Seismic design guidelines for solid and perforated hybrid precast concrete shear walls: Appendix: Design example

Brian J. Smith and Yahya C. Kurama

The following example demonstrates the use of the proposed performance-based procedure to design the reinforcement for a solid hybrid precast concrete shear wall for a four-story parking structure in Los Angeles, Calif. **Figure A1** shows the plan view of the building and the elevation view of the wall being designed. The properties of the wall are described in more detail in Smith and Kurama.[1] For brevity, only the base panel reinforcement, the reinforcement crossing the base joint, and the design of the upper panel–to–panel joint located at the first floor (Fig. A1) are discussed in this design example.

Prototype wall properties

Wall dimensions:

Wall height from top of foundation

H_w = 13,716 mm (540 in.) = 13.7 m (45 ft)

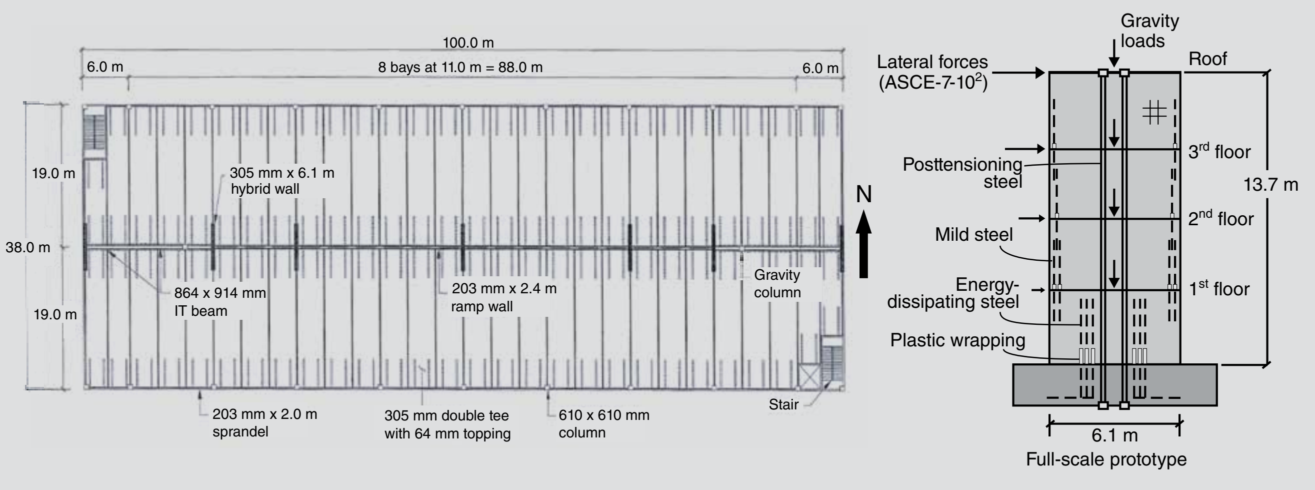

Figure A1. Plan view of full-scale structure and elevation view of full-scale wall. Note: 1 mm = 0.0394 in.; 1 m = 3.28 ft. Drawing courtesy of Consulting Engineers Group, Texas

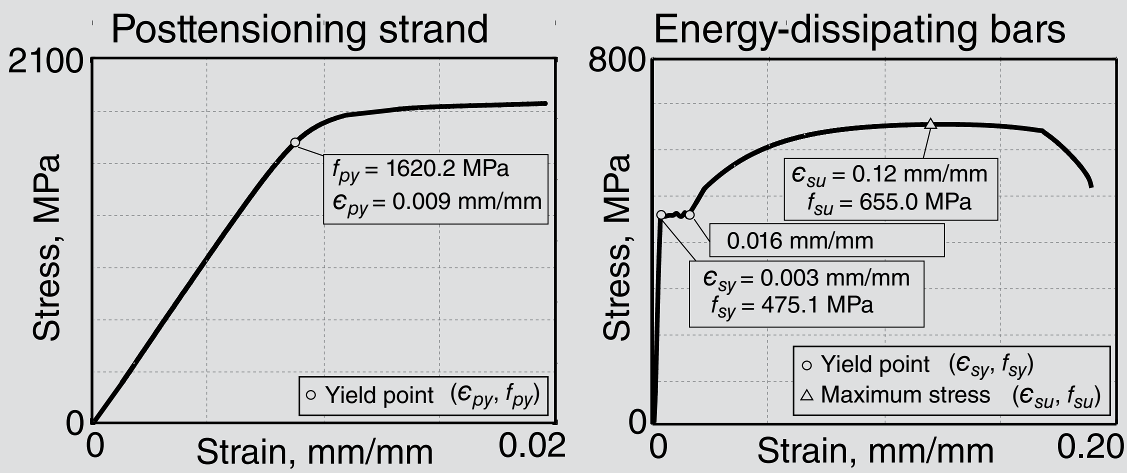

Figure A2. Stress-strain behavior of posttensioning strands and energy-dissipating bars. Note: f_{py} = yield strength of posttensioning steel determined at limit of proportionality point on strand stress-strain relationship; f_{su} = ultimate (maximum) strength of energy-dissipating steel; f_{sy} = yield strength of energy-dissipating steel; ε_{py} = yield strain of posttensioning steel, defined at limit of proportionality point on strand stress-strain relationship; ε_{su} = strain of energy-dissipating steel at f_{su}; ε_{sy} = yield strain of energy-dissipating steel at f_{sy}. 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi; ε_{py} = 0.009 mm/mm.

Wall length L_w = 6096 mm (240 in.) = 6.1 m (20 ft)

Wall thickness t_w = 381 mm (15 in.) = 0.38 m (1.25 ft)

Seismic floor and roof weights assigned to the i^{th} floor level of the prototype building W_i:

Building seismic weight at first floor
W_1 = 22,512 kN (5061 kip)

Building seismic weight at second floor
W_2 = 22,392 kN (5034 kip)

Building seismic weight at third floor
W_3 = 22,392 kN (5034 kip)

Building seismic weight at roof (fourth floor)
W_4 = 21,383 kN (4807 kip)

Fundamental period determined from modal analysis
T_1 = 0.39 sec

ASCE 7-10[2] design spectral response acceleration parameters for building site in Los Angeles:

5% damped design spectral acceleration parameter at short periods $S_{DS} = 1.00g$

5% damped design spectral acceleration parameter at a fundamental period equal to 1.0 second $S_{D1} = 0.64g$

where

g = acceleration due to gravity

Material properties

Concrete:

Compressive strength of unconfined panel concrete
f_c' = 41.41 MPa (6.0 ksi)

Modulus of elasticity of unconfined concrete
$E_c = 4730\sqrt{f_c'} = 4730\sqrt{41.41}$ = 30,438 MPa (4415 ksi)

Shear modulus of concrete G_c
$= E_c/[2(1+\nu_c)]$ = 30,438/[2(1 + 0.18)]
= 12,900 MPa (1870 ksi),
where Poisson's ratio for concrete ν_c is assumed as 0.18.

ACI 318-11[3] factor relating equivalent uniform compression stress block length a_d at Δ_{wd} to neutral axis length c_d
$\beta_1 = 0.75$

ACI ITG-5.2[4] factor relating equivalent uniform confined concrete compressive stress at toe of base panel to confined concrete strength $\gamma_m = 0.92$

ACI ITG-5.2 factor relating equivalent uniform compression stress block length a_d at Δ_{wm} to neutral axis length
$\beta_m = 0.96$

Posttensioning strands (**Fig. A2**):

Ultimate strength of posttensioning steel
f_{pu} = 1861.6 MPa (270.0 ksi)

Yield strength of posttensioning steel
f_{py} = 1620.2 MPa (235.0 ksi)

Initial stress of posttensioning steel after all short-term and long-term losses (but before any lateral displacement of wall)
$f_{pi} = 0.55 f_{pu} = 0.55(1861.6) = 1023.9$ MPa (148.5 ksi)

Modulus of elasticity of posttensioning steel
$E_p = 196{,}501$ MPa (28,500 ksi)

Energy-dissipating bars (Fig. A2):

Ultimate maximum strength of energy-dissipating steel
$f_{su} = 655.0$ MPa (95.0 ksi)

Yield strength of energy-dissipating steel
$f_{sy} = 475.1$ MPa (68.9 ksi)

Strain of energy-dissipating steel at ultimate strength
$\varepsilon_{su} = 0.12$

Modulus of elasticity of energy-dissipating steel
$E_s = 200{,}000$ MPa (29,000 ksi)

ACI ITG-5.2 coefficient to estimate additional energy-dissipating bar debonding that is expected to occur during reversed-cyclic lateral displacements of wall to $\pm \Delta_{wm}$
$\alpha_s = 2.0$

Other mild steel bars:

Yield strength of mild steel $f_{sy} = 413.7$ MPa (60.0 ksi)

Strain of mild steel at ultimate strength $\varepsilon_{su} = 0.08$

Modulus of elasticity of mild steel
$E_s = 200{,}000$ MPa (29,000 ksi)

ASCE 7-10 design force demands

The design of the wall should be conducted under all applicable load combinations prescribed by ASCE 7-10. The response modification factor R is taken as 6.0 considering the structure as a building frame system. For brevity, this design example is demonstrated under a single load combination with the following factored design axial force, design shear force, and design bending moment at the wall base determined using the equivalent lateral force procedure in ASCE 7-10 for the exterior wall in Fig. A1:

Wall design base axial force
$N_{wd} = 1075.6$ kN (241.8 kip)

Wall design base shear force
$V_{wd} = 2385.1$ kN (536.2 kip)

Wall design base moment
$M_{wd} = 24{,}422.3$ kN-m (18,013.0 kip-ft)

This example also includes the design of the first-floor panel-to-panel joint (Fig. A1). The factored design axial force, design shear force, and design bending moment at the first-floor joint are:

Wall design axial force at first-floor panel-to-panel joint
$N_{wd,u} = 935.5$ kN (210.3 kip)

Wall design shear force at first-floor panel-to-panel joint
$V_{wd,u} = 2128.5$ kN (478.5 kip)

Wall design moment at first-floor panel-to-panel joint
$M_{wd,u} = 15{,}698.4$ kN-m (11,578.6 kip-ft)

Seismic drift demands

The linear-elastic drift of the wall can be determined from the linear-elastic flexural and shear deflections. The effective moment of inertia I_e is determined using Eq. (1) from the paper.

$$I_e = 0.5 I_{gross} = (0.5)\left(\frac{1}{12}\right) t_w L_w^3 = (0.5)\left(\frac{1}{12}\right)(0.38)(6.1)^3$$

$$= 3.60 \text{ m}^4 \ (416.7 \text{ ft}^4)$$

The resulting linear-elastic flexural deflection $\delta_{w,flex}$ at the top of the wall can be found assuming a cantilever (with stiffness $E_c I_e$) loaded under the equivalent lateral forces corresponding to V_{wd}.

$\delta_{w,flex} = 0.012$ m = 11.94 mm (0.47 in.)

Similarly, the effective shear area A_{sh} can be found using Eq. (2) from the paper.

$A_{sh} = 0.8 A_{gross} = 0.8 t_w L_w = (0.8)(381)(6096)$
$= 18{,}580{,}610$ mm^2 (2880 in.^{2}) = 1.86 m^2 (20.0 ft^2)

Then, the shear deflection at the top of the wall $\delta_{w,sh}$ can be determined assuming a shear cantilever with stiffness $G_c A_{sh}$.

$\delta_{w,sh} = 0.0013$ m =1.3 mm (0.05 in.)

The total linear-elastic drift of the wall Δ_{we} can be determined from the sum of the flexural and shear deflections.

$$\Delta_{we} = \left(\frac{\delta_{w,flex} + \delta_{w,sh}}{H_w}\right) = \left(\frac{11.94 + 1.3}{13{,}716}\right)$$

$$= 0.000965 \text{ mm/mm} = 0.0965\%$$

The design-level wall drift Δ_{wd} (corresponding to the design-basis earthquake) can be determined from the

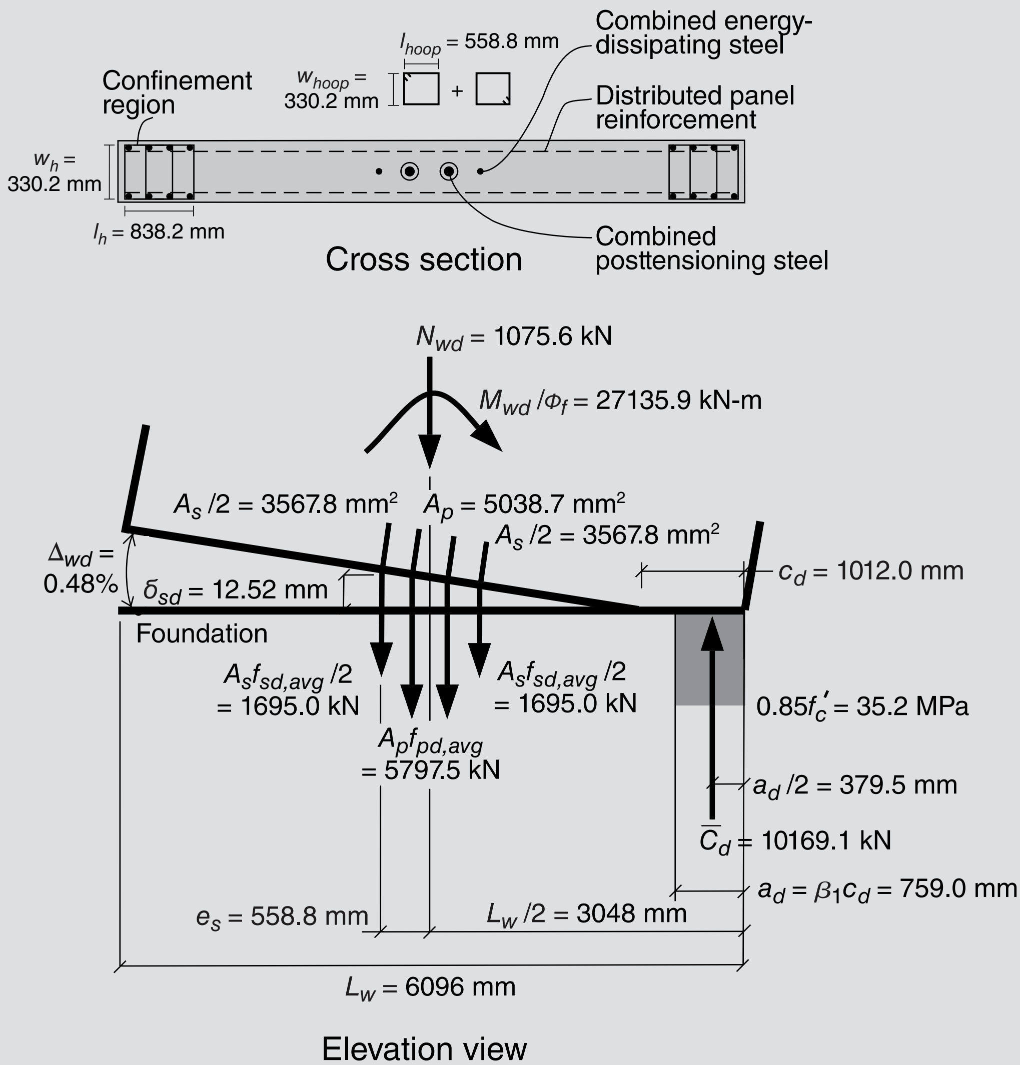

Figure A3. Free body diagram and gap opening of base joint at Δ_{wd}. Note: a_d = length of ACI 318-11 equivalent uniform concrete compression stress block; A_p = total posttensioning steel area; A_s = total energy-dissipating steel area; c_d = neutral axis length; $\bar{C}_d$ = concrete compressive stress resultant; e_s = centroidal distance of combined tension-side and compression-side energy-dissipating bars from wall centerline; f'_c = compression strength of unconfined panel concrete; $f_{pd,avg}$ = average stress for posttensioning steel ; $f_{sd,avg}$ = average stress for energy-dissipating steel; l_h = confined region length at wall toes (center-to-center of bars); l_{hoop} = length of individual confinement hoop (center-to-center of bars); L_w = wall length; M_{wd} = wall design base moment; N_{wd} = factored axial force at wall base for design load combination being considered; w_h = width of confined concrete region at wall toes; w_{hoop} = width of individual confinement hoop (center to center of bars); β_1 = ACI 318-11 factor relating a_d to c_d; δ_{sd} = elongation of energy-dissipating bars; Δ_{wd} = design-level wall drift corresponding to design-basis earthquake; ϕ_f = capacity reduction factor for axial-flexural design of base joint.1 mm = 0.0394 in.; 1 m = 3.28 ft; 1 kN = 0.225 kip; 1 MPa = 0.145 ksi.

linear-elastic drift using section 12.8.6 of ASCE 7-10 with a deflection amplification factor C_d of 5.0.

$$\Delta_{wd} = C_d\Delta_{we} = (5)(0.000965) = 0.004825 \text{ mm/mm} = 0.48\%$$

For this design example, the maximum-level wall drift Δ_{wm} (corresponding to the maximum considered earthquake) is taken as 0.95 times the drift from Eq. (3).[4]

$$\Delta_{wm} = 0.95\Delta_{wc} = (0.95)\left[\left(\frac{H_w}{L_w}\right)(0.8\%) + 0.5\%\right]$$

$$= (0.95)\left[\left(\frac{13,716}{6096}\right)(0.8\%) + 0.5\%\right]$$

$$= 2.19\% = 0.0219$$

Posttensioning and energy-dissipating steel areas crossing base joint

The posttensioning and energy-dissipating steel areas crossing the base joint can be determined using equilibrium, compatibility and kinematics, and design constitutive relationships. The reinforcement is placed in a symmetrical layout and outside the confined concrete boundary (toe) regions of the base panel (Fig. A1). To simplify the presentation of the design process, the posttensioning and energy-dissipating steel areas on the compression side and tension side of the wall centerline are assumed to be combined at their centroidal locations (**Fig. A3**). As a further simplification for the calculations, the steel stresses on the compression side and tension side of the wall are taken as the average steel stresses $f_{pd,avg}$ and $f_{sd,avg}$ considering the total areas for the posttensioning and energy-dissipating reinforcement, respectively (that is, the additional moment resistance due to the unequal tension-side and compression-side steel stresses is ignored). Then, the following equations can be used to determine the concrete compressive stress resultant $\bar{C}_d$ and the neutral axis length c_d at Δ_{wd}.

$$\bar{C}_d = 0.85 f_c' t_w \beta_1 c_d = (0.85)(41.41)(0.38)(0.75)c_d$$

$$\frac{M_{wd}}{\phi_f} = \bar{C}_d \left(\frac{L_w}{2} - \frac{\beta_1 c_d}{2} \right)$$

where

ϕ_f = axial-flexural capacity reduction factor from ACI 318-11 = 0.9

$$\frac{24{,}422.3}{0.9} = \bar{C}_d \left(\frac{6.1}{2} - \frac{0.75 c_d}{2} \right)$$

Solving this system of equations provides the following results:

$\bar{C}_d$ = 10,169.1 kN (2286.1 kip)

c_d = 1.01 m = 1012.0 mm (39.84 in.) = $0.166L_w$

The different stresses in the individual posttensioning tendons and energy-dissipating bars should be incorporated in the final design of the wall. Furthermore, the previously stated simplifications should not be made if the distance from the wall centerline to the extreme posttensioning tendon or energy-dissipating bar is greater than $0.125L_w$ (instead, estimated values for the steel stresses should be incorporated into the following iteration).

Assuming that the design-level drift is achieved by the rigid body rotation of the wall through the gap opening at the base joint (Fig. A3), the posttensioning and energy-dissipating steel strains (and stresses) can be estimated using the neutral axis length c_d. At this stage, the different strains and stresses in the combined compression-side and tension-side posttensioning and energy-dissipating steel are also calculated. The design process requires initial estimations of the combined posttensioning and energy-dissipating steel eccentricities e_p and e_s (that is, centroidal distances from the centerline of the wall).

Posttensioning steel:

e_p = 203.2 mm (8.0 in.) (initial estimate)

Unbonded length of posttensioning steel
l_{pu} = 15.2 m = 15,240 mm (600 in.)

Elongations of posttensioning steel
$\delta_{pd} = \Delta_{wd}(0.5L_w - c_d \pm e_p)$
= (0.004825)[(0.5)(6096) – 1012.0 ± 203.2]
= 8.84 and 10.80 mm (0.35 and 0.43 in.) for the combined compression-side and tension-side tendons, respectively.

Then, the posttensioning steel stresses f_{pd} at Δ_{wd} can be found.

$$f_{pd} = f_{pi} + E_{pi}\frac{\delta_{pd}}{l_{pu}}$$

$$f_{pd} = 1023.9 + 196{,}501\frac{8.84}{15{,}240}$$

= 1137.9 MPa (165.0 ksi) for the compression-side tendon

$$f_{pd} = 1023.9 + 196{,}501\frac{10.80}{15{,}240}$$

= 1163.2 MPa (168.7 ksi) for the tension-side tendon

Both stresses are in the linear-elastic range (Fig. A2).

Average stress for the total posttensioning steel area crossing the base joint $f_{pd,avg}$ = 1150.6 MPa (166.9 ksi)

Energy-dissipating steel:

e_s = 558.8 mm (22.0 in.) (initial estimate)

Wrapped length of energy-dissipating steel
l_{sw} = 812.8 mm (32.0 in.)

Elongations of energy-dissipating steel
$\delta_{sd} = \Delta_{wd}(0.5L_w - c_d \pm e_s)$
= (0.004825)[(0.5)(6096) – 1012.0 ± 558.8]
= 7.13 and 12.52 mm (0.28 and 0.49 in.) for the combined compression-side and tension-side bars, respectively.

Then, the energy-dissipating steel strains ε_{sd} at Δ_{wd} are calculated.

$$\varepsilon_{sd} = \frac{\delta_{sd}}{l_{sw}} = \frac{7.13}{812.8} \text{ and } \frac{12.52}{812.8}$$

= 0.0088 and 0.0154 for the combined compression-side and tension-side bars, respectively

Both strainss are within range of the yield plateau (Fig. A2).

Average stress for the total energy-dissipating steel area crossing the base joint $f_{sd,avg} = f_{sy}$ = 475.1 MPa (68.9 ksi)

Ignoring the differences in the compression-side and tension-side steel stresses (that is, assumingthat the steel stresses are equal to the average stress), equilibrium along the base joint combined with Eq. (4) can be used to determine the required total posttensioning and energy-dissipating steel areas. The designer must select a value for the energy-dissipating steel moment ratio κ_d within the recommended range of $0.50 \le \kappa_d \le 0.80$. For this design example, 0.50 is used for κ_d.

$$\bar{C}_d = A_s f_{sd,avg} + A_p f_{pd,avg} + N_{wd}$$

where

A_p = total posttensioning steel area (includes the tension-side and compression-side tendons)

A_s = total energy-dissipating steel area

$10{,}169.1 = A_s(475.1) + A_p(1150.6) + 1075.6$

$$\kappa_d = \frac{M_{ws}}{M_{wp} + M_{wn}} = \frac{A_s f_{sd,avg}\left(\frac{L_w}{2} - \frac{\beta_1 c_d}{2}\right)}{\left(A_p f_{pd,avg} + N_{wd}\right)\left(\frac{L_w}{2} - \frac{\beta_1 c_d}{2}\right)} = \frac{A_s f_{sd,avg}}{A_p f_{pd,avg} + N_{wd}}$$

$$0.5 = \frac{A_s(475.1)}{A_p(1150.6) + 1075.6}$$

Solving this system of equations gives the following results:

A_p = 4956.8 mm² (7.68 in.²)

A_s = 7135.5 mm² (11.06 in.²)

For this example, the provided reinforcement crossing the base joint consists of thirty-six 15.2 mm (0.6 in.) diameter posttensioning strands with A_p of 5038.7 mm² (7.81 in.²) and fourteen 25M (no. 8) energy-dissipating bars with A_s of 7135.5 mm² (11.06 in.²). All of the subsequent design calculations should be made using these values of A_p and A_s. Seven energy-dissipating bars are placed at a center-to-center spacing of 76.2 mm (3.0 in.) on each side of the wall centerline, and the posttensioning strands on each side are placed in a single tendon with eighteen strands in the tendon (**Fig. A4**). The placement of the energy-dissipating bars and the posttensioning strands satisfy the previous estimations of the steel eccentricities e_p and e_s. This design step may need to be revised if the eccentricities or unbonded lengths of the posttensioning and energy-dissipating steel are modified later in the design.

Probable base moment strength (initial calculation)

Based on the initial estimations for the eccentricities and unbonded lengths of the compression-side and tension-side posttensioning and energy-dissipating steel, the material stresses at the maximum-level wall drift Δ_{wm} can be determined and used to calculate the probable base moment strength of the wall M_{wm}. This process is iterative and requires an assumed neutral axis length c_m at Δ_{wm}.

Assume $c_m = 0.9c_d = (0.9)(1012.0) = 910.8$ mm (35.9 in.) = 0.91 m (3.0 ft) (initial estimate).

Then, the posttensioning and energy-dissipating steel strains and stresses can be calculated.

Posttensioning steel:

Elongations of posttensioning steel
$\delta_{pm} = \Delta_{wm}(0.5L_w - c_m \pm e_p)$
$= (0.0219)[(0.5)(6096) - 910.8 \pm 203.2]$
= 42.35 and 51.25 mm (1.67 and 2.02 in.) for the compression-side and tension-side tendons, respectively.

Then, the posttensioning steel strains ε_{pm} at Δ_{wm} are calculated.

$$\varepsilon_{pm} = \frac{f_{pi}}{E_p} + \frac{\delta_{pm}}{l_{pu}} = \frac{1023.9}{196{,}501} + \frac{42.35 \text{ and } 51.25}{15{,}240}$$

= 0.0080 and 0.0086 for the compression-side and tension-side tendons, respectively

The posttensioning steel stresses f_{pm} are calculated based on the assumed strand stress-strain relationship in Fig. A2.

f_{pm} = 1502.4 and 1588.6 MPa (217.9 and 230.4 ksi) for the compression-side and tension-side tendons, respectively

Average stress for the total posttensioning steel area crossing the base joint $f_{pm,avg}$ = 1545.5 MPa (224.2 ksi)

Energy-dissipating steel:

Elongations of energy-dissipating steel
$\delta_{sm} = \Delta_{wm}(0.5L_w - c_m \pm e_s)$
$= (0.0219)[(0.5)(6096) - 910.8 \pm 558.8]$
= 34.57 and 59.04 mm (1.36 and 2.32 in.) for the compression-side and tension-side energy-dissipating bars, respectively.

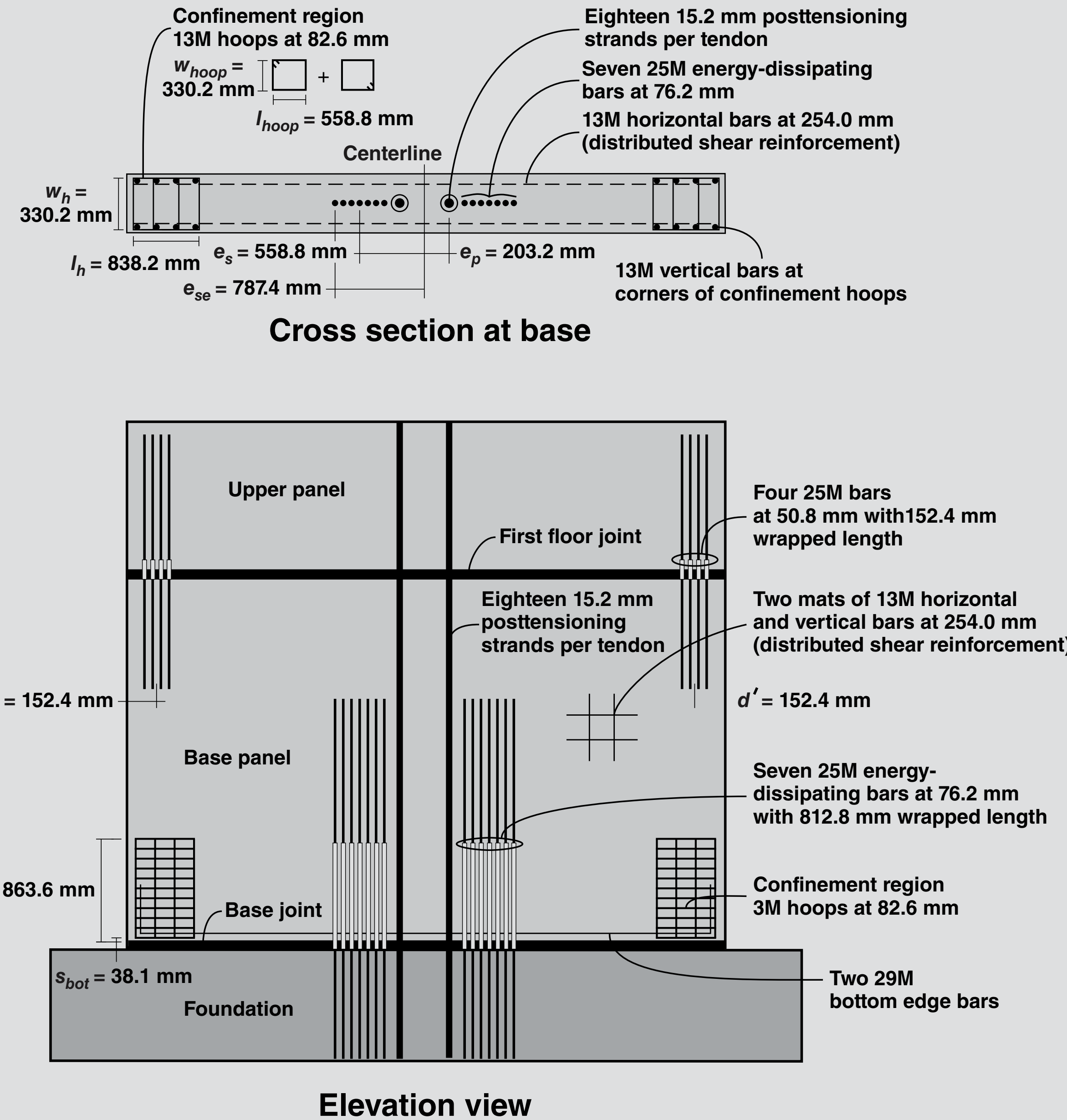

Figure A4. Cross section (at base) and elevation of wall reinforcement details. Note: d = centroid location (from panel end) of tension mild steel bars crossing upper panel–to–panel joint from panel end; d' = centroidal distance of compression mild steel bars crossing upper panel–to–panel joint from panel end; e_p = centroidal distance of tension-side and compression-side posttensioning tendons from wall centerline; e_s = centroidal distance of tension-side and compression-side energy-dissipating bars from wall centerline; e_{se} = distance of extreme energy-dissipating bar from wall centerline; l_h = confined region length at wall toes (center to center of bars); l_{hoop} = length of individual confinement hoop (center to center of bars); s_{bot} = first confinement hoop distance from bottom of base panel (to center of bar); w_h = width of confined concrete region at wall toes; w_{hoop} = width of individual confinement hoop (center-to-center of bars). 13M = no. 4; 25M = no. 8; 29M = no. 9; 1 mm = 0.0394 in.; m = 3.28 ft.

Then, the energy-dissipating steel strains ε_{sm} at Δ_{wm} are calculated.

$$\varepsilon_{sm} = \frac{\delta_{sm}}{l_{sw} + \alpha_s d_s} = \frac{34.57 \text{ and } 59.04}{812.8 + (2.0)(25.4)}$$

= 0.040 and 0.068 for the compression-side and tension-side energy-dissipating bars, respectively, where d_s = diameter of energy-dissipating bar

The energy-dissipating steel stresses f_{sm} are calculated based on the assumed steel stress-strain relationship in Fig. A2.

f_{sm} = 585.4 and 632.9 MPa (84.9 and 91.8 ksi) for the combined compression-side and tension-side energy-dissipating bars, respectively

Average stress for the total energy-dissipating steel area crossing the base joint $f_{sm,avg}$ = 609.2MPa (88.4 ksi)

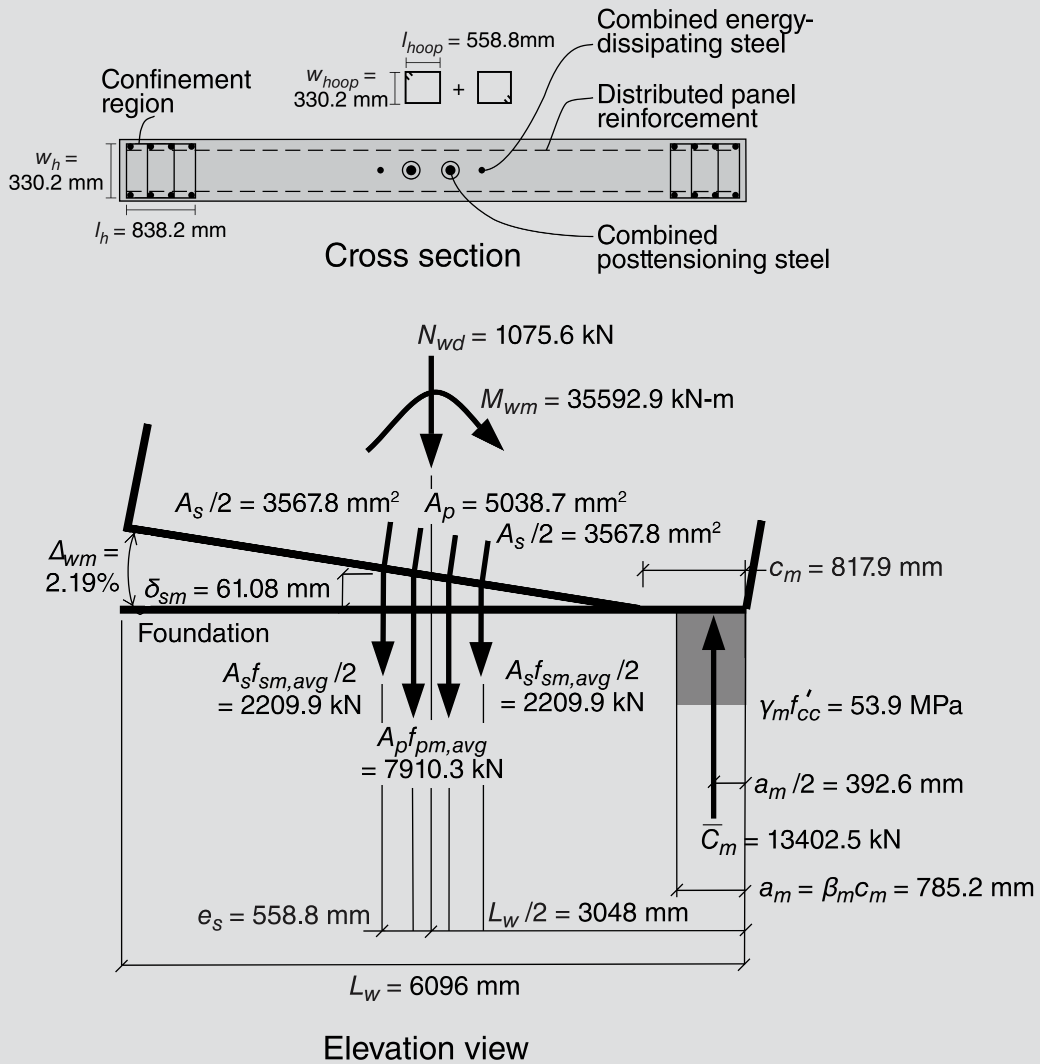

Figure A5. Free body diagram and gap opening of base joint at Δ_{wm}. Note: a_m = length of ACI ITG-5.2 equivalent uniform concrete compression stress block at Δ_{wm}; A_p = total posttensioning steel area; A_s = total energy-dissipating steel area; c_m = neutral axis length; $\bar{C}_m$ = concrete compressive stress resultant; e_s = centroidal distance of combined tension-side and compression-side energy-dissipating bars from wall centerline; f'_{cc} = compression strength of confined concrete at toes of base panel; $f_{pm,avg}$ = average stress for posttensioning steel; $f_{sm,avg}$ = average stress for energy-dissipating steel; l_h = confined region length at wall toes (center to center of bars); l_{hoop} = length of individual confinement hoop (center to center of bars); L_w = wall length; M_{wm} = probable base moment strength of wall; N_{wd} = factored axial force at wall base for design load combination being considered; w_h = width of confined concrete region at wall toes; w_{hoop} = width of individual confinement hoop (center to center of bars); β_m = ACI ITG-5.2 factor relating a_m to c_m; γ_m = ACI ITG-5.2 factor relating equivalent uniform confined concrete compression block stress at toe of base panel to f'_{cc}; δ_{sm} = elongation of energy-dissipating bars; Δ_{wm} = maximum-level wall drift corresponding to maximum-considered earthquake; ϕ_f = capacity reduction factor for axial-flexural design of base joint. 1 mm = 0.0394 in.; 1 m = 3.28 ft; 1 kN = 0.225 kip; 1 MPa = 0.145 ksi.

Using equilibrium along the base joint at Δ_{wm} (**Fig. A5** shows the final design values at the end of the iteration), the concrete compressive stress resultant $\bar{C}_m$ and the probable base moment strength of the wall M_{wm} can be determined.

$$\bar{C}_m = A_s f_{sm,avg} + A_p f_{pm,avg} + N_{wd}$$

$$= \left[\frac{(7135.5)}{10^6}\right]\left[(609.2)(1000)\right] + \left[\frac{(5038.7)}{10^6}\right]\left[(1545.5)(1000)\right] + 1075.6$$

= 13,209.9 kN (2969.7 kip)

$$M_{wm} = \bar{C}_m\left(\frac{L_w}{2} - \frac{\beta_m c_m}{2}\right) = 13,209.9\left[\frac{6.1}{2} - \frac{(0.96)(0.91)}{2}\right]$$

= 34,520.1 kN-m (25,460.7 kip-ft)

In this calculation, the differences in the compression-side and tension-side steel stresses are again ignored by using the estimated average steel stresses. At Δ_{wm}, the confined concrete in compression is represented using an equivalent uniform stress block with the concrete strength parameter γ_m of 0.92 and the stress block length parameter β_m of 0.96, as given in section 5.6.3.8(a) of ACI ITG-5.2.

Confinement reinforcement at wall toes

Confinement reinforcement is required at the ends of the base panel and can be designed using sections 5.6.3.5 through 5.6.3.9 of ACI ITG-5.2.[9] The confined concrete is designed for a maximum concrete strain ε_{cm} at Δ_{wm}, determined as follows:

Plastic hinge height
$h_p = 0.06H_w = (0.06)(13{,}716) = 823.0$ mm (32.4 in.)

Plastic curvature at wall base $\phi_{wm} = \dfrac{\Delta_{wm}}{h_p} = \dfrac{0.0219}{823.0}$

$= 0.0000266$/mm

$$\varepsilon_{cm} = \phi_{wm} c_m = (0.0000266)(910.8) = 0.0242$$

The required confined concrete compressive strength can then be calculated using Eq. (5-5) and Table 5.1 of ACI ITG-5.2, which relates the lateral confining stress provided by the transverse reinforcement f_l', the confined concrete compressive strength f_{cc}', and the unconfined concrete compressive strength f_c'.

$$\varepsilon_{cm} = 0.004 + 4.6\varepsilon_{su}\left(\frac{f_l'}{f_{cc}'}\right)$$

$$0.0242 = 0.004 + (4.6)(0.08)\left(\frac{f_l'}{f_{cc}'}\right)$$

$$\left(\frac{f_l'}{f_{cc}'}\right) = 0.055$$

Then, from Table 5.2 of ACI ITG-5.2:

$$\left(\frac{f_{cc}'}{f_c'}\right) = 1.48$$

$$f_{cc}' = 1.48 f_c' = (1.48)(41.41) = 61.29 \text{ MPa } (8.89 \text{ ksi})$$

Using axial force equilibrium at Δ_{wm}, the initial assumption of c_m equal to $0.9c_d$ should be checked and, if necessary, the previous steps to determine the probable base moment strength and required confined concrete compressive strength should be iterated until the calculated c_m from equilibrium at the wall base is sufficiently close to the assumed c_m. For this design example, a clear cover of 25.4 mm (1.0 in.) is assumed at the wall toes, resulting in a confined concrete width w_h of 330.2 mm (13.0 in.). Therefore:

$$\bar{C}_m = \gamma_m f_{cc}' w_h \beta_m c_m = (0.92)\left(\frac{61.29}{1000}\right)(330.2)(0.96)c_m$$

$$= 13{,}209.9 \text{ kN } (2969.7 \text{ kip})$$

$$c_m = 739.1 \text{ mm } (29.1 \text{ in.}) < 0.9c_d = 910.8 \text{ mm } (35.9 \text{ in.})$$

Because of the large difference between the initial assumption of c_m and the calculated c_m from equilibrium, c_m and f_{cc}' should be recalculated. Through this iteration, which is not shown here, the following final design values can be determined:

$$f_l' = 2.85 \text{ MPa } (0.414 \text{ ksi})$$

$$f_{cc}' = 58.6 \text{ MPa } (8.50 \text{ ksi})$$

$$c_m = 817.9 \text{ mm } (32.2 \text{ in.}) = 0.134L_w = 0.24c_d$$

These parameters are then used to determine the confinement hoop layout and spacing according to sections 5.6.3.5 through 5.6.3.9 in ACI ITG-5.2. For rectangular confinement hoops with f_{sy} of 413.7 MPa (60.0 ksi), the confinement reinforcement ratio ρ_h can be calculated using section 5.6.3.8(b).

$$\rho_h = \frac{f_l'}{0.35 f_{sy}} = \frac{2.85}{(0.35)(413.7)} = 0.020$$

The length of the confined concrete region at each wall end l_h is assumed to be equal to 838.2 mm (33.0 in.), thus satisfying the following design requirement:

$$l_h > 0.95c_m$$

838.2 mm > (0.95)(817.9) = 777.0 mm (33.0 in. > 30.6 in.) → OK

Then, two overlapping 13M (no. 4) hoops are selected for each wall end (Fig. A3)

Hoop bar area $a_{hoop} = 129.0$ mm^2 (0.20 in.^{2})

Hoop width $w_{hoop} = 330.2$ mm (13.0 in.)

Individual hoop length $l_{hoop} = 558.8$ mm (22.0 in.)

This satisfies the following requirement:

$$\frac{l_{hoop}}{w_{hoop}} < 2.50$$

$$\frac{558.8}{330.2} = 1.69 < 2.50 \rightarrow \text{OK}$$

The confinement hoop spacing s_{hoop} can be determined using the following equation based on the specific geometry of the selected hoop layout.

$$\rho_h = \frac{a_{hoop}\left(4l_{hoop} + 4w_{hoop}\right)}{l_h w_h s_{hoop}}$$

where

w_h = width of the confined concrete region

Assuming that $w_h = w_{hoop}$, then:

$$0.020 = \frac{(129.0)\left[(4)(558.8)+(4)(330.2)\right]}{(838.2)(330.2)s_{hoop}}$$

Solving this equation for the confinement hoop spacing yields:

s_{hoop} = 82.9 mm (3.3 in.)

For this example, the provided confinement hoop spacing is 82.6 mm (3.25 in.). The confinement hoops should extend vertically over a height of the base panel not less than the plastic hinge height h_p of 823.0 mm. (32.4 in.). Thus, placing 11 layers of hoops at each end of the wall while providing a distance from the first hoop to the bottom of the base panel s_{bot} of 38.1 mm (1.5 in.), the total height of the confinement region would be 863.6 mm (34.0 in.). For ease of handling and placement of the hoops at each wall end as well as maintain the integrity of the cnofined region under loading, a confinement cage should be constructed by tying the corners of the hoops to 13M (no. 4) vertical bars.

Probable base moment strength (final calculation)

The probable base moment strength of the wall is recalculated using the finalized neutral axis length c_m and confined concrete strength f'_{cc}, resulting in the following:

δ_{pm} = 44.4 and 53.3 mm (1.75 and 2.10 in.) for the compression-side and tension-side tendons, respectively

ε_{pm} = 0.0081 and 0.0087 for the compression-side and tension-side tendons, respectively

$f_{pm,avg}$ = 1569.9 MPa (227.7 ksi) (average for total posttensioning steel area crossing the base joint)

δ_{sm} = 36.60 and 61.08 mm (1.44 and 2.40 in.) for the compression-side and tension-side energy-dissipating bars, respectively

ε_{sm} = 0.042 and 0.071 for the compression-side and tension-side energy-dissipating bars, respectively

$f_{sm,avg}$ = 619.4 MPa (89.8 ksi) (average for total energy-dissipating steel area crossing the base joint)

$\bar{C}_m$ = 13,402.5 kN (3013.0 kip)

M_{wm} = 35,592.9 kN-m (26,252 kip-ft)

The corresponding wall base shear force can be calculated as:

$$V_{wm} = \left(\frac{M_{wm}}{M_{wd}}\right)V_{wd} = \left(\frac{35{,}592.9}{24{,}422.3}\right)(2385.1) = 3476.0 \text{ kN (781.4 kip)}$$

Maximum energy-dissipating steel strain and wrapped length

The calculated neutral axis length c_m can be used to check the initial assumption for the energy-dissipating bar wrapped length (used to estimate the energy-dissipating steel strains) based on the maximum energy-dissipating bar strain. The maximum energy-dissipating bar strain will occur in the extreme bar at the tension side of the wall (located a distance from the wall centerline e_{se} of 787.4 mm [31 in.]). The designer must select an allowable energy-dissipating bar strain ε_{sa} based on the recommended limits of $0.5\varepsilon_{su} \le \varepsilon_{sa} \le 0.85\varepsilon_{su}$. For this design example, it is assumed that:

$\varepsilon_{sa} = 0.65\varepsilon_{su}$ = (0.65)(0.12) = 0.078

Then, based on Fig. A5:

$$\begin{aligned}\delta_{sm} &= \Delta_{wm}(0.5L_w - c_m + e_{se})\\ &= (0.0219)[(0.5)(6096) - 817.9 + 787.4]\\ &= 66.1 \text{ mm (2.60 in.)}\end{aligned}$$

$$\varepsilon_{sm} = \frac{\delta_{sm}}{l_{sw} + \alpha_s d_s} = \frac{66.1}{l_{sw} + (2.0)(25.4)} \le \varepsilon_{sa} = 0.078$$

Therefore, $l_{sw} \ge$ 796.6 mm (31.4 in.)

The initial assumption for the energy-dissipating bar wrapped length l_{sw} of 812.8 mm (32.0 in.) is close to and satisfies the listed limit. No further iteration is required.

A final check should be performed to determine whether the lower limit for energy-dissipating bar strain is satisfied when using the selected wrapped length. This check should

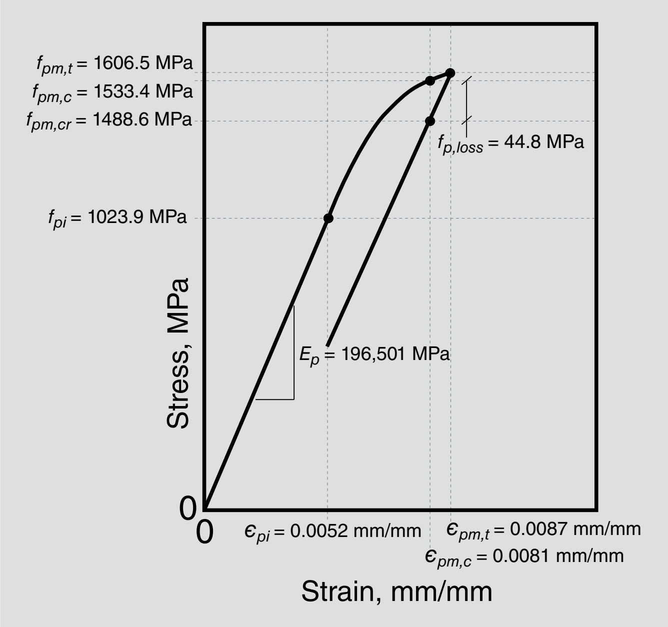

Figure A6. Posttensioning tendon stresses under repeated cycles to $\pm\Delta_{wm}$. Note: E_p = modulus of elasticity; $f_{p,loss}$ = stress loss in compression-side tendon at $+\Delta_{wm}$ due to nonlinear material behavior associated with reversed-cyclic loading of wall to $\pm\Delta_{wm}$; f_{pi} = initial stress after all short-term and long-term losses (but before any lateral displacement of wall); $f_{pm,t}$ = stress in tension-side tendon at initial and repeated loading to $+\Delta_{wm}$; $f_{pm,c}$ = stress in compression-side tendon at initial loading to $+\Delta_{wm}$; $f_{pm,cr}$ = stress in compression-side tendon after repeated loading to $+\Delta_{wm}$; Δ_{wm} = maximum-level wall drift corresponding to maximum-considered earthquake; $\varepsilon_{pm,t}$ = strain in tension-side tendon at initial and repeated loading to $+\Delta_{wm}$; $\varepsilon_{pm,c}$ = strain in compression-side tendon at initial and repeated loading to $+\Delta_{wm}$. 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

be performed using the average tension-side energy-dissipating bar elongation:

$$0.50\varepsilon_{su} = (0.50)(0.12) = 0.060$$

$$\varepsilon_{sm} = \frac{\delta_{sm}}{l_{sw} + \alpha_s d_s} = \frac{61.08}{812.8 + (2.0)(25.4)} = 0.071$$

$0.071 > 0.060 \rightarrow$ OK

Maximum posttensioning steel strain

Similarly, the calculated neutral axis length c_m can be used to determine the maximum posttensioning steel strain and ensure that it does not exceed the recommended limit of 0.01. The maximum posttensioning steel strain will occur in the tension-side tendon. Using the previously defined equations:

$$\delta_{pm} = \Delta_{wm}(0.5L_w - c_m + e_p)$$
$$= (0.0219)[(0.5)(6096) - 817.9 + 203.2]$$
$$= 53.3 \text{ mm (2.10 in.)}$$

Therefore,

$$\varepsilon_{pm} = \frac{f_{pi}}{E_p} + \frac{\delta_{pm}}{l_{pu}} = \frac{1023.9}{196{,}501} + \frac{53.3}{15{,}240} = 0.0087 < 0.01 \rightarrow \text{OK}$$

Posttensioning stress losses

A small posttensioning stress loss will occur as the wall is displaced to $\pm\Delta_{wm}$ under fully reversed and cyclic loading and the strand strains exceed the limit of proportionality (that is, as the stress-strain behavior of the strand becomes nonlinear). **Figure A6** displays the stress-strain relationship of the tension-side and compression-side posttensioning tendons as the wall is cyclically displaced to $\pm\Delta_{wm}$. Let the stresses in the tension-side and compression-side tendons on first loading to $+\Delta_{wm}$ be $f_{pm,t}$ and $f_{pm,c}$, respectively. Upon reversed loading of the wall to $-\Delta_{wm}$ and reloading back to $+\Delta_{wm}$, the tension-side tendon will return to the same point on the stress-strain curve (that is, the stress and strain of the tendon will equal $f_{pm,t}$ and $\varepsilon_{pm,t}$, respectively) and the tendon will not experience any stress loss due to the nonlinear material behavior. The compression-side tendon will also return to the same strain (that is, $\varepsilon_{pm,c}$), but it will incur stress losses because of the greater strain (equal to $\varepsilon_{pm,t}$) that it experiences in the reverse loading direction (that is, when the wall is subjected to $-\Delta_{wm}$). The stress loss $f_{p,loss}$ when the wall is displaced back to $+\Delta_{wm}$ can be determined as follows:

$$\delta_{pm} = \Delta_{wm}(0.5L_w - c_m + e_p)$$
$$= (0.0219)[(0.5)(6096) - 817.8 \pm 203.2]$$
$= 44.4$ and 53.3 mm (1.75 or 2.10 in.) for the compression-side and tension-side tendons, respectively

$$\varepsilon_{pm} = \frac{f_{pi}}{E_p} + \frac{\delta_{pm}}{l_{pu}} = \frac{1023.9}{196{,}501} + \frac{44.4 \text{ and } 53.3}{15{,}240}$$

$= 0.0081$ and 0.0087 for the compression-side and tension-side tendons, respectively.

Therefore,

$\varepsilon_{pm,t} = 0.0087$

$\varepsilon_{pm,c} = 0.0081$

$f_{pm,t}$ = 1606.5 MPa (233.0 ksi) (based on the posttensioning strand stress-strain relationship in Fig. A2)

$f_{pm,c}$ = 1533.4 MPa (222.4 ksi) (based on Fig. A2)

$f_{pm,avg}$ = 1569.9 MPa (227.7 ksi) (average for the total posttensioning steel area crossing the base joint)

Stress in compression-side posttensioning tendon after repeated loading $f_{pm,cr}$:

$$f_{pm,cr} = f_{pm,t} - E_p(\varepsilon_{pm,t} - \varepsilon_{pm,c})$$
$$= 1606.5 - 196{,}501(0.0087 - 0.0081)$$
$$= 1488.6 \text{ MPa (215.9 ksi)}$$

$$f_{p,loss} = f_{pm,c} - f_{pm,cr} = 1533.4 - 1488.6 = 44.8 \text{ MPa (6.50 ksi)}$$

Wall restoring force

Eq. (5) must be satisfied to provide a sufficient axial restoring force for the wall. For design purposes, a capacity reduction factor against loss of restoring ϕ_r of 0.90 is used. Therefore, the design equation is:

$$\phi_r\left[A_p\left(f_{pm,avg}-0.5f_{p,loss}\right)+N_{wd}\right]>A_s\left(f_{sm,avg}+f_{sy}\right)$$

$$0.9\left\{(5038.7)\left[\frac{1569.9}{1000}-(0.5)\left(\frac{44.8}{1000}\right)\right]+1075.6\right\}$$

$$>(7135.5)\left(\frac{619.4}{1000}+\frac{475.1}{1000}\right)$$

7985.7 kN (1795.3 kip) > 7809.8 kN (1755.7 kip) → OK

Shear design of base joint

To prevent significant horizontal slip of the wall during loading to the maximum-level drift Δ_{wm}, the shear friction strength at the horizontal joints V_{ss} should be greater than the maximum joint shear force V_{jm}:

$$\phi_s V_{ss} > V_{jm}$$

where

ϕ_s = 0.75 is the capacity reduction factor for shear design

At the base joint:

$V_{jm} = V_{wm}$ = 3476.0 kN (781.4 kip) as calculated previously

Then, using a shear friction coefficient μ_{ss} of 0.5 as recommended by section 5.5.3 of ACI ITG-5.2 along with the compressive force at the wall base to determine the shear friction strength V_{ss}, the design equation can be written as:

$$\phi_s\mu_{ss}\left(\bar{C}_m-0.5A_pf_{p,loss}\right)>V_{wm}$$

$$(0.75)(0.5)\left[13{,}402.5-(0.5)(5038.7)\frac{(44.8)}{1000}\right]>3476$$

4983.6 kN (1120.4 kip) > 3476.0 kN (781.4 kip) → OK

Base panel shear reinforcement

The base panel is expected to develop diagonal cracking; thus, distributed vertical and horizontal steel reinforcement should be designed based on the shear requirements in sections 21.9.2 and 21.9.4 of ACI 318-11. This distributed reinforcement should be designed to ensure that the shear (diagonal tension) strength of the base panel V_n is greater than the maximum wall base shear force V_{wm}:

$$\phi_s V_n > V_{wm}$$

As specified in section 21.9.4 of ACI 318-11, the shear strength is a function of the gross area of concrete in the direction of the shear force $L_w t_w$, the shear steel reinforcement ratio ρ_t, and the material properties of the concrete and steel. For design purposes, a capacity reduction factor ϕ_s of 0.75 is again used. Then, the design equation can be written as:

$$\phi_s\left(L_w t_w\right)\left(\alpha_c\lambda\sqrt{f_c'}+\rho_t f_{sy}\right)>V_{wm}$$

where

α_c = ACI 318-11 coefficient defining relative contribution of concrete strength to nominal wall shear strength

λ = ACI 318-11 factor reflecting reduced mechanical properties of lightweight-aggregate concrete relative to normalweight aggregate concrete

For H_w/L_w = 13.7/6.1 = 2.25, α_c = 0.17

$$(0.75)(6096)(381)\left[(0.17)(1.0)\frac{\sqrt{41.41}}{1000}+\rho_t\frac{(413.7)}{1000}\right]$$

> 3476.0 kN

Solving the design equation for the shear reinforcement ratio yields:

$\rho_t > 0.0022$

As required by section 21.9.2.1 of ACI 318-11, the reinforcement ratio ρ_t shall not be less than 0.0025. Therefore, using this minimum requirement, the required shear reinforcement area A_t can be determined:

$A_t = \rho_t t_w$ = (0.0025)(381)(1000) = 952.5 mm²/m (0.45 in.²/ft)

For this example, the provided base panel shear reinforcement consists of 13M (no. 4) mild steel bars spaced 254 mm (10 in.) on center and located at each panel face in both the horizontal and vertical directions. The provided A_t equals 1016 mm²/m (0.48 in.²/ft). Section 21.9.6.4(e) of ACI 318-11 should be satisfied for the termination and development of the base panel horizontal distributed reinforcement within the confined boundary regions at the wall toes.

Base panel bottom edge reinforcement

Bottom edge reinforcement is designedin the base panel to limit the initiation of vertical cracking near the tip of the gap (that is, at the neutral axis) at the base joint. As required by section 4.4.10 of ACI ITG-5.2, the total mild steel area along the bottom edge of the base panel A_{edge}

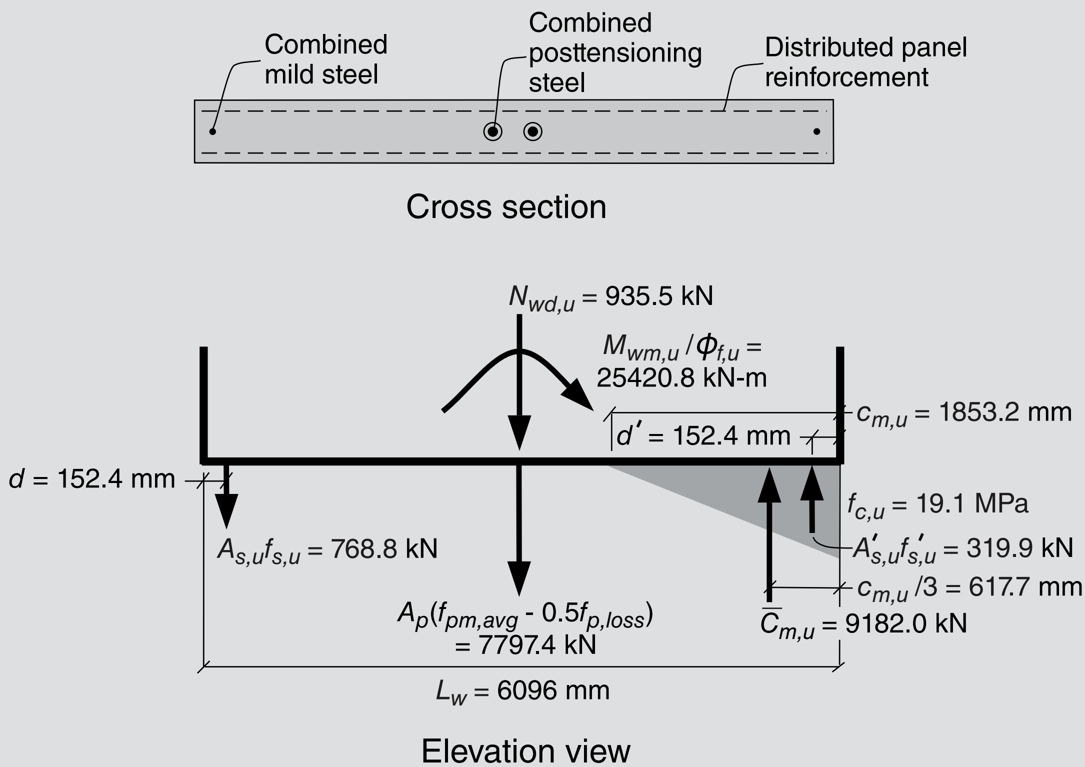

Figure A7. Free body diagram of first-floor upper panel–to–panel joint at Δ_{wm}. Note: A_p = total posttensioning steel area; $A_{s,u}$ = total tension-side mild steel area crossing upper joint at panel end; $A'_{s,u}$ = total compression-side mild steel area crossing upper joint at panel end; $c_{m,u}$ = neutral axis length; $\overline{C}_{m,u}$ = concrete compressive stress resultant; d = centroidal distance (from panel end) of tension-side mild steel bars crossing upper joint; d' = centroidal distance (from panel end) of compression-side mild steel bars crossing upper joint; $f_{c,u}$ = maximum concrete compression stress at upper joint; $f_{p,loss}$ = stress loss in compression-side posttensioning tendon at Δ_{wm} due to nonlinear material behavior associated with reversed-cyclic loading of wall to ± Δ_{wm}; $f_{pm,avg}$ = average stress in posttensioning steel; $f_{s,u}$ = stress in combined tension-side mild steel crossing upper joint; $f'_{s,u}$ = stress in combined compression-side mild steel crossing upper joint; L_w = wall length; M_{wm} = probable base moment strength of wall; $M_{wm,u}$ = upper joint moment corresponding to M_{wm}; $N_{wd,u}$ = factored axial force at upper joint; Δ_{wm} = maximum-level wall drift corresponding to maximum-considered earthquake. 1 mm = 0.0394 in.; 1 m = 3.28 ft; 1 kN = 0.225 kip; 1 MPa = 0.145 ksi.

should provide a nominal tensile strength T_{edge} of not less than 87.6 kN/m (6 kip/ft) along the length of the panel. Therefore:

$$T_{edge} = 87.6L_w = (87.6)(6.1) = 534 \text{ kN (120 kip)}$$

$$A_{edge} = \frac{T_{edge}}{f_{sy}} = \frac{\left(\frac{534}{100}\right)}{\left(\frac{413.7}{1000}\right)} = 1290 \text{ mm}^2 \text{ (2.0 in.}^2\text{)}$$

For this example, the edge reinforcement provided at the bottom of the base panel consists of two 29M (no. 9) mild steel bars with A_{edge} equal to 1290 mm² [2.0 in.²]).

Flexural design of upper panel–to–panel joints

The philosophy behind the flexural design of the upper panel–to–panel joints is to prevent significant gap opening and nonlinear behavior of the material during lateral displacements of the wall up to the maximum-level drift Δ_{wm}. Only the design of the first-floor panel–to–panel joint (Fig. A1) is demonstrated in this example. As previously discussed, the design axial force at each joint is calculated as the factored axial force for the design load combination being considered. For the first-floor panel–to–panel joint

$$N_{wd,u} = 935.5 \text{ kN (210.3 kip)}$$

The bending moment $M_{wm,u}$ is calculated from the probable base moment strength M_{wm} at Δ_{wm} as:

$$M_{wm,u} = \left(\frac{M_{wm}}{M_{wd}}\right) M_{wd,u} = \frac{(35{,}592.9)}{(24{,}422.3)}(15{,}698.4)$$

$$= 22{,}878.7 \text{ kN-m (16,874.5 kip-ft)}$$

To prevent significant gap opening at the upper panel–to–panel joints, mild steel reinforcement should be designed at the panel ends (where $A_{s,u}$ and $A'_{s,u}$ represent the total tension-side and compression-side mild steel areas crossing the upper panel–to–panel joint, respectively) and placed in a symmetrical layout (where d and d' represent the centroidal distances from panel end of the tension-side and compression-side mild steel bars crossing the upper panel–to–panel joint, respectively). For design purposes, a capacity reduction factor $\phi_{f,u}$ of 0.90 is used for the axial-flexural design of the upper panel–to–panel joints.

Using equilibrium, compatibility and kinematics, and design constitutive relationships at the upper panel–to–panel joints when Δ_{wm} is reached (**Fig. A7** for the first-floor joint), the following equations can be used to solve for the concrete compressive stress resultant $\bar{C}_{m,u}$, neutral axis length $c_{m,u}$, stresses in the combined tension-side and compression-side mild steel bars $f_{s,u}$ and $f'_{s,u}$, respectively, and maximum concrete compressive stress $f_{c,u}$:

$$\bar{C}_{m,u} = A_{s,u}f_{s,u} - A'_{s,u}f'_{s,u} + A_p(f_{pm,avg} - 0.5f_{p,loss}) + N_{wd,u}$$

$$\bar{C}_{m,u} = 0.5f_{c,u}t_w c_{m,u}$$

$$\frac{M_{wm,u}}{\phi_{f,u}} = \bar{C}_{m,u}\left(\frac{L_w}{2} - \frac{c_{m,u}}{3}\right) + A_{s,u}f_{s,u}\left(\frac{L_w}{2} - d\right) + A'_{s,u}f'_{s,u}\left(\frac{L_w}{2} - d'\right)$$

$$f_{s,u} = \left(\frac{E_s}{E_c}\right)f_{c,u}\left(\frac{L_w - c_{m,u} - d}{c_{m,u}}\right)$$

$$f'_{s,u} = \left(\frac{E_s}{E_c}\right)f_{c,u}\left(\frac{c_{m,u} - d'}{c_{m,u}}\right)$$

It is required that the tension-side steel strain be limited to the yield strain ε_{sy} to limit gap opening and the maximum concrete compressive stress be limited to $0.5f'_c$ to keep the concrete linear elastic. The designer would select an initial steel area and location. For this design example, the following steel area and location are used (Fig. A4):

$A_{s,u} = A'_{s,u}$ = 2038.7 mm² (3.16 in.²) (four 25M [no. 8] bars at a center-to-center spacing of 50.8 mm (2.0 in.) located at each end)

$d = d'$ = 152.4 mm (6.0 in.)

Then

$$\bar{C}_{m,u} = (2038.7)\left(\frac{f_{s,u}}{1000}\right) - (2038.7)\left(\frac{f'_{s,u}}{1000}\right) + (5038.7)\left[\frac{1569.9}{1000} - \frac{(0.5)(44.8)}{1000}\right] + 935.5$$

$$\bar{C}_{m,u} = (0.5)\left(\frac{f_{c,u}}{1000}\right)(381)c_{m,u}$$

$$\frac{(22,878.7)(1000)}{0.9} = \bar{C}_{m,u}\left(\frac{6096}{2} - \frac{c_{m,u}}{3}\right) + (2038.7)\left(\frac{f_{s,u}}{1000}\right)\left(\frac{6096}{2} - 152.4\right) + (2038.7)\left(\frac{f'_{s,u}}{1000}\right)\left(\frac{6096}{2} - 152.4\right)$$

$$f_{s,u} = \left(\frac{200,000}{30,438}\right)f_{c,u}\left(\frac{6096 - c_{m,u} - 152.4}{c_{m,u}}\right)$$

$$f'_{s,u} = \left(\frac{200,000}{30,438}\right)f_{c,u}\left(\frac{c_{m,u} - 152.4}{c_{m,u}}\right)$$

By solving this system of equations, the steel and concrete strain limitations are satisfied:

$\bar{C}_{m,u}$ = 9182.0 kN (2064.2 kip)

$c_{m,u}$ = 1853.2 mm (73.0 in.) = $0.31L_w$

$f_{c,u}$ = 19.0 MPa (2.77 ksi) < $0.5f'_c$ → OK

$f_{s,u}$ = 377.1 MPa (54.7 ksi) tension < f_{sy} → OK

$f'_{s,u}$ = 156.9 MPa (22.7 ksi) compression < f_{sy} → OK

Although not shown here, this design step would be repeated for each upper panel–to–panel joint in the wall. To prevent strain concentrations in the upper panel–to–panel joint steel, a short prescribed 152.4 mm (6.0 in.) length of the bars should be unbonded (wrapped in plastic) at each joint.

Shear design of upper panel–to–panel joints

The shear force at the first-floor joint can be calculated from the probable base shear force V_{wm} at Δ_{wm}:

$$V_{jm} = V_{wm,u} = \left(\frac{V_{wm}}{V_{wd}}\right)V_{wd,u} = \left(\frac{3476.0}{2385.1}\right)(2128.5)$$
$$= 3102.0 \text{ kN (697.4 kip)}$$

where

$V_{wm,u}$ = first-floor joint shear force corresponding to V_{wm}

At the upper panel–to–panel joints, a larger shear friction coefficient of μ_{ss} equal to 0.6 can be used, and the design equation can be written as

$\phi_s \mu_{ss}[(A_{s,u} + A'_{s,u})f_{sy} + A_p(f_{pm,avg} - 0.5f_{p,loss}) + N_{wd,u}] > V_{wm,u}$

Then, for the first-floor joint:

$$(0.75)(0.6)\left\{(2038.7+2038.7)\left(\frac{413.7}{1000}\right) + (5038.7)\left[\frac{1569.9}{1000} - \frac{(0.5)(44.8)}{1000}\right] + 935.5\right\}$$

> 3102.0 kN (697.4 kip)

4688.9 kN (1054.1 kip) > 3102.0 kN (697.4 kip) → OK

While not shown in this example, this design step would be repeated for each upper panel–to–panel joint in the wall.

Final wall reinforcement details

Figure A4 shows the reinforcement details of the wall based on the above design calculations. Final design checks should be conducted (not shown here) considering the strains and stresses in the individual posttensioning tendons and energy-dissipating steel bars.

References

1. Smith, B., and Y. Kurama. 2012. *Seismic Design Guidelines for Special Hybrid Precast Concrete Shear Walls*. Research report NDSE-2012-02. Notre Dame, IN: University of Notre Dame. hybridwalls.nd.edu/Project/Downloads/Design_Guidelines.pdf.

2. ASCE (American Society of Civil Engineers) and SEI (Structural Engineering Institute). 2010. *Minimum Design Loads for Buildings and Other Structures*. ASCE/SEI 7-10. Reston, VA: ASCE and SEI.

3. ACI (American Concrete Institute) Committee 318. 2011. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11).* Farmington Hills, MI: ACI.

4. ACI Innovation Task Group 5. 2009. *Requirements for Design of a Special Unbonded Post-Tensioned Precast Shear Wall Satisfying ACI ITG-5.1 (ACI ITG-5.2) and Commentary.* Farmington Hills, MI: ACI.

Notation

a_d = length of ACI 318-11 equivalent uniform concrete compression stress block at Δ_{wd}

a_{hoop} = confinement hoop steel bar area

a_m = length of ACI ITG-5.2 equivalent uniform concrete compression stress block at Δ_m

A_{edge} = total mild steel area along bottom edge of base panel

A_p = total posttensioning steel area

A_s = total energy-dissipating steel area

A_{sh} = effective shear area of wall cross section

$A_{s,u}$ = total tension-side mild steel area crossing upper panel–to–panel joint at panel end

$A'_{s,u}$ = total compression-side mild steel area crossing upper panel–to–panel joint at panel end

A_t = shear reinforcement area within panel

c_d = neutral axis length at base joint at Δ_{wd}

c_m = neutral axis length at base joint at Δ_{wm}

$c_{m,u}$ = neutral axis length at upper panel–to–panel joint at Δ_{wm}

C_d = ASCE 7-10 deflection amplification factor

$\bar{C}_d$ = concrete compressive stress resultant at base joint at Δ_{wd}

$\bar{C}_m$ = concrete compressive stress resultant at base joint at Δ_{wm}

$\bar{C}_{m,u}$ = concrete compressive stress resultant at upper panel–to–panel joint at Δ_{wm}

d = centroidal distance (from panel end) of tension-side mild steel bars crossing upper panel–to–panel joint

d' = centroidal distance (from panel end) of compression-side mild steel bars crossing upper panel–to–panel joint

d_s = diameter of energy-dissipating bar

e_p = centroidal distance of tension-side and compression-side posttensioning tendons from wall centerline

e_s = centroidal distance of tension-side and compression-side energy-dissipating bars from wall centerline

e_{se} = distance of extreme energy-dissipating bar from wall centerline

E_c = modulus of elasticity of concrete

E_p = modulus of elasticity of posttensioning steel

E_s = modulus of elasticity of energy-dissipating or other mild steel

f'_c = compressive strength of unconfined panel concrete

f'_{cc} = compressive strength of confined concrete at toes of base panel

$f_{c,u}$ = maximum concrete compression stress at upper panel–to–panel joint at Δ_{wm}

f'_l = effective lateral confining stress provided by confinement reinforcement at toes of base panel

$f_{p,loss}$ = stress loss in compression-side posttensioning tendon at $+\Delta_{wm}$ due to nonlinear material behavior associated with reversed-cyclic loading of wall to $\pm\Delta_{wm}$

f_{pd} = posttensioning steel stress at Δ_{wd}

$f_{pd,avg}$ = average stress for total posttensioning steel area crossing base joint at Δ_{wd}

f_{pi} = initial stress of posttensioning steel after all short-term and long-term losses but before any lateral displacement of wall

f_{pm} = posttensioning steel stress at Δ_{wm}

$f_{pm,avg}$ = average stress for total posttensioning steel area crossing base joint at Δ_{wm}

$f_{pm,c}$ = stress in compression-side posttensioning tendon at initial loading to $+\Delta_{wm}$

$f_{pm,cr}$ = stress in compression-side posttensioning tendon after repeated loading to $+\Delta_{wm}$

$f_{pm,t}$ = stress in tension-side posttensioning tendon at initial and repeated loading to $+\Delta_{wm}$

f_{pu} = ultimate strength of posttensioning steel

f_{py} = yield strength of posttensioning steel determined at limit of proportionality point on strand stress-strain relationship

$f_{sd,avg}$ = average stress for total energy-dissipating steel area crossing base joint at Δ_{wd}

f_{sm} = energy-dissipating steel stress at Δ_{wm}

$f_{sm,avg}$ = average stress for total energy-dissipating steel area crossing base joint at Δ_{wm}

f_{su} = ultimate (maximum) strength of energy-dissipating or other mild steel

$f_{s,u}$ = stress in combined tension-side mild steel crossing upper panel–to–panel joint at Δ_{wm}

$f'_{s,u}$ = stress in combined compression-side mild steel crossing upper panel–to–panel joint at Δ_{wm}

f_{sy} = yield strength of energy-dissipating or other mild steel

g = acceleration due to gravity

G_c = shear modulus of concrete

h_p = plastic hinge height over which plastic curvature is assumed to be uniformly distributed at wall base

H_w = wall height from top of foundation

I_e = reduced linear-elastic effective moment of inertia of wall cross section

I_{gross} = moment of inertia of gross wall cross section

l_h = confined region length at wall toes (center-to-center of bars)

l_{hoop} = length of individual confinement hoop (center-to-center of bars)

l_{pu} = unbonded length of posttensioning steel

l_{sw} = wrapped length of energy-dissipating steel

L_w = wall length

M_{wd} = wall design base moment at Δ_{wd}

$M_{wd,u}$ = upper panel-to-panel joint moment corresponding to M_{wd} at Δ_{wd}

M_{wm} = probable base moment strength of wall at Δ_{wm}

$M_{wm,u}$ = upper panel–to–panel joint moment corresponding to M_{wm} at Δ_{wm}

N_{wd} = factored axial force at wall base for design load combination being considered

$N_{wd,u}$ = factored axial force at upper panel–to–panel joint for design load combination being considered

R = ASCE 7-10 response modification factor

s_{bot} = first confinement hoop distance from bottom of base panel (to center of bar)

s_{hoop} = confinement hoop spacing (center-to-center of bars)

S_{D1} = ASCE 7-10 5% damped design spectral response acceleration parameter at a fundamental period equal to 1.0 second

S_{DS} = ASCE 7-10 5% damped design spectral response acceleration parameter at short periods

t_w = wall thickness

T_1 = fundamental period determined from modal analysis

T_{edge} = total tensile force for design of base panel bottom edge reinforcement

V_{jm} = maximum shear force at horizontal joint at Δ_{wm}

V_n = nominal shear (diagonal tension) strength of base panel

V_{ss} = shear friction strength at horizontal joint

V_{wd} = wall design base shear force corresponding to M_{wd} at Δ_{wd}

$V_{wd,u}$ = upper panel-to-panel joint shear force corresponding to V_{wd} at Δ_{wd}

V_{wm} = maximum wall base shear force corresponding to M_{wm} at Δ_{wm}

$V_{wm,u}$ = upper panel–to–panel joint shear force corresponding to V_{wm} at Δ_{wm}

w_h = width of confined concrete region at wall toes

w_{hoop} = width of individual confinement hoop (center-to-center of bars)

W_1 = seismic weight of building at first floor

W_2 = seismic weight of building at second floor

W_3 = seismic weight of building at third floor

W_4 = seismic weight of building at roof (fourth floor)

W_i = seismic weight of building at i^{th} floor/roof level.

α_c = ACI 318-11 coefficient defining relative contribution of concrete strength to nominal wall shear strength

α_s = ACI ITG-5.2 coefficient to estimate additional energy-dissipating bar debonding that is expected to occur during reversed-cyclic lateral displacements of wall to $\pm\Delta_{wm}$

β_1 = ACI 318-11 factor relating equivalent uniform concrete compression stress block length a_d to neutral axis length c_d at Δ_{wd}

β_m = ACI ITG-5.2 factor relating equivalent uniform concrete compression stress block length a_m to neutral axis length c_m at Δ_{wm}

γ_m = ACI ITG-5.2 factor relating equivalent uniform confined concrete compression block stress at toe of base panel to confined concrete strength f'_{cc}

δ_{pd} = elongation of posttensioning steel at Δ_{wd}

δ_{pm} = elongation of posttensioning steel at Δ_{wm}

δ_{sd} = elongation of energy-dissipating steel at Δ_{wd}

δ_{sm} = elongation of energy-dissipating steel at Δ_{wm}

$\delta_{w,flex}$ = wall displacement due to flexural deformations in linear-elastic effective stiffness model

$\delta_{w,sh}$ = wall displacement due to shear deformations in linear-elastic effective stiffness model

Δ_{wd} = design-level wall drift corresponding to design-basis earthquake

Δ_{we} = linear-elastic wall drift calculated using linear-elastic effective stiffness model

Δ_{wm} = maximum-level wall drift corresponding to maximum-considered earthquake

ε_{cm} = maximum concrete compression strain at base joint at Δ_{wm}

ε_{pm} = strain in posttensioning steel at Δ_{wm}

$\varepsilon_{pm,c}$ = strain in compression-side posttensioning tendon at initial and repeated loading to $+\Delta_{wm}$

$\varepsilon_{pm,t}$ = strain in tension-side posttensioning tendon at initial and repeated loading to $+\Delta_{wm}$

$\varepsilon_{py,}$ = yield strain of posttensioning steel, defined at limit of proportionality point on strand stress-strain relationship

ε_{sa} = allowable strain in energy-dissipating steel at Δ_{wm}

ε_{sd} = strain in energy-dissipating steel at Δ_{wd}

ε_{sm} = maximum strain in energy-dissipating steel at Δ_{wm}

ε_{su} = strain of energy-dissipating or other mild steel at f_{su}

ε_{sy} = yield strain of energy-dissipating or other mild steel at f_{sy}

κ_d = energy-dissipating steel moment ratio, defining relative amounts of energy-dissipating resistance from $A_s f_{sd}$ and restoring resistance from $A_p f_{pd}$ and N_{wd} at wall base

λ = ACI 318-11 factor reflecting reduced mechanical properties of lightweight-aggregate concrete relative to normalweight aggregate concrete

μ_{ss} = coefficient of shear friction at horizontal joints

ν_c = Poisson's ratio for concrete

ρ_h = confinement steel reinforcement ratio at wall toes

ρ_t = shear steel reinforcement ratio within wall panel

ϕ_f = capacity reduction factor for axial-flexural design of base joint

ϕ = capacity reduction factor for axial-flexural design of upper panel–to–panel joints

ϕ_r = capacity reduction factor against loss of restoring

ϕ_s = capacity reduction factor for shear design

ϕ_{wm} = plastic curvature at wall base at Δ_{wm}