Appendix D: Precast Structural Calculations

Basic Wind Speed (3-Second Gust) : 90 MPH

Wind Importance Factor : 1.15

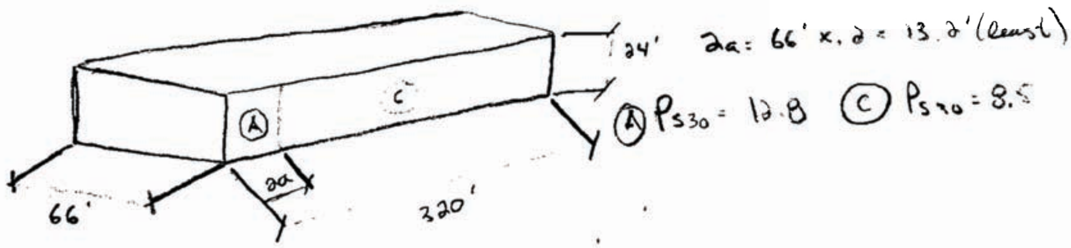
Exposure Category : B

Internal Pressure Coefficient : +/- .18

Components and Cladding per Code Requirements (Values listed based on 100 sq. ft effective wind area)

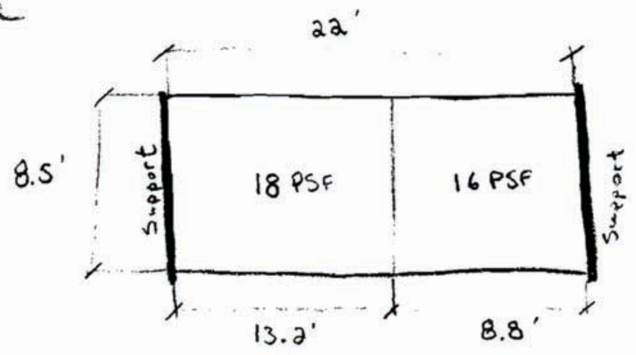
a. Interior Zone : 16 PSF

b. Exterior Zone & Corner Zone : 18 PSF



An end zone panel in zone Ⓐ will be analyzed for windload as it has the longest span and the greatest wind pressure.

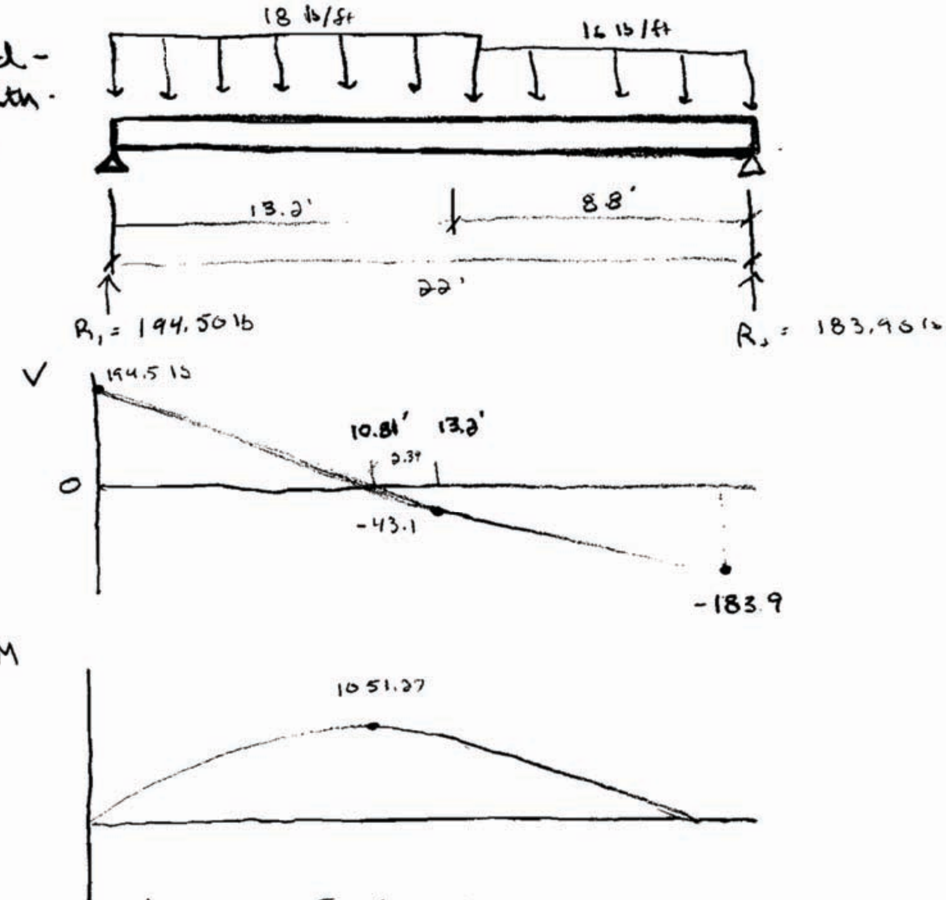
Panel



18 PSF × 1' section = 18 lb/ft

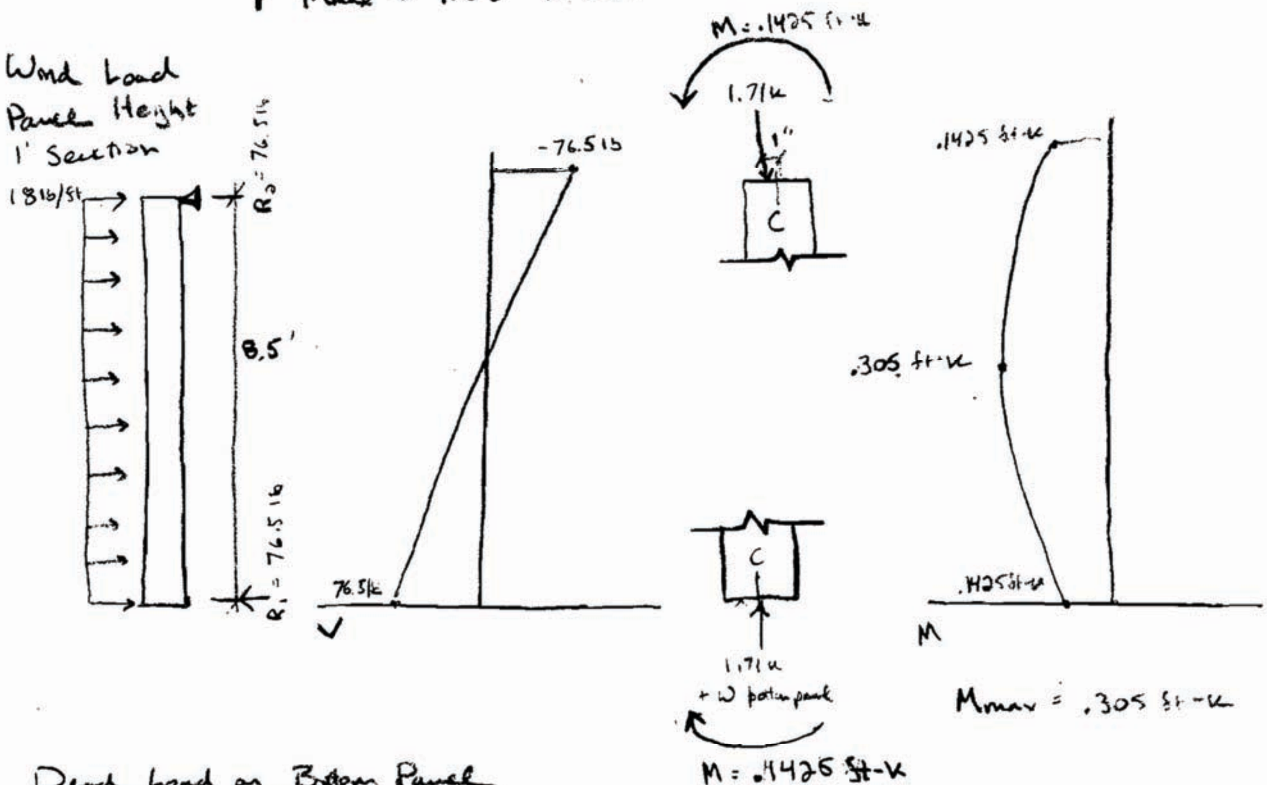
16 PSF × 1' section = 16 lb/ft

Wind Load -
Panel Width -
1' Section



M_{max} = 1.05 ft-kips

Wind Load
Panel Height
1' Section



M_{max} = .305 ft-k

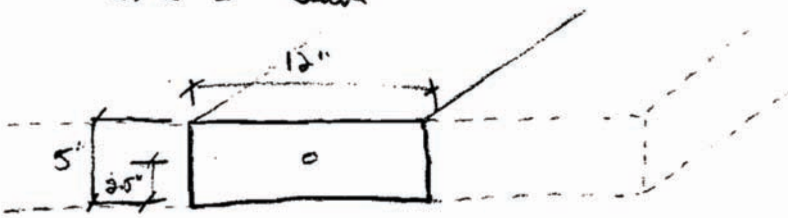
Dead Load on Bottom Panel

17' x 22' x .67' x .150 kips/ft^3 = 37.6 kips

37.6 kips / 22' = 1.71 kips / ft

Reaction @ Panel base = 1.71 + [(8.5)(.67')(.150)] = 2.56 kips

Flexure Check
of Panel Height (8.5')
as a 1' beam



$f'c = 5000 \text{ psi}$ $\beta_1 = .80$ $M_u = .305 \text{ ft-k}$ $\phi = .9$
$= 3.66 \text{ in-k}$
$d = 2.5"$ $h = 5"$ $b = 12"$ f_y steel $= 60 \text{ ksi}$

Minimum Steel

$$A_s \geq \frac{3\sqrt{f'c}}{f_y} bd = \frac{3\sqrt{5000}}{60 \times 1000}(2.5)(12) = .106 \text{ in}^2$$

$$\geq \frac{200\, bd}{f_y} = \frac{200\,(2.5)(12)}{60 \times 1000} = .1 \text{ in}^2$$

ϕM_n * use # 4 bars @ 1' - $A_s = .20 \text{ in}^2$

$$M_n = A_s f_y (d - a/2)$$

$$a = \frac{A_s f_y}{.85 f'c\, b} = \frac{.20\,(60 \text{ ksi})}{.85\,(5 \text{ ksi})(12")} = .236"$$

$$M_n = (.20)(60)(2.5 - .236/2) = 28.59 \text{ "K}$$

$$= 2.38 \text{ 'K}$$

$$\boxed{\phi M_n = 2.14 \text{ 'K}}$$

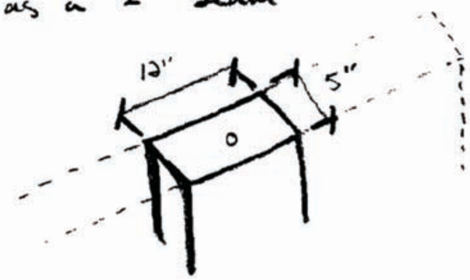
$= 25.68$ in-kips

Minimum thickness for Deflection

simply supported solid one-way slabs

$$\ell/20 = 8.5(12)/20 = 5.1"$$

Compressive Design
at Panel Height
as a 1' beam



$f'_c = 5000\,psi$ $\beta_1 = .80$ $\phi = .65$

$h = 5"$ $b = 12"$ f_y steel $= 60ksi$ $P_u = 2.56$ kips

$$P_n = .85 f'_c A_c + A_s f_y$$

$A_s = .20\,in^2$ (from flexural design at panel height)

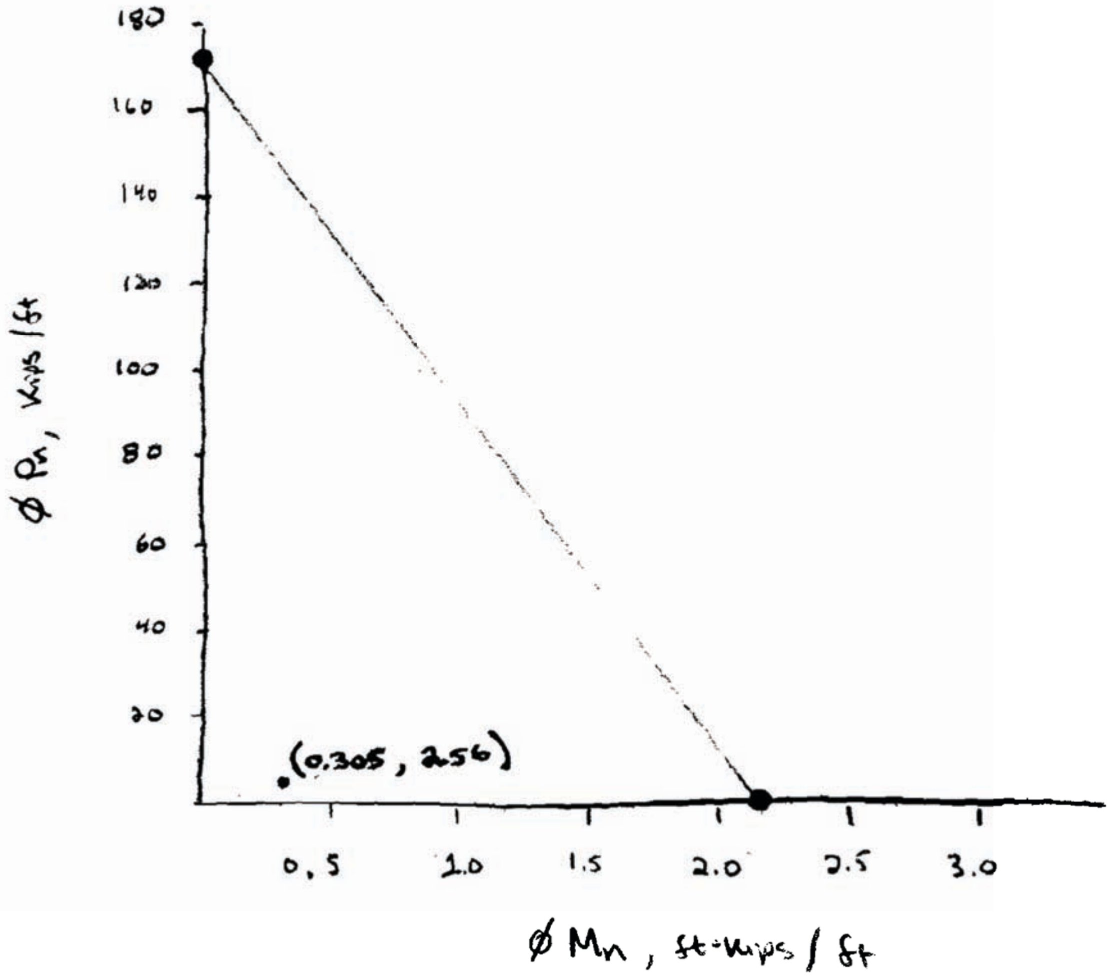
$A_c = (12" \times 5") - A_s = 59.8\,in^2$

$$P_n = .85\,(5\,ksi)\,59.8\,in^2 + .20\,in^2\,(60\,ksi)$$

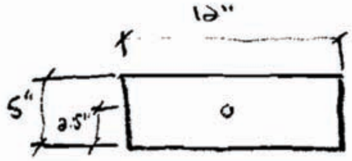
$$= 266 \text{ kips}$$

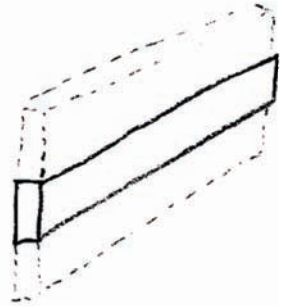
$$\boxed{\phi P_n = 173 \text{ kips}}$$

Interaction Curve for Precast, Reinforced Concrete Wall Panel



Flexural Design
of Panel Width (22')
as a 1' beam





$f'c = 5000 psi$ $\beta_1 = .80$ $M_u = 1.05$ ft-k

$d = 2.5"$ $h = 5"$ $b = 12"$ f_y steel = 60ksi

<u>Minimum Steel</u>

From Flexural Design of Panel width (same parameters)

$A_s \geq .106 \text{ in}^2$

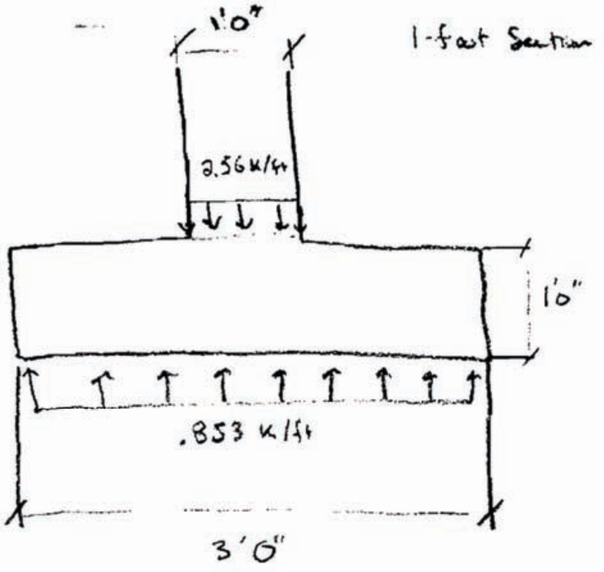
* use # 4 bars @ 1' - $A_s = .20 \text{ in}^2$

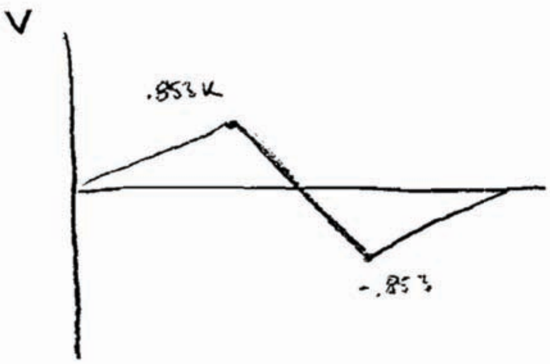
<u>ϕM_n</u>

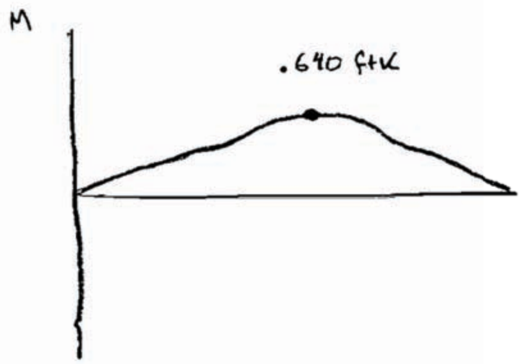
From Flexural Design of Panel Width (same parameters)

$\boxed{\phi M_n = 2.14 \text{ 'k}}$

Strip Footing Design Check







Shear

$$V_c = 2\sqrt{f'c}\ bd$$

$$= 2\sqrt{3000}\ (12")(12")$$

$$= 15.8k$$

$$(.5)\phi V_n = (.75)(15.8k)(.5)$$

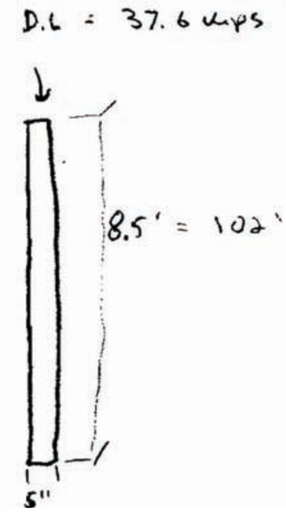
$$= 5.92K$$

$$V_u = .853K \le \tfrac{1}{2}\phi V_n = 5.92K \therefore OK$$

Second - Order Analysis of an Uncracked - Member

5in. thick , 20 ft wide wall panel

<u>Assumptions</u>



Concrete : $f_c' = 5000$ psi
$E_c = 4300$ ksi

$P_u = 1.2D + 1.6L + .8W = 1.2(37.6) = 45.12$ kips
$I_g = bh^3/12 = 240(5)^3/12 = 2500 \text{ in.}^4$
Additional wind load = 16 psf

$w_u = 16(22')(.8) = 282$ lb/ft

$$EI_{eff} = \frac{.70 E_c I_g}{1+\beta_d} = \frac{.70(4300)(2500 \text{ in}^4)}{1.0} = 7.53 \times 10^6 \text{ kip-in.}^2$$

Deflection due to wind

$$\frac{5 w_u l^4}{384 EI} = \frac{5\left(\frac{.282}{12}\right)(102)^4}{384(7.53 \times 10^6)} = .004 \text{ in}$$

Deflection due to $P_u e$

$$\Delta_i = \frac{P_u e l^2}{16 EI} = \frac{45.12(1)(102)^2}{16(7.53 \times 10^6)} = .004 \text{in}$$

Initial midspan bow including eccentricity and wind

$$e = 1.0 + .004 + .004 = 1.008$$

Deflection due to P-Δ moment at midspan

$$\Delta = \frac{Pel^2}{8EI} = \frac{(45.12)e(102)^2}{8(7.53 \times 10^6)} = .008e$$

First Iteration

$$\Delta = .008(1.008) = .008$$

Second

$$e = 1.008 + .008 = 1.016$$

$$\Delta = .008(1.016) = .008 \text{ (convergence)}$$

$$M_u = \frac{45.12(1)}{2} + 45.12\,(1.016) + \frac{\left(\frac{.287}{12}\right)(102)^2}{8} = 99 \text{ kip-in.}$$

$$M_u y / I = 99\,(2.5)(1000)\,/\,2500 = -99 \text{ psi}$$

Half Panel wt : $[100\,(4.25)\,/\,5\,(12)]\,(1.2) = 8.5$ psi

Dead Load : $37.6\,(1000)(1.2)\,/\,[5\,(240)] = 37.6$ psi

Net Stress (tension) : -52.9 psi

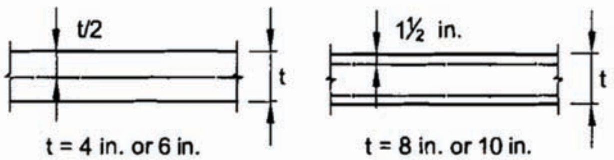
$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 530 \text{ psi} > 52.9 \text{ psi}$$

Therefore, analysis is valid

$$P_u = 45.12/22 = 2.05 \text{ kips/ft}$$
$$M_u = 99/[(12)(22)] = .375 \text{ kip-ft/ft}$$

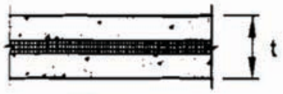
Point is below interaction curve for 5" partially developed strand ∴ OK

Figure 2.7.6 Partial interaction curves for precast, reinforced concrete wall panels

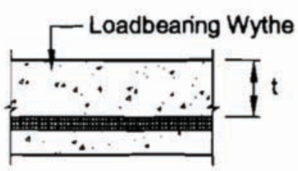


t, in.	Full Interaction Curve Data			
	ϕP_c	ϕP_{nb}	ϕM_{nb}	ϕM_c
4	134	67	5.5	0.5
6	202	100	12.4	1.0
8	269	131	22.6	1.8
10	336	164	35.5	2.8

Curves Based on Minimum
Vertical Reinforcement $\rho = 0.10\%$
$f'_c = 5000$ psi; $f_y = 60{,}000$ psi



Fully Composite Sandwich Panel



Partially Composite or Non-Composite