els and corresponding rotational demands at VBE at these levels.

Berman and Bruneau (2003a) provide a comparison of the work required to achieve two mechanisms: the yielding of the web plates over the entire height of the structure and a story mechanism. Their study indicates that to ensure the former mechanism, the thickness of the web plate must change at each story to match the story shear. Otherwise, a mechanism that to some degree concentrates inelastic deformation in some stories will form. Thus, it is recommended to proportion the web plates to the story shear as closely as possible and not to provide unnecessary overstrength.

In some cases, foundation uplift or diaphragm deformation can be the predominant mode of seismic response of SPSW structures (as is also the case for other stiff systems). ASCE 7 does not fully address these as modes of seismic response. Design of systems based on that mode of behavior is beyond the scope of this Design Guide.

3.5.1. REQUIREMENTS OF THE AISC SEISMIC PROVISIONS (ANSI/AISC 341-05)

AISC 341 addresses the high-seismic design of SPSW. As AISC 360 does not address SPSW, some of the basic requirements of the system contained in AISC 341 are also applied in low-seismic design as well.

The general SPSW requirements applicable to both high-seismic and low-seismic design pertain to the analysis of the system and certain member requirements. Foremost of these is the calculation of the angle of tension stress in the web plate (AISC 341 Equation 17–2, Equation 3–1), and the corresponding expression for web-plate shear strength as a function of the angle (AISC 341 Equation 17–1, Equation 3–20).

Equally important are limitations on the systems to which these equations are applicable. These include the panel aspect ratio (L/h) in AISC 341 Section 17.2.b (to the range between 0.8 and 2.5, as discussed under the previous section) and the required VBE stiffness given in Section 17.4.g (Equation 3–22).

Additionally, AISC 341 Section 17.2.c, requires that boundary elements be included adjacent to all openings “unless otherwise justified by testing and analysis”. This requirement is applicable to both high-seismic and low-seismic design of SPSW.

In addition to these general design requirements, AISC 341 contains many requirements that are only applicable to high-seismic design. These include requirements for the web-plate connection and for the frame.

As discussed earlier, the high-seismic design of SPSW is based on yielding of the web plate. Thus, AISC 341 requires that the web-plate connection be designed to resist the expected yield strength of the web plate ($R_y F_y t_w$). Connections of web plates must have sufficient strength to permit the plate to develop this force across the entire connection, considering the angle of the tension α as discussed in the previous section.

Likewise, the design of HBE and VBE must be based on forces corresponding to full tension yielding of the web plate. In this way, AISC 341 ensures that web-plate tension yielding is the primary yield mechanism of SPSW.

AISC 341 also requires that a SPSW be designed as a moment frame with a web-plate infill. Specifically, a number of the provisions require that boundary elements and their connections conform to requirements for Special Moment Frames (SMF) or Ordinary Moment Frames (OMF).

Connections of HBE to VBE must be designed as OMF connections; Section 17.4b gives the requirement for SPSW, referring to the OMF Section 11.2 of AISC 341. Additionally, the required shear strength of the HBE-to-VBE connection must be based on the development and strain-hardening of plastic hinges at each end of the HBE (rather than allowing use of the amplified seismic load, as is allowed for a typical OMF). The seismic portion of the required shear strength is given by AISC 341 Equation 9–1 (Equation 3–37):

$$E_m = 2M_{pr}/L_h \tag{3–37}$$

where

E_m = The maximum seismic load effect to be used in ASCE load combinations

L_h = the distance between plastic hinges

$$= L - 2s_h \tag{3–38}$$

where

L = the distance between column centerlines

s_h = the distance from the column centerline to the plastic hinge, as given in AISC 358

For unreinforced connections, such as Reduced Beam Section (RBS) and Welded Unreinforced Flange-Welded Web (WUF-W) connections, s_h can be determined as

$$s_h = ½\,(d_c + d_b) \tag{3–39}$$

AISC 358 gives limitations for this distance for the RBS connection; the value above is a reasonable preliminary estimate.

Note that the beam plastic moment strength in Equation 3–37 is typically calculated in the absence of any axial force:

$$M_{pr} = 1.1R_yF_yZ \tag{3–40}$$

Designers may wish to consider the axial force present at the HBE-to-VBE connection in order to reduce the calculated flexural strength and thus required shear strength of the connection. While not explicitly described in AISC

341, this method is consistent with the underlying capacity-design methodology in which the yield mechanism of the frame is considered. Reduction of the calculated HBE flexural strength can be done adapting the interaction equations from Chapter H of AISC 360.

For LRFD the resulting modified beam strength when $P_u/P_y < 0.2$ is

$$M^*_{pr} = \left(1.1R_yF_yZ\right)\left[1-\frac{1}{2}\left(\frac{P_{u\ HBE}}{P_y}\right)\right] \tag{3–41}$$

and otherwise is

$$M^*_{pr} = \frac{9}{8}\left(1.1R_yF_yZ\right)\left[1-\frac{P_{u\ HBE}}{P_y}\right] \tag{3–42}$$

For ASD the resulting modified beam strength when $P_u/P_y < 0.2$ is

$$M^*_{pr} = \left(1.1R_yF_yZ\right)\left[1-\frac{1}{2}\left(\frac{1.5P_{a\ HBE}}{P_y}\right)\right]$$

and otherwise is

$$M^*_{pr} = \frac{9}{8}\left(1.1R_yF_yZ\right)\left[1-\frac{1.5P_{a\ HBE}}{P_y}\right]$$

For SPSW, the additional beam shear due to web-plate tension must be considered. The total beam shear is thus

For LRFD

$$V_u = \frac{2M_{pr}}{L_h}+\frac{P_u}{2}+\frac{w_g+w_u}{2}L_{cf} \tag{3–43}$$

For ASD

$$V_a = \frac{2M_{pr}}{L_h}+\frac{P_a}{2}+\frac{w_g+w_a}{2}L_{cf}$$

where

P = concentrated gravity load on the beam (assumed to be centered on the span) based on LRFD or ASD load combinations

w_g = distributed gravity load on the beam (assumed to be uniform) based on LRFD or ASD load combinations

$$w_u = R_yF_y\left(t_i - t_{i+1}\right)\cos^2(\alpha) \tag{3–44}$$

w_a = $w_u/1.5$

Note that the appropriate load factors from LRFD or ASD load combinations must be applied to gravity forces in the above equations.

Figure 3–18 shows the free-body diagrams for the condition under which V_p is calculated.

For fully restrained connections, AISC 341 Section 11.2a requires that the connection have the strength to resist the formation of a plastic hinge in the beam (including strain hardening): $1.1R_y M_p$ (the "maximum force that can be delivered by the system" is a limitation that is not applicable to the OMF connection in a SPSW). Additionally, the section gives prescriptive requirements for continuity plates, welds, and weld access holes. The required weld-access hole configuration is shown in Figure 11–1 of AISC 341. Single-sided partial-joint-penetration groove welds or fillet welds are not allowed. For partially restrained connections, Section 11.2b requires the same strength as does Section 11.2a.

Welds of flanges in these connections must comply with the requirements in Section 7.3b for demand-critical welds. These include a Charpy V-notch toughness of 20 ft-lb at –20 °F as determined by the appropriate AWS classification test method or manufacturer certification, and 40 ft-lb at 70 °F (but not more than 20 °F above the lowest anticipated service temperature) as determined by Appendix X of AISC 341, or another method approved by the engineer.

In addition, Section 17.4a requires that boundary elements comply with the requirements for SMF in Section 9.6. That is, boundary elements must be proportioned so that the strong-column/weak-beam requirements of Equation 9–3 (Equation 3–45) are met:

$$\frac{\sum M^*_{pc}}{\sum M_{pb}} \geq 1.0 \tag{3–45}$$

where

$\sum M^*_{pc}$ = sum of column plastic moment strengths at a connection (reduced for axial force and computed at the beam centerline)

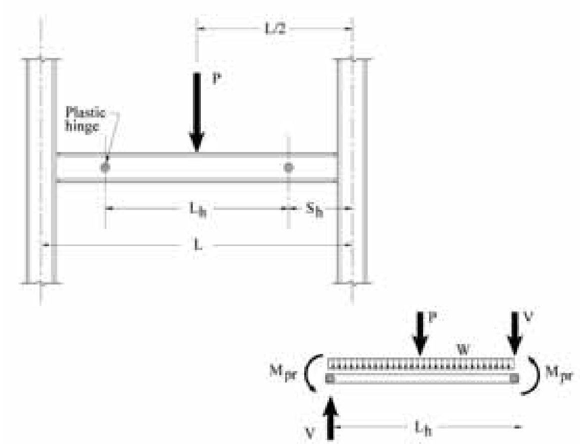

Fig. 3–18. Free-body diagram of SMF beam.

$\sum M_{pb}$ = sum of beam plastic moment strengths at a connection (computed at the column centerline)

Figure 3–19 shows this method for computing beam strength at the column centerline. The beam strength projected to the column centerline is

$$M_{pb} = M_{pr} + V_u s_h \tag{3–46}$$

Panel zones of HBE-to-VBE connections at the top and bottom of the SPSW must also comply with SMF requirements. Section 17.4.f requires compliance with Section 9.3. This section requires that the panel-zone shear strength be computed by calculating the moment at the column face due to the formation of a plastic hinge in the beam at a determined location. Figure 3–20 shows this method for computing beam strength at the column face. The authors recommend that the requirements of AISC 341 Section 17.4 be applied to panel zones at all levels.

The minimum panel-zone thickness is given in AISC Equation 9–3 (Equation 3–47):

$$t \geq \frac{d_z + w_z}{90} \tag{3–47}$$

where

t = the sum thickness of the column web and any doubler plates used

d_z = the panel-zone depth between beam flanges or continuity plates (if present)

w_z = the panel-zone width between column flanges

If doubler plate(s) are required, Section 9.3.c gives prescriptive detailing requirements. Doublers are welded along their vertical edges to develop their full shear strength.

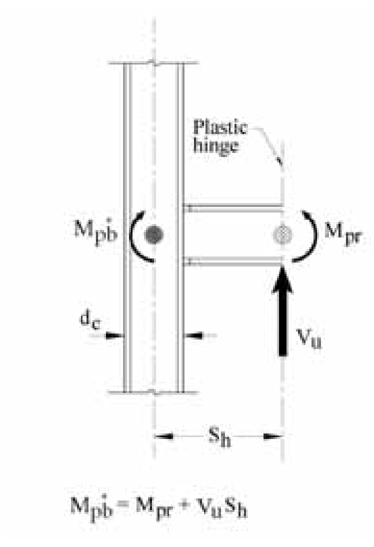

Fig. 3–19. Forces at column centerline from beam plastic hinge.

As boundary elements are configured to comprise a moment frame, the formation of plastic hinges in boundary elements (typically the HBE) under the design seismic loading is considered possible. AISC 341 therefore places certain compactness requirements on them (Section 17.4c). For flanges, the limit is

$$\frac{b_f}{2t_f} \leq 0.30\sqrt{\frac{E}{F_y}} \tag{3–48}$$

For webs, the limits are based on the axial force in the member. The axial force ratio C_a is

For LRFD

$$C_a = \frac{P_u}{\phi_b P_y}$$

For ASD (3–49)

$$C_a = \frac{\Omega_b P_a}{P_y}$$

ϕ_b and Ω_b are as defined in AISC 341 Table I–8–1.

The limiting web slenderness ratios are

for $C_a \leq \frac{1}{8}$

$$\frac{h}{t_w} \leq 3.14\sqrt{\frac{E}{F_y}}\left[1 - 1.54C_a\right] \tag{3–50}$$

for $C_a > \frac{1}{8}$

$$\frac{h}{t_w} \leq 1.12\sqrt{\frac{E}{F_y}}\left[2.33 - C_a\right] \tag{3–51}$$

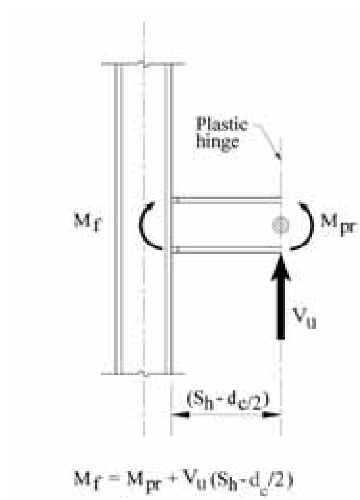

Fig. 3–20. Forces at column face from beam plastic hinge.

Using C_a of 1.0, one can see that a ratio of h/t_w of 36 will always satisfy the requirement for F_y = 50 ksi.

In keeping with the expected moment-frame behavior, Section 17.4.d gives lateral bracing requirements. The maximum brace spacing is the same as for SMF:

$$L_b \le 0.086 r_y \left(\frac{E}{F_y} \right) \tag{3–52}$$

HBE bracing force requirements are based on the section expected plastic moment:

$$P_{br} = 0.02 F_y b_f t_f \tag{3–53}$$

The brace stiffness required to satisfy AISC 360 Equation A–6–8, using the section expected plastic moment and C_d of 1.0, is

$$\beta_{br} = \frac{13.3 R_y F_y Z}{L_b \left(d - t_f \right)} \tag{3–54}$$

Finally, AISC 341 has specific requirements for VBE splices (Section 17.4e, which refers to Section 8.4). Such splices must be capable of resisting the same forces as are required for the column. For columns subject to net tension, two additional requirements apply. First, if partial-joint-penetration groove welds are used, splice required strengths must be doubled. Second, flange splices must be able to resist forces corresponding to one-half of the expected strength of the smaller flange:

$$R_u = \frac{1}{2} R_y F_y A_f \tag{3–55}$$

Splices are required to be at least four ft from the nearest HBE, or at the midpoint of the clear height of the VBE.

3.5.2. DESIGN

The application of these provisions in order to achieve the expected performance is discussed below. Designers must be aware that conformance to AISC 341 cannot by itself guarantee ductile system behavior for all configurations and applications. Attention must be given to the specifics of each design.

3.5.2.1. Web-Plate Design

The high-seismic design of web plates is the same as the low-seismic design of these elements. The design strength is computed using the calculated angle of tension stress (AISC 341 Equation 17–2, Equation 3–1) and the design shear strength based on that angle (AISC 341 Equation 17–1, Equation 3–20). This strength is compared to the required strength of the web plate as determined from analysis. This required strength is based on the horizontal shear resisted by the SPSW, some of which is resisted by the VBE.

3.5.2.2. HBE Design

Horizontal boundary elements are designed for forces corresponding to yielding of the web plate. Axial forces in HBE are largely due to the effects of web-plate tension on the VBE. Flexural forces are due in part to web-plate tension (where plates of differing thickness are used above and below the beam, or where only one web plate connects to the beam, such as at the top of the SPSW).

The required flexural strength of the HBE at the top and bottom of the SPSW can be quite large. At other levels, the required flexural strength due to web-plate yielding is limited to the difference in web-plate strength above and below and to any difference in the angle of the tension stress α. The load that the web plates are expected to exert on the HBE can be estimated using Equation 3–44.

Where the same web plate thickness is provided both above and below the HBE, Equation 3–44 will result in no flexural requirement for the beam. While this is consistent with achieving the full yielding of the web plates, use of a very flexible beam will result in the contribution of the moment frame being negligible, which is not consistent with the assumed system behavior. At a minimum, the beam must be designed to resist the differential forces due to the calculated story shears tributary to the frame (Equation 3–24). Providing beams of radically different strengths from one level to the next is not recommended.

At the base, a steel beam in the foundation may be provided. Alternatively, a concrete foundation may be designed to resist these forces, typically by acting as a beam spanning between column footings. Strong-column/weak-beam proportioning is not addressed by the provisions at this location. While flexural yielding in the grade beam is preferable to flexural yielding at the base of the column, this may not be feasible for beams designed to span from column to column resisting the tension yielding of the web plate.

While the web local yielding limit state in the HBE is only required to resist the stress σ_{11} (Figure 3–6), this stress combines in the web with the shear stress σ_{12}. It is therefore advisable to use sections with webs that are at least as strong as the expected strength of the web plate. For sections of a different material grade, the recommended minimum thickness of the HBE web is

$$t_{w\,HBE} \ge \frac{t_w R_y F_y}{F_{y\,HBE}} \tag{3–56}$$

where

$F_{y\,HBE}$ = the yield stress of the HBE material

$R_y F_y$ = the expected yield stress of the web-plate material

t_w = the thickness of the web plate

$t_{w\,HBE}$ = the thickness of the HBE web

Flexural forces from frame deformation must also be resisted by the HBE. These flexural forces can be assumed to cause plastic hinges to form at the ends of the beam. Thus, the flexural forces from frame deformation can be ignored if the HBE are designed to have sufficient strength to resist web-plate tension assuming a simple span. Thus, the required midspan flexural strength of the HBE is

For LRFD

$$M_u = \frac{\left(w_u + w_g\right)L_h^2}{8} + \frac{P_u L_h}{4}$$

For ASD (3–57)

$$M_a = \frac{\left(w_a + w_g\right)L_h^2}{8} + \frac{P_a L_h}{4}$$

The *PL*/4 terms above can be modified appropriately when the arrangement of framing beam(s) is not one beam at midspan of the HBE.

This flexural force is combined with the axial force, which has two sources. The first is VBE reactions due to the inward force from the web plate. The second is a difference in the effects of the webs above and below, due to any difference in thickness and angle α and possibly material.

Figure 3–21 shows the assumed yield mechanism of a two-story SPSW, with internal forces due to (a) web-plate tension and (b) flexural deformation.

The axial force from VBE can be estimated by assuming that VBE deliver forces equally to the top and bottom of each story. Thus the axial force from this source is

$$P_{HBE\ (VBE)} = \sum \frac{1}{2} R_y F_y \sin^2(\alpha) t_w h_c \qquad (3\text{–}58)$$

From the web plates, the axial force (assuming equal collector conditions on each side of the SPSW) is the additional collector force required to cause web-plate yielding at that level

$$P_{HBE(web)} = \frac{1}{2} R_y F_y \left[t_i \sin(2\alpha_i) - t_{i+1} \sin(2\alpha_{i+1})\right] L_{cf} \qquad (3\text{–}59)$$

This force should not be less than the required strength of the collector.

At the VBE in tension, both the collector and the VBE tend to cause compression in the HBE-to-VBE connection. At the VBE in compression, the collector tends to cause tension while the VBE tends to cause compression in the HBE-to-VBE connection.

Equations 3–58 and 3–59 give seismic load effects, which are combined with other loads according to the appropriate load combinations (LRFD or ASD).

The required shear strength of HBE was previously established in the discussion of the AISC 341 requirements (Equation 3–43). As hinging is expected in the HBE, the web connection should be designed to resist both the shear and axial forces.

As noted earlier, the probable beam moment may be reduced considering the axial force present in the HBE-to-VBE connections.

3.5.2.3. VBE Design

The high-seismic design of SPSW requires that web-plate tension yielding be the primary source of system inelasticity. Failure of VBE under overturning forces must be precluded at forces corresponding to yielding of the web plate.

The most direct method of achieving this is to design the web plates for the calculated forces with as little overstrength as possible (i.e., with demand-to-capacity ratios as close to unity as possible), and to design the VBE for the sum of the shear strengths of the connected web plates (plus the gravity load). The seismic axial compressive force is thus limited to the sum of the web-plate strengths plus the sum of the HBE shears derived above

$$E_m = \sum \frac{1}{2} R_y F_y \sin(2\alpha) t_w h + \sum V_u \qquad (3\text{–}60)$$

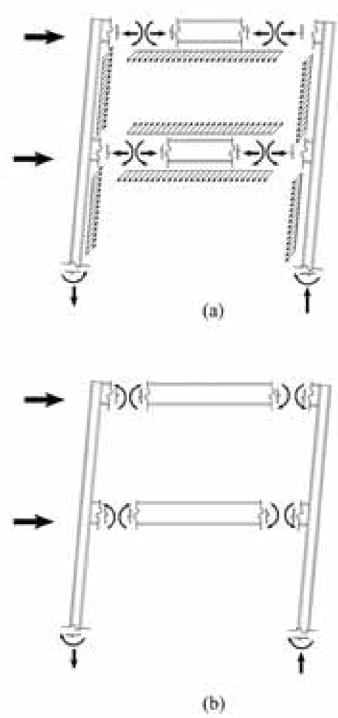

Fig. 3–21. Internal SPSW forces due to (a) web-plate tension (b) flexural deformation.

where

$\sum V_p =$ the sum of beam shears from Equation 3–43.

This force should not be amplified by the overstrength factor Ω_0, as it represents the capacity of the SPSW. The final term is especially important for shorter buildings, as the compression delivered to the column by the top HBE can be significant. For simplicity of calculation, ΣV_p can be bounded by the sum of the beam shear strengths.

Column tension forces can be established similarly. However, across any horizontal section of the SPSW, the seismic tension is shared between the web plate and the VBE, and thus seismic tensile forces in VBE are significantly lower than the corresponding compressive seismic forces in the opposite VBE. In the context of Equation 3–60, the term ΣV_p must be separated into the part that acts upward (the beam shear due to plastic-hinge formation) and the part that acts downward (the force from web-plate tension on the HBE). The expression for seismic axial tension force is

$$E_m = \sum \frac{1}{2} R_y F_y \sin(2\alpha) t_w h_c + \sum \left[\frac{2M_{pr}}{L_h} - \frac{w_u}{2} L_{cf} \right] \quad (3\text{–}61)$$

Note that the forces from web-plate tension on the HBE reduce the tension in the column.

The most accurate method of establishing VBE flexural forces (shears and moments), outside a nonlinear analysis, is to model the VBE as a continuous member on multiple supports (Berman, 2005). Applied to this VBE model are the inward forces due to web-plate tension and the moments from beam plastic hinging (computed at the column centerline as shown in Figure 3–19 and Equation 3–46). Beam supports may be calculated as rigid or as a spring with axial stiffness equivalent to the HBE axial stiffness calculated based on a length equal to ½ L_{cf}. Figure 3–22 shows such a model. HBE axial flexibility is neglected in the figure.

Alternatively, the shears and moments in VBE may be approximated considering the conditions at each story individually. VBE shear is due to both the web-plate tension and the portion of the story shear not resisted by the web plate. The shear due to web-plate tension is

$$V_{VBE(web)} = \frac{1}{2} R_y F_y \sin^2(\alpha) t_w h_c \quad (3\text{–}62)$$

The shear due to hinging of the HBE is

$$V_{VBE(HBE)} = \sum \frac{\frac{1}{2} M_{pb}^*}{h_c} \quad (3\text{–}63)$$

This shear should be at least equal to the portion of the story shear not resisted by the web plate. This force is determined by frame analysis and can be assumed as being shared equally by the two VBE. The total shear is

$$V_u = V_{VBE(web)} + V_{VBE(HBE)} \quad (3\text{–}64)$$

Similarly, VBE moments are due to both the web-plate tension and hinging of the HBE. For a fixed-fixed condition, the moment from web-plate tension at the connection is

$$M_{VBE(web)} = \frac{R_y F_y \sin^2(\alpha) t_w h_c^2}{12} \quad (3\text{–}65)$$

The moment due to hinging of the HBE can be determined from analysis, or, conservatively, one-half of the flexural strengths of the beams can be applied to each column segment at a connection, as indicated by AISC 341 Section 9.6.

It should be noted that Section 17.4a (which invokes Section 9.6, the SMF strong-column/weak-beam check) specifically excludes "consideration of the effects of the webs," and thus Equation 3–64 is not required. It is the opinion of the authors, however, that the flexure from web-plate tension should be considered in conjunction with the forces corresponding to beam hinging. Thus, under this method, this design check is similar to the strong-column/weak-beam check of Section 17.4a.

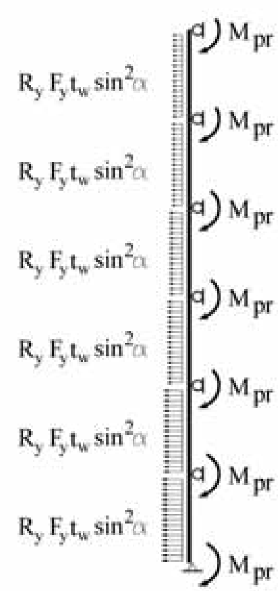

Fig. 3–22. Model of a VBE for computing flexural forces.

The procedure recommended here is twofold. In the VBE design, the moments applied from HBE hinging are not amplified by the factor R_y, nor by the strain-hardening factor of 1.1. The resulting design is therefore likely to result in HBE hinging prior to VBE hinging, although it is not assured that inelastic rotational demands will be precluded in VBE as the HBE strain hardens. In this design check, the critical VBE is the one in compression for three reasons. First, the VBE axial compression force is substantially larger than the tension force. Second, the compression force is additive to gravity forces. Finally, the HBE axial force is less at this connection (due to the collector force and the VBE inward reaction being in opposite directions). Thus, the moment resulting from HBE hinging is larger.

In order to help prevent any VBE hinging from leading to a weak-story condition, a strong-column/weak-beam check is performed. In this case, the factors R_y and 1.1 are used (but the resistance factor on the VBE strength is not). The strong-column/weak-beam check is modified to address the entire SPSW. The greater flexural strength that can be utilized from the VBE in tension is used to supplement that of the VBE in compression, and thus a weak-story condition is avoided. This is presented under "Connection Design."

The moment from HBE hinging is

$$M_{VBE(HBE)} \leq \frac{\frac{1}{2}\sum M^*_{pb}}{1.1R_y} \quad (3\text{–}66)$$

where

M^*_{pb} = the moment at the column centerline due to beam plastic hinging (see "Connection Design")

If the VBE flexural forces are taken from the analysis instead of from capacity design, they should be amplified to reflect the condition at yielding of the web plate or evaluated at the expected displacement. Where a nonlinear analysis is used to model web-plate yielding, the VBE flexural forces from the analysis at the expected drift may be used directly.

The VBE flexure due to beam hinging is typically greater than that due to web-plate tension. In such cases, the flexure away from the connection does not govern the design.

As the required HBE flexural strength is governed by flexure in the span due to web-plate tension, it is convenient to use a Reduced Beam Section (RBS) connection in the HBE to limit the required flexural strength of the VBE. See AISC 358 for a detailed treatment of the design of RBS connections.

The RBS connection is thus proposed for economy in the design of the VBE. Alternatively, the connection may be a more typical welded connection (WUF-W). Such a connection will not reduce the required flexural strength of the HBE, as this is based on resisting web-plate tension after formation of plastic hinges; it will, however, require a greater VBE flexural strength to maintain strong-column/weak-beam proportioning. This should be considered in weighing the economy of the two connections.

It should be noted that in neither case are the quality requirements of SMF applicable to the connection, as these connections are not expected to undergo the large rotations expected for SMF.

3.5.2.4. Axial Force Reduction in VBE

Axial forces corresponding to web-plate yielding at all levels simultaneously can be extremely high. For this reason, alternative methods for estimating maximum forces corresponding to the expected mechanism have been proposed. Three of these are outlined in the Commentary to AISC 341.

The first method is "nonlinear push-over analysis" (POA). This method involves an analysis with incrementally increasing load and element stiffness properties correspondingly modified as yielding occurs. The force distribution selected should favor high overturning moments for purposes of design of the VBE. POA methods are outlined in detail in FEMA 356.

The second method is the "combined linear-elastic computer programs and capacity design concept" (LE+CD). This method involves the design of the VBE at a given level by applying loads from the expected strength of the connecting web plate and adding the overturning loads from levels above using the amplified seismic load:

$$E_m = \frac{1}{2}\, R_y F_y\, \sin(2\alpha)\, t_w h_c + \Omega_0\, E_{(above)} \quad (3\text{–}67)$$

For SPSW, the overstrength factor Ω_0 is 2.0 for the basic system and 2.5 for SPSW in a dual system. Figure 3–23 shows a free-body diagram of the VBE under these seismic loads.

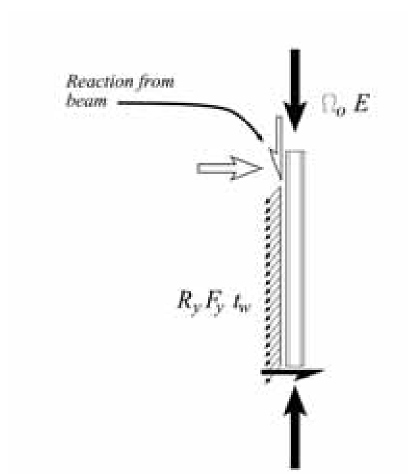

Fig. 3–23. Free-body-diagram of column under LE+CD loading.

To be a true capacity design, the reaction from the beam above should include the effect of web-plate tension.

The third method described is the "indirect capacity design approach" (ICD); this method is based on the CSA-S16-01 code used in Canada (CSA, 2001). In this method, an overstrength factor B is calculated based on the web-plate at the first level.

For LRFD

$$B = \frac{[0.5R_y F_y \sin(2\alpha) t_w L]_1}{(V_u)_1}$$

For ASD (3–68)

$$B = \frac{[0.5R_y F_y \sin(2\alpha) t_w L]_1}{(1.5V_a)_1}$$

where the subscript "1" denotes that values are taken at the first level of the SPSW.

The base overturning is then calculated as B times the overturning moment due to the design seismic forces. This overturning moment is used for the first two levels. Above that, the overturning moment is taken as a linear function between that value and B times the overturning moment due to the design seismic forces at the bottom of the top web plate. The overstrength of the web plates at levels other than the first is not considered in this method. Figure 3–24 shows this diagrammatically.

This moment profile corresponds to a force distribution that is fairly severe with respect to overturning moment. The corresponding loading profile is shown in Figure 3–25.

For convenience, designers may wish to use a computer model to obtain axial forces corresponding to the ICD method. The value of the force can be calculated as

$$F = \frac{B\left[M_1 - M_n\right]}{H_{n-1} - H_2} \qquad (3\text{–}69)$$

where

M_1 = the calculated moment at the bottom of the first level

M_n = the calculated moment at the bottom of the top level

$H_{n\text{-}1}$ = the height above the base of level $(n - 1)$

H_2 = the height above the base of the second level

The height at which this force acts can be calculated as

$$H = \left[H_{n-1} - H_2\right]\left[\frac{1}{1 - \dfrac{M_n}{M_1}}\right] + H_2 \qquad (3\text{–}70)$$

where

H = the height above the base at which the force acts

Figure 3–26 shows the different overturning column compression and tension forces for the design example in Chapter 5 using the sum of web-plate capacities (CAP), the combined linear-elastic computer programs and capacity design concept (LE+CD), the indirect capacity design approach (ICD), and push-over analysis (POA). Tension forces are shown on the left, and compression forces on the right.

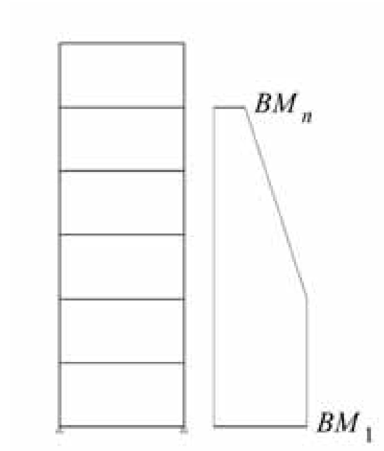

Fig. 3–24. Schematic of ICD overturning moment.

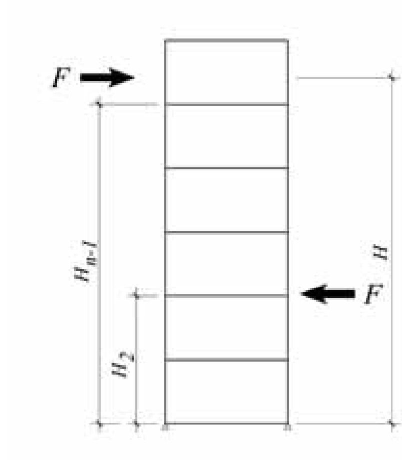

Fig. 3–25. ICD implied force distribution.

Note that for this case, tension forces can be overestimated (above the capacity-design method) using the LE+CD and ICD methods.

It should be noted that using methods other than capacity design for the VBE will lead to designs in which VBE failure is possible, although unlikely. The methods presented are intended to reasonably estimate VBE axial forces based on studies of model buildings, which are typically regular. Where structural irregularities exist, designers should consider carefully whether the LE+CD and ICD methods are sufficient.

3.5.2.5. Configuration

The high overturning forces expected in SPSW can be mitigated by the use of special configurations to distribute the overturning over multiple bays. Figure 3–27 shows four of these configurations: (a) web plate offset at one level; (b) web plate offset at each level; (c) additional web plates at certain levels acting as outriggers to deliver overturning forces to outer columns; and (d) additional web plates at certain levels acting as coupling beams between shear walls.

Designers should be aware that each of these configurations incorporates structural irregularities. All use an in-plane offset, which requires consideration of the structural overstrength in designing both the horizontal elements that transfer the seismic forces from one panel to another, as well as in the vertical elements that resist the overturning. Additionally, the configurations introduce more HBE with web plates only above or only below; these HBE are thus subject to both large axial forces and (simultaneous) large flexural forces. Additionally, where coupling or outrigger web plates are provided at a certain level, that level may have too much strength to participate in the inelastic response. Drift may then be concentrated at other levels.

Additionally, beams can be used as outriggers or couplers between walls. Figure 3–28 shows two such configurations: (a) outrigger beams that deliver overturning forces to outer

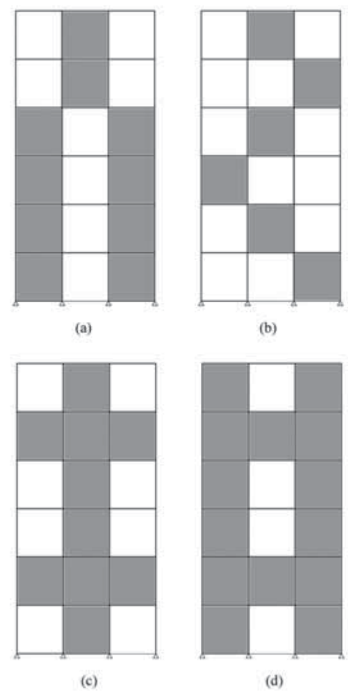

Fig. 3–27. Configurations that reduce overturning by means of web-plate location.

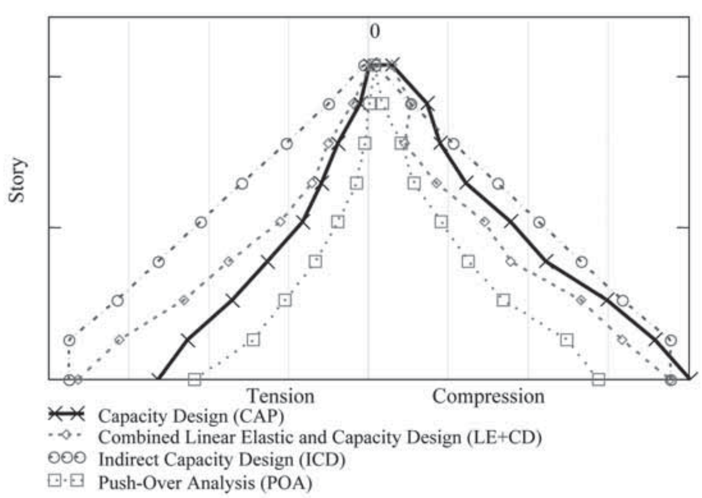

Fig. 3–26. Column axial forces.

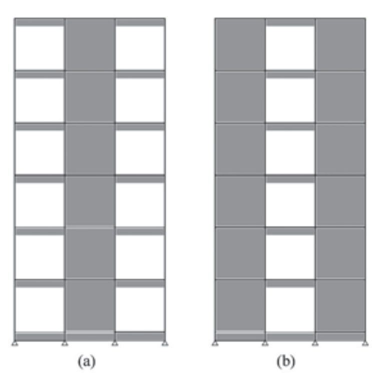

Fig. 3–28. Configurations that reduce VBE overturning forces by means of beams.

columns; and (b) coupling beams between shear walls. Beams used to distribute overturning forces should comply with the requirements of HBE. As in the case of HBE-to-VBE connections, it is preferable that a tested beam-to-column connection be used, and that designs conform to limitations in AISC 358, such as span-to-depth ratio.

3.5.2.6. Connection Design

AISC 341 contains numerous requirements pertaining to the connection of beams (HBE) to columns (VBE) in SPSW. Two of these, the strong-column/weak-beam requirement and the panel-zone strength requirement, require calculation of moments corresponding to plastic-hinge formation in the HBE. The HBE probable moment strength is combined with shear in the beam to calculate the moment at the column centerline (for the strong-column/weak-beam requirement) or at the column face (for the panel-zone strength requirement). The probable moment for each beam is

$$M^*_{pb} = M_{pr} + V_u e \tag{3–71}$$

where

V_p = the shear at the plastic hinge

e = the distance from the plastic hinge to the point at which moments are computed (the column centerline for the strong-column/weak-beam requirement and the column face for the panel-zone strength requirement)

M_{pr} = the probable beam moment as given in Equation 3–40, 3–41, or 3–42

For the strong-column/weak-beam requirement, the eccentricity e is

$$e = s_h \tag{3–72}$$

For the panel-zone strength requirement, the eccentricity e is

$$e = s_h - \frac{1}{2}d_c \tag{3–73}$$

where

d_c = the column depth

The plastic hinge location should be established by plastic analysis where the flexural forces due to gravity loading exceed 30 percent of the beam plastic moment. This concept may be extended to include the flexural forces due to the web-plate tension for purposes of the beam design. However, typical beams (those at levels other than the top and bottom of the SPSW) have moderate flexural demand due to web-plate tension.

As mentioned earlier, designers may wish to calculate a reduced beam flexural strength based on the axial force present in the HBE-to-VBE connection. This will permit the calculation of a lesser required plastic section modulus for the VBE.

As discussed earlier, for the strong-column/weak-beam check, designers may wish to consider both VBE to ensure that a weak-story condition does not exist. This permits utilization of the flexural strength of the VBE in tension, which is far greater due to its lower axial force. For this check, both VBE are considered, as is the axial force in each end of the HBE, and the flexural strength of the adjoining beams outside the SPSW (if rigidly connected).

The required column plastic section modulus (assuming a VBE above and below the connection) is

For LRFD

$$Z_c \ge \frac{1}{2}\frac{\sum M^*_{pb}}{\left[2F_{yc} - \frac{|P_{uC}| + |P_{uT}|}{A_g}\right]}$$

For ASD (3–74)

$$Z_c \ge \frac{1}{2}\frac{\sum M^*_{pb}}{\left[2F_{yc} - \frac{|1.5P_{aC}| + |1.5P_{aT}|}{A_g}\right]}$$

where

F_{yc} = the VBE yield strength

P_{uC} = the axial compression force in the VBE (including the effects of web-plate tension) for LRFD

P_{uT} = the axial tension force in the VBE (including the effects of web-plate tension) for LRFD

P_{aC} = the axial compression force in the VBE (including the effects of web-plate tension) for ASD

P_{aT} = the axial tension force in the VBE (including the effects of web-plate tension) for ASD

A_g = the VBE area

$\sum M^*_{pb}$ = sum of the expected flexural strengths of the beams framing into each VBE (i.e., each end of the HBE, plus the adjoining beams outside the SPSW, if rigidly connected)

The required column web thickness is based on the required panel-zone shear