State of the Art of Precast/Prestressed Concrete Sandwich Wall Panels

Second Edition

Prepared by

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NOTE

This issue of the *PCI Journal* contains only the appendix of this committee report. The body of the report was published in the Spring 2011 issue.

APPENDIX – DESIGN EXAMPLES

EXAMPLE A1. NONCOMPOSITE CLADDING PANEL

Design Criteria (see Fig. A.1)

Wind load:

Direct pressure = 10 lb/ft^2

Suction pressure = 15 lb/ft^2

a = 8 ft = 96 in. (panel width)

b_p = 1.5 ft (height of parapet)

f'_c = 5000 psi

f'_{ci} = 3500 psi

f_{pu} = 270 ksi

l = 23 ft = 276 in. (clear span)

t_s = 4 in. (inner wythe thickness)

t_{ns} = 2 in. (outer wythe thickness)

$$w_{dead} = \frac{(4+2)}{12}(150) = 75 \text{ lb/ft}^2 = 0.075 \text{ kip/ft}^2$$

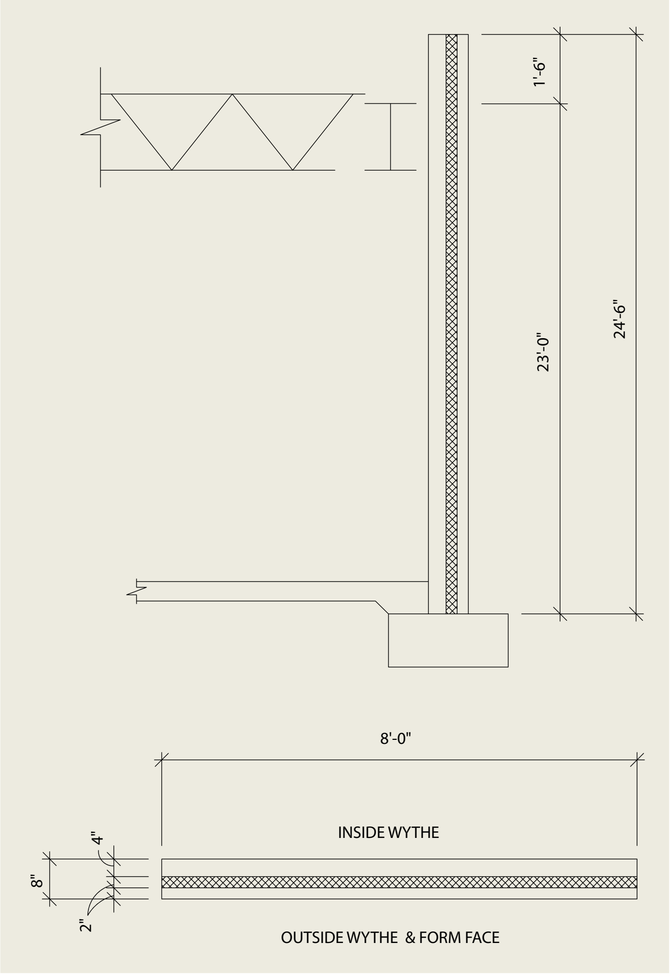

Figure A.1. Example A1.

Noncomposite behavior.

Note: The inner wythe t_s is 4 in. thick and is used as the structural wythe. It is assumed to resist 100% of the service and handling loads.

Inner Wythe Section Properties

$A = t_s a = 4(96) = 384$ in.^{2}

$I = a(t_s^3)/12 = 96\,(4^3)/12 = 512$ in.^{4}

$S = a(t_s^2)/6 = 96\,(4^2)/6 = 256$ in.^{3}

Structural Analysis

Analyze the section for handling and service load conditions before determining the prestress required.

$U = 1.2D + 1.6W + 1.0L + 0.5\,(L_r \text{ or } S \text{ or } R)$

Suction pressure governs because it is larger than direct pressure.

Note: In calculating wind moment, reduction caused by 1 ft 6 in. cantilevered parapet is neglected. (A separate design check of the parapet should be done.)

w_{wind} = (15 lb/ft^2)(8 ft) = 120 lb/ft = 0.12 kip/ft = 0.01 kip/in.

$$M_{service} = \frac{w_{wind}l^2}{8} = \frac{(120)(23)^2}{8}$$

$$= 7935 \text{ lb-ft} = 7.94 \text{ kip-ft}$$

$$M_{u,wind} = (7.94)(1.6)$$

$$= 12.7 \text{ kip-ft}$$

In calculating the deflection due to wind, assume $\phi_k = 0.85$ and $\beta_d = 0$.

(Stiffness-reduction factor is assumed to be at least 0.85 considering the stringent dimensional accuracy provided by a precasting plant.)

$$\Delta = \frac{5}{384}\left(\frac{w_{u,wind}l^4}{EI}\right)$$

$$E_c = 57\sqrt{f'_c} = 57\sqrt{5000}$$

$$= 4030 \text{ ksi}$$

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(512)}{1+0}$$

$$= 1.75 \times 10^6 \text{ kip-in.}^2$$

$$\Delta = \frac{5}{384}\left[\frac{1.6(0.01)(276)^4}{1{,}750{,}000}\right]$$

$$= 0.69 \text{ in.}$$

To continue the analysis, a load-deflection (P-Δ) analysis is carried out to demonstrate the P-Δ effects due to the relatively high flexibility of the noncomposite panel.

$P_D = (a)(w_{dead})[(l/2) + b_p]$

$P_D = (8)(0.075)[(23/2) + 1.5] = 7.8$ kip

$P_u = 1.2\ P_D = 1.2\ (7.8)$

$= 9.36$ kip at mid-height

$\beta_d = 1.0$

$$EI = \frac{\phi_k E_c I}{1 + \beta_d} = \frac{0.85(4030)(512)}{1 + 1.0}$$

$= 8.77 \times 10^5$ kip-in.^{2}

Assume that the initial bow of the panel is ±0.77 in. (l/360) due to differential shrinkage during storage and is additive to the deflection due to the wind.

Because this panel is noncomposite, thermal bowing will be minimal. The deflection of the panel at midspan is the sum of the initial bow and the wind deflection.

$\varDelta_{midspan} = 0.77 + 0.69$

$= 1.46$ in.

Secondary deflection from self-weight:

$\varDelta = P_u e l^2/8EI$
$= (9.36)\ e\ (23 \times 12)^2/[(8)(\ 8.77 \text{ x } 10^5)]$
$= 0.102e$

First iteration:
$\varDelta = 0.102\ (1.46) = 0.15$ in.

Second iteration:
$e = 1.46 + 0.15 = 1.61$ in.

$\varDelta = 0.102\ (1.61) = 0.16$ in.

Third iteration:
$e = 1.46 + 0.16 = 1.62$ in.

$\varDelta = 0.102\ (1.62) = 0.16$ in.
resulting in convergence.

The ultimate moment due to wind and P-$\varDelta$ effects is:

$M_u = M_{u,wind} + M_{u,P\text{-}\Delta}$
$= (12.7$ kip-ft$) + (9.36$ kip$)(1.62$ in.$)/12$ in./ft
$= 14$ kip-ft

Note that the P-$\varDelta$ calculations were based on the uncracked moment of inertia, so M_u should be compared with M_{cr}. Handling stresses often dictate the required amount of prestress, so this will be checked later.

Table A.1 Static load multiplierss

Condition	Multiplier on Static Load
Stripping	1.3
Shipping	1.5
Yard Handling and Erection	1.2

Stripping and Yard Handling (see Fig. A.2)

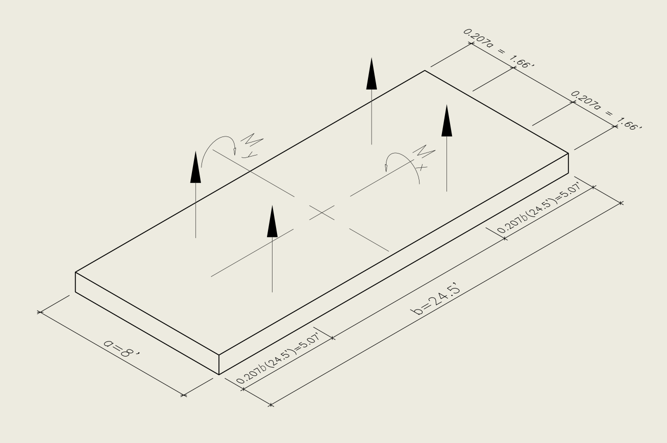

Figure A.2. Stripping diagram.

$f'_{ci} = 3500$ psi

Stripping equivalent static multiplier = 1.3 (Table A.1)

Yard handling equivalent static multiplier = 1.2 (Table A.1)

Therefore, stripping governs.

$w = 1.3\ (0.075)$
$= 0.098$ kip/ft^2

From *PCI Design Handbook*, Fig. 8.3.2:

$+M_x = -M_x = (0.0107)wa^2b$
$= (0.0107)(0.098)(8)^2(24.5)$
$= 1.6$ kip-ft

M_x is resisted by an effective width of

$15t_s = (15)(4) = 60$ in. (governs)

or $b/2 = (24.5)(12)/2 = 147$ in. (does not govern)

$S_x = 4^2(60)/6$
$= 160$ in.^{3}

$M_x/S_x = (1.6)(12000)/160$
$= 120$ psi

which is less than $5\sqrt{3500} = 296$ psi (OK)(see *PCI Design Handbook*, 7th ed., Eq. 8.3.4.2)

$+M_y = -M_y = (0.0107)\ wab^2$
$= (0.0107)(0.098)(8)(24.5^2)$
$= 5.04$ kip-ft

M_y is resisted by an effective width of:

$a/2 = (8)(12)/2 = 48$ in.

Stripping and yard handling moment required for total panel width:

$M_{u,strip} = (2)(5.04)$
$= 10.1$ kip-ft

Check stresses after prestress level is determined.

Shipping

Because the panel will be supported at the same locations as at stripping, shipping stresses will not govern. The ratio of the shipping equivalent static multiplier of 1.5 (Table A.1) to the stripping multiplier of 1.3 is 1.5/1.3 = 1.15. The ratio of the square root of shipping concrete strength of 5000 psi to stripping concrete strength of 3500 psi is $\sqrt{5000}/\sqrt{3500}$ = 1.19. This means that the concrete strength increase is greater than the higher shipping stresses.

Erection

Try a two-point pick (see Fig. A.3) with equal +*M* and -*M*: $M = 0.044wb^2$.

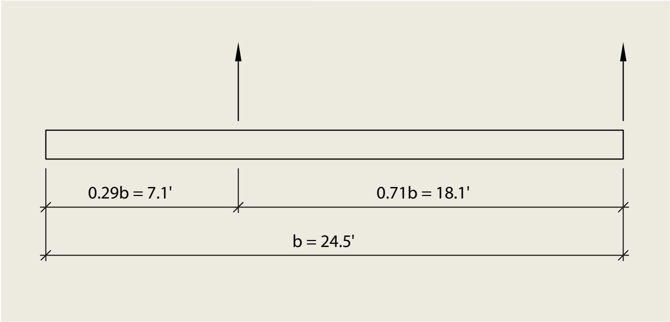

Figure A.3. Two-point pick.

Equivalent static multiplier = 1.2 for erection. (Table A.1)

w = 1.2 (0.075)(8)
= 0.72 kip/ft

M = $(0.044)wb^2$ (see *PCI Design Handbook*, 7th ed., Fig. 8.6.1)

M = (0.044)(0.72)(24.5^2)
= 19.0 kip-ft

f_{bv} =(19.0)(12,000)/256
= 890 psi

$f_{pc,req.} = 890 - 5\sqrt{5000}$
= 537 psi

This prestress value is too high; therefore, revise the handling scheme. Try a three-point pick (see Fig. A.4). Note that the rigging shown is marginally stable during rotation and the sling length required to achieve the minimum sling angle is critical.

$+M_y$ = 0.034 wb^2

$-M_y$ = 0.0054 wb^2 (does not govern)

$+M_y$ = 14.7 kip-ft
governs over

$M_{u,wind}$ = 12.7 kip-ft

$M_u = M_{u,wind} + M_{u,P\text{-}\Delta}$ = 14 kip-ft and $M_{y\ handling}$ = 10.1 kip-ft

$f_{pc,req.} = M/S - 5\sqrt{f_c'}$
= (14.7)(12,000)/256 – $5\sqrt{5000}$
= 689 – 354 = 335 psi

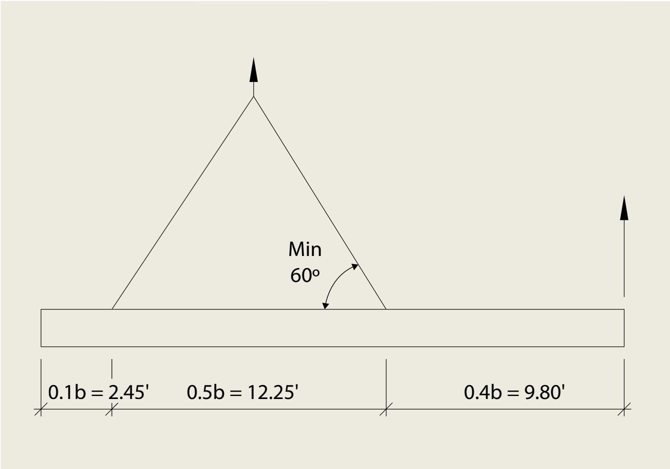

Figure A.4. Three-point pick.

Determination of Prestress Levels

Assume 15% prestress losses. For a 4 in. wythe, use ½-in.-diameter, low-relaxation, 270 ksi strands.

A_{ps} = 0.153 in.^{2} per strand. Determine number of strands required:

Number of strands required:

$$= \frac{f_{pc}A_{wythe}}{\left(0.7A_{ps}f_{pu}\right)\left(1-loss\right)}$$

$$= \frac{(0.335)(384)}{\left[0.7(0.153)(270)\right](1-0.15)}$$

= 5.3 (Use six ½ in. strands)

For a 2 in. wythe, use ⅜-in.-diameter, low-relaxation, 270 ksi strands.

A_{ps} = 0.085 in.^{2}

To satisfy the minimum prestress requirements of ACI 318-05, section 18.11.2.3,

$f_{pc,min.}$ = 225 psi

Number of strands required

$$= \frac{f_{pc,min}A}{\left(0.7A_{ps}f_{pu}\right)\left(1-loss\right)}$$

$$= \frac{(0.225)(2)(96)}{\left[0.7(0.085)(270)\right](1-0.15)}$$

= 3.2

(Use four ⅜ in. strands)

Evaluate ϕM in structural wythe

The prestressing steel stress f_{ps} = 264 ksi is determined from methods given in the *PCI Design Handbook*.

$$a = \frac{A_{ps}f_{ps}}{0.85f'_c b}$$

$$= \frac{(6 \times 0.153)264}{0.85(5)(96)} = 0.59 \text{ in.}$$

$$\phi M_n = \phi A_{ps}f_{ps}(d - a/2)$$
$$= 0.9\ (6\times0.153)(264)(2\text{–}0.59/2)/12 = 31.0 \text{ kip-ft}$$

This moment is greater than M_u = 14 kip-ft (wind + P-Δ)

Therefore:

$$M_{cr} = (f_{pc} + f_r)S$$
$$= (0.384 + 0.53)(256)/12$$
$$= 19.5 \text{ kip-ft}$$

$\phi M_n/M_{cr}$ = 31.0/19.5 = 1.6 > 1.2

Therefore, the section is satisfactory and basing the P-Δ calculations on uncracked section properties is valid.

EXAMPLE A2. NONCOMPOSITE LOAD-BEARING PANEL

Design Criteria (see Fig. A.5)

Wind load:

Direct pressure = 17 lb/ft^2
Suction pressure = 24 lb/ft^2

Panel dead load D_p = 30 kip
Roof dead load D_r = 20 kip
Roof live load L_r = 6.2 kip

f'_c = 5000 psi

f'_{ci} = 3500 psi

f_{pu} = 270 ksi

t_s = 6 in.

a = 10 ft = 120 in.

l = 28.5 ft = 342 in.

As panel is noncomposite, the thermal bowing effect is negligible and is not considered in this design.

The inner 6-in.-thick wythe is considered to be the structural wythe and will be designed to resist 100% of all loadings.

Allowable flexural tension stress at ultimate = $7.5\sqrt{f'_c}$

Inner Wythe Section Properties

$A = t_s a = 6(120) = 720$ in.^{2}
$I = a(t_s^3)/12 = 120(6^3)/12 = 2160$ in.^{4}

$S = a(t_s^2)/6 = 120(6^2)/6 = 720$ in.^{3}

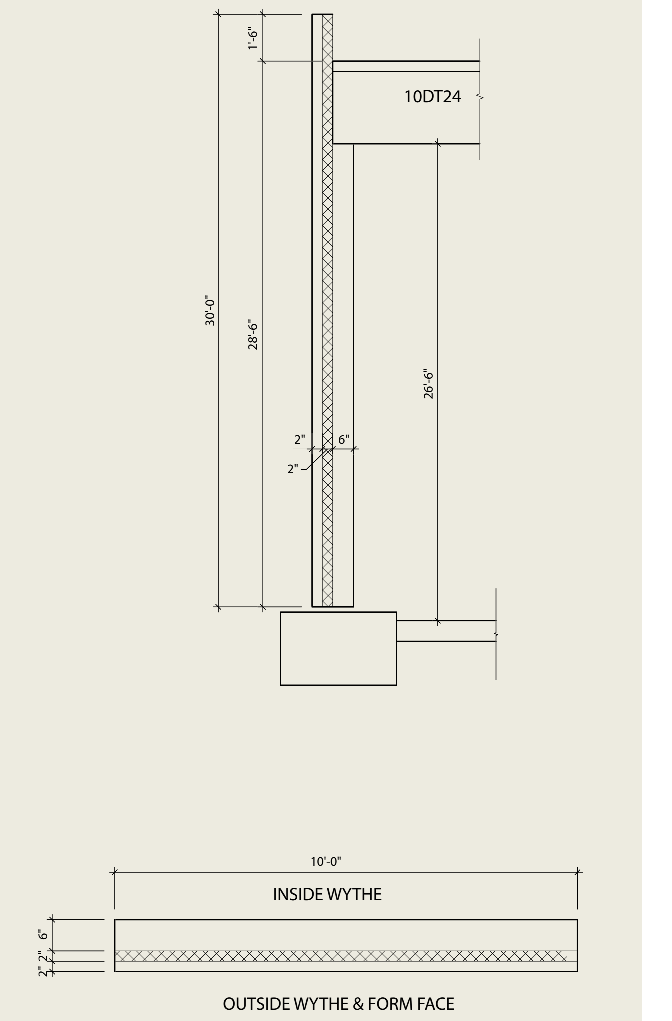

Figure A.5. Example A2.

Selection of Prestressing

Assume f_{pc} minimum = 225 psi per ACI 318-05 Section 18.11.2.3 so that minimum requirements of ACI 318-05 Section 14.3 may be waived.

6 in. structural wythe, try seven 1/2-in.-diameter strands at 70% initial stress located at the center of gravity of the wythe.

A_{ps} = 0.153 in.^{2} / strand
Assume 15% prestress losses.

$$f_{pc} = \frac{A_{ps}f_{se}}{A}$$

$$f_{pc} = \frac{7(0.153)(0.7)(270)(1 - 0.15)}{720}$$

$$f_{pc} = 0.239 \text{ ksi} > 0.225 \text{ ksi}$$

Satisfies ACI 318-05 section 18.11.2.3 and minimum reinforcement requirements of ACI 318-05 section 14.3 may be waived.

For 2 in. wythe, try four 3/8-in.-diameter strands located at the center of gravity of the wythe.

A_{ps} = 0.085 in.^{2} / strand

Assume 15% prestress losses.

$$f_{pc} = \frac{A_{ps} f_{se}}{A}$$

$$= \frac{4(0.085)(0.7)(270)(1-0.15)}{(2)(120)}$$

f_{pc} = 0.228 ksi > 0.225 ksi

Satisfies ACI 318-05 section 18.11.2.3 and minimum reinforcement requirements of ACI 318-05 section 14.3 may be waived.

It could be argued that the face wythe of a noncomposite panel is a nonstructural veneer, so ACI 318-05 requirements would not apply. Therefore, the prestress level in the face wythe would be governed only by crack control requirements (under wind and handling effects).

Stripping, Handling, Shipping, and Erection

Noncomposite properties of the panel are used to resist forces generated by stripping, handling, shipping, and erection procedures. Analysis in accordance with the *PCI Design Handbook* results in the following:

Stripping – Use two-point pick at about the 1/5 points from ends and sides of panel (Example A1 and Fig. A.3).

Erection – Use three-point pick with lower points at 4 ft and 14 ft from the lower end and inserts in the ledge area near the top of the panel (Example A1 and Fig. A.4).

Analysis:

All of the load cases of ACI 318-05 section 9.2 must be considered. The controlling load case for this example, Eq. (9-4), was found to govern.

$U = 1.2D+1.6W+1.0L+0.5(L_r \text{ or } S \text{ or } R)$

$W_{suction} > W_{pressure}$

$w_{suction}$ = (10)(0.024)

= 0.240 kip/ft = 0.02 kip/in.

Determine axial load:

P_{u1} = 1.2[(30/2)+20]+0.5[6.2] = 45.1 kip
Includes half of the panel self-weight.

P_{u2} = 1.2[20]+0.5[6.2] = 27.1 kip
Externally applied loads only.

Calculate stiffness for deflection computation:

ϕ_k = 0.85

Stiffness-reduction factor is assumed to be at least 0.85 considering the stringent dimensional accuracy provided by a precasting plant.

Ultimate dead load at midheight = (30 kip /2 + 20 kip)1.2 = 42 kip

Total load at midheight = 42 kip + (6.2 kip)0.5 = 45.1 kip

β_d = 42/45.1 = 0.93

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(2160)}{1+0.93}$$

$= 3.83 \times 10^6$ ksi

Assume a minimum eccentricity of 1 in. for construction tolerances.

Deflection at midheight of panel due to externally applied loads at e = 1 in.:

$$\Delta = \frac{Pel^2}{16EI}$$

$$\Delta = \frac{(27.1)(1)(342)^2}{16(3.83\times 10^6)} = 0.05 \text{ in.}$$

Note that the eccentricity is applied in the direction that achieves maximum total moment and deflection. In this case, the wind suction load governs and the eccentricity should be applied toward the inside face of the panel.

Based on the *PCI Design Handbook* maximum bowing tolerance (section 13.2.8), assume an initial panel bow of l/360 = 0.95 in.

Wind deflection (suction)

Use β_d = 0, ϕ_k = 0.85

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(2160)}{1+0}$$

= 7.40 x 10^6 ksi

$$\Delta = \frac{5}{384}\left(\frac{w_u l^4}{EI}\right)$$

$$= \frac{5}{384}\left(\frac{1.6(0.02)(342)^4}{7{,}400{,}000}\right)$$

= 0.77 in.

Δ_{total} = 0.05 + 0.95 + 0.77
= 1.77 in.

A P-Δ analysis is performed to account for secondary moments.

P-Δ analysis at mid-height of wall

$$\Delta = \frac{Pel^2}{8EI}$$

$$= \frac{(45.1)e(342)^2}{8(3.83 \times 10^6)}$$

$= 0.17e$

First iteration:
Δ = 0.17 (1.77) = 0.30 in.

Second iteration:
e = 1.77 + 0.30 = 2.07 in.
Δ = 0.17 (2.07) = 0.35 in.

Third iteration:
e = 1.77 + 0.35 = 2.12 in.
Δ = 0.17(2.12) = 0.36

Fourth iteration:
e = 1.77 + 0.36 = 2.13 in.
Δ = 0.17(2.13) = 0.36 (converges)

M_u = total factored moment

$M_u = 0.5P_{u,applied}\,(e_{applied}) + M_{u,wind} + P_{u,total}\,(e_{p\text{-}\Delta})$

$M_u = 0.5(27.1)(1) + 1.6(0.24)(28.5)^2(1.5) + (45.1)(2.13)$
= 577 kip-in.

Verify that the panel remains uncracked and that gross section properties apply.

Determine cracking moment

$$M_{cr} = \left(\frac{P_{u,total}}{A} + f_{pc} + f_r\right)S$$

$$= \left(\frac{45.1}{720} + 0.239 + 0.53\right)(720)$$

= 599 kip-in.

$M_{cr} > M_u$ Therefore, analysis is valid.

Check flexural strength of structural wythe

f_{ps} = 255 ksi ACI 318-05 Eq. (18-3)

$$a = \frac{(nA_{ps}f_{ps}) + P_{u,total}}{0.85f'_c b}$$

$$= \frac{[(7)(0.153)(255) + 45.1]}{0.85(5)(120)}$$

= 0.62 in.

$c = a/\beta_1$
= 0.62/0.8
= 0.77 in.

$\varepsilon_t = [(\varepsilon_c)(d_p - c)] / (c)$

$\varepsilon_t = [(0.003)(3–0.77)]/(0.77)$
= 0.009

Since $\varepsilon_t > 0.005$, section is tension controlled and $\phi = 0.9$.

$$\phi M_n = \phi\left[(nA_{ps}f_{ps}) + P_{u,total}\right]\left(d - \frac{a}{2}\right)$$

$$= 0.9\left[\left((7)(0.153)(255)\right) + 45.1\right]\left(3 - \frac{0.62}{2}\right)$$

= 770 kip-in. > 577 kip-in.

$\phi M_n > M_u$ (OK)

$\phi M_n/M_{cr}$ = 770/599 = 1.29 > 1.2 (OK)

The upper row of sandwich panel connectors will be about 2 ft below the top of the parapet. The controlling moment in the 2-in.-thick parapet will be governed by handling forces. Unit weight of the 2 in. wythe = 0.025 psf

$W = 1.3(0.025) = 0.033$ kip/ft^2 where 1.3 is the PCI load factor for handling

$M_y = wal^2/2$. For l = 2 ft, and a = 1 ft (a unit strip of the face wythe),

$M_y = 0.033(2)^2/2$
= 0.066 kip-ft/ft width

The transfer length for 3/8 in. strand is about 19 in., so the cracking moment for the parapet can include f_{pc}:

$S = 12(2)^2/6 = 8$ in.^{3}/ft

$M_{cr} = [f_{pc} + f_r]S$
= [0.228+0.530](8)/12
= 0.51 kip-ft/ft width (OK)

The sandwich panel connectors should be checked to ensure that the parapet is adequately anchored. The connector reaction can be calculated by making the simplifying assumption that the parapet is a cantilever with a short backspan.

EXAMPLE A3. NONCOMPOSITE SHEAR WALL PANEL

Design criteria

Same panel as in Example A2. Each panel is to be subjected to a seismic force Q_E of 16.1 kip. The seismic design category of the structure is B, with S_{DS} = 0.25.

Individual panel resists shear (see Fig. A.6)

From ASCE 7-05, section 12.4.2.3:

$U = (0.9\text{-}0.2S_{DS})D + \rho Q_E$ where: ρ = 1.0 for $SDC\ B$

$U = 0.85D + 1.0Q_E$

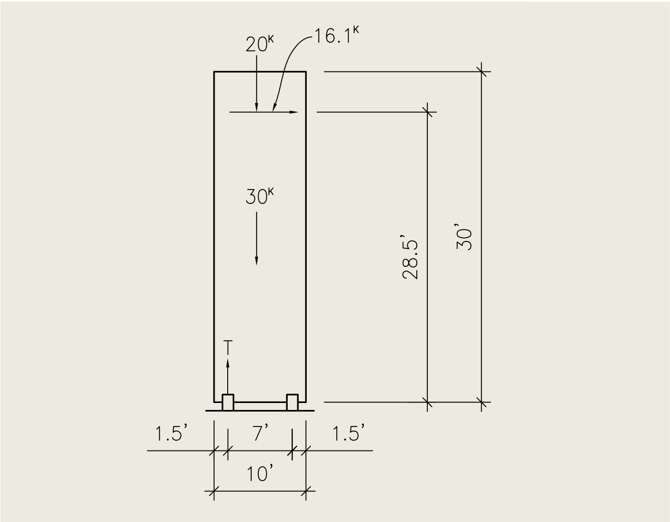

Figure A.6. Individual shear wall panel.

Total panel weight $= (100 \text{ lb/ft}^2 / 1000 \text{ lb/k})(10)(30)$
$= 30$ kip

Panel base assumed to be fully grouted, sum moments about toe:

Total uplift (single panel) $= [(16.1)(1.0)(28.5 \text{ ft}) - (30 + 20)(0.85)(10/2)]/8.5$ ft
$= 29.0$ kip uplift

Try connecting panels together to eliminate uplift.

Panels connected to resist shear (see Fig. A.7)

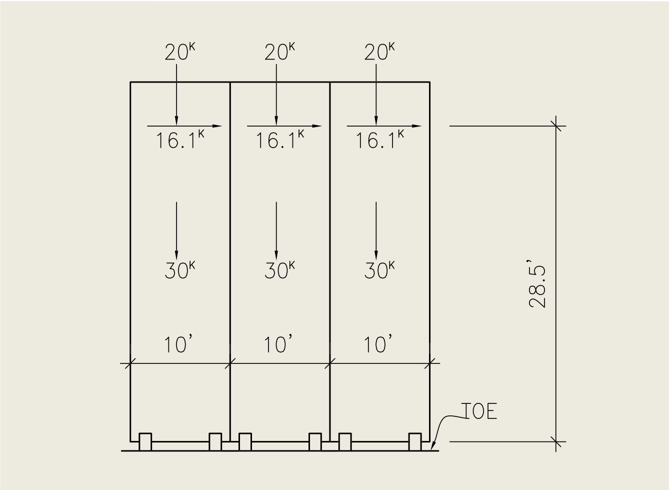

Figure A.7. Connected shear wall panels.

Try connecting three panels:

Summation of moments at toe:

Due to dead load $(30 + 20)(5 + 15 + 25) = 2250$ kip-ft

Due to seismic load $(16.1)(3)(28.5) = 1376$ kip-ft

Strength design: $U = 0.85D + 1.0Q_E$

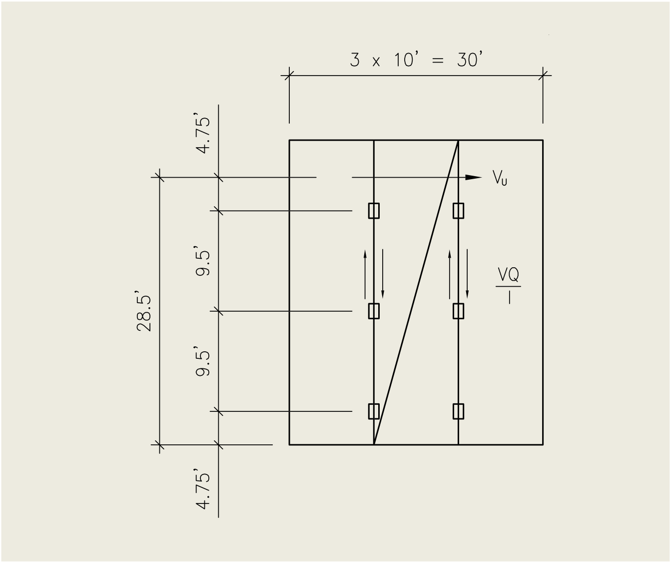

Figure A.8. Panel connections.

T_u (3 panels) $= 0.85(2250) - 1.0(1376)$
$= 536.5$ kip-ft
(+) No uplift present

Panel-to-panel connections (see Fig. A.8)

$V_u = 0.85D + 1.0Q_E$ (No D in direction of Q_E, therefore $V_u = 1.0Q_E$)

$V_u = (16.1)(3)(1.0)$
$= 48.3$ kip

Shear flow equation: $v = VQ/I$, $Q = A\bar{y}$

$Q = (10)(10)$ (per unit width)
$= 100 \text{ ft}^2$

$I = h^3/12$
$I = (30)^3/12$
$= 2250 \text{ ft}^3$

$V_uQ/I = (48.3)(100)/2250$
$= 2.15$ kip/ft

V_u (joint) $= (2.15 \text{ kip/ft})(28.5 \text{ ft}) = 61.3$ kip

Check dead load demand (determine portion of dead load that must be mobilized to resist overturning):

Dead load demand $= 1376/\{0.85(2250)\} = 0.72$

Therefore, max joint shear $= 0.72(30+20)(2 \text{ panels}) = 72$ kip (controls)

If three connections per panel are used:

$V_{u\,Connx} = (72 \text{ kip})/3$
$= 24.0$ kip each

Each connection is required to resist this shear force.

A connection between panels similar to that shown in section 2.10.7 can be designed for the 24.0 kip load using design methods in chapter 6 of the *PCI Design Handbook*.

Panel-to-Foundation Connections and Panel-to-Roof Connections

Since there is no uplift tension, the panel connection to foundation must transfer the shear force either by individual connection shear or shear friction, with the panel connectors serving as the shear-friction steel.

Use two connections per panel.

V_u = 16.1(1.0)/(2 connections)
= 8.05 kip each

A connection between the panel and the foundation similar to that shown in section 2.10.2 can be designed for the 8.05 kip load using design methods in chapter 6 of the *PCI Design Handbook*. Connections similar to that shown in section 2.10.3 can be designed for the panel-to-roof connections. See Example A2 for prestress design and handling design.

EXAMPLE A4. FULLY COMPOSITE CLADDING PANEL

Design criteria (see Fig. A.9)

Wind load = 30 lb/ft^2
Differential temperature = 30 ºF

f'_c = 5000 psi

f_{pu} = 270 ksi

Normalweight concrete

Strands:

Use $^3/_8$-in.-diameter, 270 ksi, low-relaxation strands.
Minimum prestress = 225 psi on gross section.
Try a composite insulated wall panel, 3/2/3. Assume 100% composite action for ultimate strength.

Section properties

A = $t_s a$ = (3 in.)(96 in.)(2)
= 576 in.^{2}

I = [(96)(3)^3/12 + (96)(3)(2.5)^2](2)
= 4032 in.^{4}

w = 2(3 in.)/12 in./ft (150 lb/ft^3)(8 ft)

w = 600 lb/ft (self-weight)

Prestress

Find A_{ps} by using the minimum prestress stress of 225 psi (according to the ACI code, transverse reinforcement is not required if the panel is prestressed to at least 225 psi):

$$A_{ps,\,min.} = \frac{0.225A}{P}$$

Note: To calculate P, 0.75 is the percent strand pull, 270 is the ultimate strength in ksi, and the 0.85 coefficient approximates the strand stress after all losses.

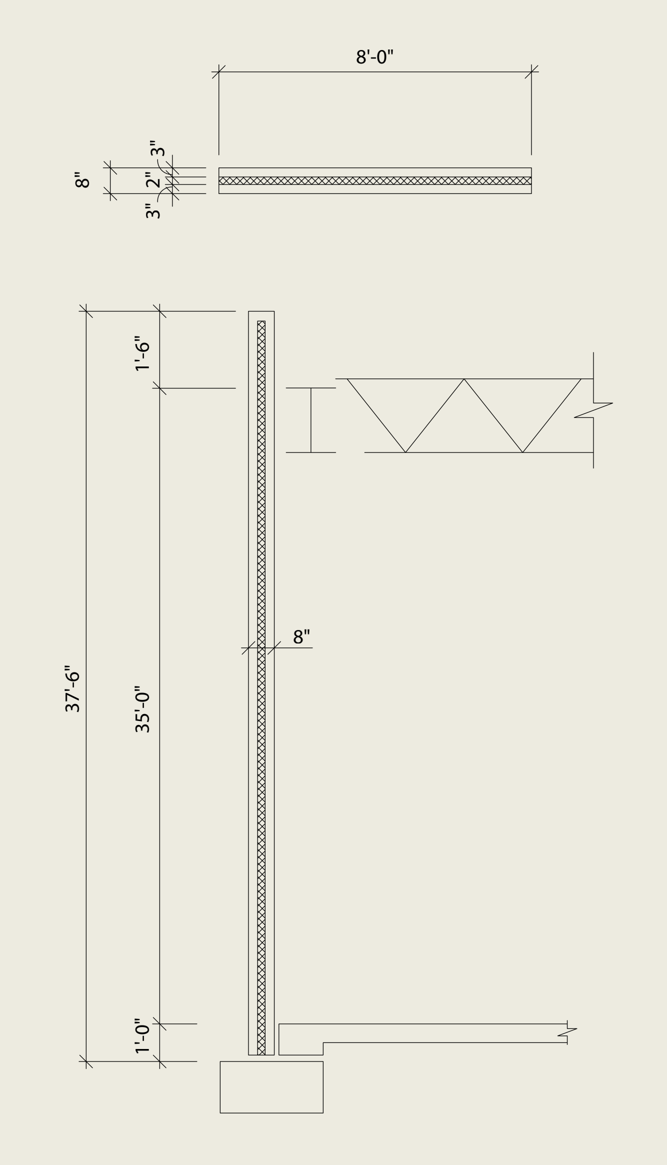

Figure A.9. Example A4.

$$= \frac{0.225\left(576\right)}{0.75\left(0.85\times 270\right)}$$
$$= 0.752 \text{ in.}^2$$

Number of strands (minimum) = 0.752/0.085 = 8.85 ($^3/_8$ in. diameter)

Try ten $^3/_8$-in.-diameter strands = 0.85 in.^{2}.

Use five $^3/_8$-in.-diameter strands in each wythe.

Normally, a P-Δ check is not necessary for a non-load-bearing panel. The calculation is included here to demonstrate that P-Δ does not control in this case.

Analysis

- Check conditions at mid-height
- Consider P-Δ effect

At mid-height:

Dead load and lateral wind load:

Dead load (self-weight):
P_D = (0.600)(36/2 + 1.5)
= 11.7 kip

Wind load:
$M_w = (0.030)(8)(36)^2/8$
= 38.9 kip-ft

Consider thermal bow because the panel is composite:

$\Delta_T = C(T_1 - T_2)\ l^2/8h$ (T_1 = outside temperature, T_2 = inside temperature)
$= (6 \text{ x } 10^{-6})(30)(36 \times 12)^2/(8 \text{ x } 8)$
= 0.5 in.

Initial bow:

Assume an initial outward bow of l/360, based on the PCI specified maximum bow tolerance, Δ_i = 1.17 in.

Determine EI from *PCI Design Handbook*:

$\phi_k = 0.85$

(The stiffness-reduction factor is assumed to be at least 0.85 considering the stringent dimensional accuracy provided by a precasting plant.)

$$E_c = 57\sqrt{f_c'} = 57\sqrt{5000}$$

= 4030 ksi

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(4032)}{1+0}$$

$= 1.38 \times 10^7$ kip-in.^{2}

in which $\beta_d = 0$ because sustained self-weight is small.

Wind deflection:

$$\Delta = \frac{5}{384}\left(\frac{w_{wind} l^4}{EI}\right)$$

w = 30 lb/ft^2 x 8 ft / 1000 lb/kip = 0.24 kip/ft

$$\Delta_w = \frac{5}{384}\left(\frac{(1.6\times0.24)(432)^4}{(1.38\times10^7)}\right)\left(\frac{1}{12}\right)$$

= 1.05 in.

Case 1, Wind Load: $U = 1.2D + 1.6W + 1.0L + 0.5$ (L_r or S or R)

$L = L_r = S = R = 0$ so $= 1.2D + 1.6W$

P_u = 1.2(11.7)
= 14.0 kip

$EI = \phi_k EI/(1+\beta_d)$

$M_u = (1.6)(38.9) + P\Delta$
$= 62.2 + P\Delta$

$\Delta_i + \Delta_w$ = 1.17 + 1.05
= 2.22 in.

***P*-Δ Analysis**

$$\Delta = \frac{Pel^2}{8EI}$$

$$= \frac{(14)e(432)^2}{8(1.38\times10^7)}$$

$= 0.024e$

First iteration:
Δ = 0.024(2.22)
= 0.053 in.

Second iteration:
Δ = 0.024(2.22 + 0.053)
= 0.055 in.

Third iteration:
Δ = 0.024(2.22 + 0.055)
= 0.055 in.
(converges)

M_u = 62.2 + 14(2.22 + 0.055)/12
= 64.9 kip-ft

Case 2: $U = 1.2(D+F+T) + 1.6\,(L+H) + 0.5\,(L_r$ or S or $R)$

P_u = 11.7(1.2)
= 14.0 kip

where $\beta_d = 1$ due to sustained dead load and $EI = 6.90 \times 10^6$ k-in.^{2}

$\Delta_T + \Delta_i$ = 0.5 + 1.17 = 1.67 in.

$$\Delta = \frac{Pel^2}{8EI}$$

$$= \frac{(14)e(432)^2}{8(6.9\times10^6)}$$

$= 0.047e$

First iteration:
Δ = 0.047(1.67) = 0.078 in.

Second iteration:
Δ = 0.047(1.67 + 0.078)
= 0.08 in. (converges)

M_u = (14)(1.75)/12
= 2.04 kip-ft

Therefore, the critical case is Case 1: $1.2D + 1.6W + 1.0L + 0.5$ (L_r or S or R)

P_u = 14.0 kip

M_u = 64.9 kip-ft

From Fig. 3.12.5, *PCI Design Handbook*, the interaction capacities for an 8 in. composite panel section are adequate.

Check for cracking at mid-height of panel (use ultimate loads):

P_u = 14.0 kip

M_u = 61.5 kip-ft

$f_{DL+wind} = P/A + Mc/I$:

$$f_{DL+wind} = \left[\frac{14}{576} - \frac{64.9(12)(4)}{4032}\right](1000)$$

= -749 psi (tension)

$f_{pc} = P/A$:

$$f_{pc} = \frac{(10)(0.085)(0.75 \times 270)(10^3)(0.85)}{576}$$

= +254 psi (compression)

Net stress = -749 + 254 = -495 psi (tension)

$$f_r = 7.5\sqrt{f'_c}$$

$$= 7.5\sqrt{5000}$$

= 530 psi max tension
OK

Therefore, section is uncracked.

Check to see whether $\phi M_n > 1.2\,M_{cr}$.

Determine M_n:

Assume the strand stress at ultimate equals the release stress:

f_{ps} = 269 ksi

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b}$$

$$= \frac{(5 \times 0.085)269}{0.85(5)(96)}$$ (Strands in compression wythe are ignored – contribution is small)

= 0.28 in.

d = 8 – 3/2 = 6.50 in.

$\phi M_n = \phi A_{ps} f_{ps}(d - a/2)$

ϕM_n = (0.9)(114.3)[6.5 – (0.28/2)]/12
= 54.5 kip-ft

$$1.2\,M_{cr} = \frac{(1.2)(0.530 + 0.254)(4032)}{(4)(12)}$$

= 79.0 kip-ft
$> \phi M_n$

Add mild reinforcing steel to increase ϕM_n

$$A_{s,\,req.} = \frac{(1.2M_{cr} - \phi M_n)}{d f_y}$$

$$= \frac{(79.0 - 54.5)(12\ in./ft)}{(6.5)(60)} = 0.75 \text{ in.}^2$$

Add no. 4 bars in each wythe at strand level:

n = $A_{s,\,req}$/0.2 in.² = 3.4, add four no. 4 bars

a = 0.40 in., ϕM_n = 76.7 kip-ft vs 79 kip-ft, (OK)(strands in compression wythe ignored)

Composite action of concrete and insulation

Using internal couple force transfer described in Section 2.7.4:

$T_u = nA_{ps}f_{ps}$ = (5)(0.085)(270)
= 114.75 kip [use this value—see following]

C_u = (0.85)(5)(3)(96)
= 1224 kip

Provide wythe interconnectors to resist 114.75 kip ultimate in each half height of panel.

Stripping (see Fig. A.10)

f'_{ci} = 3500 psi

w = 75 lb/ft²

a = 8 ft

b = 37.5 ft

Static load multiplier = 1.3 (Table A.1)

w_i = (75)(1.3)
= 98 lb/ft²

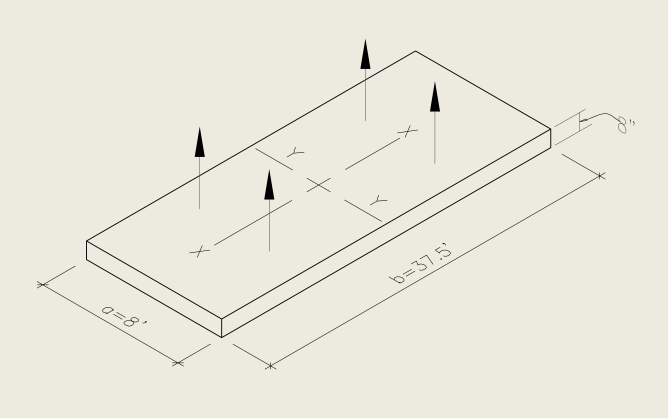

Figure A.10. Stripping diagram.

x-x Direction

$M_x = 0.0107wa^2b$
$= (0.0107)(0.098)(8)^2(37.5)$
$= 2.52$ kip-ft

over a width of $15t = 15(8) = 120$ in.

$I_y = [(120)(3)^3/12 + (120)(3)(2.5)^2](2)$
$= 5040$ in.^{4}

$f_b = MC/I$

$f_b = (2520)(12)(4)/5040$
$= 24$ psi

$f_{pc} = 0$, transverse direction – no prestress

Net stress = 24 – 0 = 24 psi

$f'_r = 5\sqrt{f'_c}$

$= 5\sqrt{3500}$

$= 296$ psi (OK)

y-y Direction

$M_y = 0.0107\ wab^2$
$= (0.0107)(0.098)(8)(37.5)^2$
$= 11.80$ kip-ft

over a width equal to $0.50a = 0.50(8)(12) = 48$ in.

$I_x = [(48)(3)^3/12 + (48)(3)(2.5)^2](2)$
$= 2016$ in.^{4}

$f_{by} = -(11800)(12)(4)/2016$
$= -281$ psi

$f_{pc} = (254)(0.9)/0.85$ (final prestress stress converted to initial stress by ratio of 0.9/0.85)
$= +269$ psi

Net stress = -281 + 269 = -12 psi

Note that only 10% stress loss occurred at this stage.

Yard Handling

Use the same arrangement as shown previously.

Static load multiplier = 1.2 (Table A.1)

Shipping (see Fig. A.11)

Static load multiplier = 1.5 (Table A.1)

$w = 75\ (1.5) = 113$ lb/ft^2

$f_{by} = (-281)(113)/98$
$= -324$ psi

$f_{pc} = +254$ psi (use final prestress stress for this stage)

Net stress = -324 + 254

= -70 psi (OK)

The optimal balance between positive and negative moments is achieved by bunking at 0.207 times the panel length:

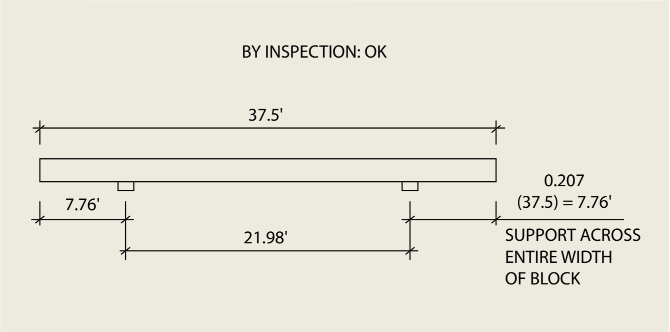

Figure A.11. Shipping diagram.

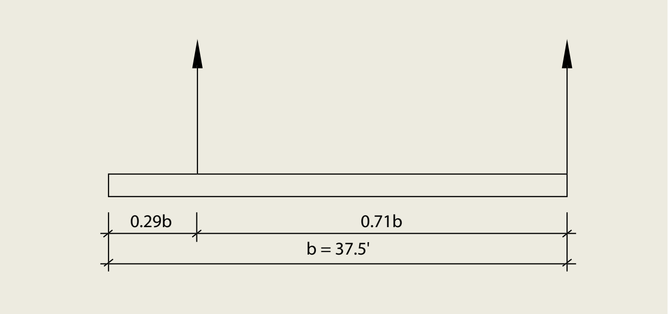

Figure A.12. Erection two-point pick.

Erection (see Fig. A.12)

Refer to *PCI Design Handbook*.

Try two-point pick-up.

Static load multiplier = 1.2 (Table A.1)

$w = 75(1.2) = 90$ lb/ft^2

or (90)(8)/1000 = 0.72 kip/ft

$M^{\pm} = 0.044wb^2$
$= (0.044)(0.72)(37.5)^2$
$= 44.6$ kip-ft

Effective width = 96 in.

$f_{by} = (44.6)(12000)(4)/4032$
$= -530$ psi

$f_{pc} = +254$ psi

Net stress = -530 + 254 = -276 psi

which is less than $5\sqrt{5000} = 354$ psi (OK, $7.5\sqrt{5000}$ cracking stress with a 1.5 factor)

EXAMPLE A5. FULLY COMPOSITE LOAD-BEARING PANEL

Design criteria (see Fig. A.13)

Wind load:
Direct pressure = 20 lb/ft^2
Suction pressure = 10 lb/ft^2
Differential temperature = 30 °F
Service roof dead loads:
8-ft-wide double tee (8*DT*18) spanning 40 ft = 45 lb/ft^2
Roofing and mechanical = 10 lb/ft^2
Service roof snow load = 30 lb/ft^2

Roof reaction:

Dead load = (0.045+0.010)(20)(8)
= 8.8 kip

Snow load = (0.030)(20)(8)
= 4.8 kip

f'_c = 5000 psi

E_c = 4030 ksi

f_{pu} = 270 ksi

The roof acts as a diaphragm supported for lateral loads by shear walls at the ends of the building. Therefore, the wall panel is braced at the top and bottom. The wall panels are connected to the double tee flanges at 1 ft 6 in. below the top of the panel and are connected to the top of the floor slab at 1 ft 0 in. above the panel base. Therefore, the unbraced length of the panels is assumed to be 35 ft 0 in.

Section properties

$A = at = (6)(96) = 576$ in.^{2}

$I = b(h_1^3 - h_2^3)/12 = 4032$ in.^{4}

Note: The value of I for deflection should be verified by full-scale testing of panels with a similar configuration to the design panel. Overestimating the effective moment of inertia I is unconservative, as the stability calculations are directly affected by the value of EI.

$S = 1008$ in.^{3}

w = (6 in./12 in./ft)150 lb/ft^3(8 ft) = 600 lb/ft

Strands

Use 3/8-in.-diameter, low-relaxation, 270 ksi strands.

Minimum prestress = 225 psi = f_{pc}

Assume prestress losses = 15%

Prestress

$A_{ps,min.} = 0.225\ (A/f_{se})$

Percent pull = 75%, final losses estimated at 15%:

$A_{ps,min.}$ = (0.225)(576)/[(0.75)(270)(0.85)]
= 0.753 in.^{2}

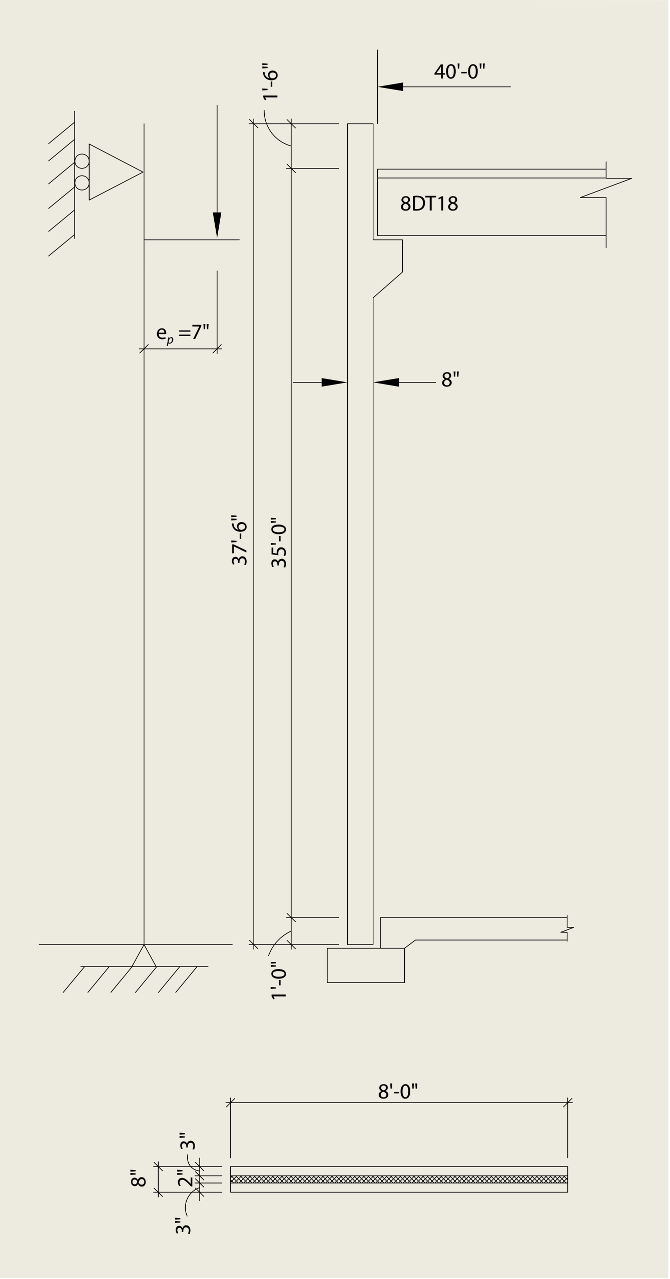

Figure A.13. Example A5.

Number of strands = 0.753/0.085
= 8.86 (use 10 strands)

f_{pc} = [(0.75)(270)(0.85)(10)(0.085)(10^3)]/576 = 254 psi with 10 strands > 225 psi (OK)

Use five 3/8-in.-diameter strands in each wythe.

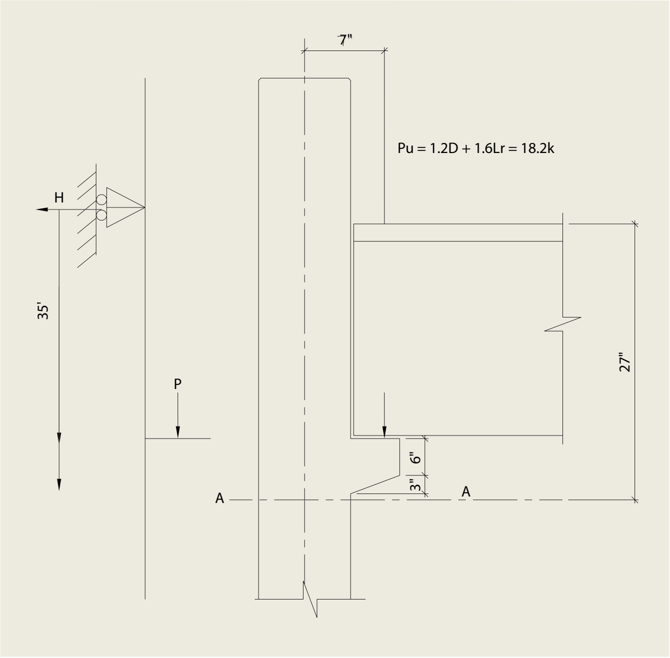

Figure A.14. Top detail.

Analysis

Case 1: $U = 1.2D + 1.6\ (L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)$

For wind, use 10 lb/ft^2 suction, which causes moments additive to moments due to dead and snow load and eccentricity.

Load at top of panel:

$P_{u\text{-}top} = 1.2(8.8) + 1.6(4.8)$
$= 18.2$ kip

Panel weight at midheight:
$P_{u\text{-}mid(self\text{-}wt)} = 1.2(0.600)(35/2) + 1.5)$
$= 13.7$ kip

Total factored load at midheight = $P_{u\text{-}mid}$ = 18.2 + 13.7 = 31.9 kip

Note: Midheight may not necessarily be the point of maximum bending moment because of the large moment applied at the corbel. It may be necessary to check several sections along the panel height to confirm where the maximum moment in the panel occurs.

$$EI = \frac{\phi_k E_c I}{1+\beta_d}$$

where

$\beta_d = [13.7 + 1.2(8.8)]/31.9$
$= 0.76$

$\phi_k = 0.85$

Stiffness-reduction factor is assumed to be at least 0.85, considering the stringent dimensional accuracy provided by a precasting plant.

Therefore:

$$EI = \frac{0.85(4030)(4032)}{1+0.76}$$

$= 7.85 \times 10^6$ kip-in.^{2}

Deflection at midheight due to factored load on corbel of panel:

$\Delta = P_{u\text{-}top}e_p l^2/16EI$

See Fig. A.13 for 7 in. eccentricity = e_p

$$\Delta = \frac{(18.2)(7)(420)^2}{16(7.85\times10^6)}$$ (The unbraced span length is 35 ft = (35)(12) = 420 in.)

$= 0.18$ in.

Deflection due to wind:

For wind, $\beta_d = 0$

$$EI = \frac{0.85(4030)(4032)}{1+0}$$

$= 1.38 \times 10^7$ kip-in.^{2}

$$\Delta = \frac{5}{384}\left(\frac{w_{u,wind}\ell^4}{EI}\right)$$

$$= \frac{5}{384}\left(\frac{(0.01)(0.8)(8)(420)^4}{12(1.38\times10^7)}\right)$$

$= 0.16$ in.

Assume initial bow = 1.17 in. (b/360)

Total deflection = 0.18 + 0.16 + 1.17

= 1.51 in.

P-Δ Analysis

$\Delta = P_{u\text{-}mid}el^2/8EI$

$$= \frac{(31.9)e(420)^2}{8(7.85\times10^6)}$$

$= 0.09e$

First iteration:
$e = 1.51$ in.
$\Delta = (0.09)(1.51) = 0.14$ in.

Second iteration:
$e = 1.51 + 0.14 = 1.65$ in.
$\Delta = (0.09)(1.65) = 0.15$ in.

Third iteration:
$e = 1.51 + 0.15 = 1.66$ in.
$\Delta = (0.09)(1.66) = 0.15$ in.
(converges)

Moment at midheight:

$M_u = P_{u\text{-}top}(e_P)/2 + P_{u\text{-}mid}\,e = (18.2)(7)/2 + (31.9)(1.66)$
$= 116.7$ kip-in.

Check for cracking:

$f = (31.9)(10^3)/576 - (116.7)(10^3)/1008$
$= -60$ psi

$f_{pc} = 254$ psi

Net stress $= -60 + 254$
$= +194$ psi (compression)

Therefore, stresses are satisfactory; that is, the panel is uncracked and the P-Δ analysis is valid.

Check interaction curve:

$M_u = 116.7/(8\times12) = 1.21$ ft kip/ft

$P_{u\text{-}mid} = 31.9/8 = 3.99$ kip/ft

From *PCI Design Handbook*, Fig. 3.12.5, the interaction diagram shows that the section is satisfactory.

Case 2: $U = 1.2D + 1.6W + 1.0L + 0.5\ (L_r$ or S or $R)$

For wind, use 10 lb/ft² suction, which causes moments additive to moments due to dead and snow load and eccentricity.

Load at top of panel:

$P_{u\text{-}top} = [1.2(8.8) + 0.5(4.8)]$
$= 13$ kip

Panel weight at midheight:

$P_{u\text{-}mid(self\text{-}wt)} = 13.7$ kip

Total factored load at midheight

$P_{u\text{-}mid} = 13 + 13.7 = 26.7$ kip

$$EI = \frac{\phi_k E_c I}{1 + \beta_d}$$

Where

$\beta_d = [13.5 + (1.2)(8.8)]/26.5$
$= 0.91$

Therefore, for $\beta_d = 0.91$, $EI = 7.23 \times 10^6$ kip-in.²

for wind $\beta_d = 0$, $EI = 1.38 \times 10^7$ kip-in.²

Deflection at midheight due to load at top:

$\Delta = 0.14$ in.

Deflection due to wind:

$$\Delta = \frac{5}{384}\left(\frac{w_{wind} l^4}{EI}\right)$$

$$= \frac{5}{384}\left(\frac{(0.01\times1.6\times8)(420)^4}{12(1.38\times10^7)}\right)$$

$= 0.31$ in.

Assume initial bow = 1.17 in.

Total deflection $= 0.14 + 0.31 + 1.17$
$= 1.62$ in.

P-Δ Analysis

$\Delta = P_{u\text{-}mid}el^2/8EI$

$$= \frac{(26.7)e(420)^2}{8(7.23\times10^6)}$$

$= 0.081e$

First iteration:
$e = 1.62$ in.

$\Delta = (0.081)(1.62) = 0.13$ in.

Second iteration:
$e = 1.75$ in.

$\Delta = (0.081)(1.75) = 0.14$ in.

Third iteration:
$e = 1.76$ in.

$\Delta = (0.081)(1.76) = 0.14$ in.
(converges)

$M_w = wl^2/8 = (0.01)(8)(35 \times 12)^2/(12 \times 8)$
$= 147$ kip-in.

$M_{uw} = 147 \times 1.6$
$= 235.2$ kip-in.

$M_{uPu} = P_{u\text{-}top}(e_P)/2 + P_{u\text{-}mid}\,e = (13.0 \text{ x } 7)/2 + (26.7)(1.76)$
$= 92.5$ kip-in.

Total moment at midheight:

$M_u = 235.2 + 92.5 = 327.7$ kip-in.

Load at midheight:

$P_{u\text{-}mid} = 26.7$ kip

Check for cracking:

$f = (26.7)(10^3)/576 - (327.7)(10^3)/1008$
$= -279$ psi

$f_{pc} = 254$ psi

Net stress $= -279 + 254$
$= -25$ psi

$fr = 7.5\sqrt{5000} = 530$ psi > 25 psi

Therefore, stresses are satisfactory; that is, the panel is uncracked and the P-Δ analysis is valid.

Check interaction curve:

$M_u = 327.7/(8\times12) = 3.41$ kip-ft/ft

$P_{u\text{-}mid} = 26.7/8 = 3.34$ kip/ft

From *PCI Design Handbook*, Fig. 3.12.5, the interaction diagram is satisfactory.

Check for cracking at the base of the corbel, where the moments due to factored gravity loads are highest. Use 35 ft as the unbraced length.

Case 1: $U = 1.2D + 1.6\ (L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)$

At section A-A (Fig. A.14)

$P_{u\text{-}top} = 1.2(8.8) + 1.6(4.8)$
$= 18.2$ kip

$w_{u\text{-}wind} = 0.8(0.01)(8) = 0.064$ kip/ft

Calculate the horizontal reaction at the double tee–to–wall panel connection:

Gravity:

$H_u = P_{u\text{-}top}(e_P)/l = (18.2 \times 7)/(35 \times 12)$
$= 0.30$ kip

Wind:

$$H_u = \frac{36.5^2}{2}\left(0.064\right)/35$$

$= 1.22$ kip

Moment at section A-A:

Gravity:

$M_u = 18.2(7) - 0.30(27)$

$= 119.3$ kip-in.

Wind:

$$M_u = 1.22\left(27\right) - \frac{\left(27 + 18\right)^2}{2}\frac{\left(0.064\right)}{12}$$

$= 27.5$ kip-in.

$f = (18.2 \times 10^3)/576 - (119.3 + 27.5)(10^3)/1008$
$= -114$ psi

Note: Because the base of the corbel is beyond the transfer point, use the full value of f_{pc}.

$f_{pc} = +254$ psi

Net stress = 254 – 114
= 140 psi (compression)

Therefore, stresses are satisfactory; that is, the panel is uncracked and the P-Δ analysis is valid.

Check interaction curve:

$M_u = (119.3 + 27.5)/(8)(12) = 1.53$ kip-ft/ft

$P_{u\text{-}top} = 18.2/8 = 2.28$ kip/ft

From *PCI Design Handbook*, Fig. 3.12.5, the interaction diagram is satisfactory.

Horizontal Shear at Interface of Concrete and Insulation

Strand force transfer will be critical.

$T_u = (5)(0.085)(270)$
$= 114.8$ kip

$C_u = (0.85)(5)(3)(96)$
$= 1224$ kip

Provide wythe connectors to resist 114.8 kip (ultimate) in each half height of panel.

Note: Wythe connectors should be stiff enough to ensure that the relative end slip between wythes is close to zero at ultimate loads. Solid zones of concrete may be necessary to achieve full composite action for panel stiffness.

Check to see if ϕM_n is greater than $1.2M_{cr}$.

See Example A4. Add four no. 4 reinforcing bars in each wythe at strand level.

Handling and Erection

See composite cladding panel, Example A4.