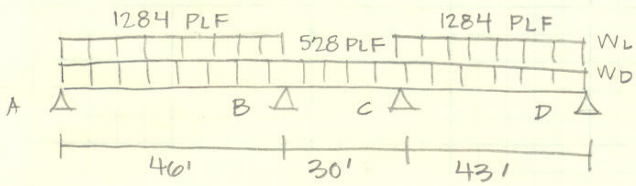
Appendices

Appendix A

CIP Joist Design Calculations

Joist Positive Moment – Moment Distribution

Joist Positive Moment



Dead Loads

24" Joists

Self: 119 PSF (4') = 476 PLF
SDL: 15 PSF (4') = 60 PLF
536 PLF

$1.2(536) = 644$ PLF

16" Joists

Self: 95 PSF (4') = 380 PLF
SDL: 15 PSF (4') = 60 PLF
440 PLF

$1.2(440) = 528$ PLF

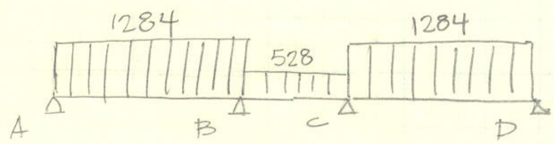
Live Loads

LL_{office} = 100 PSF

LL_{office} = 100 PSF (4') = 400 PLF $1.6(400) = 640$ PLF

2

$I_{16''} = 12,128 \text{ in}^4$
$I_{24''} = 32,297 \text{ in}^4$



$K_{AB} = \dfrac{E(32297)}{46(12)} = 58.51\,E \qquad 1.74K$

$K_{BC} = \dfrac{E(12128)}{30(12)} = 33.69\,E \qquad K$

$K_{CD} = \dfrac{E(32297)}{43(12)} = 62.59\,E \qquad 1.86K$

$FEM_{AB} = -\dfrac{1.284(46)^2}{12} = -226.41$

$FEM_{BA} = +226.41$

$FEM_{BC} = -\dfrac{0.528(30)^2}{12} = -39.6$

$FEM_{CB} = +39.6$

$FEM_{CD} = -\dfrac{1.284(43)^2}{12} = -197.84$

$FEM_{DC} = +197.84$

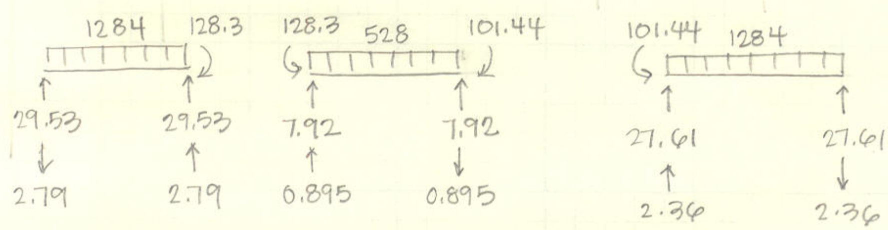
$K_{AB}^{m} = 0.75(1.74K) = 1.31\,K$

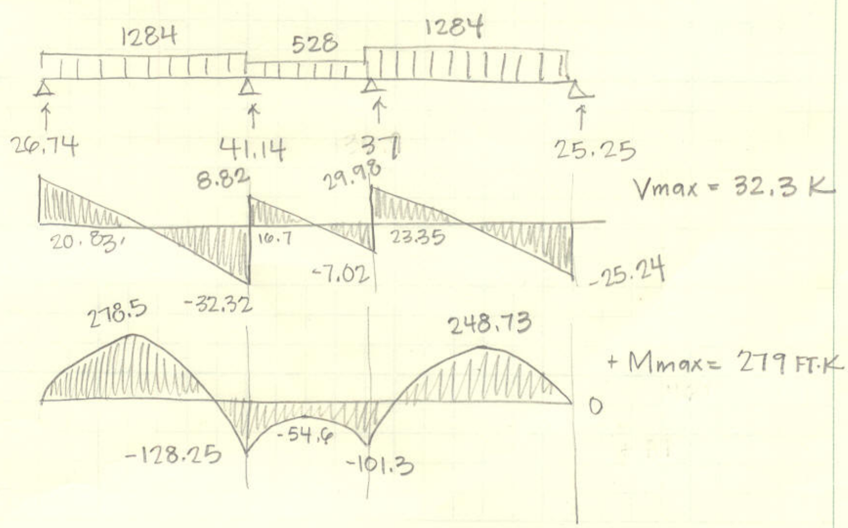
$K_{CD}^{m} = 0.75(1.86K) = 1.40K$

3

	A	B		C		D
	AB	BA	CB	BC	CD	DC
K	1.74K	1.74K	K	K	1.86K	1.86K
K^m		1.31K			1.40K	
DF	1	0.57	0.43	0.42	0.58	1
FEM	-226.4	226.41	-39.6	39.6	-197.84	197.84
	+226.4	+113.2			-98.92	-197.84
	0	-171	-129	-64.5		0
			+67.55	+135.1	+186.56	
		-38.5	-29.05	-14.53		
			+3.05	+6.1	+8.43	
		-1.74	-1.31	-0.55		
			+0.115	+0.23	0.32	
		-0.066	-0.049	-0.025		
				+0.01	+0.015	
	0	128.3	-128.3	101.44	-101.44	0

Max positive moment = 128.3 FT·K



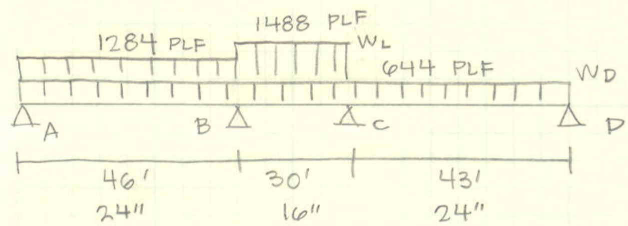


Joist Negative Moment – Moment Distribution

1

Joist Negative Moment

Joists 48" o.c.



Dead Loads

24" Joists:

self: 119 PSF (4') = 476 PLF
SDL: 15 PSF (4') = 60 PLF
536 PLF

1.2 (536) = 644 PLF

16" Joists:

self: 95 PSF (4') = 380 PLF
SDL: 15 PSF (4') = 60 PLF
440 PLF

1.2 (440) = 528 PLF

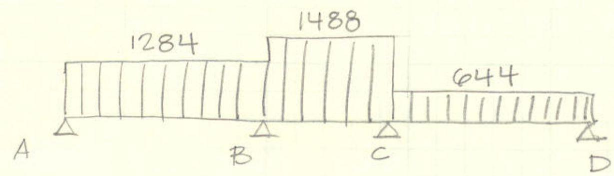
Live Loads

LL_{mech} = 150 PSF
LL_{office} = 100 PSF

LL_{mech} = 150 PSF (4') = 600 PLF 1.6 (600) = 960 PLF
LL_{office} = 100 PSF (4') = 400 PLF 1.6 (400) = 640 PLF

2

$I_{16''} = 12128$
$I_{24''} = 32,297$



$K_{AB} = \dfrac{E(32,297)}{46'(12''/FT)} = 58.51 \text{ in}^3 E$ 1.74K

$K_{BC} = \dfrac{E(12,128)}{30(12)} = 33.69 \text{ in}^3 E$ K

$K_{CD} = \dfrac{E(32,297)}{43(12)} = 62.59 \text{ in}^3 E$ 1.86K

$FEM_{AB} = -\dfrac{wl^2}{12} = -\dfrac{1.284(46)^2}{12} = -226.41$ FT·K

$FEM_{BA} = 226.41$ FT·K

$FEM_{BC} = -\dfrac{1.488(30)^2}{12} = -111.6$ FT·K

$FEM_{CB} = 111.6$ FT·K

$FEM_{CD} = -\dfrac{0.644(43)^2}{12} = -99.23$ FT·K

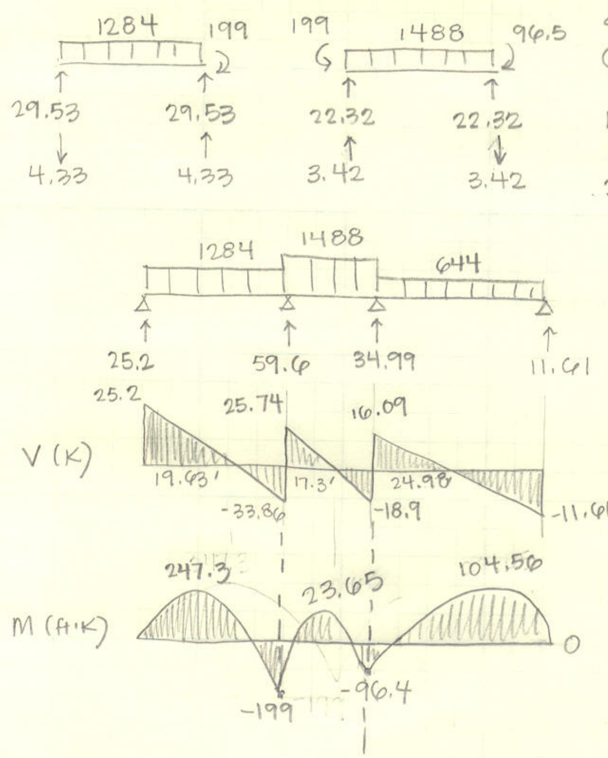
$FEM_{DC} = 99.23$ FT·K

$K_{ab}^m = 0.75(1.74K) = 1.31K$
$K_{cd}^m = 0.75(1.86K) = 1.40K$

3

	A	B		C		D
	AB	BA	BC	CB	CD	DC
K	1.74K	1.74K	K	K	1.86K	1.86K
Km		1.31K			1.40K	
DF	1	0.57	0.43	0.42	0.58	1
FEM	-226.41	+226.41	-111.6	+111.6	-99.23	+99.23
	+226.41	+113.21			-49.62	-99.23
	0	-129.97	-98.05	-49.03		0
			+18.12	+36.24	+50.04	
		-10.33	-7.79	-3.90		
			+0.82	+1.64	+2.26	
		-0.47	-0.35	-0.175		
			+0.037	+0.074	+0.102	
		-0.021	-0.016			
	0	198.83	-199.1	96.45	-96.45	0

Max Negative moment = 199.1 FT·K



Vmax = 33.9 K

-Mmax = -199 FT·K

+M16" = 23.65 FT·K

Joist Positive Reinforcement [46'/43' Span (24")Joists]

Mu+ = 279 ft-k
fy = 60 ksi
f'c = 4 ksi
#3 Stirrups

Try (2) #10

d = 28.5"-1.5"-0.375"-0.5(1.27")
d = 25.99"

As = $2(1.27in^2) = 2.54in^2$

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{2.54in^2(60ksi)}{0.85(4ksi)(48")}$$
$$a = 0.934"$$

$$c = \frac{a}{\beta}$$
$$c = \frac{0.934"}{0.85}$$
$$c = 1.099" \qquad \leq 0.375(25.99") = 9.75" \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(2.54in^2)(60ksi)\left(25.99" - \frac{0.934"}{2}\right)$$
$$\phi M_n = 3500.7in - k$$
$$\phi M_n = 291.73 ft - k$$

$$\phi M_n = 291.73 ft - k \geq M_u = 279 ft - k \therefore OK$$

$$b_{min} = 2(1.5") + 2(1.27") + (1)1.27" + (0.375")$$
$$b_{min} = 7.2" \leq 8" \therefore OK$$

Use (2) #10 bottom bars

Joist Positive Reinforcement [30’ Span (16”) Joists]

Mu+ = 23.65 ft-k
fy = 60 ksi
f’c = 4 ksi
#3 Stirrups

Try (1) #6

d = 20.5”-1.5”-0.375”-0.5(0.75”)
d = 18.25”

As = 1(0.44in^2) = 0.44in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{0.44in^2(60ksi)}{0.85(4ksi)(48")}$$
$$a = 0.162"$$

$$c = \frac{a}{\beta}$$
$$c = \frac{0.162"}{0.85}$$
$$c = 0.191" \qquad \leq 0.375(18.25") = 6.84" \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(0.44in^2)(60ksi)\left(18.25" - \frac{0.162"}{2}\right)$$
$$\phi M_n = 431.7in - k$$
$$\phi M_n = 35.97 ft - k$$

$$\phi M_n = 35.97 ft - k \geq M_u = 23.65 ft - k \therefore OK$$

$$b_{\min} = 2(1.5") + 1(0.75") + 1(0.375")$$
$$b_{\min} = 4.12" \leq 8" \therefore OK$$

Use (1) #6 bottom bar

Joist Negative Reinforcement [46'/43' Span (24") Joists and 30' Span (16") Joists]

Mu- = -199 ft-k
fy = 60 ksi
f'c = 4 ksi
#3 Stirrups

Try (9) #5

$d_{16"}$ = 20.5"-0.75"-0.375"-0.5(0.625")
$d_{16"}$ = 19.06" ***CONTROLS**

$d_{24"}$ = 28.5"-0.75"-0.375"-0.5(0.625")
$d_{24"}$ = 27.06"

As = 9(0.31in^2) = 2.79 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{2.79in^2(60ksi)}{0.85(4ksi)(8")}$$
$$a = 6.15"$$

$$c = \frac{a}{\beta}$$
$$c = \frac{6.15"}{0.85}$$
$$c = 7.24" \qquad \leq 0.375(19.06") = 7.14" \therefore \phi \neq 0.9$$

$$\varepsilon_t = \frac{0.003(d-c)}{c}$$
$$\varepsilon_t = \frac{0.003(19.06"-7.24")}{7.24"}$$
$$\varepsilon_t = 0.004898$$

$$\varepsilon_y = \frac{f_y}{E_s}$$
$$\varepsilon_y = \frac{60ksi}{29{,}000ksi}$$
$$\varepsilon_y = 0.002069$$

$$\phi = 0.65 + 0.25\left(\frac{\varepsilon_t - \varepsilon_y}{0.005 - \varepsilon_y}\right)$$
$$\phi = 0.65 + 0.25\left(\frac{0.004898 - 0.002069}{0.005 - 0.002069}\right)$$
$$\phi = 0.891$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.891(2.79 in^2)(60 ksi)\left(19.06'' - \frac{6.15''}{2}\right)$$
$$\phi M_n = 2384..2 in - k$$
$$\phi M_n = 199 ft - k$$

$$\phi M_n = 199 ft - k \geq M_u = 199 ft - k \therefore OK$$

$$b_{\min} = 9(0.625'') + 8(1'') + 1(0.375'')$$
$$b_{\min} = 14'' \leq 48'' \therefore OK$$

Use (9) #5 top bars

Joist Shear Calculations

Joist Shear

$V_u = 33.9\ K$

$\boxed{24\text{" Joists}}$ $d = 25.99"$

$\phi V_n = 0.5(0.75)(2)\sqrt{f'c}\ b_w d$
$= 0.5(0.75)(2)\sqrt{4000}\ (8")(25.99")$
$= 9.9K < 33.9K$ ∴ stirrups required

$V_s = \frac{A_s f_y d}{s}$

$V_s = \frac{0.11(60)(25.99")}{12"}$

$V_s = 14.3K$

$S_{max} = \frac{d}{2} = \frac{25.99"}{2} = 13"$

$V_c = (2)\sqrt{4000}(8")(25.99")$
$V_c = 26.3K$

$\phi V_n = V_c + V_s = 26.3K + 14.3K = 40.6K > 33.9K$ ∴ OK

$\boxed{\text{use single leg \#3 stirrups @ 12" o.c.}}$

$\boxed{16\text{" Joists}}$ $d = 18.25"$

$\phi V_n = 0.5(0.75)(2)\sqrt{4000}\ (8)(18.25)$
$= 6.9K$

$V_s = \frac{0.11(60)(18.25)}{6"}$

$V_s = 20.1\ K$

$V_c = 2\sqrt{4000}\ (8)(18.25) = 18.5K$

$\phi V_n = 20.1\ K + 18.5K = 38.6K > 33.9K$ ∴ OK

$\boxed{\text{use single leg \#3 stirrups @ 6" o.c.}}$

Joist Stirrups

46'/43' Span (24") Joists	#3 stirrups @12" o.c.
30' Span (16") Joists	#3 stirrups @6" o.c.

Joist Deflection Calculations

46'/43' Span (24") Joists

L = 46'
D = 536 PLF
L = 400 PLF
f'c = 4,000 psi
I = 32,297 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$
$$E = (150PCF)^{1.5}(33)(\sqrt{4000}psi$$
$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$
$$\Delta_{TL,allow} = \frac{46'(12''/ft)}{360}$$
$$\Delta_{TL,allow} = 1.5''$$

$$\Delta_{LL,allow} = \frac{l}{480}$$
$$\Delta_{LL,allow} = \frac{46'(12''/ft)}{480}$$
$$\Delta_{LL,allow} = 1.15''$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$
$$\Delta_{TL} = \frac{5(78lb/in)(552in)^4}{384(3{,}834{,}254psi)(32{,}297in^4)}$$
$$\Delta_{TL} = 0.76'' < 1.5'' \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$

$$\Delta_{LL} = \frac{5(33.3lb/in)(552in)^4}{384(3{,}834{,}254\,psi)(32{,}297in^4)}$$

$$\Delta_{LL} = 0.325'' < 1.15'' \therefore OK$$

30' Span (16") Joists

L = 30'
D = 440 PLF
L = 600 PLF
f'c = 4,000 psi
I = 12,128 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$

$$E = (150PCF)^{1.5}(33)(\sqrt{4000}psi$$

$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$

$$\Delta_{TL,allow} = \frac{30'(12''/ft)}{360}$$

$$\Delta_{TL,allow} = 1''$$

$$\Delta_{LL,allow} = \frac{l}{480}$$

$$\Delta_{LL,allow} = \frac{30'(12''/ft)}{480}$$

$$\Delta_{LL,allow} = 0.75''$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$

$$\Delta_{TL} = \frac{5(86.7lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(12{,}128in^4)}$$

$$\Delta_{TL} = 0.41'' < 1'' \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$

$$\Delta_{LL} = \frac{5(50lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(12{,}128in^4)}$$

$$\Delta_{LL} = 0.24'' < 0.75'' \therefore OK$$

Summary of Actual and Allowable Loads

46'/43' Span (24") Joists   24 + 8 + 40 Joists

$\mathbf{M_u^+}$	279 ft-k	ϕM_n^+	292 ft-k	OK
$\mathbf{M_u^-}$	199 ft-k	ϕM_n^-	199 ft-k	OK
$\mathbf{V_u}$	33.9 k	ϕV_n	40.6 k	OK
$\mathbf{\Delta_{TL}}$	0.75"	$\mathbf{\Delta_{TL,allow}}$ **(l/360)**	1.5"	OK
$\mathbf{\Delta_{LL}}$	0.325"	$\mathbf{\Delta_{TL,allow}}$ **(l/480)**	1.15"	OK

30' Span (16") Joists   16 + 8 + 40 Joists

$\mathbf{M_u^+}$	23.65 ft-k	ϕM_n^+	36 ft-k	OK
$\mathbf{M_u^-}$	199 ft-k	ϕM_n^-	199 ft-k	OK
$\mathbf{V_u}$	33.9 k	ϕV_n	38.6	OK
$\mathbf{\Delta_{TL}}$	0.41"	$\mathbf{\Delta_{TL,allow}}$ **(l/360)**	1"	OK
$\mathbf{\Delta_{LL}}$	0.24"	$\mathbf{\Delta_{TL,allow}}$ **(l/480)**	0.75"	OK

Appendix B

CIP Girder Design Calculations

Interior Girder Design

Loads

Dead

Self Weight: $\left(\frac{24''}{12''/ft}\right)\left(\frac{24''}{12''/ft}\right)(150PCF) = 600PLF$

SDL: $15PSF\left(\frac{46'+30'}{2}\right) = 570PLF$

Joists: 95PSF(15') + 119PSF(23') = 4162 PLF

Live: ~~100PSF(23')~~ + 150PSF(15') = ~~4550 PLF~~
56PSF(23') + 150PSF(15') = 3538 PLF

$$A_T = 38'(30') = 1140SF$$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{2x1140SF}}\right)$$

$$L = 0.56L_o$$

w_D = 600 PLF + 570 PLF + 4162 PLF = 5332 PLF = 5.33 KLF
w_L = 3538 PLF = 3.6 KLF

Moments

l_n = 30'
w_D = 5.33 KLF
w_L = 3.6 KLF

ACI 8.3.3 Moments	Dead	Live	Seismic
$\pm\frac{wl^2}{16}$	300 ft-k	203 ft-k	33.5 ft-k
$+\frac{wl^2}{14}$	343 ft-k	231 ft-k	33.5 ft-k
$-\frac{wl^2}{10}$	480 ft-k	324 ft-k	33.5 ft-k
$-\frac{wl^2}{11}$	436 ft-k	295 ft-k	33.5 ft-k

Red = Worst Case (+) Moment
Blue= Worst Case (-) Moment

Load Combinations

1. 1.2D + 1.6L
2. 1.23D + L + E
3. 0.93D + E

$$E = \rho Q_E + 0.2 S_{DS} D$$
$$E = (1) Q_E + 0.2(0.153) D$$
$$E = Q_E + 0.03 D$$

(+) Moment = 782 ft-k	
Load Case 1	**782 ft-k**
Load Case 2	686 ft-k
Load Case 3	352 ft-k

(-) Moment = -1094 ft-k	
Load Case 1	**-1094 ft-k**
Load Case 2	-948 ft-k
Load Case 3	-480 ft-k

Determine B_{eff}

$16h_f + b_w = 16(4.5") + 24"$ = 96"

$\frac{L}{4} = \frac{30'(12''/ft)}{4}$ = 90" *CONTROLS

Web spacing = 28'(12"/ft)/2 = 168"

Flexural Reinforcement

(+)M

+M = 782 ft-k
f'c = 4 ksi
fy = 60 ksi
#5 stirrups
1.5" cover – bottom
0.75" cover – top

Try (8) #9

d = 28.5" – 1.5" – 0.625" – 0.5(1.25")
d = 25.81"

$A_s = 8(1\ in^2)$
$A_s = 8\ in^2$

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{8in^2(60ksi)}{0.85(4ksi)(90")}$$
$$a = 1.57"$$

$$c = \frac{a}{\beta}$$
$$c = \frac{1.57"}{0.85}$$
$$c = 1.85" \qquad \leq 0.375(25.81") = 9.68" \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(8in^2)(60ksi)\left(25.81" - \frac{1.57"}{2}\right)$$
$$\phi M_n = 10{,}810 in - k$$
$$\phi M_n = 901 ft - k$$

$$\phi M_n = 901 ft - k \geq M_u = 782 ft - k \therefore OK$$

$$b_{\min} = 2(1.5") + 2(0.625") + 8(1.125") + 7(1.125")$$
$$b_{\min} = 21.13" \leq 24" \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c} b_w d}{f_y}$$
$$A_{s,\min} = \frac{3\sqrt{4000} psi(24")(25.81")}{60000 psi}$$
$$A_{s,\min} = 1.96 in^2 < 8 in^2 \therefore OK$$

$$A_{s,\min} = \frac{200 b_w d}{f_y}$$
$$A_{s,\min} = \frac{200(24")(25.81")}{60000 psi}$$
$$A_{s,\min} = 2.06 in^2 < 8 in^2 \therefore OK$$

Use (8) #9 bottom bars

(-)M

-M = 1094 ft-k
f'c = 4 ksi
fy = 60 ksi
#5 stirrups
1.5" cover – bottom
0.75" cover – top

Try (12) #9

d = 28.5" – 0.75" – 0.625" - 0.625" – 0.5(1.125")
d = 25.94"

A_s = 12(1 in^2)
A_s = 12 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$

$$a = \frac{12in^2(60ksi)}{0.85(4ksi)(24'')}$$

$$a = 8.82''$$

$$c = \frac{a}{\beta}$$

$$c = \frac{8.82''}{0.85}$$

$$c = 10.38'' \qquad \leq 0.375(26.56'') = 9.73'' \therefore \phi \neq 0.9$$

$$\varepsilon_t = \frac{0.003(d-c)}{c}$$

$$\varepsilon_t = \frac{0.003(25.94''-10.38'')}{10.38''}$$

$$\varepsilon_t = 0.004497$$

$$\varepsilon_y = \frac{f_y}{E_s}$$

$$\varepsilon_y = \frac{60ksi}{29{,}000ksi}$$

$$\varepsilon_y = 0.002069$$

$$\phi = 0.65 + 0.25\left(\frac{\varepsilon_t - \varepsilon_y}{0.005 - \varepsilon_y}\right)$$

$$\phi = 0.65 + 0.25\left(\frac{0.004497 - 0.002069}{0.005 - 0.002069}\right)$$

$$\phi = 0.86$$

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

$$\phi M_n = 0.8(12in^2)(60ksi)\left(25.94'' - \frac{8.82''}{2}\right)$$

$$\phi M_n = 13{,}332 in - k$$

$$\phi M_n = 1{,}111 ft - k$$

$$\phi M_n = 1{,}111 ft - k \geq M_u = 1{,}094 ft - k \therefore OK$$

$$b_{\min} = 2(1.5'') + 2(0.625'') + 12(1.125'') + 11(1.125'')$$

$$b_{\min} = 30.125'' \leq 90'' \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c}\, b_w d}{f_y}$$

$$A_{s,\min} = \frac{3\sqrt{4000}\, psi(24'')(25.94'')}{60000\, psi}$$

$$A_{s,\min} = 1.97 in^2 < 12 in^2 \therefore OK$$

$$A_{s,\min} = \frac{200 b_w d}{f_y}$$

$$A_{s,\min} = \frac{200(24'')(25.94'')}{60000\, psi}$$

$$A_{s,\min} = 2.08 in^2 < 12 in^2 \therefore OK$$

Use (12) #9 top bars

Interior Girder Shear/Torsion Reinforcement

w_u = 1.2D + 1.6L
w_u = 1.2(5.33 klf) + 1.6(3.6 klf)
w_u = 12.16 klf

Stirrups

Shear

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(12.2klf)(28.5')}{2} = 200k * CONTROLS$$
$$V_u = \frac{12.2klf(28.5')}{2} = 174k$$

$$V_c = 2\sqrt{f'c}b_w d$$
$$V_c = 2\sqrt{4000}psi(24")(25.81")$$
$$V_c = 78.4k$$

$$0.5\phi V = 0.5(0.75)V_c$$
$$0.5\phi V_n = 0.5(0.75)(78.4k)$$
$$0.5\phi V_n = 29.4k < 200k \therefore Stirrups\ \mathrm{Re}quired$$

$$V_s = \frac{V_u}{\phi} - V_c$$
$$V_s = \frac{200k}{0.75} - 78.4k$$
$$V_s = 188.3k$$

Try #5 Stirrups with two legs $\quad$ $A_v = 2(0.31in^2) = 0.62in^2$

$$V_s = \frac{A_v f_y d}{s}$$

$$s = \frac{A_v f_y d}{V_s}$$

$$s = \frac{0.62in^2(60ksi)(25.81")}{188.3k}$$

$$s = 10.2" \rightarrow 4"$$

$$\Rightarrow A_V = 0.49in^2$$

Torsion

M_{joists} = 227 ft-k – 87.6 ft-k = 139.4 ft-k

$$T_u = \frac{139.4\,ft-k}{2} = 69.7\,ft-k$$

$$T_{n,req} = \frac{69.7\,ft-k}{0.75} = 92.9\,ft-k$$

$$T_n = \frac{2A_o A_t f_{yv} \cot\theta}{s}$$

θ = 45°

f_{yv} = 60 ksi

p_h = 2[28.5” – 2(1.5”) – 2(0.625”)] + 2[24” – 2(1.5”) – 2(0.625”)] = 88”

A_{oh} = 24.25”(19.75”) = 479 in^2

A_o = 0.83A_{oh} = 0.85(479 in^2) = 407 in^2

$$A_T = \frac{T_{n,req}\,s}{2A_o f_{yv} \cot\theta}$$

$$A_T = \frac{92.9\,ft-k(4")}{2(407in^2)(60ksi)\cot(45)} = 0.0076in^2$$

$$A = 2A_T + A_V$$

$$A = 2(0.0076in^2) + 0.49in^2$$

$$A = 0.51in^2 < 0.62in^2 \therefore OK$$

$$A_{s,\min} = \frac{50 b_w s}{f_y}$$

$$A_{s,\min} = \frac{50(24")(4")}{60000\,psi}$$

$$A_{s,\min} = 0.08 in^2 \leq 0.62 in^2 \therefore OK$$

Use #5 closed stirrups with two legs at 4" o.c.

Longitudinal Reinforcement

s = 12"

$$A_l = \frac{p_h \cot^2 \theta A_T}{s}$$

$$A_T = \frac{88" \cot^2(45)(0.0076 in^2)}{12"}$$

$$A_T = 0.056 in^2 \leq A_{\#4} = 0.2 in^2$$

Use #4 longitudinal reinforcement at 12" spacing

Interior Girder Deflection Calculations

L = 30'
D = 5332 PLF
L = 3538 PLF
f'c = 4,000 psi
I = 46,299 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$

$$E = (150PCF)^{1.5}(33)(\sqrt{4000}psi$$

$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$

$$\Delta_{TL,allow} = \frac{30'(12''/ft)}{360}$$

$$\Delta_{TL,allow} = 1''$$

$$\Delta_{LL,allow} = \frac{l}{480}$$

$$\Delta_{LL,allow} = \frac{30'(12''/ft)}{480}$$

$$\Delta_{LL,allow} = 0.75''$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$

$$\Delta_{TL} = \frac{5(739lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(46{,}299in^4)}$$

$$\Delta_{TL} = 0.91'' < 1'' \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$

$$\Delta_{LL} = \frac{5(295lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(46{,}299in^4)}$$

$$\Delta_{LL} = 0.36'' < 0.75'' \therefore OK$$

Exterior Girder Design

Loads

Dead

Self Weight: $\left(\frac{24''}{12''/ft}\right)\left(\frac{16''}{12''/ft}\right)(150PCF) = 400PLF$

SDL: $15PSF\left(\frac{46'}{2}\right) = 345PLF$

Joists: 119 PSF(23’) = 2,737 PLF

Façade: $20PSF\left(\frac{18'+13'}{2}\right) = 310PLF$

Live: ~~100PSF(23’) = 2,300 PLF~~
65 PSF(23’) = 1,495 PLF

$$A_T = 38'(23') = 690SF$$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{2x690SF}}\right)$$

$$L = 0.65L_o$$

w_D = 400 PLF + 345 PLF + 2737 PLF + 310 PLF = 3792 PLF = 3.8 KLF
w_L = 1495 PLF = 1.5 KLF

Moments

l_n = 30'
w_D = 3.8 KLF
w_L = 1.5 KLF

ACI 8.3.3 Moments	Dead	Live	Seismic
$\pm\frac{wl^2}{16}$	214 ft-k	85 ft-k	21 ft-k
$+\frac{wl^2}{14}$	244 ft-k	97 ft-k	21 ft-k
$-\frac{wl^2}{10}$	342 ft-k	135 ft-k	21 ft-k
$-\frac{wl^2}{11}$	311 ft-k	123 ft-k	21 ft-k

Red = Worst Case (+) Moment
Blue= Worst Case (-) Moment

Load Combinations

1. 1.2D + 1.6L
2. 1.23D + L + E
3. 0.93D + E

$$E = \rho Q_E + 0.2 S_{DS} D$$
$$E = (1)Q_E + 0.2(0.153)D$$
$$E = Q_E + 0.03D$$

(+) Moment = 448 ft-k	
Load Case 1	**448 ft-k**
Load Case 2	419 ft-k
Load Case 3	248 ft-k

(-) Moment = -627 ft-k	
Load Case 1	**-627 ft-k**
Load Case 2	-577 ft-k
Load Case 3	-339 ft-k

Determine B_{eff}

$6h_f$ = 6(4.5") = 27" *CONTROLS
½ clear span = 0.5(28')(12"/ft) = 168"
½ to next beam = 0.5(46')(12"/ft) = 276"

Flexural Reinforcement

(+)M

+M = 448 ft-k
f'c = 4 ksi
fy = 60 ksi
#4 stirrups
1.5" cover – bottom
0.75" cover – top

Try (6) #8

d = 28.5" – 1.5" – 0.5" – 0.5(1")
d = 26"

A_s = 6(0.79 in^2)
A_s = 4.74 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{4.74 in^2 (60ksi)}{0.85(4ksi)(27'')}$$
$$a = 3.1''$$

$$c = \frac{a}{\beta}$$
$$c = \frac{3.1''}{0.85}$$
$$c = 3.64'' \qquad \le 0.375(26'') = 9.75'' \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(4.74in^2)(60ksi)\left(26'' - \frac{3.1''}{2}\right)$$
$$\phi M_n = 6258 in - k$$
$$\phi M_n = 522 ft - k$$

$$\phi M_n = 522 ft - k \ge M_u = 448 ft - k \therefore OK$$

$$b_{\min} = 2(1.5'') + 2(0.5'') + 6(1'') + 5(1'')$$
$$b_{\min} = 15'' \le 16'' \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c}b_w d}{f_y}$$

$$A_{s,\min} = \frac{3\sqrt{4000}psi(16")(26")}{60000psi}$$

$$A_{s,\min} = 1.32in^2 < 4.74in^2 \therefore OK$$

$$A_{s,\min} = \frac{200b_w d}{f_y}$$

$$A_{s,\min} = \frac{200(16")(26")}{60000psi}$$

$$A_{s,\min} = 1.39in^2 < 4.74in^2 \therefore OK$$

Use (6) #8 bottom bars

(-)M

-M = 627 ft-k
f'c = 4 ksi
fy = 60 ksi
#4 stirrups
1.5" cover – bottom
0.75" cover – top

Try (8) #8

d = 28.5" – 0.75" – 0.5" – 0.625" - 0.5(1")
d = 26.13"

A_s = 8(0.79 in^2)
A_s = 6.32 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$

$$a = \frac{6.32in^2(60ksi)}{0.85(4ksi)(16")}$$

$$a = 6.97"$$

$$c = \frac{a}{\beta}$$

$$c = \frac{6.97"}{0.85}$$

$$c = 8.2" \qquad \leq 0.375(26.13") = 9.8" \therefore \phi = 0.9$$

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

$$\phi M_n = 0.9(6.32in^2)(60ksi)\left(26.13" - \frac{6.97"}{2}\right)$$

$$\phi M_n = 7{,}728 in - k$$

$$\phi M_n = 644 ft - k$$

$$\phi M_n = 644 ft - k \geq M_u = 627 ft - k \therefore OK$$

$$b_{min} = 2(1.5") + 2(0.5") + 8(1") + 7(1")$$

$$b_{min} = 19" \leq 27" \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c}b_w d}{f_y}$$

$$A_{s,\min} = \frac{3\sqrt{4000}psi(16'')(26.13'')}{60000psi}$$

$$A_{s,\min} = 1.32in^2 < 6.32in^2 \therefore OK$$

$$A_{s,\min} = \frac{200b_w d}{f_y}$$

$$A_{s,\min} = \frac{200(16'')(26.13'')}{60000psi}$$

$$A_{s,\min} = 1.39in^2 > 6.32in^2 \therefore OK$$

Use (8) #8 top bars

Exterior Girder Shear/Torsion Reinforcement

w_u = 1.2D + 1.6L
w_u = 1.2(3.8 klf) + 1.6(1.5 klf) = 7 klf

Stirrups

Shear

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(7klf)(29')}{2} = 115k * CONTROLS$$
$$V_u = \frac{7klf(29')}{2} = 101k$$

$$V_c = 2\sqrt{f'c}b_w d$$
$$V_c = 2\sqrt{4000}psi(16'')(26'')$$
$$V_c = 52.6k$$

$$0.5\phi V = 0.5(0.75)V_c$$
$$0.5\phi V_n = 0.5(0.75)(52.6k)$$
$$0.5\phi V_n = 19.73k < 115k \therefore Stirrups \operatorname{Re} quired$$

$$V_s = \frac{V_u}{\phi} - V_c$$
$$V_s = \frac{115k}{0.75} - 52.6k$$
$$V_s = 100.73k$$

Try #4 Stirrups with two legs $\quad$ $A_v = 2(0.2in^2) = 0.4in^2$

$$V_s = \frac{A_v f_y d}{s}$$
$$s = \frac{A_v f_y d}{V_s}$$
$$s = \frac{0.4in^2(60ksi)(26'')}{100.73k}$$
$$s = 6.19'' \to 5''$$

Use 5” spacing $\quad \Rightarrow A_v = 0.321in^2$

Torsion

M_{joists} = 227 ft-k

$$T_u = \frac{227\,ft-k}{2} = 113.5\,ft-k$$
$$T_{n,req} = \frac{113.5\,ft-k}{0.75} = 151.3\,ft-k$$

$$T_n = \frac{2A_o A_t f_{yv} \cot\theta}{s}$$

$\theta = 45°$

f_{yv} = 60 ksi

p_h = 2[28.5" – 2(1.5") – 2(0.5")] + 2[20" – 2(1.5") – 2(0.5")] = 73"

A_{oh} = 24.5"(12") = 294 in^2

A_o = 0.83A_{oh} = 0.85(294) = 250 in^2

$$A_T = \frac{T_{n,req}\,s}{2A_o f_{yv} \cot\theta}$$
$$A_T = \frac{151.3\,ft-k(5")}{2(250in^2)(60ksi)\cot(45)} = 0.025in^2$$

$$A = 2A_T + A_V$$
$$A = 2(0.025in^2) + 0.321in^2$$
$$A = 0.37in^2 < 0.4in^2 \therefore OK$$

$$A_{s,\min} = \frac{50 b_w s}{f_y}$$
$$A_{s,\min} = \frac{50(16")(5")}{60000\,psi}$$
$$A_{s,\min} = 0.067in^2 \le 0.4in^2 \therefore OK$$

Use #4 closed stirrups with two legs at 5" o.c.

Longitudinal Reinforcement

s = 12”

$$A_l = \frac{p_h \cot^2 \theta A_T}{s}$$

$$A_T = \frac{73'' \cot^2(45)(0.025in^2)}{12''}$$

$$A_T = 0.152in^2 \leq A_{\#4} = 0.2in^2$$

Use #4 longitudinal reinforcement at 12” spacing

Exterior Girder Deflection Calculations

L = 30’
D = 3792 PLF
L = 1495 PLF
f’c = 4,000 psi
I = 30,866 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$
$$E = (150PCF)^{1.5}(33)(\sqrt{4000}psi$$
$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$
$$\Delta_{TL,allow} = \frac{30'(12''/ft)}{360}$$
$$\Delta_{TL,allow} = 1''$$

$$\Delta_{LL,allow} = \frac{l}{480}$$
$$\Delta_{LL,allow} = \frac{30'(12''/ft)}{480}$$
$$\Delta_{LL,allow} = 0.75''$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$
$$\Delta_{TL} = \frac{5(441lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(30{,}866in^4)}$$
$$\Delta_{TL} = 0.81'' < 1'' \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$
$$\Delta_{LL} = \frac{5(125lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(30{,}866in^4)}$$
$$\Delta_{LL} = 0.23'' < 0.75'' \therefore OK$$

Summary of Actual and Allowable Loads

Interior Girders

M_u^+	782 ft-k	ϕM_n^+	900 ft-k	OK
M_u^-	1094 ft-k	ϕM_n^-	1111 ft-k	OK
V_u	200 k	ϕV_n	318 k	OK
T_u	69.7 ft-k	ϕT_n	92.8 ft-k	OK
Δ_{TL}	0.91”	**$\Delta_{TL,allow}$ (l/360)**	1”	OK
Δ_{LL}	0.36”	**$\Delta_{TL,allow}$ (l/480)**	0.75”	OK

Exterior Girders

M_u^+	448 ft-k	ϕM_n^+	522 ft-k	OK
M_u^-	627 ft-k	ϕM_n^-	644 ft-k	OK
V_u	115 k	ϕV_n	177 k	OK
T_u	114 ft-k	ϕT_n	150 ft-k	OK
Δ_{TL}	0.81”	**$\Delta_{TL,allow}$ (l/360)**	1”	OK
Δ_{LL}	0.23”	**$\Delta_{TL,allow}$ (l/480)**	0.75”	OK

Appendix C

CIP Column Design Calculations

Column Loads

Dead

Façade	
Glass Curtain Wall	15 PSF
Precast	20 PSF
Roof	38 PSF
Typical Floor	
Joists (24" Joists)	119 PSF
Joists (16" Joists)	95 PSF
SDL	15 PSF
Girders	600 PLF

Live

Roof	
Snow	30 PSF
Mechanical	150 PSF
Typical Floor	
Office	100 PSF
Mechanical	150 PSF

Load Combinations

1.2D + 1.6L *CONTROLS
1.23D + L + E
0.93D + E

Example of a Column Axial Load Spreadsheet

Level	DL (k)	LL (k)	RedLL (k)	1.2DL (k)	1.6RedLL (k)
Roof	51.8	20.5	11.0	62.2	17.6
12	117.5	68.4	36.7	141.0	58.7
11	117.5	68.4	36.7	141.0	58.7
10	117.5	68.4	36.7	141.0	58.7
9	117.5	68.4	36.7	141.0	58.7
8	117.5	68.4	36.7	141.0	58.7
7	117.5	68.4	36.7	141.0	58.7
6	117.5	68.4	36.7	141.0	58.7
5	117.5	68.4	36.7	141.0	58.7
4	117.5	68.4	36.7	141.0	58.7
3	117.5	68.4	36.7	141.0	58.7
2	120.5	68.4	36.7	144.6	58.7
	1347	773	415	1617	664

Column 5,4,28,27,26		
A_T =	684	SF
K_{LL}=	4	
L=	0.54	Lo

Max Pu	2281	k

	1.2DL (k)	1.6LL (k)	Pu (k)
10-12	344	135	479
6-12	908	370	1278
1-12	1617	664	2281

Appendix D

CIP Shearwall Design Calculations

Shearwall Reinforcement Calculations

1

Shearwall Reinforcement

$f'c = 4000$ psi

8" thick
30' long (horizontally)

$M_u = 34{,}501$ FT·K

$I_{n.a.} = \frac{(0.667')(30')^3}{12} = 1500 \text{ FT}^4$

$A_g = 0.667'(30') = 20 \text{FT}^2$

$f_c = \frac{P_u}{A_g} + \frac{M_u h_w/2}{I_{n.a.}}$

$= \frac{4661 \text{K}}{20 \text{ FT}^2} + \frac{34{,}501 \text{ FT·K}(30')/2}{1500 \text{ FT}^4}$

$= 4.01 \text{ KSi} > 0.2(4 \text{ KSi}) = 0.8 \text{ KSi}$

∴ boundary elements needed

$V_u = 370$ K

$2A_{cv}\sqrt{f'c} = \frac{2(8'')(30' \times 12''/\text{ft})\sqrt{4000}}{1000} = 364\text{K} < 370\text{K}$

∴ Two curtains reinf. req'd

Req'd longitudinal and transverse reinforcement in wall

$\rho_v = \frac{A_{sv}}{A_{cv}} = \rho_n > 0.0025$ max

$A_{cv} = 8(12) = 96 \text{ in}^2/\text{ft}$ of wall

$0.0025(96) = 0.24 \text{ in}^2/\text{ft}$

use #4 bars in two curtains
$A_s = 0.2(2) = 0.4 \text{ in}^2 > 0.24 \text{ in}^2$ ∴ OK

$S_{req'd} = \frac{2(0.2 \text{ in}^2)(12''/\text{ft})}{0.24} = 20'' \rightarrow 18''$

$S = \frac{l_w}{5} = 72''$ or 18" * controls

2

Determine reinforcement for shear

Assume #4 bars @ 18" e.w.

$$\frac{h_w}{l_w} = \frac{180.75'}{30'} = 6.03 > 2$$

$$\rho_n = \frac{2(0.2)}{8(12)} = 0.004167$$

$$\phi = 0.6$$

$$A_{cv} = 8(30 \cdot 12) = 2880 \text{ in}^2$$

$$\phi V_n = \phi A_{cv} \left(2\sqrt{f'_c} + \rho_n f_y\right)$$
$$= 0.6(8)(30 \cdot 12)\left(2\sqrt{4000} + 0.004167 \cdot 60000\right)$$
$$= 651 \text{ K} > V_u = 370 \text{K}$$

Use (2) curtains of #4 bars spaced at 18" o.c. in both horizontal and vertical directions

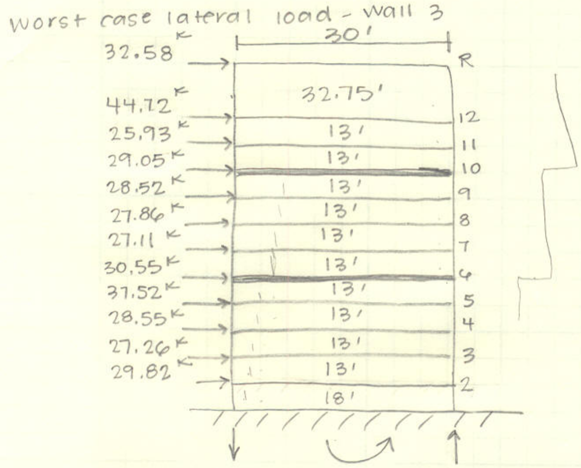
$$b_{min} = 0.75(2) + 0.5(2) + 0.5(2) + 1(1) = 4.5"$$

Shearwall Boundary Element Design Calculations

1

Shearwall Boundary Element Design

worst case lateral load - wall 3



$h_{1-12} = 180.75'$

$h_{6-12} = 110.75'$

$h_{10-12} = 58.75'$

$M_{1-12} = 29.82(18') + 27.26(31') + 28.55(44') + 37.52(57') + 30.55(70') + 27.11(83') + 27.86(96') + 28.52(109') + 29.05(122') + 25.93(135') + 44.72(148') + 32.58(180.75')$

$M_{1-12} = 34{,}501$ FT·K (factored already)

$M_{6-12} = 27.11(13') + 27.86(26') + 28.52(39') + 29.05(52') + 25.93(65') + 44.72(78') + 32.58(110.75')$
$= 12{,}482$ FT·K (factored already)

$M_{10-12} = 25.93(13') + 44.72(26') + 32.58(58.75')$
$= 3{,}414$ FT·K

$P_{w,u,1-12} = \dfrac{34{,}501}{30'} = 1{,}151$ K

$P_{wu,6-12} = \dfrac{12{,}482 \text{ FT·K}}{30'} = 417$ K

$P_{wu,10-12} = \dfrac{3{,}414}{30'} = 114$ K

2

col. 10, 11, 12, 19, 20, 21

	DL	LLR	1.6W	Mx	My
10-12	376.4	180.3	114	92	115
6-12	1005.2	452.7	417	92	115
1-12	1791.2	793.2	1151	92	115

Load combinations:

	1-12	6-12	10-12
1.4D	2508		
1.2D + 1.6L	3419		
1.2D + 0.8W	2725		
1.2D + 1.6W + L	4094	2076	746

col. 10, 11, 12, 19, 20, 21

Top
Pu = 746 K
Mx = 92 FT·K
My = 115 FT·K
Pu = 770 K

self = 19.9 K 1.2self = 24 K

$\boxed{18\times18 \; (4)\#9}$

Mid
Pu = 2100
Mx = 92
My = 115
Pu = 2159

self = 49 1.2self = 59

$\boxed{30\times30 \; (8)\#10}$

Bot.
Pu = 4177
Mx = 92
My = 115
Pu = 4332

self = 95 1.2self = 114

$\boxed{42\times42 \; (20)\#9}$

$\phi Pn = 4448$
$\phi Mnx = 93$
$\phi Mny = 116$

Hand calc. capacity check

$A_g = (36)(36) = 1296 \text{ in}^2$

$A_{st} = 43.68 \text{ in}^2$

$\rho_s = \frac{43.68}{1296} = 0.033704$ > 0.01, < 0.06

$\phi P_n = 0.8(0.7)[0.85(5\text{ ksi})(1296 - 43.68) + 60(43.68)]$

$\phi P_n = 4{,}448 \text{ K} = 4{,}448 \text{ K}$ ∴ OK

3

col. 13,18

	DL	LLR	1.6W	Mx	My
10-12	218	102	114	92	589
6-12	800	362	417	92	589
1-12	1585	714	1151	92	589

Load combinations	1-12	6-12	10-12
1.4D	2219		
1.2D+1.6L	3045		
1.2D+0.8W	2478		
1.2D+1.6W+L	3767	1739	478

col. 13,18

TOP

$P_u = 478$
$M_x = 92$
$M_y = 589$
$P_u = 521$

self - 35.4 1.2 self = 42.5

24x24
12-#9

Mid

$P_u = 1739 + 43 = 1782$
$M_x = 92$
$M_y = 589$
$P_u = 1841$

self 1.2 = 59

30x30
12-#8

Bot.

$P_u = 3767 + 43 + 59 = 3869$
$M_x = 92$
$M_y = 589$
$P_u = 3983$

self- 1.2 = 114

36x36
24-#10

$\phi P_n = 4036$ K
$\phi M_{nx} = 92$ FT·K
$\phi M_{ny} = 587$ FT·K

Hand calc: capacity check:

$A_g = 36(36) = 1296\ in^2$
$A_{st} = 30.48$
$\rho = \frac{30.48}{1296} = 0.0235 \geq 0.01,\ \leq 0.06$

$\phi P_n = 0.8(0.7)[0.85(5)(1296-30.48)+60(30.48)]$
$= 4036\text{ K} > 3983 \therefore$ OK
$= 4{,}036\text{ K} \therefore$ OK

Shearwall Deflections

Wall	Max Δx		Max Δy		Max Δz	
	E/S	W/N	E/S	W/N	E/S	W/N
1	2.053982”	2.032261”	1.503888”	1.503888”	0.164363”	-0.161891”
2	2.053982”	2.032261”	1.547330”	1.547330”	0.012486”	-0.289300”
3	2.053982”	2.032261”	1.570137”	1.570137”	0.148668”	-0.150556”
4	2.053982”	2.053982”	1.526333”	1.547330”	0.424625”	0.012286”
5	2.032261”	2.032261”	1.526333”	1.547330”	0.137942”	-0.289300”

$$\Delta_{allow} = \frac{h}{400} = \frac{180.75'(12''/ft)}{400} = 5.42''$$

All shearwall deflections are less than the allowable 5.42” ∴ OK

Appendix E

Representative Spread Footing Design Calculations

Spread Footing Design – E-7

Loads

Axial Loads from Tower

D = 1956 k
LLr = 794 k
W = 720 k

Garage Loads

Level	LL	SDL	Slab Thickness	Drop Size
1	100 PSF	15 PSF	9”	10’x10’x5.5”
G1	50 PSF	15 PSF	8”	10’x10’x5.5”
G2	50 PSF	15 PSF	8”	10’x10’x5.5”
G4	50 PSF	15 PSF	4”	------------

Level 1

LL: 100(0.5) PSF(900 SF) = 45,000 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10’(10’)(5.5”/12)(150 PCF) = 6,875 LB
Slab: 9”/12(900 SF)(150 PCF) = 101,250 LB
Column: (42”x42”)/144in^2(10’)(150 PCF) = 18,375 LB

Level G1 and G2

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10’(10’)(5.5”/12)(150 PCF) = 6,875 LB
Slab: (8”/12)(900 SF)(150 PCF) = 90,000 LB
Column: (42”x42”)/144in^2(10’)(150 PCF) = 18,375 LB

Level G3

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Slab: (4”/12)(900 SF)(150 PCF) = 45,000 LB
Column: (42”x42”)/144in^2(3’)(150 PCF) = 5,513 LB

D = 462 k

L = 113 k

Service: 4,045 k
Factored: 1.2D + 1.6W + L = 4,961 k
Moments are negligible.

Footing Size and Reinforcement

$q_a = 40KSF$ (soils report)

$$q_a \geq \frac{P}{A}$$
$$40KSF \geq \frac{4045k}{B^2}$$
$$B \geq 10.1'$$
$$B = 10.5'$$
$$q = \frac{P_u}{A}$$
$$q = \frac{4961k}{(10.5')^2}$$
$$q = 45KSF$$
$$q = 312.5PSI$$

f'c = 3000 psi (Structural Notes)
42" x 42" column
Df = 36"

Try #11 bars

$$v_c = \phi 4\sqrt{f'c}$$
$$v_c = 0.75(4)\sqrt{3000}\,psi$$
$$v_c = 164\,psi$$

$$d^2\left(v_c + \frac{q}{\beta_c}\right) + d\left(v_c + \frac{q}{2}\right)w = \frac{q}{4}\left(BL - w^2\right)$$
$$d^2\left(164\,psi + \frac{312.5\,psi}{4}\right) + d\left(164\,psi + \frac{312.5\,psi}{2}\right)42'' = \frac{312.5\,psi}{4}\left(126''^2 - 42''^2\right)$$
$$d = 45.2''$$

h = 45.2" + 3" + 1.41"
h = 49.61"

h = 50”

d = 50” – 3” – 1.41” = 45.6”

$$l = \frac{10.5' - 3.5'}{2}$$
$$l = 3.5'$$

$$M_u = \frac{ql^2}{2}$$
$$M_u = \frac{45KSF(3.5')^2}{2}$$
$$M_u = 275.6\,ft - k$$

$$a = \frac{A_s f_y}{0.85 f'cb}$$
$$a = \frac{A_s(60ksi)}{0.85(3ksi)(12'')}$$
$$a = 1.96A_s$$

$$M_u = \phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$
$$275.6\,ft - k(12in/ft) = 0.9A_s(60ksi)\left(45.6'' - \frac{1.96A_s}{2}\right)$$
$$A_s \geq 1.33in^2 < 1.56in^2 \therefore OK$$

$$\rho = \frac{A_s}{bh}$$
$$\rho = \frac{1.56in^2}{12''(51'')}$$
$$\rho = 0.002549 \geq 0.0018 \therefore OK$$

a = 1.96(1.56in^2) = 3.06in

c = 3.06in/0.85 = 3.6” ≤0.375(45.6”) = 17.1” $\phi = 0.9$

$$\phi B_n = \phi(0.85) f'cA_1$$
$$\phi B_n = 0.65(0.85)(3ksi)(42")^2$$
$$\phi B_n = 2{,}924k(2) = 5{,}848k > 4{,}961k \therefore OK$$

ACI 22.5.5

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{126^2}{42^2}} = 3 \rightarrow 2$$

$$A_{s,\min} = 0.005A_{col}$$
$$A_{s,\min} = 0.005(42in)^2$$
$$A_{s,\min} = 8.82in^2 < 9.48in^2 = A_{(12)\#8}$$

Use 10.5’ x 10.5’ x 50” square footing with (11) #11 each way and (12) #8 dowels

Spread Footing Design – E-9

Loads

Axial Loads from Tower
D = 2141 k
LLr = 715 k

Garage Loads

Level	LL	SDL	Slab Thickness	Drop Size
1	100 PSF	15 PSF	9”	10’x10’x5.5”
G1	50 PSF	15 PSF	8”	10’x10’x5.5”
G2	50 PSF	15 PSF	8”	10’x10’x5.5”
G4	50 PSF	15 PSF	4”	------------

Level 1

LL: 100(0.5) PSF(900 SF) = 45,000 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10’(10’)(5.5”/12)(150 PCF) = 6,875 LB
Slab: 9”/12(900 SF)(150 PCF) = 101,250 LB
Column: (42”x42”)/144in^2(10’)(150 PCF) = 18,375 LB

Level G1 and G2

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10’(10’)(5.5”/12)(150 PCF) = 6,875 LB
Slab: (8”/12)(900 SF)(150 PCF) = 90,000 LB
Column: (42”x42”)/144in^2(10’)(150 PCF) = 18,375 LB

Level G3

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Slab: (4”/12)(900 SF)(150 PCF) = 45,000 LB
Column: (42”x42”)/144in^2(3’)(150 PCF) = 5,513 LB

D = 462 k
L = 113 k

Service: 3,431 k
Factored: 1.2D + 1.6L = 4,448 k
Moment are negligible.

Footing Reinforcement

$q_a = 40KSF$ (soils report)

$$q_a \geq \frac{P}{A}$$

$$40KSF \geq \frac{3{,}434k}{B^2}$$

$$B \geq 9.26'$$

$$B = 9.5'$$

$$q = \frac{P_u}{A}$$

$$q = \frac{4448k}{(9.5')^2}$$

$$q = 49.3ksf$$

$$q = 342.3psi$$

f'c = 3000 psi (Structural Notes)
42" x 42" column

Try #11 bars

$$v_c = \phi 4\sqrt{f'c}$$

$$v_c = 0.75(4)\sqrt{3000}\,psi$$

$$v_c = 164\,psi$$

$$d^2\left(v_c + \frac{q}{\beta_c}\right) + d\left(v_c + \frac{q}{2}\right)w = \frac{q}{4}\left(BL - w^2\right)$$

$$d^2\left(164\,psi + \frac{342.3\,psi}{4}\right) + d\left(164\,psi + \frac{342.3\,psi}{2}\right)42'' = \frac{342.3\,psi}{4}\left(114''^2 - 42''^2\right)$$

$$d = 39.96''$$

h = 39.96" + 3" + 1.41"
h = 44.41"
h = 45"

d = 45" – 3" – 1.41" = 40.6"

$$l = \frac{10' - 3.5'}{2}$$
$$l = 3.25'$$

$$M_u = \frac{ql^2}{2}$$
$$M_u = \frac{49.3KSF(3.25')^2}{2}$$
$$M_u = 260.4\,ft - k$$

$$a = \frac{A_s f_y}{0.85 f'cb}$$
$$a = \frac{A_s(60ksi)}{0.85(3ksi)(12'')}$$
$$a = 1.96A_s$$

$$M_u = \phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right)$$
$$260.4\,ft - k(12in/\,ft) = 0.9A_s(60ksi)\left(40.6'' - \frac{1.96A_s}{2}\right)$$
$$A_s \geq 1.38in^2 < 1.56in^2 \therefore OK$$

$$\rho = \frac{A_s}{bh}$$
$$\rho = \frac{1.56in^2}{12''(50'')}$$
$$\rho = 0.0026 \geq 0.0018 \therefore OK$$

a = 1.96(1.56in^2) = 3.06 in

c = 3.06 in/0.85 = 3.6” ≤0.375(40.6”) = 15.2” $\phi = 0.9$

$$\phi B_n = \phi(0.85) f'cA_1$$
$$\phi B_n = 0.65(0.85)(3ksi)(42'')^2$$
$$\phi B_n = 2{,}924k(2) = 5{,}848k > 4{,}448k \therefore OK$$

ACI 22.5.5

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{120^2}{42^2}} = 2.85 \rightarrow 2$$

$$A_{s,\min} = 0.005A_{col}$$
$$A_{s,\min} = 0.005(42in)^2$$
$$A_{s,\min} = 8.82in^2 < 9.48in^2 = A_{(12)\#8}$$

Use 9.5' x 9.5' x 45" square footing with (10) #11 each way and (12) #8 dowels

Appendix F

Roof Design Calculations

Roof Joist Calculations and Design – 46' Span

Roof Joists - snow only

span = 46'

Loads:

Dead

SDL	= 15 PSF
Insulation	= 1.5 PSF
Built-up roof	= 6.5 PSF
Metal Deck	= 2 PSF
Joists	= 16.6
	25 PSF

4.2 PSF

$\boxed{30\ PSF}$

Live

snow - = 30 PSF

D: 25 PSF(4') = 100 PLF + 16.6 PLF = 117 PLF

L: 30 PSF(4') = 120 PLF

Wu = 1.2(117) + 1.6(120)
Wu = 332.4 plf

use 26K12
d = 26"
TL = 380 > 332.4 ∴ OK
LL = 203 > 192 ∴ OK   l/360 deflection

Roof Joist Calculations and Design – 30' Span

Roof Joists - Mechanical & snow

span = 30'

Loads:

Dead

SDL	= 15 PSF	
Insulation	= 1.5 PSF	
Built-up roof	= 6.5 PSF	
Metal Deck	= 3 PSF	
Joists	= 12.1	8.1 PSF
		$\boxed{35 \text{ PSF}}$

D: 26 PSF(1.5') + 12.1 = 51

L: 30(1.5) = 45 plf
150(1.5) = 225 plf

1. 1.2D + 1.6L + 0.5S
2. 1.2D + 1.6S + L

1. 1.2(51) + 1.6(225) + 0.5(45) = 444 PLF

use 26 K8
d = 26"
TL = 544 > 444 ∴ OK
LL = 457 > 432 ∴ OK l/360 deflection

Roof Deck Calculations and Design – 46' Span

Roof Deck - snow only

Dead :

SDL:	= 15 PSF
Insulation (Rigid)	= 1.5 PSF
Built-up Roof (5 ply felt & gravel)	= 6.5 PSF
Metal Deck	= 2 PSF
	25 PSF

Live:

Snow = 30 PSF

Span = 4'

$D = 15 + 1.5 + 6.5 + 2 = 25$ PSF
$L = 30$ PSF

Load combinations

$1.4D$
$1.2D + 1.6L + 0.5S$
* $1.2D + 1.6S + L$

$W_u = 1.2(25) + 1.6(30) = 78$ PSF $<$ 224 PSF ∴ OK

$\boxed{\text{use 22 gage Type F, Intermediate Rib Deck}}$

Roof Deck Calculations and Design – 30' Span

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

Roof Deck - Mechanical

Dead

SDL	= 15 PSF
Insulation	= 1.5 PSF
Built up roof	= 6.5 PSF
Metal Deck	= 3 PSF
	26 PSF

Live

Snow = 30 PSF
Mechanical = 150 PSF

span 2'

Load combos

1. 1.2D + 1.6L + 0.5S
2. 1.2D + 1.6S + L

1. Wu = 1.2(26) + 1.6(150) + 0.5(30) = 287 # < 367
2. Wu = 1.2(26) + 1.6(30) + 150 = 230

$\boxed{\text{use 18 gage, Type F Intermediate Rib Deck}}$

Appendix G

Cost Analysis Calculations

Concrete System Cost Analysis – Typical Floor

	Cost	Joists/Slab							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0300	Concrete	1100	CY	$91.00	$0.00	$0.00	$100,100	$0	$0
03210-600-0400	Reinf.	64	Ton	$905.00	$435.00	$0.00	$57,920	$27,840	$0
03110-420-3760	Formwork	24250	SF	$2.78	$3.45	$0.00	$67,415	$83,663	$0
03310-700-1600	Placement	1100	CY	$0.00	$10.55	$4.13	$0	$11,605	$4,543
03310-300-0010	Finishing	24250	SF	$0.00	$0.31	$0.00	$0	$7,518	$0
							$225,435	**$130,625**	**$4,543**

	Cost	Girders							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0300	Concrete	117	CY	$91.00	$0.00	$0.00	$10,647	$0	$0
	Reinf.						$0	$0	$0
03210-600-0100	#3-#7	11.33	Ton	$855.00	$790.00	$0.00	$9,687	$8,951	$0
03210-600-0150	#8-#18	24.53	Ton	$855.00	$470.00	$0.00	$20,973	$11,529	$0
	Formwork						$0	$0	$0
03110-405-2650	Interior	3464	SFCA	$0.97	$4.20	$0.00	$3,360	$14,549	$0
03110-405-1650	Exterior	4208	SFCA	$0.96	$5.10	$0.00	$4,040	$21,461	$0
03110-700-0200	Placement	117	CY	$0.00	$21.00	$8.25	$0	$2,457	$965
							$48,707	**$58,946**	**$965**

	Cost	Columns							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0400	Concrete	99	CY	$96.00	$0.00	$0.00	$9,504	$0	$0
03210-600-0250	Reinf.	14.1	Ton	$855.00	$550.00	$0.00	$12,056	$7,755	$0
	Formwork								
03110-410-6650	24"x24"	728	SFCA	$0.83	$4.51	$0.00	$604	$3,283	$0
03110-410	30"x30"	1820	SFCA	$0.79	$4.40	$0.00	$1,438	$8,008	$0
03110-410-7150	36"x36"	1560	SFCA	$0.74	$4.29	$0.00	$1,154	$6,692	$0
	Placement								
03310-700-0800	24"x24"	13.5	CY	$0.00	$20.50	$8.10	$0	$277	$109
03310-700	30"x30"	42.2	CY	$0.00	$17.03	$6.70	$0	$719	$283
03310-700-1000	36"x36"	44	CY	$0.00	$13.55	$5.30	$0	$596	$233
							$24,756	**$27,330**	**$625**

	Cost	Shearwalls							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0300	Concrete	49	CY	$91.00	$0.00	$0.00	$4,459	$0	$0
03210-600-0700	Reinforcement	0.89	Ton	$810.00	$420.00	$0.00	$721	$374	$0
03110-455-8060	Formwork	3900	SFCA	$0.83	$2.10	$0.00	$3,237	$8,190	$0
03310-700-4950	Placement	49	CY	$0.00	$19.00	$7.45	$0	$931	$365
03350-350-0020	Finishing	3900	SF	$0.03	$0.51	$0.00	$117	$1,989	$0
							$8,534	**$11,484**	**$365**

	Cost	Shoring							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03150-600-1500	Reshoring	24,250	SF	$0.42	$0.41	$0.00	$10,185	$9,943	$0
03150-600-3050	Shores (12 mon.)	24,250	SF	$5.76	$0.00	$0.00	$139,680	$0	$0
							$149,865	**$9,943**	**$0**

Total Cost	Concrete		
	Material	Labor	Equipment
Joists/Slab	$225,435	$130,625	$4,543
Girders	$48,707	$58,946	$965
Columns	$24,756	$27,330	$625
Shearwalls	$8,534	$11,484	$365
Shoring/Reshoring	$149,865	$9,943	$0
	$457,297	$238,328	$6,498
	$702,123		

Concrete System Cost Analysis – Spread Footings

Cost: Square Footing E-7 Concrete System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	17	CY	$87.00	$0.00	$0.00	$1,479	$0	$0
03210-600-0550	Reinforcement	0.61	ton	$770.00	$350.00	$0.00	$470	$214	$0
03110-430-5150	Formwork	175	SFCA	$0.59	$2.59	$0.00	$103	$453	$0
03310-700-2600	Placement (direct chute)	17	CY	$0.00	$11.55	$0.36	$0	$196	$6
							$2,052	$863	$6
							$2,921		

Cost: Square Footing E-9 Concrete System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	12.53	CY	$87.00	$0.00	$0.00	$1,090	$0	$0
03210-600-0550	Reinforcement	0.53	ton	$770.00	$350.00	$0.00	$408	$186	$0
03110-430-5150	Formwork	143	SFCA	$0.59	$2.59	$0.00	$84	$370	$0
03310-700-2600	Placement (direct chute)	12.53	CY	$0.00	$11.55	$0.36	$0	$145	$5
							$1,583	$701	$5
							$2,288		

Steel System Cost Analysis – Typical Floor

Cost	Slab			Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-1010	Concrete	356	CY	$109.00	$0.00	$0.00	$38,804	$0	$0
03210-200-0100	WWF	242.5	CSF	$12.00	$18.05	$0.00	$2,910	$4,377	$0
03110-420-6650	Edge Form	199.5	SFCA	$0.50	$4.77	$0.00	$100	$952	$0
03310-700-1400	Placement	356	CY	$0.00	$13.55	$5.30	$0	$4,824	$1,887
							$41,814	**$10,153**	**$1,887**

Cost	Metal Deck		Unit Cost			Total Cost		
		Area (SF)	Material	Labor	Equip.	Material	Labor	Equip.
05310-300-5800	3"-20 gauge	24,250	$1.71	$0.43	$0.03	$41,468	$10,428	$728
						$41,468	**$10,428**	**$728**

Cost	Beams		Unit Cost			Total Cost		
		Length (ft)	Material	Labor	Equip.	Material	Labor	Equip.
05120-640-0300	W8x10	6.5	$10.45	$3.63	$2.38	$68	$24	$15
05120-640-1100	W12x14	40	$14.65	$2.48	$1.62	$586	$99	$65
05120-640-2700	W16x26	447	$27.00	$2.18	$1.43	$12,069	$974	$639
05120-640-2900	W16x31	157	$32.50	$2.42	$1.59	$5,103	$380	$250
05120-640-3300	W18x35	148	$36.50	$3.28	$1.58	$5,402	$485	$234
05120-640-3500	W18x40	122	$42.00	$3.28	$1.58	$5,124	$400	$193
05120-640-3520	W18x46	823	$48.00	$3.28	$1.58	$39,504	$2,699	$1,300
05120-640-3700	W18x50	322	$52.50	$3.46	$1.66	$16,905	$1,114	$535
05120-640-3900	W18x55	92	$57.50	$3.46	$1.66	$5,290	$318	$153
05120-640	W18x60	76	$62.75	$3.48	$1.67	$4,769	$264	$127
05120-640-3920	W18x65	136	$68.00	$5.50	$1.68	$9,248	$748	$228
05120-640-4100	W21x44	330	$46.00	$2.96	$1.42	$15,180	$977	$469
05120-640-4900	W24x55	189	$57.50	$2.84	$1.37	$10,868	$537	$259
05120-640-5100	W24x62	103	$65.00	$2.84	$1.37	$6,695	$293	$141
05120-640-5300	W24x68	37	$71.00	$2.84	$1.37	$2,627	$105	$51
05120-640-5500	W24x76	43	$79.50	$2.84	$1.37	$3,419	$122	$59
05120-640-5800	W27x84	86	$88.00	$2.65	$1.27	$7,568	$228	$109
05120-640-6300	W30x108	43	$113.00	$2.63	$1.26	$4,859	$113	$54
05120-640	W30x124	43	$129.50	$2.72	$1.31	$5,569	$117	$56
		3243.5				**$160,851**	**$9,998**	**$4,937**

Cost	Columns		Unit Cost			Total Cost		
		Length (ft)	Material	Labor	Equip.	Material	Labor	Equip.
05120-260-7200	W12x87	26	$91.00	$2.21	$1.45	$2,366	$57	$38
05120-260	W14x90	13	$102.00	$2.24	$1.47	$1,326	$29	$19
05120-260	W14x99	13	$102.00	$2.24	$1.47	$1,326	$29	$19
05120-260	W14x132	39	$139.75	$2.30	$1.51	$5,450	$90	$59
05120-260	W14x145	39	$154.50	$2.33	$1.53	$6,026	$91	$60
05120-260	W14x159	13	$169.25	$2.36	$1.54	$2,200	$31	$20
05120-260-7450	W14x176	78	$184.00	$2.39	$1.56	$14,352	$186	$122
05120-260	W14x193	39	$198.75	$2.42	$1.58	$7,751	$94	$62
05120-260	W14x211	26	$213.50	$2.45	$1.60	$5,551	$64	$42
05120-260	W14x233	26	$228.25	$2.48	$1.62	$5,935	$64	$42
05120-260	W14x257	91	$243.00	$2.51	$1.64	$22,113	$228	$149
		403				**$74,396**	**$964**	**$631**

Cost	Braced Frame Members		Unit Cost			Total Cost		
		Length (ft)	Material	Labor	Equip.	Material	Labor	Equip.
05120-640-3700	W18x50	60	$52.50	$3.46	$1.66	$3,150	$208	$100
05120-640-3920	W18x65	30	$68.00	$3.50	$1.68	$2,040	$105	$50
05120-640	W18x71	30	$73.75	$3.50	$1.68	$2,213	$105	$50
05120-640	W18x97	30	$100.50	$3.50	$1.68	$3,015	$105	$50
05120-260	HSS 8x8x3/8	48	$39.64	$3.11	$2.04	$1,903	$149	$98
05120-260	HSS 8x8x5/8	48	$39.64	$3.11	$2.04	$1,903	$149	$98
05120-260	HSS 10x10x1/2	48	$64.06	$2.84	$1.84	$3,075	$136	$88
05120-260	HSS 10x10x5/8	35	$64.06	$2.84	$1.84	$2,242	$99	$64
05120-260	HSS 12x12x1/2	35	$83.06	$2.64	$1.69	$2,907	$92	$59
		364				**$22,447**	**$1,149**	**$659**

Total Cost	**Steel**		
	Material	Labor	Equipment
Slab on Deck	$41,814	$10,153	$1,881
Metal Deck	$41,468	$10,428	$728
Beams	$160,851	$9,998	$4,937
Columns	$74,396	$964	$631
Braced Members	$22,447	$1,149	$659
	$340,976	$32,692	$8,836
	$382,504		

Steel System Cost Analysis – Spread Footings

Cost: Square Footing E-7 Steel System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	12.5	CY	$87.00	$0.00	$0.00	$1,088	$0	$0
03210-600-0550	Reinforcement	0.54	ton	$770.00	$350.00	$0.00	$416	$189	$0
03110-430-5150	Formwork	150	SFCA	$0.59	$2.59	$0.00	$89	$389	$0
03310-700-2600	Placement (direct chute)	12.5	CY	$0.00	$11.55	$0.36	$0	$144	$5
							$1,592	$722	$5
							$2,318		

Cost: Square Footing E-9 Steel System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	7.5	CY	$87.00	$0.00	$0.00	$653	$0	$0
03210-600-0550	Reinforcement	0.33	ton	$770.00	$350.00	$0.00	$254	$116	$0
03110-430-5150	Formwork	101.3	SFCA	$0.59	$2.59	$0.00	$60	$262	$0
03310-700-2600	Placement (direct chute)	7.5	CY	$0.00	$11.55	$0.36	$0	$87	$3
							$966	$464	$3
							$1,434		

Appendix H

Schedule Analysis Calculations

Concrete System Schedule Analysis – Typical Floor

	Schedule	Joists/Slab				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-0300	Concrete	1100	CY			
03210-600-0400	Reinforcement	64	Ton	12 Rodm	8.7	7.356
03110-420-3760	Formwork	24250	SF	(5) C-2	2400	10.104
03310-700-1600	Placement	1100	CY	C-20	180	6.111
03310-300-0010	Finishing	24250	SF	4 Cefi	3600	6.736
						30.308
						31 days

	Schedule	Girders				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-0300	Concrete	117	CY			
	Reinforcement					
03210-600-0100	#3-#7	11.33	Ton	12 Rodm	4.8	2.360
03210-600-0150	#8-#18	24.53	Ton	12 Rodm	8.1	3.028
	Formwork					
03110-405-2650	Interior	3464	SFCA	(5) C-2	1977	1.752
03110-405-1650	Exterior	4208	SFCA	(5) C-2	1625	2.590
03110-700-0200	Placement	117	CY	C-20	90	1.300
						11.030
						12 days

	Schedule	Columns				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-0400	Concrete					
03210-600-0250	Reinforcement	14.1	Ton	12 Rodm	6.9	2.043
	Formwork					
03110-410-6650	24"x24"	728	SFCA	(5) C-1	1190	0.612
03110-410	30"x30"	1820	SFCA	(5) C-1	1120	1.625
03110-410-7150	36"x36"	1560	SFCA	(5) C-1	1250	1.248
	Placement					
03310-700-0800	24"x24"	13.5	CY	C-20	92	0.147
03310-700	30"x30"	44	CY	C-20	116	0.379
03310-700-1000	36"x36"	42.2	CY	C-20	140	0.301
						6.356
						7 days

	Schedule	Shearwalls				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-0300	Concrete	49	CY			
03210-600-0700	Reinforcement	0.89	Ton	12 Rodm	9	0.099
03110-455-8060	Formwork	3900	SFCA	(5) C-2	3950	0.987
03310-700-4950	Placement	49	CY	C-20	100	0.490
03350-350-0020	Finishing	3900	SF	4 Cefi	2160	1.806
						3.382
						4 days

	Schedule	Shoring				
		Quantity	Unit	Crew	Daily Output	# of Days
03150-600-1500	Reshoring	24,250	SF	5 carp.	7000	3.464
03150-600-3050	Shores (12 mon.)	24,250	SF	5 carp.	7000	3.464
						6.93
						7 days

Final Schedule	Concrete
	# of Days
Joists/Slab	30.31
Girders	11.03
Columns	6.36
Shearwalls	3.38
Shoring/Reshoring	6.93
	58.01
	58 days

Concrete System Schedule Analysis – Spread Footings

Schedule: Square Footing E-7 Concrete System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	17	CY				
03210-600-0550	Reinforcement	0.61	ton	4 Rodm	3.60	0.17	
03110-430-5150	Formwork	179	SFCA	C-1	414.00	0.43	
03310-700-2600	Placement (direct chute)	17	CY	C-6	120.00	0.14	
						0.74	**day(s)**

Schedule: Square Footing E-9 Concrete System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	12.53	CY				
03210-600-0550	Reinforcement	0.53	ton	4 Rodm	3.60	0.15	
03110-430-5150	Formwork	167	SFCA	C-1	414.00	0.40	
03310-700-2600	Placement (direct chute)	12.53	CY	C-6	120.00	0.10	
						0.66	**day(s)**

Steel System Schedule Analysis – Typical Floor

	Schedule	Slab				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-1010	Concrete	356	CY			
03210-200-0100	WWF	242.5	CSF	2 Rodm	35	6.929
03110-420-6650	Edge Form	199.5	SFCA	C-1	225	0.887
03310-700-1400	Placement	356	CY	C-20	140	2.543
						10.358
						11 days

Schedule	Metal Deck				
		Area (SF)	Crew	Daily Output	# of Days
05310-300-5800	3"-20 gauge	24,250	E-4	3000	8.083
					8.083
					9 days

Schedule	Beams				
		Length (ft)	Crew	Daily Output	# of Days
05120-640-0300	W8x10	6.5	E-2	600	0.011
05120-640-1100	W12x14	40	E-2	880	0.045
05120-640-2700	W16x26	447	E-2	1000	0.447
05120-640-2900	W16x31	157	E-2	900	0.174
05120-640-3300	W18x35	148	E-5	960	0.154
05120-640-3500	W18x40	122	E-5	960	0.127
05120-640-3520	W18x46	823	E-5	960	0.857
05120-640-3700	W18x50	322	E-5	912	0.353
05120-640-3900	W18x55	92	E-5	912	0.101
05120-640	W18x60	76	E-5	906	0.084
05120-640-3920	W18x65	136	E-5	900	0.151
05120-640-4100	W21x44	330	E-5	1064	0.310
05120-640-4900	W24x55	189	E-5	1110	0.170
05120-640-5100	W24x62	103	E-5	1110	0.093
05120-640-5300	W24x68	37	E-5	1110	0.033
05120-640-5500	W24x76	43	E-5	1110	0.039
05120-640-5800	W27x84	86	E-5	1190	0.072
05120-640-6300	W30x108	43	E-5	1200	0.036
05120-640	W30x124	43	E-5	1160	0.037
		3244			3.296
					4 days

Schedule	**Columns**	Length (ft)	Crew	Daily Output	# of Days
05120-260-7200	W12x87	78	E-2	984	0.079
05120-260	W14x90	39	E-2	972	0.040
05120-260	W14x99	39	E-2	972	0.040
05120-260	W14x132	117	E-2	948	0.123
05120-260	W14x145	117	E-2	936	0.125
05120-260	W14x159	39	E-2	924	0.042
05120-260-7450	W14x176	234	E-2	912	0.257
05120-260	W14x193	117	E-2	900	0.130
05120-260	W14x211	78	E-2	888	0.088
05120-260	W14x233	78	E-2	876	0.089
05120-260	W14x257	273	E-2	864	0.316
		1209			1.330
					2 days

Schedule	**Braced Frame Members**	Length (ft)	Crew	Daily Output	# of Days
05120-640-3700	W18x50	60	E-5	912	0.066
05120-640-3920	W18x65	30	E-5	900	0.033
05120-640	W18x71	30	E-5	900	0.033
05120-640	W18x97	30	E-5	900	0.033
05120-260	HSS 8x8x3/8	48	E-2	700	0.069
05120-260	HSS 8x8x5/8	48	E-2	700	0.069
05120-260	HSS 10x10x1/2	48	E-2	768	0.063
05120-260	HSS 10x10x5/8	35	E-2	768	0.046
05120-260	HSS 12x12x1/2	35	E-2	828	0.042
		364			0.453
					1 day

Final Schedule	Steel
	# of Days
Slab on Deck	10.36
Metal Deck	8.08
Beams	3.30
Columns	1.33
Braced Members	0.45
	23.52

24 days

Steel System Schedule Analysis – Spread Footings

Schedule: Square Footing E-7 Steel System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	12.5	CY				
03210-600-0550	Reinforcement	0.54	ton	4 Rodm	3.60	0.15	
03110-430-5150	Formwork	150	SFCA	C-1	414.00	0.36	
03310-700-2600	Placement (direct chute)	12.5	CY	C-6	120.00	0.10	
						0.62	**day(s)**

Schedule: Square Footing E-9 Steel System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	7.5	CY				
03210-600-0550	Reinforcement	0.33	ton	4 Rodm	3.60	0.09	
03110-430-5150	Formwork	101.3	SFCA	C-1	414.00	0.24	
03310-700-2600	Placement (direct chute)	7.5	CY	C-6	120.00	0.06	
						0.40	**day(s)**

Appendix I

Design Load Calculations

Wind Loads (Steel and Concrete Systems)

Assumptions

- Assumed fixed at ground level even though there is a 3-level parking garage below grade
- Building shape, in plan and elevation, was assumed rectangular with the dimensions being 222.5’ in the North / South direction and 119’ in the East / West direction and a height of 180.75’, which is the tallest height measurement for the building. See framing plans and elevations for actual building shape and dimensions.

  NOTE: These assumed building shapes and dimensions were used to calculate the pressure profiles along the height of the building for a conservative approach. When the actual forces to each floor were calculated, actual building dimensions and shapes were used.

- The wind load calculation procedures were taken from ASCE 7-02, Chapter 6. Method 2: Analytical Procedure (Sec. 6.5) was used for this building.

Building Information

- N-S direction – Shearwalls/Braced Frames
- E-W direction – Shearwalls/Braced Frames
- Location: Arlington, VA
- Exposure B
- Building Use: Office (Primary), Retail (1st Level), Parking (Below Grade)

Velocity Pressure

- $K_{zt} = 1.0$ (Fig. 6-4) area is flat
- $K_d = 0.85$ (Table 6-4) Building MWFRS
- V = 90 mph (Fig. 6-1)
- Use Group II (Table 1-1)
- I = 1.0 (Table 6-1)

From Table 6-3 (Exposure B, Case 2)

z (ft)	Kz
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
180	1.17
200	1.20

$$q_z = 0.00256 K_{zt} K_d V^2 I K_z$$

$$q_z = 0.00256(1.0)(0.85)(90)^2(1.0)K_z$$

$q_z = 17.63 K_z$ PSF

$q_h = 17.63(1.17^*)$ *linear interpolation

$q_h = 20.65$ PSF

External Pressure Coefficients (Fig. 6-6)

Windward Wall: Cp = 0.8
Leeward Wall:
N-S: L/B = 222.5'/119' = 1.87 Cp = -0.326* *linear interpolation
E-W: L/B = 119'/222.5' = 0.53 Cp = -0.5

Internal Pressure Coefficients (6.5.11.1)

GC_{pi} = +0.18
= -0.18
$q_i = q_h$ = 20.65 PSF ($q_i = q_h$ for windward and leeward walls of enclosed buildings)

Internal Pressure = $qiGC_{pi} = \pm 20.65 PSF(0.18) = \pm 3.72 PSF$

Gust Factor (N-S Direction)

N-S Direction: B = 119', L = 222.5'

Estimate Frequency (C_t = 0.02, x = 0.75 – Table 9.5.5.3.2)

$$f = \frac{1}{C_t h_n^x} = \frac{1}{0.02(180.75)^{0.75}} = 1.01Hz > 1.0 \therefore Rigid$$ (Inverse of Eq. 9.5.5.3.2-1)

G = 0.85 or

Calculate G

From Table 6-2 (Exposure B)

$$\bar{z}_{\min} = 30\,ft$$
$$c = 0.3$$
$$l = 320\,ft$$
$$\bar{\varepsilon} = 1/3$$

$$g_Q = 3.4$$
$$g_V = 3.4$$ (6.5.8.1)

$$\bar{z} = 0.6h = 0.6(180.75) = 108.45' > 30' \therefore \bar{z} = 108.45'$$ (6.5.8.1)

$$L_z = l(\bar{z}/33)^{\bar{\varepsilon}} = 320(108.45/33)^{1/3} = 475.76$$ (Eq. 6-7)

$$I_z = c(33/\bar{z})^{1/6} = 0.3(33/108.45)^{1/6} = 0.246$$ (Eq. 6-5)

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{119+180.75}{475.76}\right)^{0.63}}} = 0.82$$ (Eq. 6-6)

$$G = 0.925\left(\frac{1+1.7g_Q I_z Q}{1+1.7g_V I_z}\right) = 0.925\left(\frac{1+1.7(3.4)(0.246)(0.82)}{1+1.7(3.4)(0.246)}\right) = 0.83$$ (Eq. 6-4)

Since 0.83 < 0.85, use G=0.83

Gust Factor (E-W Direction)

E-W Direction: B = 222.5’, L = 119’

Estimate Frequency (C_t = 0.02, x = 0.75 – Table 9.5.5.3.2)

$$f = \frac{1}{C_t h_n^x} = \frac{1}{0.02(180.75)^{0.75}} = 1.01Hz > 1.0 \therefore Rigid$$ (Inverse of Eq. 9.5.5.3.2-1)

G = 0.85 or

Calculate G

From Table 6-2 (Exposure B)

$$\bar{z}_{\min} = 30\,ft$$
$$c = 0.3$$
$$l = 320\,ft$$
$$\bar{\varepsilon} = 1/3$$

$$g_Q = 3.4$$
$$g_V = 3.4$$
(6.5.8.1)

$$\bar{z} = 0.6h = 0.6(180.75) = 108.45' > 30' \therefore \bar{z} = 108.45'$$ (6.5.8.1)

$$L_z = l(\bar{z}/33)^{\bar{\varepsilon}} = 320(108.45/33)^{1/3} = 475.76$$ (Eq. 6-7)

$$I_z = c(33/\bar{z})^{1/6} = 0.3(33/108.45)^{1/6} = 0.246$$ (Eq. 6-5)

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{222.5+180.75}{475.76}\right)^{0.63}}} = 0.799$$ (Eq. 6-6)

$$G = 0.925\left(\frac{1+1.7g_Q I_z Q}{1+1.7g_V I_z}\right) = 0.925\left(\frac{1+1.7(3.4)(0.246)(0.799)}{1+1.7(3.4)(0.246)}\right) = 0.82$$ (Eq. 6-4)

Since 0.82 < 0.85, use G=0.82

N-S Windward Pressure

$$P_{wz} = q_z C_p G = q_z 0.8(0.83) = 0.664 q_z \text{ PSF}$$

N-S Leeward Pressure

$$P_{lh} = q_h C_p G = 20.65(-0.326)(0.83) = -5.59 \text{ PSF}$$

E-W Windward Pressure

$$P_{wz} = q_z C_p G = q_z 0.8(0.82) = 0.656 q_z \text{ PSF}$$

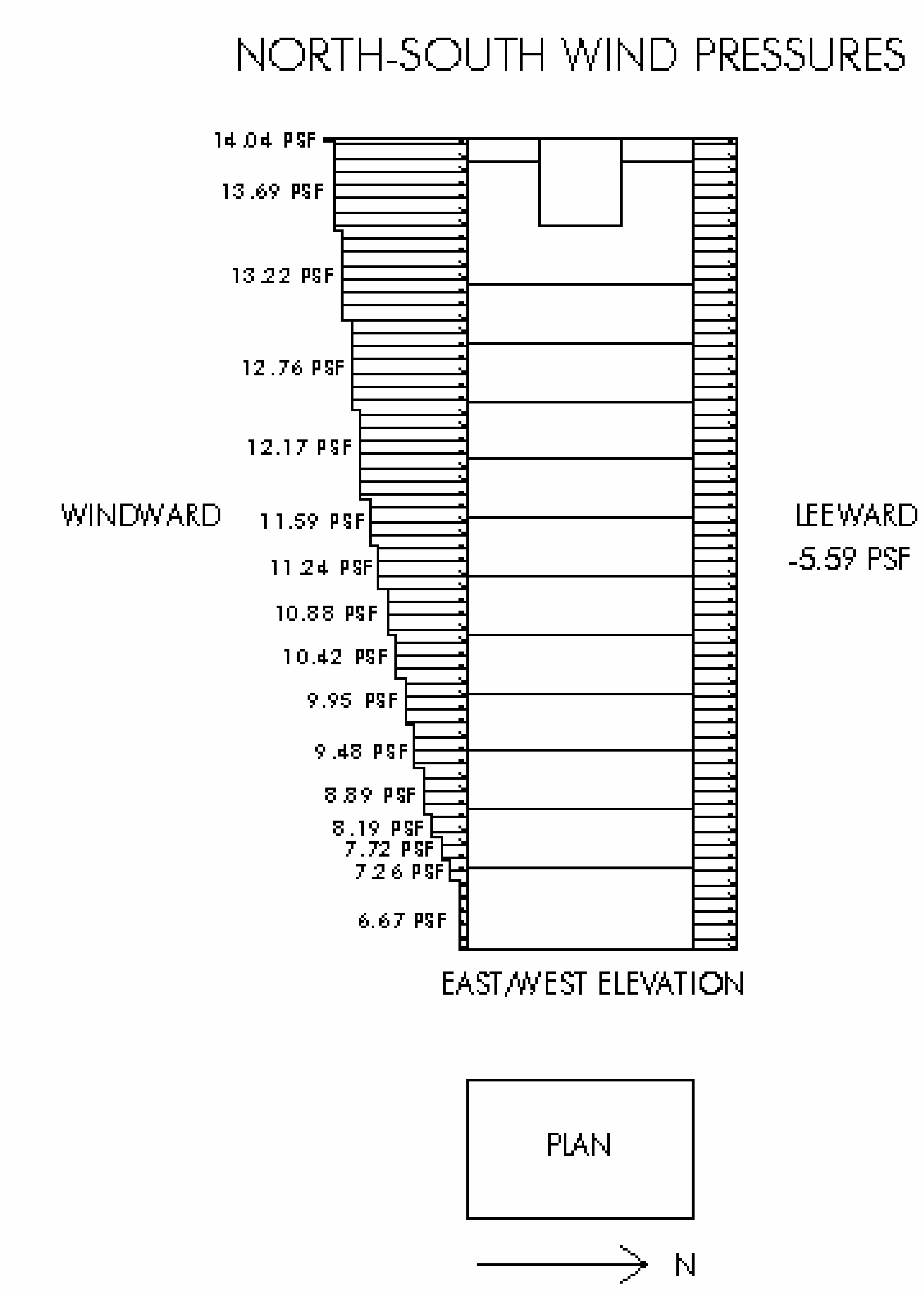
E-W Leeward Pressure

$$P_{lh} = q_h C_p G = 20.65(-0.5)(0.82) = -8.47 \text{ PSF}$$

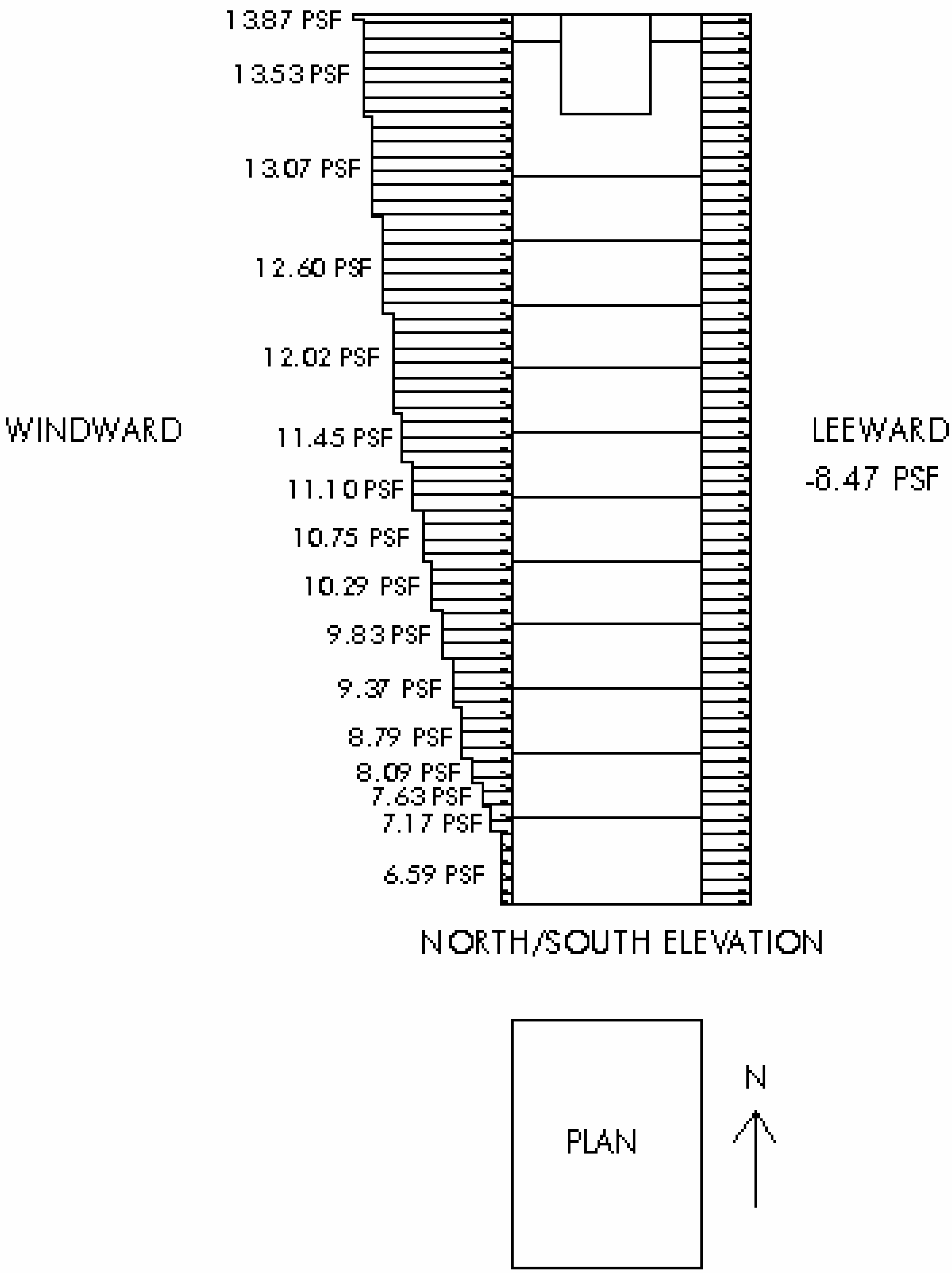
Total Pressures

z	Kz	qz	N-S Windward Pressure (PSF)	E-W Windward Pressure (PSF)	N-S Leeward Pressure (PSF)	E-W Leeward Pressure (PSF)	P_{total} (N-S) (PSF)	P_{total} (E-W) (PSF)
0-15	0.57	10.05	6.67	6.59	-5.59	-8.47	12.26	15.06
20	0.62	10.93	7.26	7.17	-5.59	-8.47	12.85	15.64
25	0.66	11.63	7.72	7.63	-5.59	-8.47	13.31	16.10
30	0.70	12.34	8.19	8.09	-5.59	-8.47	13.78	16.56
40	0.76	13.40	8.89	8.79	-5.59	-8.47	14.48	17.26
50	0.81	14.28	9.48	9.37	-5.59	-8.47	15.07	17.84
60	0.85	14.98	9.95	9.83	-5.59	-8.47	15.54	18.30
70	0.89	15.69	10.42	10.29	-5.59	-8.47	16.01	18.76
80	0.93	16.39	10.88	10.75	-5.59	-8.47	16.47	19.22
90	0.96	16.92	11.24	11.10	-5.59	-8.47	16.83	19.57
100	0.99	17.45	11.59	11.45	-5.59	-8.47	17.18	19.92
120	1.04	18.33	12.17	12.02	-5.59	-8.47	17.76	20.49
140	1.09	19.21	12.76	12.60	-5.59	-8.47	18.35	21.07
160	1.13	19.92	13.22	13.07	-5.59	-8.47	18.81	21.54
180	1.17	20.62	13.69	13.53	-5.59	-8.47	19.28	22.00
200	1.20	21.15	14.04	13.87	-5.59	-8.47	19.63	22.34

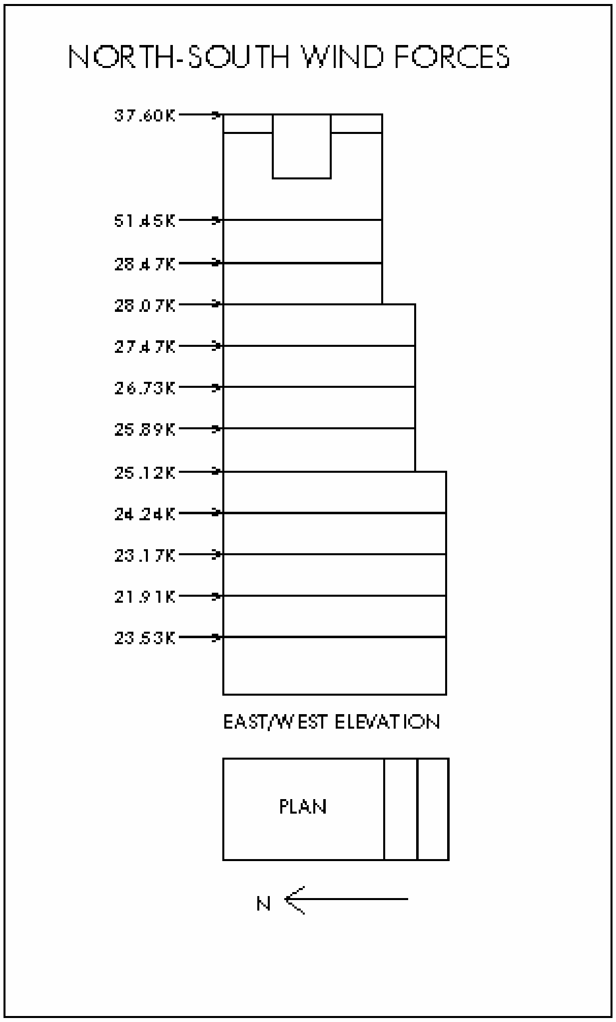
Wind Pressure Diagrams

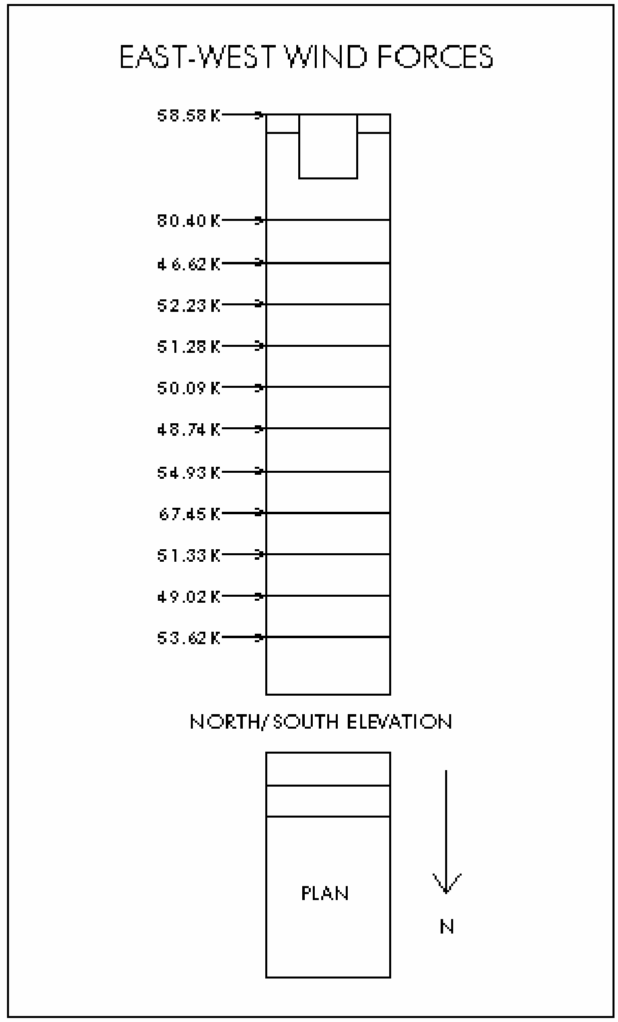


EAST-WEST WIND PRESSURES



Wind Force Diagrams





Seismic Loads (Concrete System)

Assumptions

- ASCE 7-02, Chapter 9 was used to calculate the seismic loads for this building.

Building Information

- N-S Direction: Reinforced Concrete Shearwalls
- E-W Direction: Reinforced Concrete Shearwalls
- Location: Arlington, VA
- Building Use: Office (Primary), Retail (1st Level), Parking (Below Grade)

Seismic Design Category

Occupancy Category - II	(Table 1-1)
Seismic Use Group: 1	(Table 9.1.3)
Site Class C:	(Structural Notes)
Acceleration from Maps:	
$S_s = 0.190$	(Fig. 9.4.1.1a)
$S_1 = 0.070$	(Fig. 9.4.1.1b)
Adjust for Site Class:	
$F_a = 1.2$	(Table 9.4.1.2.4a)
$F_V = 1.7$	(Table 9.4.1.2.4b)
$S_{ms} = F_aS_s = 1.2(0.19) = 0.228$	(Eq. 9.4.1.2.4-1)
$S_{m1} = F_vS_1 = 1.7(0.07) = 0.119$	(Eq. 9.4.1.2.4-2)

Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3\ S_{ms} = 2/3(0.228) = 0.152$	(Eq. 9.4.1.2.5-1)
$S_{D1} = 2/3\ S_{m1} = 2/3(0.119) = 0.0793$	(Eq. 9.4.1.2.5-2)

Seismic Design Category

(Table 9.4.2.1a)
S.D.C. based on short period response acceleration = S.D.C.-A

(Table 9.4.2.1b)
S.D.C. based on 1-sec. period response acceleration = **S.D.C.-B* worst case**

NOTE: Building does not meet any plan or vertical irregularities as specified in Tables 1616.5.1.1 or 1616.5.1.2 of the IBC 2000, therefore it is still S.D.C.-B.
Equivalent Lateral Force Procedure can be used.

Seismic Base Shear ($V=C_sW$)

R = 4 (Table 9.5.2.2)   Reinforced Concrete Shearwalls
I = 1.0 (Table 9.1.4)
$T = C_t h_n^x$ (Eq. 9.5.5.3.2-1)
N-S: $T = C_t h_n^x = 0.016(180.75)^{0.9} = 1.72\sec$ (Table 9.5.5.3.2)
E-W: $T = C_t h_n^x = 0.016(180.75)^{0.9} = 1.72\sec$ (Table 9.5.5.3.2)

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.152}{4/1} = 0.038$$

$$C_{S,\max}(N-S) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{1.72\left(\frac{4}{1}\right)} = 0.011526$$ *Controls

$$C_{S,\max}(E-W) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{1.72\left(\frac{4}{1}\right)} = 0.011526$$ *Controls

$$C_{S,\min} = 0.044IS_{DS} = 0.044(1.0)(0.152) = 0.006688 < 0.011526 \therefore OK$$

Dead Loads

Roof Dead Load

Joists	9 PSF
Metal Deck	3 PSF
Insulation	1.5 PSF
SDL	15 PSF
Built-up Roof (5-ply felt and gravel)	6.5 PSF
	35 PSF

Snow Load 30 PSF (See Snow Load Calculations)

Typical Floor Load

Joists	119 PSF
Misc. DL	15 PSF
mech. ducts, plumbing, sprinklers, ceiling, etc.	134 PSF

Exterior Wall Loads

Glass Curtain Wall (N façade)	15 PSF
Precast/Windows (S,E,W facades)	20 PSF

$$w_{roof} = 834.2k$$
$$w_{11} = 2{,}500.5k$$
$$w_{10} = 2{,}375.7k$$
$$w_{9-6} = 2{,}854k$$
$$w_{5-2} = 3{,}332.4k$$
$$w_1 = 3364k$$

$$W = w_{roof} + w_{11} + w_{10} + 4w_{9-6} + 4w_{5-2} + w_1$$
$$W = 834.2k + 2{,}500.5k + 2{,}375.7k + 4(2{,}854k) + 4(3332.4k) + 3{,}364k$$
$$W = 33{,}820k$$

$V_{N\text{-}S}$ = 0.011526(33,820 k) = 390 k
$V_{E\text{-}W}$ = 0.011526(33,820 k) = 390 k

$$F_x = C_{vx}V$$
$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$k(N-S)=1+\frac{1.72-0.5}{2}=1.61^{*}$$ *linear interpolation

$$k(E-W)=1+\frac{1.72-0.5}{2}=1.61^{*}$$ *linear interpolation

Seismic Base Shear and Overturning Moment

Level	w_x	h_x	$w_x h_x^{1.61}$	$w_x h_x^{1.61}$	C_{vx} (N-S)	C_{vx} (E-W)	F_x (N-S)	F_x (E-W)
Roof	834.1	180.75	3590163	3590163	0.079	0.079	30.61	30.61
12	2501	148	7802523	7802523	0.171	0.171	66.53	66.53
11	2376	135	6392692	6392692	0.140	0.140	54.51	54.51
10	2854	122	6523690	6523690	0.143	0.143	55.63	55.63
9	2854	109	5441400	5441400	0.119	0.119	46.40	46.40
8	2854	96	4435174	4435174	0.097	0.097	37.82	37.82
7	2854	83	3508890	3508890	0.077	0.077	29.92	29.92
6	3332	70	3113959	3113959	0.068	0.068	26.55	26.55
5	3332	57	2236987	2236987	0.049	0.049	19.08	19.08
4	3332	44	1474565	1474565	0.032	0.032	12.57	12.57
3	3332	31	839068	839068	0.018	0.018	7.15	7.15
2	3364	18	353057	353057	0.008	0.008	3.01	3.01
	33819.1		45712167	45712167	1.000	1.000	390	390

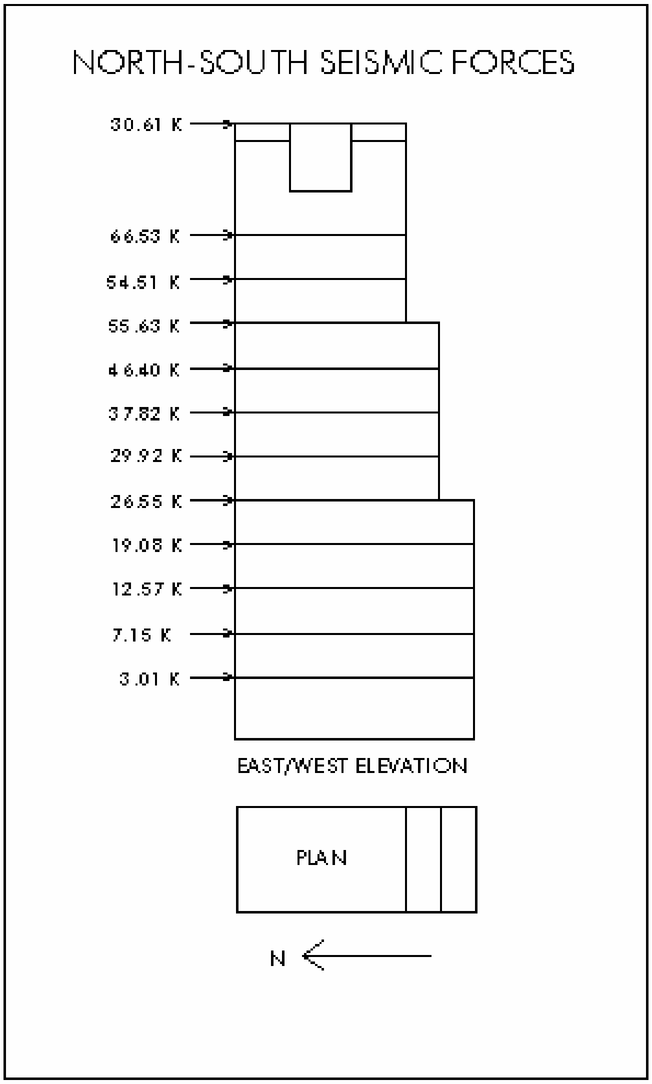
k (N-S)	1.61	
k (E-W)	1.61	
V (N-S)	390	k
V (E-W)	390	k

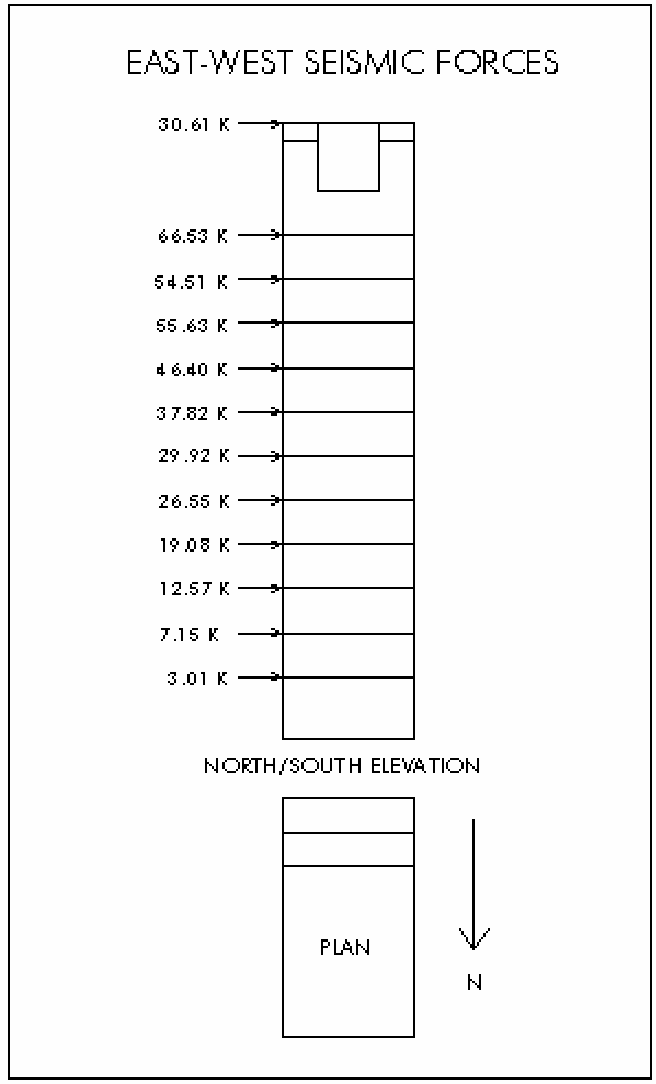
C_s (N-S)	0.011526
C_s (E-W)	0.011526

Base Shear		
N-S	390	k
E-W	390	k

Overturning Moment		
Overturning Moment (N-S)	44473.5034	ft-k
Overturning Moment (E-W)	44473.5034	ft-k

Seismic Force Diagrams





Load Cases and Controlling Lateral Forces (Concrete System)

Load Combinations Involving Wind Loads (W) and Seismic Loads (E)

ASCE 7-02 (Sec. 2.3.2)

1.2D + 1.6(Lr or S or R) + (L or **0.8W**)
1.2D + **1.6W** + L + 0.5(Lr or S or R)
1.2D + **1.0E** + L + 0.2S
0.9D + **1.6W** + 1.6H
0.9D + **1.0E** + 1.6H

Check 1.6W vs. 1.0E

Red = Controlling E-W Lateral Force, Blue = Controlling N-S Lateral Force

	1.6W (N-S)	1.6W (E-W)	1.0E (N-S/E-W)
Roof	**60.16**	**93.72**	30.61
12	**82.32**	**128.64**	66.53
11	45.55	**74.59**	**54.51**
10	44.91	**83.57**	**55.63**
9	43.95	**82.05**	**46.40**
8	**42.77**	**80.14**	37.82
7	**41.42**	**77.98**	29.92
6	**40.19**	**87.89**	26.55
5	**38.78**	**107.92**	19.08
4	**37.07**	**82.13**	12.57
3	**35.06**	**78.43**	7.15
2	**37.64**	**85.79**	3.01

Seismic Loads (Steel System)

Assumptions

- ASCE 7-02, Chapter 9 was used to calculate the seismic loads for this building.

Building Information

- N-S Direction: Steel Braced Frames
- E-W Direction: Steel Braced Frames
- Location: Arlington, VA
- Building Use: Office (Primary), Retail (1st Level), Parking (Below Grade)

Seismic Design Category

Occupancy Category - II	(Table 1-1)
Seismic Use Group: 1	(Table 9.1.3)
Site Class C:	(Structural Notes)
Acceleration from Maps:	
$S_s = 0.190$	(Fig. 9.4.1.1a)
$S_1 = 0.070$	(Fig. 9.4.1.1b)
Adjust for Site Class:	
$F_a = 1.2$	(Table 9.4.1.2.4a)
$F_V = 1.7$	(Table 9.4.1.2.4b)
$S_{ms} = F_aS_s = 1.2(0.19) = 0.228$	(Eq. 9.4.1.2.4-1)
$S_{m1} = F_vS_1 = 1.7(0.07) = 0.119$	(Eq. 9.4.1.2.4-2)

Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3\ S_{ms} = 2/3(0.228) = 0.152$	(Eq. 9.4.1.2.5-1)
$S_{D1} = 2/3\ S_{m1} = 2/3(0.119) = 0.0793$	(Eq. 9.4.1.2.5-2)

Seismic Design Category

(Table 9.4.2.1a)
S.D.C. based on short period response acceleration = S.D.C.-A

(Table 9.4.2.1b)
S.D.C. based on 1-sec. period response acceleration = **S.D.C.-B* worst case**

NOTE: Building does not meet any plan or vertical irregularities as specified in Tables 1616.5.1.1 or 1616.5.1.2 of the IBC 2000, therefore it is still S.D.C.-B.
Equivalent Lateral Force Procedure can be used.

Seismic Base Shear ($V=C_S W$)

R = 3 (Table 9.5.2.2) Structural steel systems not specifically detailed for seismic resistance.

I = 1.0 (Table 9.1.4)

$T = C_t h_n^x$ (Eq. 9.5.5.3.2-1)

N-S: $T = C_t h_n^x = 0.02(180.75)^{0.75} = 0.986$ (Table 9.5.5.3.2)

E-W: $T = C_t h_n^x = 0.02(180.75)^{0.75} = 0.986$ (Table 9.5.5.3.2)

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.152}{3/1} = 0.050667$$

$$C_{S,\max}(N-S) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{0.986\left(\frac{3}{1}\right)} = 0.02681$$ *Controls

$$C_{S,\max}(E-W) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{0.986\left(\frac{3}{1}\right)} = 0.02681$$ *Controls

$$C_{S,\min} = 0.044 I S_{DS} = 0.044(1.0)(0.152) = 0.006688 < 0.02681 \therefore OK$$

Dead Loads

Roof Dead Load

Metal Deck	5 PSF
Insulation	3 PSF
Misc. DL	10 PSF
Roofing	20 PSF
	38 PSF

Snow Load 30 PSF (See Snow Load Calculations)

NOTE: Since Snow Load is not greater than 30 PSF, 20% of the Snow Load does not need to be considered in the weight calculations.

Typical Floor Load

3 ¼" lt. wt. slab on 3" metal deck	46 PSF
Ponding of Concrete	10 PSF
Misc. DL	15 PSF
mech. ducts, plumbing, sprinklers, ceiling, etc.	71 PSF

Exterior Wall Loads

Glass Curtain Wall (N façade)	15 PSF
Precast/Windows (S,E,W facades)	20 PSF

$$w_{roof} = 909k$$
$$w_{11} = 1617k$$
$$w_{10} = 1512k$$
$$w_{9-6} = 1781k$$
$$w_{5-2} = 2050k$$
$$w_1 = 2083k$$

$$W = w_{roof} + w_{11} + w_{10} + 4w_{9-6} + 4w_{5-2} + w_1$$
$$W = 909k + 1617k = 1512k + 4(1781k) + 4(2050k) + 2083k$$
$$W = 21{,}445k$$

$V_{N\text{-}S}$ = 0.02681(21,445k) = 574.94k
$V_{E\text{-}W}$ = 0.02681(21,445k) = 574.94k

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$k(N-S) = 1 + \frac{0.986 - 0.5}{2} = 1.243*$$ *linear interpolation

$$k(E-W) = 1 + \frac{0.986 - 0.5}{2} = 1.243*$$ *linear interpolation

Seismic Base Shear and Overturning Moment

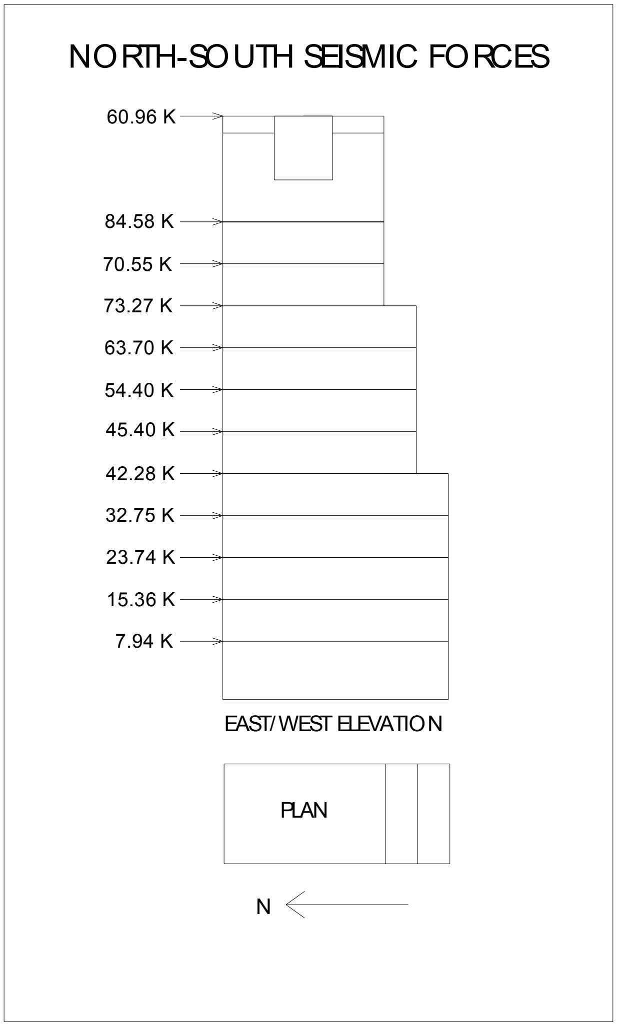
Level	w_x	h_x	$w_x h_x^{1.243}$	$w_x h_x^{1.243}$	C_{vx} (N-S)	C_{vx} (E-W)	F_x (N-S)	F_x (E-W)
12 (roof)	909	180.75	580915	580915	0.106	0.106	60.96	60.96
11	1617	148	806019	806019	0.147	0.147	84.58	84.58
10	1512	135	672290	672290	0.123	0.123	70.55	70.55
9	1781	122	698247	698247	0.127	0.127	73.27	73.27
8	1781	109	606995	606995	0.111	0.111	63.70	63.70
7	1781	96	518355	518355	0.095	0.095	54.40	54.40
6	1781	83	432592	432592	0.079	0.079	45.40	45.40
5	2050	70	402912	402912	0.074	0.074	42.28	42.28
4	2050	57	312109	312109	0.057	0.057	32.75	32.75
3	2050	44	226238	226238	0.041	0.041	23.74	23.74
2	2050	31	146392	146392	0.027	0.027	15.36	15.36
1	2083	18	75682	75682	0.014	0.014	7.94	7.94
			5478745	5478745	1.000	1.000	574.94	574.94

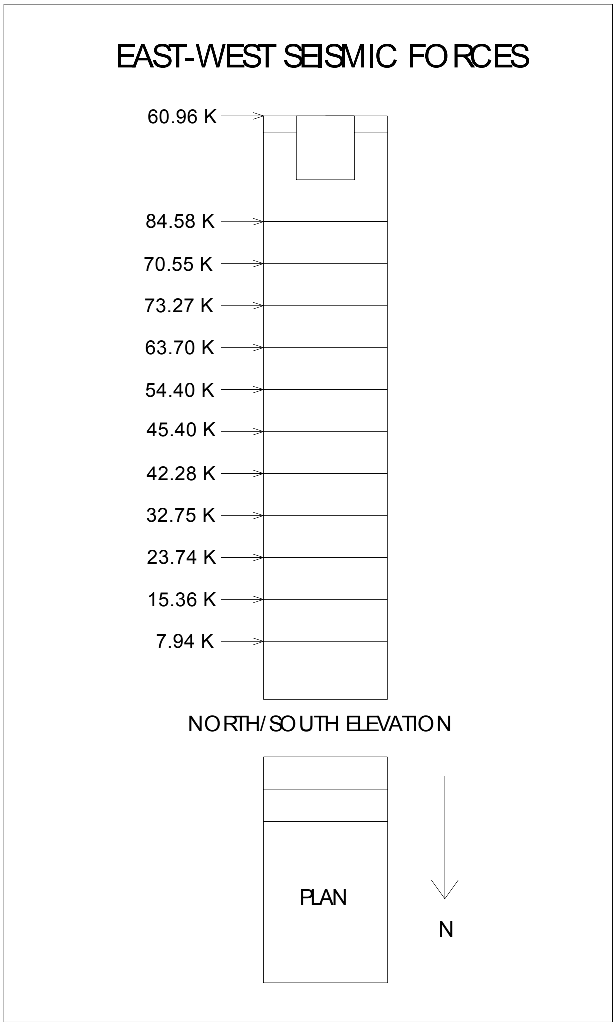
k (N-S)	1.243	
k (E-W)	1.243	
V (N-S)	574.94	k
V (E-W)	574.94	k

Base Shear		
N-S	574.94	k
E-W	574.94	k

Overturning Moment		
Overturning Moment (N-S)	64424.2942	ft-k
Overturning Moment (E-W)	64424.2942	ft-k

Seismic Force Diagrams





Load Cases and Controlling Lateral Forces (Steel System)

Load Combinations Involving Wind Loads (W) and Seismic Loads (E)

ASCE 7-02 (Sec. 2.3.2)

1.2D + 1.6(Lr or S or R) + (L or **0.8W**)
1.2D + **1.6W** + L + 0.5(Lr or S or R)
1.2D + **1.0E** + L + 0.2S
0.9D + **1.6W** + 1.6H
0.9D + **1.0E** + 1.6H

Check 1.6W vs. 1.0E

Red = Controlling E-W Lateral Force, Blue = Controlling N-S Lateral Force

	1.6W (N-S)	1.6W (E-W)	1.0E (N-S/E-W)
Roof	60.16	**93.72**	**60.96**
12	82.32	**128.64**	**84.58**
11	45.55	**74.59**	**70.55**
10	44.91	**83.57**	**73.27**
9	43.95	**82.05**	**63.70**
8	42.77	**80.14**	**54.40**
7	41.42	**77.98**	**45.40**
6	40.19	**87.89**	**42.28**
5	**38.78**	**107.92**	32.75
4	**37.07**	**82.13**	23.74
3	**35.06**	**78.43**	15.36
2	**37.64**	**85.79**	7.94

Snow Load (Steel and Concrete Systems)

Assumptions

- ASCE 7-02, Chapter 7 was used to calculate the snow loads for this building.

Building Information

- Location: Arlington, VA
- Max. Roof Slope = 4.55% or 2.62º*

*Since the maximum roof slope is less than 5º, then ASCE 7-02, Chapter 7, Section 7-3 can be used.

$p_f = 0.7 C_e C_t I p_g$ (Eq. 7-1)

C_e = 0.9	Surface roughness B (6.5.6.2)
	Fully exposed (Table 7-2)
C_t = 1.0	(Table 7-3)
I = 1.0	Category II (Table 7-4)
p_g = 25 PSF	(Fig. 7-1)

$$p_f = 0.7(0.9)(1.0)(1.0)(25PSF) = 15.75PSF$$

$p_{f,\min} = 20PSF \cdot I$ p_g>20 PSF (Sec. 7-3)

$p_{f,\min} = 20PSF \cdot 1 = 20PSF$ 20 PSF > 15.75 PSF, therefore use 20 PSF

NOTE: Structural Notes specify a snow load value of 30 PSF.

Roof Live Load (Steel and Concrete Systems)

Assumptions

- ASCE 7-02, Chapter 4 was used to check the minimum roof live load.

$L_r = 20R_1R_2$ (Eq. 4-2)

R2 = 1 F<4 (4.9.1) Max. roof slope = 2.61º

R1= 0.6 A_t > 600 SF (4.9.1) At(roof col.) = 30'[(46'+30')/2]
At(roof col.) = 1140 SF > 600 SF

$L_r = 20(1)(0.6)$
$L_r = 12$ PSF < 30 PSF*

***Therefore snow load controls with a roof live load of 30 PSF.**