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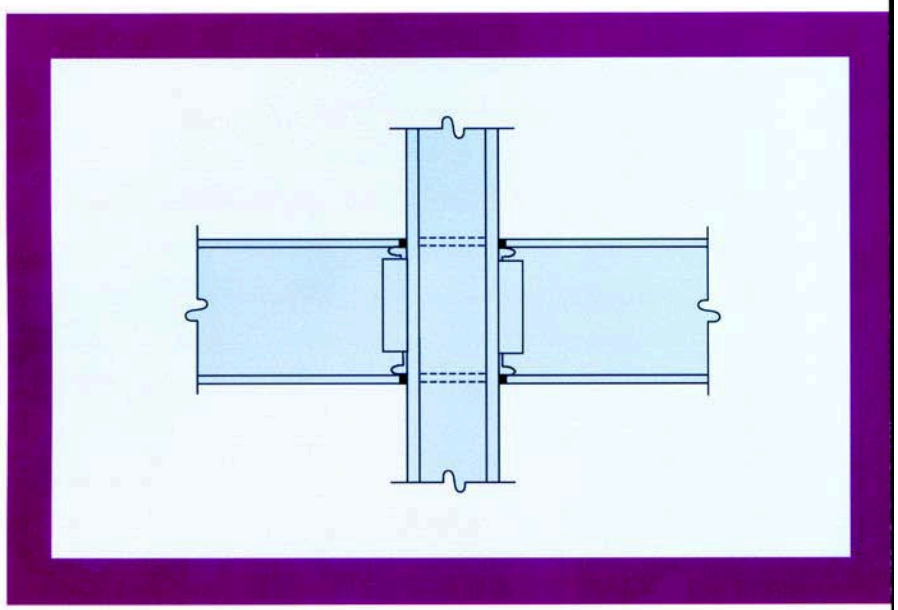
Steel Design Guide Series

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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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

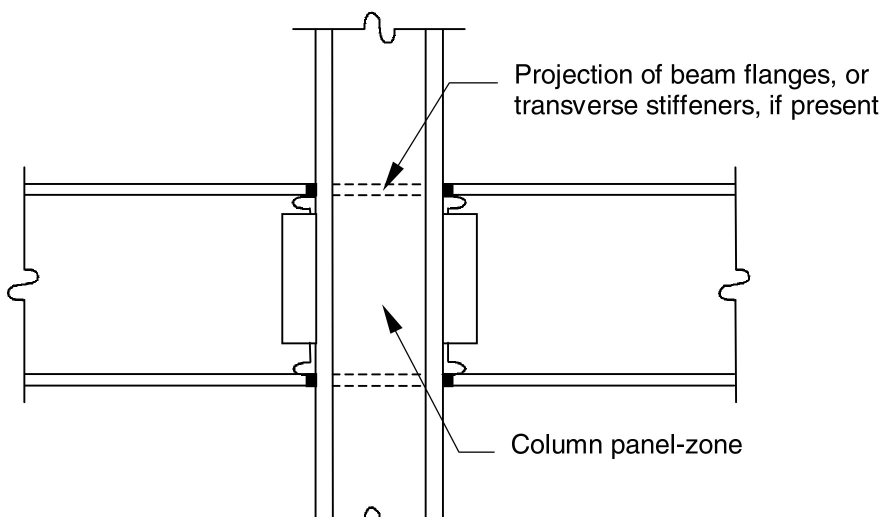


Figure 1-1 Illustration of column panel-zone.

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.

[1]From 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:

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

where

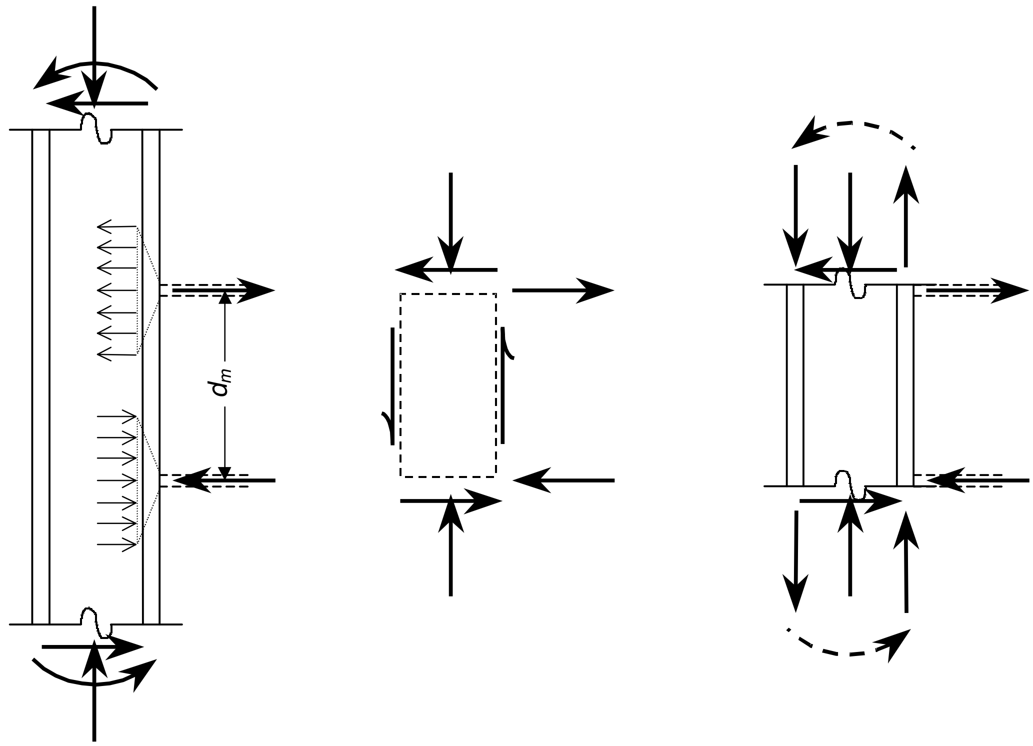
P_{uf} = factored beam flange force, tensile or compressive, kips
M_u = factored beam end moment, kip-in.
d_m = moment arm between the flange forces,[2] in.
P_u = factored beam axial force, kips

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

[2] The 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.



(a) Beam flange forces distributed through column flange and fillet

(b) Free-body diagram illustrating shear and axial force transfer through column panel-zone

(c) Free-body diagram illustrating resulting column axial forces and flange forces (moments)

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 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:

$$P_{uf} = \frac{M_u}{d_m} = \frac{1.1R_yF_yZ + V_ua}{d_m} \qquad (2.1\text{-}2)$$

[3]With 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.

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:

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

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:

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

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:

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

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-

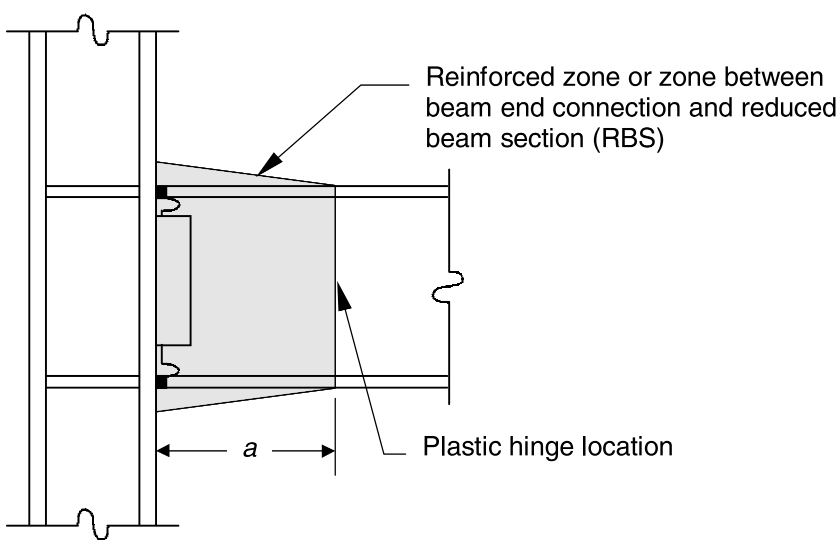


Figure 2-2 Schematic illustration of moment connection for high-seismic applications.

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:

For $P_u \le 0.4P_y$, $\quad \phi R_v = 0.9 \times 0.6 F_y d_c t_w \qquad$ (2.2-1)

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

(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:

For $P_u \le 0.75P_y$,

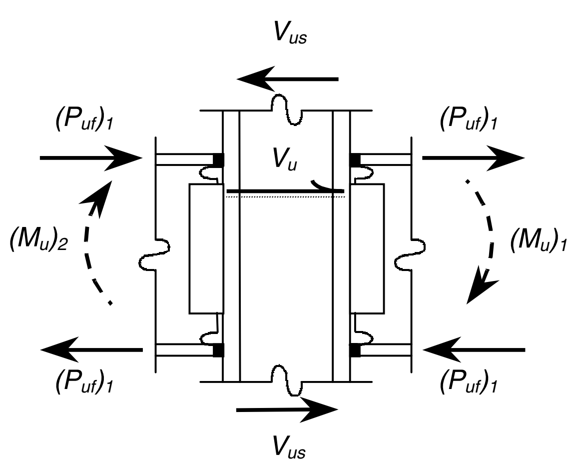
$$\phi R_v = 0.9 \times 0.6 F_y d_c t_w \left(1 + \frac{3 b_f t_f^2}{d_b d_c t_w}\right) \qquad (2.2\text{-}3)$$

For $P_u > 0.75P_y$,

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

(2.2-4)

For F_y equal to or less than 50 ksi, all W-shapes listed in ASTM A6 except a W30 × 90 and a W16 × 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).