This is an unedited, uncorrected chapter.

The final chapter will be available in time for fall.

NOTE: Figures and tables appear at the end of the chapter.

WEB CHAPTER 13

Connections

W13.1 Column Stiffening at Moment Connections

Figure W13.1.1: Beam-to-column moment connection terminology.

From the description of beam-to-column connections given in Sections 13.11, 12 and 13, it is evident that the beam flanges or flange connection plates apply concentrated forces to the column, and cause stresses in the flanges and web of the column. It is the transfer of these essentially concentrated forces to the column flange, for which provision is made in LRFDS Chapter K, that forms the subject of this section. Moment connections are called ***double-concentrated forces*** because there is one tensile flange force and one compressive flange force that form a couple acting on the same side of the column (Fig. W13.1.1*a*). When opposing connected beams coincide, a ***pair of double concentrated forces*** results, as shown in Fig. W13.1.1*b*.

The web of a column within the boundary of the column flanges, and the tensile and compressive concentrated forces imposed by moment connections is called the ***panel-zone*** (Fig. W13.1.1*c*). If the moments in two beams, connected to opposite flanges of an interior column, differ significantly in magnitude (or, when a one sided beam-to-column connection is encountered), large shear forces may develop in the column web within the panel-zone. The panel-zone undergoes both shear and moment deformations that are generally responsible for a major portion (up to 50%) of the total joint rotation of tier buildings under lateral loads even when stresses remain in the elastic domain.

The basic requirement that must be satisfied by any connection are related to its strength, stability and deformations. On the basis of these three requirements the provisions for restrained members subjected to concentrated flange forces, given in Chapter K of the LRFD Specification can be categorized as:

- Strength requirement: Section K1.3. Web local yielding
- Stability requirements: Section K1.4. Web crippling
  Section K1.6. Web compression buckling
- Deformation requirement: Section K1.2. Flange local bending

In addition, the column web safety is to be checked under shear forces that develop in the panel-zone (Section K1.7).

The general LRFDS requirement for this loading type may be written as :

$$P_d \geq P_u \tag{W13.1.1}$$

where P_u = factored concentrated load applied on the column flange (tension or compression)

P_d = design strength (= ϕP_n)

P_n = nominal resistance

ϕ = resistance factor corresponding to P_n

LRFD moment connections are designed and detailed subsequent to the design analysis and member selection, based upon the LRFD Specifications. That is, for particular beam and column selected, Eq. W13.1.1 is verified for different limit states enumerated earlier. If the relation is not satisfied, the members or the connection have to be strengthened.

W13.1.1 Web Local Yielding

Figure W13.1.2: Web local yielding.

When a heavy concentrated load is applied to the column-flange by a beam-flange or a flange-connection-plate, the load is transmitted through the relatively wide column-flange into the relatively narrow column-web (from b_{fb} or b_{fp} to t_{wc}). The critical section is considered to be at the web toe of the fillet of the column section (Fig. W13.1.2). This is located at the k-distance from the back of the flange, as listed in the dimension tables in Table 1-1 of the LRFD Manual. At low load levels the behavior is elastic. The intensity of stress in the column web will be a maximum opposite the point of application of load and this stress concentration diffuses and diminishes progressively at points away from the location of the load. It is assumed that, due to the ductility of steel, when the maximum strength of the column web is reached, the load is distributed uniformly over a length of the web contained by lines radiating at a gradient of 2½:1. The stress intensity is taken as the yield stress of steel. Thus, for equilibrium

$$P_{n,\,\mathrm{WY}} = (5k_c + N)\, t_{wc} F_{yc} \tag{W13.1.2}$$

where N = length over which the concentrated load is applied

k_c = distance between outer face of column flange and web toe of its fillet, if column is a rolled shape, or equivalent distance if column is a welded shape, in.

t_{wc} = thickness of the column web, in.

F_{yc} = yield stress of the column web, ksi

$P_{n,\,\mathrm{WY}}$ = nominal strength of the column web corresponding to the limit state of web local yielding, kips

N is taken as the thickness of the beam flange (t_{fb}) or the flange connection plate (t_{fp}) delivering concentrated force, as the case may be. Equation W13.1.2 is the relation K1-2 of the LRFD Specification. The resistance factor ϕ for this limit state is specified as, 1.0. The design strength of the column web under a concentrated (tensile or compressive) force applied to the column flange is therefore given by

$$P_{d,\,\mathrm{WY}} \equiv \phi P_{n,\,\mathrm{WY}} = 1.0\,(5k_c + N)\, t_{wc} F_{yc} = P_{wo} + NP_{wi} \tag{W13.1.3}$$

with

$$P_{wo} = 5\, k_c\, t_{wc} F_{yc} \tag{W13.1.4}$$

$$P_{wi} = t_{wc} F_{yc} \tag{W13.1.5}$$

where N = t_{fb} for directly welded moment connections

	=	t_{fp} for flange plated moment connections
$P_{d,\,\mathrm{WY}}$	=	design strength of the column web corresponding to the limit state of web local yielding, kips
t_{fb}	=	thickness of the beam flange delivering the concentrated force
t_{fp}	=	thickness of the flange plate delivering the concentrated force

The values of P_{wo} and P_{wi} for the W-shapes normally used as columns have been recorded in the Properties section of the Column Load Tables (LRFDM Table 4-2) for steels with $F_y = 50$ ksi.

When the concentrated (tensile or compressive) flange force is applied at a distance from the column end (column top) which is less than or equal to the depth of the column, d_c, the design strength of the column web is reduced to

$$P_{d,\,\mathrm{WY}} = \phi\,(2.5\,k_c + N)\,t_{wc}\,F_{yc} \qquad \text{(W13.1.6)}$$

This criterion for design is based on the compression force from the beam flange (which is usually controlling) but is applicable to the tension flange as well. Occasionally, the tension flange is controlling and must be checked.

If the thickness of the column web t_{wc} is such that design strength $P_{d,\mathrm{WY}}$ is less than the required strength P_u, column web stiffening is required.

W13.1.2 Web Crippling

Figure W13.1.3: Loading and mechanism for web crippling.

Unstiffened portions of webs of members under concentrated compressive loads may collapse by ***web crippling*** by forming a local plastic mechanism involving the flange on which the load is applied and the immediate vicinity of the web. An experimental and theoretical investigation of the crippling load of slender plate girders, subjected to localized edge loading as shown in Fig. W13.1.3, was presented by Roberts [1971]. The simply supported girders of span L were loaded by a central load P, distributed uniformly over a small distance N. The thickness of the web, t_{wc}; thickness of the flange t_{fc}; depth of the girder d_c; and yield stress of steel F_{yc} are the parameters considered in that study. The assumed collapse mechanism considered by Roberts is shown in Fig. W13.1.3*b*. Dimensions α and β define the position of the assumed yield lines in the web and plastic hinges in the flange, and θ defines the deformation of the web just before collapse. Let M_w be the plastic moment per unit width of the web, M_f the plastic moment of the flange, and F_{yc} the yield stress of steel. If the applied load moves vertically through a small distance δ_v, the rotation of the plastic hinges in the flange is δ_v/β and the rotation of the yield lines in the web is $\delta_v/(2\alpha\cos\theta)$, and twice this value along the central yield line. Equating internal and external work gives:

$$P_{pl} = \frac{4M_{fc}}{\beta} + \frac{4\beta M_{wc}}{\alpha\cos\theta} - \frac{2\eta M_{wc}}{\alpha\cos\theta} + \frac{2NM_{wc}}{\alpha\cos\theta} \qquad \text{(W13.1.7)}$$

where P_{pl} is the plastic collapse load and η defines a length of web beneath the load which is assumed to have yielded owing to the presence of compressive membrane stresses, and therefore offers no resistance to bending. Experimental evidence suggested that for slender plate girders

P_{pl} is independent of the depth, and α can be taken as $25t_{wc}$. Minimizing P_{pl} with respect to β and introducing several simplifying assumptions, results in the following expression for the collapse load:

$$P_{\text{pl}} \quad = \quad 0.5\,t_{wc}^2 \left[EF_{yc}\,\frac{t_{fc}}{t_{wc}} \right]^{\frac{1}{2}} \left[1 \; + \; 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{\frac{3}{2}} \right] \qquad \text{(W13.1.8)}$$

Based on this study, the LRFD Specification stipulates that for unstiffened portions of webs of members under concentrated loads, the web crippling strength, when the concentrated load is applied at a distance not less than ½ d_c from the end of the member is,

$$P_{n,\text{WC}} \quad = \quad 0.80\,t_{wc}^2 \left[1 \; + \; 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5} \right] \sqrt{EF_{yc}\,\frac{t_{fc}}{t_{wc}}} \qquad \text{(W13.1.9)}$$

which is Eq. K1-4 of the LRFD Specifications. With the resistance factor ϕ taken as 0.75, the design crippling strength of the column web may be written as:

$$P_{d,\text{WC}} \quad = \quad \phi P_{n,\text{WC}} \quad = \quad \phi(0.80)\,t_{wc}^2 \left[1 \; + \; 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5} \right] \sqrt{EF_{yc}\,\frac{t_{fc}}{t_{wc}}} \qquad \text{(W13.1.10)}$$

where $P_{d,\text{WC}}$ = design strength of the column web corresponding to the limit state of web crippling, kips

ϕ = resistance factor = 0.75

N = length over which the concentrated compressive force is applied, in.

t_{fc} = thickness of the flange of the column, in.

t_{wc} = thickness of the web of the column, in.

d_c = depth of the column, in.

It is recommended that the value of N/d_c used in Eq. W13.1.10 be limited to 0.2. If the thickness of the column web t_{wc} is such that the design strength $P_{d,\mathrm{WC}}$ is less than the required strength P_u, column web stiffeners are required.

For the rolled W-shapes listed in Table 1-1 of the LRFDM, with F_y equal to 50 ksi, the web crippling limit state will never control the design in an FR or PR moment connection except to a W12×50 or W10×33 column. Also, as the overstress for these two column shapes is less than 3 percent, it is considered (negligible) acceptable. Therefore, the limit state of web crippling is not considered further.

W13.1.3 Web Compression Buckling

Figure W13.1.4: Web compression buckling.

The limit state of ***web compression buckling*** applies to the web of a member when a pair of compressive single concentrated forces or the compressive components in a pair of double concentrated forces are applied at both flanges of a member at the same location. The mathematical model for this buckling failure is based on a rectangular plate of length, *a;* width, *h;* thickness, *t;* simply supported along the four edges; and subjected to a concentrated compressive load *P* at mid length of each of two opposite sides, as shown in Fig. W13.1.4*a*. The elastic buckling strength for such a simply supported plate, with a large aspect ratio, *a/h*, is given by Timoshenko and Gere [1961] as:

$$P_{cr} = \frac{4\pi E t^3}{12(1 - \mu^2)h} \tag{W13.1.11}$$

This relation is applicable to the web of a column (Fig. W13.1.4b), using $h = h_c$, the column web depth clear of fillets; $t = t_{wc}$, thickness of the column web; E, the modulus of elasticity (= 29,000 ksi for steel); and μ, the Poissons's ratio (= 0.3 for steel). If the rotational restraint along the loaded edges were to be a fully fixed condition; the buckling strength would be theoretically twice as great as that given by Eq. W13.1.11. Chen and Newlin [1973] reported that the column flanges provide essentially simple support to the webs of shapes of A36 steel because of the yielding near the junction of the web and flange. It was also observed that this local yielding does not spread throughout the compression panel until just prior to ultimate load when the panel begins to buckle. With shapes of high strength steels this local yielding did not occur prior to ultimate load, and the column flanges provide essentially fixed end supports for the web pane. Thus Chen and Newlin suggested that the increase in effective degree of fixity at the loaded edge may be accounted for in design by making the strength proportional to the square root of the yield stress. Their stability criterion can therefore be written as

$$P_{cr} = 1.15 \frac{E t_{wc}^3}{h_c} \sqrt{\frac{F_{yc}}{36}} = 32.64 \frac{t_{wc}^2}{(h_c / t_{wc})} \sqrt{E F_{yc}} \tag{W13.1.12}$$

Adjusting this semi-rational approach downward to represent a lower bound for all test results gives the LRFD requirement that

$$P_{n,\mathrm{WB}} = 24 \frac{t_{wc}^2}{(h_c / t_{wc})} \sqrt{E F_{yc}} \tag{W13.1.13}$$

This is Eq. K1-8 of the LRFDS. The resistance factor ϕ for this limit state is to be taken as 0.9. Thus, for unstiffened portions of webs of members under concentrated loads to both flanges, the design compressive strength is

$$P_{d,\mathrm{WB}} \equiv \phi P_{n,\mathrm{WB}} = 24\,\phi \frac{t_{wc}^2}{(h_c/t_{wc})}\sqrt{EF_{yc}} \equiv P_{wb} \qquad \text{(W13.1.14)}$$

where $P_{d,\mathrm{WB}}$ = design strength of the column web corresponding to the limit state of web compression buckling, kips

ϕ = resistance factor = 0.9

h_c = column web depth clear of fillets, in.

This equation is applicable to pairs of concentrated compression forces for which N/d_c is small (< 1). When N/d_c is not small, the member web should be designed as a compression member, in accordance with LRFDS Chapter E. Again, if the thickness of the column web t_{wc} is such that the design compressive strength $P_{d,\mathrm{WB}}$ is less than the required strength P_u, either a pair of transverse stiffeners (one on each side of the column web), or a doubler plate must be provided and must extend the full depth of the column web. When the pair of concentrated compressive forces to be resisted is applied at a distance from the column end that is less than ½ d_c, the design strength must be reduced by 50 percent.

W13.1.4 Flange Local Bending

Figure W13.1.5: Flange local bending.

Figure W13.1.6: Plate bending mechanism under tensile line load.

The tension force from the beam flange (or, flange connection plate), may produce excessive bending deformation of the column flange, and cause over stressing of the weld joining the flange (or, flange plate) to the column flange, directly in line with the column web. This flange local bending occurs in both longitudinal and transverse directions to the axis of the column as shown in Fig. W13.1.5. An approximate yield line analysis can be made by assuming the beam flange force to be resisted by the bending resistance of the outstanding portions of the column flange, and by the direct resistance of the thick center portion of the column flange. The following development is adapted from Graham, Sherbourne and Jensen [1959]. The column flange is considered acting as two rectangular plates (indicated as ABCD in Fig. W13.1.6). The beam flange is assumed to apply a line load on each of these two plates. The effective length of the plates, s, is assumed to be $12t_{fc}$ and they are assumed to be fixed at the ends of this length. The plate is also assumed to be fixed adjacent to the column web. Let

$$m = t_{wc} + (k_c - t_{fc}); \qquad s = 12\, t_{fc}$$

$$r = (b_{fc} - m) / 2; \qquad w = (b_{fp} - m) / 2 \qquad \text{(W13.1.15)}$$

A yield line analysis of this plate gives the ultimate capacity developed as

$$P_{u1} = C\, t_{fc}^{\;2} F_{yc} \qquad \text{(W13.1.16)}$$

where

$$C = \frac{\left(\dfrac{4}{\alpha_1}\right) + \left(\dfrac{\alpha_1}{\beta}\right)}{2 - \left(\dfrac{\beta}{\alpha_2}\right)} \qquad \text{(W13.1.17)}$$

$$\alpha_1 = \frac{s}{r}; \quad \alpha_2 = \frac{w}{r}; \quad \beta = \frac{\alpha_1}{4}\left[\sqrt{\alpha_1^2 + 8\alpha_2} - \alpha_1\right] \qquad \text{(W13.1.18)}$$

For the wide flange columns and beams used in practical connections, it has been found that C varies within the range of 3.5 to 5. A conservative approximation to P_{u1} is therefore given by

$$P_{u1} = 3.5\, t_{fc}^2 F_{yc} \qquad \text{W13.1.19)}$$

The force carried by the central rigid portion of the column in line with the web is

$$P_{u2} = t_{fp}\, m\, F_{yc} \qquad \text{(W13.1.20)}$$

The ultimate capacity of the connection is therefore given by

$$\begin{aligned} P_u &= 2P_{u1} + P_{u2} \\ &= 7.0\, t_{fc}^2 F_{yc} + t_{fp}\, m\, F_{yc} \end{aligned} \qquad \text{(W13.1.21)}$$

Reducing the strength of the column region by 20 percent and making further conservative assumptions, the above equation is reduced to the following [Graham et al., 1959]:

$$P_{n,\,\mathrm{FB}} = 6.25\, t_{fc}^2 F_{yc} \qquad \text{(W13.1.22)}$$

This is Eq. K1-1 of the LRFD Specification. The resistance factor ϕ for this limit state is specified as 0.90. The column flange local bending strength is therefore given by

$$P_{d,\,\mathrm{FB}} \equiv \phi P_{n,\,\mathrm{FB}} = \phi 6.25\, t^2_{fc}\, F_{yc} \equiv P_{fb} \qquad \text{(W13.1.23)}$$

If the length of the loading b_{fb}, or b_{fp} measured across the column flange is less than 0.15 b_{fc} where b_{fc} is the flange width of the column, flange local bending will not be critical and Eq. W13.1.24 need not be checked. The design flange local bending strength, $P_{d,\mathrm{FB}}$, is tabulated as P_{fb} for W-shapes normally used as columns in the Properties section of the Column Load Tables (LRFDM Table 4-2) for steels with $F_y =$ 50 ksi.

The effective column flange length for flange local bending is $12t_{fc}$, i.e., $6t_{fc}$ on each side from the line of the applied concentrated force. To develop the fixed edge consistent with the assumption of this model, an additional $4t_{fc}$ (resulting in a total of $10t_{fc}$) is required for the full flange-bending strength given by LRFDS Eq. K1-1. Thus, if the distance from the column end to the center line of the connected beam tension flange or flange plate is less than $10t_{fc}$, LRFDS Section K1-2 states that the flange bending strength at this column end location must be reduced to 50 percent of the strength at an intermediate column location given by Eq. W13.1.23.

If the thickness of the column flange t_{fc} is such that the design tensile strength $P_{d,\mathrm{FB}}$ is less than the required tensile strength P_u, a pair of transverse stiffeners one on each side of the column web are needed in line with the tension flange of the beam. Such stiffeners keep the column flange from deflecting and load the weld uniformly. They must extend at least one half the depth of the column webs.

W13.1.5 Shear in Panel-Zone

Figure W13.1.7: Panel-zone web shear at an interior column.

As mentioned earlier, if the moments in beams connected to opposite flanges of a column differ significantly in magnitude, they may produce large shear forces in the column web within the beam-to-column panel-zone. Figure W13.1.7*a* shows a three-bay, two-story, rigid-jointed, unbraced frame under factored loads (gravity loads and wind from right). Beams which are part of such rigid frames must resist end moments resulting from both gravity loads (dead and live loads) and horizontal loads (wind loads). Figure W13.1.7*b* shows the moment distribution for two beams, whose ends 1 and 2 are connected to joint J of the frame. Note that the contributions from gravity component and horizontal component of the factored loading are shown separately. Note also that the wind moment on the windward side of the column are additive to the gravity moments, while on the leeward side of the column they act in opposite directions and only the resulting (or, net) moment must be used. When the moments are additive, it is conservative to use the full dead and live loads. When the moments are opposing, however, the designer may elect to assume that only the gravity dead loads are effective. The story shear in the upper column is then substracted from the net force due to the beam moments to determine the shear within the column web. The resulting forces acting on the members at joint J are shown in Fig. W13.1.7*c*. Equilibrium of forces acting on the top stiffener and column web, in the horizontal direction (Fig. W13.1.7*d*), gives:

$$V_{uz} = P_{uf1} - P_{uf2} - V_{uc} \tag{W13.1.24}$$

with

$$P_{uf1} = \frac{M_{u1}}{d_{m1}}; \qquad P_{uf2} = \frac{M_{u2}}{d_{m2}} \tag{W13.1.25a}$$

$$M_{u1} = M_{u1,G} + M_{u1,H} \tag{W13.1.25b}$$

$$M_{u2} = M_{u2,G} - M_{u2,H} \tag{W13.1.25c}$$

where $M_{u1,G}$ = moment at end 1 due to factored gravity loads on the frame, in.-kips ($\frown$)

$M_{u1,H}$ = moment at end 1 due to factored horizontal loads on the frame, in.-kips ($\frown$)

$M_{u2,G}$ = moment at end 2 due to factored gravity loads on the frame, in.-kips ($\frown$)

$M_{u2,H}$ = moment at end 2 due to factored horizontal loads on the frame, in.-kips($\smile$)

$\mathrm{V_{uc}}$ = shear on the joint from column above, kips ($\leftarrow$)

V_{uz} = shear in the web panel zone, kips ($\leftarrow$)

d_{m1}, d_{m2} = distance between flange forces in a moment connection, in.

The maximum shear resistance of the web is $F_{vyc} d_c t_{wc}$, where F_{vyc} is the yield stress of the column web steel in shear. The nominal shear strength of the column web for low levels of axial load in the column (from Eq. K1-9 of the the LRFDS) may be written as:

$$V_{nc1} = (0.6F_{yc})\, d_c\, t_{wc} \tag{W13.1.26}$$

where d_c = depth of the column, in.

t_{wc} = thickness of the column web, in.

For webs subjected to high shear in combination with high axial loads (i.e., P_u exceeding $0.4P_y$) the deleterious effects of the interaction of shear and axial forces has to be considered by using the following relation (from Eq. K1-10 of the LRFDS)

$$V_{nc2} = (0.6F_{yc})d_c t_{wc}\left[1.4 - \frac{P_u}{P_y}\right] = V_{nc1}\left[1.4 - \frac{P_u}{P_y}\right] \text{ for } P_u > 0.4P_y \qquad \text{(W13.1.27)}$$

where P_u = factored axial load in the column, kips

P_y = axial yield strength of the column ($= A_c F_{yc}$), kips

Figure W13.1.8 shows the interaction of shear and axial force in column web panel-zone, as given by Eqs. W13.1.26 and 27. LRFDS requires that:

$$V_{dc} = \phi V_{nc} = 0.9V_{nc} \geq V_{uz} \qquad \text{(W13.1.28)}$$

Figure W13.1.8: Interaction of shear and axial force in column web panel-zone.

If Eq. W13.1.28 is not satisfied, the web is not thick enough to resist the shear forces in the panel- zone, the connection can be reinforced by welding a doubler plate to the web or by using diagonal stiffeners; stiffeners are preferable. We have:

$$V_{ur} = V_{uz} - V_{dc} \qquad \text{(W13.1.29)}$$

where V_{ur} = required shear strength of the reinforcement in the panel-zone, kips

V_{uz} = required shear strength of the web in the panel-zone, kips (Eq. W13.1.24)

V_{dc} = design shear strength of the web in the panel-zone, kips (Eq. W13.1. 26, or 27)

W13.1.6 Panel-Zone Web Reinforcement

To summarize:

- At the location of the tensile component of a double concentrated force, the limit state of web local yielding and flange local bending must be checked.
- At the location of the compressive component of a double concentrated force, the limit states of web local yielding and web crippling must be checked.
- At the location of the compressive components of a pair of double concentrated forces, the limit state of web buckling must be checked.
- The limit state of panel-zone web shear must be checked at all beam-to-column moment connections.

If any of the design strengths P_d (namely, $P_{d,\,\mathrm{WY}}$, $P_{d,\,\mathrm{WC}}$, $P_{d,\,\mathrm{FB}}$ or $P_{d,\,\mathrm{WB}}$) are less than the factored concentrated load P_{uf}, then panel-zone web reinforcement is required. The reinforcement provided must carry the excess concentrated flange forces that the column web or flange is unable to carry. Horizontal equilibrium gives:

$$P_{ur} \quad = \quad P_{uf} - P_d \qquad \text{(W13.1.30)}$$

where P_{ur} is the required strength of the reinforcement in the panel-zone. However, as mentioned in Section 13.11, stiffeners are very expensive, especially if they must be fitted between the column flanges. In many cases, it would be much more economical for the designer to select a heavier section than what is required for the primary beam-column (Chapter 11) loading but would not require stiffeners.

Transverse Stiffeners

The required area of the transverse stiffeners may be obtained from the relation:

$$A_{st} \geq \frac{P_{ur}}{F_{d,st}} \tag{W13.1.31}$$

where P_{ur} = required strength of the stiffeners, from Eq. W13.1.30

$F_{d,st}$ = design stress in the stiffener

= $\phi F_{y,st} = 0.9F_{y,st}$ for stiffeners in tension

= $\phi F_{cr,st} \approx 0.85F_{y,st}$ for stiffeners in compression

$F_{y,st}$ = yield stress of the stiffener material, ksi

ϕ = resistance factor

A_{st} = area of the stiffener, in.^{2}

Figure W13.1.9: Transverse stiffeners.

The standard transverse stiffeners are an efficient way to stiffen the column web (Fig. W13.1.9). It should be noted here that stiffeners on all columns are generally made of A36 steel as stiffness and stability, rather than strength, normally govern stiffener design. The following rules govern the design of these stiffeners.

1. Stiffeners must be placed in pairs at points of concentrated loads on columns whenever the required flange force P_{uf} exceeds the design strength $P_{d,\,WY}$, $P_{d,\,WC}$, $P_{d,\,WB}$, or $P_{d,\,FB}$.

2. If the concentrated force P_{uf} exceeds $P_{d,\,WY}$, $P_{d,\,WC}$, or $P_{d,\,FB}$, stiffeners need not extend more than one half the depth of the web when a beam is connected to one flange only, as

at an exterior column. When beams are connected to both flanges of a column, opposite to each other, and stiffeners are required, the stiffeners should extend continuously between the flanges of the column.

3. If the concentrated compressive load(s) P_{uf} exceed(s) $P_{d,\,\mathrm{WC}}$ or $P_{d,\,\mathrm{WB}}$, the stiffeners must be designed as axially compressed columns as per Section E2 of the LRFDS. The effective length of this column is taken as equal to $0.75h_c$. The column cross-section is assumed to consist of the two stiffeners and a strip of the web having a width of $25t_{wc}$.

4. The thickness of a transverse stiffener shall not be less than ½ t_{fp}, where t_{fp} is the thickness of the beam flange or flange connection plate delivering the concentrated load. The width of each stiffener plus half the thickness of the column web, shall not be less than ⅓ the width of the flange or flange connection plate delivering the concentrated force. That is:

$$t_{st} \geq \tfrac{1}{2}t_{fp}\,; \qquad b_{st} \geq \tfrac{1}{3}b_{fp} - \tfrac{1}{2}t_{wc} \tag{W13.1.32}$$

6. The stiffener's width-to-thickness ratio must satisfy Section B5.1 of the LRFDS. Thus, to prevent local buckling, the width-thickness ratio is limited (when the stiffener and the column are made of the same grade of steel) by

$$\frac{b_{st}}{t_{st}} \leq 0.56\sqrt{\frac{E}{F_{y,st}}} \tag{W13.1.33}$$

However, stiffeners made of A36 steel attached to columns made of high strength steel must deform inelastically, for all column stress values above the stiffener's 36 ksi yield point. It therefore appears reasonable to treat such stiffeners in the same manner as projecting elements of compact beam sections and require that:

$$\frac{b_{st}}{t_{st}} \leq 0.38\sqrt{\frac{E}{F_{y,st}}} \qquad \text{(W13.1.34)}$$

Transverse Stiffener-to-Column Welds

1. Column stiffeners, when provided, are shop welded.

2. To resist tensile concentrated forces, the stiffener must be welded directly to the flange upon which the tensile concentrated force is imposed. This weld is designed to develop the required strength of the stiffener, P_{ur}. Fillet welds are preferable; however, CJP groove welds may be required when the force in the stiffener is large.

3. When the concentrated force is always compressive, one end of a full depth stiffener is sometimes finished for bearing, with the other end welded.

4. When partial depth stiffeners for compressive concentrated forces are used, some fabricators prefer to finish the end in contact for bearing.

5. A web weld is always required for partial depth stiffener. It is sometimes economical to extend the partial depth stiffener beyond one-half the column web depth in order to reduce the weld size.

6. Fillet welds are preferable for connection between the stiffener and the column web; CJP or PJP groove weld are seldom required.

7. If concentrated forces from opposed moment connections at a joint are equal (i.e., P_{uf1} = P_{uf2}), they may theoretically be transferred entirely through stiffeners with no attachment to the column web, except as required for the limit state of web compression buckling and/or to prevent the stiffener from buckling as a column. Quite often, these forces are not equal. The differential axial force must then be transferred to the column web. In this

case appropriate weld sizes are required.

Doubler Plates

Figure W13.1.10: Reinforcement of web panel-zone by doubler plate.

A ***doubler plate*** or a ***pair of doubler plates*** may be used to strengthen a column web which is inadequate in web local yielding, web compression buckling, or panel zone shears (Fig. W13.1.10). A doubler plate requires considerable welding and can cause significant distortion of the column flanges, if the doubler plate is thick. Thus, if a doubler plate thicker than the column web or ¼ in. is required, the use of two thinner plates, one on either side of the column web, must be considered. Thin doubler plates may be subjected to shear buckling. If the doubler plate has the dimensions $t_{dp} \times d_{dp} \times h_{dp}$ and yield stress $F_{y,\,dp}$, shear buckling will not precede shear yield, if (from LRFDS Appendix F-2):

$$\lambda \;<\; \lambda_{pv} \;=\; 1.10\sqrt{\frac{k_v E}{F_{y,dp}}} \qquad \text{(W13.1.35)}$$

where

$$\lambda \;=\; \frac{d_{dp}}{t_{dp}}; \quad \alpha \;=\; \frac{h_{dp}}{d_{dp}}; \quad k_v \;=\; 5 \;+\; \frac{5}{\alpha^2} \qquad \text{(W13.1.36)}$$

Diagonal Stiffeners

To reinforce a column web which is inadequate in panel-zone shear, a pair of diagonal stiffeners may be used. They are located along the compression diagonal of the web panel-zone. They form the diagonals of a truss with the flanges as chords and verticals (see Fig. 13.10.3). The area A_{st} of the stiffener must be sufficient for the horizontal component of the stiffener compressive force to make up the deficiency in shear resistance of the column web. Thus, if θ is the inclination of the diagonal stiffeners with the horizontal and $P_{u,\,st}$ the required compressive strength of the diagonal stiffeners, we have:

$$P_{u,st} \cos\theta \quad = \quad V_{ur} \quad \rightarrow \quad P_{u,st} \quad = \quad \frac{V_{ur}}{\cos\theta} \qquad \text{(W13.1.37)}$$

The design compressive strength of the diagonal stiffener is

$$P_{d,\,st} \quad = \quad 0.85\, F_{cr,\,st}\, A_{st} \qquad \text{(W13.1.38)}$$

where $F_{cr,\,st}$ = design compressive stress of the diagonal stiffener acting as a pin-ended column of length L_{st}

≈ $F_{y,\,st}$ for preliminary calculations

A_{st} = area of the diagonal stiffener = $(2b_{st} + t_{wc})\, t_{st}$

The full force in the diagonal stiffener must be developed at each end, as for any truss diagonal, using either fillet welds or groove welds. The diagonal stiffeners will prevent column web buckling with only a nominal attachment to the web.

W13.1.7 Design Tables and Charts

The expressions for connection design strength for limit states of web local yielding $P_{d,\,\mathrm{WY}}$ (Eq.

W13.1.3), web crippling $P_{d,\,\mathrm{WC}}$ (Eq. W13.1.10), web compression buckling $P_{d,\,\mathrm{WB}}$ (Eq. W13.1.14) and flange bending $P_{d,\,\mathrm{FB}}$ (Eq. W13.1.23) can be rewritten as follows:

$$P_{d,\,\mathrm{FB}} = P_{fb}$$

$$P_{d,\,\mathrm{WY}} = P_{wo} + P_{wi}\,N$$

$$P_{d,\,\mathrm{WC}} = C_4 + C_5\,N$$

$$P_{d,\,\mathrm{WB}} = P_{wb}$$

It is seen that for a selected column and steel grade, P_{fb}, P_{wo}, P_{wi}, C_4, C_5 and P_{wb} are constants. It follows therefore that $P_{d,\,\mathrm{WB}}$ and $P_{d,\,\mathrm{FB}}$ are constants, while $P_{d,\,\mathrm{WY}}$ and $P_{d,\,\mathrm{WC}}$ vary linearly with N. The valules of P_{wb}, P_{fb}, P_{wi}, and P_{wo} for the W-shapes used as columns have been computed and given in the Properties section of the Column Load Tables (LRFDM Table 4-2). More extensive tables are given in [USS, 1979; Vinnakota, 1986; Vinnakota, Mallare and Vinnakota, 1987; Carter, 1999]on stiffening of webs subjected to concentrated loads.

EXAMPLE W13.1.1 Column Stiffening at Moment Connection

Check if the W14×90 column of Example 13.12.1 is adequate for the double concentrated forces P_{uf} = 200 kips delivered by the flange plated moment connection to W21×57 beams. Assume that the factored axial load in the column, $P_u = 0.6P_y$.

Solution

1. Data

From Example 13.12.1:

Force transmitted by flange plate, $P_{uf} = 200$ kips

Thickness of flange plate, $t_{fp} = 7/8$ in.

Width of flange plate, $b_{fp} = 8$ in.

Depth of the W21×57 beam, $d_b = 21.1$ in.

Distance between the centerlines of flange plates,

$$d_{fp} = d_b + t_{fp} = 21.1 + 0.875 = 22.0 \text{ in.}$$

W14×90 column:

$d_c = 14.0$ in.; $t_{fc} = 0.710$ in.; $t_{wc} = 0.440$ in.; $k_c = 1.31$ in.

$$\left(\frac{h}{t_w}\right)_c = 25.9 \rightarrow h_c = 11.4 \text{ in.}$$

2. Check adequacy of column flange and web

Design strength of the column in flange local bending,

$$P_{d,\mathrm{FB}} = 0.9(6.25)\, t_{fc}^2\, F_{yc}$$

$$= 0.9\,(6.25)\,(0.710^2)\,(50) = 142 \text{ kips} < 200 \text{ kips} \qquad \text{N.G.}$$

Design strength of the column web local yielding

$$P_{d,\mathrm{WY}} = 1.0(5k_c + N)\,t_{wc}\,F_{yc} = 1.0(5 \times 1.31 + 0.875)\,(0.440)\,(50)$$

$$= 144 + 0.875\,(22) = 163 \text{ kips} < 200 \text{ kips} \qquad \text{N.G.}$$

Design strength corresponding to the limit state of web compression buckling,

$$P_{d,\mathrm{WB}} = 0.9(24) \times \frac{t_{wc}^2}{(h/t_w)_c}\sqrt{EF_{yc}} = 0.9(24)\left(\frac{0.440^2}{25.9}\right)\sqrt{29{,}000\,(50)}$$

$$= 193 \text{ kips} < 200 \text{ kips} \qquad \text{N.G.}$$

Alternatively, these design strengths can be obtained from Properties section of the Column Load Tables (LRFDM Table 4-2). For a W14×90 column:

$P_{d,\,\mathrm{FB}} = 142$ kips; $P_{d,\mathrm{WB}} = 194$ kips

$P_{wo} = 144$ kips; $P_{wf} = 22.0$ kips/in.

$$P_{d,\,\mathrm{WY}} = P_{wo} + N P_{wf} = 144 + 0.875\,(22) = 163 \text{ kips}$$

Design shear strength of the web panel-zone,

$$V_{dz} = 0.9(0.6F_y)d_c t_{wc}\left[1.4 - \frac{P_u}{P_y}\right] = 0.9(0.6)(50)(14.0)(0.440)[1.4 - 0.6]$$

$$= 133 \text{ kips} < 200 \text{ kips} \qquad \text{N.G.}$$

So, either provide a heavier column or provide web stiffeners. (Ans.)

3. Select a heavier column

Eliminating the need for a doubler plate and stiffeners through the selection of a column section with a thicker web may be reasonable economical alternative.

From the relation for design shear strength of the web panel-zone, we observe that this strength is essentially proportional to the column web thickness, for a given nominal depth of W-shapes. So, required web thickness

$$t_{w\,\text{req}} \geq \frac{200}{133}(0.440) = 0.662 \text{ in.}$$

From LRFDM Table 1-1, we observe that a W14×132, with $t_w = 0.645$ is the lightest W14 shape that is likely to work. For a W14×132 column, we have:

$$d_c = 14.7 \text{ in.}; \qquad t_{wc} = 0.645 \text{ in.}$$

resulting in:

$$V_{dz} = 0.9(0.6)(50)(14.7)(0.645)[1.4 - 0.6]$$

$$= 205 \text{ kips} > 200 \text{ kips} \qquad \text{O.K.}$$

From Properties Section of the Column Load Table (LRFDM Table 4-2), we obtain for a W14×132:

$$P_{d,\text{FB}} = 298 \text{ kips} > 200 \text{ kips} \qquad \text{O.K.}$$

$$P_{d,\text{WB}} = 611 \text{ kips} > 200 \text{ kips} \qquad \text{O.K.}$$

$$P_{wo} = 263 \text{ kips}; \qquad P_{wf} = 32.3 \text{ kips/in.}$$

$$P_{d,\mathrm{WY}} = 263 + 0.875(32.3) = 291 \text{ kips} > 200 \text{ kips} \qquad \text{O.K.}$$

So, select a W14×132 for the column. (Ans.)

4. Reinforce the web panel-zone of W14×90

Transverse stiffeners

Required stiffener area, $A_{st} \geq \dfrac{P_u - P_{d,\mathrm{FB}}}{0.9F_{y,st}} = \dfrac{(200 - 142)}{0.9(36)} = 1.79 \text{ in.}^2$

Width of stiffener, $b_{st} \geq \dfrac{1}{3}b_{fp} - \dfrac{1}{2}t_{wc} = \dfrac{8.00}{3} - \dfrac{0.440}{2} = 2.45 \text{ in.}$

Thickness of stiffener, $t_{st} \geq \dfrac{1}{2}t_{fp} = \dfrac{1}{2}\left(\dfrac{7}{8}\right) = \dfrac{7}{16} \text{ in.}$

Provide 2 PL ½×4 of A36 steel as plate stiffeners. Stiffener area provided,

$$A_{st} = 2(b_{st} - \text{clip})t_{st} = 2(4.00 - 0.750)0.5$$

$$= 3.25 \text{ in.}^2 > 1.79 \text{ in.}^2 \qquad \text{O.K.}$$

From LRFDS Table J2.4, minimum weld size to connect the stiffener to:

column flange (t_2 = 0.710 in.) → 1/4 in. → D = 4

column web (t_2 = 0.500 in.) → 3/16 in. → D = 3

Length of weld connecting the stiffener to the tension flange,

$$L_w \geq \frac{(200 - 142)}{2(2)(1.392)(4)} = 2.60 \text{ in.}$$

Weld length provided = b_{st} − clip = 4.00 − 0.750 = 3.25 > 2.60 in. O.K.

Length of weld connecting the stiffener to the column web,

$$L_w \geq \frac{(200 - 142)}{2(2)(1.392)(3)} = 3.47 \text{ in.}$$

Check plate local buckling:

$$\frac{b_{st}}{t_{st}} = \frac{4.00}{0.500} = 8.00 < 0.56\sqrt{\frac{29{,}000}{36}} = 15.9$$

As the design shear strength of the web panel zone of a W14×90 column is less than the applied force of 200 kips from the flange plate, the panel zone is to be reinforced by using a doubler plate or a diagonal stiffener. Required shear capacity of the reinforcement,

$$V_{ur} = P_{uf} - V_{dz} = 200 - 133 = 67.0 \text{ kips}$$

Doubler plate

Required web doubler plate thickness

$$t_{dp} > \frac{V_{ur}}{0.9(0.6)F_{y,dp}\,h_c} = \frac{67.0}{0.9(0.6)(36)(11.4)} = 0.302 \text{ in.}$$

Required size (number of sixteenths-of-an-inch) of fillet weld using E70 electrodes,

$$D \geq \frac{V_{ur}}{1.392\,h_c} = \frac{67.0}{1.392(11.4)} = 4.22, \quad \text{say } 5$$

Minimum size of fillet weld, from LRFDS Table J2.4, is 3/16 in. So, provide 5/16 in. weld. Minimum thickness of doubler plate to provide this weld is (5/16 + 1/16) = 3/8 in. So, select a 3/8 in. doubler plate of A36 steel and provide 5/16 in. fillet welds.

Check doubler plate for shear buckling:

$$a = d_b - t_{fb} - t_{st} = 21.1 - 0.650 - 0.500 = 20.0 \text{ in.}$$

$$\frac{a}{h} = \frac{20.0}{11.4} = 1.75; \quad k_v = 5 + \frac{5}{(a/h)^2} = 6.63$$

$$\frac{d_{dp}}{t_{tp}} = \frac{11.4}{0.375} = 30.4 < 1.10\sqrt{\frac{6.63(29{,}000)}{36}} = 80.4 \quad \text{O.K.}$$

Diagonal stiffeners

Inclination of the stiffener with the horizontal:

$$\tan\theta = \frac{20.0}{11.4} \rightarrow \theta = 60.3^\circ \rightarrow \cos\theta = 0.495$$

Required axial strength of the diagonal stiffener,

$$P_{u,st} = \frac{V_{ur}}{\cos\theta} = \frac{67.0}{0.495} = 135 \text{ kips}$$

Assuming $F_{cr} = F_y$ for the stiffener subjected to compression, the required stiffener area is:

$$A_{st} \geq \frac{P_{u,st}}{0.85F_y} = \frac{135}{0.85(36)} = 4.41 \text{ in.}^2$$

Provide 2PL ½ in.× 4 ½ in. A36 steel plates as diagonal stiffeners. Area provided,

$$A_{st} = (2b_{st} + t_{wc})t_{st} = (2\times 4.50 + 0.440)(0.500) = 4.72 > 4.45 \text{ in.}^2 \qquad \text{O.K.}$$

Check plate local buckling:

$$\frac{b_{st}}{t_{st}} = \frac{4.50}{0.500} = 9.00 < 0.56\sqrt{\frac{29{,}000}{36}} = 15.9 \qquad \text{O.K.}$$

Use 3/16 in. fillet welds. Length of weld required to transmit the force,

$$L \geq \frac{P_{u,st}}{2(2)(1.392D)} = \frac{135}{2(2)(1.392)(3)} = 8.08 \text{ in.}$$

Length of weld provided = length of diagonal stiffener ≈ 23 in. > 8.08 in. O.K.

References

W13.1 Carter, C. J. [1999]: *Stiffening of Wide-Flange columns at Moment Connections : Wind and Seismic applications,* Steel Design Guide Series, no. 13, AISC, Chicago, IL.

W13.2 Chen, W. F., and Newlin, D. E. [1973]: "Column Web Strength in Beam-to-Column Connections", *Journal of Structural Division,* ASCE, vol. 99, ST-9, September, pp. 1978-84.

W13.3 Curtis, L. E. and Murray, T. M.[1989]: "Column Flange Strength at Moment End-Plate Connections," *Engineering Journal,* AISC, vol. 26, no. 2, pp. 41-50.

W13.4 Graham, J. D., Sherbourne, A. N., Khabbaz, R. N., and Jensen, C. D. [1959]:

Welded Interior Beam to Column Connections, American Institute of Steel Construction, New York, NY.

W13.5 Roberts, T. M. [1971]: "Slender Plate Girders Subjected to Edge Loading", *Proceedings, Institution of Civil Engineers*, part 2, vol. 71, September, pp. 805-819.

W13.6 Timoshenko, S. P., and Gere, J. M. [1961]: *Theory of Elastic Stability,* 2nd ed., McGraw Hill Book Company, New York.

W13.7 USS [1979]: Beam-to-Column Flange Connections - Restrained Members: A design Aid, United States Steel, Pittsburgh, December.

W13.8 Vinnakota, S. [1986]: LRFD of Steel Members, Lecture notes, Department of Civil & Environmental Engineering, Marquette University, Fall (unpublished).

W13.9 Vinnakota, S., Mallare, M. P. Jr., and Vinnakota, M. R. [1987]: "Load and Resistance Factor Design of Beam-to-Column Flange Connections for Restrained Members: Design Aids," *Journal of Structural Engineering Practice,* vol. 4, no. 182, pp. 19-66.

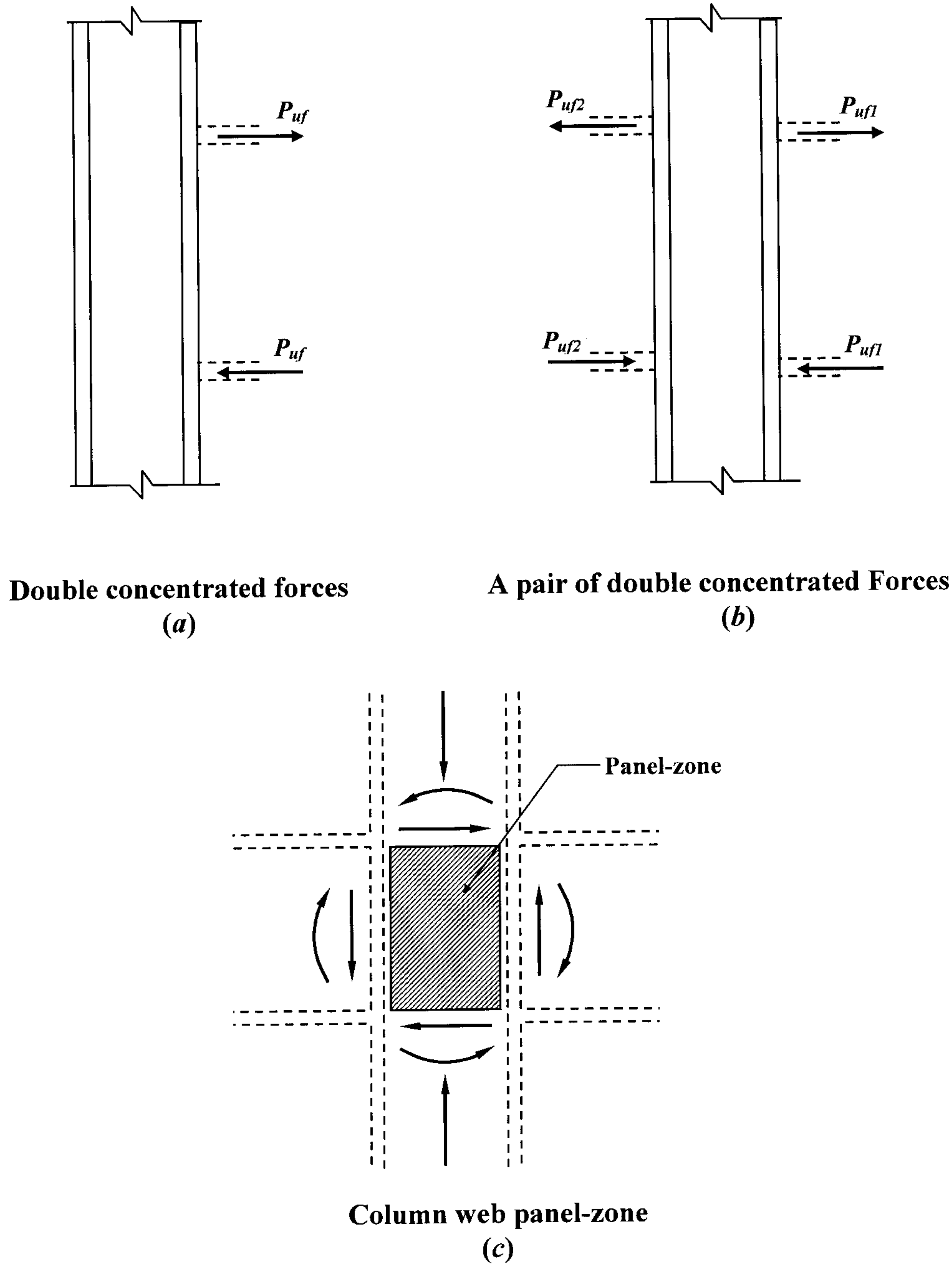
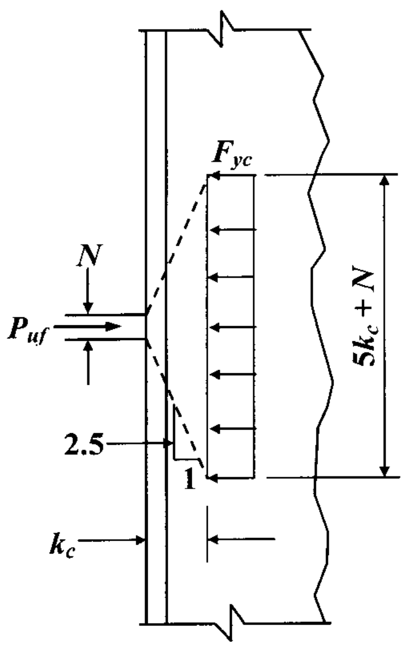
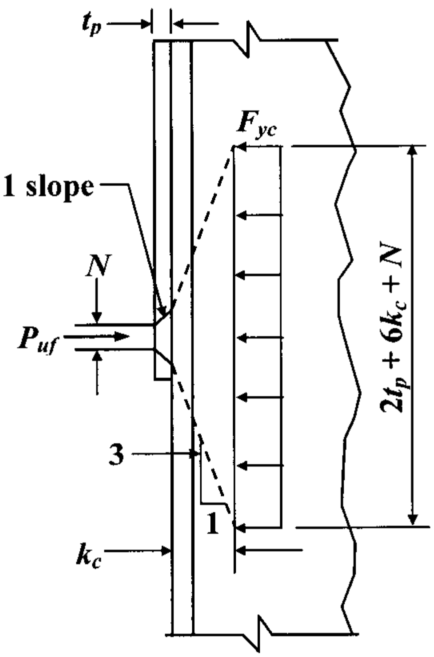


Figure W13.1.1: Beam-to-column moment connection terminology.

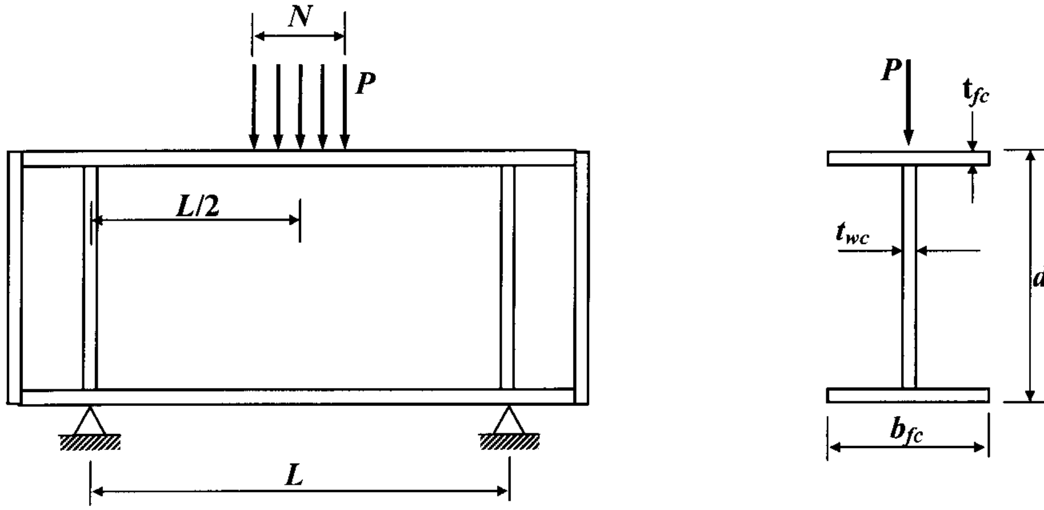


Directly welded flange or flange plated moment connection
(*a*)

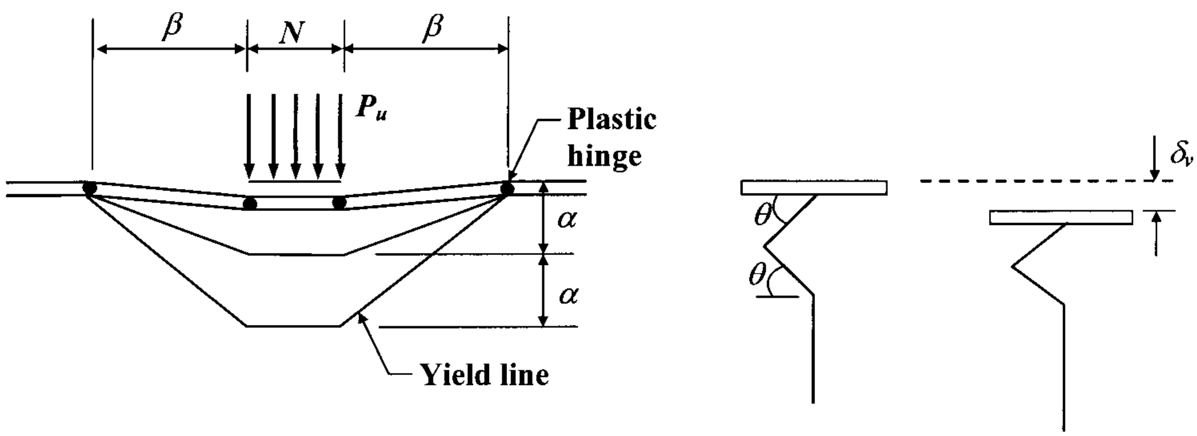


Extended end-plate moment connection
(*b*)

Figure W13.1.2: Web local yielding.



Girder subjected to edge loading
(*a*)



Assumed mechanism for web crippling
(*b*)

Figure W13.1.3: Loading and mechanism for web crippling.

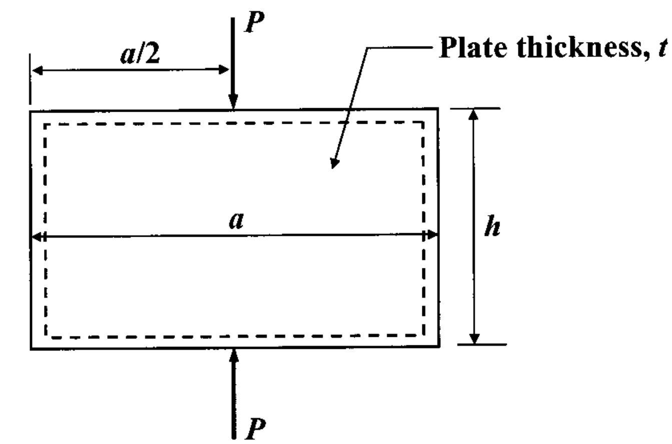
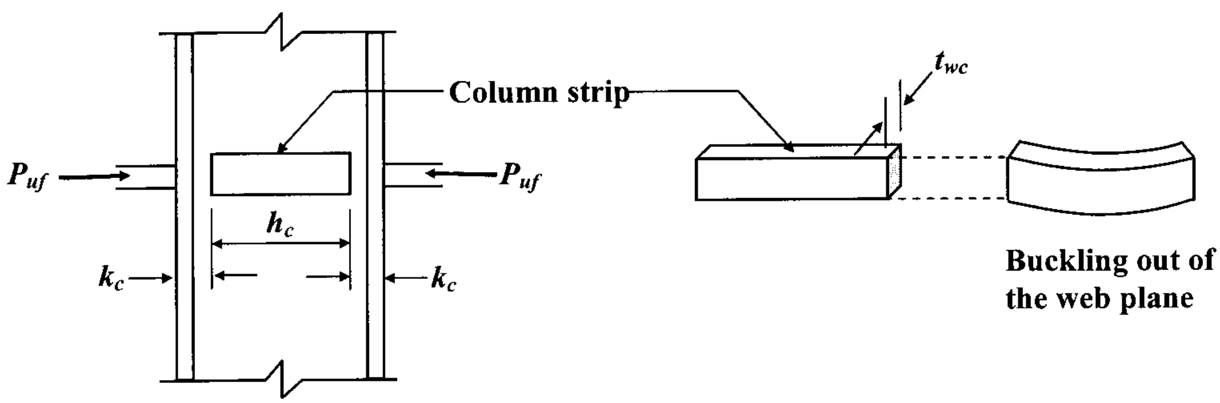


Plate with simply supported edges
(*a*)



Column web under a pair of concentrated compressive forces
(*b*)

Figure W13.1.4: Web compression buckling.

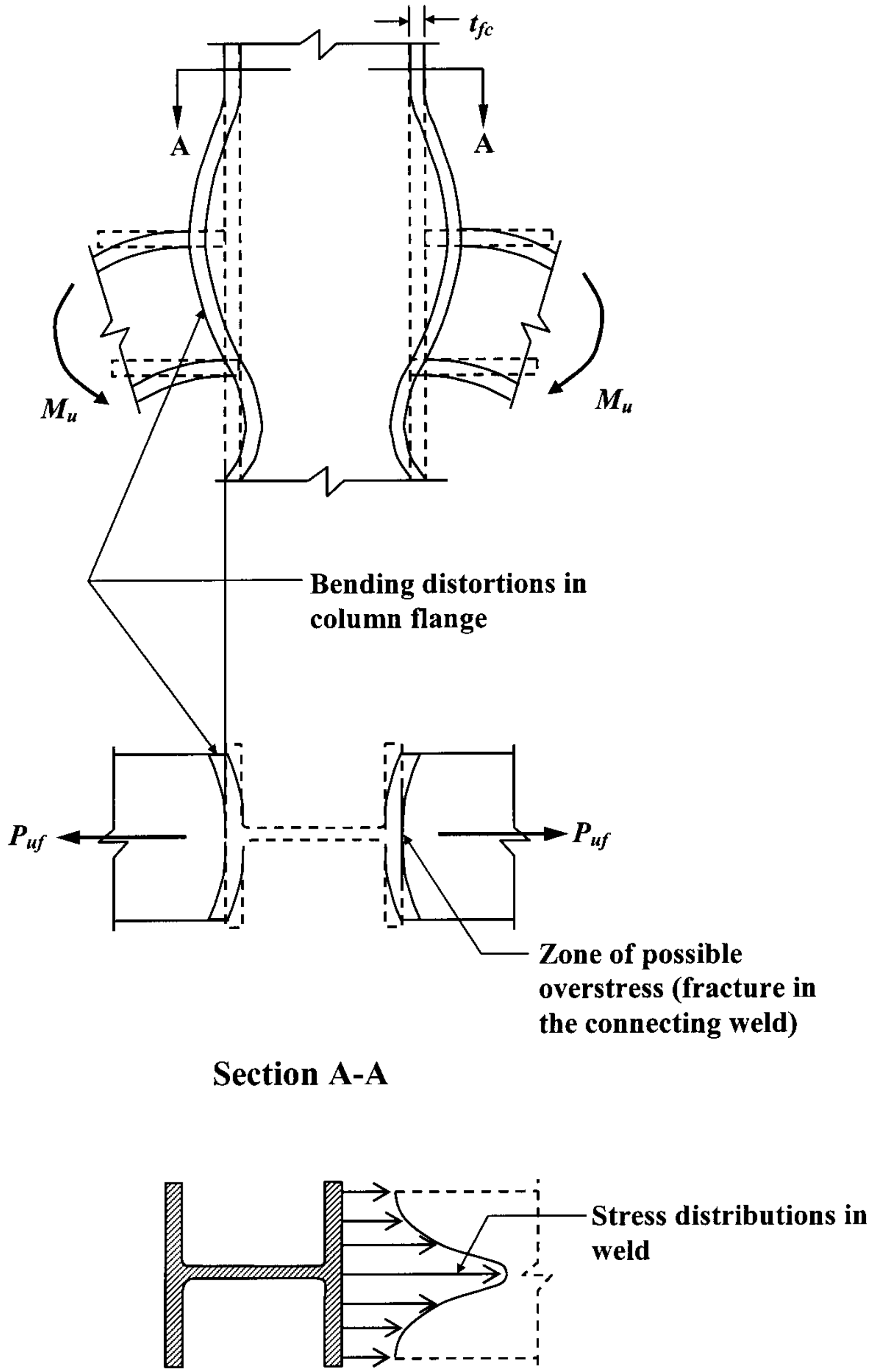


Figure W13.1.5: Flange local bending.

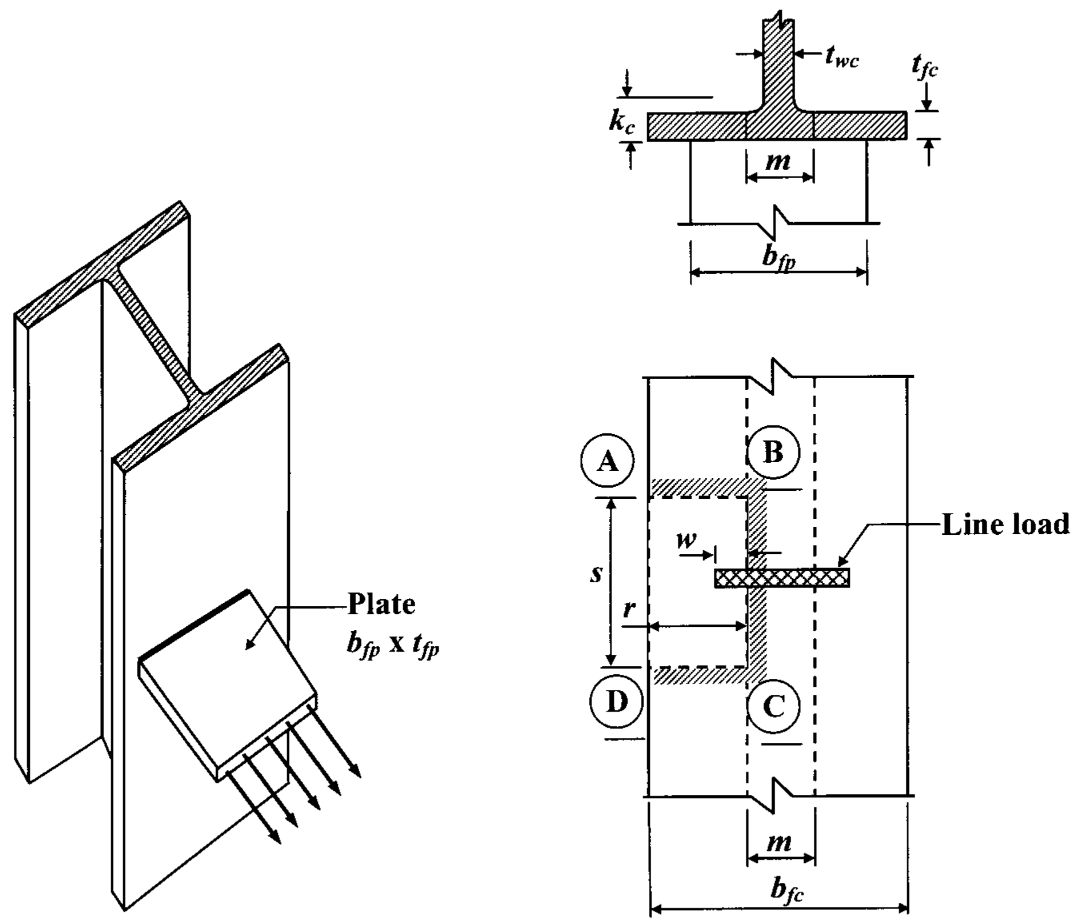
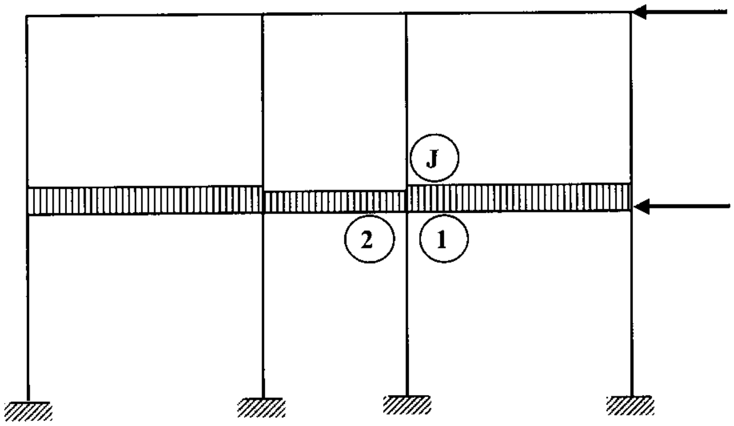
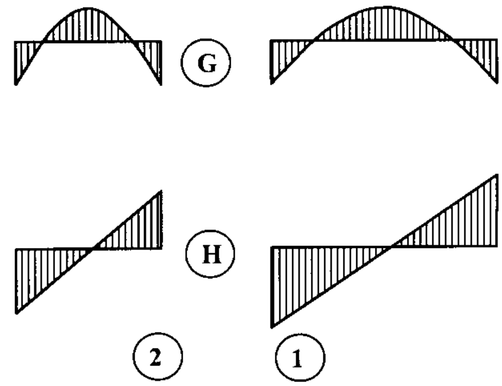


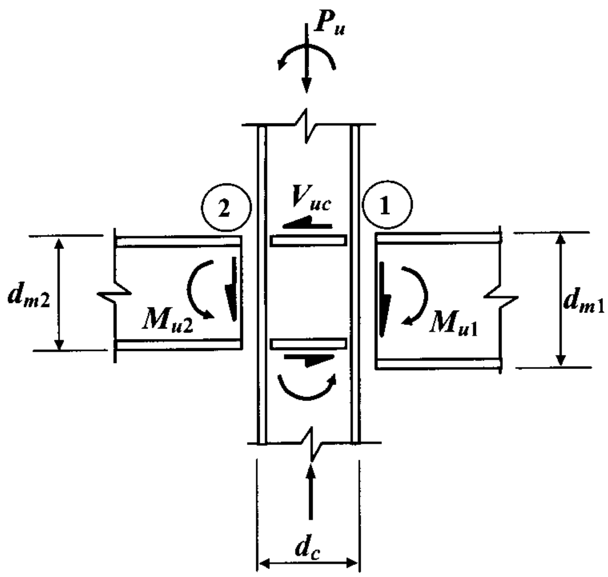
Figure W13.1.6: **Plate bending mechanism under tensile line load.**



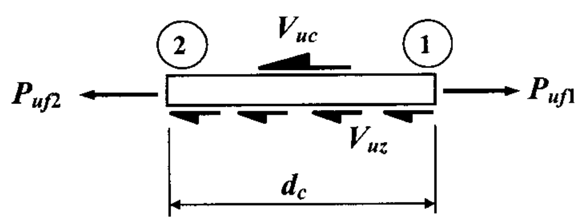
FR frame
(a)



Moments on beam-ends 1 and 2 under gravity and lateral loads
(*b*)



Forces acting at an interior beam-to-column connection
(*c*)



Forces acting on top stiffener and column web
(*d*)

Figure W13.1.7: Panel-zone web shear at an interior column.

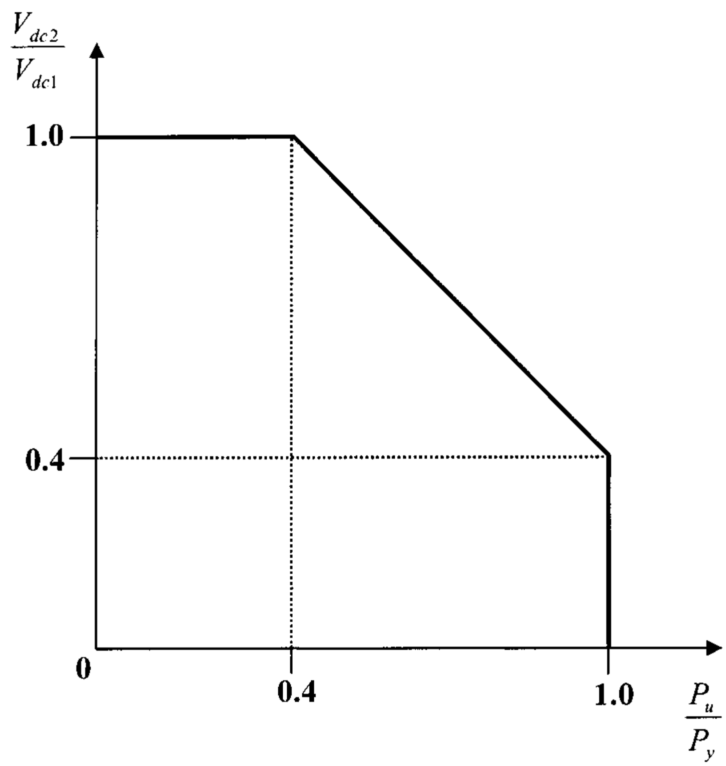


Figure W13.1.8: Interaction of shear and axial force in column web panel-zone.

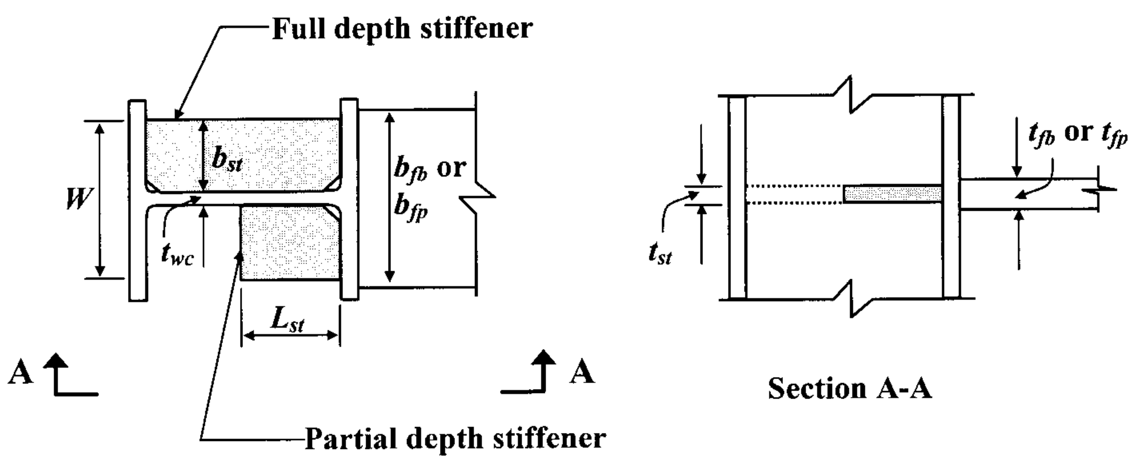


Figure W13.1.9: Transverse stiffeners.

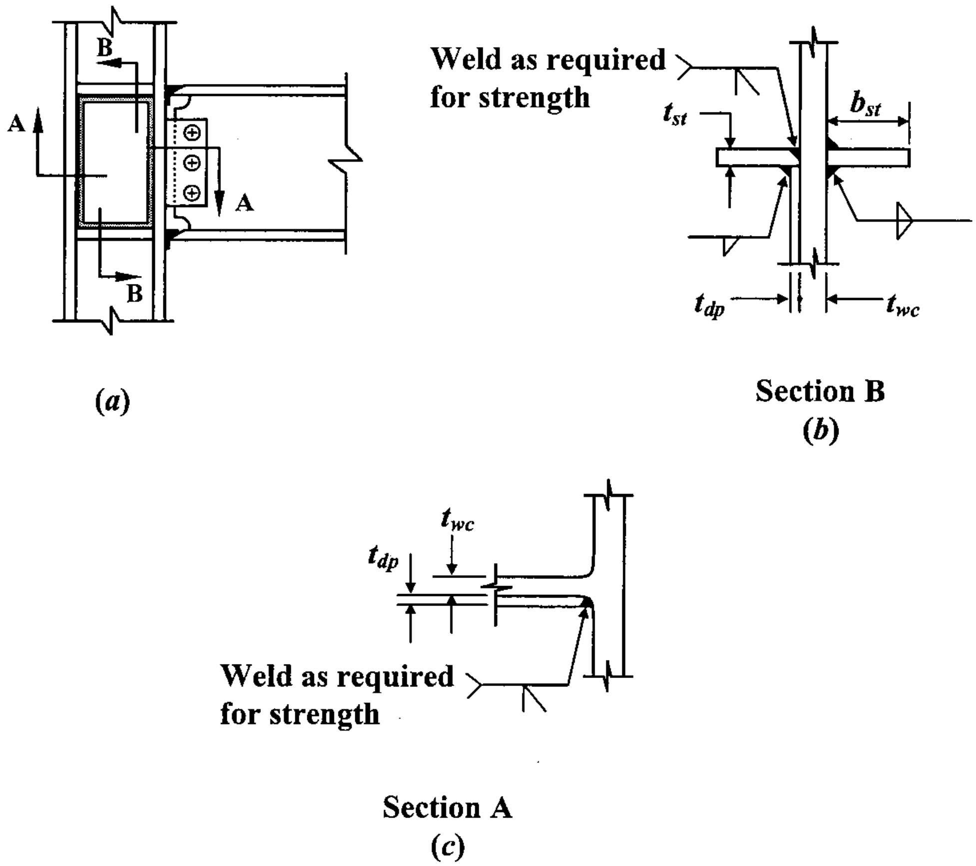


Figure W13.1.10: Reinforcement of web panel-zone by doubler plate.