Design of Free Flange Moment Connection

JAEHYUNG CHOI, BOZIDAR STOJADINOVIC, and SUBHASH C. GOEL

ABSTRACT

The Free Flange connection is a new public-domain beam-to-column moment connection developed at The University of Michigan. This connection is designed to alleviate excessive local deformation and shear force overload of beam flanges in pre-Northridge fully-restrained steel moment connections. These design goals were achieved by cutting the web of the beam back and away from the column, thus creating portions of the beam flanges that are not constrained by the web. Such free portions of beam flanges significantly reduce connection deformation constraints; allow the flange steel to yield freely; and help to redirect most of the shear force back into the web connection. Seven full size Free Flange connection specimens were tested to validate the design concept, check the design procedure, and prequalify the connection for use in practice. The connection was prequalified for the tested beam and column sizes, with all specimens exceeding 0.04 radian total rotations before failure, and behaving in a ductile manner. The original Free Flange connection design procedure was modified based on the observed specimen behavior. The new design procedure is presented and compared to the Free Flange connection design procedure in the FEMA 350 Design Guidelines.

INTRODUCTION

Design of fully-restrained beam-to-column steel moment connections has been traditionally based on classical beam theory assumptions: the beam flanges transfer the axial moment-couple forces, while the web resists the entire shear force. Recent analytical and experimental research done at The University of Michigan (Goel, Stojadinovic, and Lee, 1997; Stojadinovic, Goel, Lee, Margarian, and Coi, 2000) has shown that strain distribution and force flow in the beam near the beam-column connection are very different from those given by the classical beam theory. This finding is in accordance with St. Venant's principle: boundary conditions of the conventional pre-Northridge moment connection cause severe local deformations and major changes in the force flow in the connection region. In particular: 1) beam flanges are subject to double-curvature local bending caused by shear deformation of the connection; and 2) the beam web becomes virtually stress-free, resulting in beam flange overload as beam shear force shifts from the web to the flanges. Such excessive local curvature and global shear overload exist regardless of the quality of the beam flange welds and smoothness of the weld access hole detail: they are a consequence of connection configuration.

The Free Flange connection was developed at The University of Michigan with the goal to alleviate curvature concentration and shear overloading in beam flanges (Choi, Goel, and Stojadinovic, 2000). This connection design is in the public domain. The first Free Flange connection design procedure, called UofM98, was developed at The University of Michigan in 1998 to design the connection specimens used in prequalification tests. Seven Free Flange connection specimens were tested within the SAC Phase II Steel Project in order to validate the design concept, check the design procedure, and prequalify the connection for use in practice. These tests prequalified the Free Flange connection for the tested sizes, and provided the experimental data to improve the original connection design procedure. SAC affected one set of modifications while preparing the FEMA 350 Design Guidelines, resulting in a FEMA 350 Free Flange connection design procedure. Independently, the authors improved the UofM98 procedure to develop a new UofM-2000 Free Flange connection design procedure. The Free Flange connection design concept and the three Free Flange design procedures are presented and compared in this paper.

FREE FLANGE CONNECTION CONCEPT

Ductile behavior of a fully-restrained moment connection is achieved not only by using fracture-resistant welds and ductile weld access hole details, but also by providing con-

Jaehyung Choi is engineer, DASSE Design Inc., San Francisco, CA.

Bozidar Stojadinovic is assistant professor, University of California, Berkeley, Department of Civil and Environmental Engineering, Berkeley, CA.

Subhash C. Goel is professor, University of Michigan, Department of Civil and Environmental Engineering, Ann Arbor, MI.

straint-free conditions for deformation in the connection region (Stojadinovic et.al, 2000). The Free Flange connection is a new connection configuration designed to reduce the detrimental effect of connection boundary condition constraints. This connection design adopts the fracture-critical improvements in weld metal toughness, welding procedure and quality control, as well as the design of weld and access hole details as prescribed in FEMA 267 (FEMA, 1995), FEMA 267A (FEMA, 1997) and FEMA 350 (FEMA, 2000) documents. In addition, this design strives to alleviate severe local deformation and shear force overload of the beam flanges found in conventional pre-Northridge fully-restrained steel moment connections by cutting the web of the beam back and away from the column, thus creating portions of the beam flanges that are not constrained by the web (Figure 1). The length of the web cutback, i.e., the distance between the column face and the beginning of the tapered web cut, is called the free flange length.

The free flange portions of beam flanges alleviate local deformation in three ways. First, the web of the beam is cut back to allow "free" bending and axial deformation of the flanges. As shown in Figure 2, the flange is bending locally as a fixed-ended beam due to the overall shear deformation of the beam in the connection region. Free flange length is selected to reduce the concentration of curvature produced by such local bending. Longer free flange length results in smaller concentration of curvature in the flange. In comparison, a short free length created by a conventional weld access hole detail results in a severe concentration of curvature. Second, the deformation constraints imposed on the beam flange by the stiff column flange are relieved, even though local strain and stress states in the flange still depend on the length-to-thickness and width-to-thickness ratios of the flange (Yang and Popov, 1996). Third, the free portion of the flange allows yielding of flange steel to

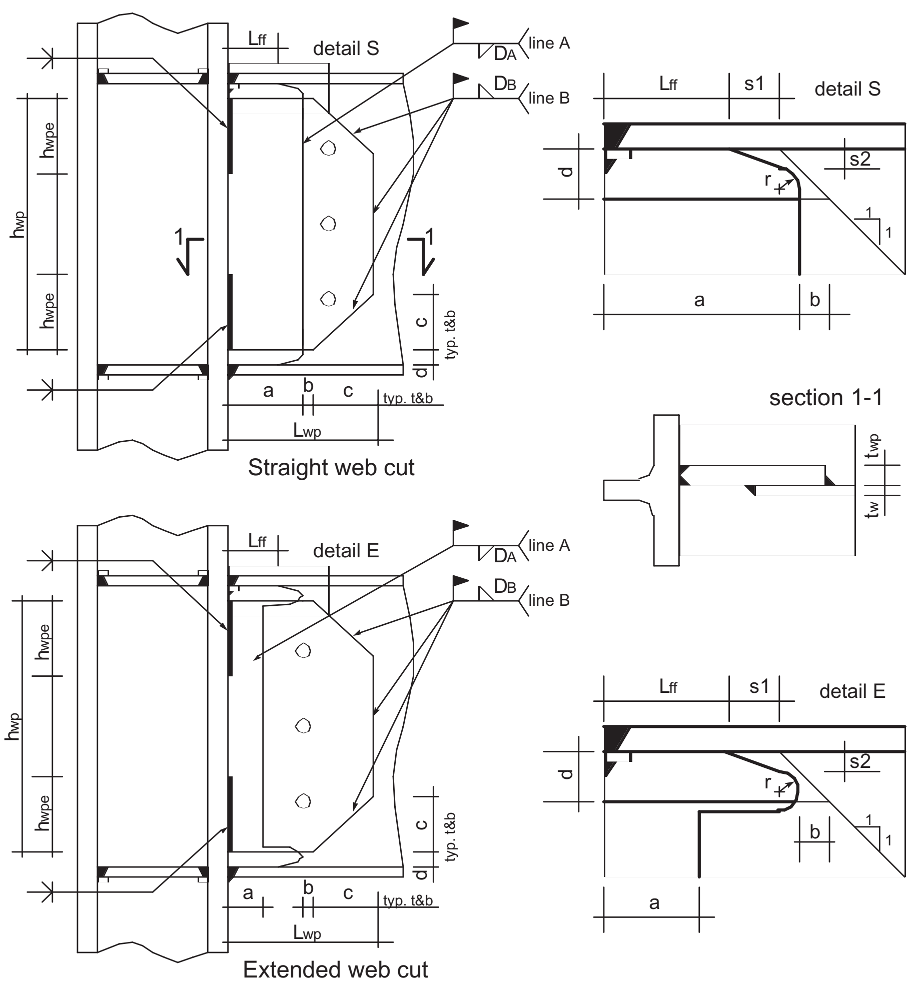

Fig. 1. Configuration of the Free Flange connection. Details S and E are designed following Figure 3-5 in FEMA 350. In particular, slope s2/s1 *must be smaller than 0.5. Radius* r *must be larger than 3/8-in. Slope and radius must be finished following Note 6 in Figure 3-5 of FEMA 350. Distance* b *must be larger than 1 in., while clearance distance* d *must be larger than 2 in. The 1-1 slope of the diagonal edge of the web plate must meet the flange outside lengths* L_{ff} *and* s1*. Effective web height* h_{wpe} *is usually taken as* $d_b/4$*, but must be between* $d_b/5$ *and* $d_b/4$*. Length of the web plate* L_{wp} *should not be larger than* $d_b/2$*.*

"flow" in a less constrained manner along the free flange, into the beam web and away from the connection region.

The second important feature of the Free Flange connection is its force transfer mechanism. The purpose of designing the force flow is to bring as much of the shear force back into the beam web as possible, and thus relieve the flanges. The Free Flange connection consists of three elements: the top flange, the beam web and a web plate connecting it to the column, and the bottom flange. These elements have the same vertical displacement when the beam undergoes shear deformation. Thus, they can be represented by three shear springs in a parallel shear-spring model shown in Figure 3. In this figure, V is the total applied shear force, V_{wp} is the shear force acting on the web plate, and V_f denotes the shear forces acting on the flanges (assuming they are identical as is the case in typical symmetric beam sections). Assuming that the free flanges and the beam web in the connection region are acting as fixed-end beams, distribution of the total shear force among the three connection elements can be calculated using shear force stiffness ratios as follows:

$$\frac{V_f}{V} = \frac{K_f}{2K_f + K_{wp}} = \frac{E\left[\dfrac{b_f t_f^3}{L_f^3}\right]}{E\left[\dfrac{(6L_{wp} b_f t_f^3) + (L_f^3 h_{wp} t_{wp})}{(3L_{wp} L_f^3)}\right]} = \frac{1}{2 + \alpha^3 \dfrac{h_{wp} t_{wp}}{3L_{wp} b_f}} \tag{1}$$

$$\frac{V_{wp}}{V} = 1 - 2\frac{V_f}{V}$$

In these equations, K_f is the stiffness of one flange and K_{wp} is the stiffness of the web. Furthermore, L_{ff} is the free flange length, and b_f and t_f are width and thickness of the free flange, respectively, L_{wp} is the web plate length, and h_{wp} and t_{wp} are height and thickness of the web plate, respectively. The free flange length to thickness aspect ratio

$$\alpha = \frac{L_{ff}}{t_f} \tag{2}$$

is defined to account for the effect of the free flange length. Note that the shear force in a flange is proportional to $1/L^3_{ff}$ (Equations 1 and 2), decreasing hyperbolically as the free flange length increases. Thus, elongation of the free flange is quite effective in reducing the magnitude of flange shear force and simultaneously shifting beam shear into the web plate.

An analytical study of the Free Flange connection (Choi, Goel, and Stojadinovic, 2000) showed that beam flange deformation due to double-curvature bending is reduced to a sufficiently small level when the shear force in one free flange is less than 10 percent of the total connection shear. In comparison, each flange in a conventional pre-Northridge connection carries approximately 25 percent of the total connection shear (Goel, Stojadinovic, and Lee, 1997). Thus, force flow in the Free Flange connection should be

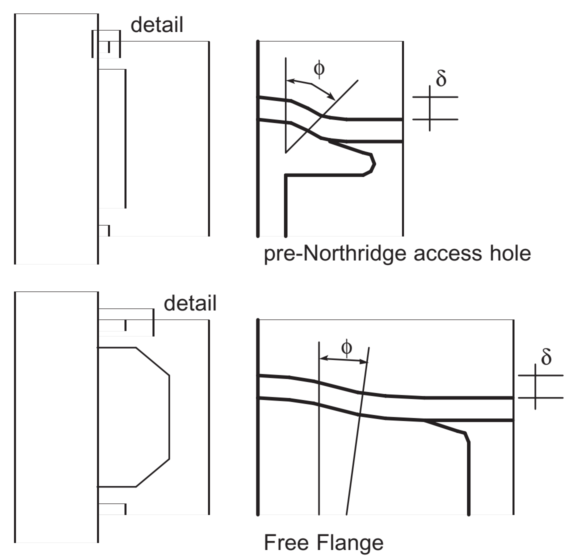

Fig. 2. Effect of flange elongation on its local bending.

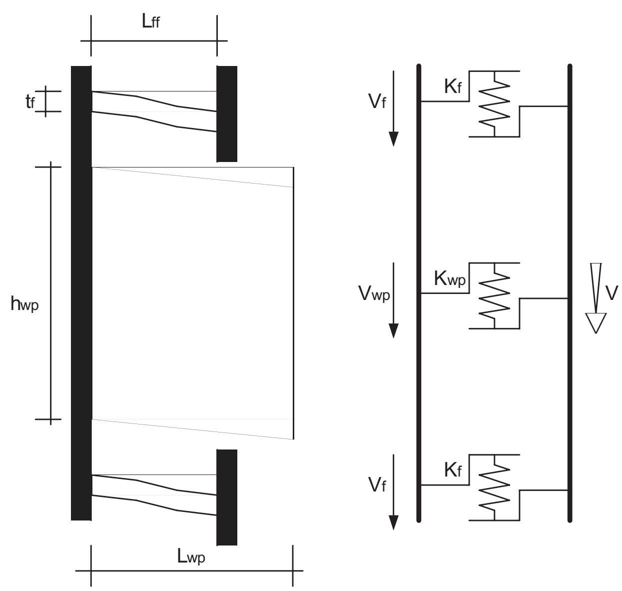

Fig. 3. Shear force transfer in Free Flange connections.

engineered to achieve a free flange shear force ratio V_f / V less then or equal to 0.1. This can be done by setting the free flange aspect ratio α, incorporated into Equations 1 and 3, larger than 5.0.

Even though it would be desirable to completely eliminate the shear force from beam flanges and make them function as axially-load-only elements, this is not possible. The free flange cannot be made too long because it may buckle in compression before the beam develops its plastic moment capacity. Buckling of the free flange can be postponed for after the beam reaches its nominal plastic moment capacity by making the flange stout enough so that its critical buckling stress equals to, or is just slightly less than, the flange yield stress. For purposes of design, the free flange can be treated as a compressed plate. The length of this plate is equal to the free flange length, and the boundary conditions may be assumed as hinged at edges perpendicular to the direction of the axial force and free at edges parallel to the axial force. Such boundary conditions allow for a first buckling mode shape shown in Figure 4, making the effective length factor for a free flange K = 1.0. Given the radius of gyration of the flange section $r_f = t_f / \sqrt{12}$, the relation between the free flange slenderness ratio λ_f and the free flange aspect ratio α is:

$$\lambda_f = \frac{KL_{ff}}{r_f} = \sqrt{12}\alpha$$

Using this slenderness ratio in AISC LRFD Equation E2-2 (AISC, 1999) and setting the free flange critical buckling stress to 95 percent of beam yield stress results in an upper bound for the slenderness ratio of approximately 20. This means that the free flange aspect ratio α should be smaller than 6.0.

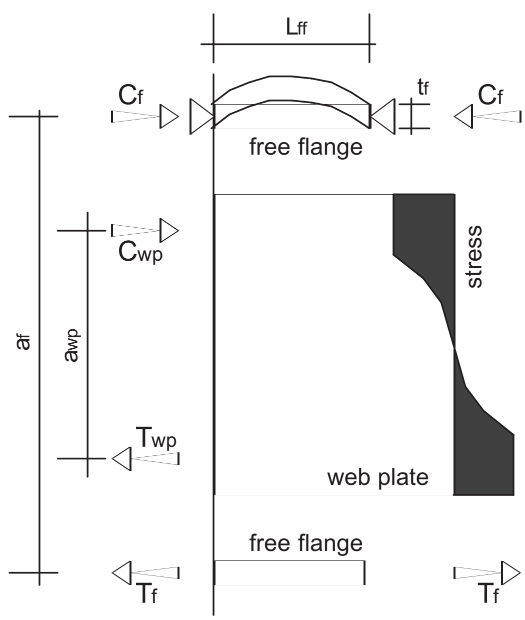

Fig. 4. Internal moment couples of the Free Flange connection.

Therefore, optimal free flange length is achieved when its aspect ratio α is in the range between 5.0 and 6.0. This range represents a good balance between reducing the flange shear force and potential for free flange buckling, as shown in Figure 5.

Shear Force in the Web Plate

The web plate carries its shear force V_{wp} as a plate fixed into a rigid column support. Such stiff support imposes deformation constraints on the web plate and causes significant changes in the force flow in this plate. Instead of classical beam theory stress distributions, normal, and particularly shear stresses concentrate in the top and bottom corners of the web plate: the middle of the web plate is virtually stress-free (Figure 6). Such non-uniform distribution of stresses is in accordance with St. Venant's Principle and the truss analogy force transfer model (Goel, Stojadinovic and Lee, 1997). Effective stress distributions in the web plate were extensively studied using finite element models (Choi, Goel

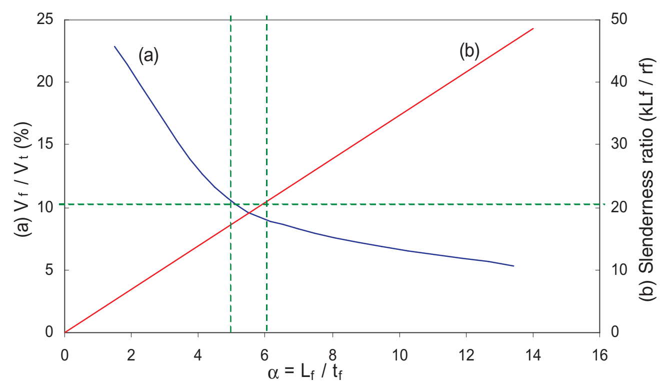

Figure 5. Selection of design values for the free flange aspect ratio α.

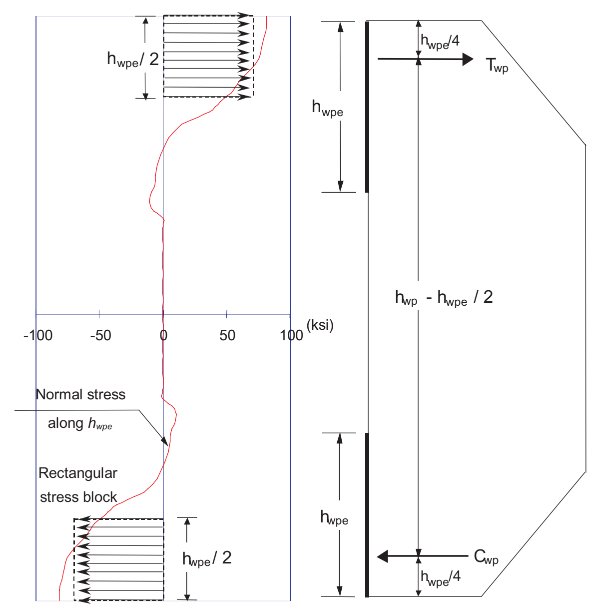

Fig. 6. Effective height of the web plate in Free Flange connections.

and Stojadinovic, 2000). First, smallest values of normal stress were found in tall web plates, ones that reach as close to the beam flanges as possible. Second, both normal and shear stress distributions in the top and bottom portions of the web plate were found to be reasonably uniform (Figure 6). Thus, an effective web plate height concept, with effective height h_{wpe} equal to between $1/6$ and $1/3$ of the web plate height, can be used to model the stress distribution in the active top and bottom portions of the web plate. Furthermore, it was shown that welding the web plate to the column along its entire height results in opposite-direction shear stresses in the middle of the web plate and a detrimental increase of stresses in web plate corners. Therefore, choosing the tallest possible web plate and welding it to the column only in the effective top and bottom portions was found to preserve the force transfer function of the web plate without unnecessarily increasing its shear stresses.

UofM98 FREE FLANGE CONNECTION DESIGN PROCEDURE

This design procedure has five parts: design of connection geometry; calculation of connection design forces; detailing of the beam flange welds; sizing of the web plate and web plate welds; and sizing of the connection panel zone and continuity plates.

Connection Geometry

Geometry of the Free Flange connection is shown in Figure 1. The free flange length, defined as the distance from the column face to the toe of the web cutback, is calculated as a product of flange thickness t_f and free flange aspect ratio α:

$$L_{ff} = \alpha t_f \tag{3}$$

Free flange aspect ratio should be between 5.0 and 6.0, with the free flange length rounded to the nearest $1/8$ in. The beam web cut in the k-line region should be made using a radius larger than $1/2$ in. and a slope prescribed for beam access holes in Figure 3-5 of FEMA 350.

The height of the web plate h_{wp} is computed so that the clear distance between the web plate and the beam flange, d, is not smaller than 2 in. to facilitate field welding of the web plate as well as welding of the backing bar reinforcement for the top flange. Thus:

$$h_{wp} = d_b - 2t_f - 2d \tag{4}$$

where d_b is the beam section depth.

Connection Design Forces

Free Flange connection force transfer mechanism is shown in Figures 3 and 4. Additional assumptions are: 1) beam plastic hinge forms at the column face; 2) normal forces forming the beam moment couple at the column face can be divided between the flanges and the effective portions of the web plate; 3) web plate resists the total shear force transferred through the connection; and 4) web plate shear is carried equally by effective web plate portions in its top and bottom half. The expected plastic moment developed in the beam plastic hinge M_{pe} is:

$$M_{pe} = C_{pr} Z_b R_y F_{yb} \tag{5}$$

where F_{yb} is nominal beam yield strength and R_y is the expected steel yield strength factor, defined in Section 2.6.2 of FEMA 350 and equal to 1.1 for A572 Gr. 50 and A992 steel. In addition, Z_b is the beam plastic section modulus and C_{pr} is the peak connection strength factor, defined in Section 3.2.4 of FEMA 350. In this procedure, C_{pr} is equal to 1.1.

Connection moment-couple tension T and compression C forces are computed by using a moment arm $a_f = d_b - t_f$ equal to the center-to-center distance between the beam flanges. Thus:

$$T = C = \frac{M_{pe} + M_g}{(d_b - t_f)}$$

where M_g is the beam moment at the plastic hinge (column face) due to gravity loads.

At the column face, moment-couple tension force T is divided between a tension force in the free flange T_f and a tension force T_{wp} acting along the effective height of the web plate in the tensile region of the connection. If the free flange length was adopted as suggested in this design procedure, distribution of normal stresses across the free flange width should be close to uniform (Choi, Goel and Stojadinovic, 2000). Then, tension force in the free flange is simply:

$$T_f = C_{prf} A_f R_y F_{yb} \tag{6}$$

where $A_f = b_f t_f$ is the area of the flange, and C_{prf} is the peak flange strength factor, equal to 1.1 in this procedure. Consequently, tension force in the web plate is:

$$T_{wp} = T - \phi_t T_f \tag{7}$$

using the AISC tension resistance factor ϕ_t =0.9. Moment couple compression force C can be divided in a similar way, making $C_f = T_f$ and $C_{wp} = T_{wp}$.

Beam flange compression force C_f is assumed to be equal to the beam flange tension force T_f, regardless of possible buckling of the free flange because the slenderness of the free flange, governed by the adopted free flange aspect ratio α, is sufficiently small.

Equation 7 can be derived using design assumption 1 and 2, stated above, as follows:

$$M_{pe} + M_g = M_f + M_{wp}$$
$$M_f = \phi_t T_f (d_b - t_f)$$
$$M_{wp} = (M_{pe} + M_g) - M_f$$
$$T_{wp} = \frac{M_{wp}}{(d_b - t_f)} = \frac{M_{pe} + M_g}{(d_b - t_f)} - \frac{M_f}{(d_b - t_f)} = T - \phi_t T_f$$

where M_f is the beam flange moment, and M_{wp} is the beam web moment at the column face, where the plastic hinge is assumed to be. The same moment arm, equal to $(d_b - t_f)$, was used for both moment couples.

Following design assumptions 3 and 4, the free flanges are assumed to carry no shear in the connection region. Thus, the web plate carries the total shear force:

$$V_{wp} = V_{pe} + V_g \tag{8}$$

a sum of the expected seismic shear force V_{pe} and the gravity shear V_g. The expected seismic shear force V_{pe} corresponding to beam plastic hinges forming at column faces is:

$$V_{pe} = \frac{M_{pe}}{\frac{L_b}{2}} \tag{9}$$

where L_b is the clear beam span between the columns. The top and the bottom halfs of the web plate carry equal parts of the web plate shear force V_{wp}.

Beam Flange Welds

Detailing of the complete-joint-penetration (CJP) field welds between the beam flanges and the column flange should be done as specified in Figure 3-8, Note 1 of FEMA 350. Namely, weld run-off tabs should be removed and ground smoothly. The backing bar on the bottom flange should be removed, the root of the weld backgouged, and reinforced with a fillet weld. The backing bar on the top flange CJP weld may be left in place, but then it must be reinforced from below using a fillet weld. All welds should be made using overmatching weld metal with sufficient toughness, as specified in Sections 3.3.2.4 and 3.3.2.5 of FEMA 350. Note that the weld metal CVN toughness requirement of 20 ft-lbs at 0°F is specified in the Errata for the FEMA 350 document.

Design of the Web Plate and its Welds

The web plate should be shaped to facilitate formation of a beam plastic hinge by allowing unrestricted spread of yielding from the flanges into the beam web and away from the column face. A tapered web plate has such a desirable shape, yet its geometry is simple enough for cost-effective fabrication. The height of the web plate (Equation 4) depends on the dimensions of the beam and the required minimum clearance for welding. The length of the plate depends on the free flange length, the required size of web plate welds, and the location of the 45° diagonally cut web plate sides. The diagonal side of the web plate should be located so that a line extending along the side meets the beam flange before the free flange length begins, i.e. in front of, or at least tangent to, the web radius cut, as shown in Figure 1. Such geometry is intended to protect the vulnerable web-to-flange transition area in the k-line region of the beam at the web cutback. Given the geometry of the web-to-flange transition in the beam k-line area prescribed in Figure 3-5 of FEMA 350, and adopted for the Free Flange connection, adequate position of the diagonally cut sides of the web plate can be achieved by extending the web plate away from the column by 1 in. beyond the edge of the radius cut, as shown in Figure 1 and in Figure 3-9 in FEMA 350.

The thickness of the web plate t_{wp} is calculated so that each half of the web plate can resist a combination of axial force T_{wp} (or C_{wp}) and shear force $0.5V_{wp}$ over its effective height h_{wpe}. The effective height of the web plate varies between $1/6$ and $1/3$ of the web plate height, based on the results of finite element analyses of boundary condition effects and force flow in the web plate. For typical beam sizes, this means that the effective web height is between $1/5^{th}$ and $1/4^{th}$ of the beam depth. Thus, the maximum value, equal to one quarter of the beam depth, is used:

$$h_{wpe} = d_b / 4 \tag{10}$$

Normal and shear stresses in the web plate are computed with respect to the effective web plate area $A_{wpe} = h_{wpe} t_{wp}$ and combined using von Mises' yield criterion. The expected yield strength factor R_y for plate steel is assumed to be equal to 1.0. The thickness of the web plate, not to be smaller than the thickness of the beam web, is computed conservatively using a strength reduction factor $\phi = 0.9$:

$$f_t^2 + 3f_v^2 = (\phi R_y F_y)^2; \quad R_y = 1.0$$
$$\left(\frac{T_w}{A_{wpe}}\right)^2 + 3\left(\frac{0.5V_{wp}}{A_{wpe}}\right)^2 = (\phi F_y)^2$$
$$A_{wpe}^2 = \frac{T_w^2 + 0.75V_{wp}^2}{(\phi F_y)^2} = (h_{wpe} t_{wp})^2$$

$$t_{wp} = \frac{\sqrt{\dfrac{T_{wp}{}^2 + 0.75 V_{wp}{}^2}{(\phi F_y)^2}}}{h_{wpe}} \tag{11}$$

CJP welds should be used to shop-weld the web plate to the column because these welds are essential for preventing progressive collapse of the beam after flange fracture. The tested Free Flange connections had double-bevel CJP web-plate welds, but other CJP welds are acceptable. These welds should be made using weld metal that overmatches the web plate and column base metal and meets the toughness requirements prescribed by FEMA 350. Two separate CJP welds should be used, starting from the top and bottom corners of the web plate and extending along each effective web plate height h_{wpe}, as shown in Figure 1. Such weld configuration is based on the effective height analysis discussed above. Eccentricity of the web plate is another reason for using CJP welds (Figure 1). This eccentricity occurs because beam flanges should be centered on the column axis to insure centric transfer of large flange forces into the column and avoid column torque. Therefore, the web plate cannot be centered on the column axis. The CJP welds insure that out-of-plane moment generated in the web plate is transferred into the column.

Fillet welds between the beam web and the web plate, along weld lines A (straight, far side) and B (tapered, near side) in Figure 1, are necessary to transfer axial and shear forces in the beam web to the web plate. These welds should, also, be made using overmatched and tough weld metal as specified in FEMA 350. It is conservative to size the fillet welds along lines A and B to resist a vector combination of normal force T_{wp} and shear force $0.5V_{wp}$ in each half of the web plate:

$$R_{wp} = \sqrt{T_{wp}^2 + (0.5 V_{wp})^2} = R_{wA} + R_{wB}$$

Resistance along weld line A is computed first, assuming that the entire length L_A is effective and welded using a fillet weld with the largest allowable size:

$$R_{wA} = \phi_v R_{wn} L_A = \phi_v (0.6 F_{EXX}) t_e L_A = 1.392\, D_A\, L_A$$

where $\phi_v = 0.75$ is the shear resistance factor, R_{wn} is nominal resistance of weld per unit length, t_e is effective weld thickness equal to 0.707 times the weld size, and F_{EXX} is nominal strength of the weld metal. If E70 weld metal is used, a shorthand AISC formula using the customary sixteenth-of-an-inch notation can be used. Required strength of weld line B is then:

$$R_{wB} = R_{wp} - R_{wA}$$

Available weld length L_B along weld line B extends to beam centerline because only one half of the web plate is considered. Thus:

L_B = (diagonal length) + (vertical length)
- (weld stop length = 0.5")

Then, weld size along line B can be computed from:

$$R_{wB} \le \phi_v R_{wn} L_B = \phi_v (0.6 F_{EXX}) t_e L_B = 1.392\, D_B\, L_B$$

Weld sizes along lines A and B are often limited by beam web (base metal) thickness. Weld lines A and B may also be treated as a fillet weld group under eccentric loading to meet fillet weld size limits and to size these welds more economically.

Design of Panel Zone and Continuity Plates

Seismic moment at the center of the column M_c is calculated by using distance L_c from the column axis to the beam inflection point multiplied by the seismic shear load V_{pe}, computed using Equation 8:

$$M_c = V_{pe} L_c$$

Then, the panel zone design shear force is:

$$V_{pzu} = \frac{M_c}{0.95 d_b} + V_{cg} - V_{cr}$$

where the seismic shear contribution is computed using an effective beam depth of $0.95d_b$, V_{cg} is the column gravity shear and V_{cr} is the column seismic shear force. The nominal shear capacity of the panel zone is computed by using Equation 9-1 from the AISC Seismic Provisions (AISC, 1997).

$$V_{pzn} = 0.6 F_{yc} d_c t_p \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p}\right)$$

where d_c is column depth, t_p is total column panel zone thickness, b_{cf} is the column flange width, and t_{cf} is the column flange thickness. The required thickness of the panel zone is then computed at column centerline to satisfy the design equation:

$$\phi_{pz} V_{pzn} \ge V_{pzu}$$

where ϕ_{pz} is the panel zone resistance factor. Panel zone should be detailed following the AISC Seismic Provisions (AISC, 1997).

Table 1. Free Flange connection specimens.

Specimen	Type	Beam	Web Cut	Column	Panel Zone (doubler plate)
FF-1	Exterior	W24x68	Straight	W14x120	Strong (5/8" Gr. 50)
FF-2	Exterior	W30x99	Straight	W14x176	Weak (none)
FF-3	Exterior	W30x99	Straight	W14x176	Strong (3/4" Gr. 50)
FF-4	Exterior	W30x124	Straight	W14x257	Medium (none)
FF-5	Exterior	W30x124	Straight	W14x257	Strong (1/2" Gr. 50)
UCSD	Exterior	W36x150	Extended	W14x257	Weak (1/4" Gr. 50)
UofT	Interior	W36x150	Extended	W14x398	Medium (1/2")

Table 2. Free Flange connection specimen performance in prequalification tests.

Specimen	Peak Values			Prequalification Values		C_{pr} From Eq. 5 and 12
	Total Drift	Total Plastic Rotation	Beam Contribution	Total Drift	Total Plastic Rotation	
FF-1	5%	4.2%	83%	5%	3.2%	1.07
FF-2	5%	3.7%	43%	5%	3.3%	1.07
FF-3	4%	3.4%	95%	3%	2.0%	0.91
FF-4	4%	2.5%	38%	4%	2.5%	1.14
FF-5	4%	2.7%	76%	4%	2.7%	1.26
UCSD	4%	2.7%	9%	3%	1.8%	1.10
UofT	5%	3.3%	33%	5%	3.3%	0.97

Rotation components are computed according to the SAC Test Protocol (SAC, 1997). Prequalification values reflect the state of the specimen at the end of two complete cycles to the same drift level without failure of excessive loss of resistance. Connection peak strength factor C_{pr} is computed using measured shear force values and Equations 5 and 12.

The panel zone resistance factor may be used to calibrate the panel zone design strength. A strong panel zone is expected when $\phi_{pz} \leq 0.75$ and is not expected to yield during a prequalification test. A medium panel zone, one that yields in a limited manner after yielding in the beam, is assumed to occur when $\phi_{pz} \leq 1.0$. A weak panel zone, one that yields before the beam yields in a prequalification test, is expected when $\phi_{pz} \geq 1.0$. The recommended value of the panel zone resistance factor for the Free Flange connection design in this design procedure is between 0.75 and 1.0.

The continuity plates of the connection should be designed and detailed following the AISC Seismic Provisions (AISC, 1997).

FREE FLANGE CONNECTION TESTS

Seven full size beam-to-column connection specimens were designed using the UofM98 procedure and tested for prequalification. Specimen beam and column sizes and classification of their panel zone strengths are listed in Table 1. Six of the specimens were exterior connection specimens without the floor slab: five at the University of Michigan (Choi, Goel, and Stojadinovic, 2000), and one at the University of California, San Diego (Gilton, Chi, and Uang, 2000). One interior connection specimen with a floor slab was tested at the University of Texas at Austin (Venti and Engelhardt, 2000). All tests were conducted following the SAC Test Protocol (SAC, 1997) and thus comply with the AISC and FEMA 350 connection prequalification test requirements according to Supplement No. 1 to the AISC Seismic Provisions (AISC, 1999). All data was reported in accordance with the SAC Test Protocol.

Typical moment-connection rotation and moment-total plastic rotation responses of specimens with strong and weak panel zones is shown in Figures 7 through 10. Response of the medium panel zone specimens was similar to the response of the weak panel zone specimens. A summary of specimen performance is listed in Table 2, but more details can be found in the test reports cited above. As can be seen in these figures and in Table 2, Free Flange connection specimens generally reached more than 4 percent total rotation and more than 3 percent total plastic rotation without fracturing, albeit with some loss of resistance. Contribution of the beam to specimen rotation in specimens with strong panel zones was larger than 75 percent, while deformation of the specimens with weak and moderate panel zone strengths was dominated by the panel zone and the column. Causes of specimen resistance degradation were local buckling of the free flanges, observed at 2 percent to 3 percent drift levels, and lateral-torsional deformation of the beam, observed after local buckling at 3 percent and 4 percent drift level. Magnitude of post-peak resistance degradation ranged between a few percent for the weak

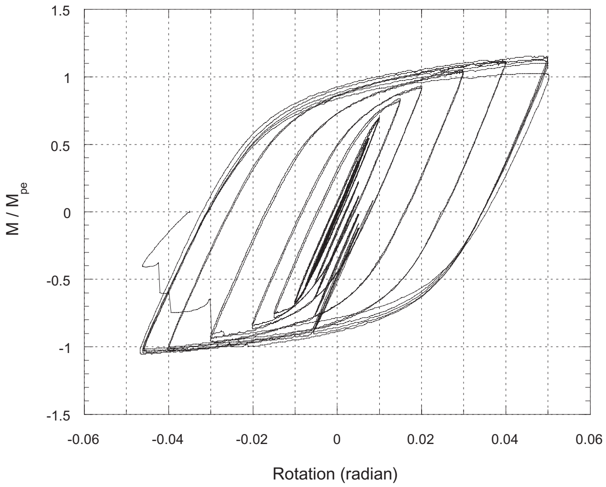

Fig. 7. Moment-rotation response of a weak panel zone Free Flange connection specimen (FF-2). Moments are computed at the center-line of the column and normalized with respect to the expected plastic design moment M_{pe} *for the connection, according to the SAC Test Protocol (SAC, 1997).*

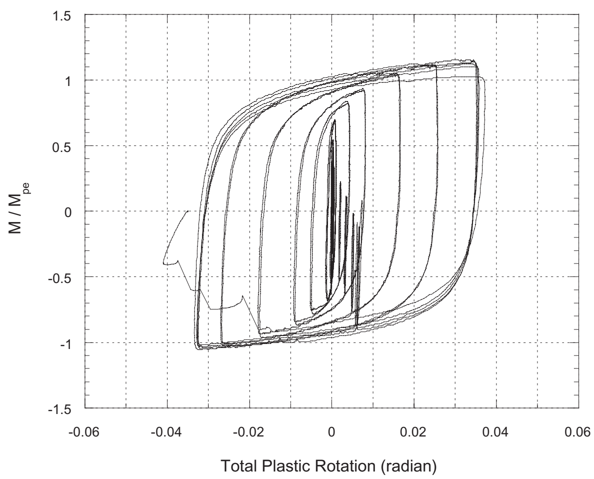

Fig. 8. Moment-total plastic rotation response of a weak panel zone Free Flange connection specimen (FF-2).

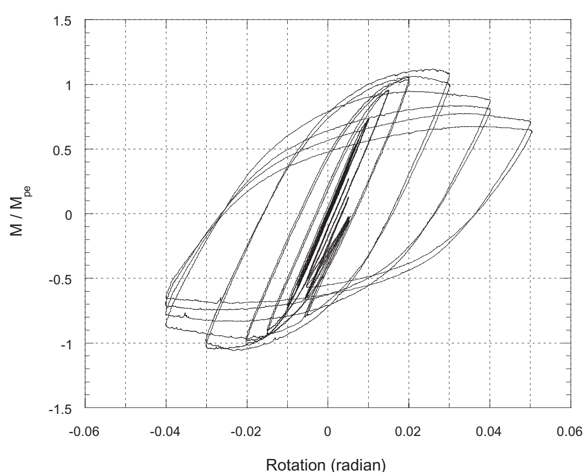

Fig. 9. Moment-rotation response of a strong panel zone Free Flange connection specimen (FF-1).

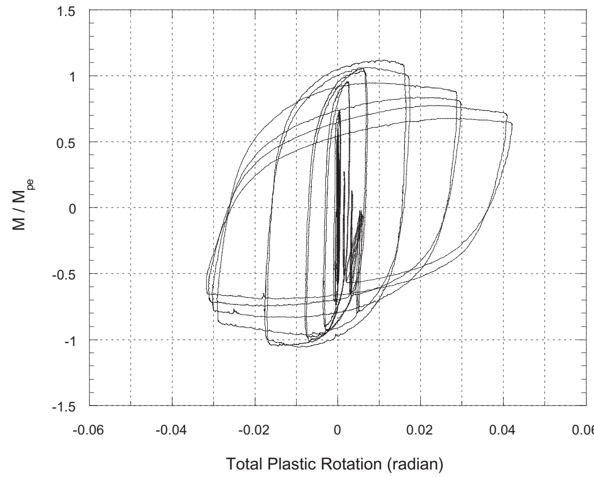

Fig. 10. Moment-total plastic rotation response of a strong panel zone Free Flange connection specimen (FF-1).

Fig. 11. Panel zone deformation and column kink in a weak panel zone Free Flange connection specimen (FF-2).

Fig. 12. Beam yielding and buckling in a strong panel zone Free Flange connection specimen (FF-3). Note that the panel zone shows no signs of yielding.

panel zone specimens to approximately 20 percent after two cycles at the same drift level for the strong panel zone specimens. Such behavior is similar to RBS connections behavior (Venti and Engelhardt, 2000). Lateral-torsional buckling of the beam in strong panel zone specimens, made possible by the weak lateral restraining system in the test setup, was responsible for relatively high loss of resistance under repeated loading. Use of code-prescribed bracing and the presence of a floor slab is expected to control lateral-torsional buckling of beams in real building structures and significantly reduce post-peak strength degradation of Free Flange connections.

Figures 11, 12 and 13 show deformations and yielding patterns of Free Flange connection specimens with weak, strong, and medium panel zones, respectively. Evidently,

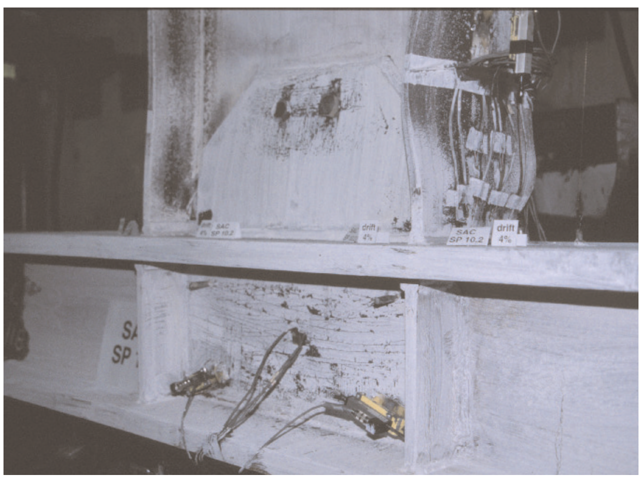

Fig. 13. Balanced yielding of the beam and the panel zone in a medium-strength panel zone Free Flange connection specimen (FF-4).

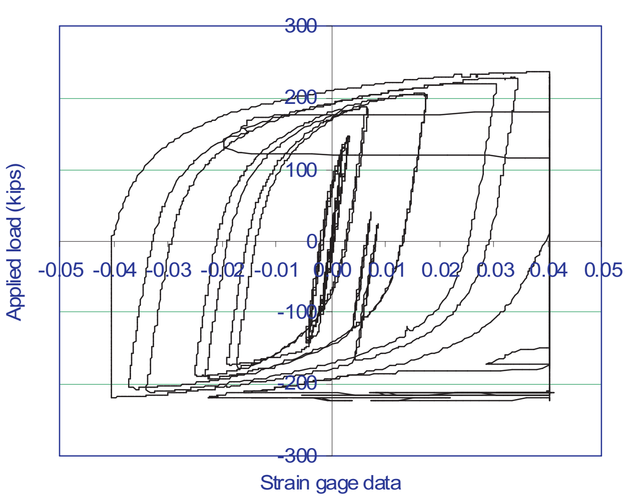

Fig. 14. Axial strain measured using a strain gage located in the middle of the free flange near the column face in Free Flange Specimen FF-5.

local connection behavior depends on panel zone strength. A weak panel zone specimen exhibited very ductile behavior in terms of rotation capacity. However, excessive panel zone deformation caused column flange kinking, leading to beam flange fracture at drift levels larger than 5 percent. Another undesirable effect of excessive weak panel zone deformation is an "incomplete" formation of the beam plastic hinge, meaning that only a part of the beam web yielded. In strong panel zone specimens, a "complete" plastic hinge formed in the beam at a distance away from the column face, and the panel zone did not yield. Such a yielding pattern may not be desirable either, because excessive beam yielding caused severe local buckling in the beam web and flanges, resulting in rapid connection stiffness and strength degradation as well as lateral instability problems. In a medium panel zone specimen, a balanced behavior between the beam and panel zone was observed. A "complete" beam plastic hinge formed without severe local or lateral-torsional buckling problems, while limited panel zone yielding still occurred.

IMPROVEMENT OF THE UOFM98 DESIGN PROCEDURE

The Free Flange connection test specimens were designed by using the UofM98 design procedure as presented earlier. Although all Free Flange connection specimens behaved in a ductile manner and satisfied prequalification test requirements, some improvements to the design procedure are warranted based on test observations. In addition, the entire design procedure was upgraded to the most recent provisions specified in FEMA 350.

Plastic Hinge Location and Connection Design Forces

Maximum loads sustained by the specimens were somewhat larger than design forces M_{pe} and V_{pe} calculated using

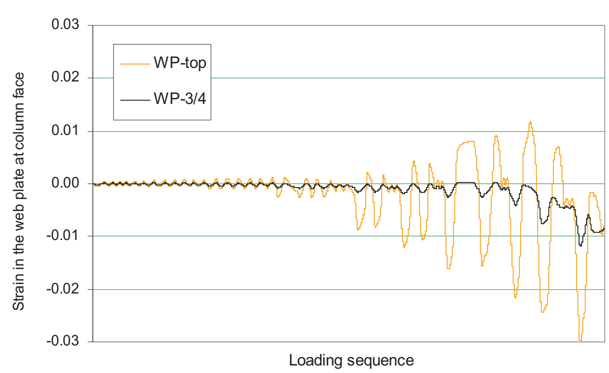

Fig. 15. Strain measurements in the web plate of Free Flange Specimen FF-5.

Equations 5 and 9. In UofM98 design procedure, the beam plastic hinge was located at the column face. This was a good assumption for the weak panel zone specimen. However, plastic hinges in medium and strong panel zone specimens were located approximately a half beam depth away from the column face. Assuming that a medium to strong panel zone will generally be provided in new buildings, the expected location of the beam plastic hinge can be taken at a half beam depth away from the column face. Based on this assumption, connection design forces should be revised as follows:

$$M_{pe} = C_{pr} Z_b R_y F_{yb} \tag{5}$$

$$V_{pe} = \frac{M_{pe}}{\left(\frac{L_b}{2} - \frac{d_b}{2} \right)} \tag{12}$$

where M_{pe} is the expected beam plastic moment (Equation 5) and V_{pe} is the corresponding shear. Design of the web plate connection is based on the moment at the face of the column. Establishing the plastic hinge at $d_b/2$ away from the column face results in the following values of beam shear and moment at the column face:

$$V_{cf} = V_{pe} + V_g \tag{13}$$

$$M_{cf} = V_{pe} \frac{L_b}{2} + M_g \tag{14}$$

where V_g and M_g are beam shear and moment due to gravity load computed at the face of the column.

Magnitude of the Flange Forces

A realistic estimate of the normal force resisted by the beam flange T_f can be obtained from strain gage data. Figure 14 shows the strain history measured in the middle of the beam flange at the column face during a test of Specimen FF-5. The strain in the beam flange exceeded 4 percent, which was the maximum capacity of the strain gage, but remained well below 15 percent based on manually measured elongations of the free flange. In common structural steel, strain hardening generally begins at approximately 1.5 percent strain, while ultimate strength, equal to approximately 1.4 times the yield strength, is attained at approximately 15 percent strain. The flange peak strength factor C_{prf} was set to 1.1 in Equation 6 of the UofM98 procedure. Based on the strain history shown in Figure 14, this factor should be increased to 1.25 to capture the increase of flange steel resistance due to strain hardening. Consequently, use of the tension resistance factor $\phi_t = 0.9$ in Equation 7 to compute T_{wp}, normal force resisted by the web plate, is not justified because the flange force has already been accurately estimated. Therefore, this resistance factor is dropped from Equation 7. Note that the beam plastic hinge peak strength factor C_{pr} remains at 1.1.

Web Plate Moment Arm

In the UofM98 design procedure, beam moment in the plastic hinge, M_{pe}, was divided between the moment couple formed by tension and compression forces in the flanges, M_f, and the moment in the web plate, M_{wp}. However, the assumption that both moment couples have the same moment arm, equal to $(d_b - t_f)$, was erroneous: it contradicts principles of mechanics, and it leads to an underestimate of the web plate forces. Both strain gage data and finite element analysis results are used to make a better estimate of the web plate moment arm. Strains measured at the two locations in the web plate, the top and ¾-point of web plate height, during the test of Specimen FF-5 are shown in Figure 15. The strains near the top of the web plate are approximately 3 times higher than the ones at ¾-height, but both are well above yield strain levels, suggesting similar stress values at these two locations. This finding is supported in Figure 6 that shows a normal stress distribution along the web plate height at 4 percent drift level taken from a finite element analysis of a Free Flange connection. The web plate of this connection model was connected to the column using two separate CJP welds, each $d_b/4$ long, as prescribed in the UofM98 design procedure. An equivalent stress block, with a depth equal to $h_{wpe}/2$, can be used to approximate the computed stress distribution (Figure 6). The stress block resultants are, thus, located at a distance of $h_{wpe}/4$ from the edges of the web plate. Thus, the moment arm for the web plate moment couple is $a_{wp} = h_{wp} - (h_{wpe}/2)$ (Figure 6).

Web Plate Forces

Considering the revision of web plate moment arm, revised flange peak strength factor C_{prf} = 1.25, and a revised assumption about the location of the beam plastic hinge, a modified procedure for computing the web plate normal force is:

$$M_f = T_f (d_b - t_f) = C_{prf} A_f R_y F_{yb} (d_b - t_f) \tag{15}$$

$$M_{wp} = M_{cf} - M_f \tag{16}$$

$$T_{wp} = \frac{M_{wp}}{(h_{wp} - \frac{h_{wpe}}{2})} \tag{17}$$

The shear force in the web plate is still computed assuming the web plate carries the entire beam shear:

$$V_{wp} = V_{cf} = V_{pe} + V_g \tag{18}$$

Geometry of the Web Plate

Change of the beam plastic hinge location from the column face to half-beam-depth away from the column face affects web plate geometry: length of the web plate must be limited to facilitate formation of the plastic hinge. Thus, it is reasonable to limit the length of the web plate (L_{wp} in Figure 1) to $d_b/2$. The rest of the UofM98 guidelines for web plate design remain in effect.

Shape of the Beam Web Cut

Connection between the beam web and the web plate has a small eccentricity. Nevertheless, web plate normal force acting at this eccentricity induces a bending moment about the connection vertical axis. This moment is initially resisted by the flanges, but when they buckle at large drift demand levels, the web plate takes this moment and bends out-of-plane. Resistance of the web plate to such bending depends on the CJP weld between the web plate and the column, and on the shape of the beam web cutback.

Connections designed for Specimens FF-1 through FF-5 had a straight beam web cutback as shown in Figure 1. Note that the length of web cutback is not restricted by the slope and radius of the transition between the web and the flange in the k-line region of the beam. Instead, a minimum 2-in. clearance *a*, needed to weld the vertical fillet weld along weld line A governs. Thus, the beam web may be extended towards the column face to strengthen the web plate by effectively shortening its span for out-of-plane bending. Interior Free Flange connection specimens UofT and UCSD in Table 1 featured extended web cuts with $a = 3t_{wp}$, roughly equal to half of the free flange length. These tests showed that the extended web cut is indeed effective in reducing web plate bending. The geometry of the web cutback in the k-line region of the beam for specimens UofT and UCSD was designed following the standard access hole geometry prescribed in Figure 3-5 of FEMA 350. Tests also showed that such cutback geometry reduced strain concentrations and precluded early failure of the flange near the web cutback.

UofM2000 FREE FLANGE CONNECTION DESIGN PROCEDURE

The principal assumptions in this design procedure are: 1) a beam plastic hinge forms $d_b/2$ away from the column face; 2) the beam moment at the column face can be divided between the flange moment couple and the web plate moment; 3) the web plate resists the total shear force transferred through the connection at the column face; 4) the web plate shear is carried equally by effective web plate portions in its top and bottom half; and 5) the web plate moment couple, established by normal forces in the top and bottom effective portions of the web plate, has a moment arm equal to $h_{wp} - (h_{wpe}/2)$.

The steps of the UofM2000 Free Flange connection design procedure are:

1. Set beam free flange length (Equation 3):

   $$L_{ff} = \alpha t_f$$

   The free flange aspect ratio α should have a value between 5.0 and 6.0. Round the free flange length to the nearest ⅛th of an in.
2. Choose between the straight and the extended web cut. In either case, provide a weld clearance distance *a* between the end of the beam web and the column that is at least 2 in. long. Specify the geometry of the web cut in the beam k-line area following Figure 1 and the access hole detail prescribed in Figure 3-5 of FEMA 350.
3. Set the height of the web plate (Equation 4):

   $$h_{wp} = d_b - 2t_f - 2d$$

   Weld clearance distance *d* should not be smaller than 2 in. (Figure 1). Round the height of the web plate to the nearest ⅛th in.
4. Calculate the effective height of the web plate (Equation 10):

   $$h_{wpe} = d_b / 4$$

5. Determine the length of the web plate and location of the 45°-diagonal corners of the web plate so that the extended corner diagonal intersects the beam k-line before the web transitions into the free flange length (Figure 1). Limit web plate length to no more than $d_b/2$.
6. Calculate beam shear and moment at the column face due to gravity loads. Then, compute beam shear and moment at the column face using the following procedure (Equations 5, 12, 13 and 14):

   $$M_{pe} = C_{pr} Z_b R_y F_{yb}; \quad C_{pr} = 1.1$$

   $$V_{pe} = \frac{M_{pe}}{\left(\frac{L_b}{2} - \frac{d_b}{2}\right)}$$

   $$V_{cf} = V_{pe} + V_g$$

   $$M_{cf} = V_{pe} \frac{L_b}{2} + M_g$$

7. Calculate moments carried by the beam flanges and by the web plate using the following procedure (Equa-

tions 6, 15 and 16):

$$T_f = C_{prf} A_f R_y F_{yb}; \quad C_{prf} = 1.25$$

$$M_f = T_f (d_b - t_f)$$

$$M_{wp} = M_{cf} - M_f$$

8. Calculate the normal force in the moment couple acting on the web plate (Equation 17):

$$T_{wp} = \frac{M_{wp}}{(h_{wp} - \frac{h_{wpe}}{2})}$$

9. Calculate the shear force acting on the web plate (Equation 18):

$$V_{wp} = V_{cf} = V_{pe} + V_g$$

10. Calculate the thickness of the web plate using Equation 11 with a strength reduction factor $\phi = 0.9$:

$$t_{wp} = \frac{\sqrt{\dfrac{T_{wp}{}^2 + 0.75 V_{wp}{}^2}{(\phi F_y)^2}}}{h_{wpe}}$$

Web plate should be at least as thick as the beam web.

11. Design the fillet welds between the beam web and the web plate (weld lines A and B) to resist a combination of normal force T_{wp} and shear force V_{wp} in each half of the web plate. Specify these welds as field welds to be made in vertical position.
12. Specify two complete-joint-penetration welds along the effective height h_{wpe} of the top and bottom portion of the web plate to shop-weld the web plate to the column.
13. Specify complete-joint-penetration field welds between beam flanges and the column following Figure 3-8, Note 1 in FEMA 350.
14. Design connection panel zone and doubler plate(s) following Section 3.3.3.2 of FEMA 350.
15. Design connection continuity plates following Section 3.3.3.1 of FEMA 350.

All base and weld material should satisfy material property requirements specified in FEMA 350.

Design Example

Free Flange connection Specimen FF-4 with a W30×124 beam and a W14×257 column is redesigned in this example. A572 Grade 50 steel is used throughout. The clear span of the beam, L_b , is 268 in., while the story height is 144 in. UofM2000 Free Flange connection design procedure results in the following:

1. Selecting free flange aspect ratio $a = 5.4$. Adopt free flange length:
   $L_{ff} = \alpha t_f = 5.4 \times 0.93$ in. = 5 in.

2. Choose a straight web cutback with length $a = 6.5$ in.

3. Set the height of the web plate to:
   $h_{wp} = d_b - 2t_f - 2b = 30.2$ in. – 2 × 0.93 in. – 2 × 2 in.
   = 24 in.

4. Effective web plate height is:
   $h_{wpe} = d_b/4 = 30.2$ in./4 in. = 7.5 in.

5. Choose the length of the web plate so that the extended diagonal cut intersects the beam k-line before the web transitions into the free flange length, as shown in Figure 16.

6. Neglect beam shear and moment due to gravity loads. Compute:
   $M_{pe} = C_{pr} Z_b R_y F_{yb} = 1.1 \times 408 \times 1.1 \times 50$
   = 24,684 kip-in.

$$V_{pe} = \frac{M_{pe}}{\left(\frac{L_b}{2} - \frac{d_b}{2}\right)} = \frac{24,684}{\left(\frac{268}{2} - \frac{30.2}{2}\right)} = 208$$

$$V_{cf} = V_{pe} + V_g = 208 + 0 = 208 \text{ kips}$$

$$M_{cf} = V_{pe}\frac{L_b}{2} + M_g = 208 \times \frac{268}{2} + 0 = 27,872 \text{ kip-in.}$$

7. Moments carried by the beam flanges and by the web plate:

$$T_f = C_{prf} A_f R_y F_{yb} = 1.25 \times (10.5 \times 0.93) \times 1.1 \times 50$$
$$= 671.3 \text{ kips}$$

$$M_f = T_f (d_b - t_f) = 671.3 \times (30.2 - 0.93)$$
$$= 19,650 \text{ kip-in.}$$

$$M_{wp} = M_{cf} - M_f = 27,872 - 19,650$$
$$= 8,222 \text{ kip-in.}$$

Note that approximately 30% of the column face moment is carried by the web plate.

8. Calculate the normal force in the moment couple acting on the web plate:

$$T_{wp} = \frac{M_{wp}}{\left(h_{wp} - \frac{h_{wpe}}{2}\right)} = \frac{8222}{\left(24 - \frac{7.5}{2}\right)} = 406 \text{ kips}$$

9. Calculate the shear force acting on the web plate:
$V_{wp} = V_{cf} = 208$ kips

10. Calculate the thickness of the web plate:

$$t_{wp} = \frac{\sqrt{\dfrac{T_{wp}^{\;2} + 0.75 V_{wp}^{\;2}}{(\phi F_y)^2}}}{h_{wpe}} = \frac{\sqrt{\dfrac{406^2 + 0.75 \times 208^2}{(0.9 \times 50)^2}}}{7.5}$$

$$= 1.31 \text{ in.} \simeq 1\frac{3}{8} \text{ in.}$$

Note that the web plate is approximately twice as thick as the web of the beam.

11. Design the fillet welds between the beam web and the web plate (weld lines A and B) to resist a combination of normal force T_{wp} and shear force V_{wp} in each half of the web plate. Assume E70 weld metal. Using the instantaneous center of rotation method for eccentrically loaded weld groups, approximate fillet weld group A and B as to parallel vertical welds and use Table 8-38 from AISC LRFD Manual (AISC, 1994), as shown in Figure 16. Angle of the resultant R_{wp} with respect to the vertical:

$$\phi = \tan^{-1}\left(\frac{T_{wp}}{0.5V_{wp}}\right) = \tan^{-1}\left(\frac{406}{104}\right) = 75.6^{\circ}$$

Therefore, use the 75° angle part of Table 8-38 with:

$l = L_A = 11.5$ in.; $k = 7$ in./11.5 in. $= 0.6$; $e_x = 0$ in.; $C_1 = 10.0$

to find $C = 4.11$ and

$$D_A = D_B = \frac{R_{wp}}{CC_1 L_A} = \frac{416.4}{4.11 \times 1.0 \times 11.5 \text{ in.}} = 8.8 \simeq 9$$

Therefore, adopt 9/16-in. fillet welds along weld lines A and B as shown in Figure 16. Welds on weld line A must be built out to full height.

12. Web plate CJP welds along the 7.5-in. top and bottom effective height of the web plate are specified in Figure 16.

13. Beam flange CJP are specified following Figure 3-8, Note 1 in FEMA 350.

14. Design of the connection panel zone following Section 3.3.3.2 of FEMA 350 is omitted for brevity.

15. Design of the connection continuity plates following Section 3.3.3.1 of FEMA 350 is omitted for brevity.

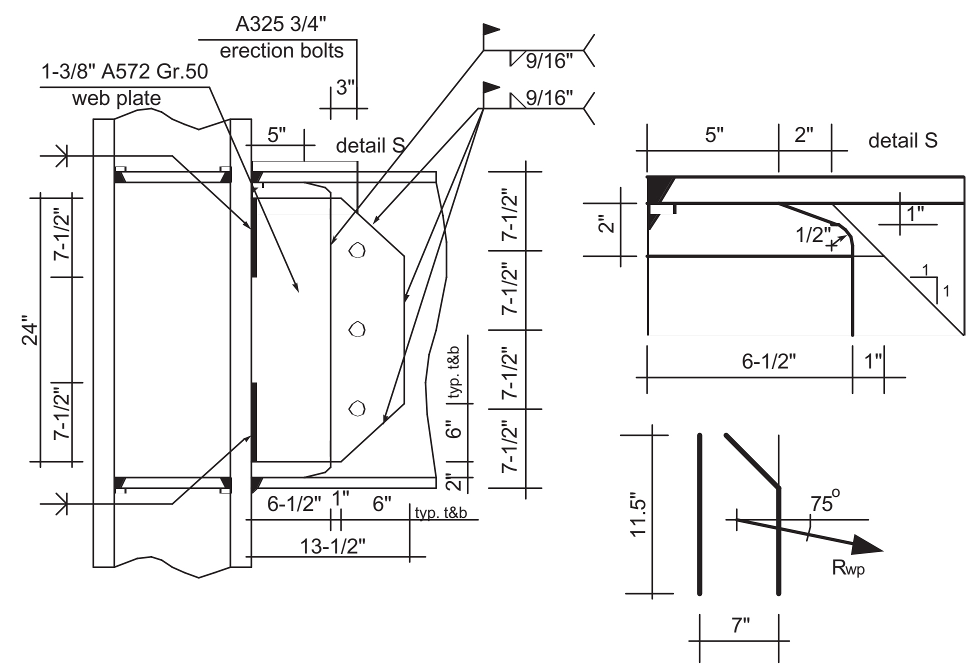

Fig. 16. Free Flange connection designed in the design example.

COMPARISON OF FREE FLANGE CONNECTION DESIGN PROCEDURES

The Free Flange connection has been included among the prequalified welded moment connections in FEMA 350. Connection specimens used in the prequalification tests were designed using the UofM98 design procedure. After these tests, the UofM98 design procedure was modified by SAC Joint Venture TAP for Connection Performance for inclusion in FEMA 350. In a separate effort, the improved UofM2000 design procedure presented herein was also developed. It is important to compare these procedures with respect to the prequalified connection specimens and establish if they are applicable for prequalified connection design.

UofM98 and FEMA 350 Procedures

There are two principal differences between the UofM98 and FEMA 350 design procedures: 1) location of the beam plastic hinge; and 2) connection peak strength factor C_{pr}. Locating the beam plastic hinge half-beam-depth away from the column is appropriate for panel zone designs prescribed in FEMA 350, while the UofM98 assumption that the plastic hinge forms at the column face is adequate only for weak panel zones. In practice, FEMA 350 should be used.

Connection peak strength factor C_{pr} for the Free Flange connection is 1.1 in the UofM98 design procedure, while in FEMA 350 this factor is set to 1.2. Table 2 shows the values of the C_{pr} factor back-calculated from the peak loads recorded in the seven Free Flange connection tests that have been carried out to date. The calculated values range between 0.91 and 1.26 with a mean value of 1.07. Thus, the FEMA 350 C_{pr} value of 1.2 appears to be too high, leading to over-design of the web plate and web plate fillet welds. The UofM98 C_{pr} value of 1.1 is closer to prequalification test results and appropriate for use in practice.

Both UofM98 and FEMA 350 design procedures adopt the same moment arm value for the internal flange and web plate moment couples.

UofM98 and UofM2000 Procedures

There are three principal differences between the UofM98 and UofM2000 design procedures: 1) location of the beam plastic hinge; 2) length of the moment arm for internal flange and web plate moment couples; and 3) flange peak strength factor C_{prf}.

The UofM2000 design procedure adopts the same plastic hinge location as FEMA 350: half-beam-depth away from the column face. However, at the column face, the difference in moment arm lengths for the flange and the web plate moment couples is explicitly recognized in the UofM2000 design procedure. The recommended web plate moment arm, equal to $h_{wp} - (h_{wpe}/2)$, is shorter than $(d_b - t_f)$, the value used in the UofM98 design procedure, while the effective web plate length remained the same in both procedures. This change was justified using measured strain gage data and post-test finite element analysis, and should be adopted in practice. In addition, different levels of strain in the flange and the web plate, and the associated different magnitudes of strain hardening, are recognized in the UofM2000 design procedure through the use of a separate flange peak strength factor C_{prf}. A single connection peak strength factor C_{pr} used in UofM98 and FEMA 350 design procedures does not enable such differentiation. Together, change in the web plate moment arm length and use of a separate flange peak strength factor lead to a realistic estimate of the web plate forces and better web plate design.

FEMA 350 and UofM2000 Procedures

It should be noted that the required web plate thickness and fillet weld sizes are the smallest by the UofM98 procedure, somewhat larger by the UofM2000 procedure, and the largest if the FEMA 350 design procedure is used. The role of the web plate in the Free Flange connection is such that stronger web plate leads to a more conservative connection design. Therefore, if UofM98 connections passed the prequalification tests, it may be concluded that both UofM2000 and FEMA 350 connections would pass the same tests, too.

The principal differences between the FEMA 350 and UofM2000 design procedures are: 1) length of the web plate moment arm; and 2) use of a separate connection and flange peak strength factor. The UofM2000 design procedure is more rational on both counts, because both experimental and finite element analysis data was used to justify the valued adopted in this procedure.

Only right-angle in-plane Free Flange connections were tested and prequalified. Consequently, UofM2000 and FEMA 350 design procedures address only such connections. Furthermore, all prequalified connections listed in FEMA 350 assume the same right-angle in-plane configuration. Use of FEMA 350 connections, including the Free Flange connection, in configurations where the beam is skewed with respect to the column either in the plane of the connection or out of the connection plane requires additional prequalification testing.

CONCLUSION

The Free Flange connection is a new public-domain beam-to-column moment connection developed at The University of Michigan. This connection design is based on the finding that improvements in connection welds alone are not sufficient to insure ductile response of the connection (Stojadinovic et al., 2000). In addition to better welds, geometry of the Free Flange connection is designed to prevent excessive local deformation of the flanges and to change the force flow in the connection to reduce flange shear forces. These design goals were achieved by cutting the web of the beam back and away from the column. Portions of the beam flanges that are not constrained by the web significantly reduce connection deformation constraints; allow the flange steel to yield freely; and help to redirect most of the shear force back into the web connection.

The Free Flange connection was prequalified for seismic design of new moment resisting frame buildings on the basis of seven standard connection tests and is included in FEMA 350 Design Guidelines. Prequalification criteria listed in Table 3-4 of FEMA 350 limit Free Flange connection applicability in accordance with the number of conducted tests and beam and column sizes used in these tests. More prequalification tests are needed to extend the range of the Free Flange connection. Nevertheless, prequalification tests conducted to date demonstrated that the Free Flange connection is quite robust and that it compares well with other FEMA 350 connection types. Even though it may not be as economical as the RBS or the WUF-W connection due to the amount of welding required on the web plate, the Free Flange connection concept is important for designers to understand the force flow and the deformation mechanism of fully-restrained moment connections. For example, the explicit design procedure for the connection between the beam web and the column is a unique feature of the Free Flange connection.

The prequalified connections were designed using the UofM98 design procedure, developed on the basis of finite element analysis alone. Test results and further analyses were used to modify this design procedure. The Free Flange connection design procedure in FEMA 350 was developed within the SAC Joint Venture Steel Project, while the improved UofM2000 procedure was developed by the authors and is presented herein. The primary differences among these procedures are in: 1) location of the beam plastic hinge; 2) moment arm lengths for internal moment couples; and 3) connection and flange peak strength factors. A comparison of these three procedures shows that the UofM2000 procedure has a better foundation in the principles of mechanics and better represents test results than UofM98 and FEMA 350 design procedures. In addition, connections designed using the UofM2000 procedure are somewhat more conservative than the ones designed using the UofM98 procedure, and somewhat less conservative than FEMA 350 designs. Based on this finding, the authors recommend the proposed UofM2000 design procedure for use in practice.

ACKNOWLEDGMENT

The research presented in this paper was carried out with financial support from the National Science Foundation (Grant No. CMS 9615273) and FEMA/SAC Joint Venture Steel Project (Subtask 7.02a). The authors gratefully acknowledge these sources of funding. Nevertheless, the opinions presented in the paper are solely those of the authors and do not necessarily represent the views of the sponsors.

REFERENCES

AISC (1994), *Load and Resistance Factor Design Manual of Steel Construction*, American Institute of Steel Construction, Vol. I and II, 2nd edition, Chicago, IL.

AISC (1997), *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, April 15, Chicago, IL.

AISC (1999), *Seismic Provisions for Structural Steel Buildings (1997) Supplement No. 1*, American Institute of Steel Construction, February 15, Chicago, IL.

AISC (1999), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, December 27, Chicago, IL.

Choi, J., Goel, S.C., and Stojadinovic, B. (2000), *Development of Free Flange Moment Connection*, Research Report No. UMCEE 00-15, Department of Civil and Environmental Engineering, The University of Michigan, Ann Arbor, MI.

FEMA (1995), *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, Federal Emergency Management Agency, Report No. FEMA 267.

FEMA (1997), *Interim Guidelines Advisory No. 1*, Federal Emergency Management Agency, Report No. FEMA 267A.

FEMA (2000), *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, Federal Emergency Management Agency, Report No. FEMA 350.

Gilton, C., Chi, B., and Uang, C.M. (2000), *Cyclic testing of a Free Flange Moment Connection*, SAC Joint Venture, Technical Report No. SAC/BD-00/19.

Goel, S.C., and Stojadinovic, B., and Lee, K.H. (1997), "Truss Analogy for Steel Moment Connections," *Engineering Journal*, American Institute of Steel Construction, Vol. 33, No. 2, 2nd Quarter, pp. 43-53.

SAC Joint Venture (1997), *Protocol for Fabrication, Inspection, Testing and Documentation of Beam-Column Connection Tests and Other Experimental Specimens*, Technical Report No. SAC/BD-97/02.

Stojadinovic, B., Goel, S.C., Lee, K.H., Margarian, A.G., and Choi, J. (2000), "Parametric Tests on Unreinforced Steel Moment Connections," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 126, No. 1, pp. 40-49.

Venti, M., and Engelhardt, M. (2000), *Test of a Free Flange Connection with a Composite Floor Slab*, SAC Joint Venture, Technical Report No. SAC/BD-00/18.

Yang, T.-S. and Popov, E.P. (1996), "Analytical Studies of Pre-Northridge Steel Moment-Resisting Connections," *Proceedings of 11th World Conference on Earthquake Engineering*, CD 2, Paper No. 661, Elsevier Science Ltd, Oxford, England.