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The Effect of Plastic Hinge Location on the Flexural Strength Demand of Welded Flange Plate Connections

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ABSTRACT: A Welded Flange Plate (WFP) connection consists of top and bottom flange plates that are fillet welded to the flanges of a beam and are complete joint penetration groove welded to a column flange. Four WFP design examples including various beam sizes, beam lengths, and/or assumed plastic hinge locations are presented in this paper. In these design examples, the relationships among the following three factors are investigated: the assumed location of the plastic hinge, the probable peak flexural demand at the end of the beam, and the required length of the flange plates for the WFP connections. Nonlinear static finite element analyses considering both the material and geometric nonlinearities are also carried out for these examples to verify the actual location of the plastic hinge as well as the relationship between the actual location of the plastic hinge and the length of the flange plates used for the WFP connections.

KEYWORDS: Finite element method; Flexural strength; Nonlinear analysis; Plastic hinges; Steel frames; Welded connections.

1 INTRODUCTION

Two typical types of flat-plate-reinforced connections that have been widely studied are Cover-Plate and Flange-Plate connections. These types of connections differ in that both the cover plates and the beam flanges are welded to the column flange in a Cover-Plate connection, while only the flange plates are welded to the column flange in a Flange-Plate connection. Studies of these connections have shown that Flange-Plate connections are marginally less likely to suffer brittle or ductile fracture than Cover-Plate connections [1]. Since Flange-Plate connections perform better than Cover-Plate connections, this paper concentrates on the study of the flexural strength of the Welded Flange Plate (WFP) connections and the effects of the location of the plastic hinge on the flexural strength demand of the WFP connections.

A WFP connection is a fully restrained moment connection that possesses sufficient rigidity to maintain the angle between connected members at the strength limit states [2]. As shown in Figure 1, WFP connections utilize plates that connect beam flanges to column flanges without any direct connection of the beam flanges to the column flanges. The connection between the flange plate and the column flange is a complete joint penetration groove weld, whereas the top and bottom flange plates are fillet welded to the beam top and bottom flanges, respectively [3].

In previous studies, the plastic hinge of WFP connections has been positioned at various locations. These locations include: (1) at the end of the flange plate, as shown in Figure 2(a) [3], (2) at a short distance beyond the end of the flange plate, as shown in Figure 2(b) [4], and (3) at a distance of 1.0 to 1.5 d_b from the face of the column, as shown in Figure 2(c) [5].

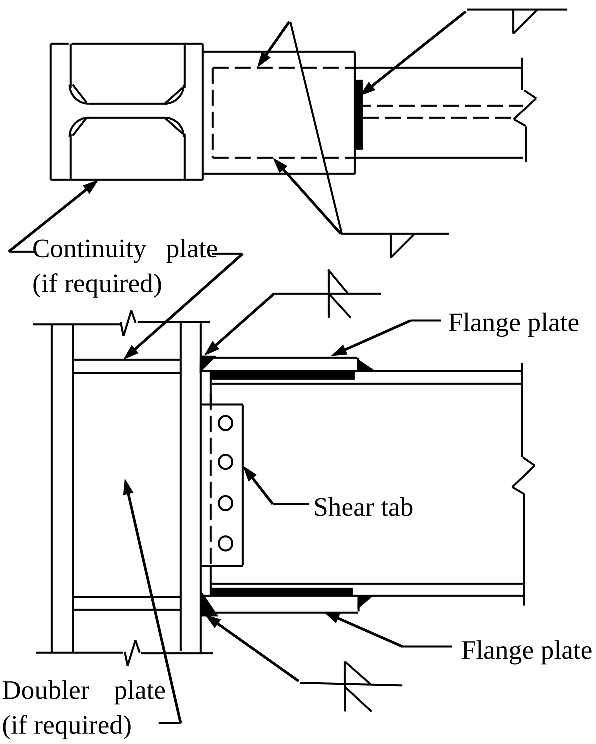
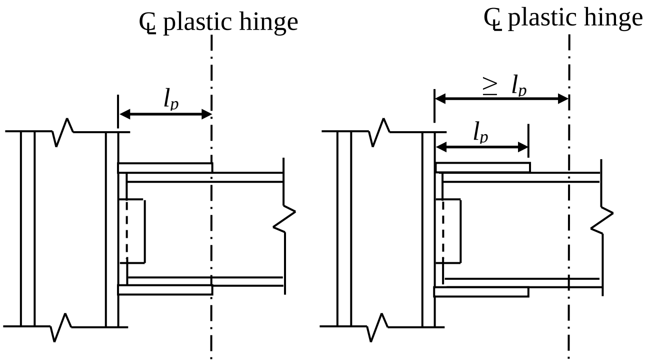
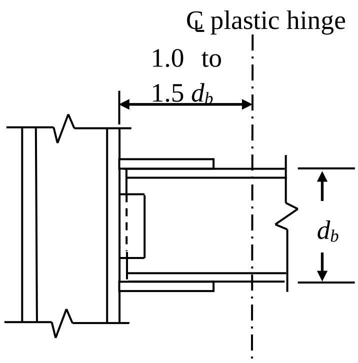


Figure 1. Welded Flange Plate Connection



(a) Plastic hinge located at end of flange plate

(b) Plastic hinge located at a short distance beyond end of flange plate



(c) Plastic hinge located 1.0 to 1.5 d_b from face of column

Figure 2. Plastic Hinge Locations

2 WELDED FLANGE PLATE CONNECTION DESIGN PROCEDURE

Referring to the multistory steel moment frame elevation shown in Figure 3, the procedure for designing the Welded Flange Plate connection of the moment frame is summarized as follows:

(1) Calculate the probable peak plastic hinge moment at the centerline of the plastic hinge.
The probable peak plastic hinge moment at the centerline of the plastic hinge of the beam may be determined using Eq. (1) [3]:

$$M_{pr} = C_{pr} R_y Z_e F_y \tag{1}$$

where M_{pr} = the probable peak plastic hinge moment; R_y = the ratio of the expected yield strength to the minimum specified yield strength of the steel to be used [a value of 1.1 is specified for ASTM A992 steel and ASTM A572 Grade 50 [3,6]; Z_e = the effective plastic modulus of the section at the location of the plastic hinge; F_y = the specified minimum yield stress of the material of the yielding element (F_y = 50 ksi (345 Mpa) for A992 steel [2]); and C_{pr} = a factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. For most connection types, C_{pr} may be computed using Eq. (2):

$$C_{pr} = \frac{F_y + F_u}{2F_y} \tag{2}$$

where F_u = the specified minimum tensile stress of the material of the yielding element (F_u = 65 ksi (448 Mpa) for A992 steel [2]).

(2) Determine the probable peak flexural strength demand at the face of the column and at the centerline of the column due to the formation of the plastic hinge in the beam.
Referring to Figure 3(b), the probable peak flexural strength demand at the face of the column may be determined using Eq. (3):

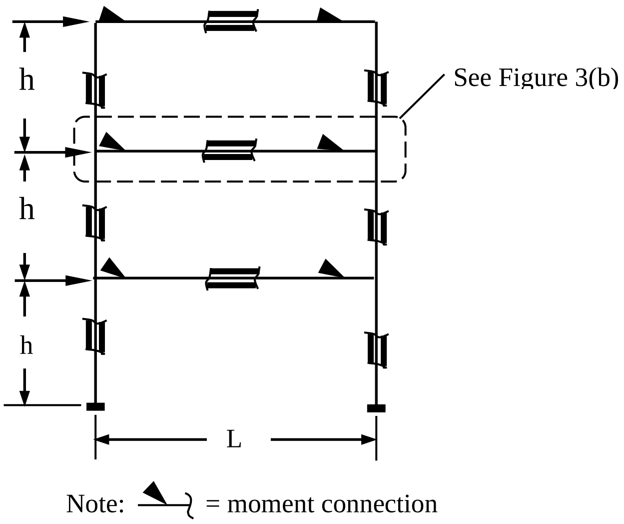
$$M_f = M_{pr} + V_p(x) \tag{3}$$

where M_f = the probable peak flexural strength demand at the face of the column due to the formation of the plastic hinge in the beam; V_p = the probable peak shear force at the centerline of the plastic hinge; and x = the distance from the face of the column to the centerline of the plastic hinge.
Also, referring to Figure 3(b), one has:

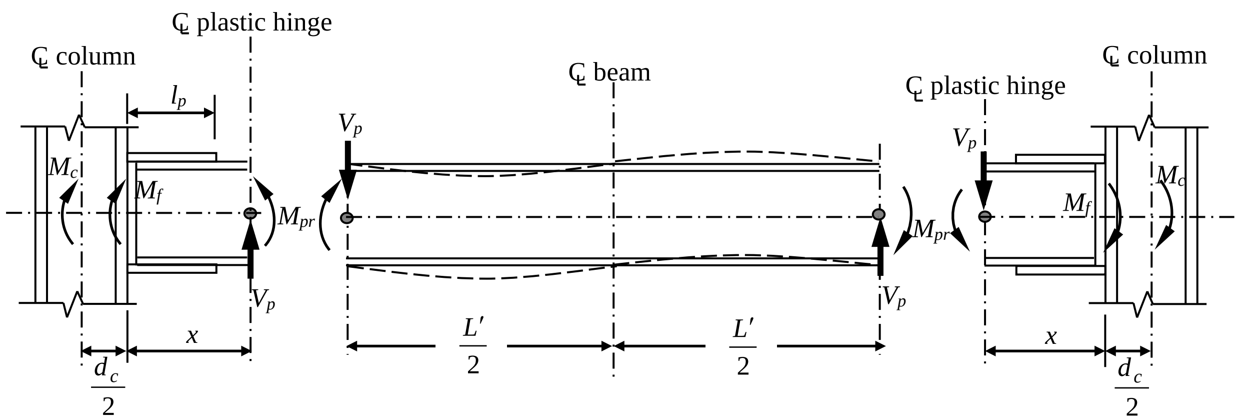
$$V_p = \frac{M_{pr} + M_{pr}}{L'} = \frac{M_{pr}}{\left(\frac{L'}{2}\right)} \tag{4}$$

From Eqs. (3) and (4), one has:

$$M_f = M_{pr} + \left(\frac{M_{pr}}{\left(\frac{L'}{2}\right)} \right)(x) \tag{5}$$



(a) Elevation of moment frame subjected to lateral forces



(b) Moments and shear forces at critical sections

Figure 3. A steel moment frame with welded flange plate connections

Also, referring to Figure 3, the probable peak flexural strength demand at the centerline of the column may be determined using the following equation:

$$M_c = M_{pr} + V_p\left(x + \frac{d_c}{2}\right) \tag{6}$$

where M_c = the probable peak flexural strength demand at the centerline of the column due to the formation of the plastic hinge in the beam; and d_c = the depth of the column.

From Eqs. (4) and (6), one has:

$$M_c = M_{pr} + \left(\frac{M_{pr}}{\left(\frac{L'}{2}\right)}\right)\left(x + \frac{d_c}{2}\right) \tag{7}$$

(3) Determine the width of the top and bottom flange plates.

Referring to Figure 4, the width of the top and bottom flange plates may be determined as:

$$b_p \geq b_f + 2\, t_f \tag{8}$$

where b_p = the width of the flange plate; b_f = the flange width of the beam; and t_f = the flange thickness of the beam.

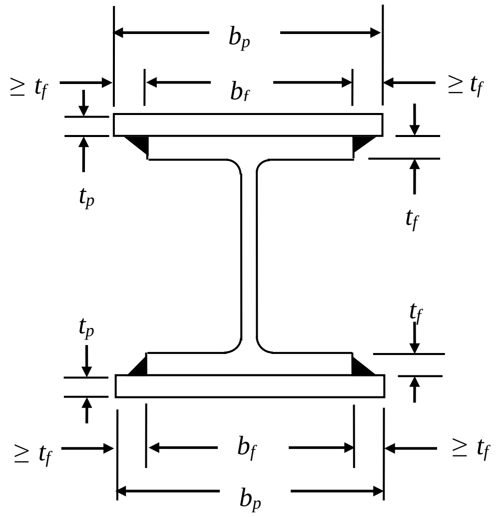


Figure 4. Cross-Section of Beam with Flange Plates

(4) Determine the thickness of the top and bottom flange plates.

Referring to Figure 5, the flexural strength at the column face, developed by the strength of the flange plates, may be computed using Eq. (9).

$$M_f = R_y F_{yp} b_p t_p \left(d_b + t_p\right) \tag{9}$$

where F_{yp} = the specified minimum yield stress of the flange plates; d_b = the beam depth; and t_p = the thickness of the top and bottom flange plates.

From Eq. (9), the thickness of the flange plates may be determined using Eq. (10) [7]. Note that the magnitude of M_f in Eq. (10) shall be equal to that determined in Eq. (5).

$$t_p = \frac{M_f}{R_y F_{yp} b_p \left(d_b + t_p\right)} \tag{10}$$

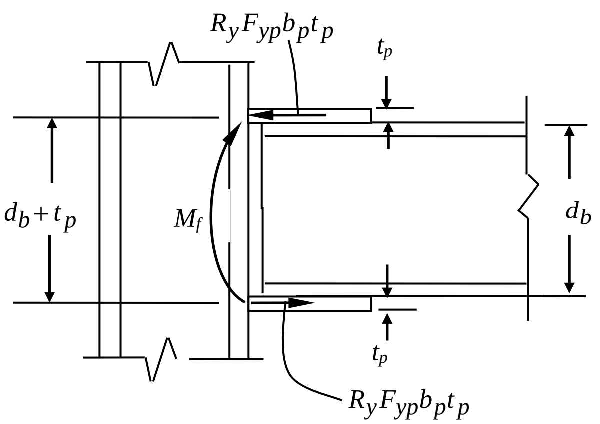


Figure 5. Moment at Face of Column Developed by Flange Plates

(5) Design the fillet welds joining the top and bottom flange plates to the top and bottom flanges of the beam, respectively. Referring to the fillet welds shown in Figure 6, it is difficult to deposit a weld continuously around the corner from one side (the side for the end weld) to the other (the side for the side weld) without causing a gouge in the corner of the parts joined; therefore, the welds must be discontinued at the corner [8].

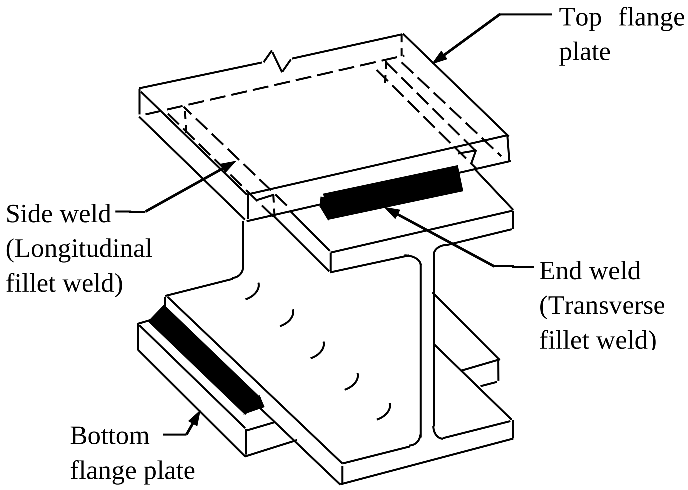


Figure 6. Fillet weld details for flange plates to flanges of beam connection

Based on the considerations mentioned above, the lengths of the side welds and the end welds shown in Figure 7 may be determined. Referring to Figure 7, the nominal design strength of the weld, F_w, may be calculated using Eq. (11).

$$F_w = [(0.707t_w)(l_p - 1.5'')(0.60)F_{EXX}] \times 2 + \left(0.707t_w\right)\left(b_f - 2''\right)\left(0.60F_{EXX}\right) \tag{11}$$

where t_w = fillet weld size; and F_{EXX} = weld metal tensile strength.

Furthermore, referring to Figure 7, the flexural strength at the column face, developed by the fillet welds, may be computed using Eq. (12).

$$M_f = F_w \times d_b \tag{12}$$

Note that M_f determined using Eq. (12) must be larger than or equal to that determined using Eq. (5). If the welds do not provide sufficient strength, return to Step 2 and select a longer flange plate length.

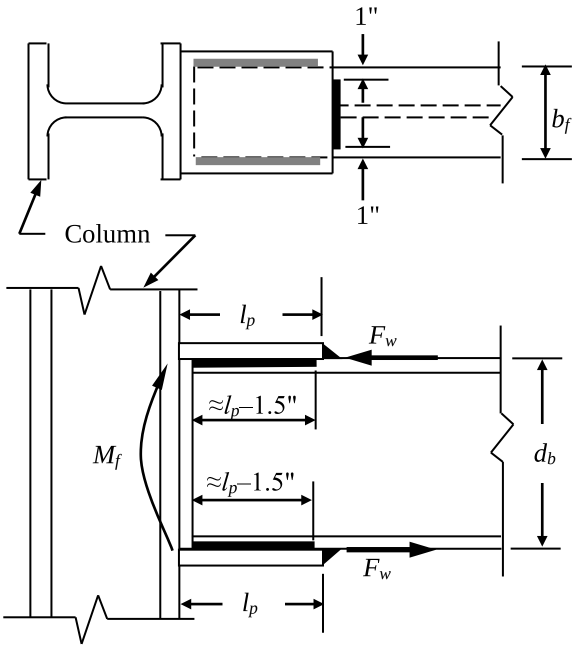


Figure 7. Flexural strength at face of column developed by fillet welds

(6) Check whether beam flange continuity plates are required.

Continuity plates across the column web are required if the thickness of the column flange is less than the value given either by Eq. 13 or 14 [3]:

$$t_{cf} < 0.4\sqrt{1.8 b_p t_p \frac{F_{yb}R_{yb}}{F_{yc}R_{yc}}} \tag{13}$$

$$t_{cf} < \frac{b_p}{6} \tag{14}$$

where t_{cf} = the thickness of the column flange; b_p = the width of the flange plate at the column face; t_p = the thickness of the flange plate; F_{yb} = the specified minimum yield stress of the beam; F_{yc} = the specified minimum yield stress of the column; R_{yb} = the ratio of the expected yield strength of the beam to the minimum specified yield strength of the steel to be used for the beam; and R_{yc} = the ratio of the expected yield strength of the column to the minimum specified yield strength of the steel to be used for the column.

(7) Check the strength of the panel zone and determine whether web doubler plate(s) is/are required.

The thickness of the panel zone that results in simultaneous yielding of the panel zone and beam may be determined using Eq. (15) [3]:

$$t_{pz} = \frac{C_y M_c\left[\frac{h-(d_b+t_p)}{h}\right]}{0.9(0.6)F_{yc}R_{yc}d_c(d_b+t_p)} \tag{15}$$

where t_{pz} = the thickness of the panel zone that results in simultaneous yielding of the panel zone and beam; M_c = the probable peak flexural strength demand at the centerline of the column due to the formation of the plastic hinge in the beam; h = the average story height of the stories above and below the panel zone; d_b = the beam depth; F_{yc} = the specified minimum yield stress of the column; R_{yc} = the ratio of the expected yield strength of the column to the minimum specified yield strength of the steel to be used for the column; d_c = the column depth; and

$$C_y = \frac{1}{C_{pr}\frac{Z_{be}}{S_b}} \tag{16}$$

where Z_{be} = the effective plastic section modulus of the beam at the zone of plastic hinging; and S_b = the elastic section modulus of the beam at the zone of plastic hinging.

If the required thickness of the panel zone determined using Eq. (15) is greater than the web thickness of the column, web doubler plate(s) is/are required.

The following procedure demonstrates the derivation of Eq. (15):

Figure 8 shows a moment frame subjected to lateral forces. The frame deformation due to lateral forces is shown in Figure 8(a). The inflection points of the column and beam deflections are assumed to be located at the mid-height and mid-span of the column and beam, respectively. Referring to Figure 8(b), one has:

$M_{cpz} = V_{pl}(d_b + t_p)$; $M_{cpz} = V_c(h)$; and

$$V_{pz} = V_{pl} - V_c$$

where M_{cpz} = the moment at the centroid of the panel zone; V_{pl} = the compressive or tensile force in the flange plate; V_c = the shear force in the column; and V_{pz} = the shear force in the panel zone.

From the above equations, one has:

$$V_{pz} = V_{pl} - V_c = \frac{M_{cpz}}{d_b + t_p} - \frac{M_{cpz}}{h}$$

$$= \frac{M_{cpz} - \frac{(d_b + t_p)M_{cpz}}{h}}{d_b + t_p} = \frac{M_{cpz}\left(1 - \frac{d_b + t_p}{h}\right)}{d_b + t_p}$$

$$= \frac{M_{cpz}\left(\frac{h - (d_b + t_p)}{h}\right)}{d_b + t_p}$$

At the time of simultaneous yielding of the panel zone and beam, one has:

$$V_{pz} = R_y \phi_v V_n = R_y (0.9)(0.6) F_{yc} (d_c) t_{pz} \text{ ; and}$$

$$M_{cpz} = C_y M_c$$

where $\phi_v V_n =$ the design shear strength of the panel zone; $\phi_v =$ the resistance factor for shear; and V_n = the nominal shear strength [9].

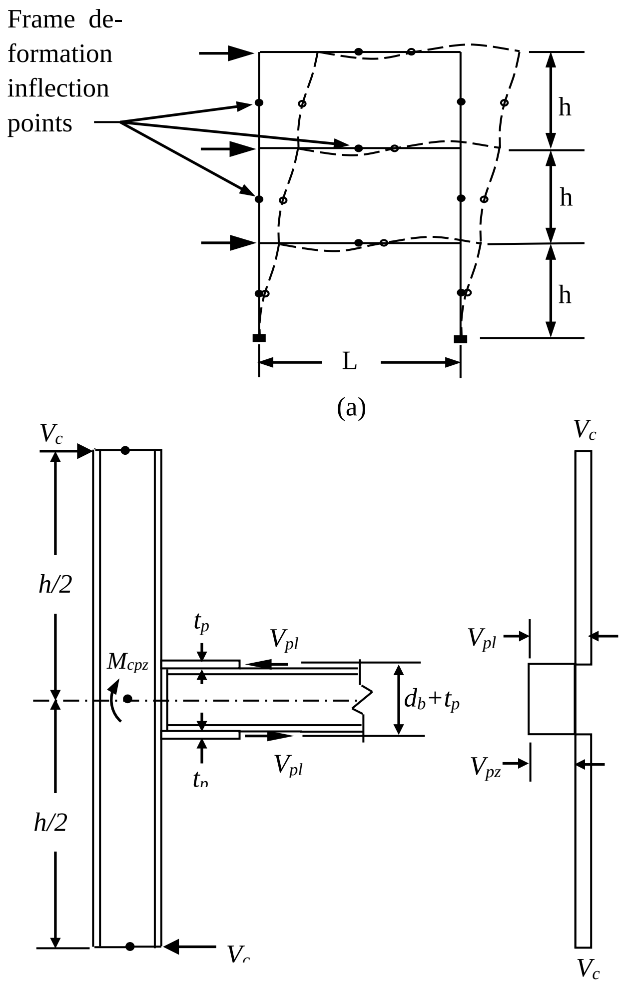


Figure 8. Free body diagram for calculation of panel zone shear stress

From the above equations, one has

$$V_{pz} = R_y (0.9)(0.6) F_{yc} (d_c) t_{pz} = = \frac{M_{cpz}\left(\frac{h - (d_b + t_p)}{h}\right)}{d_b + t_p}$$

From the above equation, one has Eq. (15).

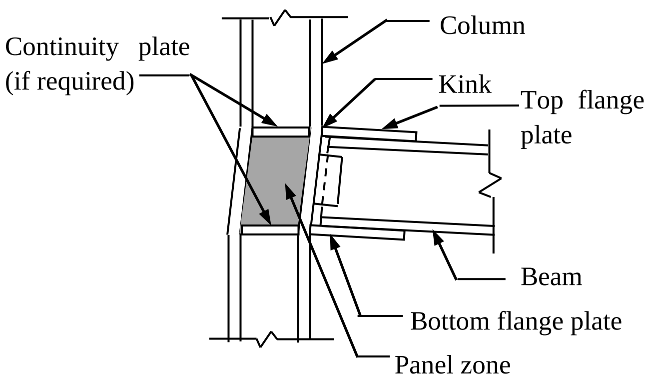
The panel zone thickness shall conform to the following requirement [6]:

$$t \geq (d_z + w_z)/90 \qquad (17)$$

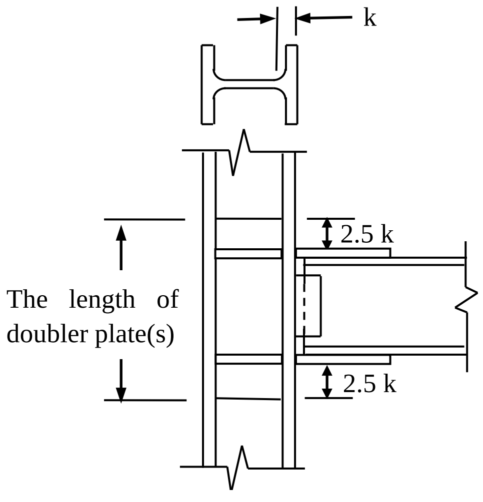
where t = the thickness of the column web (if doubler plate(s) is/are not required); the thickness of the column web or the doubler plate(s) (both the column web and the doubler plate(s) shall satisfy the thickness requirement if the doubler plate(s) is/are required); or the total thickness of the column web and the doubler plate(s) (if local buckling of the column web and the doubler plate(s) is prevented by using plug welds joining them); d_z = the depth of the panel zone between the continuity plates; and w_z= the width of the panel zone between the column flanges. In order to eliminate the effects of kinks on the column flanges [10], as shown in Figure 9(a), the doubler plate shall be extended to a distance of 2.5 k beyond the top and bottom of the top and bottom flange plates, respectively, as shown in Figure 9(b) [11], where k is the distance from the outer face of the column flange to the web toe of the fillet of the column, as shown in Figure 9(b).

(8) Design the shear tab.

Referring to Figure 1, the length of the shear tab equals d_b-2k-2" (where k is the distance from the outer face of the beam flange to the web toe of the fillet of the beam), and the thickness of the shear tab should match that of the beam web. The number, type, and size of the erection bolts shall be selected based on erection loads [3].



(a)



(b)

Figure 9. Effects of kinks on column flanges and length of doubler plate(s)

3 WELDED FLANGE PLATE CONNECTION DESIGN EXAMPLES

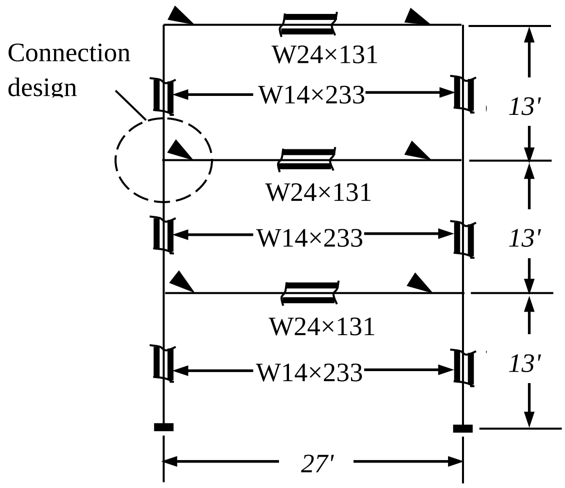
As shown in Figure 10, two steel moment frames are utilized to demonstrate the following four design examples for the Welded Flange Plate connections using the design procedure previously mentioned:

Design Example (I): Design the Welded Flange Plate connection of the frame with a beam size of W24×131 and a column size of W14×233 as shown in Figure 10(a) assuming the centerline of the plastic hinge in the beam is located at a distance of $l_p + d_b/2$ from the face of the column.

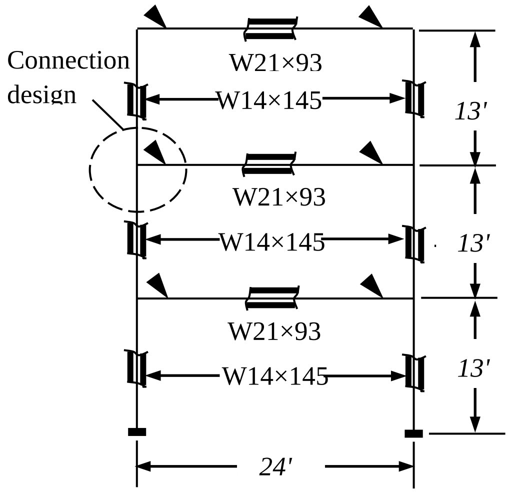
Design Example (II): Design the Welded Flange Plate connection of the frame with a beam size of W24×131 and a column size of W14×233 as shown in Figure 10(a) assuming the centerline of the plastic hinge in the beam is located at a distance of l_p from the face of the column.

Design Example (III): Design the Welded Flange Plate connection of the frame with a beam size of W21×93 and a column size of W14×145 as shown in Figure 10(b) assuming the centerline of the plastic hinge in the beam is located at a distance of $l_p + d_b/2$ from the face of the column.

Design Example (IV): Design the Welded Flange Plate connection of the frame with a beam size of W21×93 and a column size of W14×145 as shown in Figure 10(b) assuming the centerline of the plastic hinge in the beam is located at a distance of l_p from the face of the column.



(a)The steel moment frame for Examples (I) and (II)



(b) The steel moment frame for Examples (III) and (IV)

Figure 10. Steel Moment Frames for Design Examples (I), (II), (III), and (IV)

Design Example (I):

Referring to Figure 10(a), assume that the centerline of the plastic hinge in the beam is located at a distance of $l_p + d_b/2$ from the face of the column. The section properties of the W24×131 beam are: d_b = 24.5 in., b_f = 12.9 in., S_x = 329 in.^{3}, Z_x = 370 in.^{3}, and t_{bf} = 0.96 in. The section properties of the W14×233 column are: d_c = 16.0 in., t_{wc} = 1.07 in., and t_{cf} = 1.72 in. [2]. ASTM A992 steel is used for the beams

and columns, A572 Gr. 50 steel is used for the flange plates, and E7018 electrode is used for the welds.
The design procedure for the Welded Flange Plate connection is demonstrated as follows:

(1) Calculate the probable peak plastic hinge moment at the centerline of the plastic hinge:
The beam W24×131 is a compact section ($b_f/2t_f \le \lambda_p$ for the flange and $h/t_w \le \lambda_p$ for the web, where λ_p is the limiting slenderness parameter for a compact element; $\lambda_p = 0.38\sqrt{E/F_y}$ for the flange and $\lambda_p = 3.76\sqrt{E/F_y}$ for the web). Also, assume that the unbraced length of the compression flange of the beam is less than L_p (where $L_p = 1.76 r_y \sqrt{E/F_y}$, the limiting laterally unbraced length for the full plastic flexural strength of the beam [8]). Therefore, $Z_e = Z_x$ (where Z_x is the plastic section modulus about the major axis of the beam). From Eqs. (1) & (2), one has:

$$M_{pr} = \left(\frac{F_y + F_u}{2F_y}\right) R_y Z_e F_y$$

$$= \left(\frac{50+65}{2(50)}\right)(1.1)(370)(50) = 23{,}402.5 \text{ kip-in}$$

(2) Referring to Figure 3 and assuming l_p = 16 in., one has $x = l_p + (d_b/2) = 16 + (24.5/2) = 28.25$ in., and $L' = L - d_c - 2(l_p + d_b/2) = 324 - 16 - 2(28.25) = 251.5$ in. The probable peak flexural strength demand at the face of the column may be determined using Eq. (5):

$$M_f = M_{pr} + \left(\frac{M_{pr}}{\left(\frac{L'}{2}\right)}\right)(x)$$

$$= 23402.5 + \left(\frac{23402.5}{\left(\frac{251.5}{2}\right)}\right)(28.25) = 28{,}660 \text{ kip-in}$$

The probable peak flexural strength demand at the centerline of the column due to the formation of the plastic hinge in the beam may be determined using Eq. (7):

$$M_c = M_{pr} + \left(\frac{M_{pr}}{\left(\frac{L'}{2}\right)}\right)\left(x + \frac{d_c}{2}\right)$$

$$= 23402.5 + \left(\frac{23402.5}{\left(\frac{251.5}{2}\right)}\right)\left(28.25 + \frac{16.0}{2}\right)$$

$$= 30{,}149 \text{ kip-in}$$

(3) The width of the top and bottom flange plates may be determined using Eq. (8):

$b_p \ge b_f + 2\,t_f = 12.9 + 2(0.96) = 14.82$ in.
Use b_p = 15 in.

(4) Determine the thickness of the top and bottom flange plates: From Eq. (10), one has:

$$t_p = \frac{M_f}{R_y F_{yp} b_p (d_b + t_p)} = \frac{28660}{(1.1)(50)(15)(24.5 + t_p)}$$

From the above equation, one has t_p = 1.344 in.
Use t_p = 1⅜ in. ≥ 1.344 in. OK

(5) Design the fillet weld: Referring to Figure 6, note that since the thickness of the flange of the beam is ≥ ¼ in., the maximum fillet weld size that may be used along the edge of the flange is $t_{w(\max)} = t_{bf} - {}^1/_{16}$ in. = 0.96 in. – ${}^1/_{16}$ in. = 0.8975 in. Therefore, a size of ${}^7/_8$ in. (≤ 0.8975 in.) is used for the fillet weld along the flange of the beam. Moreover, since the thickness of the flange plate is ≥ ¼ in., the maximum fillet weld size that may be used along the end of the flange plate is $t_{w(\max)} = t_p - {}^1/_{16}$ in. = $1^3/_8$ in. – ${}^1/_{16}$ in. = 1.3125 in. Therefore, a size of $1^5/_{16}$ in. (≤ 1.3125 in.) is used for the fillet weld along the end of the flange plate.
Referring to Figure 7, the nominal design strength of the weld, F_w, may be calculated using Eq. (11):

$$F_w = [(0.707t_w)(l_p - 1.5'')(0.60)F_{EXX}] \times 2 + (0.707t_w)(b_f - 2'')(0.60F_{EXX})$$

= [(0.707)(0.875)(16–1.5)(0.60)(70)](2) +(0.707)(1.3125)(12.9–2)(0.60)(70)=1178.3 kips

Furthermore, referring to Figure 7, the flexural strength at the column face, developed by the fillet welds, may be computed using Eq. (12):

$$M_f = F_w \times d_b = 1178.3 \times 24.5 = 28868 \text{ kip-in} \geq 28660 \text{ kip-in}$$

Since the flexural strength at the face of the column developed by the fillet welds is larger than the probable peak flexural strength demand at the face of the column due to the formation of the plastic hinge in the beam, the weld sizes provided are adequate.

(6) Check whether beam flange continuity plates are required.
From Eq. (13), one has:

$$0.4\sqrt{1.8 b_p t_p \frac{F_{yb}R_{yb}}{F_{yc}R_{yc}}} = 0.4\sqrt{1.8(15)(1.375)\frac{50(1.1)}{50(1.1)}} = 2.437"$$

From Eq. (14), one has: $\dfrac{b_p}{6} = \dfrac{15}{6} = 2.5\,\text{in.}$

Since 2.5 in. > 2.437 in., the value 2.5 in. controls. Also, since t_{cf} = 1.72 in. < 2.5 in., continuity plates (with a thickness equal to that of the flange plates) across the column web are required.

(7) Check the strength of the panel zone and determine whether web doubler plate(s) is/are required:
From Eqs. (15) and (16), one has:

$$t_{pz} = \frac{\left(\dfrac{1}{C_{pr}\dfrac{Z_{be}}{S_b}}\right) M_c \left[\dfrac{h-(d_b+t_p)}{h}\right]}{0.9(0.6)F_{yc}R_{yc}d_c(d_b+t_p)} = \frac{\left(\dfrac{1}{1.15\left(\dfrac{370}{329}\right)}\right)(30149)\left(\dfrac{156-(24.5+1.375)}{156}\right)}{0.9(0.6)(50)(1.1)(16.0)(24.5+1.375)}$$
$$=1.581 \text{ in.}$$

Since $t_{pz} > t_{wc}$, the required minimum thickness of the doubler plate is: $t_{pz} - t_{wc}$ = 1.581 – 1.07 = 0.511 in. Providing a ⅜ in. doubler plate on each side of the panel zone, the total thickness of doubler plates provided = 2 ×⅜ in. = 0.75 in. > 0.511 in. OK

Referring to Figure 9, the panel zone depth between continuity plates may be computed as $d_z = d_b$ = 24.5 in., and panel zone width between column flanges may be computed as $w_z = d_c - 2t_{cf}$ = 16.0 – 2(1.72) = 12.56 in. Furthermore, from Eq. (17), one has $(d_z+w_z)/90$ = 0.412 in. Since the doubler plate thickness, t = ⅜ in. < 0.412 in., local buckling of the doubler plate must be prevented by using plug welds joining the column web and the doubler plate. Using the plug welds, the total thickness of the panel zone may be computed as $t = t_{wc}$ + ⅜ + ⅜ = 1.82 in., which satisfies the requirement as specified in Eq. (17).
Referring to Figure 9(b), the length of the doubler plate may be computed as $l_{dp} = d_b + 2t_p$ + 5k = 24.5 + 2(1⅜) + 5(3) = 42.25 in. (use 42 in.)

(8) Design the shear tab.
The length of the shear tab may be computed as d_b – 2k – 2" = 24.5" – 2(2¼") – 2" = 18 in. The thickness of the shear tab should match that of the beam web, which is $^5/_8$ in.
The design process of Design Example (I) demonstrated above results in the Welded Flange Plate connection details shown in Figure 11. The design processes of Design Examples (II), (III), and (IV) are also carried out by following the same design approach used by Design Example (I). Their results are summarized in Tables 1 and 2.

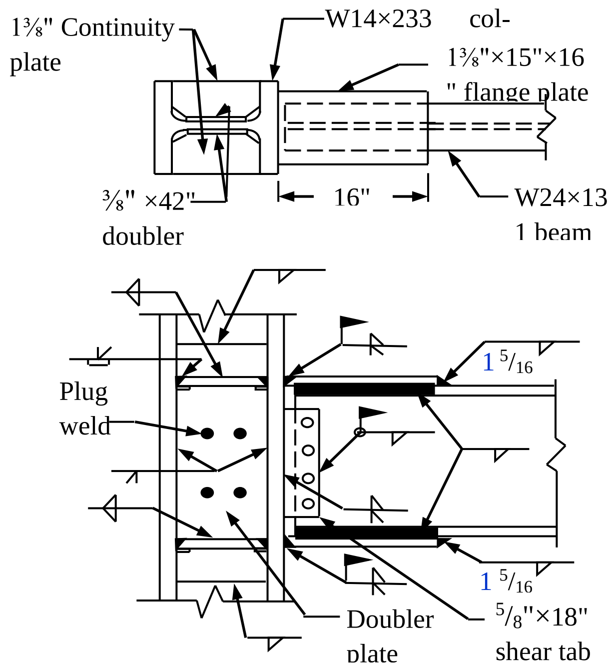


Figure 11. Welded Flange Plate connection details derived from Design Example (I)

Table 1. Probable peak flexural strength demand at the face of the column derived from Design Examples (I), (II), (II), & (IV)

Design Example	Beam size	Bay width	Assumed centerline of plastic hinge location (measured from the outer face of the column flange)	Probable peak flexural strength demand at the face of the column (kip-in)
(I)	W24×131	27'-0"	$l_p + d_b/2$	28,660
(II)	W24×131	27'-0"	l_p	25,835
(III)	W21×93	24'-0"	$l_p + d_b/2$	16,928
(IV)	W21×93	24'-0"	l_p	15,294

Table 2. Welded Flange Plate connection details derived from Design Examples (I), (II), (II), and (IV)

Design Example	Dimensions of flange plate (thickness × width × length)	Longitudinal fillet weld size	Transverse fillet weld size
(I)	1⅜"×15"×16"	7/8"	1 5/16"
(II)	1¼"×15"×14½"	7/8"	1 3/16"
(III)	1 5/16"×10½"×13"	13/16"	1¼"
(IV)	1 3/16" ×10½"×11 3/4"	13/16"	1⅛"

Table 2. (Continued) Welded Flange Plate connection details derived from Design Examples (I), (II), (II), and (IV)

Design Example	Dimensions of doubler plate on each face of the column web (thickness × length)	Continuity plate thickness	Dimensions of shear tab (thickness × length)
(I)	⅜"×42"	1⅜"	5/8"×18"
(II)	⅜"×42"	1¼"	5/8"×18"
(III)	¼"×36"	1 5/16"	9/16"×16½"
(IV)	¼"×36"	1 3/16"	9/16"×16½"

4 NONLINEAR STATIC FINITE ELEMENT ANALYSIS FOR WELDED FLANGE PLATE CONNECTIONS

The assumed locations of the centerline of the plastic hinge for the Welded Flange Plate connections specified in Design Examples (I), (II), (III), and (IV) may be verified using the nonlinear static finite element analysis method. Figure 12 illustrates the size, length, and boundary condition of the beam and column for the finite element analysis computer modeling for Design Examples (I), (II), (III), and (IV). The frame in Figure 12 is a portion of the multistory moment frame subjected to lateral forces, as shown in Figure 8(a). The end(s) of the beam and column in Figure 12 correspond to the deformation inflection point(s) in the beam and column in Figure 8(a) since the magnitude of the bending moment at these points equal zero.

Referring to the computer model for Design Example (I) shown in Figure 12(a), the magnitude of the probable peak lateral force capacity P (which corresponds to the probable peak flexural strength demand, M_f, at the interface of the column and the flange plates) to be applied to the free end of the column may be computed using the following equation: $M_f = [P(2)(78'')/162''][162''-(d_c/2)]$

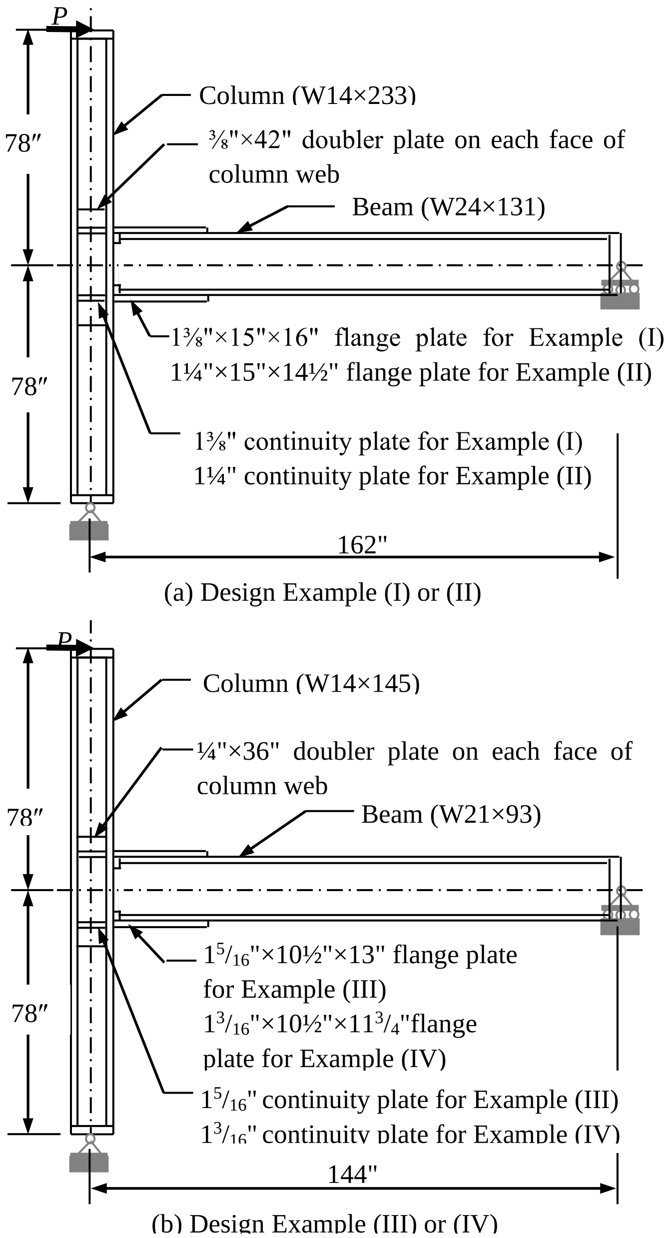


Figure 12. The size, length, and boundary condition of the beam and column for the finite element analysis computer modeling for Design Examples (I), (II), (III), and (IV)

From Table 1, one has M_f = 28,660 kip-in. Also, since the column depth d_c = 16.0 in. for Design Example (I), the above equation results in P = 193.26 kips. Repeating the procedure mentioned above, one has P = 174.21 kips, 114.39 kips, and 103.35 kips for Design Examples (II), (III), and (IV), respectively.

The lateral load P is applied in one direction in increments over the duration of a pseudo time of 100 seconds at the free end of the column, as shown in Figure 12. The computer models for Design Examples (I), (II), (III), and (IV) were analyzed using the computer software NISA/DISPLAY [12].

The nonlinear static analysis takes both the material and geometric nonlinearities into account. The elastic, linear hardening, true stress-strain curve for ASTM A992 steel (to be used for the beam and column) is shown in Figure 13; the curve was derived from Bartlett et al. [13]. The elastic, linear hardening, true stress-strain curve for ASTM A572 Gr.50 steel (to be used for the flange plates and doubler plates) is shown in Figure 14; the curve was derived from Salmon et al. [14]. The elastic, linear hardening, true stress-strain curve for E7018 electrode (to be used for welding) is shown in Figure 15; the curve was derived from the Lincoln Electric Company [15].

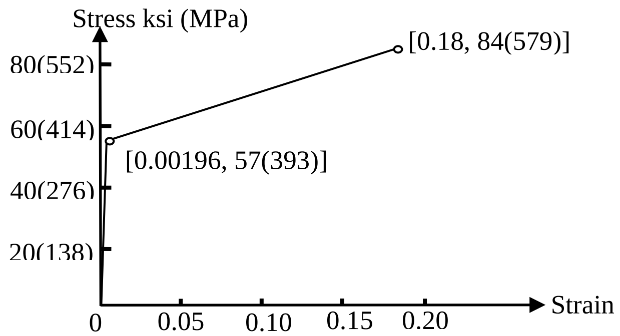


Figure 13. Elastic, linear hardening, true stress-strain curve for A992 steel

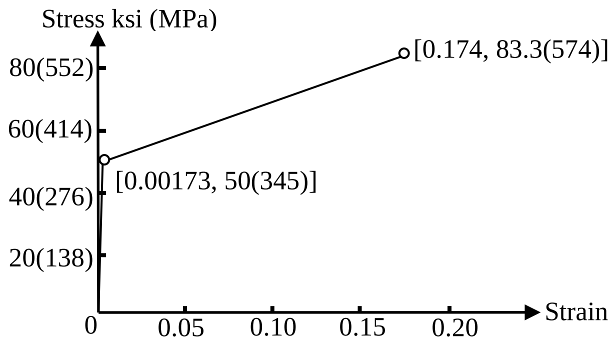


Figure 14. Elastic, linear hardening, true stress-strain curve for A572 Gr. 50 steel

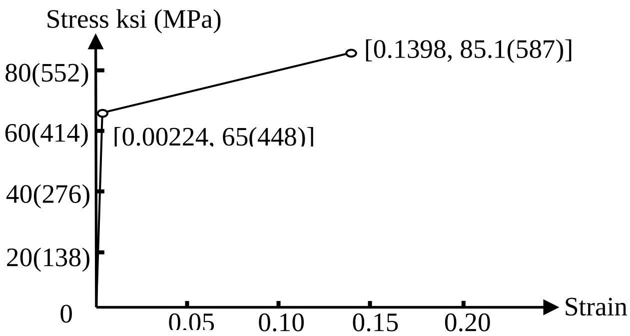


Figure 15. Elastic, linear hardening, true stress-strain curve for E7018 electrode

Referring to Figure 13, yielding occurs in A992 steel when the von-Mises stress reaches 57 ksi (based on the concept that the von-Mises stress is the yielding criterion for isotropic materials [16]); furthermore, fracturing occurs in A992 steel when the first-principal stress reaches 84 ksi (based on the concept that the first-principal stress is the fracturing criterion for isotropic materials [16])

The results of the finite element for Design Examples (I) through (IV) are shown in Figures 16 through 23 and are discussed below:

Figure 16 shows that at the time step of 88.4 seconds, the first-principal stress in the beam has just passed 84 ksi (which is the true ultimate tensile strength of A992 steel). At this time step, fracturing in the beam occurs and the flexural strength demand at the face of the column may be determined to be M_f = 28,660 kip-in × 0.884 = 25,335 kip-in. Furthermore, at this time step, the corresponding lateral force applied at the free end of the column may be determined to be P = 193.26 kips × 0.884 = 170.84 kips.

Figure 17 shows the von-Mises stress distribution in the beam at the time of fracturing (that is, at the time step at which the first-principal stress has just passed the ultimate tensile strength of the beam). Since the yielding areas (where the von-Mises stresses are larger than the true yielding stress of 57 ksi) have extended from the flange into the web of the beam, a plastic hinge has been formed at this time step. By observation, the centerline of the plastic is located at a distance of about 8 in. (about 33% of the beam depth) from the end of the flange plates.

Table 3 summarizes the results obtained from Figures 16 through 23, which are the results of the finite element analyses for Design Examples (I), (II), (III), and (IV).

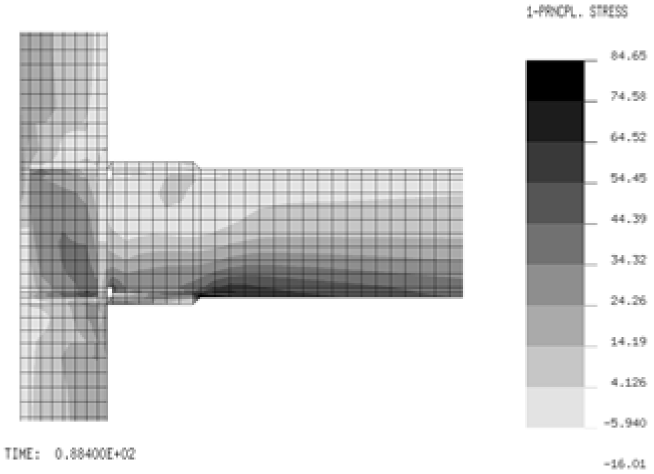


Figure 16. First-principal stress distribution for Design Example (I)

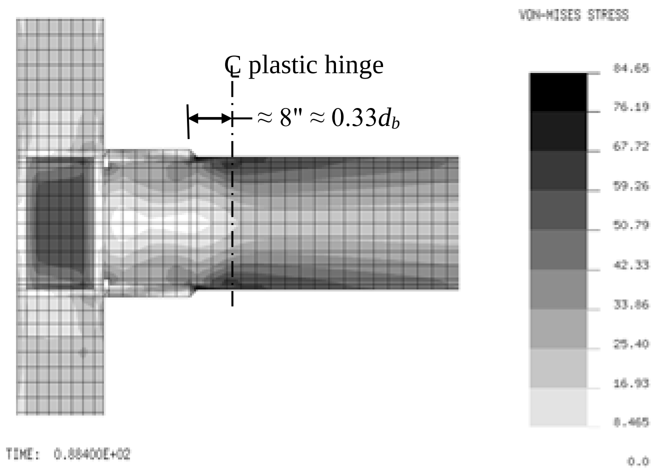


Figure 17. Von-Mises stress distribution for Design Example (I)

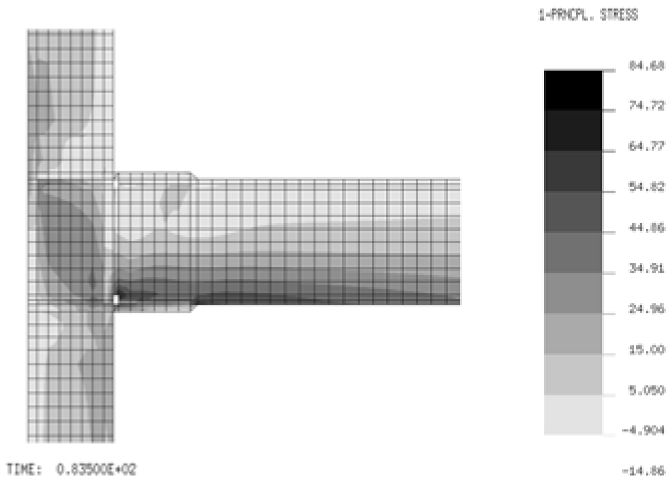


Figure 18. First-principal stress distribution for Design Example (II)

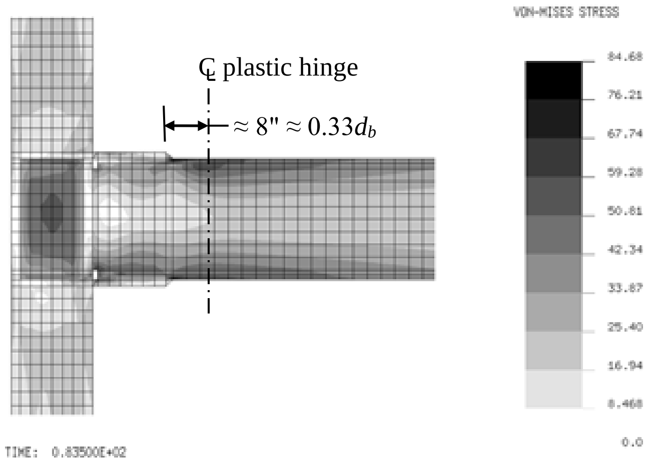


Figure 19. Von-Mises stress distribution for Design Example (II)

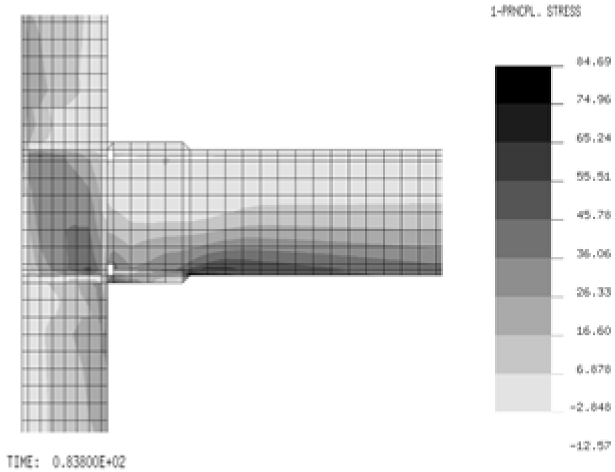


Figure 20. First-principal stress distribution for Design Example (III)

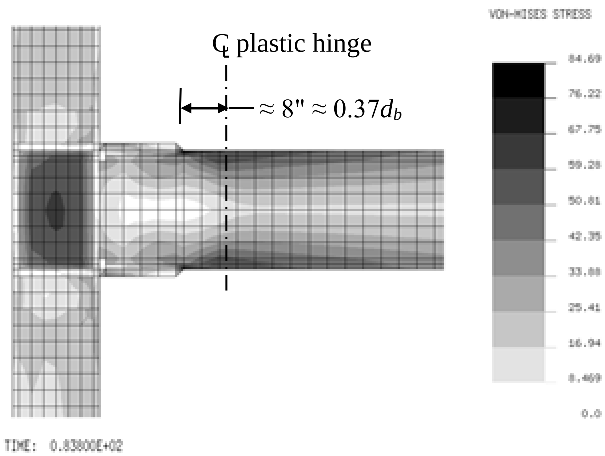


Figure 21. Von-Mises stress distribution for Design Example (III)

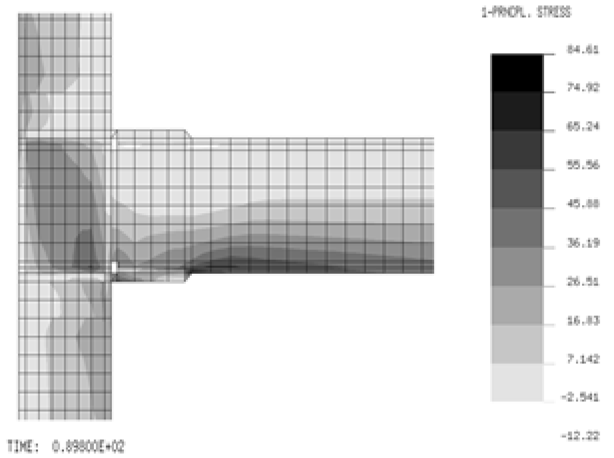


Figure 22. First-principal stress distribution for Design Example (IV)

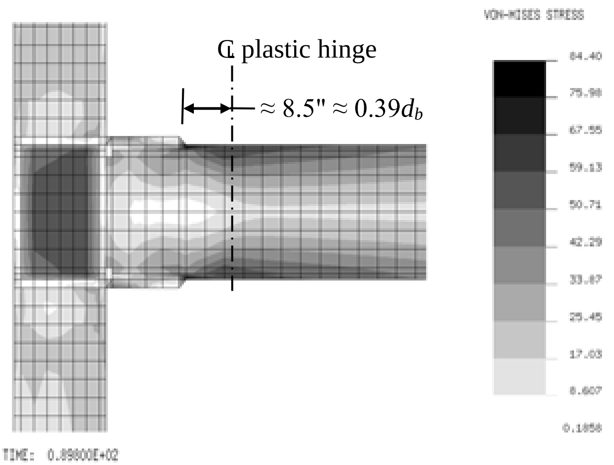


Figure 23. Von-Mises stress distribution for Design Example (IV)

Table3. A summary of the results derived from the finite element analysis for Design Examples (I), (II), (III), and (IV)

Design Example	Probable peak lateral force capacity (kips)	Actual lateral force capacity (kips)	Actual centerline of plastic hinge location (measured from the outer face of the column flange)
(I)	193.26	170.84	$l_p + 0.33\ d_b$ = 16" + 0.33×24.5" = 24.09"
(II)	174.21	145.47	$l_p + 0.33\ d_b$ = 14.5" + 0.33×24.5" = 22.59"
(III)	114.39	95.86	$l_p + 0.37\ d_b$ = 13" + 0.37×21.6" = 20.99"
(IV)	103.35	92.81	$l_p + 0.39\ d_b$ = 11.75" + 0.39×21.6" = 20.17"

5 RESULTS AND DISCUSSION

Tables 1 and 2 summarize the results of the four WFP Design Examples. Table 1 indicates that for the same beam size and beam length, the longer the assumed distance between the face of the column and the plastic hinge location, the higher the probable peak flexural strength demand at the face of the column. Table 2 indicates that for the same beam size and beam length, the longer the assumed distance between the face of the column and the plastic hinge location, the longer the top and bottom flange plates that are needed (that is, the stronger the WFP connection is needed).

Table 3 summarizes the results derived from the finite element analyses for the four design examples. The table indicates that (1) The distances measured from the face of the column to the actual location of the centerline of the plastic hinge for the four design examples ranged from l_p + 0.33 d_b to l_p + 0.39 d_b (where l_p is the length of the flange plate and d_b is the depth of the beam); these distances are longer than l_p (the assumed location for Design Examples (II) and (IV)) but are shorter than l_p + 0.5 d_b (the assumed location for Design Examples (I) and (III)). (2) For the same beam size and beam length, the longer the distance between the face of the column and the plastic hinge location, the larger the lateral force capacity applied at the free end of the column.

6 CONCLUSIONS

Since the location of the plastic hinge depends upon the beam size, the beam length, as well as the length of the flange plates being used for the WFP connection, one has to use an assumed plastic hinge location to begin designing a WFP connection. Four examples for the design of the WFP connections are presented in this paper. The assumed plastic hinge locations used in the design examples were also verified using the nonlinear static finite element analysis method. The following conclusions are inferred from the results of the four design examples and the finite element analyses for these design examples: (1) The designed WFP connection details presented in Design Example (I) were based on the assumption that the plastic hinge is located at a distance of l_p + 0.5 d_b from the outer face of the column flange. Since the assumed distance is longer than the actual distance (l_p + 0.33 d_b), the designed WFP connection is capable of producing a higher flexural strength capacity than the actual probable peak flexural strength demand (developed by the formation of the plastic hinge at its actual location). Therefore, the designed WFP connection details presented in Design Example (I) are adequate. (2) The designed WFP connection details presented in Design Example (II) were based on the assumption that the plastic hinge is located at a distance of l_p from the outer face of the column flange. Since the assumed distance is shorter than the actual distance ($l_p + 0.33\ d_b$), the designed WFP connection is not capable of producing a higher flexural strength capacity than the actual probable peak flexural strength demand (developed by the formation of the plastic hinge at its actual location). Therefore, the designed WFP connection details presented in Design Example (II) are not adequate. (3) The designed WFP connection details presented in Design Example (III) were based on the assumption that the plastic hinge is located at a distance of $l_p + 0.5\ d_b$ from the outer face of the column flange. Since the assumed distance is longer than the actual distance ($l_p + 0.37\ d_b$), the designed WFP connection is capable of producing a higher flexural strength capacity than the actual probable peak flexural strength demand (developed by the formation of the plastic hinge at its actual location). Therefore, the designed WFP connection details presented in Design Example (III) are adequate. (4) The designed WFP connection details presented in Design Example (IV) were based on the assumption that the plastic hinge is located at a distance of l_p from the outer face of the column flange. Since the assumed distance is shorter than the actual distance ($l_p + 0.39\ d_b$), the designed WFP connection is not capable of producing a higher flexural strength capacity than the actual probable peak flexural strength demand (developed by the formation of the plastic hinge at its actual location). Therefore, the designed WFP connection details presented in Design Example (IV) are not adequate.

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