PCI COMMITTEE REPORT

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

Prepared by

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NOTE: Chapters 1 through 8 plus the References and Bibliography were published in the March-April 1997 PCI JOURNAL. The six Design Examples and Notation are presented in this issue.

APPENDIX — DESIGN EXAMPLES

EXAMPLE A1. NON-COMPOSITE CLADDING PANEL

Design Criteria (see Fig. A1)

Wind load:
Direct pressure = 15 lb per sq ft (0.72 kPa)
Suction pressure = 7 lb per sq ft (0.34 kPa)
Differential temperature = 30°F (17°C)

f'_c = 5000 psi (34.5 MPa)
f'_{ci} = 3500 psi (24.1 MPa)

For handling and service load conditions, use non-composite analysis.
Note: The inner wythe [4 in. (102 mm) thick] is used as the structural wythe and is assumed to resist 100 percent of the service and handling loads.

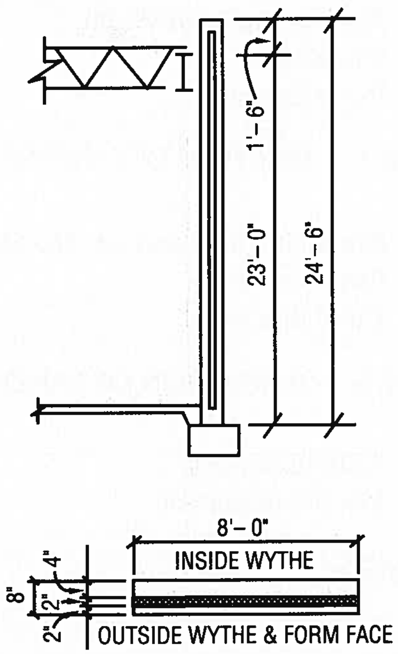


Fig. A1

Inner Wythe Section Properties

A = 4(96) = 384 in.² (2.48 × 10⁵ mm²)
I = 96(4³)/12 = 512 in.⁴ (2.13 × 10⁸ mm⁴)
S = 512/2 = 256 in.³ (4.20 × 10⁶ mm³)

Analysis

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

$U = 0.75\,(1.4D + 1.7L + 1.7W)$

Direct pressure governs since it is larger than suction pressure.
Note: In calculating wind moment, reduction caused by 1 ft 6 in. (0.46 m) parapet is neglected.

$$M_{max} = (1/8)(0.015 \times 8)(23^2)$$
$$= 7.94 \text{ ft-kips (10.8 kN-m)}$$

$$M_u = (7.94)(0.75)(1.7)$$
$$= 10.1 \text{ ft-kips (13.7 kN-m)}$$

In calculating the deflection due to wind, assume $\phi = 0.9$ and $\beta_d = 0$.

$$\Delta = (5/384)(wl^4 / EI)$$

$$E_c = 57\sqrt{5000}$$
$$= 4030 \text{ ksi (27790 MPa)}$$

$$EI = \frac{\phi E_c I}{1+\beta_d}$$
$$= \frac{(0.9)(4030)(512)}{1+0}$$
$$= 1.86 \times 10^6 \text{ kips-in.}^2$$

$$\Delta_w = (5/384)\frac{(0.12/12)(23 \times 12)^4}{1.86 \times 10^6}$$
$$= 0.41 \text{ in. (10.4 mm)}$$

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 non-composite panel.

$$P_u = 1.4\,(6/12)(8)(0.15)\,[(23/2) + 1.5]$$
$$= 10.9 \text{ kips (48.5 kN) at midheight}$$

$$\phi = 0.7 \text{ at } P_u = 0.1\, f'_c\, A$$

$$P_u = (0.1)(5)(384)$$
$$= 192 \text{ kips (854 kN)}$$

$$\phi = 0.9 \text{ at } P_u = 0$$

$$\phi = 0.9 - (0.2)(10.9/192)$$
$$= 0.89$$

$$\beta_d = 1.0$$

$$EI = \phi\, E_c I/(1 + \beta_d)$$
$$= (0.89)(4030)(512)/(1 + 1)$$
$$= 918{,}000 \text{ kips-in.}^2$$

Assume that the initial bow of the panel is ±¹/₂ in. (13 mm) and is additive to the deflection due to the wind.

Note that thermal bowing, Δ_T, is not additive to the combined deflection due to initial conditions plus wind ($\Delta_{initial}$ + Δ_{wind}). This is because Δ_T is covered in the ACI 318-95 provisions for loading case:

$U = 0.75\,(1.4D + 1.4T + 1.7L)$
but not less than $U = 1.4\,(D + T)$.

In other words, ACI 318-95 does not require the combination of wind loads and thermal loads. Therefore, the deflection of the panel at midspan is:

$$\Delta_{midspan} = 0.5 + 0.41$$
$$= 0.91 \text{ in. (23 mm)}$$

$$\Delta = Pel^2/8EI$$
$$= (10.9)\, e\, (23 \times 12)^2/[(8)(918{,}000)]$$
$$= 0.113e$$

First iteration:
$\Delta = 0.113\,(0.91) = 0.10$ in. (2.54 mm)

Second iteration:
$e = 0.91 + 0.10 = 1.01$ in. (25.7 mm)
$\Delta = 0.113\ (1.01) = 0.12$ in. (3.05 mm)

Third iteration:
$e = 0.91 + 0.12 = 1.03$ in. (26.2 mm)
$\Delta = 0.113\ (1.03) = 0.12$ in. (3.05 mm)
resulting in convergence.

The ultimate moment due to wind and $P\text{-}\Delta$ effects is:

$$M_u = M_{u,\,wind} + M_{u,\,P\text{-}\Delta}$$
$$= (10.1)(12) + (10.9)(1.03)$$
$$= 121 + 11 = 132 \text{ in.-kips or } 11 \text{ ft-kips } (14.9 \text{ kN-m})$$

Note that wind contributes 92 percent of M_u while $P\text{-}\Delta$ effects contribute only 8 percent. Since the validity of the $P\text{-}\Delta$ analysis is determined by comparing $M_{u,\,P\text{-}\Delta}$ with M_{cr}, the level of prestress must be determined; then, M_{cr} is compared against $M_{u,\,P\text{-}\Delta}$. However, since the $P\text{-}\Delta$ effect is so small compared to wind, it is not reasonable to compare $M_u = M_{u,\,wind} + M_{u,\,P\text{-}\Delta}$ vs. M_{cr} when normally the service moments, not the ultimate moments, are compared to M_{cr}. Comparison of $M_u = M_{u,\,wind} + M_{u,\,P\text{-}\Delta}$ vs. M_{cr} is logical only when the panel is loadbearing and the $P\text{-}\Delta$ effect is more significant.

Thermal Bow

Since this is a non-composite panel, the panel thickness is assumed to be the distance between the center of gravity of the wythes:

$h = 2 + 2 + 1 = 5$ in. (127 mm)

$$\Delta_T = C(T_1 - T_2)(l^2)/8h$$
$$= (6 \times 10^{-6})(30)(23 \times 12)^2/(8 \times 5)$$
$$= 0.34 \text{ in. } (8.6 \text{ mm})$$

Note that since $\Delta_T = 0.34$ in. (8.6 mm) is less than $\Delta_{wind} = 0.41$ in. (10.4 mm) and since thermal bow in this case does not contribute to panel stresses (because the bowing is not restrained), the load case $U = 0.75\ (1.4D + 1.4T + 1.7L)$, but not less than $U = 1.4\ (D + T)$ (will not govern).

Stripping and Yard Handling (see Fig. A2)

$f'_{ci} = 3500$ psi (24.1 MPa)
Stripping equivalent static multiplier = 1.3
Yard handling equivalent static multiplier = 1.2
Therefore, stripping governs.

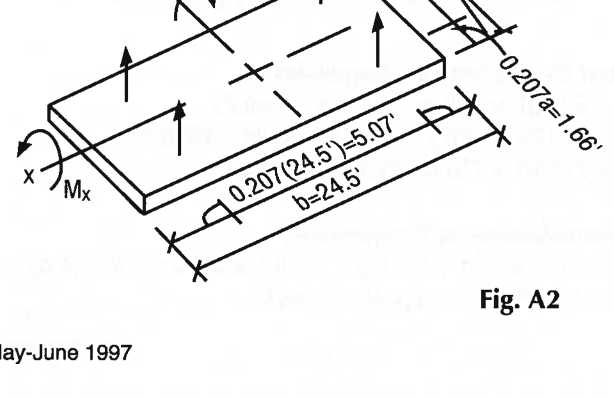


Fig. A2

$$w = 1.3\ (0.075)$$
$$= 0.098 \text{ kips per sq ft } (4.69 \text{ kPa})$$

$$+M_x = -M_x = (0.0107)wa^2b$$
$$= (0.0107)(0.098)(8)^2(24.5)$$
$$= 1.6 \text{ ft-kips } (2.17 \text{ kN-m})$$

M_x is resisted by an effective width of
$15t = (15)(4) = 60$ in. (1524 mm) (governs)
or $b/2 = (24.5)(12)/2 = 147$ in. (3734 mm) (does not govern)

$$S_x = (1/6)(4^2)(60)$$
$$= 160 \text{ in.}^3\ (2.62 \times 10^6 \text{ m}^3)$$

$$M_x/S_x = (1.6)(12000)/160$$
$$= 120 \text{ psi } (0.83 \text{ MPa})$$

which is less than $5\sqrt{3500} = 296$ psi (2.04 MPa) (ok)

$$+M_y = -M_y = (0.0107)wab^2$$
$$= (0.0107)(0.098)(8)(24.5^2)$$
$$= 5.04 \text{ ft-kips } (6.85 \text{ kN-m})$$

M_y is resisted by an effective width of:
$a/2 = (8)(12)/2 = 48$ in. (1219 mm)

Moment required for total panel width:

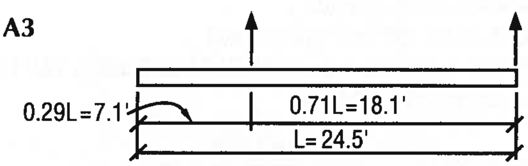
$$M_y = (2)(5.05)$$
$$= 10.1 \text{ ft-kips } (13.7 \text{ kN-m})$$

Check stresses after prestress level is determined.

Travel

Since the panel will be supported at the same locations as at stripping, travel stresses will not govern. The ratio of the travel equivalent static multiplier of 1.5 to the stripping multiplier of 1.3 is 1.5/1.3 = 1.15. The ratio of the square root travel concrete strength of 5000 psi to stripping concrete strength of 3500 psi is $\sqrt{5000}/\sqrt{3500} = 1.19$.

Fig. A3



Erection

Try a two-point pick (see Fig. A3) with equal $+M$ and $-M$: $M = 0.044wl^2$.

Equivalent static multiplier = 1.2

$$w = 1.2\ (0.075)(8)$$
$$= 0.72 \text{ kips per ft } (10.5 \text{ kN/m})$$

$$M = (0.044)(0.72)(24.5^2)$$
$$= 19.1 \text{ ft-kips } (25.9 \text{ kN-m})$$

$$f_{bv} = (19.1)(12{,}000)/256$$
$$= 896 \text{ psi } (6.18 \text{ MPa})$$

$$f_{pc,\,req.} = 896 - 5\sqrt{5000} = 543 \text{ psi } (3.74 \text{ MPa})$$

This value is too high; therefore, revise the handling scheme.
Try a three-point pick (see Fig. A4):

$$+M_y = 0.034wl^2$$
$$= (0.034)(0.72)(24.5^2)$$
$$= 14.7 \text{ ft-kips } (19.9 \text{ kN-m})$$

$-M_y = 0.011wl^2$ (does not govern)

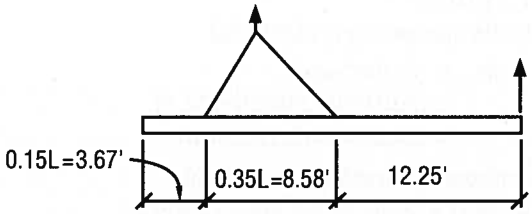


Fig. A4

$+M_y$ = 14.7 ft-kips (19.9 kN-m) governs over

$M_{u,\ wind}$ = 10.1 ft-kips (13.7 kN-m) and

$M_{y,\ handling}$ = 10.1 ft-kips (13.7 kN-m)

$$f_{pc,\ req.} = M/S - 5\sqrt{f_c'}$$
$$= (14.7)(12{,}000)/256 - 5\sqrt{5000}$$
$$= 689 - 354 = 335 \text{ psi } (2.31 \text{ MPa})$$

Determination of Prestress Levels

Assume 15 percent prestress losses. For a 4 in. (102 mm) wythe, use 1/2 in. (13 mm) diameter low-relaxation 270K strands.

A_{ps} = 0.153 sq in. (99 mm²) per strand. Determine number of strands required:

$$\text{No. strands req.} = \frac{f_{pc}A_{wythe}}{(0.7A_{ps}f_{pu})(1-\text{Loss})}$$
$$= \frac{(0.335)(384)}{(0.7)(0.153)(270)(0.85)}$$
$$= 5.3 \text{ [Use six 1/2 in. (13 mm) strands]}$$

For a 2 in. (51 mm) wythe, use 3/8 in. (9.5 mm) diameter low-relaxation 270K strands.

A_{ps} = 0.085 sq in. (55 mm²) per strand

To satisfy the requirements of ACI 318-95, Section 18.11.2.3, $f_{pc,\ min.}$ = 225 psi (1.55 MPa)

$$\text{No. strands req.} = \frac{(0.225)(2)(96)}{(0.7)(0.085)(270)(0.85)}$$
$$= 3.2 \text{ [Use four 3/8 in. (9.5 mm) strands]}$$

Evaluate $\phi M_{n,\ inner}$:

The prestressing steel stress, f_{ps} = 264 ksi (1822 MPa), is determined from methods given in the PCI Design Handbook, Fourth Edition.

$$a = A_{ps}f_{ps}/[(0.85)(f_c')(b)]$$
$$= (6)(0.153)(264)/[(0.85)(5)(96)]$$
$$= 0.59 \text{ in. } (15 \text{ mm})$$

$$\phi M_n = \phi A_{ps}f_{ps}(d - a/2)$$
$$= 0.9\,(6 \times 0.153)(264)(2 - 0.59/2)/12$$
$$= 31.0 \text{ ft-kips } (42.0 \text{ kN-m})$$

This moment is greater than M_u = 11 ft-kips (14.9 kN-m) Therefore:

$$M_{cr} = (f_{pc} + f_r)S$$
$$= (0.384 + 0.53)(256)/12$$
$$= 19.5 \text{ ft-kips } (26.4 \text{ kN-m})$$

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

Therefore, the section is satisfactory.

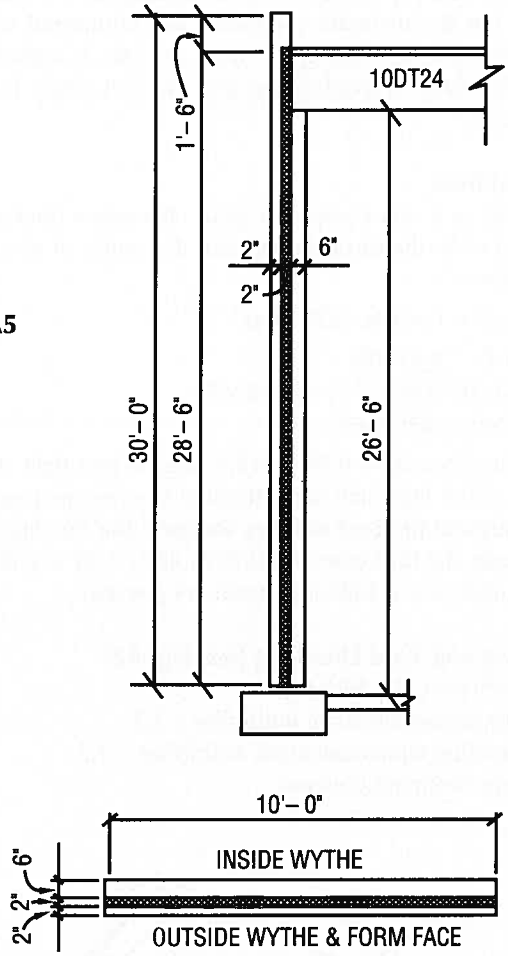
EXAMPLE A2. NON-COMPOSITE LOADBEARING PANEL

Design Criteria (see Fig. A5)

Wind load:
Direct pressure = 28 lbs per sq ft (1.34 kPa)
Suction pressure = 17 lbs per sq ft (0.81 kPa)
Panel dead load = 26.5 kips (95.9 kN)
Roof dead load = 20 kips (89.0 kN)
Roof live load = 6.2 kips (27.6 kN)
f_c' = 5000 psi (34.5 MPa)
f_{ci}' = 3500 psi (24.1 MPa)

Differential temperature = 35°F (19°C).
For handling and service load conditions, use non-composite analysis.
Note: The inner wythe [6 in. (152 mm) thick] is used as the structural wythe and is assumed to resist 100 percent of the service and handling loads.

Fig. A5



Inner Wythe Section Properties

A = 6 (120) = 720 in.² (4.64 × 10⁵ mm²)
I = (1/12)(120)(6³) = 2160 in.⁴ (8.99 × 10⁸ mm⁴)
S = 2160/3 = 720 in.³ (1.18 × 10⁷ mm³)

Determination of Prestressing

For the 2 in. (51 mm) thick wythe, use 3/8 in. (9.5 mm) diameter 270K low-relaxation strands.

$f_{pc,\,min.} = 225$ psi (1.55 MPa)
Assume 15 percent prestress losses.

$$\text{No. strands req.} = \frac{(0.225)(2\times120)}{(0.7\times270)(0.085)(0.85)} = 3.96$$

Try using four 3/8 in. (9.5 mm) diameter strands.

6 in. (152 mm) Wythe

Use 1/2 in. (13 mm) diameter 270K low-relaxation strands.
$f_{pc,\,min.} = 225$ ksi (1.55 MPa)
Assume 15 percent prestress losses.

$$\text{No. strands req.} = \frac{(0.225)(720)}{(0.7\times270)(0.153)(0.85)} = 6.59$$

Try using eight 1/2 in. (13 mm) diameter strands at center of gravity of wythe. Determine prestressing steel stress from methods given in PCI Design Handbook, Fourth Edition.

$f_{ps} = 264$ ksi (1820 MPa)
$a = A_{ps}f_{ps}/(0.85\, f'_c b)$
$= (8)(0.153)(264)/[(0.85)(5)(120)]$
$= 0.63$ in. (16 mm)
$\phi M_n = \phi A_{ps} f_{ps}(d - a/2)$
$= (0.9)(8 \times 0.153)(264)(3 - 0.63/2)/12$
$= 65.1$ ft-kips (88.3 kN-m)
$f_{pc} = (8)(0.153)(0.7 \times 270)(0.85)/720$
$= 0.273$ ksi [or 273 psi (1.88 MPa)]
$M_{cr} = (f_{pc} + f_r)S$
$= (0.273 + 0.530)(720)/12$
$= 48.2$ ft-kips (65.4 kN-m)
$\phi M_n/M_{cr} = 65.1/48.2 = 1.35 > 1.2$ (ok)

Stripping, Handling, Shipping and Erection

Non-composite 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 one-fifth points from ends and sides of panel (see Example A1).
Erection – Use three-point pick with lower points at 4 ft (1.22 m) and 14 ft (4.27 m) from the lower end and inserts in the ledge area near the top of the panel.

Analysis

Case 1: Full factored loads
Loads due to wind suction, dead and live load eccentricity ($P_u e$) and initial bow:

$U = 0.75\,(1.4D + 1.7L + 1.7W)$
$w_{suction} = (10)(0.017)$
$= 0.170$ kips per ft (2.48 kN/m)
$P_u = 0.75[1.4(26.5/2 + 20) + 1.7(6.2)]$
$= 34.9 + 7.9$
$= 42.8$ kips (190 kN)
$\phi = 0.9$ at $P_u = 0$
$\phi = 0.7$ at $P_u = (0.1)(5)(720) = 360$ kips (1601 kN)
$\phi = 0.9 - 0.2\,(42.8/360)$
$= 0.88$
$\beta_{d,\,P\text{-}\Delta} = 34.9/42.8$
$= 0.82$
$EI = \phi E_c I/(1 + \beta_d)$
$= (0.88)(4030)(2160)/(1 + 0.82)$
$= 4.21 \times 10^6$ kips-in.^{2} (2.90×10^7 MPa)

Deflection at mid-height of panel due to roof load at $e = 1$ in. (25.4 mm):

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

$P_u = 0.75[1.4(20) + 1.7(6.2)] = 28.9$ kips (129 kN)

$$\Delta = \frac{(28.9)(1)(26.5\times12)^2}{(16)(4.21\times10^6)} = 0.04 \text{ in. (1 mm)}$$

Assume initial bow = ±1/2 in. (13 mm).

Wind Deflection (Suction)

Use $\beta_d = 0$, $\phi = 0.9$
$EI = (0.9)(4030)(2160)/(1 + 0)$
$= 7.83 \times 10^6$ kips-in.^{2}
$\Delta = (5/384)(wl^4/EI)$
$= (5/384)(0.170/12)(26.5 \times 12)^4/(7.83 \times 10^6)$
$= 0.24$ in. (6.10 mm)
$\Delta_{total} = 0.50 + 0.04 + 0.24$
$= 0.78$ in. (20 mm)

Note that thermal bow is not included in this load case. Refer to Example A1.

P-Δ Analysis

$$\Delta = \frac{Pel^2}{8EI} = \frac{(42.8)e(26.5\times12)^2}{(8)(4.21\times10^6)} = 0.13e$$

First iteration:
$\Delta = 0.13\,(0.78) = 0.10$ in. (2.5 mm)

Second iteration:
$e = 0.78 + 0.10 = 0.88$
$\Delta = 0.13\,(0.88) = 0.11$ in. (2.8 mm)

Third iteration:
$e = 0.78 + 0.11 = 0.89$
$\Delta = 0.13\,(0.89) = 0.12$ in. (3.0 mm) (converges)
$e = 0.78 + 0.12 = 0.90$

$M_{u_1} = (1/2)(P_{u,\,roof})(1) + (P_{u,\,wind\text{-}eccentr.})(0.90) + M_{u,\,wind}$
$= (1/2)(28.9)(1) + (42.8)(0.90) + (0.75)(1.7)(1/8)(0.17) \times (26.5)^2(12)$
$= 14 + 39 + 228$
$= 281$ in.-kips [or 23.5 ft-kips (31.9 kN-m)]
$< \phi M_n = 65.1$ ft-kips (88.3 kN-m) and
$< M_{cr} = 48.2$ ft-kips (65.4 kN-m) (ok)

Case 1a: Wind direct plus initial bow

$U = 0.75\ (1.4D + 1.7L + 1.7W)$

This analysis assumes $e = 0$ and leads to:

$M_u = P_u\Delta + M_{u,\ wind}$

$= (42.8)(1.03) + 28/17(228)$

$= 420$ in.-kips [or 35.0 ft-kips (47.5 kN-m)]

$< \phi M_n = 65.1$ ft-kips (88.3 kN-m)

$< M_{cr} = 48.2$ ft-kips (65.4 kN-m) (ok)

Case 2: Full factored loads

$U = 0.75\ (1.4D + 1.4T + 1.7L)$

but not less than $U = 1.4\ (D + T)$ (does not govern)

Thermal Bow

Since this is a non-composite panel, the thickness is assumed to be the distance between the center of gravities of the wythes:

$\Delta_T = C(T_1 - T_2)(l^2)/8h$

$= (6 \times 10^{-6})(35)(26.5 \times 12)^2/[(8)(6)]$

$= 0.44$ in. (11.2 mm)

$P_{u,\ mid\text{-}height} = 42.8$ kips (190 kN) (same as Case 1)

Therefore:

$EI = 4.21 \times 10^6$ kips-in.^{2} (same as Case 1)

$\Delta_{total} = \Delta_{initial} + \Delta_T$

$= 0.5 + 0.44$

$= 0.94$ in. (23.9 mm)

P-Δ Analysis

$$\Delta = \frac{Pel^2}{8EI} = 0.13e$$

First iteration:

$\Delta = 0.13\ (0.94) = 0.12$ in. (5.08 mm)

Second iteration:

$e = 0.94 + 0.12 = 1.06$

$\Delta = 0.13(1.06) = 0.14$ in. (3.56 mm)

Third iteration:

$e = 0.94 + 0.14 = 1.08$

$\Delta = 0.13(1.08) = 0.14$ in. (3.56 mm) (converges)

$M_{u_2} = (1/2)(P_{u,\ roof})(1) + (P_{u,\ mid\text{-}height})(1.08)$

$= (1/2)(28.9)(1) + (42.8)(1.08)$

$= 14 + 46$

$= 60$ in.-kips [or 5 ft-kips (6.78 kN-m)]

(Does not govern relative to Case 1)

Case 3: $U = 1.4D + 1.7L$

$P_{u,\ roof} = 1.4(20) + 1.7(6.2)$

$= 38.5$ kips (171 kN)

$P_{u,\ mid\text{-}height} = 38.5 + 1.4(26.5/2)$

$= 57$ kips (254 kN)

$\phi\ = 0.9 - 0.2\ (57/360)$

$= 0.87$

$\beta_d = 46.6/57 = 0.82$

$EI = \phi E_c I/(1 + \beta_d)$

$= (0.87)(4030)(2160)/(1 + 0.82)$

$= 4.16 \times 10^6$ kips-in.^{2}

Mid-height deflection due to roof load at $e = 1$ in. (25.4 mm):

$$\Delta_{mid\text{-}height} = \frac{Pel^2}{16EI}$$

$$= \frac{(38.5)(1)(26.5 \times 12)^2}{(16)(4.16 \times 10^6)}$$

$= 0.06$ in. (1.52 mm)

$\Delta_{total} = \Delta_{initial} + \Delta_{Pe}$

$= 0.5 + 0.06$

$= 0.56$ in. (14.2 mm)

P-Δ Analysis

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

$= (57)\ e\ (26.5 \times 12)^2/[(8)(4.16 \times 10^6)]$

$= 0.17e$

First iteration:

$\Delta = 0.17(0.56) = 0.10$ in. (2.54 mm)

Second iteration:

$e = 0.56 + 0.10 = 0.66$

$\Delta = 0.17(0.66) = 0.11$ in. (2.79 mm)

Third iteration:

$e = 0.56 + 0.11 = 0.67$

$\Delta = 0.17(0.67) = 0.11$ in. (2.79 mm) (converges)

$M_{u_3} = (1/2)(P_{u,\ roof})(1) + (P_{u,\ mid\text{-}height})(0.67)$

$= (1/2)(38.5)(1) + (57)(0.67)$

$= 19 + 38$

$= 57$ in.-kips [or 4.8 ft-kips (6.51 kN-m)]

This moment is less than:

$\phi M_n = 65.1$ ft-kips (88.3 kN-m) and

$M_{cr} = 48.2$ ft-kips (63.5 kN-m)

EXAMPLE A3. NON-COMPOSITE SHEAR WALL PANEL

Design Criteria

Same panel as in Example A2. The panel has a seismic shear force of 11.8 kips (52.5 kN) from the roof diagram.

Individual Panel Resists Shear (see Fig. A6)

$U = 0.9D + 1.43E$

Total panel weight $= (100/10^3)(10)(30)$

$= 30$ kips (133 kN)

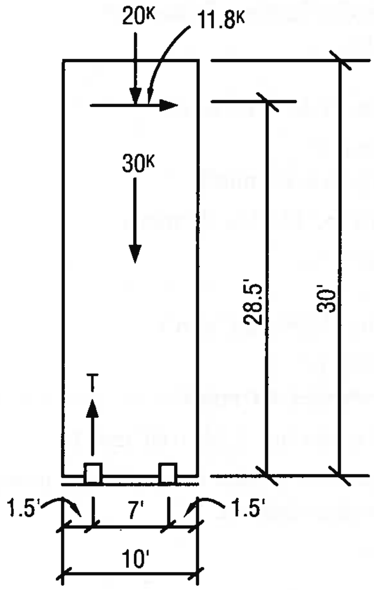
$T_u\ (7 + 1.5) = (11.8)(1.43)(28.5) - (30 + 20)(0.9)(10/2)$

$T_u = (481 - 225)/8.5$

$= 30.2$ kips (134 kN) uplift

Try connecting panels together to eliminate uplift.

Fig. A6



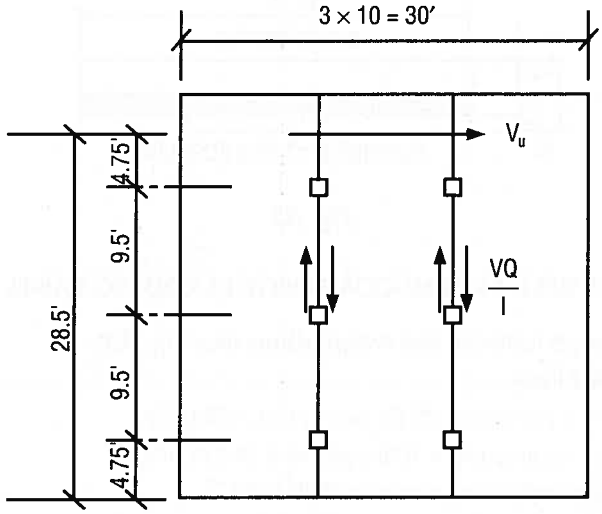


Fig. A8

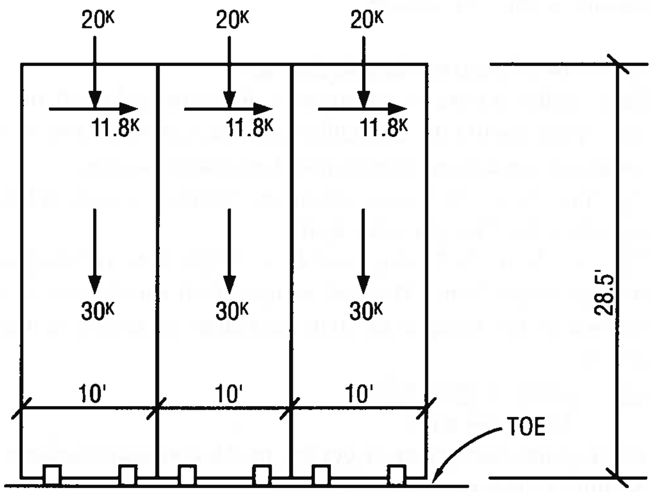


Fig. A7

Panels Connected to Resist Shear (see Fig. A7)

Summation of moments at toe:

Due to dead load:

(30 + 20)(5 + 15 + 25) = 2250 ft-kips (3050 kN)

Due to seismic considerations:

(11.8)(3)(28.5) = 1009 ft-kips (1368 kN)

Strength design: $U = 0.90 + 1.43E$

Due to dead load:

(30 + 20)(0.9)(45) = 2025 ft-kips (2745 kN)

Due to seismic considerations:

(11.8)(1.43)(3)(28.5) = 1443 ft-kips (-1956 kN)

Net moment = 2025 – 1443
= +582 ft-kips (+789 kN)

Due to compression, no uplift connections are required.

Note: Some building codes may require a check of providing a 1.5 factor of safety against uplift when using service loads. In this example, the resistance to uplift is calculated on an ultimate load basis. Since there is no net uplift in the ultimate condition, the *VQ/It* analysis is valid.

Panel-to-Panel Connections (see Fig. A8)

V_u = (11.8)(3)(1.43)
= 50.7 kips (226 kN)

Q = 10(10/2)
= 50 sq ft (4.65 m^2)

I = (1/12)(30^3)
= 2250 ft^3 (63.7 m^3)

V_uQ/I = (50.7)(50)/2250
= 1.13 kips per ft (16.5 kN/m)

If three connections per panel are used:

V_u = 1.13 (30)/3
= 11.3 kips (50.3 kN)

Each connection is required to resist this shear force.

A connection between panels similar to that shown in Section 2.10.7 of the State-of-the-Art Report body can be designed for the 11.3 kips (50.3 kN) using design methods of Chapter 6 of the PCI Design Handbook, Fourth Edition.

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

Use two connections per panel.

V_u = 11.8 (1.43)/2
= 8.5 kips (37.8 kN) each

A connection between the panel and the foundation similar to that shown in Section 2.10.2 of the State-of-the-Art Report body can be designed for the 8.5 kip (37.8 kN) load using design methods of Chapter 6 of the PCI Design Handbook, Fourth Edition. Connections similar to that shown in Section 2.10.3 of the State-of-the-Art Report body can be designed for the panel-to-roof connections. See Example A2 for prestress design and handling design.

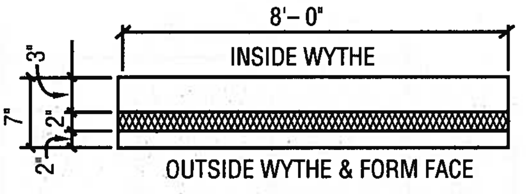


Fig. A9

EXAMPLE A4. SEMI-COMPOSITE CLADDING PANEL

Design Criteria and Assumptions (see Fig. A9)

Wind load:

Direct pressure = 20 lbs per sq ft (0.958 kPa)

Suction pressure = 10 lbs per sq ft (0.479 kPa)

Differential temperature = 30°F (17°C)

Assume initial bow, $\Delta_i = \pm 0.5$ in. (13 mm)

$f'_c = 5000$ psi (34.5 MPa)

$f'_{ci} = 3500$ psi (24.1 MPa)

For stripping, handling and erection, use composite section properties. For service loads, use non-composite section properties.

Composite Section Properties (see Fig. A10)

Dimensions in inches (1 in. = 25.4 mm).

Section	A	y	Ay	Ay^2	I_o
2 x 96	192	1	192	192	64
3 x 96	288	5.5	1584	8712	216
Sum	480		1776	8904	280

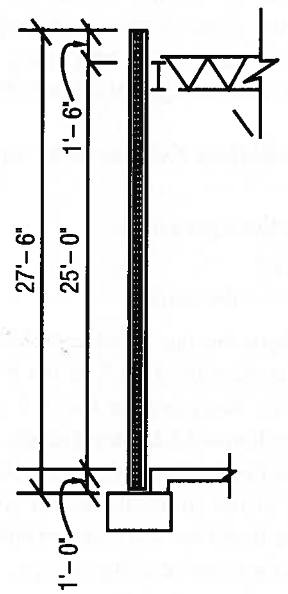
$y = 1776/480$
$= 3.70$ in. (94 mm)

$I = 280 + 8904 - 480(3.7)^2$
$= 2613$ in.^{4} (1.09×10^9 mm^4)

$S_{out} = 2613/3.70$
$= 706$ in.^{3} (1.16×10^7 mm^3)

$S_{in} = 2613/3.30$
$= 792$ in.^{3} (1.30×10^7 mm^3)

Fig. A10



Non-Composite Section Properties

Outside wythe:

$A = 2 \times 96$
$= 192$ sq in. (1.24×10^5 mm^2)

$I = (1/12)(96)(2)^3$
$= 64$ in.^{4} (2.66×10^7 mm^4)

$S = 64/1 = 64$ in.^{3} (1.05×10^6 mm^3)

Inside wythe:

$A = 3 \times 96$
$= 288$ sq in. (1.86×10^5 mm^2)

$I = (1/12)(96)(3)^3$
$= 216$ in.^{4} (8.99×10^7 mm^4)

$S = 216/1.5 = 144$ in.^{3} (2.36×10^6 mm^3)

The distribution of service flexural loads to wythes is based on relative wythe stiffness.

Inside wythe:

$$\frac{216}{64+216} \times 100 = 77 \text{ percent}$$

Outside wythe: 23 percent

Determine Prestress Requirements

Each wythe should be prestressed concentrically and prestress pulls should be adjusted to concentrically prestress the composite section in order to avoid eccentric prestress.

Try four 3/8 in. (9.5 mm) diameter, 270K, low-relaxation strands in the 2 in. (51 mm) wythe.

Try six 3/8 in. (9.5 mm) diameter, 270K, low-relaxation strands in the 3 in. (76 mm) wythe. If all the strands are stressed to the same level, then the center of gravity of the steel is:

cgs $= [(4)(1) + (6)(5.5)]/10$
$= 3.7$ in. (94 mm)

which equals the center of gravity of the composite section.

Assume 15 percent prestress losses.

Outside wythe:

$f_{pc} = (4)(0.085)(0.7)(270)(0.85)/192$
$= 0.284$ ksi (1.96 MPa)
[greater than 0.225 ksi (1.55 MPa)]

Inside wythe:

$f_{pc} = (6)(0.085)(0.7)(270)(0.85)/288$
$= 0.284$ ksi (1.96 MPa)
[greater than 0.225 ksi (1.55 MPa)]

Case 1: $U = 0.75\,(1.4D + 1.7L + 1.7W)$

Inside wythe:

$w_{direct} = (77/100)(0.02)(8)$
$= 0.123$ kips per ft (1.79 kN/m)

$M_w = (1/8)(0.123)(25)^2$
$= 9.6$ ft-kips (13.02 kN-m)

$M_{w_u} = (9.6)(1.7)(0.75)$
$= 12.2$ ft-kips (16.60 kN-m)

Deflection due to wind ($\phi = 0.9$, $\beta_d = 0$)

$EI = \phi EI/(1 + \beta_d)$
$= (0.9)(4030)(216)/(1 + 0)$

$= 783{,}000$ kips-in.^{2}

$\Delta_w = (5/384)wl^4/EI$
$= (5/384)(0.123/12)(25 \times 12)^4/783{,}000$
$= 1.38$ in. (35 mm)

Check $P\text{-}\Delta$ effects on inside wythe:

$P_{u,\,self\text{-}weight} = (288/144)(0.15)(25/2 + 1.5)(1.4)$
$= 4.2(1.4)$
$= 5.9$ kips (26.2 kN)

$0.1 f'_c A_g = (0.1)(5)(288)$
$= 144$ kips (640 kN)

$\phi = 0.9$ at $P_u = 0$; $\phi = 0.7$ at $P_u = 144$ kips (640 kN)

$\phi = 0.9 - 0.2\,(5.9/144) = 0.89$

$\beta_d = 1.0$ $EI = (0.89)(4030)(221)/(1 + 1)$
$= 396{,}000$ kips-in.^{2}

$\Delta = Pel^2/8EI$
$= (5.9)e(25 \times 12)^2/[(8)(396{,}000)]$
$= 0.168e$

At midspan:

$\Delta_{total} = \Delta_w + \Delta_i$
$= 1.38 + 0.5$
$= 1.43$ in. (36 mm)

First iteration:
$\Delta = 0.168(1.43) = 0.24$ in. (6.1 mm)

Second iteration:
$e = 1.43 + 0.24 = 1.67$ in. (42 mm)
$\Delta = 0.168(1.67) = 0.28$ in. (7.1 mm)

Third iteration:
$e = 1.43 + 0.28 = 1.71$ in. (43 mm)
$\Delta = 0.168(1.71) = 0.28$ in. (7.1 mm) (converges)

$M_{u_l} = M_{u,\,w_u} + M_{u,\,P\text{-}\Delta}$
$= 12.2 + (1.71/12)(5.9)$
$= 12.2 + 0.84$
$= 13.04$ ft-kips (17.7 kN-m)

Note that for the outside wythe use:

$M_{u_l} = [(23/100)/(77/100)](13.04)$
$= 3.9$ ft-kips (5.3 kN-m)

Case 2: $U = 0.75\,(1.4D + 1.4T + 1.7L)$
but not less than $U = 1.4(D + T)$

Thermal Bow

Since this is a non-composite panel in the service condition, use the thickness as the distance between the wythe center of gravities.

$h = 1 + 2 + 1.5$
$= 4.5$ in. (114 mm)

$\Delta_T = C(T_1 - T_2)(l^2)/8h$
$= (6 \times 10^{-6})(30)(25 \times 12)^2/[(8)(4.5)]$
$= 0.45$ in. (11 mm)

$\Delta_{(total)_2} = \Delta_T + \Delta_i$
$= 0.45 + 0.50$
$= 0.95$ in. (24 mm)

Since $\Delta_T < \Delta_w$ and unrestrained thermal bow contributes no moment, Case 1 governs over Case 2.

Determine ϕM_n

Inside wythe:

$A_{ps} = 6(0.085)$
$= 0.51$ sq in. (329 mm^2)

f_{ps} was determined to be 264 ksi (1820 MPa).

$a = A_{ps} f_{ps}/(0.85 f'_c b)$
$= (0.51)(264)/[(0.85)(5)(96)]$
$= 0.33$ in. (8.4 mm)

$\phi M_n = (0.9)(0.51)(264)(1.5 - 0.33/2)/12$
$= 13.5$ ft-kips (60 kN-m)

which is greater than $M_{u_l} = 13.04$ ft-kips (17.7 kN-m) (ok)

Outside wythe:

$A_{ps} = 4(0.085)$
$= 0.34$ sq in. (219 mm^2)

Assume $f_{ps} = 264$ ksi (1820 MPa).

$a = (0.34)(264)/[(0.85)(5)(96)]$
$= 0.22$ in. (5.6 mm)

$\phi M_n = (0.9)(0.34)(264)(1 - 0.22/2)/12$
$= 6.0$ ft-kips (8.13 kN-m)

which is greater than $M_{u_l} = 3.9$ ft-kips (5.29 kN-m) (ok)

Determine M_{cr}

Inside wythe:

$M_{cr} = (f_{pc} + f_r)(S_{inside})$
$= (0.284 + 0.530)(144)/12$
$= 9.8$ ft-kips (13.3 kN-m)

Case 1:

$M_{cr} = 9.8 + (4.2/288)(144)/12$
$= 9.8 + 0.2$
$= 10.0$ ft-kips (13.6 kN-m)

Note: A $P\text{-}\Delta$ analysis resulted in $M_{u_l} = 13.0$ ft-kips (17.6 kN-m). However, since $P\text{-}\Delta$ added only 0.8 ft-kips (1.08 kN-m) and $M_w = 9.6$ ft-kips (13.0 kN-m) is less than M_{cr}, the section is satisfactory. See Example A1 for further discussion of this topic.

$1.2\,M_{cr}$ vs. ϕM_n
$1.2\,M_{cr} = 1.2(9.8) = 11.8$ ft-kips (16.0 kN-m)
which is less than $\phi M_n = 13.5$ ft-kips (18.3 kN-m) (ok).

Outside wythe:

$M_{cr} = (f_{pc} + f_r)(S_{outside})$
$= (0.284 + 0.53)(64)/12$
$= 4.34$ ft-kips (5.88 kN-m)

$1.2\,M_{cr}$ vs. ϕM_n
$1.2\,M_{cr} = 1.2(4.34) = 5.2$ ft-kips (7.05 kN-m)
which is less than $\phi M_n = 6.0$ ft-kips (8.13 kN-m) (ok).

Stripping (see Fig. A11)

Use composite section properties.

$f'_{ci} = 3500$ psi (24.1 MPa)
$w = 64$ lb per sq ft (3.06 kPa)
$a = 8.0$ ft (2.44 m)
$b = 27.5$ ft (8.38 m)
Static multiplier = 1.3

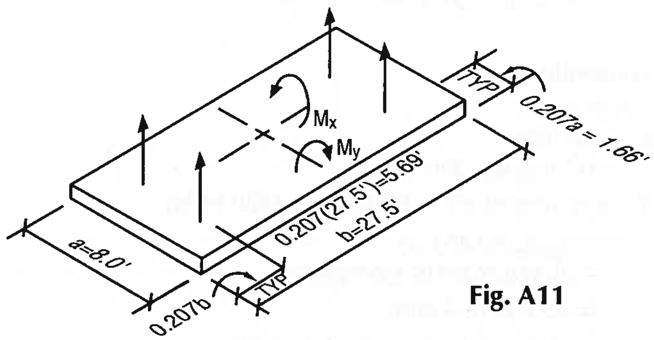


Fig. A11

x-x Direction

$M_x = 0.0107wa^2b$
$= 0.0107\ (0.064 \times 1.3)(8)^2(27.5)$
$= 1.56$ ft-kips (2.11 kN-m) over a width equal to $15t = (15)(7) = 105$ in. (2670 mm)

Section properties of 105 in. (2670 mm) width by ratio to 96 in. (2440 mm) width values:

$I_{xx} = 2613(105/96) = 2858$ in.^{4} (1.19×10^9 mm^4)

$f_p = 0$

$S_{xx,\,outer} = 2858/3.70$
$= 772$ in.^{3} (1.27×10^7 mm^3)

$f_{bx,\,outer} = \pm 1.56(12000)/772$
$= \pm 24$ psi (0.17 MPa) (ok)

$S_{xx,\,inner} = 2858/3.30$
$= 866$ in.^{3} (1.42×10^7 mm^3)

$f_{bx,\,inner} = \pm 1.56(12000)/866$
$= \pm 22$ psi (0.15 MPa) (ok)

y-y Direction

$M_y = 0.0107wab^2$
$= (0.0107)(0.064 \times 1.3)(8)(27.5)^2$
$= 5.37$ ft-kips (7.28 kN-m) over a width equal to $0.50a = 48$ in. (1220 mm).

Section properties of 48 in. (1220 mm) width by a ratio to 96 in. (2440 mm) width values:

$I_{yy} = 2613\ (48/96)$
$= 1307$ in.^{4} (5.4×10^8 mm^4)

Adjust f_{pc} for only 10 percent losses at stripping.

$f_{pc,\,outer} = \pm(284)(0.9/0.85)$
$= \pm 301$ psi (2.08 MPa)

$f_{pc,\,inner} = (284)(0.9/0.85)$
$= 301$ psi (2.08 MPa)

$S_{yy,\,outer} = 1307/3.70$
$= 353$ in.^{3} (5.78×10^6 mm^3)

$f_{by,\,outer} = \pm 5.37(12000)/353 + 301$
$= +118$ psi or $+483$ psi (ok)
(0.81 MPa or 3.33 MPa)

$S_{yy,\,inner} = 1307/3.30$
$= 396$ in.^{3} (6.49×10^6 MPa)

$f_{by,\,inner} = \pm 5.37(12000)/396 + 301$
$= +138$ psi or $+464$ psi (ok)
(0.95 MPa or 3.20 MPa)

Yard Handling

Calculations are similar to stripping except that static multiplier is equal to 1.2 instead of 1.3. Therefore, the section is satisfactory by inspection.

Travel

Support on dunnage at same locations as above.
Static multiplier = 1.5

$f_{by,\,outer} = +90$ psi or $+512$ psi (ok)
(0.62 MPa or 3.53 MPa)

$f_{by,\,inner} = +113$ psi or $+489$ psi (ok)
(0.78 MPa or 3.37 MPa)

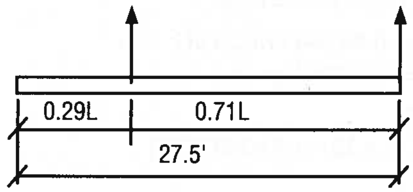


Fig. A12

Erection Stresses (see Fig. A12)

Use composite properties.
Try two-point pick
Static load multiplier = 1.2

$w = (0.064)(1.2)(8)$
$= 0.614$ kips per ft (8.95 kN/m)

$M^{\pm} = 0.044wl^2$
$= 0.044(0.616)(27.5)^2$
$= 20.5$ ft-kips (27.8 kN-m)
over full 8 ft (2.44 m) width

$M/S_{outside} = (20.5)(12)/706$
$= 0.348$ ksi (2.40 MPa)

$f_{pc} = 0.284$ ksi (1.96 MPa)

$f = -0.348 + 0.284$
$= -0.064$ ksi (0.44 MPa)
which is less than $5\sqrt{5000}/1000 = 0.354$ ksi (2.44 MPa)(ok)

EXAMPLE A5. COMPOSITE CLADDING PANEL

Design Criteria (see Fig. A13)

Wind load = 30 lbs per sq ft (1.44 kPa)
Differential temperature = 30°F (17°C)
$f'_c = 5000$ psi (34.5 MPa)
Strands:

Use 3/8 in. (9.5 mm) diameter, 270K, low-relaxation strands.
Minimum prestress = 225 psi (1.55 MPa) on gross section.

Section Properties

$A = (3)(96)(2)$
$= 576$ sq in. (3.72×10^5 mm^2)

$I = [(96)(3)^3/12 + (96)(3)(2.5)^2](2)$
$= 4032$ in.^{4} (1.68×10^9 mm^4)

$w = 600$ lbs per ft (8.76 kN/m)

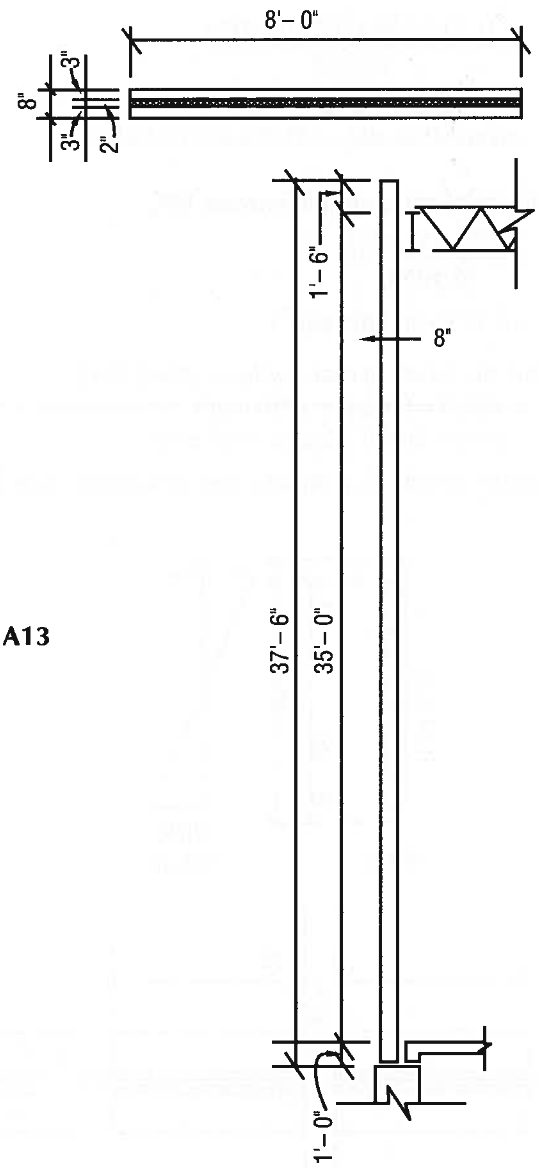


Fig. A13

Prestress

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

$$= \frac{(0.225)(576)}{(0.75)(270)(0.85)}$$

$$= 0.752 \text{ sq in. } (485 \text{ mm}^2)$$

No. of strands = 0.752/0.085 = 8.85

Try ten 3/8 in. (9.5 mm) diameter strands.

Use five 3/8 in. (9.5 mm) diameter strands in each wythe.

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

Analysis

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

At midheight:

Dead load and wind:

$P = (0.600)(36/2 + 1.5)$
$= 11.7$ kips (52 kN)

$M_w = (0.030)(8)(35)^2/8$
$= 36.8$ ft-kips (50 kN-m)

Thermal bow:

$\Delta_T = C(T_1 - T_2)l^2/8h$
$= (6 \times 10^{-6})(30)(35 \times 12)^2/(8 \times 8)$
$= 0.5$ in. (13 mm)

Initial bow:

Assume $\Delta_i = 0.50$ in. (13 mm) outward.

Determine EI from PCI Design Handbook, Fourth Edition.

$EI = \phi EI_g/(1 + \beta_d)$

$\phi = 0.70$ at $P_u = (0.1)(5)(576) = 288$ kips (1280 kN)

$\phi = 0.90$ at $P_u = 0$

For this example:

$P_u = 1.3P = 1.3(11.7) = 15.21$ kips (67.65 kN)

Thus, $\phi = 0.89$

$EI = (0.89)(4030)(4032)/(1 + 0)$
$= 1.45 \times 10^7$

in which $\beta_d = 0$.

Wind deflection:

$\Delta = (5/384)(wl^4/EI)$

$$\Delta_w = \frac{5}{384}\frac{0.240(35\times12)^4}{(1.45\times10^7)(12)}$$

$$= 0.56 \text{ in. } (14 \text{ mm})$$

Case 1: $U = 0.75\,(1.4D + 1.7L + 1.7W)$
$= 1.05D + 1.28W$

$P_u = 1.05(11.7)$
$= 12.3$ kips (54.7 kN)

in which $\beta_d = 1$ and $EI = 7.25 \times 10^6$ kips-in.^{2}

$M_u = (1.28)(36.8) + P\Delta$
$= 47.1 + P\Delta$

$\Delta_i + \Delta_w = 1.05(0.5) + 1.28(0.56)$
$= 1.24$ in. (31.5 mm)

***P*-Δ Analysis**

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

$$= \frac{(12.3)e(35\times12)^2}{(8)(7.25\times10^6)}$$

$$= 0.037e$$

First iteration:

$\Delta = 0.037(1.24)$
$= 0.046$ in. (1.2 mm)

Second iteration:

$\Delta = 0.037(1.24 + 0.046)$
$= 0.047$ in. (1.2 mm) (converges)

$M_u = 47.1 + 12.3(1.24 + 0.047)/12$
$= 48.4$ ft-kips (65.6 kN-m)

Case 2: $U = 1.4(D+T)$

$P_u = 11.7(1.4)$
$= 16.4$ kips (72.9 kN-m)

where $\beta_d = 1$ and $EI = 7.25 \times 10^6$

$\Delta_T + \Delta_i = 0.5 + 0.50$
$= 1.0$ in. (25.4 mm)

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

$$= \frac{(16.4)e(35 \times 12)^2}{(8)(7.25 \times 10^6)}$$

$$= 0.049e$$

First iteration:
$\Delta = 0.049(1.00) = 0.05$ in. (1.3 mm)

Second iteration:
$\Delta = 0.049(1.00 + 0.05)$
$= 0.05$ in. (1.3 mm) (converges)

$M_u = (16.4)(1.05)/12$
$= 1.44$ ft-kips (1.95 kN-m)

Therefore, the critical case is $1.4D + 1.28W$.

$P_u = 12.3$ kips (54.7 kN-m)

$M_u = 48.4$ ft-kips (65.6 kN-m)

From Fig. 2.6.5, PCI Design Handbook, Fourth Edition, the interaction capacities for an 8 in. (203 mm) panel section are adequate.

Check for cracking at mid-height of panel (use ultimate loads):
$P_u = 12.3$ kips (54.7 kN-m)
$M_u = 48.4$ ft-kips (65.6 kN-m)

$$f_{DL+wind} = \frac{(12.3)(1000)}{576} - \frac{(48.4)(1000)}{4032}$$

$$= -555 \text{ psi (3.83 MPa)}$$

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

$$= +254 \text{ psi (1.75 MPa)}$$

Net stress = -555 + 254 = -301 psi (2.08 MPa)

$f_r = 7.5\sqrt{f_c'}$
$= 7.5\sqrt{5000}$
$= 530$ psi (3.65 MPa)
greater than 301 psi (2.08 MPa)

Therefore, section is uncracked.

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

Determine M_n:

From Fig. 4.10.3, PCI Design Handbook, Fourth Edition:
$f_{ps} = 269$ ksi (1855 MPa)

$a = (5)(0.085)(269)/[(0.85)(5)(96)]$
$= 0.028$ in. (0.71 mm)

$d = 8 - 3/2 = 6.50$ in. (165 mm)

$\phi M_n = (0.9)(114.3)[6.5 - (0.028/2)]/12$
$= 55.6$ ft-kips (75.4 kN-m)

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

$= 79$ ft-kips (107 kN-m)
greater than $\phi M_n = 55$ ft-kips (75.4 kN-m)

Add mild reinforcing steel to increase ϕM_n.

$$A_{s,\,req.} = \frac{(79 - 55.6)(12)}{(6.5)(60)}$$

$= 0.72$ sq in. (465 mm^2)

Add four No. 4 bars in each wythe at strand level.

$A_{s,\,prov.} = 4(0.2) = 0.8$ sq in. (516 mm²)
greater than 0.72 sq in. (465 mm²)

Composite action of concrete and insulation (see Fig. A14):

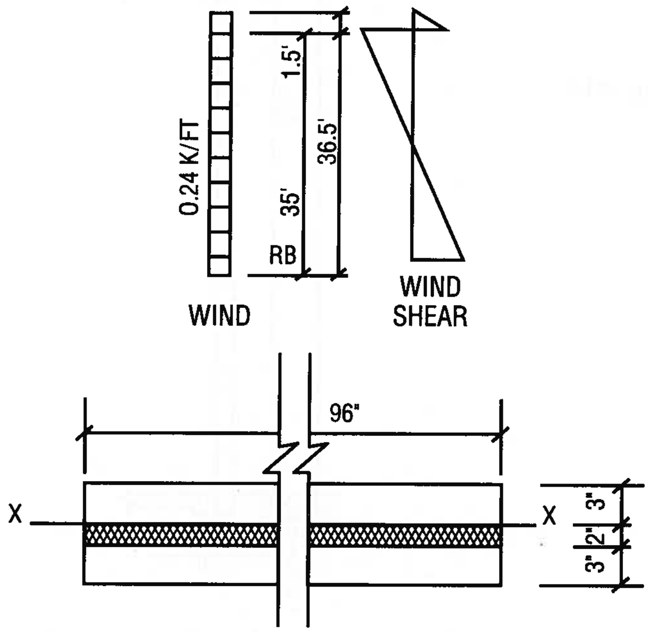


Fig. A14

$R_b = (0.24)(36.5)(16.75/35)$
$= 4.19$ kips (18.6 kN)

Case 3: $0.9D + 1.3W$
$D = 0$

$1.3W = 1.3(4.19)$
$= 5.45$ kips (24.2 kN)

$v = VQ/It$

$Q_{xx} = (96)(3)(1.5)$
$= 432$ in.³ (7.08 × 10⁶ mm³)

$$v_{h,xx} = \frac{(5450)(432)}{(4032)(96)}$$

$= 6.1$ psi (0.042 MPa)

or (6.1)(12)(96) = 7.03 kips per ft (103 kN/m)

Total shear = (7.03)(35)/(2 × 2)
= 61.5 kips (274 kN)

Strand force transfer:
$T_u = (5)(0.85)(270)$
= 114.75 kips (510 kN) [use this value - see following]

$C_u = (0.85)(5)(3)(96)$
$= 1224$ kips (54444 kN)

Provide wythe interconnectors to resist 114.75 kips (510 kN) ultimate in each half height of panel.

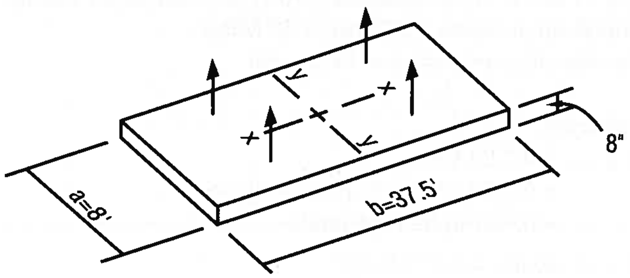


Fig. A15

Stripping (see Fig. A15)

$f'_{ci} = 3500$ psi (24 MPa)
$w = 75$ lb per sq ft (3.6 kPa)
$a = 8$ ft (2.44 m)
$b = 37.5$ ft (11.4 m)
Static load multiplier = 1.3
$w_i = (75)(1.3)$
$= 98$ lb per sq ft (4.7 kPa)

x-x Direction

$M_x = 0.0107wa^2b$
$= (0.0107)(0.098)(8)^2(37.5)$
$= 2.52$ ft-kips (3.42 kN-m)
over a width of $15t = 15(8) = 120$ in. (3048 mm)
$I_x = [(120)(3)^3/12 + (120)(3)(2.5)^2](2)$
$= 5040$ in.^{4} (2.10×10^9 mm^4)
$f_b = (2520)(12)(4)/5040$
$= 24$ psi (0.165 MPa)
$f_{pc} = 0$
Net stress = 24 - 0 = 24 psi (0.165 MPa)
$f'_r = 5\sqrt{f'_c}$
$= 5\sqrt{5000}$
$= 296$ psi (2.04 MPa) (ok)

y-y Direction

$M_y = 0.0107wab^2$
$= (0.0107)(0.098)(8)(37.5)^2$
$= 11.80$ ft-kips (16.0 kN-m)
over a width equal to $0.50a = 0.50(8)(12) = 48$ in. (1219 mm)
$I_{x,48} = [(48)(3)^3/12 + (48)(3)(2.5)^2](2)$
$= 2016$ in.^{4} (8.39×10^8 mm^4)
$f_{by} = -(11800)(12)(4)/2016$
$= -281$ psi (-1.94 MPa)
$f_{pc} = (254)(0.9)/0.85$
$= +269$ psi (1.85 MPa)
Net stress = -281 + 269 = -12 psi (-0.083 MPa)
Note that only 10 percent stress loss occurred at this stage.

Yard Handling

Use the same arrangement as shown above.
Static load multiplier = 1.2

Travel (see Fig. A16)

Static load multiplier = 1.5
$w = 75(1.5) = 113$ lbs per sq ft (5.41 kPa)
$f_{by} = (-281)(113)/98$
$= -324$ psi (-2.23 MPa)
$f_{pc} = +254$ psi (1.75 MPa)
Net stress = -324 + 254 = -70 psi (-0.48 MPa) (ok)

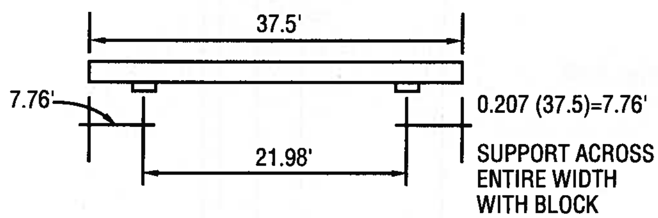


Fig. A16

Erection (see Fig. A17)

Refer to PCI Design Handbook, Fourth Edition.
Try two-point pick-up.
Static load multiplier = 1.2
$w = 75(1.2) = 90$ lb per sq ft (4.31 kPa)
or $(90)(8)/1000 = 0.72$ kips per ft (0.26 kN/m)

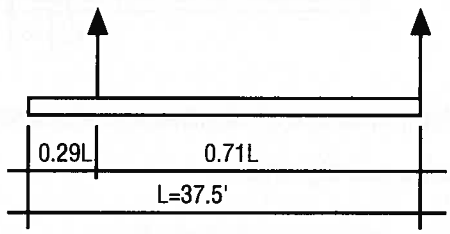


Fig. A17

$M^{\pm} = 0.044wL^2$
$= (0.044)(0.72)(37.5)^2$
$= 44.6$ ft-kips (60.5 kN-m)
Effective width = 96 in. (2438 mm)
$f_{by} = (44.55)(12000)(4)/4032$
$= -530$ psi (-3.65 MPa)
$f_{pc} = +254$ psi (1.75 MPa)
Net stress = -530 + 254 = -276 psi (-1.90 MPa)
which is less than $5\sqrt{5000} = 354$ psi (2.44 MPa) (ok)

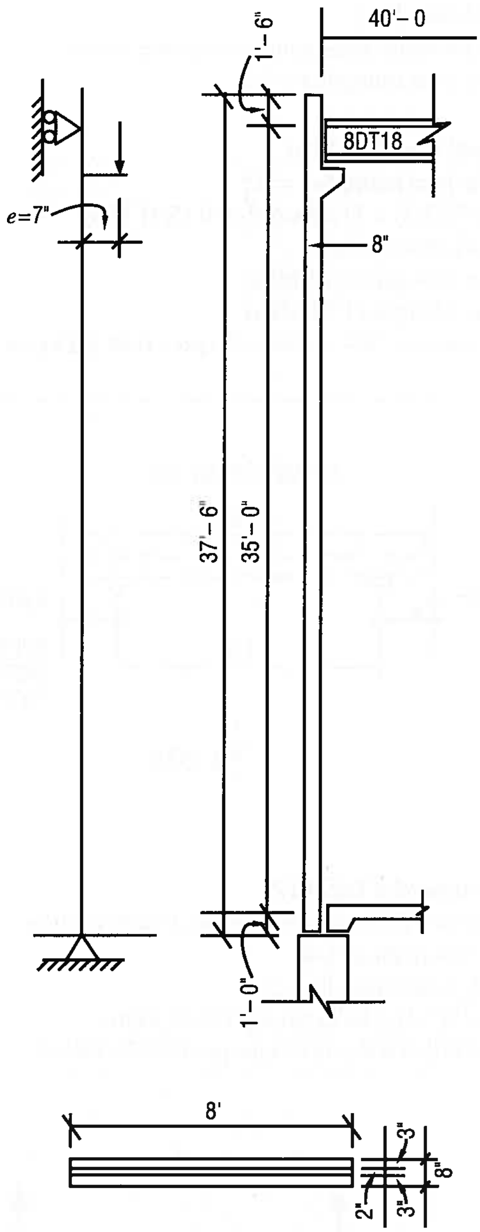


Fig. A18

EXAMPLE A6. COMPOSITE LOADBEARING PANEL

Design Criteria (see Fig. A18)

Wind load:
Direct pressure = 20 lbs per sq ft (0.96 kPa)
Suction pressure = 10 lbs per sq ft (0.48 kPa)
Differential temperature = 30°F (17°C)
Roof loads:
Double tee (8DT18) = 45 lbs per sq ft (2.15 kPa)
Roofing and mechanical = 10 lbs per sq ft (0.48 kPa)
Live load = 30 lbs per sq ft (1.44 kPa)
Roof reaction:
Dead load = (0.055)(20)(8)
= 8.8 kips (39 kN)
Live load = (0.030)(20)(8)
= 4.8 kips (21 kN)

Section Properties

$A = 576$ sq in. (3.72×10^5 mm²)
$I = 4032$ in.⁴ (1.68×10^9 mm⁴)
$S = 1008$ in.³ (1.65×10^6 mm³)
$w = 0.600$ kips per ft (8.76 kN/m)

Strands

Use 3/8 in. (9.5 mm) diameter, 270K, low-relaxation strands.
Minimum prestress = 225 psi (1.55 MPa)
Assume prestress losses = 15 percent

Prestress

$A_{ps,\,min.} = 0.225(A/P)$
$= (0.225)(576)/[(0.75)(270)(0.85)]$
$= 0.753$ sq in. (486 mm²)

No. of strands = 0.753/0.085
= 8.86 (use 10 strands)

Use five 3/8 in. (9.5 mm) diameter strands in each wythe.
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.

Analysis

Case 1: $U = 1.4D + 1.7L$
Deflection of panel at mid-height:

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

For details of this calculation, see PCI Design Handbook, Fourth Edition.
Load at top of panel:
$P_u = 1.4(8.8) + 1.7(4.8)$
$= 20.5$ kips (91 kN)
Panel weight at mid-height:
$P_u = 1.4[0.600(37.5/2)]$
$= 15.8$ kips (70 kN)
Total load at mid-height = 20.5 + 15.8 = 36.3 kips (161 kN)
$EI = \phi E_c I_g/(1 + \beta_d)$
where
$\beta_d = [15.8 + 1.4(8.8)]/36.3$
$= 0.77$
Therefore:
$EI = [(0.86)(4030)(4032)]/(1 + 0.77)$
$= 7.89 \times 10^6$ kips-in.²
Deflection due to load at top of panel:

$$\Delta = \frac{(20.5)(7)(35\times 12)^2}{(16)(7.89\times 10^6)}$$

$= 0.20$ in. (5.1 mm)
Assume initial bow = 0.50 in. (13 mm)
Total deflection = 0.20 + 0.50 = 0.70 in. (18 mm)

P-Δ Analysis

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

$$= \frac{(36.3)e(35\times 12)^2}{(8)(7.89\times 10^6)}$$

$= 0.10e$

First iteration:
$e = 0.70$
$\Delta = (0.70)(0.1) = 0.07$ in. (1.8 mm)

Second iteration:
$e = 0.70 + 0.07 = 0.77$ in. (20 mm)
$\Delta = (0.10)(0.77) = 0.08$ in. (2 mm)

Third iteration:
$e = 0.70 + 0.08 = 0.78$ in. (20 mm)
$\Delta = (0.10)(0.78) = 0.08$ in. (2 mm) (converges)

Moment at mid-height:
$M_u = (20.5)(7)/2 + (36.3)(0.76)$
$= 99.3$ in.-kips (11.2 kN-m)
$99.3/(8 \times 12) = 1.03$ ft-kips (1.40 kN-m)

Load at mid-height:
$P_u = 36.3$ kips (161 kN)
$36.3/8 = 4.53$ kips (20.1 kN)

From PCI Design Handbook, Fourth Edition, Fig. 2.6.5, the interaction diagram shows that the section is satisfactory.

Case 2: $U = 0.75(1.4D + 1.7L + 1.7W)$
For wind, use 10 lbs per sq ft (0.4 kPa) suction, which causes moments additive to moments due to dead and live load and eccentricity.

Load at top of panel:
$P_u = 0.75[1.4(8.8) + 1.7(4.8)]$
$= 15.4$ kips (68.5 kN)

Panel weight at mid-height:
$P_u = 0.75(15.8)$
$= 11.8$ kips (52.5 kN)

Total load at mid-height = 15.4 + 11.8 = 27.2 kips (121 kN)
$EI = \phi E_c I_g/(1 + \beta_d)$
where
$\beta_d = [11.8 + 0.75(1.4)(8.8)]/27.2$
$= 0.77$

Therefore, for $\beta_d = 0.77$, $EI = 7.89 \times 10^6$ kips-in.^{2} and for $\beta_d = 0$, $EI = 14.0 \times 10^6$ kips-in.^{2}

Deflection due to load at top:
$\Delta = 0.75(0.20) = 0.15$ in. (3.8 mm)

Deflection due to wind:

$$\Delta = \frac{5}{384}\frac{wL^4}{EI}$$

$$= \frac{5}{384}\left[\frac{(0.010\times 8)(35\times 12)^4}{(12)(14.0\times 10^6)}\right]$$

$= 0.19$ in. (4.8 mm)

Assume initial bow = 0.5 in. (13 mm)
Total deflection = 0.15 + 0.19 + 0.50 = 0.84 in. (21 mm)

P-Δ Analysis

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

$$= \frac{(27.2)e(35\times 12)^2}{(8)(7.89\times 10^6)}$$

$= 0.076e$

First iteration:
$e = 0.84$
$\Delta = (0.076)(0.84) = 0.06$ in. (1.5 mm)

Second iteration:
$e = 0.90$
$\Delta = (0.076)(0.90) = 0.07$ in. (1.8 mm)

Third iteration:
$e = 0.91$
$\Delta = (0.076)(0.91) = 0.07$ in. (1.8 mm) (converges)

$M_w = (0.01)(8)(35 \times 12)^2/(12 \times 8)$
$= 147$ in.-kips (16.6 kN-m)
$M_{u_w} = 147 \times 1.28 = 188$ in.-kips (21.2 kN-m)
$M_{u_{P_u}} = (15.4 \times 7)/2 + (27.2)(0.91)$
$= 79$ in.-kips (8.9 kN-m)

Total moment at mid-height:
$M_u = 188 + 79 = 267$ in.-kips (30 kN-m)
$267/(8 \times 12) = 2.77$ ft-kips per ft (12.3 kN)

Load at mid-height:
$27.2/8 = 3.40$ kips (15.1 kN)

From PCI Design Handbook, Fourth Edition, Fig. 2.6.5, the interaction diagram is satisfactory.

Case 3: $U = 0.75(1.4D + 1.7L + 1.4T)$

Check thermal bow (Δ_T).
EI is the same as in Case 2.

$\Delta_T = C(T_1 - T_2)(l^2)/8h$
$= (6 \times 10^{-6})(30)(35 \times 12)^2/[(8)(8)]$
$= 0.5$ in. (13 mm)

By inspection, the *P-Δ* analysis will converge and the moments will be less than in Case 2. Therefore, the section is satisfactory.

Check for cracking:
Use ultimate loads:
$P_u = 27.2$ kips (121 kN)
$M_u = 267$ in.-kips (30 kN-m)

Case 2: $D + L + W$ at Midheight
$f = (27.2)(10^3)/576 - (267)(10^3)/1008$
$= -218$ psi (-151 MPa)

$f_{pc} = 254$ psi (1.75 MPa)

Net stress = -218 + 254 = +36 psi (0.248 MPa) compression
Therefore, stresses are satisfactory.

Case 4: $D + L$ at Section A-A (see Fig. A19)
$H_{D_u} = (8.8 \text{ x}7)/(35 \times 12)(1.4)$
$= 0.147$ kips $\times 1.4$
$= 0.206$ kips (0.92 kN)

$H_{L_u} = (4.8 \times 7)/(35 \times 12)(1.7)$
$= 0.080$ kips $\times 1.7$
$= 0.136$ kips (0.60 kN)

Total load = 0.206 + 0.136 = 0.342 kips (1.52 kN)

Moment at Section A-A:
$M_u = 20.5$ kips $\times$ 7 in. - 0.342 kips $\times$ 27 in.
$= 143 - 9 = 134$ kip-in. (15.1 kN-m)
$f = (20.5 \times 10^3)/576 - (134 \times 10^3)/1008$
$= -97$ psi (0.67 MPa)

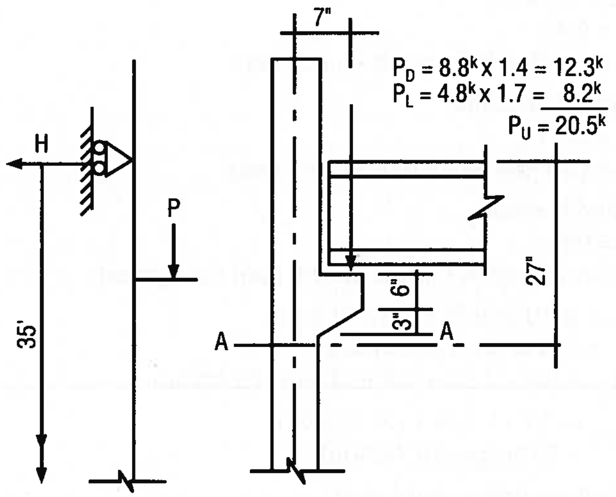


Fig. A19

f_{pc} = +254 psi (1.75 MPa)

Net stress = -97 + 254 = 157 psi (1.08 MPa) (compression) (stress is satisfactory)

Horizontal Shear at Interface of Concrete and Insulation

Strand force transfer will be critical.

T_u = (5)(0.085)(270)
= 114.8 kips (511 kN)

C_u = (0.085)(5)(3)(96)
= 1224 kips (5444 kN)

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

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

See Example A5. Add four No. 4 reinforcing bars in each wythe at strand level.

Handling and Erection

See Composite Cladding Panel (Example A5).

NOTATION

a = depth of equivalent rectangular stress block
a = width of panel being stripped
A = area of concrete at cross section considered
A_b = area of reinforcing bar or stud
A_{cr} = area of crack interface
A_{ps} = area of prestressed steel in tension zone
A_s = area of mild steel reinforcement
A_{vf} = area of shear-friction reinforcement
b = width of compression face of member
b = length of panel being stripped
c = distance from extreme fiber to neutral axis
C = resultant compressive force
C = coefficient of thermal expansion
C_u = factored compressive force
C_w = stud group adjustment factor
d_p = distance from compression fiber to centroid of prestressed reinforcement
D = dead load
e = eccentricity of design load or prestressing force measured from centroid of section
E_c = modulus of elasticity of concrete
EI = flexural stiffness of compression member
f_a = unit stress of structural steel
f_b = bending stress due to stripping; subscript denotes direction
f'_c = specified compressive strength of concrete
f'_{ci} = concrete compressive strength at time considered
f_{pc} = compressive stress in concrete at centroid of cross section due to prestress (after allowance for all prestress losses)
f_{ps} = stress in prestressed reinforcement
f_{pu} = specified tensile strength of prestressing steel
f_w = resultant stress on weld
F_a = allowable bending stress of structural steel
F_h = resultant horizontal shear force
F_w = design strength of weld
F_y = yield strength of structural steel
h = total depth of section
H = horizontal force
I = moment of inertia of section resisting external loads
I_g = moment of inertia of gross section
I_{xx}, I_{yy} = moment of inertia of weld group with respect to its own x and y axes, respectively
I_p = polar moment of inertia
K'_u = coefficient = $M_u(12{,}000)/bd_p^2$
l = total span length
l_d = development length
l_e = embedment length
l_w = length of weld
L = live load
M = unfactored service load moment
M_{cr} = cracking moment
M_n = nominal moment strength at section
M_u = factored moment due to applied loads
M_x = moment due to stripping with respect to x-axis
M_y = moment due to stripping with respect to y-axis
P = applied axial load
P_c = nominal tensile strength of concrete element
P_n = nominal axial load capacity
P_s = nominal tensile strength of steel element
P_u = factored applied axial load
q = load per unit area
Q = statical moment about neutral axis
r = radius of gyration at cross section of a compression member
R = fire endurance of composite assembly
R_1, R_2, R_3 = fire endurance of individual course
R_{DL} = reaction due to dead load
R_u = reaction due to total factored load
S = section modulus
S_w = section modulus of weld group
t = thickness of section
T = resultant tensile force
T = temperature
T_u = factored tensile force
U = required strength to resist factored loads
v = shear force per unit
V = total applied shear force
V_c = nominal shear strength of concrete element
V_{nh} = nominal horizontal shear at plane considered
V_s = nominal shear strength of steel element
V_u = factored applied shear force
w = uniform load
W = wind load
x = overall dimension of a stud group
x = distance to centroid of a weld group
y = distance from centroid of individual area to centroid of gross section
y = overall dimension of a stud group
y = distance from centroid to fiber considered
y = distance to centroid of a weld group
$\bar{y}$ = distance from one surface to centroid of section
Z = with subscript; plastic section modulus
α = with subscript; modification for development length
α = strain gradient across thickness of wall panel
β_d = ratio of factored axial dead load to factored axial total load
Δ = deflection
Δ_{temp} = thermal bow in wall panel
μ_e = effective shear-friction coefficient
ρ = A_s/bd = ratio of non-prestressed reinforcement
ρ_p = A_{ps}/bd_p = ratio of prestressed reinforcement
ϕ = strength reduction factor
ω_{pu} = $\rho_p f_{ps}/f'_c$