided partial-

joint penetration groove welds with fillet-weld reinforcement, or complete-joint-penetration groove welds. When using double-sided fillet welds, the weld size required is:

$$w_{\min} = \frac{0.9F_{yst}t_s}{0.75(1.5 \times 0.6F_{EXX})\sqrt{2}} = \frac{0.943F_{yst}t_s}{F_{EXX}} \quad (4.3-6)$$

where

F_{yst} = transverse stiffener specified minimum yield strength, ksi

t_s = transverse stiffener thickness, in.

F_{EXX} = welding electrode specified minimum strength, ksi

The 1.5 factor in the denominator of the second term in Equation 4.3-6 is the weld strength increase factor for the 90-degree angle of loading determined from LRFD Specification Appendix J2.4.

When the transverse stiffener is required for a compressive flange force only (due to local web yielding, web crippling, or compression buckling of the web), it must either bear on or be welded to the column flange to develop the force transmitted to the transverse stiffener. For proper force transfer in bearing, R_{ust} must be equal to or less than ϕR_n as given in LRFD Specification Section J8(a). From this section, it can be derived that, for a pair of transverse stiffeners, the width b_s and thickness t_s of each of the transverse stiffeners must be such that:

$$(b_s - clip)t_s \geq \frac{0.370R_{ust}}{F_{yst}} \quad (4.3-7)$$

Alternatively, when using double-sided fillet welds, the weld size required is:

$$\begin{aligned} w_{\min} &= \frac{R_{ust}}{0.75(1.5 \times 0.6F_{EXX})(b_s - clip)(2)\sqrt{2}} \\ &= \frac{0.524R_{ust}}{F_{EXX}(b_s - clip)} \end{aligned} \quad (4.3-8)$$

where

$clip$ = transverse stiffener corner clip dimension, in.

R_{ust} = transverse stiffener required strength (see Section 4.2.1), kips

F_{yst} = transverse stiffener specified minimum yield, ksi

F_{EXX} = welding electrode specified minimum strength, ksi

The 1.5 factor in the denominator of the second term in Equation 4.3-8 is the weld strength increase factor for the 90-degree angle of loading determined from LRFD Specification Appendix J2.4.

¹⁶Subsequent research (El Tawil et al., 1998) indicates that transverse stiffness with thickness equal to or greater than 60 percent of the beam flange or flange-plate thickness can provide for the required cross-sectional stiffness when a beam is moment-connected to one column flange only.

In high-seismic applications, the transverse stiffener must be welded to develop the strength of the welded portion of the transverse stiffener. As illustrated in Figure 4-8, this can be done with double-sided fillet welds, double-sided partial-joint penetration groove welds with fillet-weld reinforcement, or complete-joint-penetration groove welds. When using double-sided fillet welds, the weld size required can be determined as given previously in Equation 4.3-6.

4.3.5 Connecting Transverse Stiffeners to Column Panel-Zones

In wind, low-seismic and high-seismic applications, the transverse stiffener is welded to transmit the unbalanced force, if any, in the transverse stiffener to the column panel-zone. As illustrated in Figure 4-9b, welding to the column panel-zone is not required if the opposing beam flange forces are equal and opposite, except when compression buckling of the web governs or to stabilize the

transverse stiffeners.¹⁷ Welding to the column panel-zone will always be required when:

1. Partial-depth transverse stiffeners are used (see Figure 4-9a);
2. A beam is moment-connected to one flange of the column only; or,
3. Beams are moment-connected to both column flanges and reverse-curvature bending is anticipated (see Figure 4-9c).

The latter case is common for moment connections, especially in high-seismic applications, and results in a tensile force on one end of the transverse stiffener combined with a compressive force on the other end of the transverse stiffener. The sum of these forces is equilibrated by shear

¹⁷In such cases, minimum-size fillet welds per LRFD Specification Table J2.4 are commonly used.

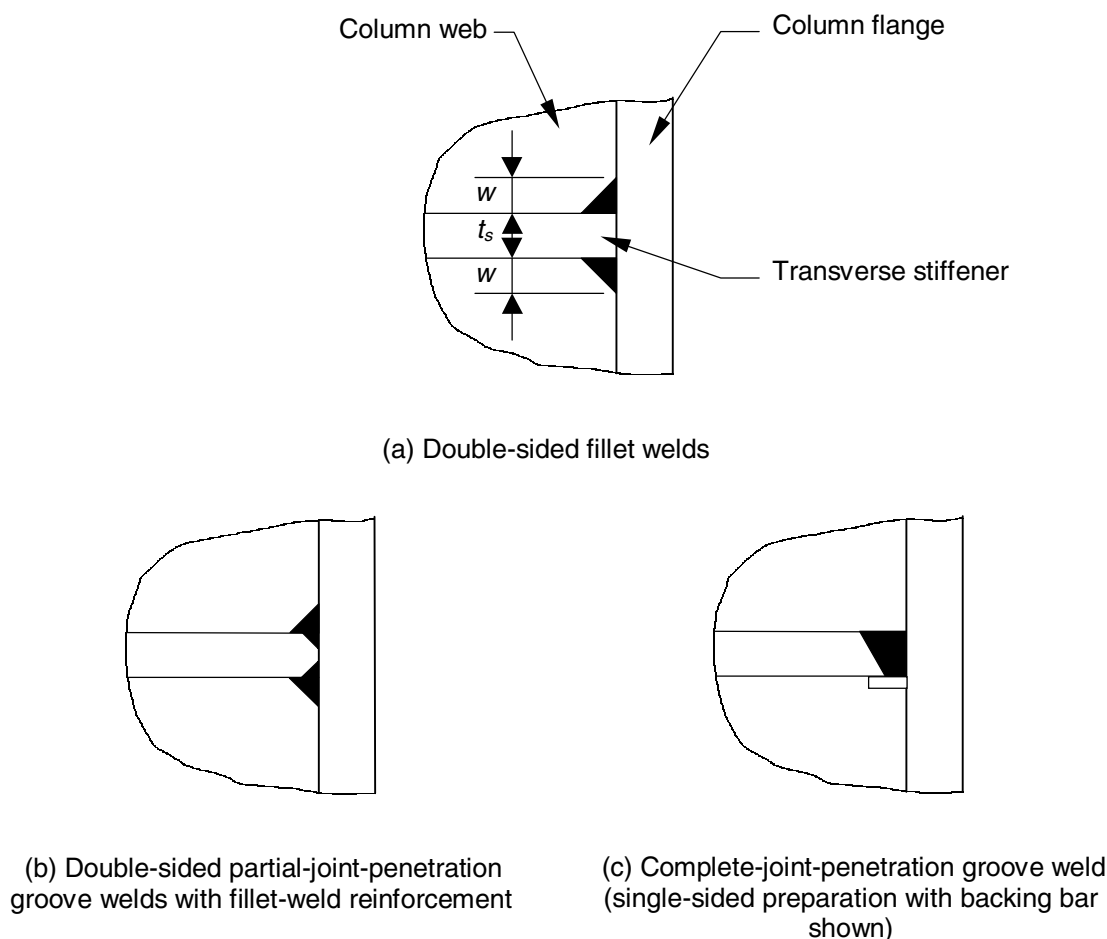


Figure 4-8 Welded joint details for transverse stiffener ends (welding to column flange).

that is distributed along the column-web edge of the transverse stiffener as illustrated in Figure 4-10.

For a pair of partial-depth transverse stiffeners, the fillet-weld size required for shear strength (with double-sided fillet welds on each transverse stiffener) is:

$$w \geq \frac{R_{ust}}{0.75 \times 0.6F_{EXX}(l - clip) \times 2 \times \sqrt{2}} \quad (4.3-9)$$

For a pair of full-depth transverse stiffeners, the fillet-weld size required for shear strength (with double-sided fillet welds on each transverse stiffener) is:

$$w \geq \frac{(R_{ust})_1 + (R_{ust})_2}{0.75 \times 0.6F_{EXX}(l - 2 \times clip) \times 2 \times \sqrt{2}} \quad (4.3-10)$$

where

R_{ust} = transverse stiffener required strength (see Section 4.2.1), kips; the subscripts 1 and 2 in Equation 4.3-10 indicate the forces at each end of the transverse stiffener as illustrated in Figure 4-10

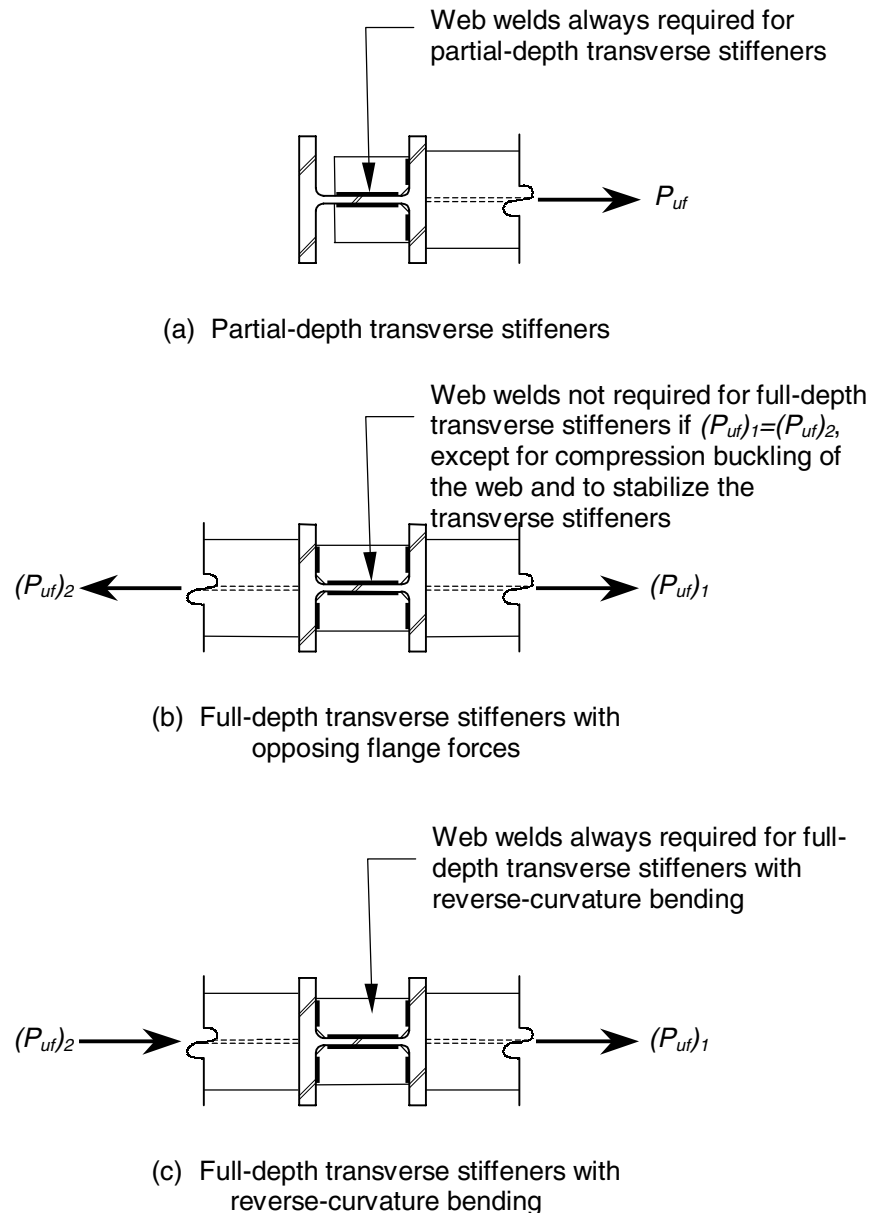


Figure 4-9 Web welding requirements for transverse stiffeners.

F_{EXX} = weld electrode specified minimum strength, ksi

l = transverse stiffener length, in.

$clip$ = transverse stiffener corner clip dimension, in.

In Equation 4.3-10, $(R_{ust})_1$ and $(R_{ust})_2$ can add, as for lateral moments, or subtract, as for gravity moments. The most critical case for weld size will usually result for the case wherein they add. However, the welds need not be sized to develop a force that is larger than that due to any of the following criteria:

1. The sum of the design strengths at the connections of the transverse stiffener to the column flanges (see Equations 4.3-11 and 4.3-14 or 4.3-17);
2. The design shear strength of the contact area of the transverse stiffener with the column panel-zone (see Equations 4.3-12 and 4.3-15); nor
3. The shear yield strength of the column panel-zone (see Equations 4.3-13 and 4.3-16).

Note that the second and third criteria should not govern unless the transverse stiffener was provided for stiffness rather than strength.

Thus, for a pair of partial-depth transverse stiffeners, the design shear strength of the welds ϕR_n need not exceed any of the following three forces:

$$\phi R_{n \max} = 0.9F_{yst}(2)(b_s - clip) \times t_s \quad (4.3-11)$$

$$\phi R_{n \max} = 0.9 \times 0.6F_{yst}(l - clip) \times 2t_s \quad (4.3-12)$$

$$\phi R_{n \max} = 0.9 \times 0.6F_y d_c \times t_{pz} \quad (4.3-13)$$

Similarly, for a pair of full-depth transverse stiffeners, the design shear strength of the welds ϕR_n need not exceed any of the following three forces:

$$\phi R_{n \max} = 0.9F_{yst}(4)(b_s - clip) \times t_s \quad (4.3-14)$$

$$\phi R_{n \max} = 0.9 \times 0.6F_{yst}(l - 2 \times clip) \times 2t_s \quad (4.3-15)$$

$$\phi R_{n \max} = 0.9 \times 0.6F_y d_c \times t_{pz} \quad (4.3-16)$$

where

$\phi R_{n \max}$ = maximum force for which the welds connecting the transverse stiffeners to the column panel-zone must be designed, kips

F_{yst} = transverse stiffener specified minimum yield strength, ksi

b_s = transverse stiffener width, in.

$clip$ = transverse stiffener corner clip dimension, in.

t_s = transverse stiffener thickness, in.

l = transverse stiffener length, in.

F_y = panel-zone specified minimum yield strength (column web and/or web doubler plate), in.

t_{pz} = panel-zone material thickness (column web and/or web doubler plate), in.

Note that, if a pair of full-depth transverse stiffeners is used, but a beam is moment connected to one column flange only, Equation 4.3-17 should be used in lieu of Equation 4.3-14, where:

$$\phi R_{n \max} = 0.9F_{yst}(2)(b_s - clip) \times t_s \quad (4.3-17)$$

When transverse stiffeners transmit an unbalanced load to both the column web and the web doubler plate simultaneously, the welded detail must be configured for proper force transfer from the transverse stiffener to the column web and web doubler plate. See Section 4.4.4.

4.4 Design of Web Doubler Plates

4.4.1 Width and Depth of Web Doubler Plates

In wind, low-seismic and high-seismic applications, the width and depth of web doubler plates are selected based upon the dimensions of the panel-zone, with due consideration of the details to be used to connect the web doubler plate.

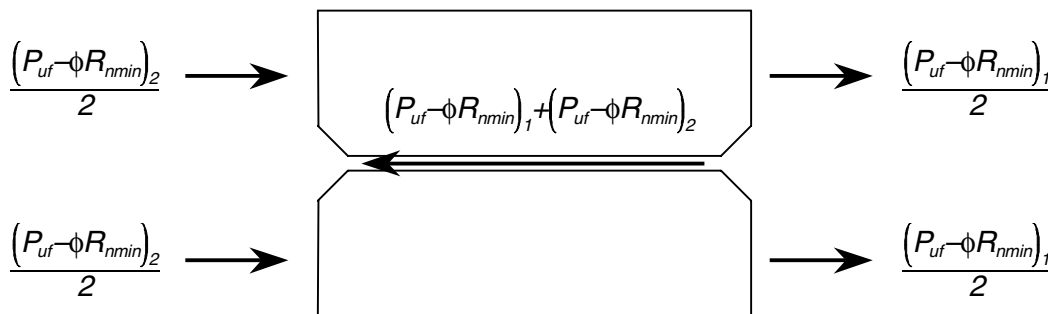


Figure 4-10 Force transfer for transverse stiffeners (reverse curvature moment case).

If full-depth transverse stiffeners are present, the web doubler plate(s) can be extended to the transverse stiffeners and one of the weld details illustrated in Figures 4-11 and 4-12 can be used. Alternatively, the web doubler plate may be extended past the transverse stiffener to clear the zone of the column web subject to crippling and buckling. As a minimum, this distance is 2.5 times the column k -distance for a directly welded flange or flange-plated moment connection and 3 times the column k -distance plus the end-plate plate thickness for an extended end-plate moment connection. The choice between these stiffening alternatives should be an economic one made by the fabricator with the approval of the Engineer of Record. Extending the web doubler plate past the transverse stiffener may be desirable in some cases because the top and bottom edges of the web doubler plate can be square-cut and the corners of transverse stiffeners may not need to be clipped to clear the column flange-to-web fillets.¹⁸ Additionally,

¹⁸A corner clip may still be desirable to separate and simplify the welds on the ends and edge of the transverse stiffener.

this detail may be preferable when partial-depth transverse stiffeners are used. However, if a web doubler plate is extended beyond the transverse stiffener, its thickness must be sufficient to transmit the full unbalanced force in the transverse stiffener, if any, into the panel-zone.

If transverse stiffeners are not present, the web doubler plate should extend beyond the beam flange or moment-connection flange plate to clear the zone of the column web subject to crippling and buckling. As a minimum, this distance is 2.5 times the column k -distance for a directly welded flange or flange-plated moment connection and 3 times the column k -distance plus the end-plate plate thickness for an extended end-plate moment connection.

4.4.2 Thickness of Web Doubler Plates

The web doubler plate thickness is selected to provide that required in excess of the column web thickness to resist panel-zone web shear. For strength, the required web doubler plate thickness is

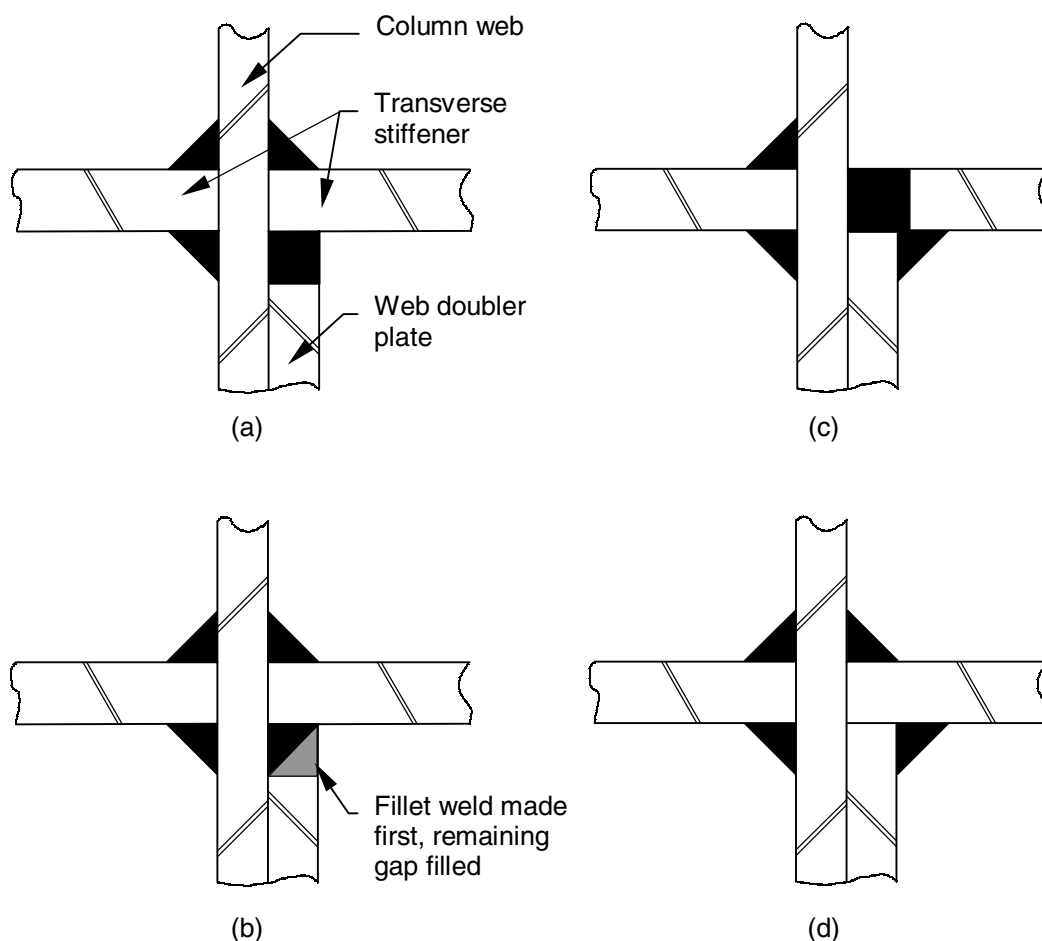


Figure 4-11 Common welded joint details at top and bottom edges with one web doubler plate and a pair of transverse stiffeners.