APPENDIX
CHAPTER 2

SEISMIC WEIGHT

- Total Dead Load
- 25% Live Load from storage
- Partition Loads ≥ 10 psf
- Perminent Equipment
- 20% Flat Roof Snow ≥ 30 psf

· Wall / Parapet DL

- Level 2-5 Wall = 35psf
- Level 1 Wall = 15 psf
- Elev. Tower Wall = 30psf
- Roof screen wall = 40 psf
- Typical Parapit = 260 plf

· Typical Slab Area = 13,910 SF / floor

· Column / SW SF = $8(24" \times 24") + 20(20" \times 20") + 2[2 \times 8' + 2 \times 18'] + 2[(16/12) \times 10' + (16/12) \times 10'] = 204$ SF

· Elevator Tower = $210\ ft^2 (51 psf) + 2(21.6' + 9.6')(6/2)(30psf) = 16,326$ lbs

· $W_R = 13,910 ft^2 (100 psf) + 2(26' + 26')(14.33'/2)(7psf) + 204\ ft^2 (14.33'/2)(150psf) + 2(244' + 61')(15/2)(35psf) + 260plf (244' + 244'/2 + 61') + 15(8')(48' + 128')(2)$
$= 1,923^k$

· $W_5 = 13,910\ ft^2 (100psf) + 204[12.67'/2 + 14.33'/2](150 pcf) + (243' + (2)61')(35psf)(12.67/2 + 14.33'/2)$
$= 1,977^k$

· $W_{4/3} = 13,910\ ft^2 (100 psf) + 204\ ft^2 (12.67')(150 pcf) + (243' + (2)(61'))(35 psf)(13.33')$
$= 1,948^k$

· $W_2 = 13,910\ ft^2 (100 psf) + 204\ ft^2 (12.67')(150pcf) + (243' + (2)61')(15psf)(12.69'/2 + 18'/2)$
$= 1,940^k$

$W_{T\ TOTAL} = 9,762^k$

SEISMIC CALCULATIONS

- Solve for C_s

  - Building Ht = 74'-4"
  - $I_e \Rightarrow$ II, $I = 1.0$

$$C_s = \text{MIN} \begin{cases} S_{DS} / (R/I) \\ S_{D1} / [T\,(R/I)] \\ S_{D1} \cdot T_L / [T^2 (R/I)] \end{cases} \geq 0.01$$

  - LAT/LONG = -77.008, 38.795

    $\Rightarrow S_s = 0.177g \quad S_1 = 0.063g$

  - $F_a = 1.6$, $F_v = 2.4$

  - $S_{MS} = F_a S_s = 1.6(.177) = 0.2832g$
    $S_{M1} = F_v S_1 = 2.4(.063) = 0.1512g$

  - $S_{DS} = 2/3\, S_{MS} = 2/3\,(.2832) = 0.1888g$
    $S_{D1} = 2/3\, S_{M1} = 2/3\,(.1512) = 0.101g$

  - Both directions use same lateral system type:

    - Ordinary Reinforced Concrete Shear Walls
      - $R = 4.0$, $C_d = 4.0$

      $C_t = .02$, $x = .75$ (All other structural sys.)

C_u =	.1	.101	.15
	1.7	$\boxed{1.698}$	1.6

      $T_a = .02\,(74.33)^{.75} = 0.506$ sec
      $C_u T_a = 1.698(.506) = .859$ sec
      $T_b = 0.6678$sec (From Model)

  - $C_s \geq$ $.189/(4/1) = 0.0473$

    $.101/.6678(4/1) = 0.0378$ ← CONTROLS

    $.101(8)/.6678^2(4/1) = 0.4230$

    $V_b = C_s W_t = 0.0378\,(9762^k) = 369^k$

SEISMIC LOAD DISTRIBUTION

- $K = 1.08$

$$T = \quad 0.5 \qquad .6678 \qquad 2.5$$

$$K = \quad 1 \qquad \boxed{1.08} \qquad 2$$

- LONG. DISTRIBUTION:

$$C_{VR} = \frac{(16.3^k + 1{,}923^k)(74.33)^{1.08}}{(16.3+1{,}923)(74.33)^{1.08} + 1977(59)^{1.08} + 1948(45.67)^{1.08} + 1948(32.33)^{1.08} + 1940(19)^{1.08}}$$

$$= .329$$

$$C_{V5} = \frac{1977(59)^{1.08}}{'' \quad ''} = .263$$

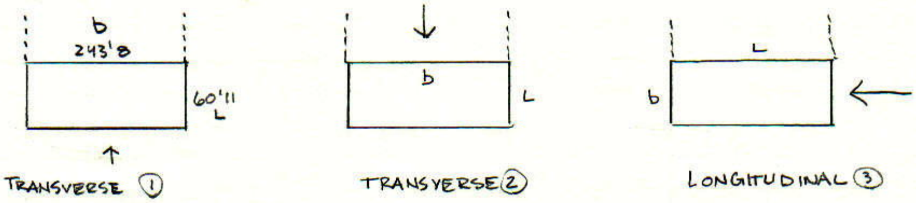
$$C_{V4} = \frac{1948(45.67)^{1.08}}{'' \quad ''} = .197$$

$$C_{V3} = \frac{1948(32.33)^{1.08}}{'' \quad ''} = .135$$

$$C_{V2} = \frac{1940(19)^{1.08}}{'' \quad ''} = .076$$

$F_R = .329\ (369) = 121.7^k$
$F_5 = .263\ (369) = 97.3^k$
$F_4 = .197\ (369) = 72.9^k$
$F_3 = .135\ (369) = 50.0^k$
$F_2 = .076\ (369) = 28.1^k$

WIND CALCULATIONS:



$$p = q\,G\,C_p - q_i\cancelto{0}{(GC_{pi})}$$

$V = 90$ MPH
$K_{zt} = 1.0$
$1/T_a = 1/.851 = 1.17 > 1 \Rightarrow$ RIGID (LONG.) , $1/.752 = 1.33 \Rightarrow$ RIGID (TRANSV.)
EXPOSURE D
IMPORTANCE $= 1.0$

GUST FACTOR (G):

$$G = 0.925\left(\frac{(1+1.7\,g_Q\,I_z\,Q)}{(1+1.7g_v\,I_z)}\right) \qquad I_z = c\left(\frac{33}{z}\right)^{1/6}$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_z}\right)^{.63}}} \qquad L_z = \ell\left(\frac{z}{33}\right)^{\bar{\epsilon}}$$

$z = .6h = 44'$, $\ell = 650$, $\bar{\epsilon} = 1/8.0$, $g_Q = g_v = 3.4$, $c = .15$

$$L_z = 650\left(\frac{.6(73.33')}{33}\right)^{1/8} = 673.8 \qquad I_z = .15\left(\frac{33}{44}\right)^{1/6} = .1430$$

$$Q_1 = \sqrt{\frac{1}{1+.63\left(\frac{243.7+73.33}{673.8}\right)^{.63}}} = \boxed{.848}, \quad Q_3 = \quad 243.7 \rightarrow 60.92 = \boxed{.902}$$

$$G_1 = .925\left(\frac{1+1.7(3.4)(.1430)(.848)}{1+1.7(3.4)(.1430)}\right) = \boxed{.861}$$

$$G_3 = .848 \rightarrow .902 \qquad = \boxed{.884}$$

C_p: WALL PRESSURE ⇒ WINDWARD = .80

$L/B = .25$ LEEWARD$_1$ = −.50

$L/B = 4$ LEEWARD$_2$ = −.20

$q_z = 0.00256\, K_z\, K_{zt}^{\to 1}\, K_D^{\to .85}\, V^{2\,\to 90^2}\, I^{\to 1}$

K_z VARIES

$K_D = .85$

	K_z	q_z
0-19'	1.08	19.04
19'-32'4"	1.22	21.50
32'4"-45'8"	1.27	22.38
45'8"-59'	1.31	23.09
59'-74'	1.38	24.32

CASE 1 (E-W) / CASE 2 (W-E)

	CASE 1 WINDWARD (P)	CASE 1 LEEWARD (P)	CASE 2 WW (P)	CASE 2 LW (P)
0-19	13.11	0	0	−10.47
19-32'4	14.81	0	0	−10.47
32'4-45'8	15.42	0	0	−10.47
45'8-59	15.91	0	0	−10.47
59'-74	16.75	0	0	−10.47

CASE 3 (N-S)

	WW (P)	LW (P)	TOTAL (P)
0-19	13.47	−4.30	17.77
19-32'4	15.20	−4.30	19.50
32'4-45'8	15.83	−4.30	20.13
45'8-59	16.33	−4.30	20.63
59-74	17.20	−4.30	21.50

* CASE 4 (E-W) NEGLECTING ADDITIONAL BUILDING

	WW (P)	LW (P)	TOTAL (P) (psf)
0-19	13.11	−10.47	23.58
19-32'4	14.81	−10.47	25.28
32'4-45'8	15.42	−10.47	25.89
45'8-59'	15.91	−10.47	26.38
59'-74'	16.75	−10.47	27.22

· WIND BASE SHEAR:

- TRANSVERSE EW

$16.8\,psf(243.67')(14.33') = 58,660\ lbs$
$+15.9\,psf(243.67')(13.33') = +51,650$
$+15.4\,psf(243.67')(13.33') = +50,020\ lbs$
$+14.8\,psf(243.67')(13.33') = +48,070\ lbs$
$+13.1\,psf(243.67')(19') = +60,650\ lbs$
$269,050 = 269^k$ ← CONTROLS

- TRANSVERSE W-E

$243.67'(73.33')(10.5\,psf)$

$= 181,620\ lbs = 182^k$

- LONGITUDINAL N-S/S-N

$21.5\,psf(60.92')(14.33') = 18,770\ lbs$
$+20.6\,psf(60.92')(13.33') = 16,730\ lbs$
$+20.1\,psf(60.92')(13.33') = 16,320\ lbs$
$+19.5\,psf(60.92')(13.33') = 15,840\ lbs$
$+17.8\,psf(60.92')(19') = 20,600\ lbs$
$88,260\ lbs = 88^k$

- TRANSVERSE REVISED (EW)

$27.22\,psf(243.67')(14.33') = 95,050\ lbs$
$+26.38\,psf(243.67')(13.33') = +85,685\ lbs$
$+25.89\,psf(243.67')(13.33') = +84,090\ lbs$
$+25.28\,psf(243.67')(13.33') = +82,110\ lbs$
$+23.58\,psf(243.67')(19') = +109,169\ lbs$
$456,104\ lbs = 456^k$

· CONROLLING TRANSVERSE STORY LOADING:

· ROOF: $27.22\,psf(243.67')(14.33'/2) = 47.5^k$
· 5TH FLR: $27.22\,psf(243.67')(14.33'/2) + 26.38\,psf(243.67')(13.33'/2) = 90.4^k$
· 4TH FLR: $26.38\,psf(243.67')(13.33'/2) + 25.89\,psf(243.67')(13.33'/2) = 84.9^k$
· 3RD FLR: $25.89\,psf(243.67')(13.33'/2) + 25.28\,psf(243.67')(13.37'/2) = 83.1^k$
· 2ND FLR: $25.28\,psf(243.67')(13.33'/2) + 23.58\,psf(243.67')(19'/2) = 95.6^k$

$= 401.5^k$

POST - TENSIONED CHECK

$W_o = (8/12)150 = 100psf$, $W_D = 25psf \Rightarrow$ Balanced Load = 125psf

$$P_{eT} = \frac{.6(125)(30.5)^2(12)}{8(4.125)} = 25,370 \text{ lb/width}$$

$$P_{eL} = \frac{.4(125)(30)^2(12)}{8(4.125)} = 16,364 \text{ lb/ft width}$$

Initial Prestress:

$P_{iT} = 25,370/.85 = 29,847$ lb/ft $\quad P_{iL} = 16,364/.85 = 19,252$ lb/ft

Tendons: 0.6", $A_p = .215$, Grade 250 $\Rightarrow$ $P_{i\,allow} = .7(250)(.215) = 37.6^k$
↑ force in one tendon.

Transverse: 4 tendons every 5' => 1 tendon every 1.25'

$29,847$ lb/ft $(1.25') = 37,308$ lb/tendon $< 37,600$ ok

Longitudinal @ Center: 14 + 3 tendons every 30.25' => 1 tendon every .56'

$19,252(.56) = 10,934$ lb/tendon $< 37,600$ ok

Longitudinal @ Edge: 10 tendons + 3 every $\frac{70.25}{2}$' => 1 tendon every .86
$= 16,557$ lb/tendon $< 37,600$
ok

Service Load Stresses

$$f_{eT} = \frac{-25,370}{12(8)} = -264 \text{ psi} \qquad f_{eL} = \frac{-16,364}{12(8)} = -170 \text{ psi}$$

Unbalanced Moment Stresses: $M = l_a/l_b = 30.5/30 = 1.02 \approx 1.0$

Positive Moments:
T $M_a^+ = .032(100)(30.5)^2 = 2977$ lb
L $M_b^+ = .032(100)(30.5)^2 = 2880$ lb
Negative Moments: T $M_a^- = .05(100)(30.5)^2 = 4651$ lb
L $M_b^- = .05(100)(30.5)^2 = 4500$ lb

Moment of Inertia of 12" strip of slab

$$I = \frac{12 \times 8^3}{12} = 512 \text{ in}^4 \qquad f = \frac{Mc}{I} = \frac{M \times 12 \times 4''}{512} = 0.09375 M \text{ in ft-lb}$$

f_a due to $M_a^+ = 2977(.09375) = 279.1$ psi
f_b due to $M_b^+ = 2880(.09375) = 270$ psi
f_a due to $M_a^- = 4651(.09375) = 436$ psi
f_b due to $M_b^- = 4500(.09375) = 422$ psi

- Allowable Stresses After Losses:

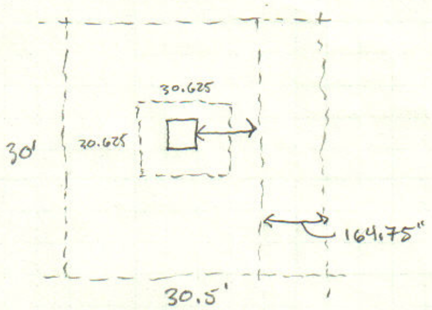
$f_{c\,allow} = 0.45 f'_c = .45(5000) = 2250 \text{ psi } C$

$f_{t\,allow} = 6\sqrt{5000} = 424 \text{ psi } T$

PUNCHING SHEAR

~~davg = 24.75"~~ , assume .75" cover   davg = 8 - .75 - .625 = 6.625

col = 24×24 $W_u = 1.2(125) + 1.6(100) = 310\ psf = .310\ ksf$



$\left(\frac{30.5' \times 12}{2}\right) - 12" - 6.25 = 164.75"$

WIDEBEAM SHEAR:

$V_u = l_2' l_1 W_u = 30'\left(\frac{164.75"}{12}\right)(.310\ ksf) = 127.68^k$

$V_c = 2\sqrt{f'c}\, b_w d = 2\sqrt{5000}(30' \times 12)(6.625)(1/1000) = 337.3^k$

$V_u = 127.68^k < \phi V_c = .75(337.3) = 253^k$ OK

TWO WAY ACTION:

$b_o = 4(30.625) = 122.5"$

$V_c = 4\sqrt{f'c}\, b_o d = 4\sqrt{5000}(122.5)(6.625) = 229.5^k$

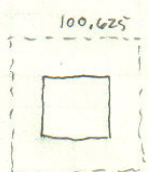
$\phi V_c = .75(229.5) = 172^k$

$V_u = [(30 \times 30.5) - (30.625 \times 30.625)(1/144)] .310 = 281.6^k$

$V_u = 281.6^k > 172^k$ Not OK

Check w/ 7.5' × 7.5' × 4" shear cap

davg = 12 - .75 - .625 = 10.625 , d/2 = 5.3125



$b_o = 4(100.625) = 402.5$

$V_c = 4\sqrt{5000}(402.5)(10.625) = 1210^k$, $\phi V_c = 907.2^k$

$V_u = [(30 \times 30.5) - (100.625^2)(1/144)] .310 = 261.8^k < 907.2^k$ OK

Column on shear cap:

$d = 10.625$ $b_o = 4(24 + 10.625) = 138.5$

$V_c = 4\sqrt{5000}(138.5)(10.625) = 416.2^k$ $\phi V_c = 312.15^k$

$V_u = [(7.5 \times 7.5) - (24 + 10.625)^2(1/144)] .310 = 14.9^k < 312.15^k$ OK

• Check Shear Cap for Additional flexure:

Interior $M_u = .07\left[(1.2 \times .125 + .5(1.6 \times .100))(30)(30.5-2)^2 - (1.2 \times .125)(30)(30.5-2)^2\right]$

$= .07\left[5605 - 3655\right] = 136.5^{1k}$ (Also see table)

$V_u = 261.8 < .75\phi V_c = 680^k \Rightarrow$ use $\gamma_F = 1$

Unbalanced Moment due to flexure

$M_{ub} = \gamma_F M_u = 1(136.5^{1k}) = 136.5^{1k}$

$M_{uv} = M_u - M_{ub} = 0 \Rightarrow$ No unbalanced moment resisted by shear

$V_L = V_{LB} - V_{LU} = V_u/A_c - \cancel{\frac{M_{uv}\,c\,\gamma}{J_c}}^{\,0}$, $A_c = d(b_o) = 10.625(402.5) = 4277\ in^2$

$= 261{,}800/4277 = 61.21$ psi

$V_E = V_L = 61.21$ psi

Check Capacity: $\phi V_n = \phi V_c / b_o d = 907{,}200/(402.5)(10.625)$

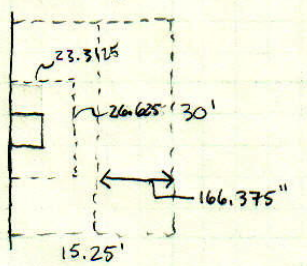
$= 212.1$ psi

212.1 psi > 61.21 psi <u>ok</u>

* Interior Columns ok w/ 7.5' x 7.5' x 12" deep Shear Cap

Edge Column

col 20 x 20 , .75" cover, $d_{avg} = 6.625$, $w_u = .310$ ksf



$(15.25' \times 12) - 10" - 6.625" = 166.375$

WIDEBEAM SHEAR:

$V_u = 30'\left(\frac{166.375}{12}\right)(.310) = 128.9^k$

$V_c = 337.3^k$ see previous

$V_u = 128.9 < \phi V_c = 253^k$ <u>OK</u>

TWO WAY ACTION:

$b_o = 2(23.3125) + 26.625 = 73.25''$

$V_c = 4\sqrt{5000}\,(73.25)(6.625) = 137.3^k$
$V_u = [(15.25 \times 30) - 20^2(1/144)]\,.310 = 140.0^k$
$V_u = 141^k > \phi V_c = 103.0^k$ Not OK

Try w/ $7.5' \times 4.583' \times 4''$ shear cap

$d_{avg} = 10.625$, $d/2 = 5.3125$, $b_o = (7.5 \times 12 + 10.625) + 2(4.583 \times 12 + 5.3125) = 221''$

$V_c = 4\sqrt{5000}\,(221)(10.625) = 664.2^k$, $\phi V_c = 498^k$

$V_u = [15.25' \times 30']\,.310 = 141.8^k < 498^k$ OK

· Check Shear Cap for additional flexure:

$V_u = 141.8 < .75\phi V_c = 373.5^k \Rightarrow$ OK, $\gamma_F = 1$

No unbalanced moment is resisted by vertical shears

* Exterior Columns OK w/ $7.5' \times 4.583' \times 4''$ Shear Cap

Column Moments

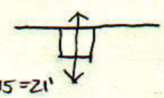
Groups:

I: Exterior Short Span Col.
II: Exterior Long Span Col.
III: Interior

Moments:

I: $W_D = 125 psf$, $W_L = 100 psf$
$M_U = .3 M_O$

- Case 1



$\ell_2 = 6+15 = 21'$
$\ell_n = 28.5'$

$$M_{OD} = \frac{.125(21')(28.5)^2}{8} = 266.5'^k$$

$$M_{UD} = .3(266.5) = 79.95'^k$$

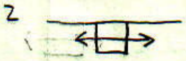
$\Rightarrow 40'^k$ to each col.

$$M_{OL} = \frac{.1(21')(28.5)^2}{8} = 213.2'^k$$

$$M_{UL} = .3(213.2) = 63.96'^k$$

$\Rightarrow 32'^k$ to each col.

- Case 2



$\ell_2 = 30.5'$
$\ell_n = 10'$

$$M_{OD} = \frac{.125(30.5')(10)^2}{8} = 47.65'^k$$

$$M_{UD} = .3(47.65) = 14.29'^k$$

$\Rightarrow 7.15'^k$ to each col.

$$M_{OL} = \frac{.1(30.5)(10)^2}{8} = 38.13'^k$$

$$M_{UL} = .3(38.13) = 11.44'^k$$

$\Rightarrow 5.71'^k$ to each col

II: $W_D = .125$, $W_L = .100$, $M_u = .3M_o$

$\ell_2 = 15+30 = 35$
$\ell_n = 28.5$

$M_{oD} = \dfrac{.125(35')(28.5)^2}{8} = 444.2'^k$

$M_{uD} = .3(444.2) = 133.2'^k$

$\Rightarrow 67'^k$ to each col.

$M_{oL} = \dfrac{.100(35')(28.5)^2}{8} = 355.4'^k$

$M_{uL} = .3(355.4) = 106.6'^k$

$\Rightarrow 53.3'^k$ to each col.

III. $W_D = .125$, $W_L = .1$, $M_u = 0.07[(q_{Du} + .5q_{Lu})\ell_2\ell_n^2 - q_{Du}\ell_2'(\ell_n')^2]$

- case 1: $\ell_2 = 40'$, $\ell_2' = 30'$, $\ell_n' = \ell_n = 28.5$

$M_{uD} = 0.07[(.125)(40')(28.5)^2 - .125(30')(28.5')^2]$

$= 71.1'^k \Rightarrow 35.6'^k$ to each col.

$M_{uL} = 0.07[(.5(.100))(40')(28.5')^2]$

$= 113.7'^k \Rightarrow 56.85'^k$ to each col.

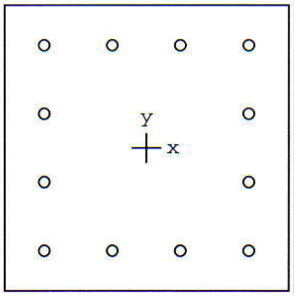
-case 2: $\ell_2 = 30.5'$, $\ell_2' = 30.5'$, $\ell_n = 38'$, $\ell_n' = 28'$

$M_{uD} = .07[.125(30.5)(38)^2 - .125(30.5)(28)^2]$

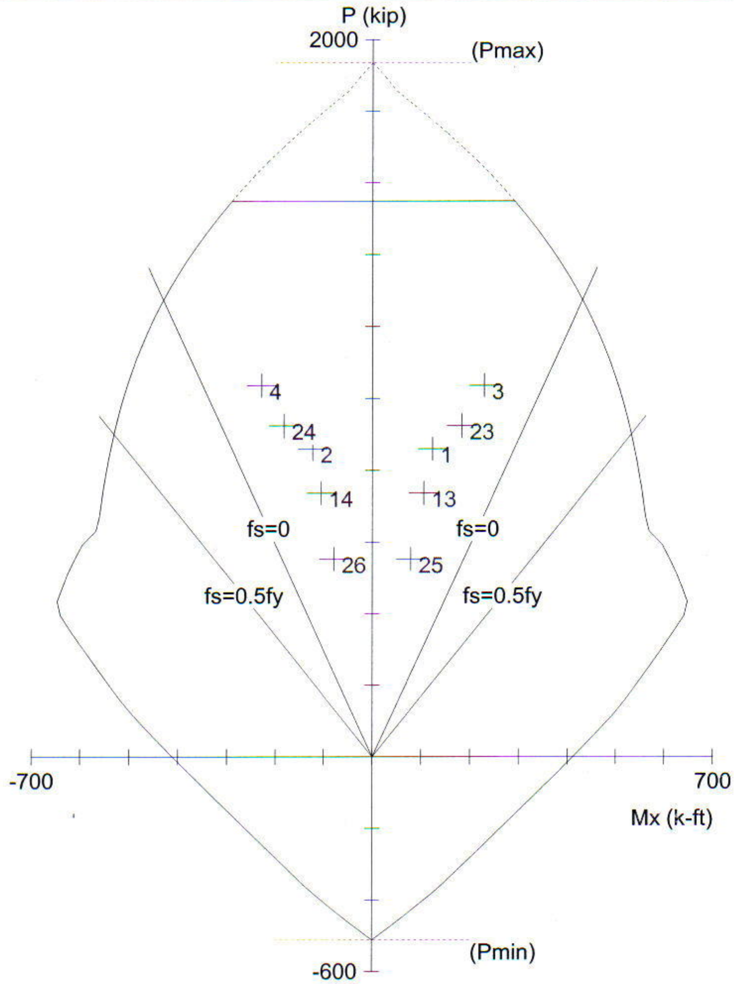
$= 176'^k \Rightarrow 88'^k$ to each col.

$M_{uL} = .07[.5(.100)(30.5')(38)^2]$

$= 154'^k \Rightarrow 77'^k$ to each col.



24 x 24 in



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Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

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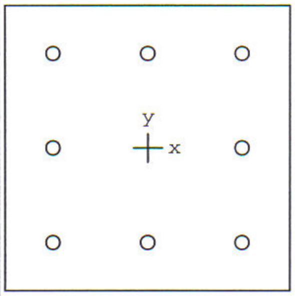
Project: National Harbor

Column: Typ. Int.		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 576 in^2	12 #8 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 9.48 in^2	Rho = 1.65%
fc = 4.25 ksi	fc = 4.25 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.8		Clear spacing = 4.75 in	Clear cover = 2.88 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		



20 x 20 in

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Run axis: About X-axis

Run option: Design

Slenderness: Not considered

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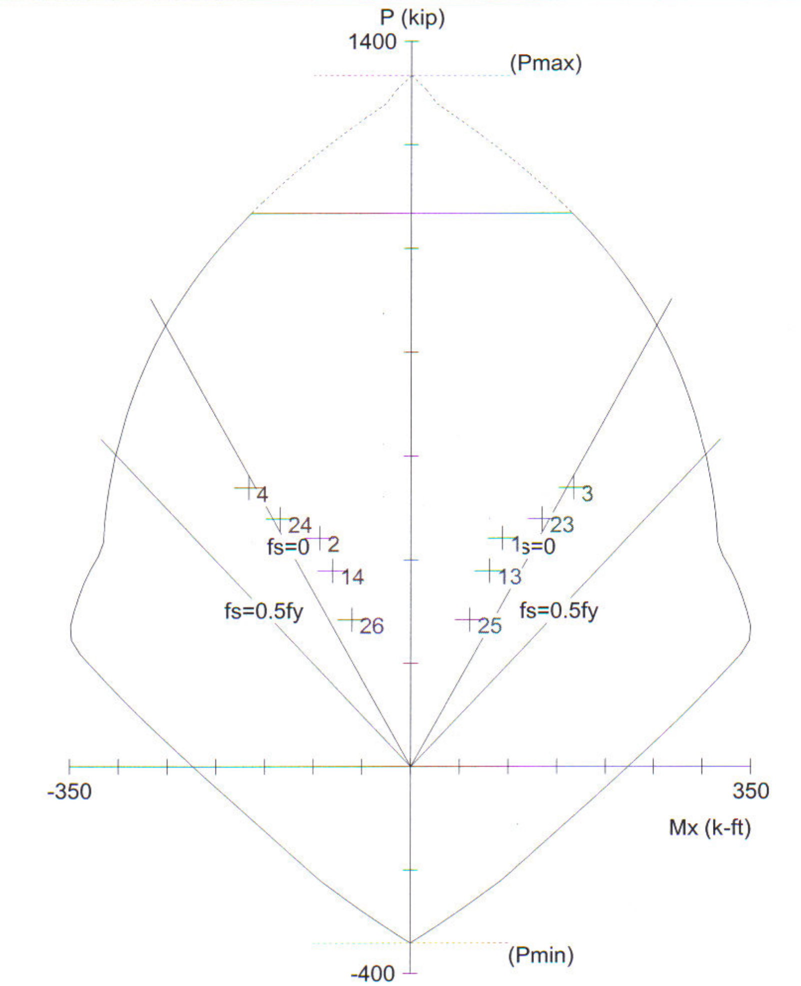
Bars: ASTM A615

Date: 03/28/08

Time: 11:37:44



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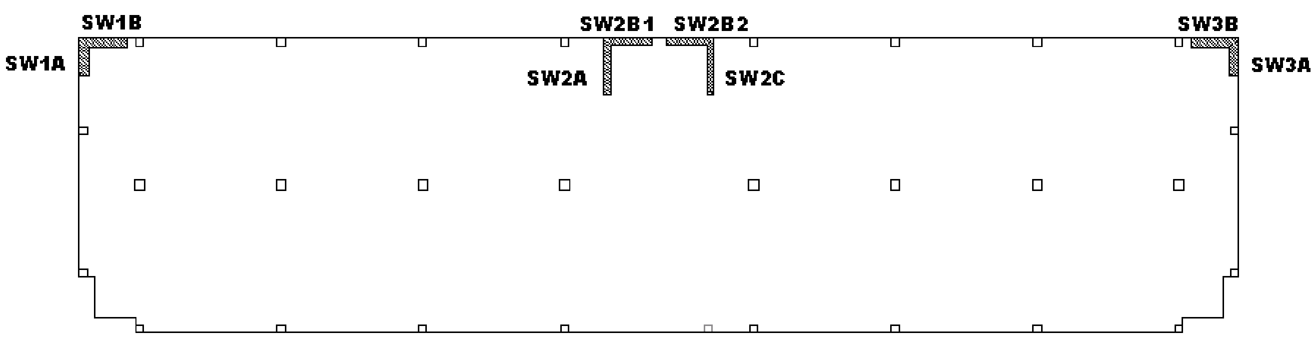
Project: National Harbor

Column: Typ. Int.		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 400 in^2	8 #8 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 6.32 in^2	Rho = 1.58%
fc = 4.25 ksi	fc = 4.25 ksi	Xo = 0.00 in	Ix = 13333.3 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 13333.3 in^4
Beta1 = 0.8		Clear spacing = 5.63 in	Clear cover = 2.88 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

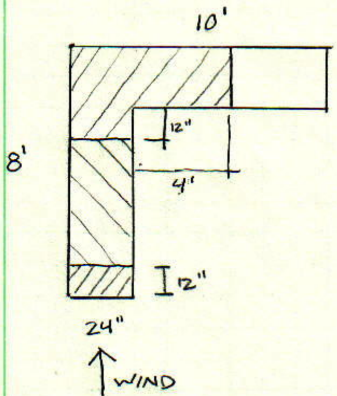
Summary of Shear Wall Forces Obtained from ETABS

Story	Pier	Load	Loc	V2 (K)	M3 (FT-K)
STORY1	SW1A	WIND	Top	77.30	592.69
STORY1	SW1A	WIND	Bottom	77.30	1320.95
STORY1	SW3A	WIND	Top	77.30	581.66
STORY1	SW3A	WIND	Bottom	77.30	1312.69
STORY1	SW1B	EQ	Top	88.54	2250.49
STORY1	SW1B	EQ	Bottom	88.54	4426.17
STORY1	SW2A	WIND	Top	123.65	1459.33
STORY1	SW2A	WIND	Bottom	123.65	2567.85
STORY1	SW2C	WIND	Top	124.50	1498.56
STORY1	SW2C	WIND	Bottom	124.50	2668.02
STORY1	SW3B	EQ	Top	88.77	2250.72
STORY1	SW3B	EQ	Bottom	88.77	4425.56
STORY1	SW2B1	EQ	Top	95.98	1546.75
STORY1	SW2B1	EQ	Bottom	95.98	2887.29
STORY1	SW2B2	EQ	Top	95.65	1541.58
STORY1	SW2B2	EQ	Bottom	95.65	2879.79



Shear Wall - Transverse Design # 2

SW3A/SW1A



SERVICE:

$P_u = 294^k$

$M_{u_{BASE}} = 1321^{1k}$

$V_u = 77.3^k$

FACTORED:

$P_u = .9(294) = 265^k$

$M_{u_{BASE}} = 1.6(1321) = 2114^{1k}$

$V_u = 1.6(77.3) = 124^k$

· Check for B.E.

$A_g = 2' \times 8' = 16\,ft^2$, $I = 2(8)^3/12 = 85.33\,ft^4$

$P_u/A_g + M_u(l_w/2)/I = 265/16 + 2114(8/2)/85.33 = 116ksf = .803\,ksi$

$.2f'_c = .2(5) = 1\,ksi$

$f_c = .803 < 1\,ksi \Rightarrow$ No need for B.E.

· Determine Long. + Trans. Reinforcement

$V_u = 124^k$

$2A_{cv}\sqrt{f'_c} = 2(24\times 96)\sqrt{5000}/1000 = 326^k > 124^k$

$\Rightarrow$ only 1 curtain req'd, 2 curtains may be used

$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_y)$

$h_w/l_w = 18.33/8 = 2.29 > 2 \Rightarrow \alpha_c = 2$

$V_u > \phi V_n \Rightarrow$

$124^k > .6\left[(24\times 96)(2\sqrt{5000} + \rho_t\, 60{,}000)\right]/1000$

$\Rightarrow \rho_t = .0009$, $\rho_t \geq .0025$ min controls

$A_{s\,req'd} = 0.0025(24\times 12) = 0.72\ in^2/ft$

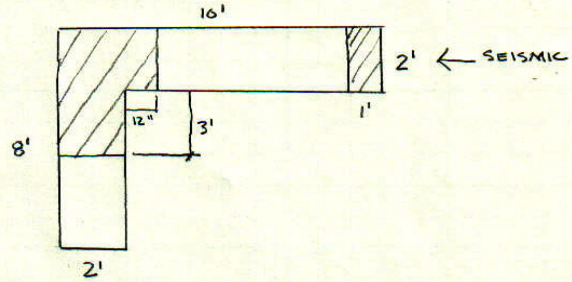
try (2) curtains of #5, $A_s = 2(.31) = .62\ in^2/spacing$

$.72/12" = .62/s$, $s_{req'd} = 10.33" < 18"$ ok

* Use (2) curtains of #5 @ 10" horozontally/vertically

Shear Wall - Longitudinal Design #2

SW1B / SW3B



SERVICE:

$P_U = 223^K$

$V_U = 88.8^K$

$M_{U_{BASE}} = 4426^{1K}$

LOAD COMBO: $(0.9 - 0.2 S_{DS}) D + \rho Q_E$

$P_U = (.9 - .2(.109))(223) = 192.3^K$

$V_U = 88.8^K$

$M_{U_{BASE}} = 4426^{1K}$

- Check for B.E.

$A_g = 2' \times 10' = 20 ft^2$, $I = \frac{2(10)^3}{12} = 167 ft^4 (.75) = 125 ft^4$

$\frac{192.3}{20} + \frac{4426(10/2)}{125} = 187 ksf = 1.3 ksi$

$.2 f'_c = .2(5) = 1 ksi < 1.3 ksi \Rightarrow$ B.E. req'd

- Determine Long. + Trans. Reinforcing

$V_U = 88.8^K$

$2 A_{cv} \sqrt{f'_c} = 2(24 \times 120)\sqrt{5000}/1000 = 407 > 88.8^K$

$\Rightarrow$ only 1 curtain needed

$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$h_w / l_w = 18.33/10 = 1.83 \Rightarrow \alpha_c = 2.34$

$V_U > \phi V_n \Rightarrow$

$88.8^K > .6\left[(24 \times 120)(2.34\sqrt{5000} + \rho_t 60{,}000)\right]/1000 \Rightarrow \rho = .001$

USE $\rho_{min} = .0025$

$A_{s\,req'd} = .0025(24 \times 12) = .72 \, in^2/ft$

try (2) curtains of #5 , $A_s = 2(.31) = .62 \, in^2/spacing$

$.72/12" = .62/s$ $\quad s_{req'd} = 10.33" < 18$ ok

* Use (2) curtains of #5 @ 10" horoz./vertical

· Design Reinforcement for B.E.

— BE # 1 24" x 12"

Axial Force: $192.3/2 + 4426/10 = 539^k$

$\phi P_{u max} = 0.8\phi \left[.85 f'c (A_g - A_{st}) + f_y A_{st}\right]$, $\phi = .7$, $A_g = 288$

$539 = .8(.7)\left[.85(5)(288 - A_{st}) + 60 A_{st}\right]$

$A_{st\,min} < \rho_{min}$, use $\rho_{min} = .01 = .01(288) = 2.88\ in^2$

try (12) # 7 bars , $A_{st} = 7.2\ in^2$ $\rho = 7.2/288 = .025 > .01$ <u>OK</u>

· Confinement Reinforcement

$S_{max} = \begin{matrix} 1/4(12) = 3" \leftarrow \text{controls} \\ 4 = 4" \end{matrix}$

Short: $b_c = 24 - 6 + 2(.625) + .875 = 20.125$ $A_{ch} = 163.5$

$A_{sh} \geq .09(3)(20.125)(5/60) = .42$
$.3(3)(20.125)(288/163.5 - 1) = 1.15 \leftarrow$ controls

use (4) #5 = $1.24\ in^2 > 1.15\ in^2$ <u>ok</u>

Long: $b_c = 12 - 6 + 2(.625) + .875 = 8.125$ $A_{ch} = 163.5$

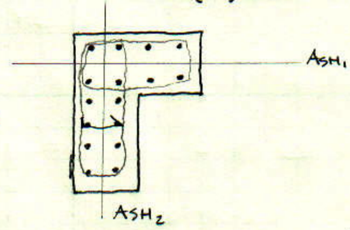
$A_{sh} \geq .09(3)(8.125)(5/60) = .183$
$.3(3)(8.125)(288/163.5 - 1) = .46 \leftarrow$ controls

USE (2) #5 = $.62 > .46\ in^2$ <u>ok</u>

— BE # 2 $A_g = 1728\ in^2$

$\rho_{min} = .01(1728) = 17.3\ in^2$ => (18) #9 or (14) #10

use (16) # 10 bars



A_{SH_1}: $S_{max} = \begin{matrix} 1/4(24) = 6" \\ 4" = 4" \leftarrow \text{cont.} \end{matrix}$

$b_c = 36 - 6 + .625(2) + 1.27 = 32.52$

$A_{ch} = 667.3\ in^2$

$A_{sh} \geq .09(3)(32.52)(5/60) = .73 \leftarrow$ cont.
$.3(3)(32.52)(5/60)(864/667.3 - 1) = .72$

(3) #5 = $.93\ in^2 > .73\ in^2$ <u>ok</u>

A_{SH_2}: $S_{max} = 4''$, $b_c = 60 - 6 + 2(.625)(1.128) = 53.88$

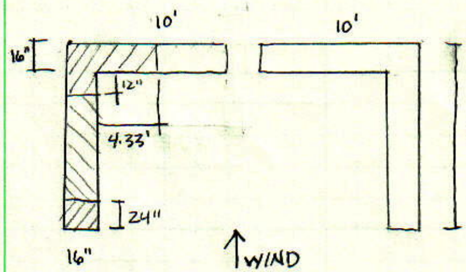
$A_{CH} = 1098$

$$A_{SH} \geq \begin{cases} .09\,(3)\,(53.88)\,(5/60) & = 1.21 \\ .3\,(3)\,(53.88)\,(5/60)\,(1440/1098 - 1) & = 1.26 \end{cases}$$

use (4) #5 bars $A_s = 1.24\,in^2 \approx 1.26\,in^2$

Shear Wall - Transverse Design #2

SW2A / SW2C



SERVICE:

P_U: 443^k

$M_{U\,BASE}$: $2668'^k$

V_U: 124.5^k

FACTORED:

$P_U: 0.9(443) = 399^k$

$M_{U\,BASE} = 4269'^k$

$V_U = 1.6(125) = 200^k$

· Check for B.E.

$A_g = (16/12) \times 12' = 16\ ft^2 \qquad I = \frac{(16/12)(12)^3}{12} = 192\ ft^4$

$399/16 + \frac{4269(12/2)}{192} = 158\ ksf = 1.1\ ksi$

$f_c = 1.1\ ksi > .2f'c = .2(5) = 1\ ksi$
$\Rightarrow$ Need B.E.

· Determine Long. & Trans. Reinforcement

$V_u = 200^k$

$2A_{cv}\sqrt{f'c} = 2(16 \times 144)\sqrt{5000}/1000 = 326^k > 200^k \Rightarrow$ only 1 curtain req'd

$V_n = A_{cv}(\alpha_c\sqrt{f'c} + \rho_t f_y)$

$h_w/l_w = 18.33/12 = 1.53 \Rightarrow \alpha_c = 2.94$

$V_u > \phi V_n \Rightarrow \quad 200 > .6\left[(16 \times 144)(2.94\sqrt{5000} + \rho_t\, 60{,}000)\right]$

$\rho_t = < .0025 \Rightarrow$ use $\rho_{min} = .0025$

$A_{s\,req'd} = .0025\,(16(12)) = .48\ in^2/ft$

try (2) curtains of #5 = $.62\ in^2$/spacing

$\Rightarrow \frac{.48}{12"} = \frac{.62}{s}$, $s = 15.5$ use 2 curtains @ 15" hor./vert.

or $\frac{.48}{12"} = \frac{.31}{s}$, $s = 7.75$ use 1 curtain @ 7.5" hor./vert.

· Design Reinforcement for B.E.

$BE \# 1 = 16'' \times 24''$

$BE\ Axial = P_u/2 + M_u/z = 399/2 + 4269/12 = 555^k$

$\phi P_{u\,max} = 0.8\phi\,[.85f'c\,(A_g - A_{st}) + f_y A_{st}]$, $\phi = .7$, $A_g = 384$

$555{,}100 = .8(.7)[.85(5000)(384 - A_{st}) + 60{,}000(A_{st})]$

$A_{st} \leq$ $\rho_{min} = 0.01$, $A_{st} = .01(24 \times 16) = 3.84\ in^2$

try (12) #7 bars , $A_{st} = 12(.6) = 7.2\ in^2$ $\rho = 7.2/(24 \times 16) = .019$

$\rho_{min} = .01 < \rho = .019 < \rho_{max} = .06$ <u>ok</u>

· Confinement Reinforcing (assume #5 bars)

$S_{max} = \min \left| \begin{array}{l} 1/4(16) = 4'' \\ 4'' \end{array} \right.$ 4" spacing

Short: $A_{CH} = (16 - 6 + 2(.625))(24 - 6 + 2(.625)) = 217\ in^2$
$b_c = (24 - 6 + .625) = 18.625$

$A_{SH} \geq \begin{array}{l} .09\, S\, b_c\, f'c/f_{yt} \\ .3\, S\, b_c\, (A_g/A_{CH} - 1)\, f'c/f_{yt} \end{array}$

$\geq \begin{array}{ll} .09(4)(18.625)(5/60) & = .559\ in^2 \\ .3(4)(18.625)(384/217 - 1)(5/60) & = 1.43\ in^2 \end{array}$

use (5) #5 $= .31(5) = 1.55\ in^2 > 1.43\ in^2$ <u>ok</u>

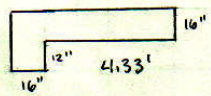
Long: $b_c = (18 - 6 + .625) = 12.625$

$A_{SH} \geq \begin{array}{ll} .09(4)(12.625)(5/60) & = .379\ in^2 \\ .3(4)(12.625)(384/217 - 1)(5/60) & = .972\ in^2 \end{array}$

Use (3) #6 $= .44(3) = 1.32\ in^2 > .972\ in^2$ <u>ok</u>

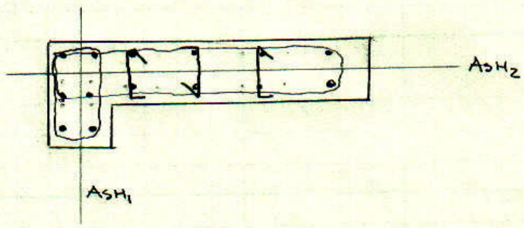
for short try (4) #6 $= .44(4) = 1.74\ in^2 > 1.43$ <u>ok</u>

BE # 2 :



$A_g = 1279 \text{ in}^2$

A_{ST} req'd $= .01(1279 \text{ in}^2) = 12.79 \text{ in}^2 \Rightarrow$ (13) #9 bars



• Confinement Reinforcing

1. $S_{max} = 4''$, $b_c = (28 - 6 + 2(.625) + 1.128) = 24.378''$

$A_{CH} = 24.378(16 - 6 + 2(.625) + 1.128) = 301.75 \text{ in}^2$

$A_{SH_1} > .09(4)(24.378)(5/60) = .73$

$.3(4)(24.378)(5/60)(448/301 - 1) = 1.19 \text{ in}^2 > 3(.31)$ Not ok

use 3" spacing $\Rightarrow A_{SH_1} \geq .89 < .93$ OK

Use (3) #5

2. $S_{max} = 4''$, $b_c = (67.96 - 6 + 2(.625) + 1.128) = 64.338$, $A_{CH} = 796.38 \text{ in}^2$

$A_g = 1087$

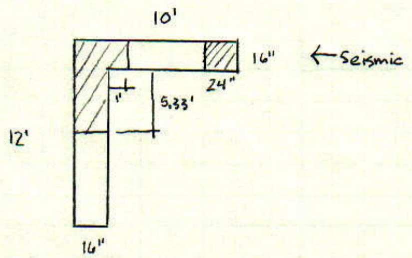
$A_{SH_2} \geq .09(3)(64.34)(5/60) = 1.45 \text{ in}^2$

$.3(3)(64.34)(5/60)(1087/796.4 - 1) = 1.76 \text{ in}^2 \Rightarrow 6(.31) = 1.86 \text{ in}^2$

use (6) #5

Shear Wall - Longitudinal Design #2

SW2B1/SW2B2



SERVICE:

$P_u = 149^k$

$M_{u_{BASE}} = 2887^{1k}$

$V_u = 95.98^k$

LOAD COMBO: $(0.9 - .2S_{DS})D + \rho Q_E$

$P_u = (.9 - .2(.189))(149) = 128^k$

$M_{u_{BASE}} = 2887^{1k}$

$V_u = 95.98^k$

· Check for B.E.

$A_g = 10' \times (16/12) = 13.33 ft^2 \quad I = \frac{(16/12)(10^3)}{12} = 111.1 ft^4 (.75) = 83.3 ft^4$

$\frac{149}{13.33} + \frac{2887(10/2)}{83.3} = 184 \text{ ksf} = 1.28 \text{ ksi}$

$2f'c = 2(.5) = 1 < 1.28 \text{ ksi} \Rightarrow$ Need B.E.

· Determine Long. + Trans. Reinforcing

$V_u = 95.98^k \qquad 2A_{cv}\sqrt{f'c} = 2(16 \times 120)\sqrt{5000}/1000 = 271.5^k$

$95.98^k < 271.5^k \Rightarrow$ only 1 curtain req'd

$V_n = A_{cv}(\alpha_c \sqrt{f'c} + \rho_t f_y)$

$h_w/l_w = 18.33/10 = 1.83 \Rightarrow \alpha_c = 2.34$

$V_u > \phi V_n \Rightarrow$

$95.98^k > .6[(16 \times 120)(2.34\sqrt{5000} + \rho\, 69000)]/1000 \Rightarrow \rho < \rho_{min}$

use $\rho_{min} = .0025$

A_s req'd $= .0025(16 \times 12) = .48 \text{ in}^2/\text{ft}$

try (2) curtains of #5 $= .62 \text{ in}^2/s$

$\frac{.48}{12} = \frac{.62}{s} \Rightarrow s = 15"$ horiz./vert.

· For B.E. use reinforcing similar to that use in design of SW's 2A + 2C.

FOUNDATION CHECKS

- Typical interior column:

Total Axial Load = $1.2(614) + 1.6(186.33) = 1035^k$

$1035^k / 220^k/\text{PILE} = 4.7$ or 5 piles required

- MODIFICATION REQ'D

- Typical exterior column:

Total Axial Load = $1.2(316) + 1.6(99.5) = 538^k$

$538^k / 220^k/\text{PILE} = 2.4$ or 3 piles required

- Exterior - short span column:

Total Axial Load = $1.2(110) + 1.6(30.09) = 180^k$

$180^k / 220^k/\text{PILE} = .81$ or 1 pile required

- Shear Walls 1-3

Total Axial Load = $1.2(586) + 1.6(86.48) = 842^k$

$842^k / 220^k/\text{PILE} = 3.8$ or 4 piles required

CONTROLING UPLIFT FORCE = 539^k

$539^k / 110^k/\text{PILE} = 4.9$ or 5 uplift piles required

CONTROLLING LATERAL FORCE = 124^k

$124^k / 15^k/\text{PILE} = 8.2$ or 9 piles required

* 10 provided for Axial/Uplift ok

USE 5 UPLIFT PILES & 4 AXIAL PILES

- MODIFICATION REQ'D

- Shear Wall 2

Total Axial Load = $1.2(1186) + 1.6(177) + 289 = 1995.4^k$

↑ ELEVATOR REACTIONS (FACTORED)

$1995.4^k / 220^k/\text{PILE} = 9.07$ PILES or 10 PILES

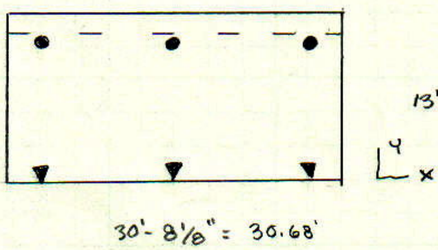
CONTROLING UPLIFT FORCE = 1110^k (FACTORED)

$1110^k / 220^k/\text{PILE} = 5.1$ or 6 UPLIFT PILES

CONTROLING LATERAL FORCE = 400^k (FACTORED)

$400^k / 15^k/\text{PILE} = 26.7$ or 27 PILES REQUIRED

PRECAST PANEL SEISMIC CALC



4" thick SAND-LIGHTWEIGHT PANEL

$$F_p = \frac{.4 a_p S_{DS} W_p}{R_p}\left(1 + 2\frac{z}{h}\right) \quad \text{Eq. 3.2.4.18}$$

	a) Wall Element	b) Body of Panel	c) Fastner of Connection
a_p	1.0	1.0	1.25
R_p	2.5	2.5	1.0

* assume top of building $\Rightarrow z/h = 1 \Rightarrow (1 + 2 z/h) = 3$

$W_p = (4/12)(115\text{pcf})(30.68' \times 13.33') = 15{,}700 \text{ lbs} = 15.7^k$

$S_{DS} = .189$

a)=b) $F_p = \dfrac{.4(1.0)(.189)(15{,}700)}{2.5}(3) = 1424$ lbs

c) $F_p = \dfrac{.4(1.25)(.189)(15{,}700)}{1.0}(3) = 4451$ lbs

$$.3 W_p S_{DS} < F_p < 1.6 W_p S_{DS}$$
$$.3(15{,}700)(.189) < F_p < 1.6(15{,}700)(.189)$$
$$890 < F_p < 4748 \quad \underline{ok}$$

$F_p/3 = 4451/3 = 1484$ lbs/connection

FACADE JOINT CALCS

- Horozontal Movement

$$J = \frac{100A}{X} + B$$

A = thermal movement = $5 \times 10^{-6} (100°F)(30' \times 12") = 0.18"$

X = Sealant extensibility = 25%

B = Construction tolerance, .25" typ.

$$J = \frac{100(.18)}{25} + .25 = .97"$$ use 1" joints

1" joint < 2" max joint width of sealant ok

- Required Sealant depth = ½ joint width of joint between ½" to 1"

$$\Rightarrow ½(1") = ½" \text{ deep}$$

Comparable Cost Manual Take offs:

- Concrete in Slabs:

Original - 75,773.5 ft² total fl area (3.75"/12)

= 23,679 ft³ => 877 CY

Redesign - 1922.5 CY - from Concept

- Drop Panels

8 full, 18 Half, 8 Quarter => 19 Equiv. Full

1 Panel => 7.5' (4 sides)(4"/12) + (7.5 × 7.5 - 22/12 × 22/12)

= 62.89 SFCA each => 1195 SFCA per FL

=> 5975 SFCA total

19 × (7.5' × 7.5' × 4"/12) = 356.25 × 5 floors

= 1826.25 ft³

= 67.65 CY

- Shear Walls

12 : Forms: 22LF (74.67') = 1643 SFCA

Conc: 2765F => 76.22 CY

Reinf.

	Bar	Length	Wt.
Vert.	24#7	1776'	3630 lbs
	22#9	1628'	5535 lbs
	14#5	1036'	1080 lbs
			10,246 lbs = 5.12 tons
Horiz.	(178)#5 @10'	1780'	1857 lbs
	(178) #5 @8'	1424'	1485 lbs
			3342 lbs = 1.67 tons

total wt = 6.79 tons

1-3: Forms : 36 LF (74.67) = 2688 SFCA
CONC : 32 SF => 88.50 CY

Reinf:

	BAR	LENGTH	WT.
VERT	12#7	888'	1815.1
	16#10	1184'	5094.8
	22#5	1628'	1698
			8608 lbs => 4.3 tons
HOR.	(178)#5@6'	1068'	
	(178)#5@8'	1424'	
			2599 lbs => 1.3 tons

Total Wt = 5.6 tons

• CMU REINF.

WALL LENGTH = 230' or 2760"   #6 plf = 1.502
S.W. LENGTH = 120' or 1440"

St. Ht.	BARS	LENGTH
19'	(690)#6 + (16)#6	13,414'
13.33'	(345)#6 + (12)#6	4,759'
13.33'	(115)#6 + (8)#6	1,640'
13.33'	(115)#6 + (4)#6	1,613'
14.33'	(690)#6 + (4)#6	9,945'
		31,371' (1.502 plf)

= 47,119 lbs = 23.56 tons

EXISTIMG SYSTEM COMPARABLE COST:

STEEL SYSTEM			
	Length	R.S. Means Quantity	Total Cost:
	(ft)	Price ($/L.F.)	(Labor +Equipment+Material)
Gravity Beams:			
W8X10	852.60	18.62	$15,875.41
W10X12	133.00	21.02	$2,795.66
W12X14	632.03	21.39	$13,519.12
W12X40	9.21	53.45	$492.27
W12X16	167.17	21.39	$3,575.77
W12X19	192.21	30.94	$5,946.98
W12X22	49.13	30.94	$1,520.08
W12X30	42.71	35.94	$1,535.00
W14X22	1346.14	35.45	$47,720.66
W14X43	30.46	56.82	$1,730.74
W16X26	4248.03	35.41	$150,422.74
W16X31	274.12	41.84	$11,469.18
W18X35	654.17	47.8	$31,269.33
W18X40	152.29	53.8	$8,193.20
W18X50	782.75	66.08	$51,724.12
W18X55	120.00	72.08	$8,649.60
W18X86	40.00	109.66	$4,386.40
W21X44	774.50	57.79	$44,758.36
W21X48	30.46	65.29	$1,988.73
W21X50	213.21	65.29	$13,920.48
W21X62	30.00	79.91	$2,397.30
W21X68	30.38	87.41	$2,655.52
W21X73	80.00	87.41	$6,992.80
W21X83	100.00	105.09	$10,509.00
W21X122	30.00	153.09	$4,592.70
W24X55	429.00	71.09	$30,497.61
W24X62	90.00	79.59	$7,163.10
W24X68	128.84	87.09	$11,220.68
W24X76	90.38	96.59	$8,729.80
W24X84	90.38	106.71	$9,644.45
W24X94	30.00	118.71	$3,561.30
W24X104	30.00	130.85	$3,925.50
W24X131	30.38	163.75	$4,974.73
W27X84	365.03	106.28	$38,795.39
W27X94	30.00	118.28	$3,548.40
W30X90	40.00	116.54	$4,661.60
W30X116	78.47	144.39	$11,330.28
			$586,693.98
Gravity Columns			
W12X45	83.30	64.29	$5,355.36
W12X50	41.70	64.29	$2,680.89
W12X53	208.30	67.91	$14,145.65
W12X58	41.70	77.57	$3,234.67
W12X65	117.70	86.02	$10,124.55
W12X72	125.00	94.5	$11,812.50
W12X79	68.20	102.93	$7,019.83
W12X87	109.90	108.97	$11,975.80
W12X96	34.40	119.91	$4,124.90
W14X99	67.20	123.69	$8,311.97

W14X109	67.70	135.77	$9,191.63
W12X120	33.80	149.07	$5,038.57
			$93,016.32
Studs	**EQ. WEIGHT**	**R.S. Means Quantity**	**Total Cost:**
	(LBS)	**Price ($/LBS)**	**(Labor +Equipment+Material)**
8031 studs	80,310	$1.30	**$104,403.00**
FRAME SYSTEM:			
	Length	**R.S. Means Quantity**	**Total Cost:**
	(ft)	**Price ($/L.F.)**	**(Labor +Equipment+Material)**
Lateral Beams			
W8X10	19.10	18.62	$355.64
W12X40	21.50	53.45	$1,149.18
W12X45	21.50	59.59	$1,281.19
W14X43	9.60	56.82	$545.47
W14X22	19.10	35.45	$677.10
W14X48	9.60	56.82	$545.47
W14X30	38.30	40.84	$1,564.17
W14X38	19.10	45.82	$875.16
W18X55	38.20	72.08	$2,753.46
W18X65	122.60	84.16	$10,318.02
W21X44	23.70	57.79	$1,369.62
W21X101	91.40	127.09	$11,616.03
W21X83	271.50	105.09	$28,531.94
W21X111	182.80	122.59	$22,409.45
W21X93	90.90	118.09	$10,734.38
W21X132	121.80	164.39	$20,022.70
W21X147	182.80	181.85	$33,242.18
W24X68	30.00	87.09	$2,612.70
W24X84	60.00	106.71	$6,402.60
W24X117	19.10	146.85	$2,804.84
W24X94	87.30	118.71	$10,363.38
W24X103	158.20	130.85	$20,700.47
W24X146	60.00	181.85	$10,911.00
W27X84	30.00	106.28	$3,188.40
			$204,974.53
Lateral Columns			
W12X65	41.70	86.02	$3,587.03
W12X58	41.70	77.57	$3,234.67
W14X90	88.70	111.60	$9,898.92
W14X99	263.00	122.76	$32,285.88
W14X109	255.40	135.16	$34,519.86
W14X120	153.40	149.07	$22,867.34
W14X132	83.30	163.68	$13,634.54
W14X145	179.70	180.13	$32,369.36
W14X159	138.00	197.52	$27,257.76
W14X176	204.70	217.29	$44,479.26
W14X211	67.70	260.50	$17,635.85
			$241,770.48
Lateral Braces			
W12X79	68.40	102.16	$6,987.74

W12X106	32.80	137.08	$4,496.22
W12X152	32.80	194.12	$6,367.14
W12X190	45.20	242.65	$10,967.78
			$28,818.88
		STEEL COST	**$1,259,677.21**
DECKING/ SLAB:			
	Area	**R.S. Means Quantity**	**Total Cost:**
	(sq. ft)	**Price ($/S.F.)**	**(Labor +Equipment+Material)**
3" 18 gauge	7527	2.11	**$15,881.34**
Roof Deck			
3"Deep Galvanized	68207	2.36	**$160,968.05**
20 gauge Deck			
	(C.Y.)	**Price ($/C.Y.)**	
3.75" Lightweight	877	125	**$109,626.25**
Concrete			
Placing- Elevated Slab	877	20.45	**$17,934.65**
pumped			
	(C.S.F.)	**Price ($/C.S.F.)**	
WWM Reinforcing	757	32.9	**$24,915.17**
		DECKING/SLAB COST	**$329,325.46**
CMU WALL:			
	Area	**R.S. Means Quantity**	**Total Cost:**
	(sq. ft)	**Price ($/S.F.)**	**(Labor +Equipment+Material)**
Exterior Block	17792	7.42	**$132,016.64**
	(ton)	**Price ($/ton)**	
Reinforcing	23.56	1350.00	**$31,806.00**
	(L.F.)	**Price ($/L.F.)**	
Bond Beam	2193	9.14	**$20,044.02**
		CMU WALL TOTAL	**$183,866.66**
		TOTAL COMPARABLE COST	**$1,772,869.32**

REDESIGN COMPARABLE COST:

	Unit	R.S. Means Quantity	Total Cost:
	(CY)	Price ($/CY)	(Labor +Equipment+Material)
COLUMNS			
24"x24"	88.5	887.5	$78,543.75
20"x20"	153.64	1019.55	$156,643.66
		COLUMN TOTAL	**$235,187.41**
SHEAR WALLS			
Forms	**(SFCA)**	**Price ($/SFCA)**	**(Labor +Equipment+Material)**
Shear Wall 1	2688.12	6.46	$17,365.26
Shear Wall 2a	1642.74	6.46	$10,612.10
Shear Wall 2b	1642.74	6.46	$10,612.10
Shear Wall 3	2688.12	6.46	$17,365.26
			$55,954.71
Concrete	**(CY)**	**Price ($/CY)**	**(Labor +Equipment+Material)**
Shear Wall 1	88.50	109.00	$9,646.50
Shear Wall 2a	76.22	109.00	$8,307.98
Shear Wall 2b	76.22	109.00	$8,307.98
Shear Wall 3	88.50	109.00	$9,646.50
			$35,908.96
Concrete Placing	**(CY)**	**Price ($/CY)**	**(Labor +Equipment+Material)**
Shear Wall 1	88.50	23.85	$2,110.73
Shear Wall 2a	76.22	23.85	$1,817.85
Shear Wall 2b	76.22	23.85	$1,817.85
Shear Wall 3	88.50	23.85	$2,110.73
			$7,857.14
Reinforcment	**(tons)**	**Price ($/ton)**	**(Labor +Equipment+Material)**
Shear Wall 1	5.60	1350.00	$7,560.00
Shear Wall 2a	6.79	1350.00	$9,166.50
Shear Wall 2b	6.79	1350.00	$9,166.50
Shear Wall 3	5.60	1350.00	$7,560.00
			$33,453.00
		SHEAR WALL TOTAL	**$133,173.82**

PRECAST WALL	**(S.F.)**	**Price ($/S.F.)**	**(Labor +Equipment+Material)**
Uninsulated 4"	17792	6.87	**$122,231.04**
smooth grey low			
rise, 30'x13'x4"			
DROP CAPS			
Forms	**(SFCA)**	**Price ($/SFCA)**	**(Labor +Equipment+Material)**
(95) EQ.	5975	6.16	**$36,806.00**
7.5'x7.5'x4"			
Concrete	**(CY)**	**Price ($/CY)**	**(Labor +Equipment+Material)**
(95) EQ.	67.65	109.00	**$7,373.85**
7.5'x7.5'x4"			
Concrete Placing	**(CY)**	**Price ($/CY)**	**(Labor +Equipment+Material)**
(95) EQ.	67.65	20.45	**$1,383.44**
7.5'x7.5'x4"			
		DROP CAP TOTAL	**$45,563.29**
FOUNDATION			
Piles	**Amount**	**Price ($/pile)**	**(Labor +Equipment+Material)**
14" sq. prestressed	12	3000.00	**$36,000.00**
precast piles			
SLAB (from Concept)			
Concrete	**(CY)**	**Price ($/CY)**	**(Labor +Equipment+Material)**
	1922.5	159	**$305,677.50**
Post- Tensioning	**(LBS)**	**Price ($/LBS)**	**(Labor +Equipment+Material)**
	95150	1.5	**$142,725.00**
Formwork	**(SFCA)**	**Price ($/SFCA)**	**(Labor +Equipment+Material)**
	74750	4.85	**$362,537.50**
Mild Steel Reinf.	**(tons)**	**Price ($/ton)**	**(Labor +Equipment+Material)**
	42.4	1500	**$63,600.00**
		SLAB TOTAL	**$874,540.00**
		TOTAL COMPARABLE COST	**$1,446,695.56**

	Task Name	Duration	Start	Finish
1	Detailing/ Shop Drawings	21 days	Tue 4/1/08	Tue 4/29/08
2	1st Floor Col./S.W. stage 1	3 days	Wed 4/30/08	Fri 5/2/08
3	1st Floor Col./S.W. stage 2	4 days	Mon 5/5/08	Thu 5/8/08
4	2nd Floor Col./S.W. stage 1	3 days	Tue 5/20/08	Thu 5/22/08
5	2nd Floor Col./S.W. stage 2	4 days	Fri 5/23/08	Wed 5/28/08
6	3rd Floor Col./S.W. stage 1	3 days	Mon 6/9/08	Wed 6/11/08
7	3rd Floor Col./S.W. stage 2	4 days	Thu 6/12/08	Tue 6/17/08
8	4th Floor Col./S.W. stage 1	3 days	Tue 6/24/08	Thu 6/26/08
9	4th Floor Col./S.W. stage 2	4 days	Fri 6/27/08	Wed 7/2/08
10	5th Floor Col./S.W. stage 1	3 days	Mon 7/14/08	Wed 7/16/08
11	5th Floor Col./S.W. stage 2	4 days	Thu 7/17/08	Tue 7/22/08
12	1st Floor Slab stage 1	5 days	Fri 5/9/08	Thu 5/15/08
13	1st Floor Slab stage 2	3 days	Fri 5/16/08	Tue 5/20/08
14	1st Floor Slab stage 3	2 days	Wed 5/21/08	Thu 5/22/08
15	2nd Floor Slab stage 1	5 days	Thu 5/29/08	Wed 6/4/08
16	2nd Floor Slab stage 2	3 days	Thu 6/5/08	Mon 6/9/08
17	2nd Floor Slab stage 3	2 days	Tue 6/10/08	Wed 6/11/08
18	3rd Floor Slab stage 1	5 days	Wed 6/18/08	Tue 6/24/08
19	3rd Floor Slab stage 2	3 days	Wed 6/25/08	Fri 6/27/08
20	3rd Floor Slab stage 3	2 days	Mon 6/30/08	Tue 7/1/08
21	4th Floor Slab stage 1	5 days	Thu 7/3/08	Wed 7/9/08
22	4th Floor Slab stage 2	3 days	Thu 7/10/08	Mon 7/14/08
23	4th Floor Slab stage 3	2 days	Tue 7/15/08	Wed 7/16/08
24	5th Floor Slab stage 1	5 days	Wed 7/23/08	Tue 7/29/08
25	5th Floor Slab stage 2	3 days	Wed 7/30/08	Fri 8/1/08
26	5th Floor Slab stage 3	2 days	Mon 8/4/08	Tue 8/5/08