Appendix A: Gravity System Design

Snow Load Calculation

Snow Drift | tech 1 | page 1 of 2

step 1: ground snow load, p_g → from fig 7-1, $p_g = 25$ psf

step 2: Exposure factor, C_e, → from table 7-2, Terrain category B
roof fully exposed
⇒ $C_e = 0.90$

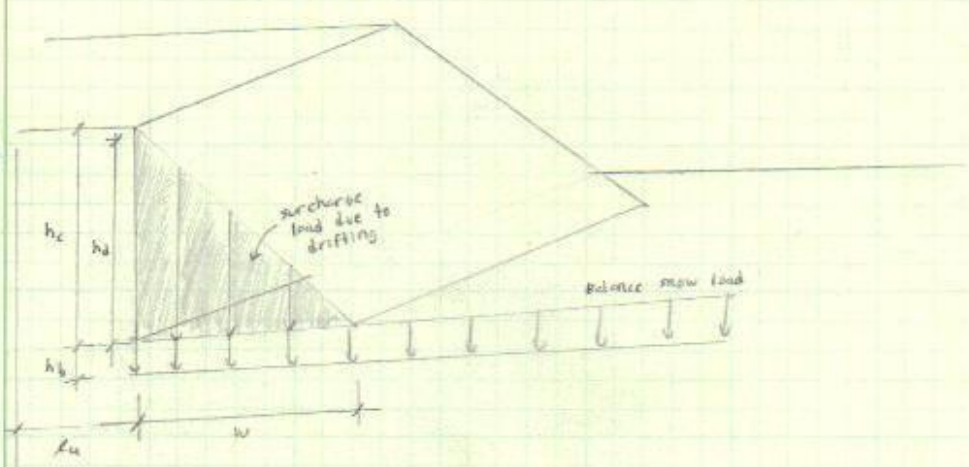
step 3: Thermal factor, C_t → from table 7-3, $C_t = 1.0$

step 4: importance factor, I → from table 1.5-1, $I = 1.0$ (occ II)

step 5: flat roof snow load, p_f → Sect 7.3,

$$p_f = 0.7\, C_e C_t I_s p_g$$
$$= 0.7\,(0.90)(1.0)(1.0)(25\text{ psf})$$
$$= 15.75 \text{ psf}$$

Snow drift: Penthouse level



step 6: maximum intensity of the drift surcharge load, p_0 → Sect 7.7.1

$p_0 = h_d \gamma$ ⇒ snow density $\gamma = 0.13 p_g + 14$
$= 0.13(25\text{ psf}) + 14$
$= 17.25 \text{ pcf} < \gamma_{max} = 30 \text{ pcf}$

leeward drift
E-W wind: $l_u = 135'$ ⇒ snow drift height, h_d → from fig 7-9,

$$h_d = 0.43\sqrt[3]{135}\sqrt[4]{25+10} - 1.5 = 3.7 \text{ ft}$$

leeward drift
N-S wind: $l_u = 55.9'$ ⇒ $h_d = 0.43\sqrt[3]{55.9}\sqrt[4]{25+10} - 1.5 = 2.5$ ft

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leeward drift

E-W wind: $P_d = 3.9\ ft(17.25\ pcf) = 67.3\ psf$; width of snow drift, $w = 4h_d = 4(3.9\ ft) = 15.6\ ft$

leeward drift

N-S wind: $P_d = 2.5\ ft(17.25\ pcf) = 42.1\ psf$; $w = 4(2.5) = 10\ ft$

step 7: balanced snow load height, h_b → from sect 2.1

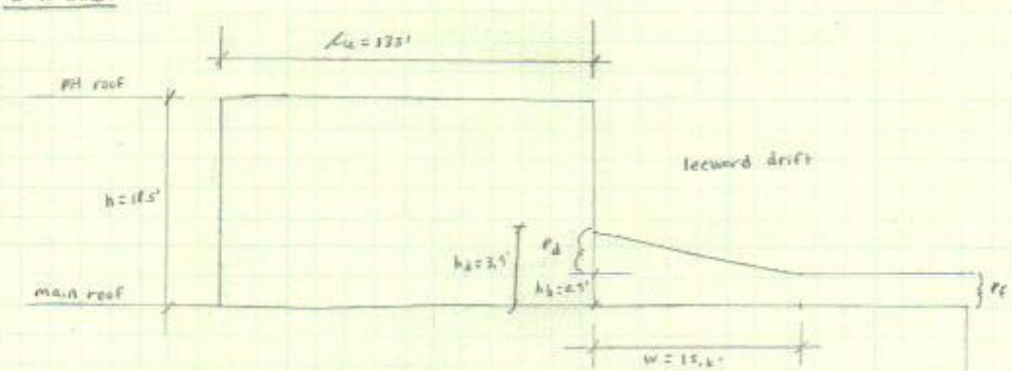
$$h_b = \frac{P_f}{\gamma} = \frac{15.75\ psf}{17.25\ pcf} = 0.9\ ft$$

step 8: $h = \underset{\text{PH roof}}{148.5'} - \underset{\text{main roof}}{130'} = 18.5\ ft$

step 9: $h_c = h - h_b = 18.5' - 0.9' = 17.6\ ft$

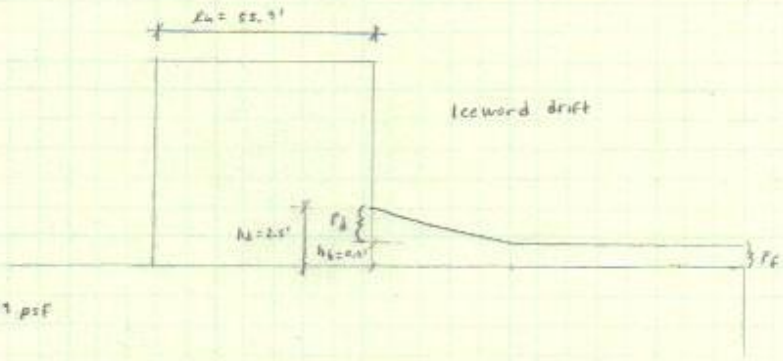
step 10: total snow load, $P = P_d + P_f$

E-W wind



$P_{total} = 67.3\ psf + 15.75\ psf = 83\ psf$

N-S wind

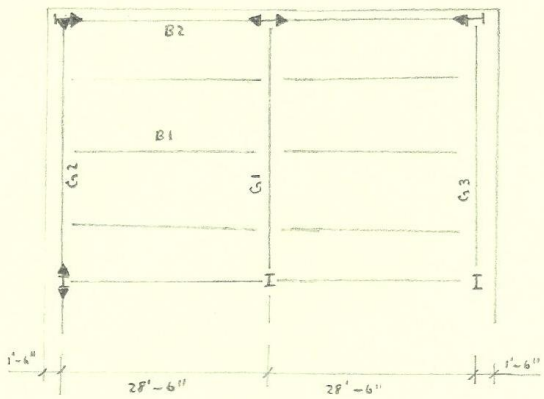


$P_{total} = 42.1 + 15.75 = 59\ psf$

since $P_{tot,E-W} > P_{tot,N-S}$, use $P_{tot} = 83\ psf$ for a conservative design

Gravity Floor System Design

composite Flr System Design | 28'-6" x 40'-0" Bay | page 1 of 9



interior bm B1

① total load

dead load:

SDL = 10 psf
slab = 75 psf

$w_D = (10\ psf + 75\ psf)(10ft) = 850\ plf$

live load:

$L_o = 100\ psf$

influence area $= k_{LL} A_T = 2(28.5\ ft)(10ft) = 570\ ft^2 > 400\ ft^2$ ∴ reduce LL

$L = 100 \times \begin{cases} 0.5 \\ 0.25 + \frac{15}{\sqrt{570}} = 0.878 \end{cases}$ max $= 87.8\ psf$

$w_L = 87.8\ psf\ (10ft) = 878\ plf$

② required moment, M_u → assumptions: - bm simply supported
- add 5 psf for bm self-wt

$w_u = 1.2(5\ psf)(10ft) + 1.2(850) + 1.6(878) = 2485\ plf$

$M_u = \frac{w_u L^2}{8} = \frac{(2485)(28.5)^2}{8} = 252,305\ lb\text{-}ft = 252\ kft$; $V_u = \frac{w_u L}{2} = \frac{2.485(28.5)}{2} = 35.4\ K$

③ starting moment arm for the concrete, y_2

assume $a = 1"$

$y_2 = t - \frac{a}{2} = 7.5 - \frac{1}{2} = 7"$

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④ determine I_{LB}

$\Delta_{LL} \le \Delta_{LL,max} = L/360$

$$\frac{5(0.878)(28.5)^4(1728)}{384EI_{LB}} \le \frac{28.5(12)}{360}$$

$$I_{LB} \ge \frac{5(0.878)(28.5)^4(1728)}{384(29000)(0.95)} = 473 \text{ in}^4$$

$\Delta_{TL} \le \Delta_{TL,max} = L/240$

$$I_{LB} \ge \frac{5(0.878+0.850)(28.5)^4(1728)}{384(29000)(1.425)} = 621 \text{ in}^4$$

⑤ select potential W-shapes from table 3-19 and 3-20 in manual
($\phi M_n \ge 252$ kft, $I_{LB} \ge 621$ in^4)

W10X26, $I_{LB} = 655$ in^4, $\phi M_n = 313$ kft, $\Sigma Q_n = 317$ K (y1=2)

W12X22, $I_{LB} = 631$ in^4, $\phi M_n = 274$ kft, $\Sigma Q_n = 238$ K (y1=3)

W14X22, $I_{LB} = 672$ in^4, $\phi M_n = 270$ kft, $\Sigma Q_n = 199$K (y1=4)

W14X26, $I_{LB} = 634$ in^4, $\phi M_n = 265$ kft, $\Sigma Q_n = 135$K (y1=6)

⑥ # of studs/bm

W10X26: $\frac{\Sigma Q_n}{Q_n} \times 2 = \frac{317}{17.2} \times 2 = 38 > 28.5'$ ∴ use 2 studs/rib
(↑ 1 stud/rib)

$\frac{317}{14.6} \times 2 = 44$ studs $< 28.5'(2) = 57'$ ∴ ok

W14X26: $\frac{135}{17.2} \times 2 = 16$ studs $< 28.5'$

W12X22: $\frac{238}{17.2} \times 2 = 28$ studs $< 28.5'$

W14X22: $\frac{199}{17.2} \times 2 = 24$ studs $< 28.5'$

⑦ check economy

W10X26 w/ 44 studs: 26# x 28.5' + 44 studs x 10#/stud = 1181 #

W12X22 w/ 28 studs: 22# x 28.5' + 28 x 10#/stud = 907 #

W14X22 w/ 24 studs: 22# x 28.5' + 24 x 10#/stud = 867 #

W14X26 w/ 16 studs: 26# x 28.5' + 16 x 10#/stud = 901 #

28'-6" x 40'-0" Bay page 3 of 9

⑧ check a

$a = \frac{\Sigma Q_n}{0.85 f'c\ b_{eff}}$

$b_{eff} = \frac{28.5(12)}{8} \times 2 = 85.5'' \leftarrow$

$\frac{1}{2}(10)(12) \times 2 = 120''$

min $\frac{28.5(12)}{8} + \frac{10(12)}{2} = 102.75''$

W14X22: $a = \frac{199}{0.85(4)(85.5)} = 0.68'' <$ assumed 1" ∴ ok

⑨ check unshored strength

W14X22, $\phi_b M_p = 125$ kft

$w_u = 1.2(75\ psf)(10ft) + 1.2(22\ plf) + 1.6(20\ psf)(10ft) = 1246$ plf

$M_u = \frac{1.246\ klf\ (28.5)^2}{8} = 126.5$ kft $> \phi_b M_p$ ∴ increase section

try W14X26

$w_u = 900 + 1.2(26) + 320 = 1251$ plf

$M_u = 127$ kft $< \phi_b M_p = 151$ kft ∴ ok for no shoring

⑩ check Δ_{wc}

$w_{wc} = 75\ psf\ (10ft) + 26\ plf = 776$ plf

$\Delta_{wc} = \frac{5(0.776\ klf)(28.5)^4(1728)}{384(29000)(245)} = 1.62''$

$\Delta_{wc,max} = \frac{28.5(12)}{240} = 1.43'' < \Delta_{wc}$ not ok

try W16X26: $\Delta_{wc} = \frac{5(0.776)(28.5)^4(1728)}{384(29000)(301)} = 1.32'' < \Delta_{wc,max}$ ok

⑪ check LL and TL deflection

W16X26:

$Y2 = 7$, point $Y1 = 7$ ($\Sigma Q_n = 76$ K), $I_{LB} = 640$ in^4

$\Delta_{LL} = \frac{5(0.878)(28.5)^4(1728)}{384(29000)(640)} = 0.70'' < \Delta_{LL,max} = 0.95''$

$\Delta_{TL} = \frac{5(1.728)(28.5)^4(1728)}{384(29000)(640)} = 1.38'' < \Delta_{TL,max} = 1.43''$

28'-6" X 40'-0" Bay page 4 of 9

(12) check M_u, V_u and brn self-wt assumption

$M_u = 252$ kft $< \phi M_n = 296$ kft ∴ ok

$V_u = 35.4$ k $< \phi V_n = 106$ k ∴ ok

self wt $= \frac{26 \text{ plf}}{10 \text{ ft}} = 2.6$ psf < 5 psf ∴ ok

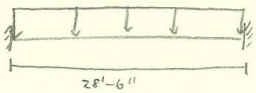
↑ W trib.

(13) check economy: W16x26: $\frac{\Sigma Q_n}{Q} \times 2 = \frac{96 \text{ k}}{12.2 \text{ k}} \times 2 \Rightarrow 12$ studs $< 28.5'$ ok

<u>use W16x26 w/ 12 studs</u>

28'-6" X 40'-0" Bay page 5 of 9

exterior bm B2



① total load

dead load:

slab - 75 psf

SDL - 10 psf

self-wt allow. - 5 psf

curtain wall - 250 plf

$w_D = (75 + 10 + 5)\ psf\ (6.5\ ft) + 250\ plf = 835\ plf$

live load:

$L_o = 100\ psf$

influence area = 28.5 ft (11.5 ft) = 327.75 ft^2 < 400 ft^2, LL reduction not applicable

$w_L = 100\ psf\ (6.5\ ft) = 650\ plf$

$w_u = 1.2(835) + 1.6(650) = 2042\ plf$

② $M_u = \frac{2.042\ klf\ (28.5\ ft)^2}{12} = 138.2\ kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{138.2\ kft\ (12)}{0.9\ (50\ ksi)} = 36.9\ in^3$

$V_u = \frac{2.042\ klf\ (28.5\ ft)}{2} = 29\ k \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.425"$

$\Delta_{TL} = \frac{wL^4}{384EI} \Rightarrow I_x \ge \frac{2.042\ (28.5)^4\ (1728)}{384\ (29000)(1.425)} = 146.7\ in^4$

④ select non-composite W-shape

try W12X26 beam

$Z_x = 37.2\ in^3 > 36.9\ in^3$ ok

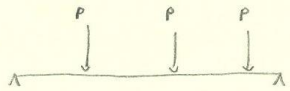
$I_x = 204\ in^4 > 146.7\ in^4$ ok

$\phi M_n = 140\ kft > 138.2\ kft$ ok

$\phi V_n = 84.2\ k > 29\ k$ ok

note: required bm size to support gravity load.
For bm size due to combined lateral and gravity analysis, refer to moment frame E elevation in report

Interior girder cal



① total load

dead load: slab = 75 psf
SDL = 10 psf
bm self wt allowance = 5 psf
girder self wt allowance = 5 psf

live load: $L_o = 100$ psf

$I.A. = (28.5 + 28.5)(40) = 2280\ ft^2 > 400\ ft^2 \therefore$ reduce LL

$$L = 100 \times \max \begin{cases} 0.50 \\ 0.25 + \frac{15}{\sqrt{2280}} = 0.56 \end{cases} = 56 \text{ psf}$$

28'-6" x 40'-0" Bay page 7 of 9

$w_u = 1.2(75 + 10 + 5 + 5) + 1.6(56) = 204$ psf

$P_u = 204 \text{ psf} \left(\frac{28.5 + 28.5}{2}\right)(10) = 58.1$ K

② $M_u = 0.50PL = 0.50(58.1 \text{ K})(40 \text{ ft}) = 1162$ kft

$V_u = 1.5P = 1.5(58.1 \text{ K}) = 87.2$ K

③ $\Delta_{LL} \leq \Delta_{LL,max}$

$\frac{0.050PL^3}{EI} \leq \frac{L}{360} = \frac{40(12)}{360} = 1.33''$

$I_{LB} \geq \frac{0.050(16 \text{ K})(40 \text{ ft})^3(1728)}{29000(1.33 \text{ in})} = 2294 \text{ in}^4$

$\Delta_{TL} \leq \Delta_{TL,max} = \frac{L}{240} = \frac{40(12)}{240} = 2''$

$I_{LB} \geq \frac{0.05(43 \text{ K})(40 \text{ ft})^3(1728)}{29000(2)} = 4100 \text{ in}^4$

④ required unshored strength

girder self-wt allowance

$P_u = 1.2[(75 + 5)\text{psf}(10 \text{ ft})](28.5 \text{ ft}) + 1.6(20 \text{ psf})(10 \text{ ft})(28.5 \text{ ft}) = 36.5$ K

slab

$M_u = 0.50PL = 0.50(36.5 \text{ K})(40 \text{ ft}) = 730 \text{ kft} \leq \phi_b M_p$

⑤ assume $a = 2''$, $Y2 = 7.5 - \frac{2}{2} = 6.5''$

⑥ select potential W-shapes

$(\phi M_n \geq 1162 \text{ kft}, I_{LB} \geq 4100 \text{ in}^4, \phi_b M_p \geq 730 \text{ kft})$

W24x76: $Y1 = BFL$, $I_{LB} = 4480 \text{ in}^4$, $\phi M_n = 1240$ kft, $\phi_b M_p = 750$ kft, $\Sigma Q_n = 509$ K

W24x84: $Y1 = 6$, $I_{LB} = 4550 \text{ in}^4$, $\phi M_n = 1290$ kft, $\phi_b M_p = 840$ kft, $\Sigma Q_n = 425$ K

⑦ # of studs/girder and economy

from table 3-21 in manual, $q_n = 21.5$ K

W24x76: $\frac{\Sigma Q_n}{q_n} \times 2 = \frac{509}{21.5} \times 2 = 48$ studs ; 76# x 40' + 48 studs x 10#/stud = 3520 #

W24x84: $\frac{425}{21.5} \times 2 = 40$ studs ; 84# x 40' + 40 studs x 10#/stud = 3760 #

try W24x76 since it's more economical

28'-6" x 40'-0" Bay page 8 of 9

⑧ check a

$a = \frac{\Sigma q_n}{0.85 f'c\ b_{eff}}$

$= \frac{509}{0.85(4)(120)} = 1.25'' < 2''$ ok

for interior girder, $b_{eff} = \begin{cases} \frac{2 \times span}{8} = \frac{2 \times 40(12)}{8} = 120'' \leftarrow \\ \text{min } 2 \times \frac{1}{2} \text{trib width} = 2 \times \frac{28.5(12)}{2} = 342'' \end{cases}$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumpt.

$\Delta_{LL} = \frac{0.050(16k)(40')^3(1728)}{29000(4480\ in^4)} = 0.68'' < 1.33''$ ok

$\Delta_{TL} = \frac{0.050(43k)(40')^3(1728)}{29000(4480)} = 1.83'' < 2''$ ok

unshored strength: $P_u = 1.2[(75psf)(10ft) + 76\ plf](28.5ft) + 1.6(20)(10ft)(28.5ft) = 37.4\ k$

$M_u = 0.50(37.4k)(40ft) = 748\ kft < \phi_b M_p = 750\ k$ ok

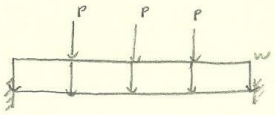
$M_u = 1162\ kft < \phi M_n = 1240\ kft$ ok

$V_u = 87.2\ k < \phi V_n = 315k$ ok

self wt $= \frac{76\ plf}{28.5ft} = 2.67\ psf < 5psf$ ok

<u>use W24x76 girder with 48 studs</u>

<u>Exterior Girder G2</u>



① load:

dead: slab - 75 psf

SDL - 10 psf

bm-wt allow. - 5 psf

girder self-wt allow. - 5 psf

wall - 250 plf

live: 100 psf x $\begin{cases} 0.50 \\ \text{max } 0.25 + \frac{15}{\sqrt{1200}} = 0.68 \end{cases}$ = 68 plf

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bru-wt allow.

$P_u = [1.2(75+10+5)\,psf + 1.6(68\,psf)](15.75\,ft)(10\,ft) = 34.2\,K$

$w_u = 1.2(5\,psf)(15.75\,ft) + 1.2(250\,plf) = 395\,plf$

girder self-wt allow.

③ $M_u = 0.313PL + \frac{wL^2}{12} = 0.313(34.2\,K)(40\,ft) + \frac{0.395\,klf(40\,ft)^2}{12} = 481\,kft \le \phi M_n = \phi M_p$

$V_u = 1.5P + \frac{wL}{2} = 1.5(34.2\,K) + \frac{0.395\,klf(40\,ft)}{2} = 59.2\,K \le \phi V_n$

$\phi M_p = \frac{\phi F_y Z_x}{12} \Rightarrow Z_x = \frac{481\,kft(12)}{0.9(50\,ksi)} = 128.3\,in^3$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 2.0''$

$\Delta_{TL} = \frac{0.010PL^3}{EI} + \frac{wL^4}{384EI} = \frac{0.010(34.2)(40)^3(1728)}{29000\,I_x} + \frac{0.395(40)^4(1728)}{384(29000)\,I_x} = \frac{1304}{I_x} + \frac{157}{I_x}$

$2.0'' = \frac{1461}{I_x} \Rightarrow I_x \ge 731\,in^4$

④ select non-composite W-section

W24x55 girder required to support gravity load

w/ $Z_x = 134\,in^3 > 128.3\,in^3$ ok

$I_x = 1350\,in^4 > 731\,in^4$ ok

$\phi M_n = 503\,kft > 481\,kft$ ok

$\phi V_n = 252\,K > 59.2\,K$ ok

note: Etabs used for lateral analysis

self wt $= \frac{55\,plf}{15.75\,ft} = 3.5\,psf < 5\,psf$ ok

- refer to moment frame 1 elevation in report for member sizes due to combined lateral and gravity analysis

Exterior Girder G3

use W24x55 w/ 32 studs

- refer to excel spreadsheet for design

28'-6" x 40'-0" Bay

Exterior Girder G3- simply supported with 3 point loads and a distributed dead load

Step 1 Dead Load: slab 75 psf
SDL 10 psf
bm self-wt allow. 5 psf
girder self-wt allow. 5 psf
wall 250 plf

Live Load: office space 100 psf

Trib. Width: 15.75 ft
Span, L: 40 ft
Influence Area: 1200 ft^2
LL_{red} 0.68

P_u= 34.2 kips
W_u= 0.395 klf

step 2 M_u= 763 k-ft ≤ ØMn
V_u= 59.2 kips ≤ ØVn

step 3 assume a=2" Y2=7-a/2=6.5"

step 4 $\Delta_{TL} \leq \Delta_{TL,max}$= L/240= 2 in
$P_{unfactored}$= 24.9 kips
$W_{unfactored}$= 0.329 klf
$I_{LB} \geq$ 2703 in^4

step 5 unshored strength $\phi_b M_p$

Dead Load: slab 75 psf
girder self-wt allow. 5 psf

Live Load: Constr. LL 20 psf

Trib. Width: 15.75 ft
Span, L: 40 ft

P_u= 20.2 kips
M_u= 403.2 k-ft ≤ $\phi_b M_p$

step 6 Select potential w-shapes
($\phi M_n \geq$ 763 kft, $I_{LB} \geq$ 2703 in4, ØbMp ≥ 403 kft

W21x57	Y1=	BFL		W24x55	Y1=	6	
	∑Qn	409	kips		∑Qn	329	kips
	ØMn=	850	kft		ØMn=	853	kft
	ØbMp=	484	kft		ØbMp=	503	kft
	I_{LB}=	2770			I_{LB}=	2920	

step 7 # of studs per girder and economy
stud strength 21.5 kips

W21x57	∑Qn/Q x2	38.05	→	40	studs
	economy:	2680	#		
W24x55	∑Qn/Q x2	30.6	→	32	studs
	economy:	2520	#		

try W24x55 w/ 32 studs since it's more economical

step 8 check a

b_{eff}= 120 in
231 in
78 in
min 360 in

∑Qn= 329 kips
f'c= 4 ksi
beff= 78 in

a= ∑Qn/(0.85x f'c xbeff) 1.24 ≤ 2.0 in ok

step 9 check ΔLL, ΔTL, unshored strengh, Mu, Vu, and girder self-wt assumption

$P_{L,\ unfactored}$= 10.8 kips
I_{LB}= 2920.0 in^4

Δ_{LL}= 0.70 in ≤ $\Delta_{LL,max}$= L/360 = 1.33 in ok

Δ_{TL}= 1.85 in ≤ $\Delta_{TL,max}$= L/240 = 2.0 in ok

Unshored strength ØbMp= 503 kft ≥ 403 kft ok

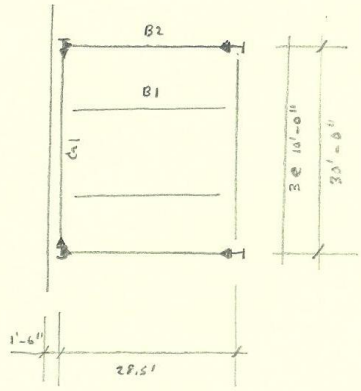
Mu= 763 kft ≤ ØMn = 853 kft ok

Vu= 59.2 kips ≤ ØVn=252 kips ok

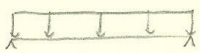
girder-self wt assumpt= girder wt/trib. Width = 55 plf/15.75 ft= 3.5 psf ≤ 5 psf ok

Girder Design: Use W24x55 with 32 studs

composite Flr System Design | 30'-0" x 28'-6" Bay | page 1 of 4



Interior Beam B1



① $w_D = (75 + 10 + 5)\ psf\ (10ft) = 900\ plf$

$LL = 100\ psf \times \begin{cases} 0.50 \\ \max\left(0.25 + \frac{15}{\sqrt{570}} = 0.878\right) \end{cases} = 87.8\ psf$

$w_L = 87.8\ psf\ (10ft) = 878\ plf$

$w_u = 1.2(900) + 1.6(878) = 2485\ plf$

② $M_u = \frac{w_u L^2}{8} = \frac{2.485(28.5)^2}{8} = 252\ kft$

$V_u = \frac{w_u L}{2} = \frac{2.485(28.5)}{2} = 35.4\ k$

③ assume $a = 1''$, $Y2 = 7''$

④ $\Delta_{TL} \le \Delta_{TL,max} = \frac{L}{240}$

$I_{LB} \ge \frac{5(0.90 + 0.878)(28.5)^4(1728)}{384(29000)(1.425)} = 639\ in^4$

$\Delta_{WC} \le \Delta_{WC,max} = L/240$

$w_{WC} = (75 + 5)\ psf\ (10ft) = 0.800\ klf$

$I_{non\text{-}comp} \ge \frac{5(0.80)(28.5)^4(1728)}{384(29000)(1.425)} = 287\ in^4$

③ required unshore strength, $\phi_b M_p$

bm self-wt allowance

$w_u = 1.2(75 + 5)\ psf\ (10ft) + 1.6(20\ psf)(10ft) = 1280\ plf$

$M_u = \frac{1.28(28.5)^2}{8} = 130\ kft \le \phi_b M_p$

30'-0" x 28'-6" Bay page 2 of 4

⑥ select potential w-shapes

($\phi M_n \geq 252$ kft, $I_{LB} \geq 639$ in^4, $\phi_b M_p \geq 130$ kft, $I \geq 282$ in^4)

W14x30: $Y1 = 6$, $\Sigma Q_n = 147k$, $I_{LB} = 717$ in^4, $I = 291$ in^4, $\phi M_n = 300$ kft, $\phi_b M_p = 177$ kft

W 16x26: $Y1 = 7$, $\Sigma Q_n = 96k$, $I_{LB} = 640$ in^4, $I = 301$ in^4, $\phi M_n = 259$ kft, $\phi_b M_p = 166$ kft

⑦ # of studs/bm and economy

W14x30: $\frac{147k}{17.2k} \times 2 \Rightarrow 18$ studs ; $30\# \times 28.5' + 18$ studs $\times 10\#/\text{stud} = 1035\#$

W 16x26: $\frac{96k}{17.2} \times 2 \Rightarrow 12$ studs ; $26\# \times 28.5' + 12$ studs $\times 16\#/\text{stud} = 861\#$

try W16x26 w/ 12 studs

⑧ check a

$b_{eff} = 85.5''$

$a = \frac{96}{0.85(4)(85.5)} = 0.33'' < 1''$ ok

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt assumption

$\Delta_{LL} = \frac{5(0.878\ \text{klf})(28.5\ \text{ft})^4(1728)}{384(29000)(640)} = 0.70'' < L/360 = 0.95''$ ok

$\Delta_{TL} = \frac{5(0.900 + 0.878)(28.5)^4(1728)}{384(29000)(640)} = 1.42'' < L/240 = 1.43''$ ok

$\Delta_{WC} = \frac{5(0.800)(28.5)^4(1728)}{384(29000)(301)} = 1.36'' < L/240$ ok

$M_u = 252$ kft $< \phi M_n = 259$ kft

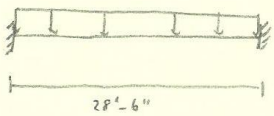
$V_u = 35.4$ k $< \phi V_n =$

bm self-wt $= \frac{26\ \text{plf}}{10\text{ft}} = 2.6$ psf < 5 psf ok

<u>use W16x26 bm w/ 12 studs</u>

30'-0" x 28'-6" Bay page 3 of 4

interior brm B2



① $w_D = (75 + 10 + 5)\,psf\,(9.75\,ft) = 877.5\,plf$

SDL; slab; brm self-wt allowance

$LL = 100\,psf \times \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{555.75}} = 0.886 \end{cases}$ (max) $= 88.6\,psf$

$w_L = 88.6\,psf\,(5 + 4.75)\,ft = 864\,plf$

$w_u = 1.2(877.5\,plf) + 1.6(864\,plf) = 2435\,plf$

② $M_u = \dfrac{w_u L^2}{12} = \dfrac{2.435(28.5)^2}{12} = 165\,kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \dfrac{165(12)}{0.9(50)} = 44\,in^3$

$V_u = \dfrac{2.435(28.5)}{2} = 34.7\,k \le \phi V_n$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240$

$I_x \ge \dfrac{(0.878 + 0.864)(28.5)^4(1728)}{384(29000)(1.425)} = 125\,in^4$

⑤ select non-composite w-shape

try W W16x26

$Z_x = 44.2\,in^3 > 44\,in^3$

$I_x = 301\,in^4 > 125\,in^4$ ok

$\phi M_n = 166\,kft > 165\,kft$ ok

$\phi V_n = 106\,k > 34.7\,k$ ok

note: W16x26 required to support gravity load.
Refer to moment frames B and C elevations in report for member sizes due to combined lateral and gravity analysis

30'-0" x 28'-6" Bay

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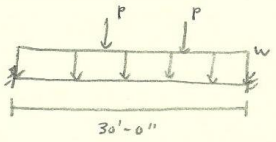
Exterior Girder Gr1



- refer to excel spreadsheet for design

30'-0" x 28'-6" Bay

Exterior Girder G1 - fixed with 2 point loads and a distributed dead load

Step 1 Dead Load: slab 75 psf
SDL 10 psf
bm self-wt allow. 5 psf
girder self-wt allow. 5 psf
wall 250 plf

Live Load: office space 100 psf

Trib. Width: 15.75 ft
Span, L: 30 ft
Influence Area: 900 ft^2
LL_{red} 0.75

P_u= 35.9 kips
W_u= 0.395 klf

step 2 M_u= 269 k-ft ≤ ØMn
V_u= 41.8 kips ≤ ØVn

step 3 assume a=2" Y2=7-a/2=6.5"

step 4 $\Delta_{TL} \leq \Delta_{TL,max}$= L/240= 1.5 in
$P_{unfactored}$= 26.0 kips
$W_{unfactored}$= 0.329 klf
$I_{LB} \geq$ 251 in^4

step 5 unshored strength $Ø_bM_p$

Dead Load: slab 75 psf
girder self-wt allow. 5 psf

Live Load: Constr. LL 20 psf

Trib. Width: 15.75 ft
Span, L: 30 ft

P_u= 20.2 kips
M_u= 134 k-ft ≤ $Ø_bM_p$

step 6 Select potential w-shapes
($ØM_n$≥269 kft, I_{LB} ≥ 251 in4, ØbMp ≥ 134 kft

16x26			W16x31		
Y1=	6		Y1=	7	
∑Qn	145	kips	∑Qn	114	kips
ØMn=	290	kft	ØMn=	309	kft
ØbMp=	166	kft	ØbMp=	203	kft
I_{LB}=	734	in^4	I_{LB}=	756	in^4

step 7 # of studs per girder and economy
stud strength 21.5 kips

W16x26 ∑Qn/Qx2 13.49 → 14 studs
economy: 920 #

W16x31 ∑Qn/Qx2 10.6 → 12 studs
economy: 1050 #

try W16x26 w/ 14 studs since it's more economical

step 8 check a

b_{eff}= 90 in
216 in
63 in
min 189 in

∑Qn= 145 kips
f'c= 4 ksi
beff= 63 in

a= ∑Qn/(0.85x f'c xbeff) 0.68 ≤ 2.0 in ok

step 9 check ΔLL, ΔTL, unshored strengh, Mu, Vu, and girder self-wt assumption

$P_{L,\ unfactored}$= 11.8 kips
I_{LB}= 734.0 in^4

Δ_{LL}= 0.21 in ≤ $\Delta_{LL,max}$= L/360 = 1.0 in ok

Δ_{TL}= 0.51 in ≤ $\Delta_{TL,max}$= L/240 = 1.5 in ok

Unshored strength ØbMp= 166 kft ≥ 134 kft ok

Mu= 269 kft ≤ ØMn = 290 kft ok

Vu= 41.8 kips ≤ ØVn=106 kips ok

girder-self wt assumpt= girder wt/trib. Width= 26 plf/15.75 ft= 1.65 psf ≤ 5 psf ok

Girder Design: Use W16x26 with 14 studs to support gravity load only

W16x26 w/ 14 studs required to support gravity load. Refer to moment frame 1 elevation in report for member sizes due to combined lateral and gravity load analysis
Note: The beams and girders in the moment frames were initially deigned as composite, but the design was changed to non-composite. The moment frame elevations consists of non-composite beam/girder members

41'-0" X 28'-6" Bay

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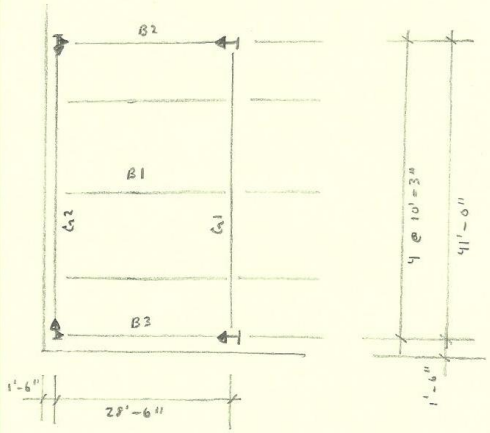
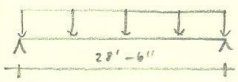


interior bm B1



① $w_D = (75 + 10 + 5)\,psf\,(10.25\,ft) = 923\ plf$

$$LL = 100\,psf \times \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{584.25}} = 0.871 \end{cases} = 87.1\ psf$$

$w_L = 87.1\,psf\,(10.25\,ft) = 893\ plf$

$w_u = 1.2(923) + 1.6(893) = 2536\ plf$

② $M_u = \dfrac{2.536\,(28.5)^2}{8} = 257\ kft$

$V_u = \dfrac{2.536\,(28.5)}{2} = 36.1\ k$

③ assume $a = 1''$, $Y2 = 7''$

④ $\Delta_{TL} \le \Delta_{TL,\,max} = L/240$

$$I_{LB} \ge \frac{5(0.923 + 0.893)(28.5)^4\,(1728)}{384\,(29000)(1.425)} = 652\ in^4$$

$\Delta_{WC} \le \Delta_{WC,\,max} = L/240$

$w_{WC} = (75 + 5)\,psf\,(10.25\,ft) = 0.820\ klf$

$$I_{non\text{-}comp.} = \frac{5(0.820)(28.5)^4\,(1728)}{384\,(29000)(1.425)} = 295\ in^4$$

⑤ required unshore strength, $\phi_b M_p$

$w_u = 1.2(75+5)(10.25) + 1.6(20)(10.25) = 1312\ plf$

$M_u = \dfrac{1.312\,(28.5)^2}{8} = 133\ kft \le \phi_b M_p$

41'-0" x 28'-6" Bay page 2 of 6

⑥ select potential w-shapes

($\phi M_n \geq 257$, $I_{LB} \geq 652\ in^4$, $\phi_b M_p \geq 133$ kft , $I \geq 295\ in^4$)

W14x34: $Z = 7$, $\Sigma Q = 125K$, $I_{LB} = 732\ in^4$, $I = 340\ in^4$, $\phi M_n = 316$ kft , $\phi_b M_p = 205$ kft

W16x26: $Z = 6$, $\Sigma Q_n = 145K$, $I_{LB} = 765\ in^4$, $I = 301\ in^4$, $\phi M_n = 296$ kft, $\phi_b M_p = 166$ kft

W16x31: $Z = 7$, $\Sigma Q_n = 114\ K$, $I_{LB} = 783\ in^4$, $I = 375\ in^4$, $\phi M_n = 313$ kft, $\phi_b M_p = 203$ kft

⑦ # of studs/bm and economy

W14x34: $\frac{125}{17.2} \times 2 \Rightarrow 16$ studs $< 28.5'$ ok ; $34\# \times 28.5' + 16$ studs $\times 10\#/stud = 1129\#$

W16x26: $\frac{145}{17.2} \times 2 \Rightarrow 18$ studs $< 28.5'$ ok ; $26\# \times 28.5' + 18$ studs $\times 10\#/stud = 921\#$

W16x31: $\frac{114}{17.2} \times 2 \Rightarrow 14$ studs $< 28.5'$ ok ; $31\# \times 28.5' + 14$ studs $\times 10\#/stud = 1024\#$

try W16x26 w/ 14 studs

⑧ check a

$a = \frac{145}{0.85(4)(85.5)} = 0.5'' < 1''$ ok

$b_{eff} = \begin{cases} 2 \times \frac{28.5(12)}{8} = 85.5'' \leftarrow \\ \min \begin{cases} 2 \times \frac{1}{2}(10.25)(12) = 123'' \end{cases} \end{cases}$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt assumpt.

$\Delta_{LL} = \frac{5(0.893)(28.5)^4(1728)}{384(29000)(765)} = 0.60'' < L/360 = 0.95''$ ok

$\Delta_{TL} = \frac{5(0.893 + 0.923)(28.5)^4(1728)}{384(29000)(765)} = 1.22'' < L/240 = 1.43''$ ok

$\Delta_{WC} = \frac{5(0.820)(28.5)^4(1728)}{384(29000)(301)} = 1.39'' < L/240$ ok

unshored strength: $M_u = 133$ kft $\leq \phi_b M_p = 166$ kft ok

$M_u = 257$ kft $\leq \phi M_n = 296$ kft ok

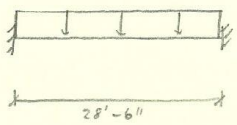
$V_u = 36.1\ K \leq \phi V_n = 106K$ ok

bm self-wt $= \frac{26\ plf}{10.25\ ft} = 2.54\ psf < 5\ psf$ ok

$\boxed{\text{use W16x26 w/ 14 studs}}$

41'-0" X 28'-6" Bay page 3 of 6

interior bm B2



① $w_D = (75 + 10 + 5)\,psf\,(10.125\,ft) = 911\,plf$

$$LL = 100\,psf \times \max\left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{577.13}} = 0.874 \end{array} \right. = 87.4\,psf$$

$w_L = 87.4\,psf\,(10.125\,ft) = 885\,plf$

$w_u = 1.2(911) + 1.6(885) = 2509\,plf$

② $M_u = \dfrac{w_u L^2}{12} = \dfrac{2.509\,klf\,(28.5\,ft)^2}{12} = 170\,kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \dfrac{170(12)}{0.9(50)} = 45.3\,in^3$

$V_u = \dfrac{2.509\,klf\,(28.5)}{2} = 35.8\,K \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240$

$I_x \ge \dfrac{(0.911 + 0.885)(28.5)^4(1728)}{384(29000)(1.425)} = 129\,in^4$

④ Select non-composite W-shape

W14X30 bm required to support gravity load

w/ $Z_x = 47.3\,in^3 > 45.3\,in^3$ ok

$I_x = 291\,in^4 > 129\,in^4$ ok

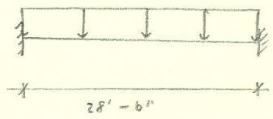
$\phi_b M_{px} = 177\,kft > 170\,kft$ ok

$\phi V_n = 112\,K > 35.8\,K$ ok

- refer to moment frame B elevation in report for member sizes due to combined lateral and gravity analysis

41'-6" x 28'-6" Bay page 4 of 6

exterior bm B3



① $w_D = (75 + 10 + 5)\,psf\,(6.625\,ft) + 250\,plf = 846.25\,plf$

$LL = 100\,psf$ $A_T = (10.25 + 1.5)\,ft\,(28.5\,ft) = 335\,ft^2 < 400\,ft^2$ ∴ LL reduction not required

$w_L = 100\,psf\,(6.625\,ft) = 662.5\,plf$

$w_u = 1.2\,(846.25) + 1.6\,(662.5) = 2076\,plf$

② $M_u = \frac{2.076\,(28.5)^2}{12} = 141\,k\,ft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{141\,(12)}{0.9\,(50)} = 37.6\,in^3$

$V_u = \frac{2.076\,(28.5)}{2} = 29.6\,k \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = \frac{28.5\,(12)}{240} = 1.425"$

$I_x \ge \frac{(0.846 + 0.663)(28.5)^4\,(1728)}{384\,(29000)\,(1.425)} = 108\,in^4$

④ select non-composite w-shape

W14X26 bm required to support gravity load

w/ $Z_x = 40.2\,in^3 > 37.6\,in^3$ OK
$I_x = 245\,in^4 > 108\,in^4$ OK

$\phi_b M_{px} = 151\,k\,ft > 141\,k\,ft$ OK
$\phi V_n = 106\,k > 29.6\,k$ OK

- refer to moment frame A.1 elevation in report for member sizes due to combined lat. and gravity analysis

41'-0" X 28'-6" Bay page 5 of 6

interior girder G1

girder self-wt allow.

① $W_D = (75 + 10 + 5 + 5)\ psf = 95\ psf$

bm wt allow.

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{2603.5}} = 0.544 \end{array} \right. = 54.4\ psf$

$W_u = 1.2(95\ psf) + 1.6(54.4\ psf) = 201\ psf$

$P_u = 201\ psf \left(\dfrac{28.5 + 35}{2}\right) ft\ (10.25 ft) = 65.4\ k$

② $M_u = 0.50\,PL = 0.50(65.4\ k)(41\ ft) = 1341\ kft$

③ assume $a = 2"$, $y_2 = 6.5"$

$V_u = 1.5P = 1.5(65.4) = 98.1\ k$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 41(12)/240 = 2.05"$

$I_{LB} \ge \dfrac{0.050\,(48.6\ K)(41\ ft)^3(1728)}{29000\,(2.05)} = 4868\ in^4$

⑤ required unshored strength, $\phi_b M_p$

$P_u = 1.2[(75+5)\ psf\ (10.25\ ft)](31.75\ ft) + 1.6(20)(10.25)(31.75) = 41.7\ k$

$M_u = 0.50(41.7k)(41ft) = 855\ kft \le \phi_b M_p$

⑥ select potential W-shapes
($\phi M_n \ge 1341\ kft$, $I_{LB} \ge 4868\ in^4$, $\phi_b M_p \ge 855\ kft$)

W24X94: $y_1 = 6$ $\Sigma Q_n = 469$, $\phi M_n = 1450\ kft$, $\phi_b M_p = 953\ kft$, $I_{LB} = 5140\ in^4$

W27X84: $y_1 = 6$ $\Sigma Q_n = 452\ K$, $\phi M_n = 1420\ kft$, $\phi_b M_p = 915\ kft$, $I_{LB} = 5460\ in^4$

⑦ # of studs / girder and economy

W24X94: $\dfrac{469}{21.5K} \times 2 = 44$ studs ; $94\# \times 41' + 44$ studs $\times 10\#/stud = 4294\#$

W27X84: $\dfrac{452}{21.5K} \times 2 = 44$ studs ; $84\# \times 41' + 44$ studs $\times 10\#/stud = 3884\#$

try W27X84 w/ 44 studs since it's more economical

41'-0" X 28'-6" Bay page 6 of 6

⑧ check a

$a = \frac{452}{0.85(4)(123)} = 1.08" < 2"$ ok

$b_{eff} = \begin{cases} 2 \times \frac{41(12)}{8} = 123" \leftarrow \text{controls} \\ \text{min} \begin{cases} \frac{1}{2}(28.5)(12) + \frac{1}{2}(35)(12) = 381" \\ \frac{41(12)}{8} + \frac{1}{2}(28.5)(12) = 61.5 + 171 = 232.5" \\ \frac{41(12)}{8} + \frac{1}{2}(35)(12) = 61.5 + 210 = 271.5" \end{cases} \end{cases}$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumption

$\Delta_{LL} = \frac{0.050(17.7K)(41ft)^3(1728)}{29000(5460 in^4)} = 0.67" < L/360 = 1.37"$ ok

$\Delta_{TL} = \frac{0.050(48.6K)(41ft)^3(1728)}{29000(5460 in^4)} = 1.83" < L/240 = 2.05"$ ok

unshored strength: $P_u = 1.2[(75psf)(10.25ft) + 84plf](31.75ft) + 1.6(20psf)(10.25)(31.75) = 42.9K$

$M_u = 0.50(42.9K)(41ft) = 879 Kft \leq \phi_b M_p = 915 Kft$ ok

$M_u = 1341 Kft \leq \phi M_n = 1420 Kft$ ok

$V_u = 98.1 K \leq \phi V_n = 368 K$ ok

girder self-wt $= \frac{84 plf}{31.75 ft} = 2.65 psf < 5psf$ assumed ok

$\boxed{\text{use W27 X 84 w/ 44 studs}}$

Exterior Girder G2

- refer to excel spreadsheet for design

41'-0" x 28'-6" Bay

Exterior Girder G2- fixed with 3 point loads and a distributed dead load

Step 1	Dead Load:	slab	75	psf
		SDL	10	psf
		bm self-wt allow.	5	psf
		girder self-wt allow.	5	psf
		wall	250	plf
	Live Load:	office space	100	psf
	Trib. Width:	15.75	ft	
	Span, L:	41.0	ft	
	Influence Area:	1230	ft^2	
	LL_{red}	0.68		
	P_u=	34.9	kips	
	W_u=	0.395	klf	
step 2	M_u=	504	k-ft	≤ ØMn
	V_u=	60.5	kips	≤ ØVn
step 3	assume a=2"	Y2=7-a/2=6.5"		
step 4	$\Delta_{TL} \leq \Delta_{TL,max}$=	L/240=	2.05	in
	$P_{unfactored}$=	25.5	kips	
	$W_{unfactored}$=	0.329	klf	
	$I_{LB} \geq$	581	in^4	
step 5	unshored strength $Ø_bM_p$			
	Dead Load:	slab	75	psf
		girder self-wt allow.	5	psf
	Live Load:	Constr. LL	20	psf
	Trib. Width:	15.75	ft	
	Span, L:	41	ft	
	P_u=	20.7	kips	
	M_u=	265	k-ft	≤ $Ø_bM_p$

step 6 Select potential w-shapes

($ØM_n \geq$ 504 kft, $I_{LB} \geq$ 581 in4, ØbMp ≥ 265 kft

W16x40	Y1=	4		W18x40	Y1=	BFL	
	∑Qn	325	kips		∑Qn	274	kips
	ØMn=	509	kft		ØMn=	523	kft
	ØbMp=	274	kft		ØbMp=	294	kft
	I_{LB}=	1400	in^4		I_{LB}=	1510	in^4

step 7 # of studs per girder and economy

stud strength 21.5 kips

W16x40	∑Qn/Q x2	30.23	→	32 studs
	economy:	1960 #		
W18x40	∑Qn/Q x2	25.5	→	26 studs
	economy:	1900 #		

try W18x40 w/ 26 studs since it's more economical

step 8 check a

b_{eff}=	123	in
	232.5	in
	79.5	in
min	189	in
∑Qn=	274	kips
f'c=	4	ksi
beff=	79.5	in

a= ∑Qn/(0.85xf'c xbeff) 1.01 ≤ 2.0 in ok

step 9 check ΔLL, ΔTL, unshored strengh, Mu, Vu, and girder self-wt assumption

$P_{L\ unfactored}$=	10.9	kips		
I_{LB}=	1510.0	in^4		
Δ_{LL}=	0.30	in ≤ $\Delta_{LL,max}$= L/360 =	1.4	in ok
Δ_{TL}=	0.79	in ≤ $\Delta_{TL,max}$= L/240 =	2.05	in ok

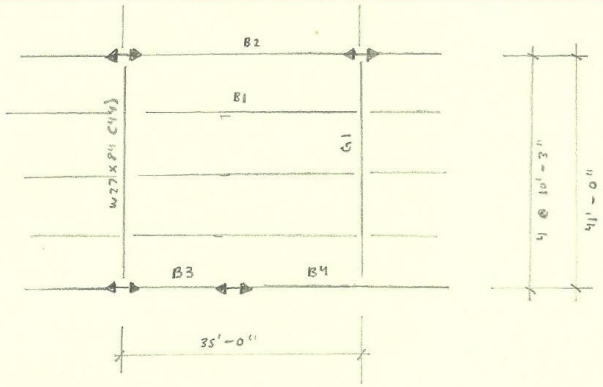
Unshored strength	ØbMp=	294 kft ≥ 265 kft	ok for no shoring
Mu=	504	kft ≤ ØMn = 523 kft	ok
Vu=	60.5	kips ≤ ØVn=106 kips	ok
girder-self wt assumpt= girder wt/trib. Width = 40 plf/15.75 ft = 2.54 psf			≤ 5 psf ok

Girder Design: Use W18x40 with 26 studs to support gravity load only

W18x40 w/ 26 studs required to support gravity load. Refer to moment frame 1 elevation in report for member sizes due to combined lateral and gravity load analysis

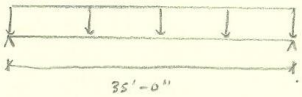
Note: The beams and girders in the moment frames were initially deigned as composite, but the design was changed to non-composite. The moment frame elevations consists of non-composite beam/girder members

41'-0" x 35'-0" Bay page 1 of 5



interior brn B1

① $w_D = (75 + 10 + 5)\,psf\,(10.25\,ft) = 922.5\ plf$



$LL_o = 100\ psf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{717.5}} = 0.81 \end{array} \right. = 81\ psf$

$w_L = 81\ psf\,(10.25\,ft) = 830.25\ plf$

$w_u = 1.2(922.5) + 1.6(830.25) = 2435\ plf$

② $M_u = \dfrac{2.435\ klf\,(35\,ft)^2}{8} = 373\ kft \le \phi M_n$

$V_u = \dfrac{2.435\,(35)}{2} = 42.6\ k \le \phi V_n$

③ assume $a = 1"$, $y_2 = 7"$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.75"$

$I_{LB} \ge \dfrac{5(0.923 + 0.830)(35)^4\,(1728)}{384\,(29000)\,(1.75)} = 1166\ in^4$

$\Delta_{wc} \le \Delta_{wc,max} = L/240$

$w_{wc} = (75 + 5)\,psf\,(10.25\,ft) = 0.820\ klf$

$I_{non\text{-}comp} \ge \dfrac{5(0.820)(35)^4\,(1728)}{384\,(29000)\,(1.75)} = 546\ in^4$

⑤ required unshored strength, $\phi_b M_p$

$w_u = 1.2(75 + 5)(10.25) + 1.6(20)(10.25) = 1312\ plf$

$M_u = \dfrac{1.312\,(35)^2}{8} = 201\ kft \le \phi_b M_p$

41'-0" x 35'-0" Bay page 2 of 5

⑥ select potential W-shapes

($\phi M_n \geq 373$, $I_{LB} \geq 1166\ in^4$, $\phi_b M_p \geq 201$ kft, $I \geq 546\ in^4$)

W16x45: $y_1 = 7$, $\Sigma Q_n = 166$ k, $I_{LB} = 1190\ in^4$, $I = 586\ in^4$, $\phi M_n = 467$ kft, $\phi_b M_p = 305$ kft

W18x40: $y_1 = 7$, $\Sigma Q_n = 148$ K, $I_{LB} = 1210\ in^4$, $I = 612\ in^4$, $\phi M_n = 446$ kft, $\phi_b M_p = 294$ kft

⑦ # of studs/ bm and economy

W16x45: $\frac{166K}{17.2k} \times 2 \Rightarrow$ 20 studs < 35' ok ; 45# x 35' + 20 studs x 10#/stud = 1775 #

W18x40: $\frac{148K}{17.2\ k} \times 2 \Rightarrow$ 18 studs < 35' ok ; 40# x 35' + 18 studs x 10 #/stud = 1580 #

try W18x40 w/ 18 studs since it's more economical

⑧ check a

$a = \frac{148}{0.85(4)(105)} = 0.41" < 1"$

$b_{eff} = 2 \times \frac{35(12)}{8} = 105" \leftarrow$

min: $2 \times \frac{1}{2}(10.25)(12) = 123"$

$\frac{35(12)}{8} + \frac{1}{2}(10.25)(12) = 114"$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt assumption

$\Delta_{LL} = \frac{5(0.830)(35)^4(1728)}{384(29000)(1210)} = 0.80" < L/360 = 1.17"$ ok

$\Delta_{TL} = \frac{5(1.753)(35)^4(1728)}{384(29000)(1210)} = 1.69" < L/240 = 1.75"$ ok

$\Delta_{WC} = \frac{5(0.820)(35)^4(1728)}{384(29000)(612)} = 1.56" < L/240 = 1.75"$ ok

unshored strength: $M_u = 201$ kft $\leq \phi_b M_p = 294$ kft ok

$M_u = 373$ kft $\leq \phi M_n = 446$ kft ok

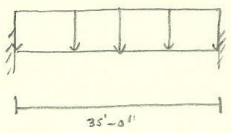
$V_u = 42.6K \leq \phi V_n = 169$ K ok

bm self-wt $= \frac{40\ plf}{10.25\ ft} = 3.9$ psf < 5psf assumed ok

$\boxed{\text{use W18x40 w/ 18 studs}}$

41'-0" X 35'-0" Bay page 3 of 5

interior bm B2



① $w_D = (75 + 10 + 5)\,psf\,(10.125\,ft) = 911.25\ plf$

$L_o = 100\ psf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{708.75}} = 0.813 \end{array} \right. = 81.3\ psf$

$w_L = 81.3\ psf\,(10.125\,ft) = 823\ plf$

$w_u = 1.2(911.25) + 1.6(823) = 2410\ plf$

② $M_u = \dfrac{2.410\ klf\,(35\,ft)^2}{12} = 246\ kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \dfrac{246\,(12)}{0.9\,(50)} = 65.6\ in^3$

$V_u = \dfrac{2.410\,(35)}{2} = 42.2\ K \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.75"$

$I_x \ge \dfrac{(0.911 + 0.823)(35)^4(1728)}{384(29000)(1.75)} = 231\ in^4$

④ select non-composite W-shape

W18X35 bm required to support gravity load

w/ $Z_x = 66.5\ in^3 > 65.6\ in^3$ ok

$I_x = 510\ in^4 > 231\ in^4$ ok

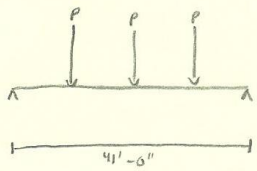
$\phi_b M_{px} = 249\ kft > 246\ kft$ ok

$\phi V_n = 159\ K > 42.2\ K$ ok

- refer to moment frame B elevation in report for member sizes due to combined lat. and gravity analysis

41'-0" x 35'-0" Bay page 4 of 5

interior girder Bal



① $w_D = (75 + 10 + 5 + 5) = 95$ psf

(bm wt allow.; girder self-wt allow.)

$LL = 100 \text{ psf} \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{260S}} = 0.54 \end{array} \right. = 54.1$ psf

$w_u = 1.2(95) + 1.6(54.1) = 200.6$ psf

$P_u = 200.6 \text{ psf} \left(\frac{30+35}{2}\right) \text{ft} (10.25 \text{ ft}) = 66.8 \text{ K}$

② $M_u = 0.5PL = 0.5(66.8 \text{ K})(41 \text{ ft}) = 1369$ kft

$V_u = 1.5P = 1.5(66.8 \text{ k}) = 100$ k

③ assume $a = 2$, $Y2 = 6.5''$

④ $\Delta_{TL} \leq \Delta_{TL,max} = L/240 = 2.05''$

$I_{LB} \geq \frac{0.050(49.7k)(41ft)^3(1728)}{29000(2.05)} = 4978 \text{ in}^4$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2[(75+5)\text{psf}(10.25\text{ft})](32.5\text{ft}) + 1.6(20)(10.25)(32.5) = 42.6$ k

$M_u = 0.50(42.6k)(41ft) = 873 \text{ kft} \leq \phi_b M_p$

⑥ select potential W-shapes

($\phi M_n \geq 1369$ kft, $I_{LB} \geq 4978 \text{ in}^4$, $\phi_b M_p \geq 873$ kft)

W24x94: $Y1 = 6$, $I_{LB} = 5140 \text{ in}^4$, $\phi M_n = 1450$ kft, $\phi_b M_p = 953$ kft, $\Sigma Q = 469$ K

W27x84: $Y1 = 6$, $\Sigma Q = 452$ K, $I_{LB} = 5460 \text{ in}^4$, $\phi M_n = 1420$ kft, $\phi_b M_p = 915$ kft

⑦ # of studs / girder and economy

W24x94: $\frac{469K}{21.5k} \times 2 \Rightarrow 44$ studs ; $94\# \times 41' + 44 \text{ studs} \times 10\#/\text{stud} = 4294\#$

W27x84: $\frac{452K}{21.5K} \times 2 \Rightarrow 44$ studs ; $84\# \times 41' + 44 \text{ studs} \times 10\#/\text{stud} = 3884\#$

try W27x84 w/ 44 studs

41'-0" X 35'-0" Bay page 5 of 5

⑧ check a

$$a = \frac{452}{0.85(4)(123)} = 1.08'' < 2'' \text{ ok}$$

$$b_{eff} = \begin{cases} 2 \times \frac{41(12)}{8} = 123'' \leftarrow \\ \min \begin{cases} \frac{1}{2}(35)(12) + \frac{1}{2}(30)(12) = 390'' \end{cases} \end{cases}$$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumption

$$\Delta_{LL} = \frac{0.050(18K)(41ft)^3(1728)}{29000(5460)} = 0.68'' < L/360 = 1.37''$$

$$\Delta_{TL} = \frac{0.050(49.7K)(41ft)^3(1728)}{29000(5460)} = 1.87'' < L/240 = 2.05''$$

unshored strength: $P_u = 1.2[75psf(10.25ft) + 84plf](32.5ft) + 1.6(20psf)(10.25)(32.5) = 43.9K$

$M_u = 0.50(43.9K)(41ft) = 900 \text{ kft} \leq \phi_b M_p = 915 \text{ kft ok}$

$M_u = 1369 \text{ kft} \leq \phi M_n = 1420 \text{ kft ok}$

$V_u = 100K \leq \phi V_n = 368K$ ok

girder self-wt $= \frac{84plf}{32.5ft} = 2.58 \text{ psf} < 5psf$ assumed ok

$\boxed{\text{use W27x84 w/ 44 studs}}$

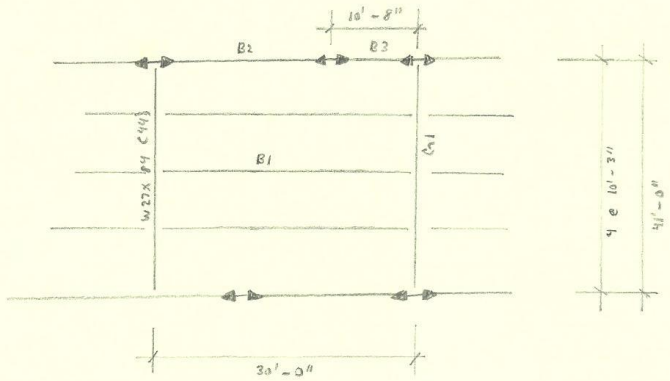
Exterior bm B3

- refer to excel spreadsheet for design

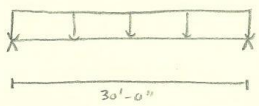
41'-0"x 35'-0" Bay								
Exterior Beam B3- fixed with a distributed load								
Step 1	Dead Load:	slab	75	psf				
		SDL	10	psf				
		bm self-wt allow.	5	psf				
		girder self-wt allow.	not applicable					
		wall	250	plf				
	Live Load:	office space	100	psf				
	Trib. Width:	8	ft					
	Span, L:	17.5	ft					
	Influence Area:	280	ft^2					
	LL_{red}	1.00						
	P_u=	0.0	kips					
	W_u=	2.44	klf					
step 2	M_u=	62	k-ft	≤ ØMn=ØMp		Zx ≥	16.6	in^3
	V_u=	21.4	kips	≤ ØVn				
step 3	$\Delta_{TL} \leq \Delta_{TL,max}$=	L/240=	0.875	in				
	$P_{unfactored}$=	0.0	kips					
	$W_{unfactored}$=	0.970	klf					
	$I_x \geq$	16	in^4					
step 4	Select non-composite W-shape							
	W 12x14 required to support gravity load							
	note: refer to moment frame A.1 elevation in report for member sizes due to combined lateral and gravity							
	analysis							

41'-0" x 30'-0" Bay

page 1 of 6



interior bm B1



① $w_D = (75 + 10 + 5)\ psf\ (10.25\ ft) = 922.5\ plf$

$L_o = 100\ psf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{615}} = 0.855 \end{array} \right. = 85.5\ psf$

$w_L = 85.5\ psf\ (10.25\ ft) = 876\ plf$

$w_u = 1.2(922.5) + 1.6(876) = 2509\ plf$

② $M_u = \frac{2.509(30)^2}{8} = 282\ kft \le \phi M_n$

$V_u = \frac{2.509(30)}{8} = 37.6\ K \le \phi V_n$

③ assume $a = 1''$, $y_2 = 7''$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = \frac{30(12)}{240} = 1.5''$

$I_{LB} \ge \frac{5(0.923 + 0.876)(30)^4(1728)}{384(29000)(1.5)} = 754\ in^4$

$\Delta_{wc} \le \Delta_{wc,max} = L/240$

$w_{wc} = (75 + 5)\ psf\ (10.25\ ft) = 0.820\ klf$

$I_{non\text{-}comp} \ge \frac{5(0.820)(30)^4(1728)}{384(29000)(1.5)} = 344\ in^4$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75 + 5)\ psf\ (10.25\ ft) + 1.6(20\ psf)(10.25\ ft) = 1312\ plf$

$M_u = \frac{1.312(30)^2}{8} = 148 \le \phi_b M_p$

41'-0" x 30'-0" Bay page 2 of 6

⑥ select potential w-shapes

($\phi M_n \geq 282$ kft, $I_{LB} \geq 754$ in^4, $\phi_b M_p \geq 148$ kft, $I_{NC} \geq 344$ in^4)

W16x31: 41 = 7 $\Sigma \phi_n = 114k$, $I_{LB} = 783$ in^4, $I = 375$ in^4, $\phi M_n = 313$ kft, $\phi_b M_p = 203$ kft

⑦ # of studs/bm and economy

W16x31: $\frac{114k}{17.2k} \times 2 \Rightarrow$ 14 studs < 30' ok ; 31# x 30' + 14 studs x 10#/stud = 1070 #

⑧ check a

$a = \frac{114}{0.85(4)(90)} = 0.37" < 1"$ ok

$b_{eff} = \frac{2 \times 30(12)}{8} = 90"$ ←

min $2 \times \frac{1}{2}(10.25)(12) = 123"$

$\frac{30(12)}{8} + \frac{1}{2}(10.25)(12) = 106.5"$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt assumption

$\Delta_{LL} = \frac{5(0.876)(30)^4(1728)}{384(29000)(783)} = 0.70" < 4/360 = 1"$ ok

$\Delta_{TL} = \frac{5(1.80)(30)^4(1728)}{384(29000)(783)} = 1.44" < 4/240 = 1.5"$ ok

$\Delta_{WC} = \frac{5(0.820)(30)^4(1728)}{384(29000)(375)} = 1.37" < 4/240 = 1.5"$ ok

unshored strength: $M_u = 148$ kft $\leq \phi_b M_p = 203$ kft ok

$M_u = 282$ kft $\leq \phi M_n = 313$ kft ok

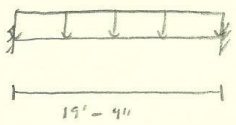
$V_u = 37.6$ kft $\leq \phi V_n = 131k$ ok

bm self-wt $= \frac{31 \text{ plf}}{10.25 \text{ ft}} = 3.02$ psf < 5psf assumed ok

$\boxed{\text{use W16x31 w/ 14 studs}}$

41'-0" X 30'-0" Bay page 3 of 6

interior bm B2



① $w_D = (75 + 10 + 5)\ psf\ (10.125\ ft) = 911.25\ plf$

$L_o = 100\ psf$, $I.A. = (10.25 + 10)\ ft\ (19.33\ ft) = 391.4\ ft^2 < 400\ ft^2$ ∴ LL reduction not required

$w_L = 100\ (10.125) = 1012.5\ plf$

$w_u = 1.2\ (911.25) + 1.6\ (1012.5) = 2714\ plf$

② $M_u = \frac{2.714\ klf\ (19.33)^2}{12} = 84.5\ kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{84.5\ (12)}{0.9\ (50)} = 22.5\ in^3$

$V_u = \frac{2.714\ klf\ (19.33\ ft)}{2} = 26.2\ K \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 0.97"$

$I_x \ge \frac{(0.911 + 1.013)(19.33)^4\ (1728)}{384\ (29000)(0.97)} = 43\ in^4$

④ select non-composite W-shape

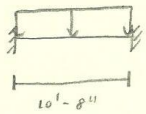
W 12X19 bm required to support gravity load w/ $Z_x = 24.7\ in^3 > 22.5\ in^3$ ok
$I_x = 130\ in^4 > 43\ in^4$ ok

$\phi_b M_{px} = 92.6\ kft > 84.5\ kft$ ok
$\phi V_n = 86\ K > 26.2\ K$ ok

- refer to moment frame B elevation in report for member sizes due to combined lat. and gravity load analysis

41'-0" X 30' X 0" Bay page 4 of 6

interior bm B3



① $w_D = (75 + 10 + 5)\ psf\ (10.125\ ft) = 911.25\ plf$

$L_o = 100\ psf$, $T.A. = (10.25 + 10)ft\ (10.67\ ft) = 216\ ft^2 < 400\ ft^2$ ∴ LL reduction not required

$w_L = 100\ psf\ (10.125\ ft) = 1012.5\ plf$

$w_u = 1.2\ (911.25) + 1.6\ (1012.5) = 2714\ plf$

② $M_u = \frac{2.714\ klf\ (10.67\ ft)^2}{12} = 25.7\ kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{25.7\ (12)}{0.9\ (50)} = 6.9\ in^3$

$V_u = \frac{2.714\ (10.67)}{2} = 14.5\ k \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 0.53"$

$I_{LB} \ge \frac{(0.912 + 1.013)(10.67)^4\ (1728)}{384\ (29000)\ (0.53)} = 7.3\ in^4$

④ select non-composite W-shape

W8x10 bm required to support gravity load

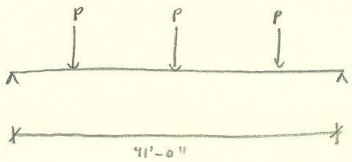
w/ $Z_x = 8.87\ in^3 > 6.9\ in^3$ ok
$I_x = 30.8\ in^4 > 7.3\ in^4$ ok

$\phi_b M_{px} = 32.9\ kft > 25.7\ kft$ ok
$\phi V_n = 40.2\ k > 14.5\ k$ ok

- refer to moment frame B elevation in report for member sizes due to combined lat. and gravity load analysis

41'-0" x 30'-0" Bay | page 5 of 6

interior girder Cal



① $W_D = (75 + 10 + 5 + 5) = 95$ psf

bm wt. allow.

girder self-wt allow.

$LL = 100 \text{ psf} \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{2255}} = 0.566 \end{array} \right|_{max} = 56.6 \text{ psf}$

$W_u = 1.2(95) + 1.6(56.6) = 204.6$ psf

$P_u = 204.6 \text{ psf} \left(\dfrac{30+25}{2}\right) ft (10.25 ft) = 57.7$ K

② $M_u = 0.5(57.7 k)(41 ft) = 1183$ kft $\leq \phi M_n$

$V_u = 1.5(57.7 k) = 86.6$ K $\leq \phi V_n$

③ assume $a = 2"$ $Y2 = 6.5"$

④ $\Delta_{TL} \leq \Delta_{TL, max} = L/240 = 2.05"$

$I_{LB} \geq \dfrac{0.050(42.7 k)(41 ft)^3 (1728)}{29000(2.05)} = 4277 \text{ in}^4$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2[(75+5) psf (10.25 ft)](27.5 ft) + 1.6(20 psf)(10.25 ft)(27.5 ft) = 36.1$ K

$M_u = 0.50(36.1 k)(41 ft) = 740 \leq \phi_b M_p$

⑥ select potential w-shapes

($\phi M_n \geq 1183$ kft, $I_{LB} \geq 4277 \text{ in}^4$, $\phi_b M_p \geq 740$ kft)

w24x84: $Y1 = 6$, $\Sigma Q_n = 425$ K, $I_{LB} = 4550 \text{ in}^4$, $\phi M_n = 1290$ kft, $\phi_b M_p = 840$ kft

w27x84: $Y1 = 7$, $\Sigma Q_n = 309$ K, $I_{LB} = 4800 \text{ in}^4$, $\phi M_n = 1300$ kft, $\phi_b M_p = 915$ kft

⑦ # of studs/girder and economy

w24x84: $\dfrac{425 k}{21.5 k} \times 2 \Rightarrow 40$ studs ; $84\# \times 41' + 40 \text{ studs} \times 10\#/\text{stud} = 3844\#$

w27x84: $\dfrac{309}{21.5} \times 2 \Rightarrow 30$ studs ; $84\# \times 41' + 30 \text{ studs} \times 10\#/\text{stud} = 3744\#$

try w27x84 w/ 30 studs

41'-0" x 30'-0" Bay page 6 of 6

⑧ check a

$a = \frac{309}{0.85(4)(123)} = 0.74" < 2"$ ok

$b_{eff} = 2 \times \frac{41(12)}{8} = 123"$ ←

min $\frac{1}{2}(30)(12) + \frac{1}{2}(25)(12) = 330"$

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumption

$\Delta_{LL} = \frac{0.050(16K)(41ft)^3(1728)}{29000(4800 in^4)} = 0.68" < L/360 = 1.37"$ ok

$\Delta_{TL} = \frac{0.050(42.7K)(41ft)^3(1728)}{29000(4800 in^4)} = 1.83" < L/240 = 2.05"$ ok

unshored strength: $P_u = 1.2[(75 psf)(10.25ft) + 84 plf](27.5ft) + 1.6(20 psf)(10.25ft)(27.5ft) = 37.2K$

$M_u = 0.50(37.2K)(41ft) = 763 kft < \phi_b M_p = 915 kft$ ok

$M_u = 1183 < \phi M_n = 1300 kft$ ok

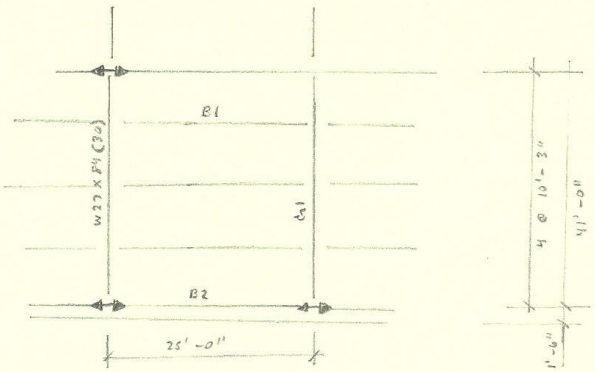
$V_u = 86.6K < \phi V_n = 368K$ ok

girder self-wt $= \frac{84 plf}{27.5 ft} = 3.05 psf < 5 psf$ assumed ok

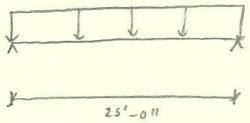
$\boxed{\text{use W27x84 w/ 30 studs}}$

41'-0" x 25'-0" Bay

page 1 of 5



interior bm B1



① $w_D = (75 + 10 + 5)\ psf\ (10.25\ ft) = 922.5\ plf$

$L_o = 100\ psf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{512.5}} = 0.913 \end{array} \right. = 91.3\ psf$

$w_L = 91.3\ psf\ (10.25\ ft) = 936\ plf$

$w_u = 1.2(922.5) + 1.6(936) = 2605\ plf$

② $M_u = \dfrac{2.605\ klf\ (25\ ft)^2}{8} = 204\ kft \le \phi M_n$

$V_u = \dfrac{2.605\ (25)}{2} = 32.7\ K \le \phi V_n$

③ assume $a = 1"$, $y_2 = 7"$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.25"$

$I_{LB} \ge \dfrac{5(0.923 + 0.936)(25)^4(1728)}{384\ (29000)(1.25)} = 451\ in^4$

$\Delta_{WL} \le \Delta_{WL,max} = L/240$

$w_{WL} = (75 + 5)\ psf\ (10.25\ ft) = 0.820\ klf$

$I_{non\text{-}comp} \ge \dfrac{5(0.820)(25)^4(1728)}{384\ (29000)(1.25)} = 199\ in^4$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75 + 5)\ psf\ (10.25\ ft) + 1.6(20\ psf)(10.25\ ft) = 1312\ plf$

$M_u = \dfrac{1.312\ (25)^2}{8} = 103\ kft \le \phi_b M_p$

41'-0" x 25'-0" Bay page 2 of 5

⑥ select potential w-shape

($\phi M_n \geq 204$ kft, $I_{LB} \geq 451$ in^4, $\phi_b M_p \geq 103$ kft, $I_{NC} \geq 199$ in^4)

w12x26: $41 = 7$, $\Sigma Q_n = 95.6$ k, $I_{LB} = 467$ in^4, $I = 204$ in^4, $\phi M_n = 218$ kft, $\phi_b M_p = 140$ kft

w14x22: $41 = 6$, $\Sigma Q_n = 119$k, $I_{LB} = 532$ in^4, $I = 199$ in^4, $\phi M_n = 225$ kft, $\phi_b M_p = 125$ kft

⑦ # of studs/bm and economy

w12x26: $\frac{95.6\ k}{17.2\ k} \times 2 \Rightarrow 12$ studs $< 25'$ ok ; $26\# \times 25' + 12$ studs $\times 10\#/\text{stud} = 770\#$

w14x22: $\frac{119k}{17.2k} \times 2 \Rightarrow 14$ studs $< 25'$ ok ; $22\# \times 25' + 14$ studs $\times 10\#/\text{stud} = 690\#$

try w14x22 w/ 14 studs since it's more economical

⑧ check a

$a = \frac{119}{0.85(4)(75)} = 0.47'' < 1''$ ok

$b_{eff} = 2 \times \frac{25(12)}{8} = 75''$ ←

min $2 \times \frac{1}{2}(10.25)(12) = 123''$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm-self-wt

$\Delta_{LL} = \frac{5(0.936)(25)^4(1728)}{384(29000)(533)} = 0.53'' < L/360 = 0.83''$ ok

$\Delta_{TL} = \frac{5(1.86)(25)^4(1728)}{384(29000)(533)} = 1.06'' < L/240 = 1.25''$ ok

$\Delta_{WC} = \frac{5(0.820)(25)^4(1728)}{384(29000)(199)} = 1.25'' \leq L/240 = 1.25''$ ok

unshored strength: $M_u = 103$kft $\leq \phi_b M_p = 125$ kft ok

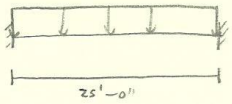
$M_u = 204$ kft $\leq \phi M_n = 225$ kft ok
$V_u = 32.7$ k $\leq \phi V_n = 94.5$ k ok

bm self-wt $= \frac{22\ plf}{10.25\ ft} = 2.15$ psf < 5 psf assumed ok

$\boxed{\text{use w14x22 w/ 14 studs}}$

41'-0" x 25'-0" Bay | page 3 of 5

exterior bm B2



① $w_D = (10 + 75 + 5)\,psf\,(6.625\,ft) + 250\,plf = 846.25\,plf$ (curtain wall wt)

$LL = 100\,psf$, $I.A. = (10.25 + 1.5)\,ft\,(25\,ft) = 293.75\,ft^2 < 400\,ft^2$ ∴ LL reduction not required

$w_L = 100\,(6.625) = 662.5\,plf$

$w_u = 1.2\,(846.25) + 1.6\,(662.5) = 2076\,plf$

② $M_u = \frac{2.076\,klf\,(25\,ft)^2}{12} = 108\,kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \frac{108\,(12)}{0.9\,(50)} = 28.8\,in^3$

$V_u = \frac{2.076\,(25)}{2} = 26\,k \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.25"$

$I_{LB} \ge \frac{(0.846 + 0.663)(25)^4\,(1728)}{384\,(29000)(1.25)} = 73\,in^4$

④ select non-composite W-shape

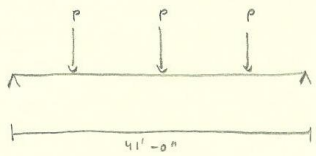
W12x22 required to support gravity load w/ $Z_x = 29.3\,in^3 > 28.8\,in^3$ ok
$I_x = 156\,in^4 > 73\,in^4$ ok

$\phi_b M_{p_x} = 110\,kft > 108\,kft$ ok
$\phi V_n = 95.9\,k > 26\,k$ ok

- refer to moment frame A.1 elevation in report for member sizes due to combined lat. and gravity load analysis

41'-0" x 25'-0" Bay page 4 of 5

interior girder Cal



① $W_o = (75 + 10 + 5 + 5) = 95$ psf

girder self-wt allow.

bm wt allow.

$LL = 100 \text{ psf} \times \max\left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{1742.5}} = 0.61 \end{array} \right. = 61$ psf

$w_u = 1.2(95) + 1.6(61) = 211.6$ psf

$P_u = 211.6 \text{ psf} \left(\frac{25 + 17.5}{2}\right) \text{ft} (10.25 \text{ ft}) = 46.1$ K

② $M_u = 0.5(46.1)(41) = 945$ kft $\leq \phi M_n$

$V_u = 1.5(46.1 \text{ k}) = 69.2$ k

③ assume $a = 1''$, $y_2 = 6.5''$

④ $\Delta_{TL} \leq \Delta_{TL,max} = L/240 = 2.05''$

$I_{LB} \geq \frac{0.050(34k)(41ft)^3(1728)}{29000(2.05)} = 3406 \text{ in}^4$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2[(75+5) \text{ psf } (10.25 \text{ ft})](21.25 \text{ ft}) + 1.6(20 \text{ psf})(10.25 \text{ ft})(21.25 \text{ ft}) = 27.9$ K

$M_u = 0.50(27.9 \text{ k})(41 \text{ ft}) = 572$ kft $\leq \phi_b M_p$

⑥ select potential W-shapes

($\phi M_n \geq 945$ kft, $I_{LB} \geq 3406 \text{ in}^4$, $\phi_b M_p \geq 572$ kft)

W24x68: $y_1 = 6$, $\Sigma Q_n = 366$K, $I_{LB} = 3640 \text{ in}^4$, $\phi M_n = 1050$ kft, $\phi_b M_p = 664$ kft

W24x76: $y_1 = 7$, $\Sigma Q_n = 280$ k, $I_{LB} = 3630 \text{ in}^4$, $\phi M_n = 1070$ kft, $\phi_b M_p = 750$ kft

41'-0" x 25'-0" Bay page 5 of 5

⑦ # of studs / girder and economy

W 24x68: $\frac{366\ K}{21.5\ K} \times 2 \Rightarrow 36$ studs ; $68\# \times 41' + 36$ studs $\times 10\#/\text{stud} = 3148\#$

W 24x76: $\frac{280\ K}{21.5\ K} \times 2 \Rightarrow 28$ studs ; $76\# \times 41' \times 28$ studs $\times 10\#/\text{stud} = 3396\#$

try W24x68 w/ 36 studs since it's more economical

⑧ check a

$a = \frac{366}{0.85(4)(123)} = 0.88" < 2"$ OK

$b_{eff} = \begin{cases} 2 \times \frac{41(12)}{8} = 123" \leftarrow \\ \frac{1}{2}(25)(12) + \frac{1}{2}(17.5)(12) = 255" \end{cases}$ min

⑨ check Δ_{LL}, Δ_{TL}, unshored strength, M_u, V_u, girder self-wt assumpt.

$\Delta_{LL} = \frac{0.050(13.3K)(41ft)^3(1728)}{29000(3640\ in^4)} = 0.75" \leq L/360 = 1.37"$ OK

$\Delta_{TL} = \frac{0.050(34K)(41ft)^3(1728)}{29000(3640\ in^4)} = 1.92" \leq L/240 = 2.05"$ OK

unshored strength: $P_u = 1.2[(75\ psf)(10.25\ ft) + 68\ plf](21.25\ ft) + 1.6(20\ psf)(10.25\ ft)(21.25\ ft) = 28.3K$

$M_u = 0.50(28.3K)(41ft) = 580.2\ kft \leq \phi_b M_p = 664\ kft$ OK

$M_u = 945\ kft \leq \phi M_n = 1050\ kft$ OK

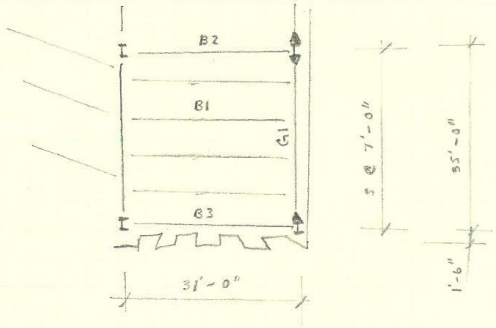
$V_u = 69.2\ K \leq \phi V_n = 295\ K$ OK

self-wt $= \frac{68\ plf}{21.25\ ft} = 3.2\ psf \leq 5\ psf$ assumed OK

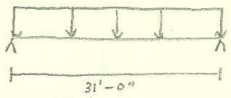
$\boxed{\text{use W24X68 w/ 36 studs}}$

35'-0" X 31'-0" Bay

page 1 of 6



interior bm B1



① $w_o = (75 + 10 + 5)\ psf\ (7ft) = 630\ plf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.56 \\ 0.25 + \frac{15}{\sqrt{434}} = 0.97 \end{array} \right. = 97\ psf$

$w_L = 97\ (7ft) = 679\ plf$

$w_u = 1.2(630) + 1.6(679) = 1842\ plf$

② $M_u = \frac{1.842\ klf\ (31ft)^2}{8} = 221\ kft \le \phi M_n$

$V_u = \frac{1.842\ (31)}{2} = 28.6\ k \le \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.55"$

$I_{LB} \ge \frac{5(0.630 + 0.679)(31)^4(1728)}{384(29000)(1.55)} = 605\ in^4$

$\Delta_{WC} \le \Delta_{WC,max} = L/240$

$w_{WC} = (75+5)\ psf\ (7ft) = 560\ plf$

$I_{non\text{-}comp.} \ge \frac{5(0.560)(31)^4(1728)}{384(29000)(1.55)} = 259\ in^4$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75+5)\ psf\ (7ft) + 1.6(20\ psf)(7ft) = 896\ plf$

$M_u = \frac{0.896\ (31)^2}{8} = 108 \le \phi_b M_p$

35'-0" x 31'-0" Bay page 2 of 6

⑥ select potential w-shapes

($\phi M_n \geq 221$ kft, $I_{LB} \geq 605$ in^4, $\phi_b M_p \geq 108$ kft, $I_{NC} \geq 259$ in^4)

W14X30: $Y1 = 7$, $\Sigma Q_n = 111K$, $I_{LB} = 634$ in^4, $I = 291$ in^4, $\phi M_n = 275$ kft, $\phi_b M_p = 177$ kft

W16X26: $Y1 = 7$, $\Sigma Q_n = 96K$, $I_{LB} = 640$ in^4, $I = 301$ in^4, $\phi M_n = 259$ kft, $\phi_b M_p = 166$ kft

⑦ # of studs / bm and economy

W14X30: $\frac{111K}{17.2K} \times 2 \Rightarrow 14$ studs < 31' ok ; $30\# \times 31' + 14$ studs $\times 10\#/\text{stud} = 1070\#$

W16X26: $\frac{96K}{17.2K} \times 2 \Rightarrow 12$ studs < 31' ok ; $26\# \times 31' + 12$ studs $\times 10\#/\text{stud} = 926\#$

try W16X26 w/ 12 studs since it's more economical

⑧ check a

$a = \frac{96}{0.85(4)(84)} = 0.34" < 1"$ ok

$b_{eff} =$
- $2 \times \frac{31(12)}{8} = 93"$
- $2 \times \frac{1}{2}(7)(12) = 84"$ ←
- min: $\frac{31(12)}{8} + \frac{1}{2}(7)(12) = 88.5"$

⑨ check Δ_{LL}, Δ_{TL}, Δ_{WC}, unshored strength, M_u, V_u, bm self-wt

$\Delta_{LL} = \frac{5(0.675)(31)^4(1728)}{384(29000)(640)} = 0.76" \leq L/360 = 1.03"$ ok

$\Delta_{TL} = \frac{5(1.31)(31)^4(1728)}{384(29000)(640)} = 1.47" \leq L/240 = 1.55"$ ok

$\Delta_{WC} = \frac{5(0.560)(31)^4(1728)}{384(29000)(301)} = 1.33" \leq L/240 = 1.55"$ ok

unshored strength: $M_u = 108$ kft $\leq \phi_b M_p = 166$ kft ok

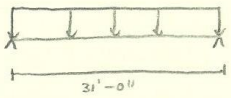
$M_u = 221$ kft $\leq \phi M_n = 259$ kft ok
$V_u = 28.6K \leq \phi V_n = 106K$ ok

self-wt $= \frac{26 \text{ plf}}{7 \text{ ft}} = 3.7$ psf ≤ 5 psf assumed ok

$\boxed{\text{use W16X26 w/ 12 studs}}$

35'-0" x 31'-0" Bay | page 3 of 6

interior bm B2



① $w_o = (75 + 10 + 5)\ psf\ (8.5\ ft) = 765\ plf$

$LL = 100\ psf \times \left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{527}} = 0.903 \end{array} \right. = 90.3\ plf$

$w_L = 90.3\ (8.5) = 768\ plf$

$w_u = 1.2\ (765) + 1.6\ (768) = 2147\ plf$

② $M_u = \frac{2.147\ klf\ (31ft)^2}{8} = 258\ kft \le \phi M_n$

$V_u = \frac{2.147\ (31)}{2} = 33.3\ k \le \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.55"$

$I_{LB} \ge \frac{5\ (0.765 + 0.768)(31)^4\ (1728)}{384\ (29000)(1.55)} = 709\ in^4$

$\Delta_{wc} \le \Delta_{wc,max} = L/240$

$w_{wc} = (75 + 5)\ psf\ (8.5\ ft) = 680\ plf$

$I_{non\text{-}comp} \ge \frac{5\ (0.680)(31)^4\ (1728)}{384\ (29000)(1.55)} = 314\ in^4$

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2\ (75 + 5)\ psf\ (8.5\ ft) + 1.6\ (20\ psf)(8.5\ ft) = 1088\ plf$

$M_u = \frac{1.088\ klf\ (31ft)^2}{8} = 131\ kft \le \phi_b M_p$

⑥ select potential W-shapes

$(\phi M_n \ge 258\ kft,\ I_{LB} \ge 709\ in^4,\ \phi_b M_p \ge 131\ kft,\ I_{NC} \ge 314\ in^4)$

W14x34: $Y1 = 7$, $\Sigma Q_n = 125\ k$, $I_{LB} = 732\ in^4$, $I = 340\ in^4$, $\phi M_n = 316\ kft$, $\phi_b M_p = 205\ kft$

W16x31: $Y1 = 7$, $\Sigma Q_n = 114\ k$, $I_{LB} = 783\ in^4$, $I = 375\ in^4$, $\phi M_n = 313\ kft$, $\phi_b M_p = 203\ kft$

⑦ # of studs/bm and economy

W14x34: $\frac{125k}{17.2k} \times 2 \Rightarrow 16$ studs < 31' ; $34\#/ft \times 31' + 16$ studs $\times 10\#/stud = 1214\#$

W16x31: $\frac{114k}{17.2k} \times 2 \Rightarrow 14$ studs < 31' ; $31\#/ft \times 31' + 14$ studs $\times 10\#/stud = 1101\#$

try W16x31 w/ 14 studs since it's more economical

35'-0" X 31'-0" Bay page 4 of 6

⑧ check a

$a = \frac{114}{0.85(4)(88.5)} = 0.38" < 1"$ ok

$b_{eff} = \frac{2 \times 31(12)}{8} = 93"$

$\frac{1}{2}(7)(12) + \frac{1}{2}(10)(12) = 102"$

$\frac{31(12)}{8} + \frac{1}{2}(7)(12) = 88.5" \leftarrow$

min $\frac{31(12)}{8} + \frac{1}{2}(10)(12) = 106.5"$

⑨ Summary

$\Delta_{LL} = \frac{5(0.768)(31)^4(1728)}{384(29000)(783)} = 0.73" \leq L/360 = 1.03"$ ok

$\Delta_{TL} = \frac{5(1.53)(31)^4(1728)}{384(29000)(783)} = 1.40" \leq L/240 = 1.55"$ ok

$\Delta_{NC} = \frac{5(0.670)(31)^4(1728)}{384(29000)(375)} = 1.30" \leq L/240 = 1.55"$ ok

unshored strength: $M_u = 131$ kft $\leq \phi_b M_p = 203$ kft ok

$M_u = 258$ kft $\leq \phi M_p = 313$ kft ok

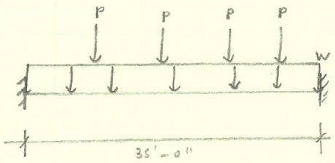
$V_u = 33.3$ k $\leq \phi V_n = 131$ k ok

self-wt $= \frac{31 \text{ plf}}{8.5 \text{ ft}} = 3.64$ psf < 5 psf assumed ok

$\boxed{\text{use W16X31 w/ 19 studs}}$

35'-0" x 31'-0" Bay | page 5 of 6

exterior girder G1



① load:

Dead: slab - 75 psf
SDL - 10 psf
bm wt allow. - 5 psf
girder self wt allow - 5 psf
wall - 250 plf

Live: 100 psf x $\left| \begin{array}{l} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{1085}} = 0.705 \end{array} \right.$ = 70.5 psf

$P_u = \left[1.2\,(75 + 10 + \underset{\text{bm self-wt allow.}}{5})\,psf + 1.6\,(70.5\,psf)\right](17\,ft)(7\,ft) = 26.3\,K$

$w_u = 1.2\,(5\,psf)(17\,ft) + 1.2\,(250\,plf) = 402\,plf$

② $M_u = 0.40PL + \dfrac{wL^2}{12} = 0.40\,(26.3\,K)(35\,ft) + \dfrac{0.402\,klf\,(35\,ft)^2}{12} = 409\,kft \le \phi M_n = \phi M_p \Rightarrow Z_x \ge \dfrac{409\,(12)}{0.9\,(50)} = 109\,in^3$

$V_u = 2.0P + \dfrac{wL}{2} = 2.0\,(26.3\,K) + \dfrac{0.402\,klf\,(35\,ft)}{2} = 59.6\,K \le \phi V_n$

③ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.75''$

$\Delta_{TL} = \dfrac{0.013\,PL^3}{EI_x} + \dfrac{wL^4}{384\,EI_x} = \dfrac{0.013\,(19.1\,K)(35\,ft)^3(1728)}{29000\,I_x} + \dfrac{0.335\,klf\,(35\,ft)^4(1728)}{384\,(29000)\,I_x} = \dfrac{634.3}{I_x} + \dfrac{78}{I_x}$

$1.75'' = \dfrac{712.3}{I_x} \Rightarrow I_x \ge 407\,in^4$

④ select non-composite W-shape

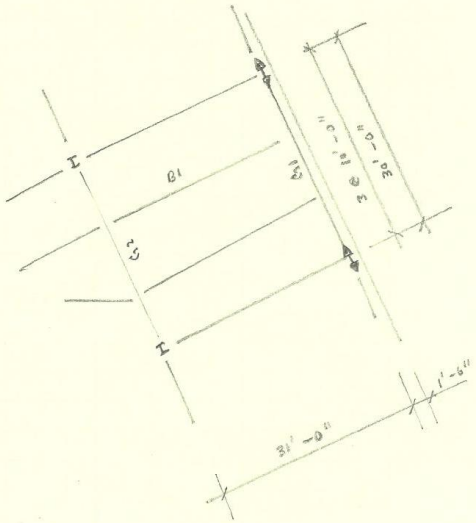
W 18x55 required to support gravity load

w/ $Z_x = 112\,in^3 > 109\,in^3$ ok
$I_x = 890\,in^4 > 407\,in^4$ ok

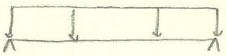
$\phi_b M_{px} = 420\,kft > 409\,kft$ ok
$\phi V_n = 212\,K > 59.6\,K$ ok

- refer to moment frame 1' elevation in report for member sizes due to combined lat. and gravity load analysis

30'-0" X 31'-0" page 1 of 4



interior bm B1



① $w_D = (75 + 10 + 5)\,psf\,(10\,ft) = 900\ plf$

$LL = 100\,psf \times \left| \begin{array}{l} 0.56 \\ 0.25 + \dfrac{15}{\sqrt{620}} = 0.852 \end{array} \right. = 85.2\ psf$

$w_L = 85.2\,psf\,(10\,ft) = 852\ plf$

$w_u = 1.2(900) + 1.6(852) = 2443\ plf$

② $M_u = \dfrac{2.443\,klf\,(31\,ft)^2}{8} = 293\ kft \le \phi M_n$

$V_u = \dfrac{2.443(31)}{2} = 38\,k \le \phi V_n$

③ assume $a = 1"$, $Y2 = 7"$

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.55"$

$I_{LB} \ge \dfrac{5(0.90 + 0.852)(31)^4(1728)}{384(29000)(1.55)} = 810\ in^4$

$\Delta_{WC} \le \Delta_{WC,max} = L/240$

$w_{WC} = (75 + 5)\,psf\,(10\,ft) = 800\ plf$

$I_{non\text{-}comp} = \dfrac{5(0.80\,klf)(31\,ft)^4(1728)}{384(29000)(1.55)} = 370\ in^4$

30'-0" X 31'-6" Bay page 2 of 4

⑤ unshored strength, $\phi_b M_p$

$w_u = 1.2(75+5)\,psf\,(10ft) + 1.6(20\,psf)(10ft) = 1280\,plf$

$M_u = \frac{1.28\,klf\,(31ft)^2}{8} = 154\,kft \le \phi_b M_p$

⑥ select potential W-shapes

$[\phi M_n \ge 293\,kft,\ I_{LB} \ge 810\,in^4,\ \phi_b M_p \ge 154\,kft,\ I_{NC} \ge 375\,in^4]$

W 16x31: 41 = 6, $\Sigma Q_n = 114k$, $I_{LB} = 914\,in^4$, $I = 375\,in^4$, $\phi M_n = 350\,kft$, $\phi_b M_p = 203\,kft$

⑦ # of studs/bm and economy

W16x31: $\frac{114\,K}{17.2\,K} \times 2 \Rightarrow 14$ studs < 31' ok; $31\,\# \times 31' + 14\,studs \times 16\,\#/stud = 1101\,\#$

⑧ check a

$a = \frac{114\,K}{0.85(4\,ksi)(93\,in)} = 0.36" < 1"$ ok

$b_{eff} = 2 \times \frac{31(12)}{8} = 93" \leftarrow$

$2 \times \frac{1}{2}(10)(12) = 120"$

min $\frac{31(12)}{8} + \frac{1}{2}(10)(12) = 106.5"$

⑨ summary

$\Delta_{LL} = \frac{5(0.852)(31)^4(1728)}{384(29000)(914)} = 0.67" \le L/360 = 1.03"$ ok

$\Delta_{TL} = \frac{5(1.75)(31)^4(1728)}{384(29000)(914)} = 1.37" \le L/240 = 1.55"$ ok

$\Delta_{NC} = \frac{5(0.80)(31)^4(1728)}{384(29000)(375)} = 1.53" \le L/240$ ok

unshored strength: $M_u = 154\,kft \le \phi_b M_p = 203\,kft$ ok

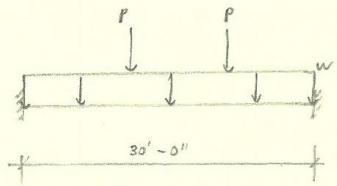
$M_u = 293\,kft \le \phi M_n = 350\,kft$ ok
$V_u = 38\,K \le \phi V_n = 131\,K$ ok

Self-wt = $\frac{31\,plf}{10\,ft} = 3.1\,psf < 5\,psf$ assumed ok

$\boxed{\text{use W16X31 w/ 14 studs}}$

30'-0" x 31'-0" Bay page 3 of 4

exterior girder G1



① dead loads: slab - 75 psf
SDL - 10 psf
bm wt allow. - 5 psf
girder self-wt allow. - 5 psf
curtain wall - 250 plf

Live Load: 100 psf x $\left| \begin{array}{l} 0.50 \\ 0.25 + \frac{15}{\sqrt{975}} = 0.73 \end{array} \right.$ = 73 psf

$P_u = [1.2\,(75 + 10 + \underset{\text{bm wt. allow.}}{5})\text{ psf} + 1.6\,(73\text{ psf})]\,(17\text{ ft})(10\text{ ft}) = 38.2\text{ K}$

$w_u = 1.2\,(5\text{ psf})(17\text{ ft}) + 1.2\,(250\text{ plf}) = 402\text{ plf}$

② $M_u = 0.222\,PL + \frac{wL^2}{12} = 0.222\,(38.2\text{ k})(30\text{ ft}) + \frac{0.402\text{ klf}\,(30\text{ ft})^2}{12} = 285\text{ kft} \le \phi M_n$

$V_u = 1.0\,P = 38.2\text{ k} \le \phi V_n$

③ assume a = 2" y2 = 6.5"

④ $\Delta_{TL} \le \Delta_{TL,max} = L/240 = 1.5"$

$\Delta_{TL} = \frac{0.008\,PL^3}{EI} + \frac{wL^4}{384EI} = \frac{0.008\,(27.7\text{ k})(30\text{ ft})^3\,(1728)}{29000\,I_{LB}} + \frac{0.335\text{ klf}\,(30\text{ ft})^4\,(1728)}{384\,(29000)\,I_{LB}} = \frac{356.5}{I_{LB}} + \frac{42.1}{I_{LB}}$

$1.5" = \frac{398.6}{I_{LB}} \Rightarrow I_{LB} \ge 266\text{ in}^4$

⑤ unshored strength, $\phi_b M_p$

$P_u = 1.2\,[(75 + \underset{\text{girder self-wt allow.}}{5})\text{ psf}\,(10\text{ ft})]\,(17\text{ ft}) + 1.6\,(20\text{ psf})(10\text{ ft})(17\text{ ft}) = 21.8\text{ K}$

$M_u = 0.222\,(21.8\text{ k})(30\text{ ft}) = 145\text{ kft} \le \phi_b M_p$

30'-0" x 31'-0" Bay | page 4 of 4

⑥ select potential W-shapes
($\phi M_n \geq 285$ kft, $I_{LB} \geq 266$ in^4, $\phi_b M_p \geq 145$ kft)

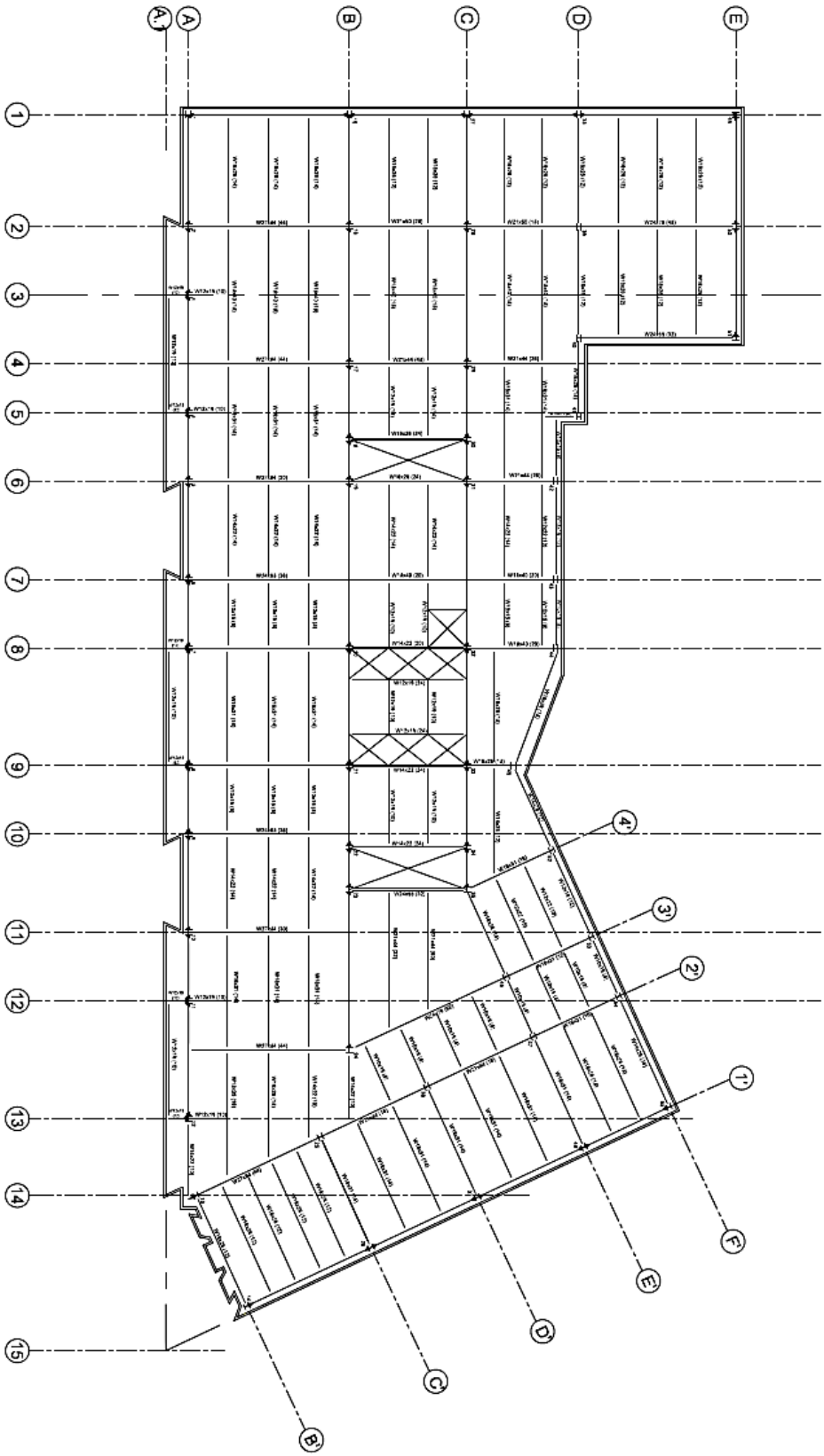
W16x31: $41 = 7$ $\Sigma Q_n = 114$ k, $I_{LB} = 783$ in^4, $\phi M_n = 313$ kft, $\phi_b M_p = 203$ kft

⑦ # of studs / girder and economy

W16x31: $\frac{114}{245} \times 2 \Rightarrow 12$ studs ; $31\# \times 30' + 12$ studs $\times$ $10\#$/stud $= 1050\#$

note: beams and girders in moment frames were initially designed as composite, but the design of the lateral beam and girder members were changed to non-composite

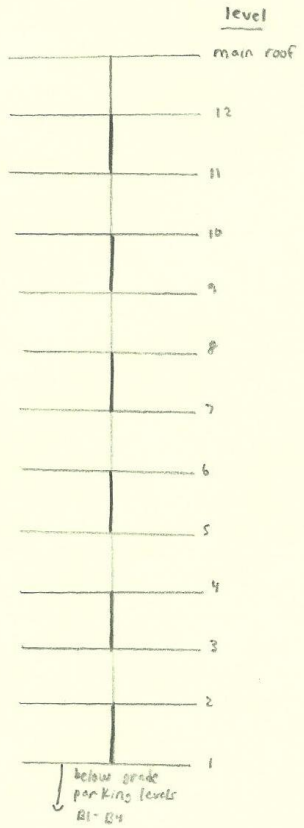
- refer to moment frame 1' elevation in report for member sizes due to combined lateral and gravity load analysis



TYPICAL FLOOR FRAMING PLAN

Column Design

Column Design | General info | page 1 of 1



Notes:

- floor-to-floor height is 15 feet
- the columns will be spliced every 2 tiers (the darkly highlighted columns in the figure to the left will be designed throughout the ht. of each column)

Loads

main roof:

dead loads: slab - 75 psf
SDL - 10 psf
member self-wt. allowance - 10 psf

live loads: balconies and terraces - 100 psf
green roof areas - 30 psf
mech. room - 150 psf
pump rm / generator rm - 150 psf

snow loads: 15.75 psf (flat roof snow load)

levels 2-12:

dead loads: slab - 75 psf
SDL - 10 psf
member self-wt allowance - 10 psf

live loads: office floors - 100 psf

penthouse roof:

dead loads: slab - 75 psf
SDL - 5 psf
member self-wt allowance - 10 psf

live loads: green roof areas - 30 psf

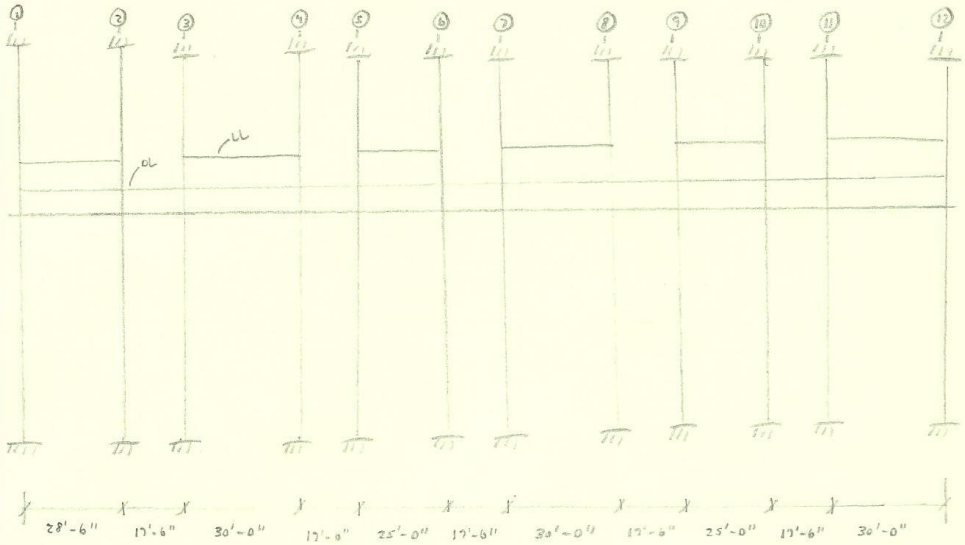
snow loads: 15.75 psf (flat roof snow load)

roof constr. LL: 20 psf

- refer to excel spreadsheets for the gravity columns analyses and sizes

Estimate column sizes | moment frame A.1 | page 1 of 13

Estimate unbalanced moment in interior and exterior columns



dead loads : slab - 75 psf
SDL - 10 psf
brn self-wt allow. - 5 psf
curtain wall - 250 plf

live loads: office space - 100 psf

note: - the slab transfers the curtain wall self-wt to the exterior brns
- the 30'-0" span brn a part of moment frame A.1 do not support the curtain wall self-wt

28'-6" span

$w_D = (75 + 10 + 5)\,psf\,(6.63\,ft) + 250\,plf = 847\,plf$

influence area = $(10.25 + 1.5)\,ft\,(28.5\,ft) = 335\,ft^2 < 400\,ft^2$, LL reduction not applicable

$w_L = 100\,psf\,(6.63\,ft) = 663\,plf$

$w_u = 1.2(847) + 1.6(663) = 2077\,plf$

$FEM_u = \frac{wL^2}{12} = \frac{2.077\,klf\,(28.5\,ft)^2}{12} = 141\,k\,ft$

17'-6" span

$w_D = (90\,psf)(10.88\,ft) + 250 = 1229\,plf$

$w_u = 1.2(1229) = 1475\,plf$

$FEM_u = \frac{1.475\,klf\,(17.5\,ft)^2}{12} = 37.6\,k\,ft$

Estimate Column sizes | moment frame A.1 | page 2 of 13

30'-0" span

$w_D = 90\ psf\ (7.38\ ft) = 664\ plf$

influence area = $(4.5 + 10.25)\ ft\ (30\ ft) = 442.5\ ft^2 > 400\ ft^2$, LL reduction required

$$LL_{red} = \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{442.5}} = 0.96 \end{cases} \quad max$$

$w_L = 0.96\ (100\ psf)(7.38\ ft) = 708.5\ plf$

$w_u = 1.2(664) + 1.6(708.5) = 1930\ plf$

$FEM_u = \dfrac{1.930\ (30)^2}{12} = 145\ kft$

25'-0" span

$w_D = 90\ psf\ (6.63\ ft) + 250 = 847\ plf$

influence area = $(10.25 + 1.5)\ ft\ (25\ ft) = 294\ ft^2 < 400\ ft^2$, LL reduction not applicable

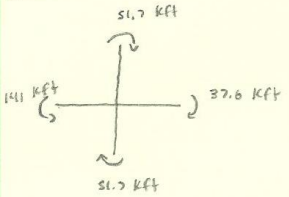
$w_L = 100\ psf\ (6.63\ ft) = 663\ plf$

$w_u = 1.2(847) + 1.6(663) = 2077\ plf$

$FEM_u = \dfrac{2.077\ (25)^2}{12} = 108\ kft$

Estimate column sizes | moment frame A.1 | page 3 of 13

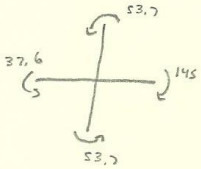
col 2



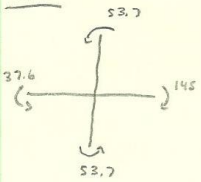
UBM = 141 - 37.6 = 103.4 kft

assume: half of UBM is distributed to col above and half to col. below

col 7

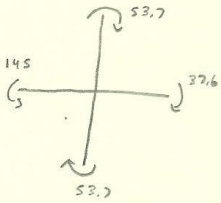


col 3

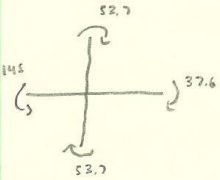


UBM = 145 - 37.6 = 107.4

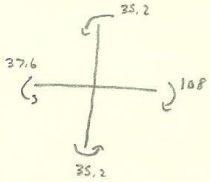
col 8



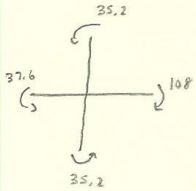
col 4



col 9

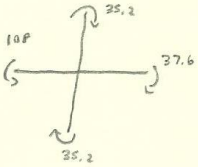


col 5

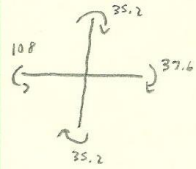


UBM = 108 - 37.6 = 70.4

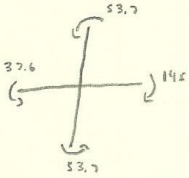
col 10



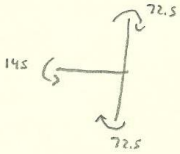
col 6



col 11



col 12



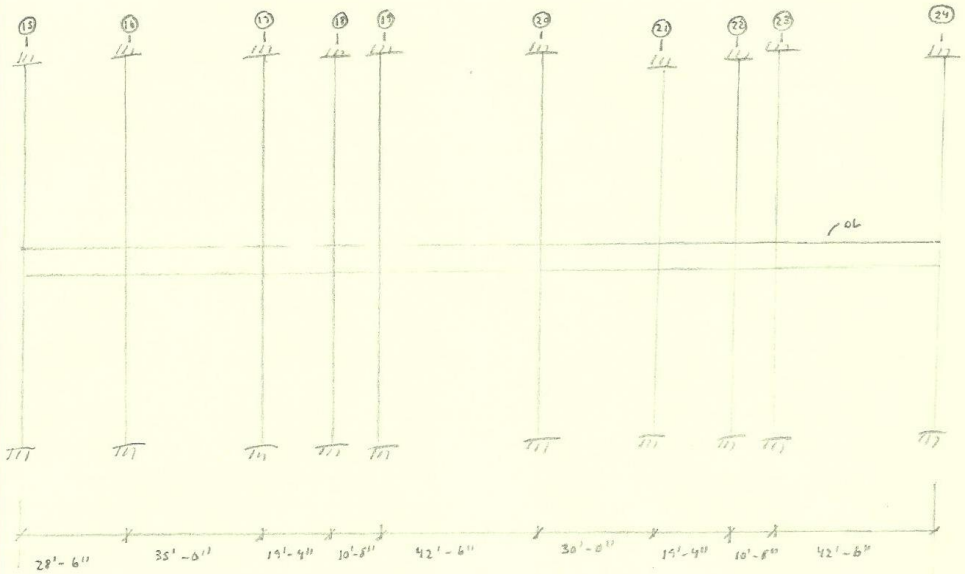
note: $k = 1.0$, $L = 15'-0''$, $d = 14''$ — W14's

$P_{eq} = P_u + \frac{24 M_u}{d}$ ← unbalanced col. mom.

axial load ↗ ↖ depth of col.

refer to excel sheets for P_{eq} and estimated col. size for each column

Estimate Column sizes | moment frame B | page 4 of 13



dead loads: slab - 75 psf
SDL - 10 psf
bm self-wt allow - 5 psf
girder self-wt allow - 5 psf

live loads: office space - 100 psf
mech. rm - 150 psf

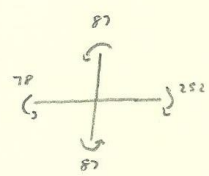
max VBM in Col 16

- load the 35'-0" span with DL and LL
- load 28'-6" span with DL only

28'-6" span: $w_D = 75\ psf\ (10.13\ ft) = 762\ plf$

$w_u = 1.2(762) = 1154\ plf$

$FEM = \frac{1.154\ (28.5)^2}{12} = 78\ kft$



35'-0" span: $w_D = 762\ plf$

influence area = $(10.25 + 10)\ ft\ (35\ ft) = 709\ ft^2 > 400\ ft^2$ ∴ reduce LL

VBM = 252 - 78 = 174

$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \frac{15}{\sqrt{709}} = 0.81 \end{cases}$

$w_L = 0.81\ (100\ psf)(10.13\ ft) = 821\ plf$

$w_u = 1.2(762) + 1.6(821) = 2468\ plf$

$FEM = \frac{2.468\ (35)^2}{12} = 252\ kft$

Estimate column size | moment frame B | page 5 of 13

max UBM in Col 17

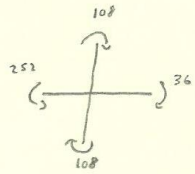
- load 35'-0" span with DL and LL
- load 19'-4" span with DL only

35'-0" span: FEM = 252 kft

19'-4" span: w_D = 95 psf (10.13 ft) = 962 plf

w_u = 1.2 (962) = 1154 plf

$FEM = \frac{1.154\,(19.33)^2}{12} = 36$ kft



UBM = 252 - 36 = 216

max UBM in col 18

- load 19'-4" span with DL and LL
- load 10'-8" span with DL only

19'-4" span: w_D = 962 plf

w_L = 150 psf (10.13 ft) = 1520 plf

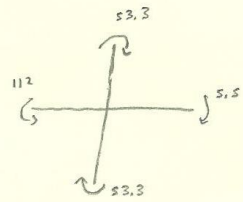
w_u = 1.2 (962) + 1.6 (1520) = 3586

$FEM = \frac{3.586\,(19.33)^2}{12} = 112$ kft

10'-8" span: w_D = 95 psf (5.13 ft) = 487 plf

w_u = 1.2 (487) = 584 plf

$FEM = \frac{0.584\,(10.67)^2}{12} = 5.5$ kft



UBM = 112 - 5.5 = 106.5

max UBM in Col 19

- load 42'-6" span with DL and LL
- load 10'-8" span with DL only

42'-6" span: w_D = 962 plf

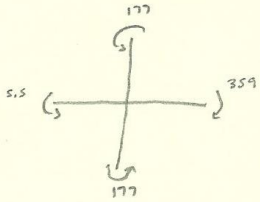
influence area: 20.25 ft (42.5 ft) = 861 ft² > 400 ft², LL reduction required

$$LL_{red} = \begin{cases} 0.50 \\ 0.25 + \frac{15}{\sqrt{861}} = 0.76 \end{cases} \max$$

w_L = 0.76 (100 psf)(10.13 ft) = 770 plf

w_u = 1.2 (962) + 1.6 (770) = 2386 plf

$FEM = \frac{2.386\,(42.5)^2}{12} = 359$ kft



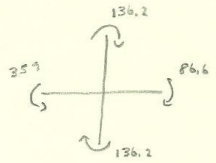
10'-8" span: FEM = 5.5 kft

Estimate Column Sizes | moment frame B | page 6 of 13

max UBM for Col 20

- load 42'-6" span with DL and LL
- load 30'-0" span with DL only

42'-6" span; FEM = 359 kft



30'-0" span: $w_D = 962$ plf

$w_u = 1.2(962) = 1154$ plf

$FEM = \frac{1.154(30)^2}{12} = 86.6$ kft

UBM = 359 - 86.6 = 272.4 kft

max UBM for Col 21

- load 30'-0" span with DL and LL
- load 19'-4" span with DL only

30'-8" span: $w_D = 962$ plf

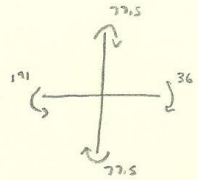
influence area = 20.25 ft (30 ft) = 608 ft² > 400 ft², LL reduction required

$LL_{red} = \max\left(0.50,\ 0.25 + \frac{15}{\sqrt{608}} = 0.86\right)$

$w_L = 0.86(100\text{ psf})(10.13\text{ ft}) = 871$ plf

$w_u = 1.2(962) + 1.6(871) = 2548$ plf

$FEM = \frac{2.548(30)^2}{12} = 191$ kft



UBM = 191 - 36 = 155 kft

19'-4" span: FEM = 36 kft

max UBM col 22

- load 19'-4" Span with DL and LL
- load 10'-8" span with DL only

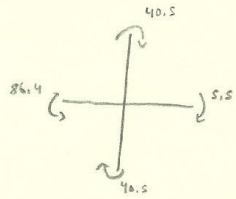
19'-4" span; $w_D = 962$ plf

influence area = 20.25 ft (19.33 ft) = 391 < 400 ft², LL reduction not applicable

$w_L = 100\text{ psf}(10.13\text{ ft}) = 1013$ plf

$w_u = 1.2(962) + 1.6(1013) = 2775$ plf

$FEM = \frac{2.775(19.33)^2}{12} = 86.4$ kft



10'-8" span: FEM = 5.5 kft

UBM = 86.4 - 5.5 = 81 kft

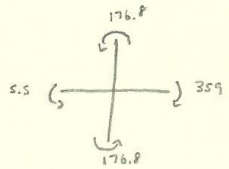
Estimate column sizes | moment frame B | page 7 of 13

max UBM on col 23

- load 42'-6" span with DL and LL
- load 10'-8" span with DL only

42'-6" span: FEM = 359 kft

10'-8" span: FEM = 5.5 kft

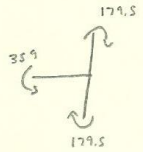


UBM = 359 - 5.5 = 353.5 kft

max UBM on col 24

- load 42'-6" span with DL and LL

42'-6" span: FEM = 359 kft



Estimate column sizes | moment frame C | page 8 of 13

max UBM in col 28 - refer to col 16

max UBM in col 29 - refer to col 17

max UBM in col 30 - refer to col 18

max UBM in col 31 - refer to col 19

max UBM in col 32 - refer to col 20

max UBM in col 33 - refer to col 21

max UBM in col 34 - refer to col 22

max UBM in col 35

- load 10'-8" span with DL and LL

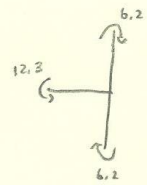
10'-8" span: $w_D = 95\,psf\,(4.75\,ft) = 451\,plf$

influence area = $9.5\,ft\,(10.67\,ft) = 101\,ft^2 < 400\,ft^2$, LL reduction not applicable

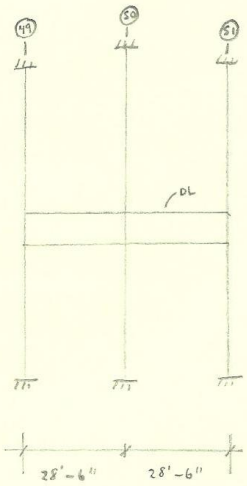
$w_L = 100\,psf\,(4.75\,ft) = 475\,plf$

$w_u = 1.2(451) + 1.6(475) = 1301\,plf$

$FEM = \frac{1.301\,(10.67)^2}{12} = 12.3\,kft$



Estimate column sizes | moment frame E | page 9 of 13



max UBM in col 49 and 51

- load 28'-6" span with DL and LL

28'-6" span: $w_D = 95\ psf\ (6.5\ ft) = 618\ plf$

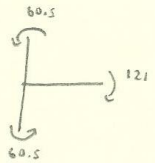
influence area = 11.5 ft (28.5 ft) = 328 ft² < 400 ft²

LL reduction not applicable

$w_L = 100\ psf\ (6.5\ ft) = 650\ plf$

$w_u = 1.2(618) + 1.6(650) = 1782\ plf$

$FEM = \frac{1.782\ (28.5)^2}{12} = 121\ kft$



max UBM in col 50

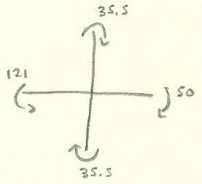
- load one of the 28'-6" spans with DL and LL
- load the other 28'-6" span with DL only

28'-6" span (DL only): $w_D = 618\ plf$

$w_u = 1.2(618) = 742\ plf$

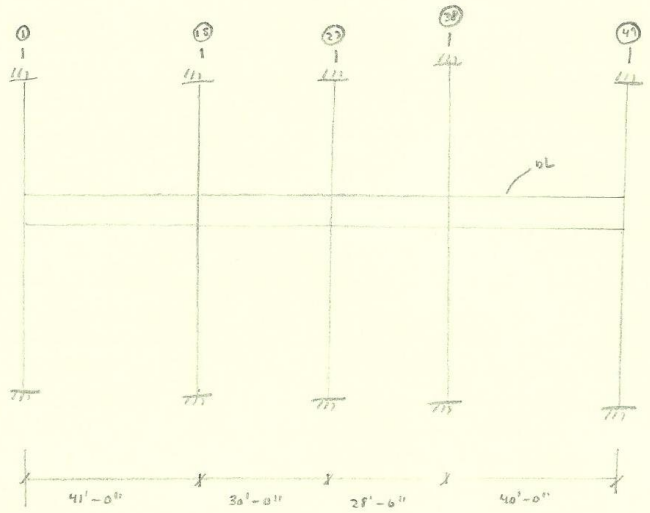
$FEM = \frac{0.742\ (28.5)^2}{12} = 50\ kft$

28'-6" span (DL and LL): FEM = 121 kft



UBM = 121 - 50 = 71 kft

Estimate column sizes | moment frame 1 | page 10 of 13



max UBM on Col 1

- load 41'-0" span with DL and LL

41'-0" span : $w_D = 95\,psf\,(15.75\,ft) = 1496\,plf$

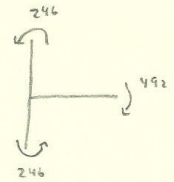
$I.A. = (28.5 + 1.5)\,ft\,(41\,ft) = 1230\,ft^2 > 400\,ft^2$, LL reduction required

$$LL_{req} = \max \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{1230}} = 0.68 \end{cases}$$

$w_L = 0.68\,(100\,psf)(15.75\,ft) = 1071\,plf$

$w_u = 1.2\,(1496) + 1.6\,(1071) = 3509\,plf$

$FEM = \dfrac{3.509\,(41)^2}{12} = 492\,kft$



max UBM in Col 15

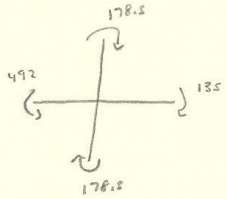
- load 41'-0" span with DL and LL
- load 30'-0" span with DL only

41'-0" span: FEM = 492 kft

30'-0" span: $w_D = 1496\,plf$

$w_u = 1.2\,(1496) = 1795\,plf$

$FEM = \dfrac{1.795\,(30)^2}{12} = 135\,kft$



UBM = 492 − 135 = 357 kft

Estimate column sizes | moment frame 1 | page 11 of 13

max UBM in col 27

- load 30'-0" span with DL and LL
- load 28'-6" span with DL only

30'-0" span: $w_D = 1496$ plf

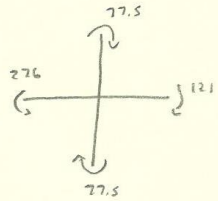
I.A. = 30ft (30ft) = 900 ft² > 400 ft², LL reduction required

$$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{900}} = 0.75 \end{cases}$$

$w_L = 0.75(100)(15.75) = 1181$ plf

$w_u = 1.2(1496) + 1.6(1181) = 3685$ plf

$FEM = \dfrac{3.685(30)^2}{12} = 276$ kft



28'-6" span: $w_D = 1496$ plf

$w_u = 1.2(1496) = 1795$ plf

$FEM = \dfrac{1.795(28.5)^2}{12} = 121$ kft

UBM = 276 − 121 = 155 kft

max UBM in col 38

- load 40'-0" span with DL and LL
- load 28'-6" span with DL only

40'-0" span: $w_D = 1496$ plf

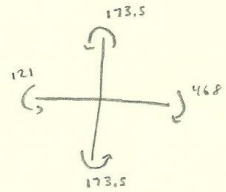
I.A. = 30ft (40ft) = 1200 ft² > 400 ft², LL reduct. req'd

$$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{1200}} = 0.68 \end{cases}$$

$w_L = 0.68(100)(15.75) = 1071$ plf

$w_u = 1.2(1496) + 1.6(1071) = 3509$

$FEM = \dfrac{3.509(40)^2}{12} = 468$ kft



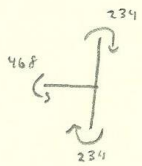
UBM = 468 − 121 = 347 kft

28'-6" span: FEM = 121 kft

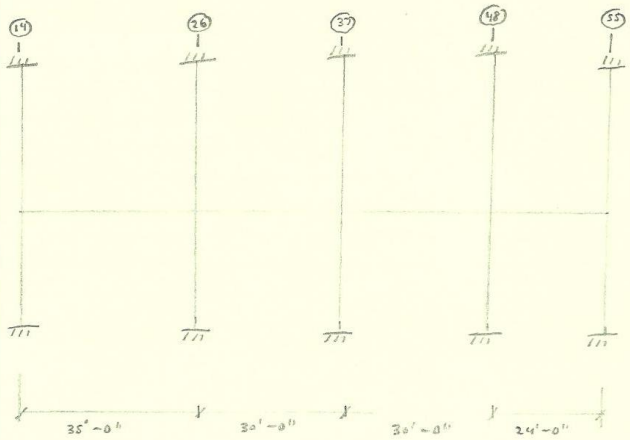
max UBM in col 49

- load 40'-0" span with DL and LL

span 40'-0": FEM = 468 kft



Estimate column size | moment frame 1' | page 12 of 13



max UBM in col 14

- load 35'-0" span with DL and LL

35'-0" span: $w_D = 95\ psf\ (17\ ft) = 1615\ plf$

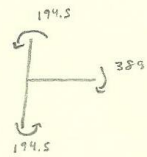
$I.A. = 32.5\ ft\ (35\ ft) = 1138\ ft^2 > 400\ ft^2$, LL reduction req'd

$$LL_{red} = \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{1138}} = 0.69 \end{cases} \quad max$$

$w_L = 0.69\ (100)(17) = 1173\ plf$

$w_u = 1.2\ (1615) + 1.6\ (1173) = 3815\ plf$

$FEM = \dfrac{3,815\ (35)^2}{12} = 389\ kft$



max UBM in col 26

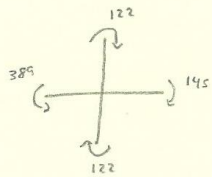
- load 35'-0" span with DL and LL
- load 30'-0" span with DL only

35'-0" span: FEM = 389 kft

30'-0" span: $w_D = 1615\ plf$

$w_u = 1.2\ (1615) = 1938\ plf$

$FEM = \dfrac{1,938\ (30)^2}{12} = 145\ kft$



UBM = 389 - 145 = 144 kft

Estimate column sizes | moment frame 1' | page 13 of 13

max UBM in col 37

- load one of the 30'-0" spans with DL and LL
- load the other 30'-0" span with DL only

30'-0" span (DL only): FEM = 145 kft

30'-0" span (DL and LL): w_D = 1615 plf

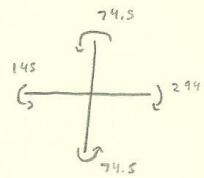
I.A. = 32.5 ft (30 ft) = 975 ft² > 400 ft², LL reduct. req'd

$$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{975}} = 0.73 \end{cases}$$

w_L = 0.73 (100)(17) = 1241 plf

w_u = 1.2 (1615) + 1.6 (1241) = 3924 plf

$$FEM = \frac{3.924\,(30)^2}{12} = 294 \text{ kft}$$



UBM = 294 − 145 = 149 kft

max UBM in col 48

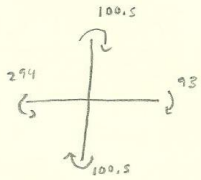
- load 30'-0" span with DL and LL
- load 24'-0" span with DL only

30'-0" span: FEM = 294

24'-0" span: w_D = 1615 plf

w_u = 1938 plf

$$FEM = \frac{1.938\,(24)^2}{12} = 93 \text{ kft}$$



UBM = 294 − 93 = 201 kft

max UBM in col 55

- load 24'-0" span with DL and LL

24'-0" span: w_D = 1615 plf

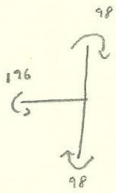
I.A. = 32.5 ft (24 ft) = 780 ft² > 400 ft², LL reduct. req'd

$$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \dfrac{15}{\sqrt{780}} = 0.79 \end{cases}$$

w_L = 0.79 (100)(17) = 1343

w_u = 1.2 (1615) + 1.6 (1343) = 4087 plf

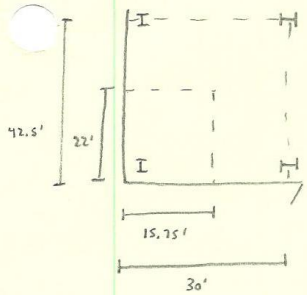
$$FEM = \frac{4.087\,(24)^2}{12} = 196 \text{ kft}$$



Estimate Column sizes

Moment Frame Column Design | column line A.1 | page 1 of 2

column 1



tributary area per floor or roof = 22 ft (15.75 ft) = 346.5 ft²

influence area per floor = 42.5 ft (30 ft) = 1275 ft²

• Load above level 12: roof + 1 floor

$$LL_{red} = \max \begin{cases} 0.50 \\ 0.25 + \frac{15}{\sqrt{1275}} = 0.67 \end{cases}$$

$$P_L = \overbrace{0.67\,(100\text{ psf})}^{\text{floor level}}(346.5\text{ ft}^2) + \overbrace{30\text{ psf}}^{\text{main roof}}(346.5\text{ ft}^2) = 33.6\text{ K}$$

$$P_S = 15.75\text{ psf}\,(346.5\text{ ft}^2) = 5.5\text{ K}$$

$$P_D = \underbrace{(75+10)\text{ psf}}_{\text{main roof}}(346.5\text{ ft}^2) + \underbrace{(75+10)\text{ psf}}_{\text{floor level}}(346.5\text{ ft}^2) = 58.9\text{ K}$$

$$P_u = 1.2(58.9\text{ K}) + 1.6(33.6\text{ K}) + 0.5(5.5\text{ K}) = 127.2\text{ K}$$

• Load above level 10: roof + 3 floors

$$LL_{red} = \max \begin{cases} 0.40 \\ 0.25 + \frac{15}{\sqrt{3 \times 1275}} = 0.49 \end{cases}$$

$P_L = (0.49(100\text{ psf})(3\text{ flrs}) + 30\text{ psf})\,346.5\text{ ft}^2 = 61.3\text{ K}$

$P_S = 15.75(346.5) = 5.5\text{ K}$

$P_D = (85\text{ psf}(3\text{ flrs}) + 85\text{ psf})\,346.5\text{ ft}^2 = 117.8\text{ K}$

$P_u = 1.2(117.8) + 1.6(61.3) + 0.5(5.5) = 242.2\text{ K}$

• Load above level 8: roof + 5 flrs

$$LL_{red} = \max \begin{cases} 0.40 \\ 0.25 + \frac{15}{\sqrt{5 \times 1275}} = 0.44 \end{cases}$$

$P_L = (0.44(100)(5\text{ flrs}) + 30)\,346.5 = 86.6\text{ K}$

$P_S = 5.5\text{ K}$

$P_D = (85(5\text{ flrs}) + 85)\,346.5 = 176.7\text{ K}$

$P_u = 1.2(176.7) + 1.6(86.6) + 0.5(5.5) = 333.4\text{ K}$

• Load above level 6: roof + 7 flrs

$$LL_{red} = \max \begin{cases} 0.40 \\ 0.25 + \frac{15}{\sqrt{7 \times 1275}} = 0.41 \end{cases}$$

$P_L = (0.41(100)(7) + 30)\,346.5 = 109.8\text{ K}$

$P_S = 5.5\text{ K}$

$P_D = (85(7) + 85)\,346.5 = 235.6\text{ K}$

$P_u = 1.2(235.6) + 1.6(109.8) + 0.5(5.5) = 461.2\text{ K}$

• Load above level 4: roof + 9 flrs

$P_L = (0.40(100)(9) + 30)\,346.5 = 135.1\text{ K}$

$P_S = 5.5\text{ K}$

$P_D = (85(9) + 85)\,346.5 = 294.5\text{ K}$

$P_u = 1.2(294.5) + 1.6(135.1) + 0.5(5.5) = 572.3\text{ K}$

• Load above level 2: roof + 11 flrs

$P_L = (0.4(100)(11) + 30)\,346.5 = 162.9\text{ K}$

$P_S = 5.5\text{ K}$

$P_D = (85(11) + 85)\,346.5 = 353.4\text{ K}$

$P_u = 1.2(353.4) + 1.6(162.9) + 0.5(5.5) = 687.5\text{ K}$

Estimate Column sizes

Moment Frame Column Design

page 2 of 2

- refer to excel spreadsheets for the estimated column sizes in moment frames A.1, B, C, E, 1, and 1'
based on gravity load analysis only

- refer to moment frame elevations for moment frame column sizes based on combined gravity and lateral loads analyses

Moment Frame A.1: Estimated column sizes based on gravity load analysis only

Column 5

Tributary Area

per floor or roof=	505 ft^2
Influence Area=	1881 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.60
P_L=	45.2 kips
P_D=	96.0 kips
P_{LV}=	10.1 kips
P_U=	192.6 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	253
Estimated Column Size:	W14x43 ΦP_n=292 k

Load above level 10: Roof + 3 Floors

LL_{red}	0.45
P_L=	83.3 kips
P_D=	191.9 kips
P_{LV}=	10.1 kips
P_U=	368.6 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	429
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8: Roof + 5 Floors

LL_{red}	0.40
P_L=	117.3 kips
P_D=	287.9 kips
P_{LV}=	10.1 kips
P_U=	538.2 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	599
Estimated Column Size:	W14x68 ΦP_n=608 k

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	156.6 kips
P_D=	383.8 kips
P_{LV}=	10.1 kips
P_U=	716.1 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	776
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	197.0 kips
P_D=	479.8 kips
P_{LV}=	10.1 kips
P_U=	895.9 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	956
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	237.4 kips
P_D=	575.7 kips
P_{LV}=	10.1 kips
P_U=	1075.7 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1136
Estimated Column Size:	W14x109 ΦP_n=1210 k

Column 6

Tributary Area

per floor or roof=	505 ft^2
Influence Area=	1881 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load, Lr	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.60
P_L=	80.6 kips
P_D=	96.0 kips
P_{LV}=	10.1 kips
P_U=	249.1 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	309
Estimated Column Size:	W14x48 ΦP_n=331 k

Load above level 10: Roof + 3 Floors

LL_{red}	0.45
P_L=	118.6 kips
P_D=	191.9 kips
P_{LV}=	10.1 kips
P_U=	425.1 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	485
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8: Roof + 5 Floors

LL_{red}	0.40
P_L=	152.7 kips
P_D=	287.9 kips
P_{LV}=	10.1 kips
P_U=	594.8 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	655
Estimated Column Size:	W14x74 ΦP_n=667 k

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	191.9 kips
P_D=	383.8 kips
P_{LV}=	10.1 kips
P_U=	772.7 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	833
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	232.3 kips
P_D=	479.8 kips
P_{LV}=	10.1 kips
P_U=	952.4 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1013
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	272.7 kips
P_D=	575.7 kips
P_{LV}=	10.1 kips
P_U=	1132.2 kips
Unbalanced Moment, M_U=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1193
Estimated Column Size:	W14x109 ΦP_n=1210 k

Column 7

Tributary Area

per floor or roof=	623.4 ft^2
Influence Area=	2221 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.57
P_L=	97.8 kips
P_D=	118.4 kips
P_{LV}=	12.5 kips
P_U=	304.8 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	397
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10: Roof + 3 Floors

LL_{red}	0.43
P_L=	143.5 kips
P_D=	236.9 kips
P_{LV}=	12.5 kips
P_U=	520.0 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	612
Estimated Column Size:	W14x68 ΦP_n=667 k

Load above level 8: Roof + 5 Floors

LL_{red}	0.40
P_L=	187.0 kips
P_D=	355.3 kips
P_{LV}=	12.5 kips
P_U=	731.9 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	824
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	236.9 kips
P_D=	473.8 kips
P_{LV}=	12.5 kips
P_U=	953.8 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1046
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	286.8 kips
P_D=	592.2 kips
P_{LV}=	12.5 kips
P_U=	1175.7 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1268
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	336.6 kips
P_D=	710.7 kips
P_{LV}=	12.5 kips
P_U=	1397.7 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1490
Estimated Column Size:	W14x145 ΦP_n=1650 k

Column 8

Tributary Area

per floor or roof=	623.4 ft^2
Influence Area=	2221 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.57
P_L=	97.8 kips
P_D=	118.4 kips
P_{Lr}=	12.5 kips
P_U=	304.8 kips
Unbalanced Moment, M_u=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	397
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10: Roof + 3 Floors

LL_{red}	0.43
P_L=	143.5 kips
P_D=	236.9 kips
P_{Lr}=	12.5 kips
P_U=	520.0 kips
Unbalanced Moment, M_u=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	612
Estimated Column Size:	W14x68 ΦP_n=667 k

Load above level 8: Roof + 5 Floors

LL_{red}	0.40
P_L=	187.0 kips
P_D=	355.3 kips
P_{Lr}=	12.5 kips
P_U=	731.9 kips
Unbalanced Moment, M_u=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	824
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	236.9 kips
P_D=	473.8 kips
P_{Lr}=	12.5 kips
P_U=	953.8 kips
Unbalanced Moment, M_u=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1046
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	286.8 kips
P_D=	592.2 kips
P_{Lr}=	12.5 kips
P_U=	1175.7 kips
Unbalanced Moment, M_u=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1268
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	336.6 kips
P_D=	710.7 kips
P_{Lr}=	12.5 kips
P_U=	1397.7 kips
Unbalanced Moment, M_u=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1490
Estimated Column Size:	W14x145 ΦP_n=1650 k

Column 9

Tributary Area

per floor or roof=	505 ft^2
Influence Area=	1881 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.60
P_L=	80.6 kips
P_D=	96.0 kips
P_{Lr}=	10.1 kips
P_U=	249.1 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	309
Estimated Column Size:	W14x48 ΦP_n=331 k

Load above level 10: Roof + 3 Floors

LL_{red}	0.45
P_L=	118.6 kips
P_D=	191.9 kips
P_{Lr}=	10.1 kips
P_U=	425.1 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	485
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8: Roof + 5 Floors

LL_{red}	0.40
P_L=	152.7 kips
P_D=	287.9 kips
P_{Lr}=	10.1 kips
P_U=	594.8 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	655
Estimated Column Size:	W14x74 ΦP_n=667 k

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	191.9 kips
P_D=	383.8 kips
P_{Lr}=	10.1 kips
P_U=	772.7 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	833
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	232.3 kips
P_D=	479.8 kips
P_{Lr}=	10.1 kips
P_U=	952.4 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1013
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	272.7 kips
P_D=	575.7 kips
P_{Lr}=	10.1 kips
P_U=	1132.2 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1193
Estimated Column Size:	W14x109 ΦP_n=1210 k

Column 10

Tributary Area

per floor or roof=	505 ft^2
Influence Area=	1881 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.60
P_L=	80.6 kips
P_D=	96.0 kips
P_{Lr}=	10.1 kips
P_U=	249.1 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	309
Estimated Column Size:	W14x48 ΦP_n=331 k

Load above level 10: Roof + 3 Floors

LL_{red}	0.45
P_L=	118.6 kips
P_D=	191.9 kips
P_{Lr}=	10.1 kips
P_U=	425.1 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	485
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8: Roof + 5 Floors

LL_{red}	0.40
P_L=	152.7 kips
P_D=	287.9 kips
P_{Lr}=	10.1 kips
P_U=	594.8 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	655
Estimated Column Size:	W14x74 ΦP_n=667 k

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	191.9 kips
P_D=	383.8 kips
P_{Lr}=	10.1 kips
P_U=	772.7 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	833
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	232.3 kips
P_D=	479.8 kips
P_{Lr}=	10.1 kips
P_U=	952.4 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1013
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	272.7 kips
P_D=	575.7 kips
P_{Lr}=	10.1 kips
P_U=	1132.2 kips
Unbalanced Moment, M_u=	35.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1193
Estimated Column Size:	W14x109 ΦP_n=1210 k

Column 11

Tributary Area	
per floor or roof=	623.4 ft^2
Influence Area=	2221 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.57
P_L=	97.8 kips
P_D=	118.4 kips
P_{LV}=	12.5 kips
P_U=	304.8 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	397
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.43
P_L=	143.5 kips
P_D=	236.9 kips
P_{LV}=	12.5 kips
P_U=	520.0 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	612
Estimated Column Size:	W14x74 ΦP_n=667 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.39
P_L=	184.6 kips
P_D=	355.3 kips
P_{LV}=	12.5 kips
P_U=	728.1 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	820
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	236.9 kips
P_D=	473.8 kips
P_{LV}=	12.5 kips
P_U=	953.8 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1046
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	286.8 kips
P_D=	592.2 kips
P_{LV}=	12.5 kips
P_U=	1175.7 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1268
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	336.6 kips
P_D=	710.7 kips
P_{LV}=	12.5 kips
P_U=	1397.7 kips
Unbalanced Moment, M_U=	53.7 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1490
Estimated Column Size:	W14x145 ΦP_n=1650 k

Column 12

Tributary Area	
per floor or roof=	663 ft^2
Influence Area=	2361 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.56
P_L=	103.3 kips
P_D=	126.0 kips
P_{LV}=	13.3 kips
P_U=	323.1 kips
Unbalanced Moment, M_U=	72.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	447
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.43
P_L=	151.5 kips
P_D=	251.9 kips
P_{LV}=	13.3 kips
P_U=	551.3 kips
Unbalanced Moment, M_U=	72.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	676
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	198.9 kips
P_D=	377.9 kips
P_{LV}=	13.3 kips
P_U=	778.4 kips
Unbalanced Moment, M_U=	72.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	903
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	251.9 kips
P_D=	503.9 kips
P_{LV}=	13.3 kips
P_U=	1014.4 kips
Unbalanced Moment, M_U=	72.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1139
Estimated Column Size:	W14x109 ΦP_n=1210 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	305.0 kips
P_D=	629.9 kips
P_{LV}=	13.3 kips
P_U=	1250.4 kips
Unbalanced Moment, M_U=	72.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1375
Estimated Column Size:	W14x132 ΦP_n=1480 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	358.0 kips
P_D=	755.8 kips
P_{LV}=	13.3 kips
P_U=	1486.4 kips
Unbalanced Moment, M_U=	72.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1611
Estimated Column Size:	W14x145 ΦP_n=1650 k

Note: Refer to Moment Frame A.1 elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame B: Estimated column sizes based on gravity load analysis only

Column 15

Tributary Area	
per floor or roof=	559 ft^2
Influence Area=	2130 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.58
P_L=	48.9 kips
P_D=	106.2 kips
P_U=	11.2 kips
P_U=	211.3 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	517
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.44
P_L=	90.2 kips
P_D=	212.4 kips
P_U=	11.2 kips
P_U=	404.8 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	711
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	128.6 kips
P_D=	318.6 kips
P_U=	11.2 kips
P_U=	593.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	900
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	173.3 kips
P_D=	424.8 kips
P_U=	11.2 kips
P_U=	792.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1099
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	218.0 kips
P_D=	531.1 kips
P_U=	11.2 kips
P_U=	991.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1298
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	262.7 kips
P_D=	637.3 kips
P_U=	11.2 kips
P_U=	1190.7 kips
Unbalanced Moment, M_u=	178.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1497
Estimated Column Size:	W14x145 ΦP_n=1650 k

Column 16

Tributary Area	
per floor or roof=	1127 ft^2
Influence Area=	4508.5 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Cooling Towers)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.50
P_L=	225.4 kips
P_D=	214.1 kips
P_U=	22.5 kips
P_U=	628.9 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	778
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.40
P_L=	304.3 kips
P_D=	428.3 kips
P_U=	22.5 kips
P_U=	1012.0 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1161
Estimated Column Size:	W14x109 ΦP_n=1210 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	394.5 kips
P_D=	642.4 kips
P_U=	22.5 kips
P_U=	1413.3 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1562
Estimated Column Size:	W14x145 ΦP_n=1650 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	484.6 kips
P_D=	856.5 kips
P_U=	22.5 kips
P_U=	1814.5 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1964
Estimated Column Size:	W14x176 ΦP_n=2010 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	574.8 kips
P_D=	1070.7 kips
P_U=	22.5 kips
P_U=	2215.7 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2365 kips
Estimated Column Size:	W14x211 ΦP_n=2420 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	664.9 kips
P_D=	1284.8 kips
P_U=	22.5 kips
P_U=	2616.9 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2769 kips
Estimated Column Size:	W14x257 ΦP_n=2960 k

Column 17

Tributary Area	
per floor or roof=	964.5 ft^2
Influence Area=	3857.4 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Cooling Towers)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	1.00
P_L=	289.4 kips
P_D=	183.3 kips
P_U=	19.3 kips
P_U=	692.5 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	878 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.80
P_L=	491.9 kips
P_D=	366.5 kips
P_U=	19.3 kips
P_U=	1236.5 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1422 kips
Estimated Column Size:	W14x132 ΦP_n=1480 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.80
P_L=	723.4 kips
P_D=	549.8 kips
P_U=	19.3 kips
P_U=	1826.8 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2012 kips
Estimated Column Size:	W14x193 ΦP_n=2210 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.80
P_L=	954.9 kips
P_D=	733.0 kips
P_U=	19.3 kips
P_U=	2417.0 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2602 kips
Estimated Column Size:	W14x233 ΦP_n=2680 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.80
P_L=	1186.3 kips
P_D=	916.3 kips
P_U=	19.3 kips
P_U=	3007.3 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	3192 kips
Estimated Column Size:	W14x283 ΦP_n=3270 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.80
P_L=	1417.8 kips
P_D=	1099.5 kips
P_U=	19.3 kips
P_U=	3597.6 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	3783 kips
Estimated Column Size:	W14x342 ΦP_n=3980 k

Column 18

Tributary Area	
per floor or roof=	452.5 ft^2
Influence Area=	1810 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	1.00
P_L=	135.8 kips
P_D=	86.0 kips
P_U=	9.1 kips
P_U=	359.6 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	451 kips
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.80
P_L=	230.8 kips
P_D=	172.0 kips
P_U=	9.1 kips
P_U=	614.8 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	706 kips
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.80
P_L=	339.4 kips
P_D=	257.9 kips
P_U=	9.1 kips
P_U=	891.7 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	983 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.80
P_L=	448.0 kips
P_D=	343.9 kips
P_U=	9.1 kips
P_U=	1168.7 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1260 kips
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.80
P_L=	556.6 kips
P_D=	429.9 kips
P_U=	9.1 kips
P_U=	1445.6 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1537 kips
Estimated Column Size:	W14x145 ΦP_n=1650 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.80
P_L=	665.2 kips
P_D=	515.9 kips
P_U=	9.1 kips
P_U=	1722.5 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1908 kips
Estimated Column Size:	W14x176 ΦP_n=2010 k

Tributary area per PH roof=	209 ft^2
Influence area on PH roof=	835 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof:	PH roof+ 0 Floors
P_L=	6.3 kips
P_D=	18.8 kips
P_U=	4.2 kips
P_U=	34.7 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	35 kips
Estimated Column Size:	W14x43 ΦP_n=196 k

Column 19

Tributary Area

per floor or roof=	864 ft^2
Influence Area=	3455 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.51
P_L=	173.2 kips
P_D=	164.2 kips
P_{LL}=	17.3 kips
P_U=	554.5 kips
Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	858 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.40
P_L=	233.3 kips
P_D=	328.3 kips
P_{LL}=	17.3 kips
P_U=	847.8 kips
Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1151 kips
Estimated Column Size:	W14x109 ΦP_n=1210 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	302.4 kips
P_D=	492.5 kips
P_{LL}=	17.3 kips
P_U=	1155.2 kips
Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1459 kips
Estimated Column Size:	W14x132 ΦP_n=1480 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	371.5 kips
P_D=	656.6 kips
P_{LL}=	17.3 kips
P_U=	1462.8 kips
Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1766 kips
Estimated Column Size:	W14x159 ΦP_n=1810 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	440.6 kips
P_D=	820.8 kips
P_{LL}=	17.3 kips
P_U=	1770.3 kips
Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2074 kips
Estimated Column Size:	W14x193 ΦP_n=2210 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	509.8 kips
P_D=	985.0 kips
P_{LL}=	17.3 kips
P_U=	2077.9 kips
Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2381 kips
Estimated Column Size:	W14x211 ΦP_n=2420 k

Tributary area per PH roof=	432 ft^2
Influence area on PH roof=	1727 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof:	PH roof+ 0 Floors
P_L=	13.0 kips
P_D=	38.9 kips
P_{LL}=	8.6 kips
P_U=	71.7 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	72 kips
Estimated Column Size:	W14x43 ΦP_n=196 k

Column 20

Tributary Area

per floor or roof=	1062 ft^2
Influence Area=	4168 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.50
P_L=	212.4 kips
P_D=	201.8 kips
P_{LL}=	21.2 kips
P_U=	688.5 kips
Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	922 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.40
P_L=	286.7 kips
P_D=	403.6 kips
P_{LL}=	21.2 kips
P_U=	1049.6 kips
Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1283 kips
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	339.8 kips
P_D=	605.3 kips
P_{LL}=	21.2 kips
P_U=	1376.7 kips
Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1610 kips
Estimated Column Size:	W14x145 ΦP_n=1650 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	456.7 kips
P_D=	807.1 kips
P_{LL}=	21.2 kips
P_U=	1805.8 kips
Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2039
Estimated Column Size:	W14x193 ΦP_n=2210 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	541.6 kips
P_D=	1008.9 kips
P_{LL}=	21.2 kips
P_U=	2183.8 kips
Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2417 kips
Estimated Column Size:	W14x211 ΦP_n=2420 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	626.6 kips
P_D=	1210.7 kips
P_{LL}=	21.2 kips
P_U=	2561.9 kips
Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2795 kips
Estimated Column Size:	W14x257 ΦP_n=2960 k

Tributary area per PH roof=	578 ft^2
Influence area on PH roof=	2231 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof:	PH roof+ 0 Floors
P_L=	17.3 kips
P_D=	52.0 kips
P_{LL}=	11.6 kips
P_U=	95.9 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	96
Estimated Column Size:	W14x43 ΦP_n=196 k

Column 21

Tributary Area

per floor or roof=	1027 ft^2
Influence Area=	3022 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.52
P_L=	207.7 kips
P_D=	195.1 kips
P_{LL}=	20.5 kips
P_U=	635.8 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	769
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.41
P_L=	279.6 kips
P_D=	390.3 kips
P_{LL}=	20.5 kips
P_U=	984.9 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1118
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	359.5 kips
P_D=	585.4 kips
P_{LL}=	20.5 kips
P_U=	1346.8 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1480
Estimated Column Size:	W14x132 ΦP_n=1480 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	441.6 kips
P_D=	780.5 kips
P_{LL}=	20.5 kips
P_U=	1712.4 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1845
Estimated Column Size:	W14x159 ΦP_n=1810 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	523.8 kips
P_D=	975.7 kips
P_{LL}=	20.5 kips
P_U=	2078.0 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2211
Estimated Column Size:	W14x193 ΦP_n=2210 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	605.9 kips
P_D=	1170.8 kips
P_{LL}=	20.5 kips
P_U=	2404.7 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2538
Estimated Column Size:	W14x233 ΦP_n=2680 k

Tributary area per PH roof=	355 ft^2
Influence area on PH roof=	1419 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof:	PH roof+ 0 Floors
P_L=	10.7 kips
P_D=	32.0 kips
P_{LL}=	7.1 kips
P_U=	58.9 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	59
Estimated Column Size:	W14x43 ΦP_n=196 k

Column 22

Tributary Area

per floor or roof=	452.5 ft^2
Influence Area=	1810 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.60
P_L=	95.1 kips
P_D=	86.0 kips
P_{LL}=	9.1 kips
P_U=	294.6 kips
Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	364 kips
Estimated Column Size:	W14x53 ΦP_n=369 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.45
P_L=	129.4 kips
P_D=	172.0 kips
P_{LL}=	9.1 kips
P_U=	452.7 kips
Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	522
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.41
P_L=	160.1 kips
P_D=	257.9 kips
P_{LL}=	9.1 kips
P_U=	604.9 kips
Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	674
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	194.6 kips
P_D=	343.9 kips
P_{LL}=	9.1 kips
P_U=	763.2 kips
Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	833
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	230.8 kips
P_D=	429.9 kips
P_{LL}=	9.1 kips
P_U=	924.3 kips
Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	994
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	267.0 kips
P_D=	515.9 kips
P_{LL}=	9.1 kips
P_U=	1085.4 kips
Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1155
Estimated Column Size:	W14x109 ΦP_n=1210 k

Tributary area per PH roof=	209 ft^2
Influence area on PH roof=	707 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof:	PH roof+ 0 Floors
P_L=	6.3 kips
P_D=	18.8 kips
P_{LL}=	4.2 kips
P_U=	34.7 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	35
Estimated Column Size:	W14x43 ΦP_n=196 k

Column 23

Tributary Area	
per floor or roof=	864 ft^2
Influence Area=	3544 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	1.0
P_L=	259.2 kips
P_D=	164.2 kips
P_{LR}=	17.3 kips
P_U=	624.2 kips
Unbalanced Moment, M_{ub}=	176.8 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	927 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.80
P_L=	440.6 kips
P_D=	328.3 kips
P_{LR}=	17.3 kips
P_U=	1111.5 kips
Unbalanced Moment, M_{ub}=	176.8 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1415 kips
Estimated Column Size:	W14x132 ΦP_n=1480 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.80
P_L=	648.0 kips
P_D=	492.5 kips
P_{LR}=	17.3 kips
P_U=	1640.2 kips
Unbalanced Moment, M_{ub}=	176.8 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1943 kips
Estimated Column Size:	W14x176 ΦP_n=2010 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.80
P_L=	855.4 kips
P_D=	656.6 kips
P_{LR}=	17.3 kips
P_U=	2169.0 kips
Unbalanced Moment, M_{ub}=	176.8 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2472 kips
Estimated Column Size:	W14x233 ΦP_n=2680 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.80
P_L=	1062.7 kips
P_D=	820.8 kips
P_{LR}=	17.3 kips
P_U=	2697.8 kips
Unbalanced Moment, M_{ub}=	176.8 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	3001 kips
Estimated Column Size:	W14x283 ΦP_n=3270 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.80
P_L=	1270.1 kips
P_D=	985.0 kips
P_{LR}=	17.3 kips
P_U=	3226.5 kips
Unbalanced Moment, M_{ub}=	176.8 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	3530 kips
Estimated Column Size:	W14x311 ΦP_n=3600 k

Column 24

Tributary Area	
per floor or roof=	1083 ft^2
Influence Area=	4331 ft^2
Floor Dead Load= (slab+SDL)	95 psf
Roof Dead Load= (slab+SDL)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.50
P_L=	162.5 kips
P_D=	205.8 kips
P_{LR}=	21.7 kips
P_U=	517.7 kips
Unbalanced Moment, M_{ub}=	179.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	825
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.40
P_L=	238.3 kips
P_D=	411.5 kips
P_{LR}=	21.7 kips
P_U=	885.9 kips
Unbalanced Moment, M_{ub}=	179.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1194
Estimated Column Size:	W14x109 ΦP_n=1210 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	324.9 kips
P_D=	617.3 kips
P_{LR}=	21.7 kips
P_U=	1271.4 kips
Unbalanced Moment, M_{ub}=	179.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1579
Estimated Column Size:	W14x145 ΦP_n=1650 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	411.5 kips
P_D=	823.1 kips
P_{LR}=	21.7 kips
P_U=	1657.0 kips
Unbalanced Moment, M_{ub}=	179.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1965
Estimated Column Size:	W14x176 ΦP_n=2010 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	498.2 kips
P_D=	1028.9 kips
P_{LR}=	21.7 kips
P_U=	2042.5 kips
Unbalanced Moment, M_{ub}=	179.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2350
Estimated Column Size:	W14x211 ΦP_n=2420 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	584.8 kips
P_D=	1234.6 kips
P_{LR}=	21.7 kips
P_U=	2428.1 kips
Unbalanced Moment, M_{ub}=	179.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2736
Estimated Column Size:	W14x233 ΦP_n=2680 k

Tributary area per PH roof=	23 ft^2
Influence area on PH roof=	91 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof:	PH roof+ 0 Floors
P_L=	0.7 kips
P_D=	2.1 kips
P_{LR}=	0.5 kips
P_U=	3.8 kips
Unbalanced Moment, M_{ub}=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	4
Estimated Column Size:	W14x43 ΦP_n=196 k

Note: Refer to Moment Frame B elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame C: Estimated column sizes based on gravity load analysis only

Column 27

Tributary Area	
per floor or roof=	439 ft^2
Influence Area=	1755 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.61
P_L=	39.9 kips
P_R=	83.4 kips
P_M=	8.8 kips
P_U=	168.3 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	301
Estimated Column Size:	W14x48 ΦP_n=331 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.46
P_L=	73.3 kips
P_R=	166.8 kips
P_M=	8.8 kips
P_U=	321.9 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	455
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.41
P_L=	103.2 kips
P_R=	250.2 kips
P_M=	8.8 kips
P_U=	469.8 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	609
Estimated Column Size:	W14x68 ΦP_n=608 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	136.1 kips
P_R=	333.6 kips
P_M=	8.8 kips
P_U=	621.5 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	755
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	171.2 kips
P_R=	417.1 kips
P_M=	8.8 kips
P_U=	778.8 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	912
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	206.3 kips
P_R=	500.5 kips
P_M=	8.8 kips
P_U=	935.1 kips
Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1068
Estimated Column Size:	W14x99 ΦP_n=1100 k

Column 28

Tributary Area	
per floor or roof=	929 ft^2
Influence Area=	3715 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Cooling Towers)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.50
P_L=	185.8 kips
P_R=	176.5 kips
P_M=	18.6 kips
P_U=	518.4 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	668 kips
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.40
P_L=	250.8 kips
P_R=	353.0 kips
P_M=	18.6 kips
P_U=	834.2 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	983 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	325.2 kips
P_R=	529.5 kips
P_M=	18.6 kips
P_U=	1163.0 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1314 kips
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	399.5 kips
P_R=	706.0 kips
P_M=	18.6 kips
P_U=	1495.7 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1645 kips
Estimated Column Size:	W14x145 ΦP_n=1650 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	473.8 kips
P_R=	882.6 kips
P_M=	18.6 kips
P_U=	1826.4 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1976 kips
Estimated Column Size:	W14x176 ΦP_n=2010 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	548.1 kips
P_R=	1059.1 kips
P_M=	18.6 kips
P_U=	2157.1 kips
Unbalanced Moment, M_u=	87.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2306 kips
Estimated Column Size:	W14x211 ΦP_n=2420 k

Column 29

Tributary Area	
per floor or roof=	795 ft^2
Influence Area=	3178 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	1.00
P_L=	238.5 kips
P_R=	151.1 kips
P_M=	15.9 kips
P_U=	370.8 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	756 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.80
P_L=	405.5 kips
P_R=	302.1 kips
P_M=	15.9 kips
P_U=	1019.2 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1204 kips
Estimated Column Size:	W14x109 ΦP_n=1210 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.80
P_L=	596.3 kips
P_R=	453.2 kips
P_M=	15.9 kips
P_U=	1505.7 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1691 kips
Estimated Column Size:	W14x159 ΦP_n=1810 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.80
P_L=	787.1 kips
P_R=	604.2 kips
P_M=	15.9 kips
P_U=	1992.3 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2177 kips
Estimated Column Size:	W14x193 ΦP_n=2210 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.80
P_L=	977.9 kips
P_R=	755.3 kips
P_M=	15.9 kips
P_U=	2478.8 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2664 kips
Estimated Column Size:	W14x257 ΦP_n=2960 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.80
P_L=	1168.7 kips
P_R=	906.3 kips
P_M=	15.9 kips
P_U=	2965.4 kips
Unbalanced Moment, M_u=	108.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	3150 kips
Estimated Column Size:	W14x283 ΦP_n=3270 k

Column 30

Tributary Area	
per floor or roof=	359 ft^2
Influence Area=	1435 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	1.00
P_L=	107.7 kips
P_R=	68.2 kips
P_M=	7.2 kips
P_U=	292.5 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	384 kips
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.80
P_L=	183.1 kips
P_R=	136.4 kips
P_M=	7.2 kips
P_U=	494.9 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	586 kips
Estimated Column Size:	W14x68 ΦP_n=608 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.80
P_L=	269.3 kips
P_R=	204.6 kips
P_M=	7.2 kips
P_U=	714.6 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	806 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.80
P_L=	355.4 kips
P_R=	272.8 kips
P_M=	7.2 kips
P_U=	934.3 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1026 kips
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.80
P_L=	441.6 kips
P_R=	341.1 kips
P_M=	7.2 kips
P_U=	1154.1 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1245 kips
Estimated Column Size:	W14x120 ΦP_n=1340 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.80
P_L=	527.7 kips
P_R=	409.3 kips
P_M=	7.2 kips
P_U=	1373.8 kips
Unbalanced Moment, M_u=	53.3 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1465 kips
Estimated Column Size:	W14x132 ΦP_n=1480 k

Tributary area per PH roof=	209 ft^2
Influence area on PH roof=	835 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof:	PH roof+ 0 Floors
P_L=	6.3 kips
P_R=	18.8 kips
P_M=	4.2 kips
P_U=	34.7 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	35 kips
Estimated Column Size:	W14x43 ΦP_n=196 k

Column 31

Tributary Area

per floor or roof=	441 ft^2
Influence Area=	1767 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.61
P_L=	93.1 kips
P_D=	84.0 kips
P_U=	8.8 kips
P_u=	336.9 kips

Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	640 kips
Estimated Column Size:	**W14x74 ΦP_n=667 k**

Load above level 10: Roof + 3 Floors

LL_{red}	0.46
P_L=	126.8 kips
P_D=	168.0 kips
P_U=	8.8 kips
P_u=	491.5 kips

Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	795 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 8: Roof + 5 Floors

LL_{red}	0.41
P_L=	156.8 kips
P_D=	251.9 kips
P_U=	8.8 kips
P_u=	640.3 kips

Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	944 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	190.1 kips
P_D=	335.9 kips
P_U=	8.8 kips
P_u=	794.3 kips

Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1098 kips
Estimated Column Size:	**W14x99 ΦP_n=1100 k**

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	225.4 kips
P_D=	419.9 kips
P_U=	8.8 kips
P_u=	951.6 kips

Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1255 kips
Estimated Column Size:	**W14x120 ΦP_n=1340 k**

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	260.8 kips
P_D=	503.9 kips
P_U=	8.8 kips
P_u=	1109.0 kips

Unbalanced Moment, M_u=	177.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1412 kips
Estimated Column Size:	**W14x132 ΦP_n=1480 k**

Tributary area per PH roof=	498 ft^2
Influence area on PH roof=	1913 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof: PH roof+ 0 Floors

P_L=	14.9 kips
P_D=	44.8 kips
P_U=	10.0 kips
P_u=	82.7 kips

Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	83 kips
Estimated Column Size:	**W14x43 ΦP_n=196 k**

Column 32

Tributary Area

per floor or roof=	779 ft^2
Influence Area=	3355 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.51
P_L=	156.5 kips
P_D=	148.0 kips
P_U=	15.6 kips
P_u=	533.4 kips

Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	767 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 10: Roof + 3 Floors

LL_{red}	0.40
P_L=	210.2 kips
P_D=	296.0 kips
P_U=	15.6 kips
P_u=	797.0 kips

Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1030 kips
Estimated Column Size:	**W14x99 ΦP_n=1100 k**

Load above level 8: Roof + 5 Floors

LL_{red}	0.37
P_L=	259.3 kips
P_D=	444.0 kips
P_U=	15.6 kips
P_u=	1053.2 kips

Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1287 kips
Estimated Column Size:	**W14x120 ΦP_n=1340 k**

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	335.0 kips
P_D=	592.0 kips
P_U=	15.6 kips
P_u=	1351.8 kips

Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1585 kips
Estimated Column Size:	**W14x145 ΦP_n=1650 k**

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	397.3 kips
P_D=	740.1 kips
P_U=	15.6 kips
P_u=	1629.1 kips

Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1863 kips
Estimated Column Size:	**W14x176 ΦP_n=2010 k**

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	459.6 kips
P_D=	888.1 kips
P_U=	15.6 kips
P_u=	1906.4 kips

Unbalanced Moment, M_u=	136.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2140 kips
Estimated Column Size:	**W14x193 ΦP_n=2210 k**

Tributary area per PH roof=	588 ft^2
Influence area on PH roof=	2594 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof: PH roof+ 0 Floors

P_L=	17.6 kips
P_D=	52.9 kips
P_U=	11.8 kips
P_u=	97.6 kips

Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	98 kips
Estimated Column Size:	**W14x43 ΦP_n=196 k**

Column 33

Tributary Area

per floor or roof=	484 ft^2
Influence Area=	2177 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.57
P_L=	100.3 kips
P_D=	92.0 kips
P_U=	9.7 kips
P_u=	332.5 kips

Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	465 kips
Estimated Column Size:	**W14x61 ΦP_n=543 k**

Load above level 10: Roof + 3 Floors

LL_{red}	0.44
P_L=	135.9 kips
P_D=	183.9 kips
P_U=	9.7 kips
P_u=	499.8 kips

Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	633 kips
Estimated Column Size:	**W14x74 ΦP_n=667 k**

Load above level 8: Roof + 5 Floors

LL_{red}	0.40
P_L=	169.4 kips
P_D=	275.9 kips
P_U=	9.7 kips
P_u=	663.9 kips

Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	797 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	208.1 kips
P_D=	367.8 kips
P_U=	9.7 kips
P_u=	836.2 kips

Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	969 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	246.8 kips
P_D=	459.8 kips
P_U=	9.7 kips
P_u=	1008.5 kips

Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1141 kips
Estimated Column Size:	**W14x109 ΦP_n=1210 k**

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	285.6 kips
P_D=	551.8 kips
P_U=	9.7 kips
P_u=	1180.8 kips

Unbalanced Moment, M_u=	77.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1314 kips
Estimated Column Size:	**W14x120 ΦP_n=1340 k**

Tributary area per PH roof=	343 ft^2
Influence area on PH roof=	1370 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof: PH roof+ 0 Floors

P_L=	10.3 kips
P_D=	30.9 kips
P_U=	6.9 kips
P_u=	56.9 kips

Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	57 kips
Estimated Column Size:	**W14x43 ΦP_n=196 k**

Column 34

Tributary Area

per floor or roof=	359 ft^2
Influence Area=	1435 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	0.65
P_L=	77.0 kips
P_D=	68.2 kips
P_U=	7.2 kips
P_u=	242.2 kips

Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	312 kips
Estimated Column Size:	**W14x48 ΦP_n=331 k**

Load above level 10: Roof + 3 Floors

LL_{red}	0.48
P_L=	105.4 kips
P_D=	136.4 kips
P_U=	7.2 kips
P_u=	369.5 kips

Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	419 kips
Estimated Column Size:	**W14x61 ΦP_n=543 k**

Load above level 8: Roof + 5 Floors

LL_{red}	0.43
P_L=	130.5 kips
P_D=	204.6 kips
P_U=	7.2 kips
P_u=	491.5 kips

Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	561 kips
Estimated Column Size:	**W14x68 ΦP_n=608 k**

Load above level 6: Roof + 7 Floors

LL_{red}	0.40
P_L=	154.4 kips
P_D=	272.8 kips
P_U=	7.2 kips
P_u=	611.5 kips

Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	681 kips
Estimated Column Size:	**W14x82 ΦP_n=735 k**

Load above level 4: Roof + 9 Floors

LL_{red}	0.40
P_L=	183.1 kips
P_D=	341.1 kips
P_U=	7.2 kips
P_u=	739.3 kips

Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	809 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 2: Roof + 11 Floors

LL_{red}	0.40
P_L=	211.8 kips
P_D=	409.3 kips
P_U=	7.2 kips
P_u=	867.1 kips

Unbalanced Moment, M_u=	40.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	937 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Tributary area per PH roof=	202 ft^2
Influence area on PH roof=	805 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof: PH roof+ 0 Floors

P_L=	6.1 kips
P_D=	18.2 kips
P_U=	4.0 kips
P_u=	33.5 kips

Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	18.5 ft
Equivalent Axial Load, P_{eq}=	34 kips
Estimated Column Size:	**W14x43 ΦP_n=196 k**

Column 35

Tributary Area	
per floor or roof=	698 ft^2
Influence Area=	2790 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Mech. Room)	150 psf
Roof Live Load= (Mech. Room)	150 psf
Roof Constr. Live Load	20 psf

Load above level 12: Roof + 1 Floor

LL_{red}	1.00
P_L=	209.4 kips
P_D=	132.6 kips
P_W=	14.0 kips
P_U=	504.5 kips
Unbalanced Moment, M_U=	6.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	515 kips
Estimated Column Size:	**W14x61 ΦP_n=543 k**

Load above level 10: Roof + 3 Floors

LL_{red}	0.80
P_L=	356.0 kips
P_D=	265.2 kips
P_W=	14.0 kips
P_U=	898.2 kips
Unbalanced Moment, M_U=	6.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	909 kips
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 8: Roof + 5 Floors

LL_{red}	0.80
P_L=	523.5 kips
P_D=	397.9 kips
P_W=	14.0 kips
P_U=	1325.3 kips
Unbalanced Moment, M_U=	6.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1336 kips
Estimated Column Size:	**W14x120 ΦP_n=1340 k**

Load above level 6: Roof + 7 Floors

LL_{red}	0.80
P_L=	691.0 kips
P_D=	530.5 kips
P_W=	14.0 kips
P_U=	1752.5 kips
Unbalanced Moment, M_U=	6.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1763 kips
Estimated Column Size:	**W14x159 ΦP_n=1810 k**

Load above level 4: Roof + 9 Floors

LL_{red}	0.80
P_L=	858.5 kips
P_D=	663.1 kips
P_W=	14.0 kips
P_U=	2179.7 kips
Unbalanced Moment, M_U=	6.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2190 kips
Estimated Column Size:	**W14x193 ΦP_n=2210 k**

Load above level 2: Roof + 11 Floors

LL_{red}	0.80
P_L=	1026.1 kips
P_D=	795.7 kips
P_W=	14.0 kips
P_U=	2606.9 kips
Unbalanced Moment, M_U=	6.2 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2617 kips
Estimated Column Size:	**W14x233 ΦP_n=2680 k**

Tributary area per PH roof=	20 ft^2
Influence area on PH roof=	80 ft^2
Roof Dead Load= (slab+SDL)	90 psf
Roof Live Load= (green roof area)	30 psf
Snow Load=	15.75 psf
Roof constr Live Load=	20 psf

Load above main roof: PH roof+ 0 Floors

P_L=	0.6 kips
P_D=	1.8 kips
P_W=	0.4 kips
P_U=	3.3 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.5 ft
Equivalent Axial Load, P_{eq}=	3 kips
Estimated Column Size:	**W14x43 ΦP_n=196 k**

Note: Refer to Moment Frame C elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame E: Estimated column sizes based on gravity load analysis only

Column 49	
Tributary Area	
per floor or roof=	339 ft^2
Influence Area=	1245 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.68
P_L=	33.1 kips
P_D=	64.4 kips
P_{Lr}=	6.8 kips
P_U=	133.6 kips
Unbalanced Moment, M_U=	234.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	535
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.50
P_L=	60.6 kips
P_D=	128.8 kips
P_{Lr}=	6.8 kips
P_U=	254.9 kips
Unbalanced Moment, M_U=	234.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	656
Estimated Column Size:	W14x74 ΦP_n=667 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.44
P_L=	84.8 kips
P_D=	193.2 kips
P_{Lr}=	6.8 kips
P_U=	370.9 kips
Unbalanced Moment, M_U=	234.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	772
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.41
P_L=	107.6 kips
P_D=	257.6 kips
P_{Lr}=	6.8 kips
P_U=	484.8 kips
Unbalanced Moment, M_U=	234.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	886
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	132.2 kips
P_D=	322.1 kips
P_{Lr}=	6.8 kips
P_U=	601.4 kips
Unbalanced Moment, M_U=	234.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1003
Estimated Column Size:	W14x99 ΦP_n=1100 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	159.3 kips
P_D=	386.5 kips
P_{Lr}=	6.8 kips
P_U=	722.1 kips
Unbalanced Moment, M_U=	234.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1123
Estimated Column Size:	W14x109 ΦP_n=1210 k

Column 50	
Tributary Area	
per floor or roof=	613 ft^2
Influence Area=	2366 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.56
P_L=	52.6 kips
P_D=	116.5 kips
P_{Lr}=	12.3 kips
P_U=	230.1 kips
Unbalanced Moment, M_U=	35.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	291 kips
Estimated Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.43
P_L=	97.1 kips
P_D=	232.9 kips
P_{Lr}=	12.3 kips
P_U=	441.0 kips
Unbalanced Moment, M_U=	35.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	502 kips
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	141.0 kips
P_D=	349.4 kips
P_{Lr}=	12.3 kips
P_U=	651.0 kips
Unbalanced Moment, M_U=	35.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	712 kips
Estimated Column Size:	W14x74 ΦP_n=667 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	190.0 kips
P_D=	465.9 kips
P_{Lr}=	12.3 kips
P_U=	869.2 kips
Unbalanced Moment, M_U=	35.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	930 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	239.1 kips
P_D=	582.4 kips
P_{Lr}=	12.3 kips
P_U=	1087.5 kips
Unbalanced Moment, M_U=	35.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1148 kips
Estimated Column Size:	W14x109 ΦP_n=1210 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	288.1 kips
P_D=	698.8 kips
P_{Lr}=	12.3 kips
P_U=	1305.7 kips
Unbalanced Moment, M_U=	35.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1367 kips
Estimated Column Size:	W14x132 ΦP_n=1480 k

Column 51	
Tributary Area	
per floor or roof=	339 ft^2
Influence Area=	1245 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.68
P_L=	33.1 kips
P_D=	64.4 kips
P_{Lr}=	6.8 kips
P_U=	133.6 kips
Unbalanced Moment, M_U=	60.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	237 kips
Estimated Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.50
P_L=	60.6 kips
P_D=	128.8 kips
P_{Lr}=	6.8 kips
P_U=	254.9 kips
Unbalanced Moment, M_U=	60.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	359 kips
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.44
P_L=	84.8 kips
P_D=	193.2 kips
P_{Lr}=	6.8 kips
P_U=	370.9 kips
Unbalanced Moment, M_U=	60.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	475 kips
Estimated Column Size:	W14x61 ΦP_n=543 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.41
P_L=	107.6 kips
P_D=	257.6 kips
P_{Lr}=	6.8 kips
P_U=	484.8 kips
Unbalanced Moment, M_U=	60.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	588 kips
Estimated Column Size:	W14x68 ΦP_n=608 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	132.2 kips
P_D=	322.1 kips
P_{Lr}=	6.8 kips
P_U=	601.4 kips
Unbalanced Moment, M_U=	60.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	705 kips
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	159.3 kips
P_D=	386.5 kips
P_{Lr}=	6.8 kips
P_U=	722.1 kips
Unbalanced Moment, M_U=	60.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	826 kips
Estimated Column Size:	W14x90 ΦP_n=1000 k

Note: Refer to Moment Frame E elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame 1: Estimated column sizes based on gravity load analysis only

Column 38	
Tributary Area	
per floor or roof=	540 ft^2
Influence Area=	2055 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green roof area)	30 psf
Roof Constr. Live Load	20 psf

Column 1
Refer to Moment Frame 1
Column 15
Refer to Moment Frame 2
Column 27
Refer to Moment Frame 3
Column 49
Refer to Moment Frame 4

Load above level 12:	Roof + 1 Floor
LL_{red}	0.58
P_L=	47.6 kips
P_D=	102.6 kips
$P_{U'}$=	10.8 kips
P_U=	204.6 kips
Unbalanced Moment, M_u=	173.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	502
Estimated Column Size:	**W14x61 ΦP_n=543 k**

Load above level 10:	Roof + 3 Floors
LL_{red}	0.44
P_L=	87.6 kips
P_D=	205.2 kips
$P_{U'}$=	10.8 kips
P_U=	391.9 kips
Unbalanced Moment, M_u=	173.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	689
Estimated Column Size:	**W14x82 ΦP_n=735 k**

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	124.2 kips
P_D=	307.8 kips
$P_{U'}$=	10.8 kips
P_U=	573.5 kips
Unbalanced Moment, M_u=	173.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	871
Estimated Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	167.4 kips
P_D=	410.4 kips
$P_{U'}$=	10.8 kips
P_U=	765.7 kips
Unbalanced Moment, M_u=	173.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1063
Estimated Column Size:	**W14x99 ΦP_n=1100 k**

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	210.6 kips
P_D=	513.0 kips
$P_{U'}$=	10.8 kips
P_U=	958.0 kips
Unbalanced Moment, M_u=	173.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1255
Estimated Column Size:	**W14x120 ΦP_n=1340 k**

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	253.8 kips
P_D=	615.6 kips
$P_{U'}$=	10.8 kips
P_U=	1150.2 kips
Unbalanced Moment, M_u=	173.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1448
Estimated Column Size:	**W14x132 ΦP_n=1480 k**

Note: Refer to Moment Frame 1 elevation in final report for column sizes that support combined gravity and lateral loads

Moment Frame 1': Estimated column sizes based on gravity load analysis only

Column 14	
Tributary Area	
per floor or roof=	323 ft²
Influence Area=	1186 ft²
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf
Load above level 12:	Roof + 1 Floor
LL_{red}	0.69
P_L=	54.4 kips
P_D=	61.4 kips
P_U=	6.5 kips
P_u=	164.0 kips
Unbalanced Moment, M_u=	194.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	497
Estimated Column Size:	W14x61 ΦP_n=543 k
Load above level 10:	Roof + 3 Floors
LL_{red}	0.50
P_L=	80.9 kips
P_D=	122.7 kips
P_U=	6.5 kips
P_u=	279.9 kips
Unbalanced Moment, M_u=	194.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	613
Estimated Column Size:	W14x74 ΦP_n=667 k
Load above level 8:	Roof + 5 Floors
LL_{red}	0.44
P_L=	104.1 kips
P_D=	184.1 kips
P_U=	6.5 kips
P_u=	390.8 kips
Unbalanced Moment, M_u=	194.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	724
Estimated Column Size:	W14x82 ΦP_n=735 k
Load above level 6:	Roof + 7 Floors
LL_{red}	0.41
P_L=	126.0 kips
P_D=	245.5 kips
P_U=	6.5 kips
P_u=	499.5 kips
Unbalanced Moment, M_u=	194.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	833
Estimated Column Size:	W14x90 ΦP_n=1000 k
Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	148.6 kips
P_D=	306.9 kips
P_U=	6.5 kips
P_u=	609.2 kips
Unbalanced Moment, M_u=	194.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	943
Estimated Column Size:	W14x90 ΦP_n=1000 k
Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	174.4 kips
P_D=	368.2 kips
P_U=	6.5 kips
P_u=	724.2 kips
Unbalanced Moment, M_u=	194.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1058
Estimated Column Size:	W14x99 ΦP_n=1100 k

Column 26	
Tributary Area	
per floor or roof=	553 ft²
Influence Area=	2113 ft²
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf
Load above level 12:	Roof + 1 Floor
LL_{red}	0.58
P_L=	87.2 kips
P_D=	105.1 kips
P_U=	11.1 kips
P_u=	271.1 kips
Unbalanced Moment, M_u=	122.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	480
Estimated Column Size:	W14x61 ΦP_n=543 k
Load above level 10:	Roof + 3 Floors
LL_{red}	0.44
P_L=	128.0 kips
P_D=	210.1 kips
P_U=	11.1 kips
P_u=	462.5 kips
Unbalanced Moment, M_u=	122.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	672
Estimated Column Size:	W14x82 ΦP_n=735 k
Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	165.9 kips
P_D=	315.2 kips
P_U=	11.1 kips
P_u=	645.2 kips
Unbalanced Moment, M_u=	122.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	858
Estimated Column Size:	W14x90 ΦP_n=1000 k
Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	210.1 kips
P_D=	420.3 kips
P_U=	11.1 kips
P_u=	846.1 kips
Unbalanced Moment, M_u=	122.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1055
Estimated Column Size:	W14x99 ΦP_n=1100 k
Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	254.4 kips
P_D=	525.4 kips
P_U=	11.1 kips
P_u=	1043.0 kips
Unbalanced Moment, M_u=	122.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1252
Estimated Column Size:	W14x120 ΦP_n=1340 k
Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	298.6 kips
P_D=	630.4 kips
P_U=	11.1 kips
P_u=	1239.8 kips
Unbalanced Moment, M_u=	122.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1449
Estimated Column Size:	W14x132 ΦP_n=1480 k

Column 37	
Tributary Area	
per floor or roof=	510 ft²
Influence Area=	1950 ft²
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace/Balcony)	100 psf
Roof Constr. Live Load	20 psf
Load above level 12:	Roof + 1 Floor
LL_{red}	0.59
P_L=	81.1 kips
P_D=	96.9 kips
P_U=	10.2 kips
P_u=	251.1 kips
Unbalanced Moment, M_u=	74.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	379
Estimated Column Size:	W14x61 ΦP_n=543 k
Load above level 10:	Roof + 3 Floors
LL_{red}	0.45
P_L=	119.3 kips
P_D=	193.8 kips
P_U=	10.2 kips
P_u=	428.5 kips
Unbalanced Moment, M_u=	74.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	556
Estimated Column Size:	W14x68 ΦP_n=608 k
Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	153.3 kips
P_D=	290.7 kips
P_U=	10.2 kips
P_u=	593.5 kips
Unbalanced Moment, M_u=	74.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	727
Estimated Column Size:	W14x82 ΦP_n=735 k
Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	193.8 kips
P_D=	387.6 kips
P_U=	10.2 kips
P_u=	780.3 kips
Unbalanced Moment, M_u=	74.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	908
Estimated Column Size:	W14x90 ΦP_n=1000 k
Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	234.6 kips
P_D=	484.5 kips
P_U=	10.2 kips
P_u=	961.9 kips
Unbalanced Moment, M_u=	74.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1090
Estimated Column Size:	W14x99 ΦP_n=1100 k
Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	275.4 kips
P_D=	581.4 kips
P_U=	10.2 kips
P_u=	1143.4 kips
Unbalanced Moment, M_u=	74.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1271
Estimated Column Size:	W14x120 ΦP_n=1340 k

Column 48	
Tributary Area	
per floor or roof=	459 ft²
Influence Area=	1755 ft²
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace/Balcony)	100 psf
Roof Constr. Live Load	20 psf
Load above level 12:	Roof + 1 Floor
LL_{red}	0.61
P_L=	73.8 kips
P_D=	87.2 kips
P_U=	9.2 kips
P_u=	227.3 kips
Unbalanced Moment, M_u=	100.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	400
Estimated Column Size:	W14x61 ΦP_n=543 k
Load above level 10:	Roof + 3 Floors
LL_{red}	0.46
P_L=	108.8 kips
P_D=	174.4 kips
P_U=	9.2 kips
P_u=	388.0 kips
Unbalanced Moment, M_u=	100.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	560
Estimated Column Size:	W14x68 ΦP_n=608 k
Load above level 8:	Roof + 5 Floors
LL_{red}	0.41
P_L=	140.0 kips
P_D=	261.6 kips
P_U=	9.2 kips
P_u=	542.6 kips
Unbalanced Moment, M_u=	100.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	715
Estimated Column Size:	W14x90 ΦP_n=1000 k
Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	174.4 kips
P_D=	348.8 kips
P_U=	9.2 kips
P_u=	702.3 kips
Unbalanced Moment, M_u=	100.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	875
Estimated Column Size:	W14x90 ΦP_n=1000 k
Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	211.1 kips
P_D=	436.1 kips
P_U=	9.2 kips
P_u=	865.7 kips
Unbalanced Moment, M_u=	100.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1038
Estimated Column Size:	W14x99 ΦP_n=1100 k
Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	247.9 kips
P_D=	523.3 kips
P_U=	9.2 kips
P_u=	1029.1 kips
Unbalanced Moment, M_u=	100.5 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1201
Estimated Column Size:	W14x109 ΦP_n=1210 k

Column S5

Tributary Area	
per floor or roof=	230 ft^2
Influence Area=	829 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Constr. Live Load	20 psf

Load above level 12:

	Roof + 1 Floor
LL_{red}	0.77
P_L=	40.7 kips
P_D=	43.7 kips
P_W=	4.6 kips
P_U=	119.9 kips
Unbalanced Moment, M_u=	98.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	288
Estimated Column Size:	**W14x43 ΦP_n=292 k**

Load above level 10:

	Roof + 3 Floors
LL_{red}	0.55
P_L=	61.0 kips
P_D=	87.4 kips
P_W=	4.6 kips
P_U=	204.8 kips
Unbalanced Moment, M_u=	98.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	373
Estimated Column Size:	**W14x61 ΦP_n=543 k**

Load above level 8:

	Roof + 5 Floors
LL_{red}	0.48
P_L=	78.5 kips
P_D=	131.1 kips
P_W=	4.6 kips
P_U=	285.3 kips
Unbalanced Moment, M_u=	98.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	453
Estimated Column Size:	**W14x61 ΦP_n=543 k**

Load above level 6:

	Roof + 7 Floors
LL_{red}	0.45
P_L=	95.0 kips
P_D=	174.8 kips
P_W=	4.6 kips
P_U=	364.0 kips
Unbalanced Moment, M_u=	98.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	532
Estimated Column Size:	**W14x61 ΦP_n=543 k**

Load above level 4:

	Roof + 9 Floors
LL_{red}	0.40
P_L=	105.8 kips
P_D=	218.5 kips
P_W=	4.6 kips
P_U=	433.8 kips
Unbalanced Moment, M_u=	98.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	602
Estimated Column Size:	**W14x68 ΦP_n=508 k**

Load above level 2:

	Roof + 11 Floors
LL_{red}	0.40
P_L=	124.2 kips
P_D=	262.2 kips
P_W=	4.6 kips
P_U=	513.7 kips
Unbalanced Moment, M_u=	98.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	684
Estimated Column Size:	**W14x82 ΦP_n=735 k**

Note: Refer to Moment Frame 1' elevation in final report for column sizes that support combined gravity and lateral loads

Gravity Column Design

Column 13 - Gravity Column	
Tributary Area	
per floor or roof=	456 ft^2
Influence Area=	1608 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.62
P_L=	74.1 kips
P_D=	86.6 kips
P_U=	9.1 kips
P_U=	227.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	227
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.47
P_L=	109.3 kips
P_D=	173.3 kips
P_U=	9.1 kips
P_U=	387.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	387
Column Size:	W14x61 ΦP_n=542 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.42
P_L=	140.7 kips
P_D=	259.9 kips
P_U=	9.1 kips
P_U=	541.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	542
Column Size:	W14x61 ΦP_n=543 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	173.3 kips
P_D=	346.6 kips
P_U=	9.1 kips
P_U=	697.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	698
Column Size:	W14x82 ΦP_n=735 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	209.8 kips
P_D=	433.2 kips
P_U=	9.1 kips
P_U=	860.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	860
Column Size:	W14x90 ΦP_n=1000 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	246.2 kips
P_D=	519.8 kips
P_U=	9.1 kips
P_U=	1022.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1022
Column Size:	W14x99 ΦP_n=1100 k

Column 25 - Gravity Column	
Tributary Area	
per floor or roof=	772 ft^2
Influence Area=	3482 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.50
P_L=	116.1 kips
P_D=	146.7 kips
P_U=	15.4 kips
P_U=	369.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	370
Column Size:	W14x61 ΦP_n=543 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.40
P_L=	169.8 kips
P_D=	293.4 kips
P_U=	15.4 kips
P_U=	631.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	631
Column Size:	W14x74 ΦP_n=667 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	231.6 kips
P_D=	440.0 kips
P_U=	15.4 kips
P_U=	906.3 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	906
Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	293.4 kips
P_D=	586.7 kips
P_U=	15.4 kips
P_U=	1181.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1181
Column Size:	W14x109 ΦP_n=1210 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	355.1 kips
P_D=	733.4 kips
P_U=	15.4 kips
P_U=	1456.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1456
Column Size:	W14x132 ΦP_n=1480 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	416.9 kips
P_D=	880.1 kips
P_U=	15.4 kips
P_U=	1730.8 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1731
Column Size:	W14x159 ΦP_n=1810 k

Column 36 - Gravity Column	
Tributary Area	
per floor or roof=	712.5 ft^2
Influence Area=	2850 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Terrace)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.53
P_L=	84.1 kips
P_D=	135.4 kips
P_U=	14.3 kips
P_U=	304.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	304
Column Size:	W14x48 ΦP_n=331 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.41
P_L=	134.4 kips
P_D=	270.8 kips
P_U=	14.3 kips
P_U=	547.1 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	547
Column Size:	W14x68 ΦP_n=608 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	188.8 kips
P_D=	406.1 kips
P_U=	14.3 kips
P_U=	796.6 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	797
Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	245.8 kips
P_D=	541.5 kips
P_U=	14.3 kips
P_U=	1050.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1050
Column Size:	W14x99 ΦP_n=1100 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	302.8 kips
P_D=	676.9 kips
P_U=	14.3 kips
P_U=	1303.9 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1304
Column Size:	W14x120 ΦP_n=1340 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	359.8 kips
P_D=	812.3 kips
P_U=	14.3 kips
P_U=	1557.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1558
Column Size:	W14x145 ΦP_n=1650 k

Column 39 - Gravity Column	
Tributary Area	
per floor or roof=	976 ft^2
Influence Area=	3904.5 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.50
P_L=	78.1 kips
P_D=	185.4 kips
P_U=	19.5 kips
P_U=	357.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	357
Column Size:	W14x53 ΦP_n=369 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.40
P_L=	146.4 kips
P_D=	370.9 kips
P_U=	19.5 kips
P_U=	689.1 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	689
Column Size:	W14x82 ΦP_n=735 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	224.5 kips
P_D=	556.3 kips
P_U=	19.5 kips
P_U=	1036.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1037
Column Size:	W14x99 ΦP_n=1100 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	302.6 kips
P_D=	741.8 kips
P_U=	19.5 kips
P_U=	1384.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1384
Column Size:	W14x132 ΦP_n=1480 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	380.6 kips
P_D=	927.2 kips
P_U=	19.5 kips
P_U=	1731.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1731
Column Size:	W14x159 ΦP_n=1810 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	458.7 kips
P_D=	1112.6 kips
P_U=	19.5 kips
P_U=	2078.9 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	2079
Column Size:	W14x193 ΦP_n=2210 k

Column 40 - Gravity Column

Tributary Area		
per floor or roof=	745	ft^2
Influence Area=	2808	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.53
P_L=	62.1 kips
P_D=	141.6 kips
P_{Lr}=	14.9 kips
P_U=	276.6 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	277
Column Size:	**W14x43 ΦP_n=292 k**

Load above level 10:	Roof + 3 Floors
LL_{red}	0.41
P_L=	114.8 kips
P_D=	283.1 kips
P_{Lr}=	14.9 kips
P_U=	530.8 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	531
Column Size:	**W14x61 ΦP_n=543 k**

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	171.4 kips
P_D=	424.7 kips
P_{Lr}=	14.9 kips
P_U=	791.2 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	791
Column Size:	**W14x90 ΦP_n=1000 k**

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	231.0 kips
P_D=	566.2 kips
P_{Lr}=	14.9 kips
P_U=	1056.4 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1056
Column Size:	**W14x99 ΦP_n=1100 k**

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	290.6 kips
P_D=	707.8 kips
P_{Lr}=	14.9 kips
P_U=	1321.6 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1322
Column Size:	**W14x120 ΦP_n=1340 k**

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	350.2 kips
P_D=	849.3 kips
P_{Lr}=	14.9 kips
P_U=	1586.9 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1587
Column Size:	**W14x145 ΦP_n=1650 k**

Column 41 - Gravity Column

Tributary Area		
per floor or roof=	200	ft^2
Influence Area=	1010	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.72
P_L=	20.4 kips
P_D=	38.0 kips
P_{Lr}=	4.0 kips
P_U=	80.3 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	80
Column Size:	**W14x43 ΦP_n=292 k**

Load above level 10:	Roof + 3 Floors
LL_{red}	0.52
P_L=	37.4 kips
P_D=	76.0 kips
P_{Lr}=	4.0 kips
P_U=	153.0 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	153
Column Size:	**W14x43 ΦP_n=292 k**

Load above level 8:	Roof + 5 Floors
LL_{red}	0.46
P_L=	52.1 kips
P_D=	114.0 kips
P_{Lr}=	4.0 kips
P_U=	222.2 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	222
Estimated Column Size:	**W14x43 ΦP_n=292 k**

Load above level 6:	Roof + 7 Floors
LL_{red}	0.43
P_L=	66.0 kips
P_D=	152.0 kips
P_{Lr}=	4.0 kips
P_U=	290.0 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	290
Column Size:	**W14x43 ΦP_n=292 k**

Load above level 4:	Roof + 9 Floors
LL_{red}	0.41
P_L=	79.3 kips
P_D=	190.0 kips
P_{Lr}=	4.0 kips
P_U=	356.9 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	357
Column Size:	**W14x53 ΦP_n=369 k**

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	94.0 kips
P_D=	228.0 kips
P_{Lr}=	4.0 kips
P_U=	426.0 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	426
Column Size:	**W14x61 ΦP_n=543 k**

Column 42 - Gravity Column

Tributary Area		
per floor or roof=	203	ft^2
Influence Area=	1417	ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95	psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95	psf
Floor Live Load= (Office Space)	100	psf
Roof Live Load= (Green Roof)	30	psf
Construct. Roof LL=	20	psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.65
P_L=	19.3 kips
P_D=	38.6 kips
P_{Lr}=	4.1 kips
P_U=	79.1 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	79
Column Size:	**W14x43 ΦP_n=292 k**

Load above level 10:	Roof + 3 Floors
LL_{red}	0.48
P_L=	35.3 kips
P_D=	77.1 kips
P_{Lr}=	4.1 kips
P_U=	151.1 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	151
Column Size:	**W14x43 ΦP_n=292 k**

Load above level 8:	Roof + 5 Floors
LL_{red}	0.43
P_L=	49.6 kips
P_D=	115.7 kips
P_{Lr}=	4.1 kips
P_U=	220.2 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	220
Estimated Column Size:	**W14x43 ΦP_n=292 k**

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	63.0 kips
P_D=	154.3 kips
P_{Lr}=	4.1 kips
P_U=	288.0 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	288
Column Size:	**W14x43 ΦP_n=292 k**

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	79.2 kips
P_D=	192.9 kips
P_{Lr}=	4.1 kips
P_U=	360.1 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	360
Column Size:	**W14x53 ΦP_n=369 k**

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	95.4 kips
P_D=	231.4 kips
P_{Lr}=	4.1 kips
P_U=	432.4 kips
Unbalanced Moment, M_u=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	432
Column Size:	**W14x61 ΦP_n=543 k**

Column 43 - Gravity Column

Tributary Area	
per floor or roof=	256 ft^2
Influence Area=	1025 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.72
P_L=	26.1 kips
P_D=	48.6 kips
P_{LL}=	5.1 kips
P_U=	102.6 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	103
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.52
P_L=	47.7 kips
P_D=	97.3 kips
P_{LL}=	5.1 kips
P_U=	195.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	196
Column Size:	W14x43 ΦP_n=292 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.46
P_L=	66.5 kips
P_D=	145.9 kips
P_{LL}=	5.1 kips
P_U=	284.1 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	284
Estimated Column Size:	W14x43 ΦP_n=292 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.43
P_L=	84.2 kips
P_D=	194.6 kips
P_{LL}=	5.1 kips
P_U=	370.8 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	371
Column Size:	W14x61 ΦP_n=543 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.41
P_L=	101.3 kips
P_D=	243.2 kips
P_{LL}=	5.1 kips
P_U=	456.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	456
Column Size:	W14x61 ΦP_n=543 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	120.3 kips
P_D=	291.8 kips
P_{LL}=	5.1 kips
P_U=	545.3 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	545
Column Size:	W14x68 ΦP_n=608 k

Column 44 - Gravity Column

Tributary Area	
per floor or roof=	772 ft^2
Influence Area=	1082 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.71
P_L=	77.7 kips
P_D=	146.7 kips
P_{LL}=	15.4 kips
P_U=	308.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	308
Column Size:	W14x48 ΦP_n=331 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.51
P_L=	142.0 kips
P_D=	293.4 kips
P_{LL}=	15.4 kips
P_U=	587.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	587
Column Size:	W14x68 ΦP_n=608 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.45
P_L=	198.4 kips
P_D=	440.0 kips
P_{LL}=	15.4 kips
P_U=	853.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	853
Estimated Column Size:	W14x90 ΦP_n=1000 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.42
P_L=	251.4 kips
P_D=	586.7 kips
P_{LL}=	15.4 kips
P_U=	1114.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1114
Column Size:	W14x109 ΦP_n=1210 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	302.5 kips
P_D=	733.4 kips
P_{LL}=	15.4 kips
P_U=	1371.8 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1372
Column Size:	W14x132 ΦP_n=1480 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	362.8 kips
P_D=	880.1 kips
P_{LL}=	15.4 kips
P_U=	1644.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1644
Column Size:	W14x145 ΦP_n=1650 k

Column 45 - Gravity Column

Tributary Area	
per floor or roof=	290 ft^2
Influence Area=	1130 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.70
P_L=	28.9 kips
P_D=	55.1 kips
P_{LL}=	5.8 kips
P_U=	115.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	115
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.51
P_L=	52.9 kips
P_D=	110.2 kips
P_{LL}=	5.8 kips
P_U=	219.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	220
Column Size:	W14x43 ΦP_n=292 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.45
P_L=	73.9 kips
P_D=	165.3 kips
P_{LL}=	5.8 kips
P_U=	319.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	319
Estimated Column Size:	W14x48 ΦP_n=331 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.42
P_L=	93.7 kips
P_D=	220.4 kips
P_{LL}=	5.8 kips
P_U=	417.3 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	417
Column Size:	W14x61 ΦP_n=543 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	113.1 kips
P_D=	275.5 kips
P_{LL}=	5.8 kips
P_U=	514.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	514
Column Size:	W14x61 ΦP_n=543 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	136.3 kips
P_D=	330.6 kips
P_{LL}=	5.8 kips
P_U=	617.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	618
Column Size:	W14x74 ΦP_n=667 k

Column 46 - Gravity Column

Tributary Area	
per floor or roof=	551 ft^2
Influence Area=	2944 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.53
P_L=	45.5 kips
P_D=	104.7 kips
P_{LL}=	11.0 kips
P_U=	204.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	204
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.41
P_L=	84.2 kips
P_D=	209.4 kips
P_{LL}=	11.0 kips
P_U=	391.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	392
Column Size:	W14x61 ΦP_n=543 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	126.7 kips
P_D=	314.1 kips
P_{LL}=	11.0 kips
P_U=	585.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	585
Estimated Column Size:	W14x68 ΦP_n=608 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	170.8 kips
P_D=	418.8 kips
P_{LL}=	11.0 kips
P_U=	781.3 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	781
Column Size:	W14x90 ΦP_n=1000 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	214.9 kips
P_D=	523.5 kips
P_{LL}=	11.0 kips
P_U=	977.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	977
Column Size:	W14x90 ΦP_n=1000 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	259.0 kips
P_D=	628.1 kips
P_{LL}=	11.0 kips
P_U=	1173.6 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, K=	1.0
Column Length, L=	15.0 ft
Equivalent Axial Load, P_{eq}=	1174
Column Size:	W14x109 ΦP_n=1210 k

Column 47 - Gravity Column	
Tributary Area	
per floor or roof=	641.3 ft^2
Influence Area=	2883 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.53
P_L=	53.2 kips
P_D=	121.8 kips
P_{LL}=	12.8 kips
P_U=	237.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	238
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.41
P_L=	96.4 kips
P_D=	243.7 kips
P_{LL}=	12.8 kips
P_U=	456.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	456
Column Size:	W14x61 ΦP_n=543 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.40
P_L=	147.5 kips
P_D=	365.5 kips
P_{LL}=	12.8 kips
P_U=	681.1 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	681
Estimated Column Size:	W14x82 ΦP_n=735 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.40
P_L=	198.8 kips
P_D=	487.4 kips
P_{LL}=	12.8 kips
P_U=	909.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	909
Column Size:	W14x90 ΦP_n=1000 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	250.1 kips
P_D=	609.2 kips
P_{LL}=	12.8 kips
P_U=	1137.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	1138
Column Size:	W14x109 ΦP_n=1210 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	301.4 kips
P_D=	731.1 kips
P_{LL}=	12.8 kips
P_U=	1366.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	1366
Column Size:	W14x132 ΦP_n=1480 k

Column 52 - Gravity Column	
Tributary Area	
per floor or roof=	324 ft^2
Influence Area=	1123 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.70
P_L=	32.3 kips
P_D=	61.6 kips
P_{LL}=	6.5 kips
P_U=	128.8 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	129
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.51
P_L=	59.1 kips
P_D=	123.1 kips
P_{LL}=	6.5 kips
P_U=	245.6 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	246
Column Size:	W14x43 ΦP_n=292 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.45
P_L=	82.6 kips
P_D=	184.7 kips
P_{LL}=	6.5 kips
P_U=	357.1 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	357
Column Size:	W14x53 ΦP_n=369 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.42
P_L=	104.8 kips
P_D=	246.2 kips
P_{LL}=	6.5 kips
P_U=	466.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	466
Column Size:	W14x61 ΦP_n=543 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	126.4 kips
P_D=	307.8 kips
P_{LL}=	6.5 kips
P_U=	574.8 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	575
Column Size:	W14x68 ΦP_n=608 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	152.3 kips
P_D=	369.4 kips
P_{LL}=	6.5 kips
P_U=	690.1 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	690
Column Size:	W14x82 ΦP_n=735 k

Column 53 - Gravity Column	
Tributary Area	
per floor or roof=	275.6 ft^2
Influence Area=	1041.2 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.71
P_L=	28.0 kips
P_D=	52.4 kips
P_{LL}=	5.5 kips
P_U=	110.3 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	110
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.52
P_L=	51.1 kips
P_D=	104.7 kips
P_{LL}=	5.5 kips
P_U=	210.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	210
Column Size:	W14x43 ΦP_n=292 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.46
P_L=	71.4 kips
P_D=	157.1 kips
P_{LL}=	5.5 kips
P_U=	305.5 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	305
Column Size:	W14x48 ΦP_n=331 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.43
P_L=	90.4 kips
P_D=	209.5 kips
P_{LL}=	5.5 kips
P_U=	398.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	399
Column Size:	W14x61 ΦP_n=543 k

Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	108.7 kips
P_D=	261.8 kips
P_{LL}=	5.5 kips
P_U=	490.9 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	491
Column Size:	W14x61 ΦP_n=543 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	129.5 kips
P_D=	314.2 kips
P_{LL}=	5.5 kips
P_U=	587.0 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	587
Column Size:	W14x68 ΦP_n=608 k

Column 54 - Gravity Column	
Tributary Area	
per floor or roof=	320.6 ft^2
Influence Area=	1211 ft^2
Floor Dead Load= (slab+SDL+mem. self-wts)	95 psf
Roof Dead Load= (slab+SDL+mem. self-wts)	95 psf
Floor Live Load= (Office Space)	100 psf
Roof Live Load= (Green Roof)	30 psf
Construct. Roof LL=	20 psf

Load above level 12:	Roof + 1 Floor
LL_{red}	0.68
P_L=	31.5 kips
P_D=	60.9 kips
P_{LL}=	6.4 kips
P_U=	126.6 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	127
Column Size:	W14x43 ΦP_n=292 k

Load above level 10:	Roof + 3 Floors
LL_{red}	0.50
P_L=	57.6 kips
P_D=	121.8 kips
P_{LL}=	6.4 kips
P_U=	241.6 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	242
Column Size:	W14x43 ΦP_n=292 k

Load above level 8:	Roof + 5 Floors
LL_{red}	0.44
P_L=	80.6 kips
P_D=	182.7 kips
P_{LL}=	6.4 kips
P_U=	351.4 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	351
Column Size:	W14x53 ΦP_n=369 k

Load above level 6:	Roof + 7 Floors
LL_{red}	0.41
P_L=	102.3 kips
P_D=	243.7 kips
P_{LL}=	6.4 kips
P_U=	459.2 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	459
Column Size:	W14x61 ΦP_n=543 k

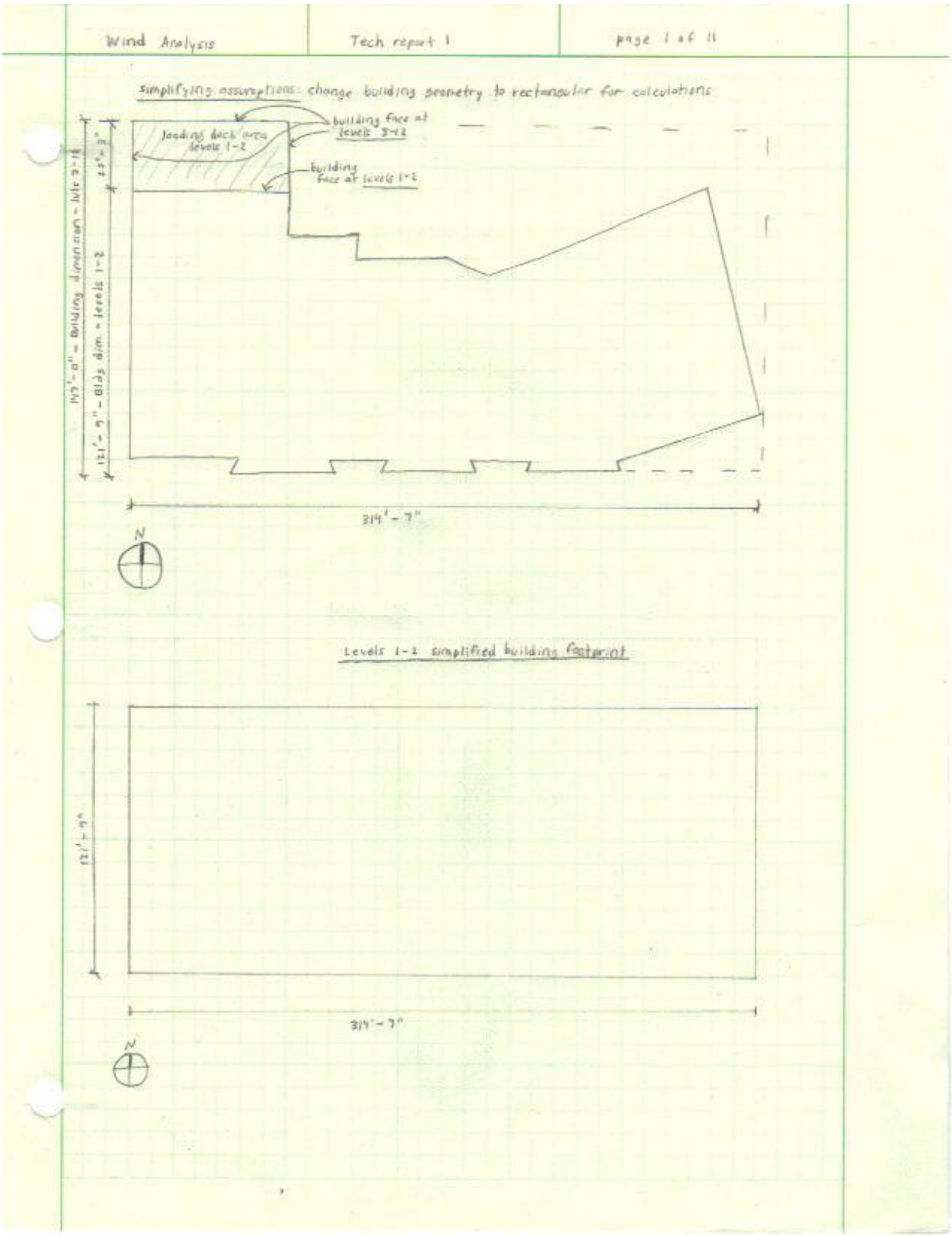
Load above level 4:	Roof + 9 Floors
LL_{red}	0.40
P_L=	125.0 kips
P_D=	304.6 kips
P_{LL}=	6.4 kips
P_U=	568.7 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	569
Column Size:	W14x68 ΦP_n=608 k

Load above level 2:	Roof + 11 Floors
LL_{red}	0.40
P_L=	150.7 kips
P_D=	365.5 kips
P_{LL}=	6.4 kips
P_U=	682.9 kips
Unbalanced Moment, M_U=	0.0 kft
Column Shape=	W14's
Column Depth=	14.0 in
Effective Length, Kx=	1.0
Column Length, Lx=	15.0 ft
Equivalent Axial Load, P_{eq}=	683
Column Size:	W14x82 ΦP_n=735 k

Note: Refer to final report for gravity column schedule

Appendix B: Wind Load Calculations

Existing Concrete System Wind Analysis



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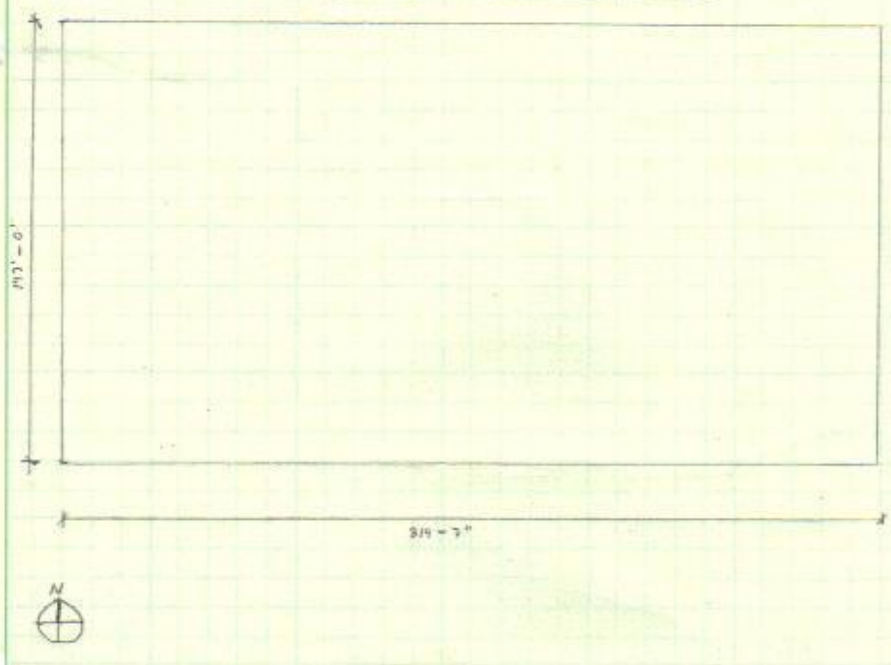
Levels 3-12 simplified building footprint



For analysis, use method 2

step 1: wind speed (V) → using Fig. 26.5-1A, V = 115 mph

step 2: directionality factor, K_d → using Table 26.6-1, $K_d = 0.85$
step 3: exposure category - B (urban, suburban)
step 4: topographic factor, K_{zt} → from sect 26.8.2, $K_{zt} = 1.0$ for homogeneous topography

step 5: gust effect factor, G_f → from sect 26.9
- since bldg ht < 300' and bldg ht < 4Leff in both N-S and E-W directions, use n_a to determine the natural frequency.
the approximate lower-bound natural frequency, n_a, for a concrete moment-resisting frame (sect 26.9.7)

$$n_1 = n_a = \frac{43.5}{h^{0.9}}$$

$$= \frac{43.5}{130^{0.9}} = 0.344 \text{ Hz} < 1 \text{ Hz} \therefore$$ calculate G_f in the event the building is flexible

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

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$g_Q = g_v = 3.4$

$$g_r = \sqrt{2\ln(3{,}600\,(0.544))} + \frac{0.577}{\sqrt{2\ln(3{,}600\,(0.544))}} = 3.8936 + \frac{0.577}{3.8936} = 4.042$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}}$$

$\bar{z} = 0.6h = 0.6(130) = 78\ ft > z_{min} = 30\ ft$ OK ✓

from table 26.9-1, $\bar{\alpha} = 1/4.0$, $\bar{b} = 0.45$, $c = 0.30$, $\ell = 320\ ft$, $\bar{\epsilon} = 1/3.0$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{78}\right)^{1/6} = 0.24$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{78}{33}\right)^{1/3} = 426.26$$

$$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{78}{33}\right)^{1/4}\left(\frac{88}{60}\right)(115) = 94.11$$

$$N_1 = \frac{0.594\,(426.26)}{94.11} = 2.46$$

$$R_n = \frac{7.47(2.46)}{(1+10.3(2.46))^{5/3}} = 0.0788$$

damping ratio, $\beta = 0.010$ (from section C26.9 in ASCE 7-10, 1% is recommended for concrete buildings)

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North - South Direction (levels 1-2)

$h = 130$ ft

$L = 121.75$ ft

$B = 314.58$ ft

$$\eta_h = 4.6\, n_1 \frac{h}{V_{\bar{z}}} = 4.6(0.544)\frac{(130)}{94.11} = 3.46$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{3.46} - \frac{1}{2(3.46)^2}\left(1 - e^{-2(3.46)}\right)$$

$$= 0.289 - 0.0418(0.999) = 0.247$$

$$\eta_B = 4.6\, n_1 \frac{B}{V_{\bar{z}}} = 4.6(0.544)\frac{(314.58)}{94.11} = 8.36$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{8.36} - \frac{1}{2(8.36)^2}\left(1 - e^{-2(8.36)}\right)$$

$$= 0.1196 - 0.00715(1.00) = 0.112$$

$$\eta_L = 15.4\, n_1 \frac{L}{V_{\bar{z}}} = 15.4(0.544)\frac{(121.75)}{94.11} = 10.84$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{10.84} - \frac{1}{2(10.84)^2}\left(1 - e^{-2(10.84)}\right)$$

$$= 0.0923 - 0.00426(1.00) = 0.088$$

$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.112)[0.53 + 0.47(0.088)]}$$

$$= 0.353$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{314.58 + 130}{426.26}\right)^{0.63}}} = 0.78$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.78)^2 + 4.04^2(0.353)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.861}$$

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East-West direction (levels 1-2)

$h = 130$ ft

$L = 314.58$ ft

$B = 121.75$ ft

$n_h = 3.46$ (see N-S direction)

$R_h = 0.247$ (see N-S direction)

$$n_B = 4.6\,(0.544)\frac{(121.75)}{44.11} = 3.24$$

$$R_B = \frac{1}{3.24} - \frac{1}{2(3.24)^2}\left(1 - e^{-2(3.24)}\right)$$

$$= 0.309 - 0.0476\,(0.998) = 0.261$$

$$n_L = 15.4\,(0.544)\frac{(314.58)}{44.11} = 28.00$$

$$R_L = \frac{1}{28.00} - \frac{1}{2(28)^2}\left(1 - e^{-2(28)}\right)$$

$$= 0.0357 - 0.000638 = 0.035$$

$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.261)[0.53 + 0.93(0.035)]}$$

$$= 0.527$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{121.75 + 130}{426.26}\right)^{0.63}}} = 0.83$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.83)^2 + 4.042^2(0.527)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.945}$$

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North - South direction (levels 3-12)

$h = 130\,ft$

$L = 147\,ft$

$B = 314.58\,ft$

$$\eta_h = 4.6\,(0.544)\frac{(130)}{44.11} = 3.46$$

$$R_h = \frac{1}{3.46} - \frac{1}{2(3.46)^2}\left(1 - e^{-2(3.46)}\right) = 0.247$$

$$\eta_B = 4.6\,(0.544)\frac{(314.58)}{44.11} = 8.36$$

$$R_B = \frac{1}{8.36} - \frac{1}{2(8.36)^2}\left(1 - e^{-2(8.36)}\right) = 0.112$$

$$\eta_L = 15.4\,(0.544)\frac{(147)}{44.11} = 13.07$$

$$R_L = \frac{1}{13.07} - \frac{1}{2(13.07)^2}\left(1 - e^{-2(13.07)}\right) = 0.073$$

$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.112)[0.53 + 0.47(0.073)]}$$

$$= 0.351$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{314.58 + 130}{426.26}\right)^{0.63}}} = 0.78$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{2.4^2(0.78)^2 + 4.042^2(0.351)^2}}{1 + 1.7(2.4)(0.26)}\right)$$

$$= \boxed{0.861}$$

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East-west direction (levels 3-12)

$h = 130$ ft

$L = 314.58$ ft

$B = 147$ ft

$n_h = 3.96$ (see N-S direction)

$R_h = 0.247$ (see N-S direction)

$$n_B = 4.6\,(0.549)\frac{(147)}{99.11} = 3.91$$

$$R_B = \frac{1}{3.91} - \frac{1}{2(3.91)^2}\left(1 - e^{-2(3.91)}\right) = 0.22$$

$$n_L = 15.4\,(0.549)\frac{(314.58)}{99.11} = 28.00$$

$$R_L = \frac{1}{28} - \frac{1}{2(28)^2}\left(1 - e^{-2(28)}\right) = 0.035$$

$$R = \sqrt{\frac{\pi}{0.01}(0.0288)(0.247)(0.22)(0.53 + 0.47)(0.035)}$$

$$= 0.484$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{147 + 130}{426.24}\right)^{0.63}}} = 0.82$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.82)^2 + 4.042^2(0.484)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.926}$$

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step 6: Enclosure classification – Fully enclosed

step 7: Internal pressure coefficient → using table 26.11-1, $(GC_{pi}) = \pm 0.18$

step 8: velocity pressure exposure coefficients, K_z or K_h → using table 27.3-1, see excel spreadsheet

step 9: velocity pressure q_z or q_h → from sect. 27.3.2, $q_z = 0.00256\, K_z K_{zt} K_d V^2$ (lbf/ft²)

see excel spread sheet for values

step 10: external pressure coefficient, C_p → using Fig. 27.3-1

wall pressure coefficients, C_p

windward wall: → all L/B values → $C_p = 0.8$

side wall: → all L/B values → $C_p = -0.7$

Leeward wall: levels 1-2

N-S wind

$L/B = \frac{121.75}{319.58} = 0.39$, $0 < 0.39 < 1 \Rightarrow C_p = -0.5$

E-W wind

$L/B = \frac{319.58}{121.75} = 2.58$, $2 < 2.58 < 4 \Rightarrow C_p$ → interpolate based on L/B values

$$\frac{C_p - C_{p_1}}{C_{p_f} - C_{p_1}} = \frac{(L/B) - (L/B)_1}{(L/B)_f - (L/B)_1}$$

$$C_p = \frac{(L/B) - (L/B)_1}{(L/B)_f - (L/B)_1}(C_{p_f} - C_{p_1}) + C_{p_1}$$

$$= \frac{2.58 - 2}{4 - 2}(-0.2 + 0.3) + (-0.3)$$

$$= -0.271$$

levels 3-12

N-S wind

$L/B = \frac{147}{319.58} = 0.47$, $0 < 0.47 < 1 \Rightarrow C_p = -0.5$

E-W wind

$L/B = \frac{319.58}{147} = 2.14$, $2 < 2.14 < 4 \Rightarrow C_p$ → interpolate based on L/B values

$$C_p = \frac{2.14 - 2}{4 - 2}(-0.2 + 0.3) + (-0.3)$$

$$= -0.297$$

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Roof Pressures coefficients, C_p

$\theta = 0°$

N-S wind

$h/L = \frac{130}{197} = 0.88$ $\quad 0.5 < 0.88 < 1.0$

horizontal distance from windward edge: 0 to $h/2 \rightarrow 0°$ to $65° \Rightarrow C_p \rightarrow$ interpolate based on h/L values

$$C_p = \frac{0.88 - 0.5}{1 - 0.5}(-1.3 + 0.9) + (-0.9)$$
$$= -1.204$$

$h/2$ to $h \rightarrow 65°$ to $130° \Rightarrow C_p = \frac{0.88 - 0.5}{1 - 0.5}(-0.7 + 0.9) + (-0.9)$
$$= -0.748$$

h to $2h \rightarrow 130°$ to $260 \Rightarrow C_p = \frac{0.88 - 0.5}{1 - 0.5}(-0.7 + 0.5) + (-0.5)$
$$= -0.652$$

$> 2h \rightarrow N/A$

E-W wind

$h/L = \frac{130}{314.58} = 0.41 < 0.5$

horizontal distance from windward edge: 0 to $\frac{h}{2} \rightarrow 0°$ to $65° \Rightarrow C_p = -0.9$

$h/2$ to $h \rightarrow 65° - 130° \Rightarrow C_p = -0.9$
h to $2h \rightarrow 130° - 260° \Rightarrow C_p = -0.5$
$> 2h \rightarrow > 260° \Rightarrow C_p = -0.7$

step 11: design wind pressures $\rightarrow$ from sect. 27.4.2, $p = q G_f C_p - q_i (G_i C_{pi})$ (lb/ft²)

windward walls $\rightarrow p = \underbrace{q_z G_f C_p}_{\text{external pressure}} - \underbrace{q_h (G_i C_{pi})}_{\text{internal pressure}}$

leeward walls, side walls, roof $\rightarrow p = \underbrace{q_h G_f C_p}_{\text{external pressure}} - \underbrace{q_h (G_i C_{pi})}_{\text{internal pressure}}$

- see excel spread sheet for pressures

note: internal wind pressures were neglected in calculating the design wind pressures since the internal pressures do not contribute towards the external wind pressures acting on the building

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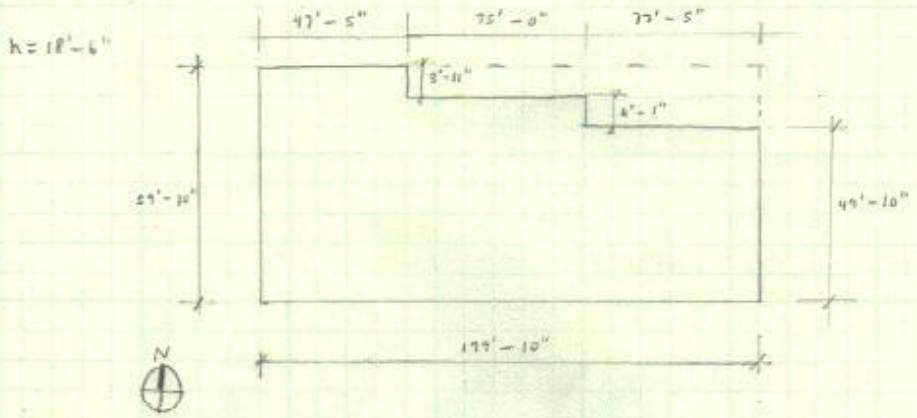
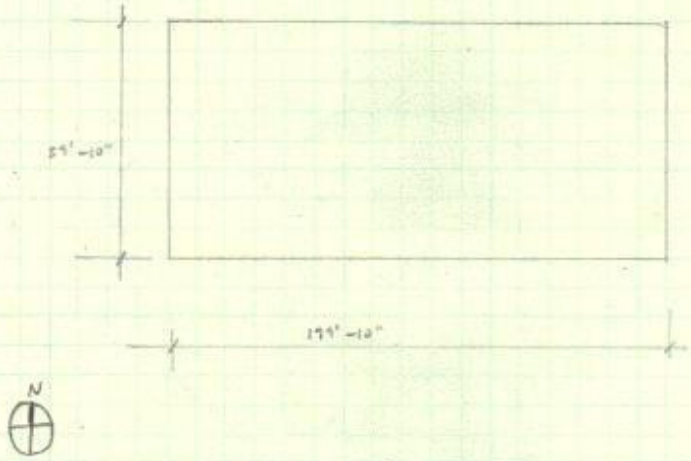
Mechanical Penthouse

Assumption: mech. Penthouse is not a part of the MWFRS
- the penthouse will be analyzed as a separate structure and therefore the guidelines listed in chapter 29 (wind loads on other structures and building appurtenances - MWFRS) will be followed for analysis



simplifying assumptions: change penthouse geometry to rectangular for calculations



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step 1: risk category → using table 1.5-1, risk category II
step 2: V = 115 mph
step 3: $K_d = 0.90$
step 4: exposure category, B
step 5: $K_{zt} = 1.0$
step 6: Gust factor, G_f → from sect. 26.9.1, $G_f = 0.85$ for other structures
step 7: K_z or K_h → using table 29.3-1, $K_z = 1.13$

step 8: q_z → using sect. 29.3.2, $q_z = 0.00256\, K_z K_{zt} K_d V^2$
$= 0.00256\,(1.13)(1.0)(0.90)(115)^2$
$= 34.43$ psf

step 9: force coefficient, C_f → using Fig. 29.5-1

$$h/D = \frac{\text{height of structure}}{\text{least dim. of str. cross-section}} = \frac{130}{59.83} = 2.17 \Rightarrow C_f = \frac{2.17-1}{7-1}(1.4-1.3) + 1.3 = 1.32$$

step 10: wind force, F → using sect. 29.5, $F = q_z G C_f A_f$

A_f - projected area normal to the wind

N-S direction

$A_f = Bh = 199.83(18.5) = 3696.86\ ft^2$

$F = 34.43\ psf\,(0.85)(1.32)(3696.86\ ft^2) = 142.8\ k$

E-W direction

$A_f = 59.83(18.5) = 1106.86\ ft^2$

$F = 34.43(0.85)(1.32)(1106.86) = 42.8\ k$

- see excel spread sheet for calculated story forces, base shear, and overturning moment

N-S Direction				E-W Direction			
Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i	Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i
1	0	121.75	0	1	0	314.58	0
2	12.54	121.75	1526.75	2	12.54	314.58	3944.833
3	23.17	147	3405.99	3	23.17	314.58	7288.819
4	33.79	147	4967.13	4	33.79	314.58	10629.66
5	44.42	147	6529.74	5	44.42	314.58	13973.64
6	55.04	147	8090.88	6	55.04	314.58	17314.48
7	65.67	147	9653.49	7	65.67	314.58	20658.47
8	76.29	147	11214.63	8	76.29	314.58	23999.31
9	86.92	147	12777.24	9	86.92	314.58	27343.29
10	97.54	147	14338.38	10	97.54	314.58	30684.13
11	108.17	147	15900.99	11	108.17	314.58	34028.12
12	118.79	147	17462.13	12	118.79	314.58	37368.96
Main roof	130	147	19110	Main roof	130	314.58	40895.4
∑=	852.34		124977.35	∑=	852.34		268129.1
L_{eff}=	146.63			L_{eff}=	314.58		

Building Effective Length, L_{eff}, used to determine the natural frequency of the building according to Sect. 26.9 of ASCE 7-10

Velocity Pressure Coefficients, K_z, and Velocity Pressures, q_z			
Level	Elevation (ft)	K_z	q_z
1	0	0.57	16.40
2	12.54	0.57	16.40
3	23.17	0.66	18.99
4	33.79	0.76	21.87
5	44.42	0.81	23.31
6	55.04	0.85	24.46
7	65.67	0.89	25.61
8	76.29	0.93	26.76
9	86.92	0.96	27.63
10	97.54	0.99	28.49
11	108.17	1.04	29.93
12	118.79	1.04	29.93
Main Roof	130	1.09	31.37
PH Roof	148.5	1.13	34.43

Gust Factor-MWFRS				
Variable	N-S Wind		E-W Wind	
$n_1=n_a$	0.544			
$g_Q=g_v$	3.4			
g_R	4.042			
Z_{mean}	78			
$I_{z,\,mean}$	0.26			
$L_{z,\,mean}$	426.26			
$V_{z,mean}$	94.11			
N_1	2.46			
R_n	0.0788			
β	0.01			
η_h	3.46			
R_h	0.247			
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
η_B	8.36	8.36	3.24	3.91
R_B	0.112	0.112	0.261	0.22
η_L	10.84	13.09	28	28
R_L	0.088	0.073	0.035	0.035
R	0.353	0.351	0.527	0.484
Q	0.78	0.78	0.83	0.82
G_f	0.861	0.861	0.945	0.926
Gust Factor-Mechnical Penthouse				
Variable	N-S Wind		E-W Wind	
G_f	0.85		0.85	

Velocity pressures (above) and gust factors (below) for the MWFRS

Wall Pressure Coeffcients, C_p				
Description	N-S Wind		E-W Wind	
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
L/B	0.39	0.47	2.58	2.14
Windward Walls	0.8			
Side Walls	-0.7			
Leeward Walls	-0.5	-0.5	-0.271	-0.293

Force Coefficient, C_f		
Description	N-S Wind	E-W Wind
Mechnical Penthouse		
h/D	1.32	1.32

Roof Pressure Coefficients, Cp		
Description	N-S Wind	E-W wind
h/L	0.88	0.41
0 to h/2	-1.204	-0.9
h/2 to h	-0.748	-0.9
h to 2h	-0.652	-0.5
>2h	N/A	-0.3

Pressure coefficients

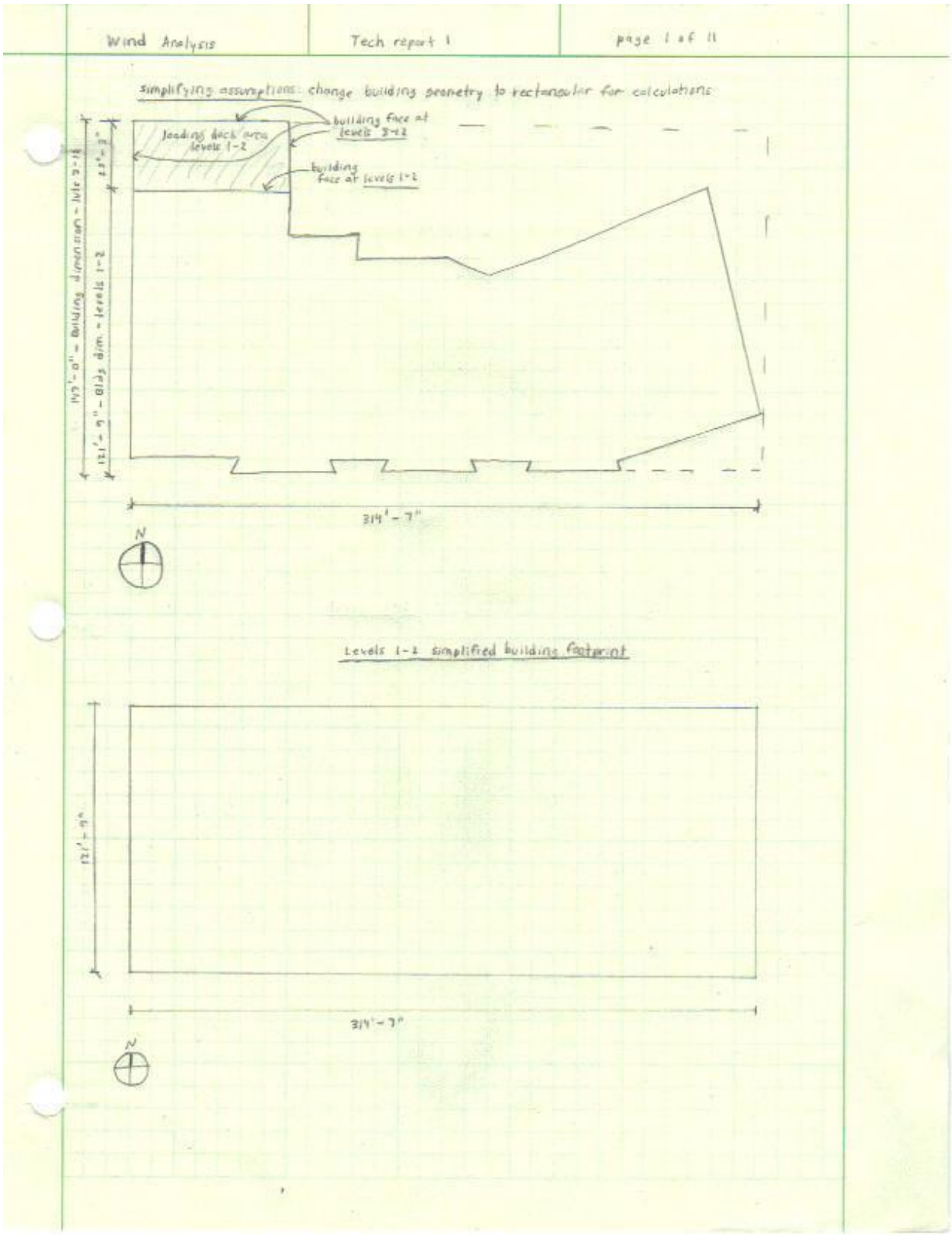
CASE 3 WIND LOAD			
Wind Forces - N-S Direction		Wind Forces - E-W Direction	
Floor	Story Force (Kips)	Floor	Story Force (Kips)
PH Roof	107.11	PH Roof	32.07
Main Roof	44.01	Main Roof	20.68
12	86.77	12	36.57
11	85.53	11	35.94
10	84.29	10	35.32
9	82.30	9	34.32
8	80.81	8	33.57
7	79.07	7	32.69
6	77.08	6	31.70
5	75.09	5	30.70
4	72.86	4	29.57
3	69.13	3	27.42
2	70.01	2	22.42
1	36.69	1	11.70

CASE 2 WIND LOAD				
Wind Forces - N-S Direction			Wind Forces - E-W Direction	
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)
PH Roof	107.11	3210.7	32.07	287.8
Main Roof	44.01	2076.5	20.68	455.9
12	86.77	4094.5	36.57	806.3
11	85.53	4035.9	35.94	792.5
10	84.29	3977.2	35.32	778.7
9	82.30	3883.4	34.32	756.7
8	80.81	3813.1	33.57	740.2
7	79.07	3731.0	32.69	720.9
6	77.08	3637.3	31.70	698.9
5	75.09	3543.5	30.70	676.9
4	72.86	3437.9	29.57	652.1
3	69.13	3262.1	27.42	604.6
2	70.01	3303.7	22.42	409.5
1	36.69	1731.3	11.70	213.7

CASE 4 WIND LOAD					
Wind Forces - N-S Direction			Wind Forces - E-W Direction		
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)	$M_{T\,N\text{-}S} + M_{T\,E\text{-}W}$ (k-ft)
PH Roof	80.41	2410.1	24.07	216.1	2626.2
Main Roof	33.03	1558.8	15.52	342.2	1901.0
12	65.14	3073.6	27.45	605.2	3678.8
11	64.20	3029.6	26.98	594.9	3624.5
10	63.27	2985.6	26.51	584.6	3570.1
9	61.78	2915.2	25.76	568.0	3483.2
8	60.66	2862.4	25.20	555.6	3418.0
7	59.35	2800.8	24.54	541.2	3341.9
6	57.86	2730.4	23.79	524.6	3255.0
5	56.37	2660.0	23.04	508.1	3168.1
4	54.69	2580.8	22.20	489.5	3070.2
3	51.89	2448.7	20.58	453.9	2902.6
2	52.56	2480.0	16.83	307.4	2787.4
1	27.54	1299.6	8.78	160.4	1460.0

Story forces for wind load cases 2 through 4

New Steel System Wind Analysis



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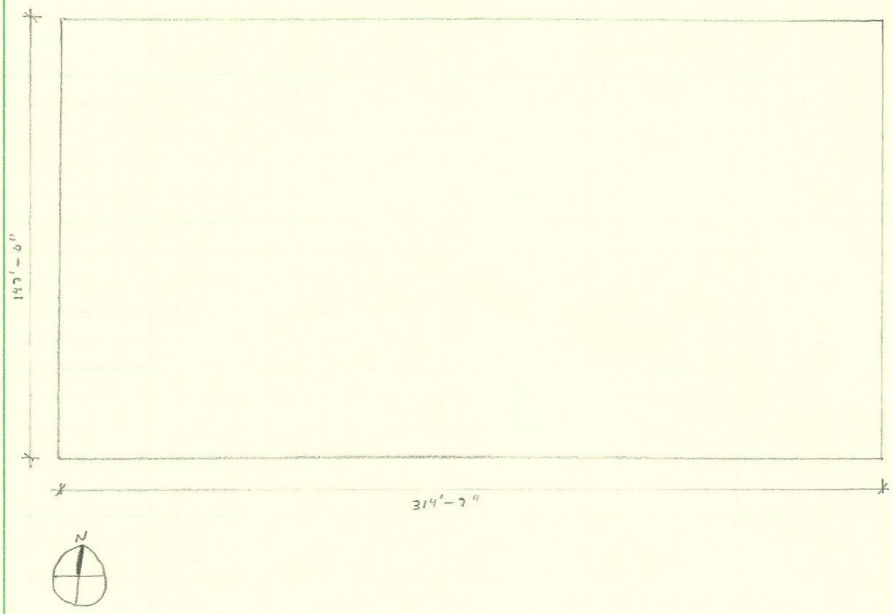
Levels 3-12 simplified building footprint



For analysis, use method 2

step 1: wind speed (V) → using Fig. 26.5-1A, V = 115 mph (location Arlington, VA)
step 2: directionality factor, K_d → using Table 26.6-1, K_d = 0.85
step 3: exposure category - B (urban, suburban)
step 4: topographic factor, K_{zt} → from sect. 26.8.2, K_{zt} = 1.0 for homogeneous topography

step 5: gust effect factor, G_f → from sect. 26.9
- since bldg ht < 300 ft and bldg ht < 4 L_{eff} in both N-S and E-W direction, use n_a to determine the natural frequency.

the approximate lower-bound natural frequency, n_a, for structural steel and concrete buildings with other lateral-force resisting systems:

$$n_1 = n_a = \frac{75}{h} = \frac{75}{180} = 0.417 \text{ Hz} < 1 \text{ Hz}$$ ∴ calculate G_f in the event the building is flexible

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

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$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3{,}600(0.417))} + \frac{0.577}{\sqrt{2\ln(3{,}600(0.417))}} = 3.825 + \frac{0.577}{3.825} = 3.976$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$

$\bar{z} = 0.6h = 0.6(180) = 108$ ft $> z_{min} = 30$ ft ok ✓

from table 26.9-1, $\bar{\alpha} = 1/9.0$, $\bar{b} = 0.45$, $c = 0.30$, $\ell = 320$ ft, $\bar{\epsilon} = 1/3.0$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{108}\right)^{1/6} = 0.246$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{108}{33}\right)^{1/3} = 475.10$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{108}{33}\right)^{1/4}\left(\frac{88}{60}\right)(115) = 102.09$$

$$N_1 = \frac{0.417(475.10)}{102.09} = 1.94$$

$$R_n = \frac{7.47(1.94)}{(1+10.3(1.94))^{5/3}} = \frac{14.49}{159.62} = 0.0908$$

damping ratio, $\beta = 1\% = 0.010$ (from sect. C26.9 in ASCE 7-10, 1% is recommended for steel and concrete buildings at serviceability levels)

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North-South Direction (levels 1-2)

$h = 180\,ft$

$L = 121.75\,ft$

$B = 314.58\,ft$

$$\eta_h = 4.6\, n_1 \frac{h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.417)(180)}{102.09} = 3.38$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{3.38} - \frac{1}{2(3.38)^2}\left(1 - e^{-2(3.38)}\right) = 0.296 - 0.0438\,(0.999) = 0.252$$

$$\eta_B = 4.6\, n_1 \frac{B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.417)(314.58)}{102.09} = 5.91$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{5.91} - \frac{1}{2(5.91)^2}\left(1 - e^{-2(5.91)}\right) = 0.1692 - 0.0143\,(1.00) = 0.155$$

$$\eta_L = 15.4\, n_1 \frac{L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.417)(121.75)}{102.09} = 7.66$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{7.66} - \frac{1}{2(7.66)^2}\left(1 - e^{-2(7.66)}\right) = 0.1305 - 0.00852\,(1.00) = 0.122$$

$$R = \sqrt{\frac{1}{0.01}(0.0908)(0.252)(0.155)(0.53 + 0.47(0.122))} = 0.456$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{314.58 + 180}{475.10}\right)^{0.63}}} = 0.78$$

$$C_{rf} = 0.925\left(\frac{1 + 1.7(0.246)\sqrt{3.4^2(0.78)^2 + 3.976^2(0.456)^2}}{1 + 1.7(3.4)(0.246)}\right)$$

$$= \boxed{0.895}$$

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East - west direction (levels 1-2)

$h = 180\,ft$

$L = 314.58\,ft$

$B = 121.25\,ft$

$\eta_h = 3.38$ (see N-S direction)

$R_h = 0.252$ (see N-S direction)

$$\eta_B = 4.6\,(0.417)\frac{(121.25)}{102.09} = 2.29$$

$$R_B = \frac{1}{2.29} - \frac{1}{2(2.29)^2}\left(1 - e^{-2(2.29)}\right) = 0.437 - 0.0953(0.9897) = 0.343$$

$$\eta_L = 15.4\,(0.417)\frac{(314.58)}{102.09} = 19.79$$

$$R_L = \frac{1}{19.79} - \frac{1}{2(19.79)^2}\left(1 - e^{-2(19.79)}\right) = 0.0505 - 0.00128 = 0.0492$$

$$R = \sqrt{\frac{1}{0.81}\left(0.0908\right)(0.252)(0.343)(0.53 + 0.47\,(0.0492))} = 0.659$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{121.25 + 180}{475.10}\right)^{0.63}}} = 0.82$$

$$C_{pf} = 0.925\left(\frac{1 + 1.7(0.246)\sqrt{3.4^2(0.82)^2 + 3.976^2(0.659)^2}}{1 + 1.7\,(3.4)(0.246)}\right) = 0.994$$

North - south direction (levels 3-12) - for C_{pf}, refer to excel spreadsheet

East - west direction (levels 3-12) - for C_{pf}, refer to excel spreadsheet

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step 6: enclosure classification - fully enclosed

step 7: internal pressure coefficient → using Table 26.11-1, $GC_{pi} = \pm 0.18$

step 8: velocity pressure exposure coefficients, K_z or K_h → using Table 27.3-1, see excel spreadsheet

step 9: velocity pressure q_z or q_h → from sect. 27.3.2, $q_z = 0.00256\, K_z K_{zt} K_d V^2$ (lb/ft²)

see excel spreadsheet for values

step 10: external pressure coefficient, C_p → using Fig. 27.4-1

wall pressure coefficients, C_p

windward wall: → all L/B values → $C_p = 0.80$

sidewall: → all L/B values → $C_p = -0.70$

Leeward wall: levels 1-2

N-S wind

$L/B = \frac{121.75}{314.58} = 0.39$, $0 < 0.39 < 1 \Rightarrow C_p = -0.50$

E-W wind

$L/B = \frac{314.58}{121.75} = 2.58$, $2 < 2.58 < 4 \Rightarrow C_p$ → interpolate based on L/B values

$$C_p = \frac{2.58-2}{4-2}(-0.2+0.3) + (-0.3)$$

$$= -0.271$$

levels 3-12

N-S wind

$L/B = \frac{147}{314.58} = 0.47$, $0 < 0.47 < 1 \Rightarrow C_p = -0.50$

E-W wind

$L/B = \frac{314.58}{147} = 2.14$, $2 < 2.14 < 4$ → interpolate based on L/B values

$$C_p = \frac{2.14-2}{4-2}(-0.2+0.3) + (-0.3)$$

$$= -0.293$$

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Roof Pressures coefficients, C_p

$\theta = 0°$

N-S wind

$h/L = \frac{180}{147} = 1.22$, $1.22 \geq 1.0$

horizontal distance from windward edge: 0 to $h/2$ -> 0' to 90' => $C_p = -1.3$

> $h/2$ -> > 90' => $C_p = -0.70$

E-W wind

$h/L = \frac{180}{314.58} = 0.57$, $0.5 < 0.57 < 1.0$

horizontal distance from windward edge: 0 to $h/2$ -> 0' to 90' => C_p -> interpolate based on h/L values

$$C_p = \frac{0.57 - 0.50}{1 - 0.5}(-1.3 + 0.9) + (-0.9)$$
$$= -0.956$$

$h/2$ to h -> 90' to 180' => $C_p = \frac{0.57 - 0.50}{1 - 0.50}(-0.7 + 0.9) + (-0.9)$
$$= -0.872$$

h to $2h$ -> 180' to 360' => $C_p = \frac{0.57 - 0.50}{1 - 0.50}(-0.7 + 0.5) + (-0.5)$
$$= -0.528$$

> $2h$ -> N/A

step 11: design wind pressures -> from sect. 27.4.2, $p = q\,G_f\,C_p - q_i\,(G_i C_{p_i})$ (lb/ft²)

windward walls -> $p = \underbrace{q_z\,G_f\,C_p}_{\text{external pressure}} - \underbrace{q_h\,(G_i C_{p_i})}_{\text{internal pressure}}$

- see excel spreadsheet for pressures

leeward walls
side walls
roof

$p = \underbrace{q_h\,G_f\,C_p}_{\text{external pressure}} - \underbrace{q_h\,(G_i C_{p_i})}_{\text{internal pressure}}$

note: internal wind pressures were neglected in calculating the design wind pressures because the internal pressures of the same magnitude will cancel and thus do not contribute towards the external wind pressures acting on the building

Wind Analysis

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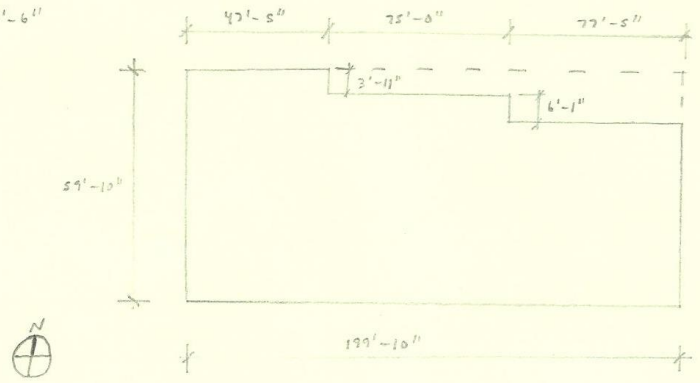
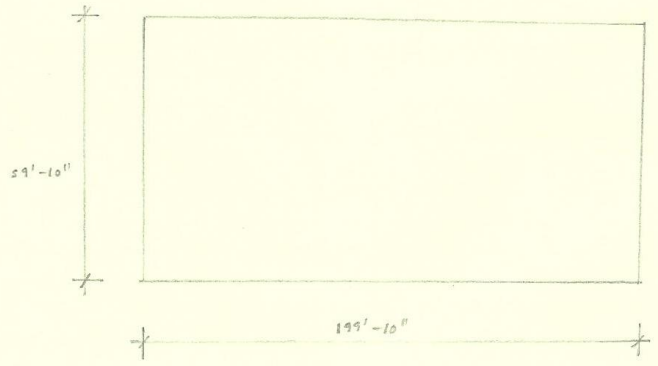
Mechanical Penthouse

Assumption: mech. Penthouse is not a part of the MWFRS
- the penthouse will be analyzed as a separate structure and therefore the guidelines listed in chapter 29 (wind loads on other structures and building appurtenances - MWFRS) will be followed for analysis

h = 18'-6"



simplified assumptions: Change penthouse geometry to rectangular for calculations



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step 1: risk category → using table 1.5-1, risk category II
step 2: V = 115 mph
step 3: k_d = 0.90
step 4: exposure category, B
step 5: k_{zt} = 1.0
step 6: Gust factor, G_f → from sect. 26.9.1, G_f = 0.85 for other structures
step 7: k_z or k_h → using table 29.3-1, k_z = 1.20 (PH roof 198'-6" above ground)

step 8: q_z → using sect. 29.3.2, $q_z = 0.00256\, k_z\, k_{zt}\, k_d\, V^2$
$= 0.00256\,(1.20)(1.0)(0.90)(115)^2$
$= 36.56$ psf

step 9: force coefficient, C_f → using Fig. 29.5-1

$$h/D = \frac{\text{ht of structure}}{\text{least dim. of sq. cross-section}} = \frac{180}{59.83} = 3.01 \Rightarrow C_f = \frac{3.01-1}{7-1}\,[1.4-1.3] + 1.3 = 1.33$$

step 10: wind force, F → using sect. 29.5, $F = q_z\, G\, C_f\, A_f$

A_f - projected area normal to the wind

N-S Direction

$A_f = Bh = 199.83\,(18.5) = 3696.86\ ft^2$

$F = 36.56$ psf $(0.85)(1.33)(3696.86\ ft^2) = 152.8$ K

E-W Direction

$A_f = 59.83\,(18.5) = 1106.86\ ft^2$

$F = 36.56\,(0.85)(1.33)(1106.86) = 45.7$ K

- see excel spreadsheet (in final report) for calculated story forces, base shears, and overturning moments for both the N-S and E-W directions

Building Effective Length, L_{eff}							
N-S Direction				E-W Direction			
Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i	Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i
1	0	121.75	0	1	0	314.58	0
2	15	121.75	1826.25	2	15	314.58	4718.7
3	30	147	4410	3	30	314.58	9437.4
4	45	147	6615	4	45	314.58	14156.1
5	60	147	8820	5	60	314.58	18874.8
6	75	147	11025	6	75	314.58	23593.5
7	90	147	13230	7	90	314.58	28312.2
8	105	147	15435	8	105	314.58	33030.9
9	120	147	17640	9	120	314.58	37749.6
10	135	147	19845	10	135	314.58	42468.3
11	150	147	22050	11	150	314.58	47187
12	165	147	24255	12	165	314.58	51905.7
Main roof	180	147	26460	Main roof	180	314.58	56624.4
∑=	1170		171611.25	∑=	1170		368058.6
L_{eff}=	146.68			L_{eff}=	314.58		

Building Effective Length, L_{eff}, used to determine the natural frequency of the building according to Sect. 26.9 of ASCE 7-10

Velocity Pressure Coefficients, K_z, and Velocity Pressures, q_z			
Level	Elevation (ft)	K_z	q_z
1	0	0.57	16.40
2	15	0.57	16.40
3	30	0.70	20.14
4	45	0.81	23.31
5	60	0.85	24.46
6	75	0.93	26.76
7	90	0.96	27.63
8	105	1.04	29.93
9	120	1.04	29.93
10	135	1.09	31.37
11	150	1.13	32.52
12	165	1.17	33.67
Main Roof	180	1.17	33.67
PH Roof	198.5	1.2	36.56

Gust Factor-MWFRS				
Variable	N-S Wind		E-W Wind	
$n_1=n_a$	0.417			
$g_Q=g_v$	3.4			
g_R	3.976			
Z_{mean}	108			
$I_{z,\,mean}$	0.246			
$L_{z,\,mean}$	475.1			
$v_{z,mean}$	102.1			
N_1	1.94			
R_n	0.0908			
β	0.01			
η_h	3.38			
R_h	0.252			
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
η_B	5.91	5.91	2.29	2.76
R_B	0.155	0.155	0.343	0.297
η_L	7.66	9.25	19.79	19.79
R_L	0.122	0.102	0.0492	0.0493
R	0.456	0.453	0.659	0.613
Q	0.78	0.78	0.82	0.817
G_f	0.895	0.894	0.994	0.972

Gust Factor-Mechnical Penthouse		
Variable	N-S Wind	E-W Wind
G_f	0.85	0.85

Velocity pressures (left) and gust factors (right) for the MWFRS

Wall Pressure Coeffcients, C_p				
Description	N-S Wind		E-W Wind	
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
L/B	0.39	0.47	2.58	2.14
Windward Walls	0.8			
Side Walls	-0.7			
Leeward Walls	-0.5	-0.5	-0.271	-0.293

Roof Pressure Coefficients, Cp		
Description	N-S Wind	E-W wind
h/L	1.22	0.57
0 to h/2	-1.3	-0.956
h/2 to h	-0.7	-0.872
h to 2h	N/A	-0.528
>2h	N/A	N/A

Force Coefficient, C_f		
Description	N-S Wind	E-W Wind
Mechnical Penthouse		
h/D	1.33	1.33

Pressure coefficients

Appendix C: Seismic Load Calculations

Existing Concrete System Seismic Load Analysis

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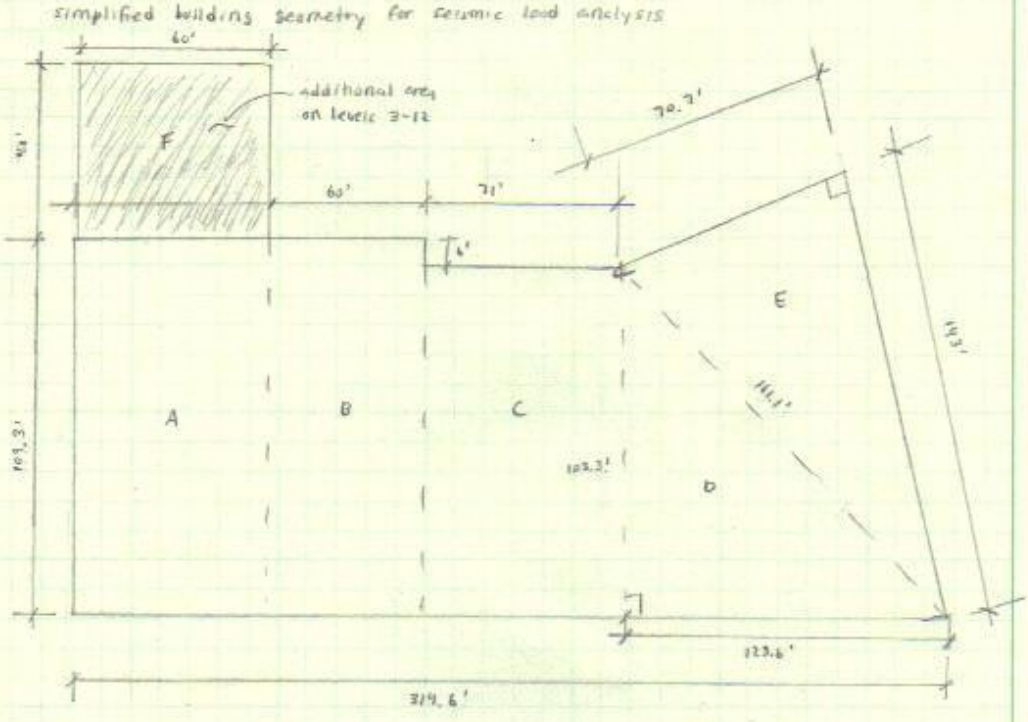
simplified building geometry for seismic load analysis



Area A = 60' (109.3') = 6558 ft²
Area B = 60' (109.3') = 6558 ft²
Area C = 71' (109.3') = 7734.3 ft²
Area D = $\frac{1}{2}$ (123.6')(103.3) = 6383.9 ft²

Area E = $\frac{1}{2}$ (143')(70.7') = 5055.1 ft²

Area F = 60(40) = 2400 ft²

building perimeter (levels 1-2) = 834.6 ft

building perimeter (levels 3-12, main roof) = 919.6 ft

floor areas for levels 1-2 = $A_A + A_B + A_C + A_D + A_E$ = 6558(2) + 7734.3 + 6383.9 + 5055.1
= 31,889 ft²

floor area for levels 3-12 = 31,889 + A_F = 31,889 + 2400 = 34,289 ft²

main roof area = 34,289 ft²

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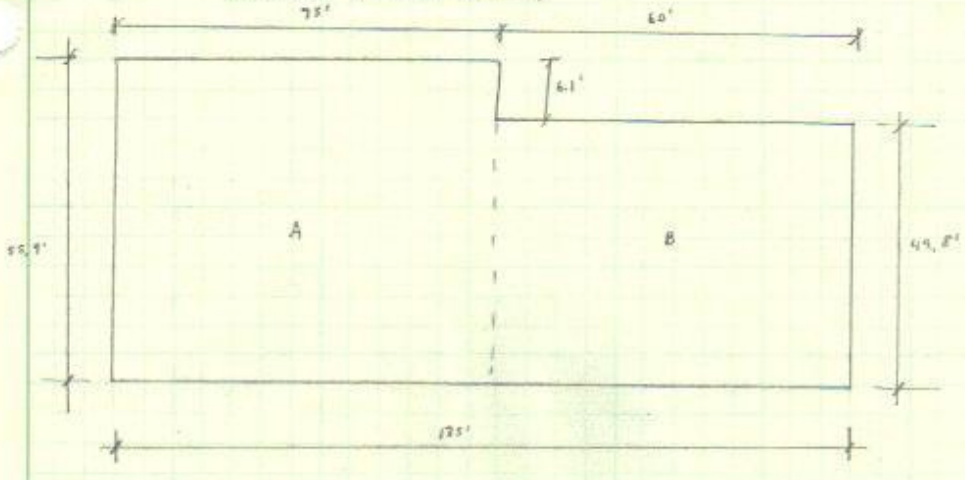
Mechanical penthouse geometry



Area A = 75'(55.9') = 4192.5 ft²

Area B = 49.8'(60') = 2988 ft²

total Mech. Plt area = $A_A + A_B$ = 4192.5 + 2988 = 7180.5

Mech. Plt perimeter = 381.8 ft

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Building weight:

- neglect 1st level weight because level 1 will not contribute towards resisting the seismic loads
- for calculation simplicity, slab openings due to stairways and elevator shafts were neglected, therefore resulting in a more conservative calculation

2nd level

Dead loads:

8" thick normal wt concrete = 150 pcf x $\frac{8}{12}$ ft = 100 psf

SDL = 10 psf

curtain wall = 250 plf

column wt = 488 k

drop panel wt = 248 k

$W_{2nd\ level}$ = (100 + 10) psf (31,284 ft²) + 250 plf (2396 ft) + 488 k + 248 k = 4452 k

levels 2 - 12

- dead loads are the same as on level 2

W = (110 psf)(34,284 ft²) + 250 plf (719.6 ft) + 488 k + 248 k = 4736 k/flr (10 flrs) = 47364 k

main roof

Dead loads:

8" thick normal wt concrete slab = 100 psf

SDL = 10 psf

curtain wall = 250 plf

$W_{main\ roof}$ = (110 psf)(34,284 ft²) + 250 plf (719.6 ft) = 4000 k

Mechanical penthouse roof

Dead loads:

8" slab = 100 psf

SDL = 5 psf

$W_{PH\ roof}$ = (105) psf (7181 ft²) = 754 k

Total building dead load = 4452 + 47364 + 4000 + 754 = 56,570 k

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step 1: site class → given in geotechnical report, "C" (very dense soil and hard rock, from table 20.3-1)

step 2: spectral response acceleration

at short periods, S_S → from Fig. 22-1, $S_S = 0.20$

at 1-second period, S_1 → from Fig. 22-2, $S_1 = 0.06$

step 3: site coefficients and adjusted maximum considered E.Q. spectral response acceleration parameters

from table 11.4-1

$S_{MS} = F_a S_S$

with $S_S < 0.25$ and site class C

$F_a = 1.2 \Rightarrow S_{MS} = 1.2(0.20) = 0.24$

from table 11.4-2

$S_{M1} = F_v S_1$

with $S_1 < 0.1$ and site class C

$F_v = 1.7 \Rightarrow S_{M1} = 1.7(0.06) = 0.102$

step 4: Design spectral response acceleration parameters at short periods, S_{DS}, and at 1-sec. period, S_{D1}, for 5% damp ⇒ from sect. 11.4.4

$S_{DS} = 2/3\ S_{MS} = 2/3\ (0.24) = 0.16$

$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.102) = 0.068$

step 5: occupancy category and importance factors

occupancy category II → from table 1.5-1

importance factor, I → from table 1.5.2, $I = 1.0$

step 6: seismic design category, SDC

SDC based on short period response acceleration parameter → from table 11.6-1

for $S_{DS} = 0.16$ and occ. II → SDC = "A"

SDC based on 1-sec. response acceleration parameter

for $S_{D1} = 0.068$ and occ. II → SDC = "B"

since risk category B is more severe than risk category A, use SDC = "B"

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step 7: response modification factor, R → from table 12.2-1

for ordinary reinforced concrete moment frames, R = 3

Equivalent Lateral Force Procedure used for analysis

step 8: approximate fundamental period, T → from sect. 12.8.2.1

$T_a = C_t h_n^x$ from table 12.8-2, "concrete moment resisting frames", $C_t = 0.016$, $x = 0.9$

$T_a = 0.016 (120)^{0.9}$
$= 1.278 s$

long transitional period → from fig. 22-12, $T_L = 6s$

step 9: seismic response coefficient, C_s → from sect. 12.8.1.1

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.16}{(3/1.0)} = 0.0533$$

$$T = 1.278 s < T_L = 6s \Rightarrow C_s < \frac{S_{D1}}{(R/I)T} = \frac{0.067}{(3/1)(1.278)} = 0.0177 \quad \text{not ok}$$

> 0.01 ✓ ok

since $C_s = 0.0533 > 0.0177$, C_s of 0.0177 controls and thus it is the value that is used for calculating the base shear V.

step 10: base shear, V

$V = C_s W = 0.0177 (56570) = 1001 k$

step 11: distribute seismic base shear, V, to story levels → from sect. 12.8.3

$F_x = C_{vx} V$

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

$$k = \frac{1.278 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.389$$

* story forces and overturning moment calculated in excel spreadsheet

Existing Superstructure Weight

Floor Weight Calculations						
Floor	Area (ft^2)	Perimeter (ft)	8" slab weight (psf)	Superimposed DL (psf)	Curtain Wall Weight (plf)	Total Weight (Kips)
2	31889.00	834.6	100	10	250.00	3716
3	34289.00	914.6	100	10	250.00	4000
4	34289.00	914.6	100	10	250.00	4000
5	34289.00	914.6	100	10	250.00	4000
6	34289.00	914.6	100	10	250.00	4000
7	34289.00	914.6	100	10	250.00	4000
8	34289.00	914.6	100	10	250.00	4000
9	34289.00	914.6	100	10	250.00	4000
10	34289.00	914.6	100	10	250.00	4000
11	34289.00	914.6	100	10	250.00	4000
12	34289.00	914.6	100	10	250.00	4000
Main Roof	34289.00	914.6	100	10	250.00	4000
PH Roof	7181.00	381.8	100	5	N/A	754
					Total Floor Weight=	48475

Typical Floor Column Weight (4th Level)					
Size (in x in)	Quantity	Length Clear Span(ft)	Unit Weight (lbs/ft^3)	Volume (ft^3)	Weight (Kips)
18X36	4	9.96	150	179.25	26.89
16X32	7	9.96	150	247.85	37.18
18X28	1	9.96	150	34.85	5.23
18X42	3	9.96	150	156.84	23.53
24X30	6	9.96	150	298.75	44.81
24X24	10	9.96	150	398.33	59.75
24 dia	12	9.96	150	375.2	56.28
16X48	2	9.96	150	106.22	15.93
16X66	1	9.96	150	73.03	10.95
14X48	2	9.96	150	92.94	13.94
12X24	5	9.96	150	99.58	14.94
22X26	1	9.96	150	39.56	5.93
20.5X24	1	9.96	150	34.02	5.10
22X24	1	9.96	150	36.51	5.48
14X96	1	9.96	150	92.94	13.94
24X36	2	9.96	150	119.50	17.93
28X28	3	9.96	150	162.65	24.40
12X48	1	9.96	150	39.83	5.98
11X24	4	9.96	150	73.03	10.95
16X96	1	9.96	150	106.22	15.93
14X66	1	9.96	150	63.90	9.58
28 dia	8	9.96	150	340.5	51.07
18X64	1	9.96	150	79.67	11.95
				Column Weight per floor (11 total flrs)=	488
				Total Column Weight=	5365
Typical Floor Drop Panel Weight (4th Level)					
Size (in x in)	Quantity	Thickness (in)	Unit Weight (lbs/ft^3)	Volume (ft^3)	Weight (Kips)
36 wide	Continuous Drop (around perimeter of all floors)	3.50	150	800.28	120
min. 68X68	40	8.00	150	856.3	128
				Drop Panel Weight per floor (11 total flrs)=	248
				Total Drop Panel Weight=	2733

New Steel System Seismic Load Analysis

Seismic Analysis

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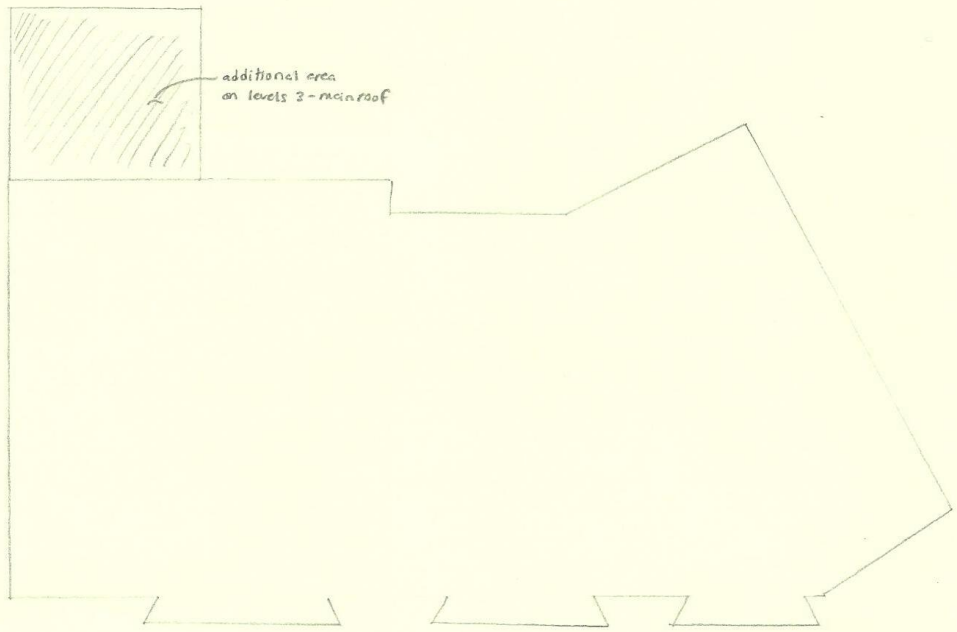


stairwell opening area = 2(10.67 ft)(30 ft) = 640 ft²

↑ # of stairwells

elevator opening area = 7(8 ft)(10 ft) = 560 ft²

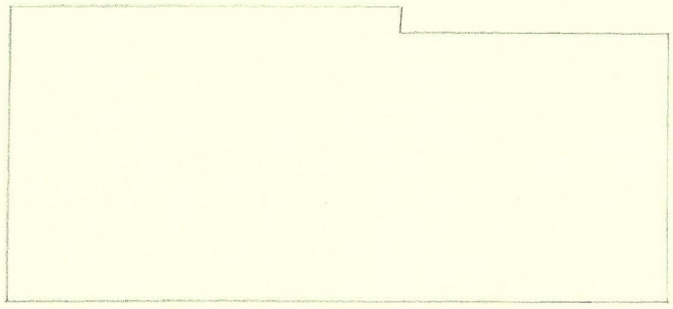
↑ # of elevators

floor area for levels 3-mainroof = 32,632 ft² − 640 ft² − 560 ft² = 31,432 ft²

building perimeter (levels 3-main roof) = 929.33 ft

floor area for levels 1-2 = 30232 ft² − 640 ft² − 560 ft² = 29,032 ft²

building perimeter = 849.3 ft



Mech. PH. area = 7180.5 ft²

Mech. PH perimeter = 381.8 ft

MECH. PH. Geometry

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Estimated
Building weight:

- neglect 1st level weight because level will not contribute towards resisting the seismic loads
- for design purposes, assume member weight (including beams, girders, and columns) will be 15 psf allowance

2nd level

dead loads: - 3VLI20 composite deck w/ 7 1/2" slab depth = 75 psf
- member wt allow. = 15 psf
- curtain wall = 250 plf
- SDL = 10 psf

$W_{2nd\ level}$ = (75 + 15 + 10) psf (29,032 ft²) + 250 plf (849.3 ft) = 3116 K

levels 3-12

- dead loads are the same as on level 2

W = (100 psf)(31,432 ft²) + 250 plf (929.33 ft) = 3376 K

W = 3376 K/flr (10 flrs) = 33760 K

main roof

- dead loads are the same as on level 2

$W_{main\ roof}$ = 3376 K

mechanical Penthouse roof

dead loads: - 3VLI20 composite deck w/ 7 1/2" slab depth = 75 psf
- SDL = 5 psf

$W_{PH\ roof}$ = (75 + 5) psf (7180.5 ft²) = 574 K

estimated total building dead load = 3116 + 33760 + 3376 + 574 = 40,826 K

- refer to excel spreadsheets for actual building weight
(after the lateral analysis, it was found that the wind load case 1 controlled the lateral system design and after designing the lateral system the actual total building wt. was calculated)

Seismic Analysis

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step 1: site class → given in geotechnical report, "C" (very dense soil and hard rock, from table 20.3-1 in ASCE 7-10)

step 2: spectral response acceleration

at short period, S_S → from Fig. 22-1, $S_S = 15\% = 0.15$

at 1-second period, S_1 → from Fig. 22-2, $S_1 = 6\% = 0.06$

step 3: site coefficients and adjusted maximum considered E.Q. spectral acceleration parameters

from table 11.4-1

$S_{MS} = F_a S_S$

with $S_S < 0.25$ and site class C

$F_a = 1.2 \Rightarrow S_{MS} = 1.2(0.15) = 0.180$

from table 11.4-2

$S_{M1} = F_V S_1$

with $S_1 < 0.1$ and site class C

$F_V = 1.7 \Rightarrow S_{M1} = 1.7(0.06) = 0.102$

step 4: design spectral response acceleration parameters at short periods, S_{DS}, and at 1-sec. period, S_{D1}, for 5% damp ⇒ from sect 11.4.4

$S_{DS} = 2/3\ S_{MS} = 2/3\ (0.180) = 0.120$

$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.102) = 0.068$

step 5: occupancy category and importance factors

occupancy category II → from table 1.5-1

importance factor, I → from table 1.5-2, $I = 1.0$

step 6: seismic design category, SDC

SDC based on short period response acceleration parameter → from table 11.6-1

for $S_{DS} = 0.12$ and occ. II - SDC = A

SDC based 1-sec response acceleration parameter → from table 11.6-2

for $S_{D1} = 0.068$ and occ. II - SDC = B

since risk category B is more severe than risk category A, use SDC = "B"

step 7: response modification factor, R → from table 12.2-1

steel systems not specifically detailed for seismic resistance, R = 3

Equivalent Lateral Force procedure used for analysis

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step 8: approximate fundamental period, T → from sect. 12.8.2.1

$T_a = C_t h_n^x$

N-S direction - resisted by both braced frames and moment frames
from table 12.8-2 "All other systems"

$C_t = 0.02$, $x = 0.75$

$T_a = 0.02 (180)^{0.75} = 0.983 s$

E-W direction
from table 12.8-2, "steel moment-resisting frames"

$C_t = 0.028$, $x = 0.80$

$T_a = 0.028 (180)^{0.80} = 1.784 s$

long transitional period → from Fig. 22-12, $T_L = 8 s$

step 9: seismic response coefficient, C_s → from sect. 12.8.1

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.12}{(3/1.0)} = 0.040$$

N-S direction

$$T = 0.983 s < T_L = 8 s \Rightarrow C_s < \frac{S_{D1}}{(R/I)T} = \frac{0.068}{(3/1)(0.983)} = 0.023 \text{ not ok}$$

> 0.01 ✓ ok

since $C_s = 0.040 > 0.023$, $C_s = 0.023$ controls and thus it is the value that is used for calculating the base shear, V, in the N-S direction.

E-W direction

$$T = 1.784 s < T_L = 8 s \Rightarrow C_s < \frac{0.068}{(3/1)(1.784)} = 0.0127$$

> 0.01 ✓ ok

since $C_s = 0.040 > 0.0127$, $C_s = 0.0127$ controls and thus it is the value that is used for calculating the base shear, V, in the E-W direction.

step 10: base shear, V

$V = C_s W$

$V_{N-S} = 0.023 (38,942 K) = 896$ kips
$V_{E-W} = 0.0127 (38,942 K) = 495$ kips

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Step 11: distribute seismic base shear, V, to story levels → sect. 12.8.3

$F_x = C_{vx} V$

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

N-S direction

$$k = \frac{0.983 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.246$$

E-W direction

$$k = \frac{1.784 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.642$$

- refer to the excel spreadsheets in the report for the base shears, story forces, and overturning moments in the N-S and E-W directions

New Steel Superstructure Weight

Gravity Beam Weight (Typical Floor- 3rd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
28'-6"	W16x26	15	28.5	26	11.12
35'-0"	W18x40	7	35	40	9.80
30'-0"	W12x19	3	30	19	1.71
	W12x19	6	4.5	19	0.51
	W12x19	2	14	19	0.53
	W16x31	10	30	31	9.30
19'-4"	W12x19	4	19.33	19	1.47
25'-0"	W14x22	9	25	22	4.95
17'-6"	W10x15	6	17.5	15	1.58
	W12x19	2	17.5	19	0.67
42'-6"	W21x44	1	36	44	1.58
	W21x44	1	31.33	44	1.38
38'-0"	W18x35	1	33	35	1.16
	W16x26	1	28.4	26	0.74
	W14x22	1	23.5	22	0.52
24'-4"	W16x26	1	24.33	26	0.63
	W12x22	2	24.33	22	1.07
	W12x16	1	24.33	16	0.39
16'-6"	W10x15	8	16.5	15	1.98
31'-0"	W14x26	1	31	26	0.81
	W16x26	7	31	26	5.64
	W16x31	7	31	31	6.73
Total Beam Weight per floor (11 typical framing levels)=					64.2
Total Beam Weight on Levels 3-Main Roof					**707**

Gravity Beam Weight (2nd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
28'-6"	W16x26	9	28.5	26	6.67
35'-0"	W18x40	7	35	40	9.80
30'-0"	W12x19	3	30	19	1.71
	W12x19	6	4.5	19	0.51
	W12x19	2	14	19	0.53
	W16x31	10	30	31	9.30
19'-4"	W12x19	4	19.33	19	1.47
25'-0"	W14x22	9	25	22	4.95
17'-6"	W10x15	6	17.5	15	1.58
	W12x19	2	17.5	19	0.67
42'-6"	W21x44	1	36	44	1.58
	W21x44	1	44	44	1.94
38'-0"	W18x35	1	33	35	1.16
	W16x26	1	28.4	26	0.74
	W14x22	1	23.5	22	0.52
24'-4"	W16x26	1	24.33	26	0.63
	W12x22	2	24.33	22	1.07
	0	0	0	0	0.00
16'-6"	W10x15	8	16.5	15	1.98
31'-0"	W14x26	1	31	26	0.81
	W16x26	6	31	26	4.84
	W16x31	7	31	31	6.73
Total Beam Weight Suppprting Level 2=					**59.2**

Gravity Girder Weight (Typical Floor- 3rd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
40'-0"	W24x76	1	40	76	3.04
	W24x55	1	40	55	2.20
28'-6"	W21x50	1	28.5	50	1.43
	W21x44	1	28.5	44	0.00
30-0"	W21x50	1	30	50	1.50
	W21x44	1	30	44	1.32
	W16x26	2	30	26	1.56
	W18x40	1	30	40	1.20
	W14x22	3	30	22	1.98
	W12x19	2	30	22	1.32
	W24x55	1	30	55	1.65
41'-0"	W27x84	5	41	84	17.22
	W24x68	4	41	68	11.15
24'-0"	W16x31	3	24	31	2.23
30'-0"	W21x44	2	30	44	2.64
35'-0"	W27x84	1	35	84	2.94
44'-0"	W24x76	1	44	76	3
Total Girder Weight per floor (11 typical framing levels)=					56.7
Total Girder Weight on Levels 3-Main Roof					**624**

Gravity Girder Weight (2nd Level)					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (Kips)
28'-6"	W21x50	1	28.5	50	1.43
	W21x44	1	28.5	44	1.25
30-0"	W21x50	1	30	50	1.50
	W21x44	1	30	44	1.32
	W16x26	2	30	26	1.56
	W18x40	1	30	40	1.20
	W14x22	3	30	22	1.98
	W12x19	2	30	22	1.32
	W24x55	1	30	55	1.65
41'-0"	W27x84	5	41	84	17.22
	W24x68	4	41	68	11.15
24'-0"	W16x31	3	24	31	2.23
30'-0"	W21x44	2	30	44	2.64
35'-0"	W27x84	1	35	84	2.94
44'-0"	W24x76	1	44	76	3
Total Girder Weight Supporting Level 2=					**52.7**

Moment Frame A.1 Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
28'-6"	W21x48	1	28.5	48	1.37
	W27x129	2	28.5	129	7.35
	W27X146	4	28.5	146	16.64
	W27X161	1	28.5	161	4.59
	W27X178	1	28.5	178	5.07
	W27X194	1	28.5	194	5.53
	W30x90	2	28.5	90	5.13
				Total Weight =	46
17'-6"	W16x26	10	17.5	26	4.55
	W27x129	8	17.5	129	18.06
	W27x146	10	17.5	146	25.55
	W27x161	8	17.5	161	22.54
	W27x178	4	17.5	178	12.46
	W27x194	15	17.5	194	50.93
	W30x90	5	17.5	90	7.88
				Total Weight =	142
25'-0"	W27x84	1	25	84	2.10
	W27x129	6	25	129	19.35
	W27x146	5	25	146	18.25
	W27x161	5	25	161	20.13
	W27x178	2	25	178	8.90
	W27x194	4	25	194	19.40
	W30x90	1	25	90	2.25
				Total Weight =	90
30'-0"	W21x62	2	30	62	3.72
	W27x84	2	30	84	5.04
	W27x129	7	30	129	27.09
	W27x146	15	30	146	65.70
	W27x161	2	30	161	9.66
	W27x178	1	30	178	5.34
	W27x194	3	30	194	17.46
	W30x99	4	30	99	11.88
				Total Weight =	146
	Total Beam Weight in Moment Frame A.1 =				**424**

Moment Frame B Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
28'-6"	W21x48	1	28.5	48	1.37
	W27x84	1	28.5	84	2.39
	W27X129	2	28.5	129	7.35
	W27X146	3	28.5	146	12.48
	W27X161	2	28.5	161	9.18
	W27x194	2	28.5	194	11.06
	W30x90	1	28.5	90	2.57
				Total Weight =	46
35'-0"	W27x84	1	35	84	2.94
	W27x129	2	35	129	9.03
	W27x146	6	35	146	30.66
	W27x194	1	35	194	6.79
	W30x90	1	35	90	3.15
	W30x99	1	35	99	3.47
				Total Weight =	56
19'-4"	W27x129	3	19.33	129	7.48
	W27x146	4	19.33	146	11.29
	W27x161	2	19.33	161	6.22
	W27x178	1	19.33	178	3.44
	W27x194	10	19.33	194	37.50
	W30x90	2	19.33	90	3.48
	W30x99	2	19.33	99	3.83
				Total Weight =	73
10'-8"	W27x84	2	10.67	84	1.79
	W27x129	3	10.67	129	4.13
	W27x146	4	10.67	146	6.23
	W27x161	2	10.67	161	3.44
	W27x194	5	10.67	194	10.35
	W30x90	1	10.67	90	0.96
	W30x99	9	10.67	99	9.51
				Total Weight =	36
42'-6"	W24x104	1	42.5	104	4
	W24x117	2	42.5	117	10
	W24x146	2	42.5	146	12
	W27x129	1	42.5	129	5
	W27x146	12	42.5	146	74
	W27x161	4	42.5	161	27
	W27x178	1	42.5	178	8
	W27x194	1	42.5	194	8
				Total Weight =	150
30'-0"	W27x84	2	30	84	5
	W27x129	2	30	129	8
	W27x146	4	30	146	18
	W27x161	1	30	161	5
	W27x178	1	30	194	6
	W27x194	1	30	194	6
	W30x90	1	30	90	3
	W30x99	1	30	99	3
				Total Weight =	52
	Total Beam Weight in Moment Frame B =				**414**

Moment Frame E Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
28'-6"	W21x55	1	28.5	55	1.57
	W27X84	1	28.5	84	2.39
	W27X129	3	28.5	129	11.03
	W27X146	5	28.5	146	20.81
	W27x161	3	28.5	161	13.77
	W27x178	3	28.5	178	15.22
	W27x194	4	28.5	194	22.12
	W30x99	2	28.5	99	5.64
	Total Beam Weight in Moment Frame E =				**93**

Moment Frame 1 Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
41'-0"	W24x104	11	41	104	46.90
	W24x131	1	131	131	17.16
				Total Weight =	64
30'-0"	W21x55	1	30	55	1.65
	W27x84	11	30	84	27.72
				Total Weight =	29
28'-6"	W27x84	12	28.5	84	28.73
				Total Weight =	29
40'-0"	W24x104	11	40	104	45.76
				Total Weight =	46
	Total Beam Weight in Moment Frame 1 =				**168**

Moment Frame 1' Beam Weight					
Bay	Size	Quantity	Span (ft)	Unit Weight (lb/ft)	Weight (kips)
35'-0"	W27X84	3	35	84	8.82
	W27x129	7	35	129	31.61
	W30x99	2	35	99	6.93
				Total Weight =	47
30'-0"	W27X84	5	30	84	12.60
	W27x129	14	30	129	54.18
	W30x90	2	30	90	5.40
	W30x99	3	30	99	8.91
				Total Weight =	81
24'-0"	W27x84	3	24	84	6.05
	W27x129	3	24	129	9.29
	W27x146	4	24	146	14.02
	W30x99	2	24	99	4.75
				Total Weight =	34
	Total Beam Weight in Moment Frame 1' =				**163**

Moment Frame Column Weight				
Size	Quantity	Span - 2 tier (ft)	Unit Weight (lbs/ft)	Weight (Kips)
W14x43	1	30.00	43	1.29
W14x193	53	30.00	193	306.87
W14x211	19	30.00	211	120.27
W14x233	23	30.00	233	160.77
W14x257	54	30.00	257	416.34
W14x283	10	30.00	283	84.90
W14x311	46	30.00	311	429.18
W14x342	6	30.00	342	61.56
W14x370	2	30.00	370	22.20
W14x500	1	30.00	500	15.00
W14x550	4	30.00	550	66.00
W14x605	2	30.00	605	36.30
W14x665	8	30.00	665	159.60
W14x730	11	30.00	730	240.90
Total Column Weight in Moment Frames =				**2121**

Bracing Weight				
Size	Quantity	Span (ft)	Unit Weight (lbs/ft)	Weight (Kips)
W6x15	5	33.54	15	2.52
W8x28	1	33.54	28	0.94
W8x31	6	33.54	31	6.24
W8x35	1	33.54	35	1.17
W10x39	2	33.54	39	2.62
W10x45	3	33.54	45	4.53
W10x49	6	35.25	49	10.36
W10x77	4	33.54	77	10.33
W12x35	1	33.54	35	1.17
W12x35	1	35.25	35	1.23
W12x53	3	33.54	53	5.33
W12x58	5	33.54	58	9.73
W12x65	3	33.54	65	6.54
W12x65	1	35.25	65	2.29
W12x72	1	33.54	72	2.41
W12x87	2	33.54	87	5.84
W12x96	2	33.54	96	6.44
W12x106	2	33.54	106	7.11
W12x120	4	33.54	120	16.10
W12x136	4	33.54	136	18.25
W12x152	2	33.54	152	10.20
W12x170	2	33.54	170	11.40
W12x190	1	33.54	190	6.37
W12x210	4	33.54	210	28.17
W12x230	1	33.54	230	7.71
W12x252	3	33.54	252	25.36
W12x279	2	33.54	279	18.72
W12x305	1	33.54	305	10.23
W12x336	23	33.54	336	259.20
W14x193	7	33.54	193	45.31
		Total Bracing Weight =		**544**

Total Superstructure Weight=	**5713**

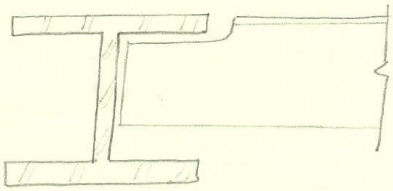
Appendix D: Typical Connections Design and Analysis

Typical Orthogonal Shear Connection

Typical orthogonal shear connection

page 1 of 6

design a typical double angle bolted/bolted connection



info: $V_u = 37.6$ K
Beam: W 16x26
Girder: W27x84
Bolts: 3/4" ϕ, A325-N
Angles: A36

limit states:

Beam:
- shear yielding (for reduced web height)
- coped beam flexural strength
- block shear
- bearing / TO

Angles:
- shear yielding
- shear rupture
- block shear
- bearing / TO

Bolts:
- bolt shear

Girder
- bearing / TO

Typical orthogonal shear connection

page 2 of 6

step 1: Design bolts

- bolt shear

using table 7-1 in steel construction manual,
$\phi r_n = 35.8$ K/bolt

$V_u = \phi R_n$ (# of bolts)

# of bolts $= \frac{37.6\ K}{35.8\ K/bolt} = 1.05 \Rightarrow$ need to use at least 2 bolts

according to table 10-1 in steel construction manual, for a double angle connection supporting a w16 using 3/4" bolts, use at least 3 rows of bolts

step 2: Design Angles

- shear yielding

$\phi R_n = \phi (0.6 F_y) A_{gv} \geq V_u$

$A_{gv} \geq \frac{37.6\ k}{1.0\,(0.6)(36\ ksi)} = 1.74\ in^2 \Rightarrow$ try 2L4x4x1/4 – 0'-8 1/2" w/ $A_{gv} = 3.64\ in^2$

- check shear rupture

$\phi R_n = \phi (0.6 F_u) A_{nv} \geq V_u$, $A_{nv} = 3.64\ in^2 - A_{holes} = 3.64\ in^2 - \underbrace{(3/4" + 1/16" + 1/16")}_{d_{hole}}(1/4")(2)(3)$

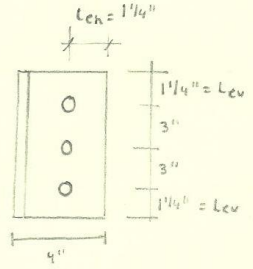
# of holes

angle thickness

# of angles

$= 2.33\ in^2$

$= 0.75\,(0.6)(58\ ksi)(2.33\ in^2)$
$= 60.8\ K > 37.6\ K$ ok



Typical orthogonal shear connection

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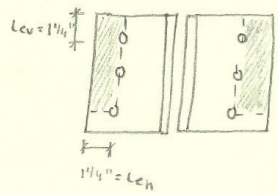
- check block shear



from table 9.3a: $\frac{\phi F_u A_{nt}}{t} = 35.3$ k/in thickness ←

from table 9.3b: $\frac{\phi 0.6 F_y A_{gv}}{t} = 117$ k/in thickness ←

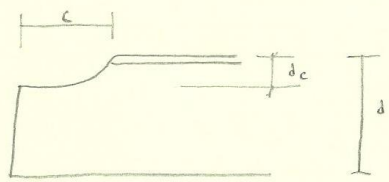
from table 9.3c: $\frac{\phi 0.6 F_u A_{nv}}{t} = 132$ k/in thickness

$\phi R_n = (35.3 + 117)\,k/in \; (1/4\ in)(2) = 76.2\ K \geq V_u$ ok

↑ angle thickness ↑ # of angles

<u>step 3</u>: check beam limit states

- design single cope



$c \leq 2d = 2(15.7\ in) = 31.4'' \Rightarrow$ use $c = 6''$

$d_c \leq d/2 = \frac{15.7}{2} = 7.85'' \Rightarrow$ use $d_c = 2''$

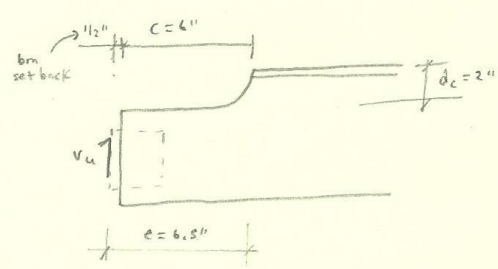
- check shear yielding

W16X26: $F_y = 50$ ksi; $F_u = 65$ ksi;
$d = 15.7$ in, $t_w = 0.25$ in

$\phi R_n = \phi(0.6F_y)\, t_w h_o = 1.0(0.6)(50)(0.25)(15.7-2) = 102.8\ K > V_u$ ok

↑ $d - d_c$

- check coped beam flexural strength



single cope:

$F_{BC} = 26{,}210\left(t_w/h_o\right)^2 f_K \leq F_y$

Typical orthogonal shear connection

page 4 of 6

$\frac{c}{d} = \frac{6}{13.7} = 0.382 < 1.0 \Rightarrow f = 2(c/d) = 2(0.382) = 0.764$

$\frac{c}{h_o} = \frac{6}{13.7} = 0.438 < 1.0 \Rightarrow K = 2.2\left(h_o/c\right)^{1.65} = 8.59$

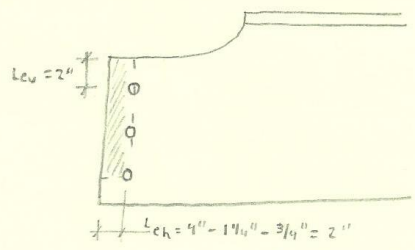
$F_{BC} = 26210\left(\frac{0.250}{13.7}\right)^2(0.764)(8.59) = 57.3\ ksi \leq F_y = 50\ ksi$

↑ controls

$\phi M_n = \phi F_y S_{net} = 0.90(50)(11.6\ in^3) = 522\ K\text{-}in$

$\phi V_n = \frac{\phi M_n}{e} = \frac{522\ K\text{-}in}{6.5\ in} = 80.3\ K > V_u$ ok

- check block shear



↑ 1/2" set back + additional 1/4" for design

from table 9.3a: $\frac{\phi F_u A_{nt}}{t} = 76.2\ k/in$ ←

from table 9.3b: $\frac{\phi 0.6 F_y A_{gv}}{t} = 180\ k/in$

from table 9.3c: $\frac{\phi 0.6 F_u A_{nv}}{t} = 170\ k/in$ ←

$\phi R_n = (76.2 + 170)\ k/in\ (0.25\ in) = 61.55\ K > V_u$ ok

Typical orthogonal shear connection

page 5 of 6

step 4: check bolt shear and bearing / TO on angle, beam, and girder

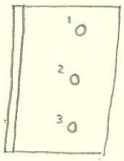
bolt shear: $\phi r_n = 35.8\ K$ ← all bolts

bearing on angles: using table 7-4

$s = 3''$

$F_u = 58\ ksi$

$d_b = 3/4''$ std



$\phi r_n = 78.3\ k/in\ (3/4\ in)(2)$

↑ # of angles

$= 39.15\ K$ ← all bolts

tear-out on angles: using table 7-5

edge dist, $L_e = 1.25''$

$F_u = 58\ ksi$

$d_b = 3/4''$

$\phi r_n = 44\ k/in\ (3/4\ in)(2) = 22\ K$ ← bolt 1

bearing on beam: using table 7-4

$s = 3''$

$F_u = 65\ ksi$

$d_b = 3/4''$ std

$\phi r_n = 87.8\ k/in\ (0.25) = 22\ K$ ← all bolts

tear-out on beam: using table 7-5

edge dist, $L_e = 2''$

$F_u = 65\ ksi$

$d_b = 3/4''$

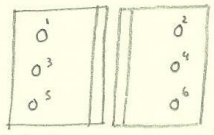
$\phi r_n = 87.8\ k/in\ (0.25) = 22\ K$ ← bolt 1

bearing on girder: using table 7-4

$s = 3''$

$F_u = 65\ ksi$

$d_b = 3/4''$



$\phi r_n = 87.8\ k/in\ (0.460\ in) = 40.4\ K$ ← all bolts

tear-out on girder: will not control by inspection

$\phi R_n = \Sigma r_{n_{i,min}} = 22K + 22K + 22K = 66\ K > V_u$ ok

Typical orthogonal shear connection

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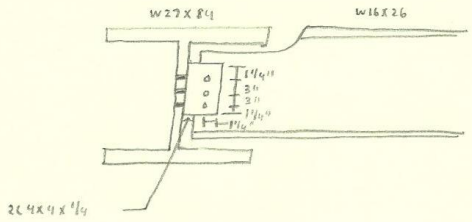
step 5: check the connection's spacing and edge distance limitations

using pg. 16.1-122 of AISC steel construction manual and table 13.4 (sect J3)

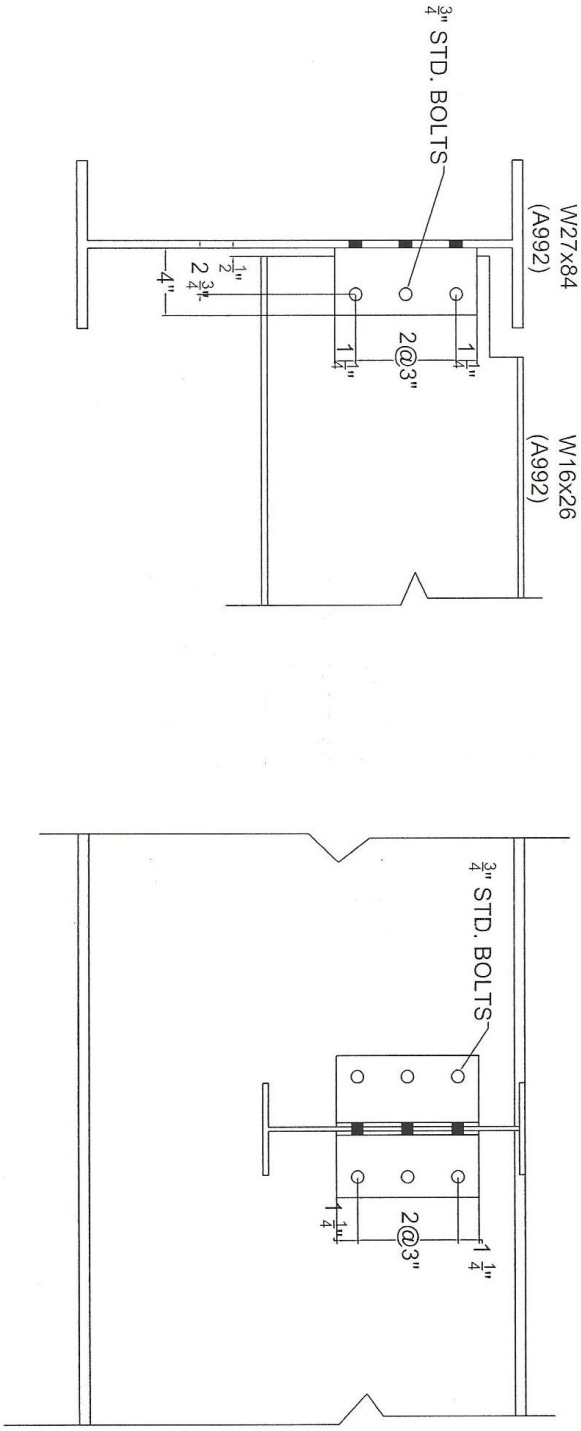
connection spacing and edge dist.	connection spacing and edge distance limitations
center-to-center spacing between bolts : 3"	min. ctr-to-ctr spacing between bolts : $3d = 3(3/4) = 2.25"$
edge dist. from ctr of std hole to edge of angles : 1.25"	min. edge dist. from std. hole to connected part : 1" for 3/4" ϕ bolts
edge dist. from ctr. of std. hole to edge of beam : 2"	max. edge dist. from ctr of std. hole to connected part : $12(0.25) = 3"$ ↑ smaller of the thicknesses of the connected parts Angle $t = 1/4\ (2) = 1/2"$ beam $t_w = 1/4"$ ← controls

The designed connection meets all spacing and edge distance limitations

Final design:



- refer to AutoCAD drawing for final detailed design



TYPICAL ORTHOGONAL SHEAR CONNECTION

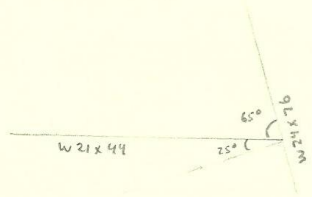
Typical Skewed Shear Connection

Typical skewed shear connection

page 1 of 6

Design a typical skewed shear connection

info: $V_u = 81.8$ k
Beam: W21x44
Girder: W24x76
plate: A36



For design, use AISC Journal "orthogonal and skewed shear connections design and detailing requirements" and chapter 10 of AISC steel manual

limit states:

Beam:
- beam gross shear yield (depth of coped beams)
- coped beam flexural strength

weld:
- check min. weld
- weld rupture
- base metal

plate:
- shear yield
- shear rupture
- block shear
- bearing / To

bolts:
- bolt shear

Girder:
- bearing / To

Typical skewed shear connection

page 2 of 6

step 1: design bolts

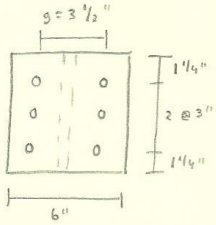
- bolt shear

using table 7-1 in steel manual

$\phi r_n = 17.9$ k/bolt

$V_u = \phi r_n$ (# of bolts)

# of bolts $= \frac{81.8k}{17.9 k/bolt} = 4.57 \Rightarrow$ use 6 bolts



step 2: design plate

- plate gross shear

$\phi V_n = \phi (0.6 F_y)(2 L_p t_p) \geq V_u$

$t_p \geq \frac{81.8k}{1.0(0.6)(36 ksi)(2)(8.5 in)} = 0.22" \Rightarrow$ use 1/4" thick plate

- shear rupture

$d_h = 3/4" + 1/16" + 1/16" = 7/8"$

$A_{net} = (8.5 - 3 \times 7/8)(t)(2) = 11.75 t_p$

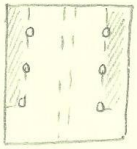
$\phi r_n = 0.75 (0.6 F_u) A_n \geq V_u$

$t_p \geq \frac{81.8k}{0.75(0.6)(58 ksi)(11.75)} = 0.27"$ ↑ controls $\Rightarrow$ use 3/8" thick plate

- check block shear

PL 6" x 3/8" x 0'-8 1/2"

$L_{eh} = 1\,1/4"$, $L_{ev} = 1\,1/4"$



from table 9.3a: $\frac{\phi F_u A_{nt}}{t} = 35.3$ k/in ←

from table 9.3b: $\frac{\phi 0.6 F_y A_{gv}}{t} = 117$ k/in ←

from table 9.3c: $\frac{\phi 0.6 F_u A_{nv}}{t} = 132$ k/in

$\phi R_n = (35.3 + 117)$ k/in $(3/8 in)(2) = 114.2k > V_u$ OK

↑ # of block shear planes

Typical skewed shear connection

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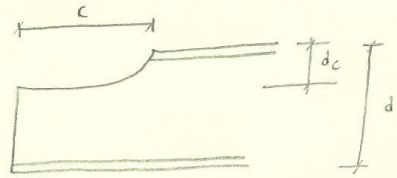
step 3: check beam limit states

- design single cope



$c \le 2d = 2(20.7) = 41.4 \Rightarrow$ use $c = 6''$

$d_c \le d/2 = \frac{20.7}{2} = 10.35'' \Rightarrow$ use $d_c = 2''$

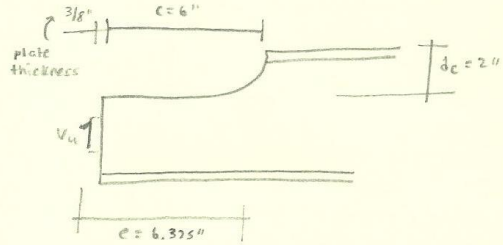
- check beam gross shear yield

W21x44: $F_y = 50$ ksi $\quad F_u = 0.65$ ksi

$d = 20.7$ in, $\quad t_w = 0.350$ in

$\phi R_n = \phi(0.6 F_y) t_w h_o = 1.0(0.6)(50)(0.350)(20.7-2) = 196.4$ K $> V_u$ ok

- check coped beam flexural strength



single cope:

$F_{BC} = 26{,}210 \left(t_w / h_o \right)^2 f k \le F_y$

$\frac{c}{d} = \frac{6}{20.7} = 0.29 < 1.0 \Rightarrow f = 2\left(c/d\right) = 2(0.29) = 0.58''$

$\frac{c}{h_o} = \frac{6}{18.7} = 0.32 < 1.0 \Rightarrow k = 2.2\left(h_o/c\right)^{1.65} = 14.36$

$F_{BC} = 26210\left(\frac{0.350}{18.7}\right)^2(0.58)(14.36) = 76.5$ ksi $\le F_y = 50$ ksi

controls

$\phi M_n = \phi F_y S_{net} = 0.90(50 \text{ ksi})(29.1 \text{ in}^3) = 1310$ k-in

$\phi V_n = \frac{\phi M_n}{e} = \frac{1310 \text{ k-in}}{6.375 \text{ in}} = 205.5$ K $> V_u$ ok

Typical skewed shear connection

page 4 of 6

Step 4: check bolt shear, bearing ⊥ to on plate, bearing ⊥ to on girder

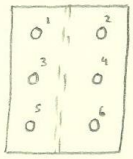
bolt shear: $\phi r_n = 17.9$ k/bolt ← all bolts

bearing on plate: using table 7-4

$s = 3''$

$F_u = 58$ ksi

$d_b = 3/4''$ std



$\phi r_n = 78.3$ k/in (3/8 in)

$= 29.4$ k / bolt ← all bolts

tear-out on plate: using table 7-5

edge dist, $L_e = 1.25''$

$F_u = 58$ ksi

$d_b = 3/4''$

$\phi r_n = 44$ k/in (3/8 in) = 16.5 k/ bolt ← bolts 1 and 2

bearing on girder: using table 7-4

$s = 3''$

$F_u = 65$ ksi

$d_b = 3/4''$

$\phi r_n = 87.8$ k/in (0.440 in) = 38.6 k/ bolt ← all bolts

tear-out on girder: will not control by inspection

$\phi R_n = \Sigma r_{n_{i,min}} = 16.5 + 16.5 + 17.9 + 17.9 + 17.9 + 17.9 = 104.6$ k $> V_u$ OK

Step 5:

- check min. weld

using table 32.4 in steel manual,
the mat. thickness of thinner part joined is the beam with $t_w = 0.35''$ ∴ min weld size = 3/16"

- beam web strength at weld

$\phi V_n = 0.75\,(0.6\,F_u)(L_p - 2\,t_{weld})\,t \geq V_u$

$$8.5 - 2\,t_{weld} \geq \frac{81.8}{0.75\,(0.6)(65)(0.35)} = 8.0$$

$$t_{weld} \leq \frac{8.0 - 8.5}{-2} = 1/4'' \implies \text{use } t_{weld} = 3/16''$$

Typical skewed shear connection

page 5 of 6

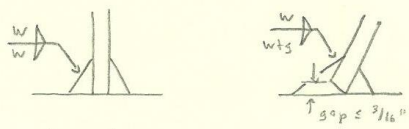
- check weld rupture

$\phi V_n = 6 \times 1.392 (L_p - 2 t_{weld})$ (# of 1/16 ths)

$= 6 \times 1.392 (8.5 - 2(3/16)) = 67.86\ k < V_u$ not good ∴ increase $t_{weld} = 1/4''$

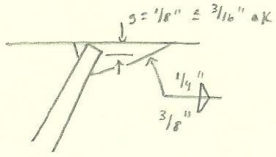
$\phi V_n = 8 \times 1.392 (8.5 - 2(1/4)) = 89.1\ k > V_u$ ok

according to "orthogonal and skewed shear connections" journal, the AISC method increases the strength on the acute side by maintaining a constant fillet size w while the increased size w+s on the obtuse side actually losses strength because of the gap g and the geometrical reduction in effective throat, due to the dihedral angle being greater than 90°.



Required orthogonal weld

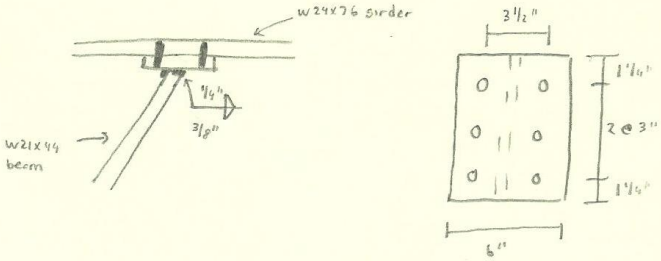
for the designed connection



$w = 1/4''$

$w + s = 1/4'' + 1/8'' = 3/8''$

Final design:



Typical skewed shear connection

page 6 of 6

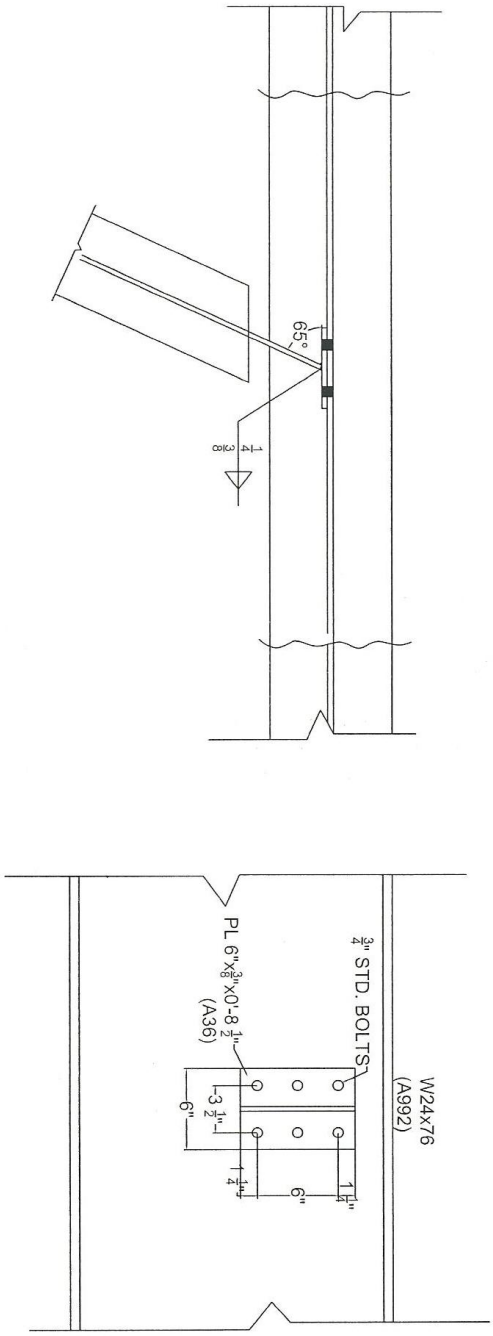
step 6: check the connection's spacing and edge distance limitations

using pg. 16.1-122 of AISC steel construction manual and table J3.4 (Sect J3)

connection spacing and edge dist.	connection spacing and edge distance limitations
ctr-to-ctr spacing between bolts : 3"	min. ctr-to-ctr spacing between bolts : $3d = 3(3/4") = 2.25"$
edge dist. from ctr of std. hole to edge of plate : 1.25"	min. edge dist. from std. hole to connected part : 1" for 3/4" ϕ bolts
	max. edge dist. from ctr of std. hole to connected part : $12(0.35) = 4.2"$ ↑ smaller of the thicknesses of the connected parts. plate $t = 0.375"$ beam $t_w = 0.35"$ ← controls

The designed connection meets all spacing and edge distance limitations

- refer to AutoCAD drawing for final detailed design



TYPICAL SKEWED SHEAR CONNECTION

Typical Moment Connection

Typical Moment connection

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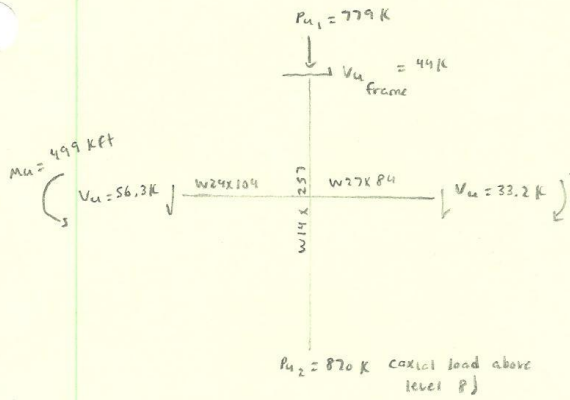
design a flange plate bolted / web bolted moment connection



controlling load combination: 1.2D + 1.0W + L

W24 X 104: $P_{u_{dead}} = 1.2(75 + 10 + 5)\,psf\,(15.75)(10)/1000$

slab, SDL, bm self wt allow.

$= 17.01$ K

$P_{u_L} = 68\,psf\,(15.75\,ft)(10\,ft)/1000$

$= 10.71$

$P_{u_{total}} = 27.7$ K

$w_D = 1.2(250 + 104)/1000 = 0.425$ KIF

curtain wall self-wt, self-wt

W27x84: $P_{u_{dead}} = 1.2(75 + 10 + 5)\,psf\,(15.75)(10)/1000$

$= 17.01$ K

$P_{u_L} = 75\,psf\,(15.75\,ft)(10\,ft)/1000$

$= 11.8$ K

$P_{u_{total}} = 17.0 + 11.8 = 28.8$ K

$w_D = 1.2(250 + 84)/1000 = 0.401$ KIF

Limit states:

- Tension Flange plate limit states
  1. tension yielding
  2. tension rupture
  3. block shear
  4. bearing / TO
- compression plate limit states
  1. buckling
  2. bolt shear / bearing / tear out
- Girder limit states
  1. reduced flexural strength
  2. beam flange block shear
- web plate
  1. shear yielding
  2. shear rupture
  3. bolt shear / bearing / tear out
  4. block shear
- weld strength
- web bolts
  1. bolt shear

Typical Moment connection

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All wide flange shapes are A992 steel.
All plates are A36

design moment connection for the W27x84

geometric properties: girder W27x84; $t_w = 0.460$ in, $d = 26.7$ in, $t_f = 0.640$ in, $b_f = 10.0$ in, $S_x = 213$ in^3

- check beam available flexural strength
  assume 2 rows of bolts in standard holes; use 7/8" bolts

$A_{fg} = b_f t_f = 10 \text{ in } (0.640 \text{ in}) = 6.40 \text{ in}^2$

$A_{fn} = A_{fg} - 2(d_b + 1/8)t_f = 6.40 - 2(7/8 + 1/8)(0.640) = 5.12 \text{ in}^2$

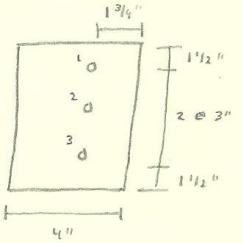
$\frac{F_y}{F_u} = \frac{50}{65} = 0.769 < 0.80 \therefore Y_t = 1.0$

$F_u A_{fn} = 65 \text{ ksi}(5.12 \text{ in}^2) = 332.8 \text{ K}$

$Y_t F_y A_{fg} = (1.0)(50 \text{ ksi})(6.40) = 320 \text{ K} < 332.8 \text{ K}$ ∴ flange rupture / web yield does not control

- design single plate web connection

from table 10-10a: Try a PL 1/4" x 4" x 0'-9 1/2", with (3) 7/8" Φ A325-N bolts and 3/16" fillet weld



- design shear strength of bolts
  single shear: from table 7-1, $\phi r_n = 24.3 \text{ K/bolt}$ ← all bolts

  $\# \text{ of bolts} = \frac{33.2 \text{ K}}{24.3 \text{ K/bolt}} = 1.36 \text{ bolts} \leq 3 \text{ bolts}$ ∴ ok

- bearing strength of bolts
  bolt spacing = 3", from table 7-4: $\phi r_n = 91.4 \text{ K/in}$

  $\phi r_n = 91.4 \text{ K/in/bolt}(1/4) = 22.9 \text{ K/bolt}$ ← all bolts

  $= \frac{33.2 \text{ K}}{22.9 \text{ K/bolt}} = 1.45 \text{ bolts} < 4 \text{ bolts}$ ∴ ok

from table 33.4, for 7/8" bolts, min. edge dist = 1 1/8" use 1 3/4" and 2 1/4" ∴ ok

- check tear-out
  $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/8)](58)(1/4) = 13.05 \text{ K/bolt}$ ← bolt 1

  $\phi R_n = 13.05 + 22.9 + 22.9 = 58.9 \text{ K} > V_u = 33.2 \text{ K}$

- plate shear yielding

  $\phi R_n = \phi 0.6 F_y A_g = 1.0(0.6)(36)(9)(1/4) = 48.6 \text{ K} > 33.2 \text{ K}$ ∴ ok

Typical moment connection

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• plate shear rupture

$\phi R_n = \phi\, 0.6\, F_u A_{nv}$

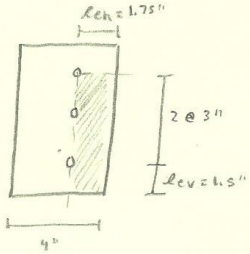
$= 0.75(0.6)(58)(1.5)$

$= 39.2\ K > 33.2\ K \therefore$ ok

$A_{hole} = (3\ \text{bolts})(7/8 + 1/8)(1/4) = 3/4\ in^2$

$A_{nv} = 7(1/4) - 3/4 = 1.5\ in^2$

• block shear rupture strength of the plate



T9.3a: $\dfrac{\phi F_u A_{nt}}{t} = 54.4\ k/in$

T9.3b: $\dfrac{\phi\, 0.6\, F_y A_{gv}}{t} = 121\ k/in \leftarrow$

T9.3c: $\dfrac{\phi\, 0.6\, F_u A_{nv}}{t} = 131\ k/in$

$\phi R_n = \frac{1}{4}(54.4 + 121) = 43.9\ K > 33.2\ K \therefore$ ok

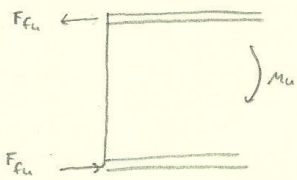
• weld strength

$\phi R_n = 1.392\, D\ell(2)$

$= 1.392\,(3\ \text{sixteenths})(7\ in)(2) = 75.2\ K > 33.2\ K \therefore$ ok

– design tension flange plate and connection

$F_{fu} = \dfrac{M_u}{d} = \dfrac{(199\ k\,ft)(12)}{26.7\ in} = 89.4\ K$



try a PL 1/2" x 8" with 7/8" ϕ A-325N bolts, $\phi r_n = 24.3\ k/bolt$

• bearing on flange

$\phi r_n = (102\ k/in)(0.640\ in) = 65.3\ K/bolt > 24.3\ K/bolt$

↑ from table 7-4

• bearing on plate

$\phi r_n = (91.4\ k/in)(0.50\ in) = 45.7\ K/bolt > 24.3\ K/bolt$

↑ from table 7-4

shear controls ∴ the number of bolts required is : $n_{min} = \dfrac{F_{fu}}{\phi r_n} = \dfrac{89.4\ K}{24.3\ K/bolt} = 3.67 \Rightarrow$ use 6 bolts

Typical moment connection

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- check flange plate tension yielding

$P_n = F_y A_g = (36)(8)(1/2) = 144\ k$

$F_{fu} = \frac{M_u}{d + t_p} = \frac{(199\ kft)(12)}{(26.7 + 1/2)in} = 87.8\ k$

$\phi P_n = 0.9(144) = 129.6\ k > F_{fu} = 87.8\ k$ ∴ ok to use 1/2" x 8" plate

- check flange plate tension rupture

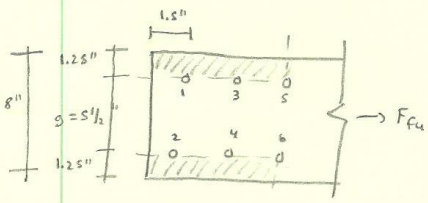
$A_n \leq 0.85 A_g = 0.85(8in)(1/2in) = 3.4\ in^2$

$A_n = [b_p - 2(d_b + 1/8)]t_p = [8 - 2(7/8 + 1/8)](1/2) = 3\ in^2$

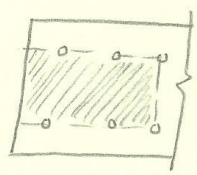
$A_e = U A_n = 1.0(3in^2) = 3\ in^2$

$\phi P_n = 0.75 F_u A_e = 0.75(58\ ksi)(3\ in^2) = 130.5\ k > 87.8\ k$ ∴ ok

- check flange plate block shear



case 1



case 2

by inspection, case 1 is more critical since it results in a smaller block shear area

$L_{ch} = 1.25"$, $L_{ev} = 1.5"$

T 9.3a: 32.6 k/in
T 9.3b: 121 k/in
T 9.3c: 131 k/in

$\phi R_n = 1/2\ in(32.6 + 121)k/in\ (2) = 153.6\ k > F_{fu} = 87.8\ k$ ∴ ok

↑ # of block shear planes

- check flange plate bolt shear / bearing / tear-out

tear-out $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/16)](58)(1/2) = 26.9\ k/bolt$ ← bolts 1 and 2

bolt shear, $\phi r_n = 24.3\ k/bolt$ ← all bolts

bearing, $\phi r_n = 91.4\ k/in/bolt\ (1/2) = 45.7\ k/bolt$ ← all bolts

$\phi R_n = 24.3(6) = 145.8\ k > 87.8\ k$ ∴ ok

Typical moment connection

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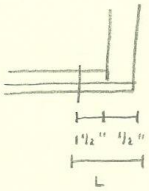
- Design compression flange plate and connection

try PL 1/2" x 8"

• flexural buckling

assume $k = 0.65$ and $L = 2''$ (1.5" edge dist. + 1/2" set back



$$\frac{kL}{r} = \frac{0.65(2)}{0.289(1/2'')} = 9 < 25 \quad \therefore F_{cr} = F_y$$

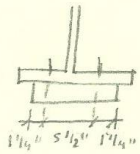
t_p

$A_g = (1/2'')(8'') = 4 in^2$

$\phi P_n = 0.9(36)(4) = 129.6 K > F_{fu} = 87.8K \therefore ok$

$\phi \cdot F_{cr} \cdot A_g$

• check local buckling



$b_f = 5\,1/2''$

$b = 1\,1/4''$

$t_p = 1/2''$

stiffened element

$$\frac{b_f}{t_p} \le \frac{253}{\sqrt{F_y}}$$

$$\frac{5.5}{1/2} \le \frac{253}{\sqrt{36}}$$

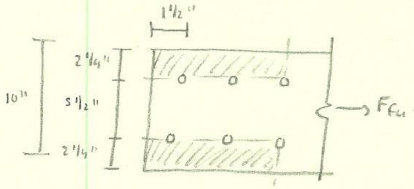
$11 \le 42.2 \quad \therefore ok$

unstiffened element

$$\frac{b}{t_p} \le \frac{95}{\sqrt{F_y}}$$

$$\frac{1.25}{1} = 1.25 \le \frac{95}{\sqrt{36}} = 15.8 \quad \therefore ok$$

- check beam flange block shear



$L_{eh} = 2\,1/4''$, $L_{ev} = 1\,1/2''$

T9.3a : 85.3 k/in

T9.3b : 186 k/in

T9.3c : 168 k/in

$\phi R_n = (85.3 + 168)\,k/in\,(0.640\,in) = 162 K > F_{fu} = 87.8K \therefore ok$

Typical Moment connection

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Design moment connection for W24x104

geometric properties: girder W24x104 ; $t_w = 0.500$ in, $d = 24.1$ in, $t_f = 0.750$ in, $b_f = 12.8$ in, $S_x = 258$ in^3

- check beam available flexural strength
  assume 2 rows of bolts in standard holes; use 7/8" bolts

$A_{fg} = b_f t_f = 12.8(0.750) = 9.6$ in^2

$A_{fn} = A_{fg} - 2(d_b + 1/8)t_f = 9.6 - 2(7/8 + 1/8)(0.750) = 5.7$ in^2

$\frac{F_y}{F_u} = \frac{50}{65} = 0.769 < 0.80 \therefore Y_t = 1.0$

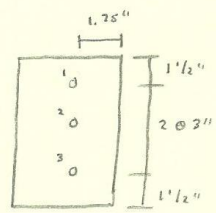
$F_u A_{fn} = 65$ ksi$(5.7$ in$^2) = 370.5$ K

$Y_t F_y A_{fg} = 1.0(50$ ksi$)(9.6$ in$^2) = 480$ K > 370.5 K $\therefore$ flange rupture / web yield will control

$M_n = \frac{F_u A_n S_x}{A_{fg}} = \frac{65(5.7)(258)}{9.6} = 9957.2$ k-in $= 830$ kft

$\phi_b M_n = 0.9(830) = 747$ kft $> M_u = 499$ kft $\therefore$ ok

- design single plate web connection
  from table 10-10a : Try PL 3/8" x 4" x 0'-9" , with (3) 7/8" ϕ A325-N bolts and 1/4" fillet weld



- design shear strength of bolts
  single shear : from table 7-1, $\phi r_n = 24.3$ k/bolt ← all bolts

  # of bolts $= \frac{56.3\text{ K}}{24.3\text{ K/bolt}} = 2.32$ bolts ≤ 3 bolts $\therefore$ ok

- bearing strength of bolts
  bolt spacing = 3", from table 7-4 : $\phi r_n = 91.4$ k/in

  $\phi r_n = 91.4$ k/in/bolt $(3/8$ in$) = 34.3$ K/bolt ← all bolts

  $= \frac{56.3\text{ K}}{34.3\text{ K/bolt}} = 1.64$ bolts < 3 bolts $\therefore$ ok

- check tear-out
  $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/8)](58)(3/8) = 19.6$ K/bolt ← bolt 1

  $\phi R_n = 19.6 + 24.3 + 24.3 = 68.2$ K $> V_u = 56.3$ K $\therefore$ ok

- plate shear yielding
  $\phi R_n = 1.0(0.60)(36)(9)(3/8) = 72.9$ K > 56.3 K $\therefore$ ok

Typical Moment Connection

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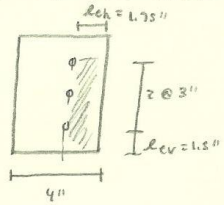
- plate shear rupture

$\phi R_n = \phi\, 0.6 F_u A_{nv}$
$= 0.75(0.6)(58)(2.25)$
$= 58.7\ K > 56.3\ K \therefore$ ok

$A_{hole} = (3\ \text{bolts})(7/8 + 1/8)(3/8) = 1.125\ in^2$
$A_{nv} = 9(3/8) - 1.125 = 2.25\ in^2$

- block shear rupture strength of the plate



T 9.3a: 54.4 k/in
T 9.3b: 121 k/in
T 9.3c: 131 k/in

$\phi R_n = 3/8\ in\,(54.4 + 121)\ k/in = 65.8\ K > 56.3\ K \therefore$ ok

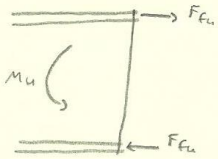
use PL 3/8" x 4" x 0'-9"

- weld strength

$\phi R_n = 1.392\,(4\ \text{sixteenths})(9\ in)(2) = 100.2\ K > 56.3\ K \therefore$ ok

- design tension flange plate and connection

$F_{fu} = \frac{M_u}{d} = \frac{(499\ k\text{-}ft)(12)}{24.1\ in} = 248.5\ K$



try a PL 1/2" x 12" with 7/8" ϕ A-325 N bolts, $\phi r_n = 24.3\ k/bolt$

- bearing on flange

$\phi r_n = (102\ k/in)(0.750\ in) = 76.5\ k/bolt > 24.3\ k/bolt$
↑ from table 7-4

- bearing on plate

$\phi r_n = (91.4\ k/in)(0.5\ in) = 45.7\ k/bolt > 24.3\ k/bolt$
↑ from table 7-4

shear controls ∴ the number of bolts is: $n_{min} = \frac{F_{fu}}{\phi r_n} = \frac{248.5\ K}{24.3\ k/bolt} = 10.22 \Rightarrow$ use 12 bolts

- check flange plate tension yielding

$P_n = F_y A_g = (36)(12)(1/2) = 216\ K$

$F_{fu} = \frac{M_u}{d + t_p} = \frac{(499\ k\text{-}ft)(12)}{(24.1 + 1/2)\ in} = 243.4\ K$

$\phi P_n = 0.9(216) = 194.4\ K < 243.4\ K \therefore$ increase plate thickness

$t_p \geq \frac{243.4}{0.9(36)(12)} = 0.63'' \Rightarrow$ try 3/4" thick plate

Typical Moment connection

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$F_{fu} = \frac{499(12)}{(24.1 + 3/4)} = 241\ K$

$\phi P_n = 0.9(36)(12)(3/4) = 291.6\ K > 241\ K$ ∴ ok to use $3/4'' \times 12''$ plate

- check flange plate tension rupture

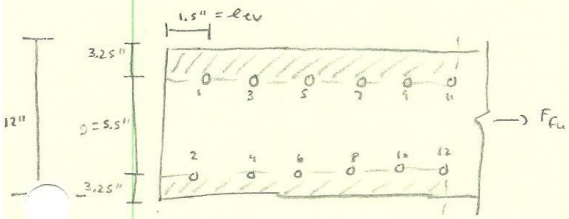
$A_n \leq 0.85 A_g = 0.85(12)(3/4) = 7.65\ in^2$ ← controls

$A_n = [12.8 - 2(7/8 + 1/8)](3/4) = 8.1\ in^2$

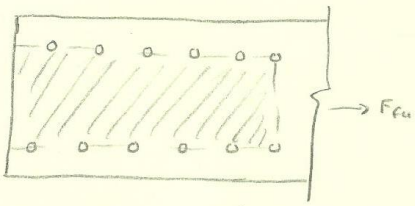
$A_e = 1.0 A_n = 1.0(7.65\ in^2) = 7.65\ in^2$

$\phi P_n = 0.75(58\ ksi)(7.65\ in^2) = 332.8\ K > 241\ K$ ∴ ok

- check flange plate block shear



case 1



case 2

by inspection, case 2 is more critical since it results in a smaller block shear area

use $l_{eh} = 2.75''$ (in tables) and $l_{ev} = 1.5''$

T 9.3a: 97.9 k/in
T 9.3b: 267 k/in
T 9.3c: 287 k/in

$\phi R_n = 3/4\ in\ (97.9 + 267)(2) = 547.9\ K > 241\ K$ ∴ ok

(2 = # of failure planes)

$A_{gv} = 2(5(3\ in) + 1.5)(3/4) = 24.75\ in^2$

$A_{nv} = 2[16.5 - 5.5(7/8 + 1/8)](3/4) = 16.5\ in^2$

$A_{nt} = (5.5 - (7/8 + 1/8))(3/4) = 3.375\ in^2$

$\phi U_{BS} F_u A_{nt} = 146.9\ K$

$\phi 0.6 F_u A_{nv} = 430.65\ K$

$\phi 0.6 F_y A_{gv} = 401\ K$

$\phi R_n = 146.9 + 401 = 547.9\ K$

- check flange plate bolt shear / bearing / tearout

tear-out: $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/16)](58)(3/4) = 40.4\ k/bolt$ ← bolts 1 and 2

bolt shear, $\phi r_n = 24.3\ k/bolt$ (from table 7-1) (all bolts)

bearing on plate, $\phi r_n = 91.4\ k/bolt(3/4) = 68.6\ k/bolt$ ← all bolts

$\phi R_n = 12(24.3\ k/bolt) = 291.6\ K > 241\ K$ ∴ ok

Typical Moment Frame

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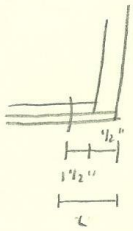
- design compression flange plate and connection

try PL 3/4" x 12"

• flexural buckling

assume K = 0.65 and L = 2' (1.5" edge dist. + 1/2" set back)



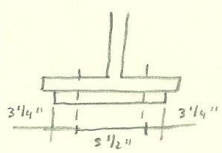
$$\frac{KL}{r} = \frac{0.65(2)}{0.289(3/4")} = 6 < 25 \therefore F_{cr} = F_y$$

t_p

$A_g = (3/4")(12") = 9 \text{ in}^2$

$\phi P_n = 0.9(36)(9) = 291.6 \text{ K} > F_{fu} = 241 \text{ K} \therefore$ ok

• check local buckling



$b_f = 5\frac{1}{2}"$

$b = 3\frac{1}{4}"$

$t_p = 3/4"$

stiffened element

$$\frac{b_f}{t_p} \le \frac{253}{\sqrt{F_y}}$$

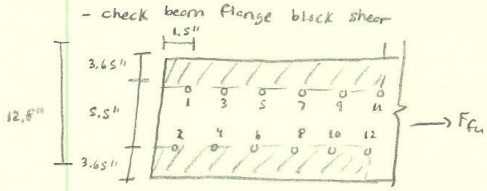
$$\frac{5.5}{3/4} = 7.33 \le \frac{253}{\sqrt{36}} = 42.2 \therefore \text{ok}$$

unstiffened element

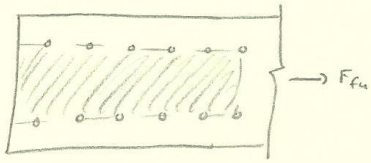
$$\frac{b}{t_p} \le \frac{95}{\sqrt{F_y}}$$

$$\frac{3\frac{1}{4}}{3/4} = 4.33 \le \frac{95}{\sqrt{36}} = 15.8 \therefore \text{ok}$$

- check beam flange block shear



case 1



case 2

by inspection, case 2 is more critical since it results in a smaller block shear area

use $\ell_{eh} = 2.75"$ and $\ell_{ev} = 1.5"$

T 9.3a: 110 K/in

T 9.3b: 371 K/in

T 9.3c: 322 K/in

$\phi R_n = 0.75(110 + 322) \text{ K/in } (2) = 648 \text{ K} > F_{fu} = 241 \text{ K} \therefore$ ok

# of shear planes

- check beam flange bolt shear / bearing / tear-out

bolt shear, $\phi r_n = 24.3$ K/bolt ← all bolts

bearing, $\phi r_n = 102$ K/in/bolt $(0.75 \text{ in}) = 76.5$ K/bolt ← all bolts

from table 7-4

tear-out, $\phi r_n = 0.75(1.2)[1.5 - 0.5(7/8 + 1/16)](65)(0.75) = 45.2$ ← bolts 1 and 2

$\phi R_n = 24.3 \text{ K/bolt } (12)$
$= 291.6 \text{ K} > 241 \text{ K} \therefore$ ok

Typical Moment Frame

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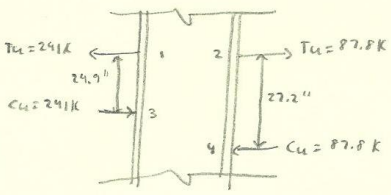
design the column side stiffener

member information: column W14X257, $d = 16.4''$, $t_w = 1.18''$, $b_f = 16''$, $t_f = 1.89''$, $k_{design} = 2.49''$

$h/t_w = 7.41''$, $A = 75.6\ in^2$



$M_{u\,left} = 499\ kft$

$M_{u\,right} = 199\ kft$

column flange bending:

$\phi R_n = 0.9\,(6.25)\,t_{fc}^2\,F_{yc} = 0.9\,(6.25)(1.89)^2(50) = 1005\,K > T_u = 241\,K$

$> T_u = 87.8\,K$

∴ do not need half depth stiffeners @ 1 and 2 for this loading condition

column web yielding

$\phi R_n = 1.0\,(5k_{des} + t_p)\,F_{yc}\,t_{wc} = 1.0\,(5(2.49) + 0.5)(50)(1.18) = 764.1\ K > T_{u_{right}}$ and $C_{u_{right}} = 87.8K$

$= 1.0\,(5(2.49) + 0.75)(50)(1.18) = 778.8\ K > T_{u_{left}}$ and $C_{u_{left}} = 241\,K$

∴ do not need half depth stiffeners @ 1, 2, 3, or 4 for this limit state

column web crippling

t_p for this problem

$$\phi R_n = 0.75\,(0.80)\,t_{wc}^2\left[1 + 3\left(\frac{\ell_b}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_{fc}}{t_{wc}}}$$

for 1/2" thick plate:

$$= 0.75(0.80)(1.18)^2\left[1 + 3\left(\frac{0.5}{16.4}\right)\left(\frac{1.18}{1.89}\right)^{1.5}\right]\sqrt{\frac{29000\,(50)(1.89)}{1.18}} = 1273\,(1.045) = 1330.3 > C_{u_{right}} = 87.8K$$

∴ do not need half depth stiffener at 4 (compression flange)

for 3/4" thick plate:

$= 1273(1.068) = 1359\ K > C_{u_{right}} = 241\ K$ ∴ do not need half depth stiffener at 3

Typical moment connection

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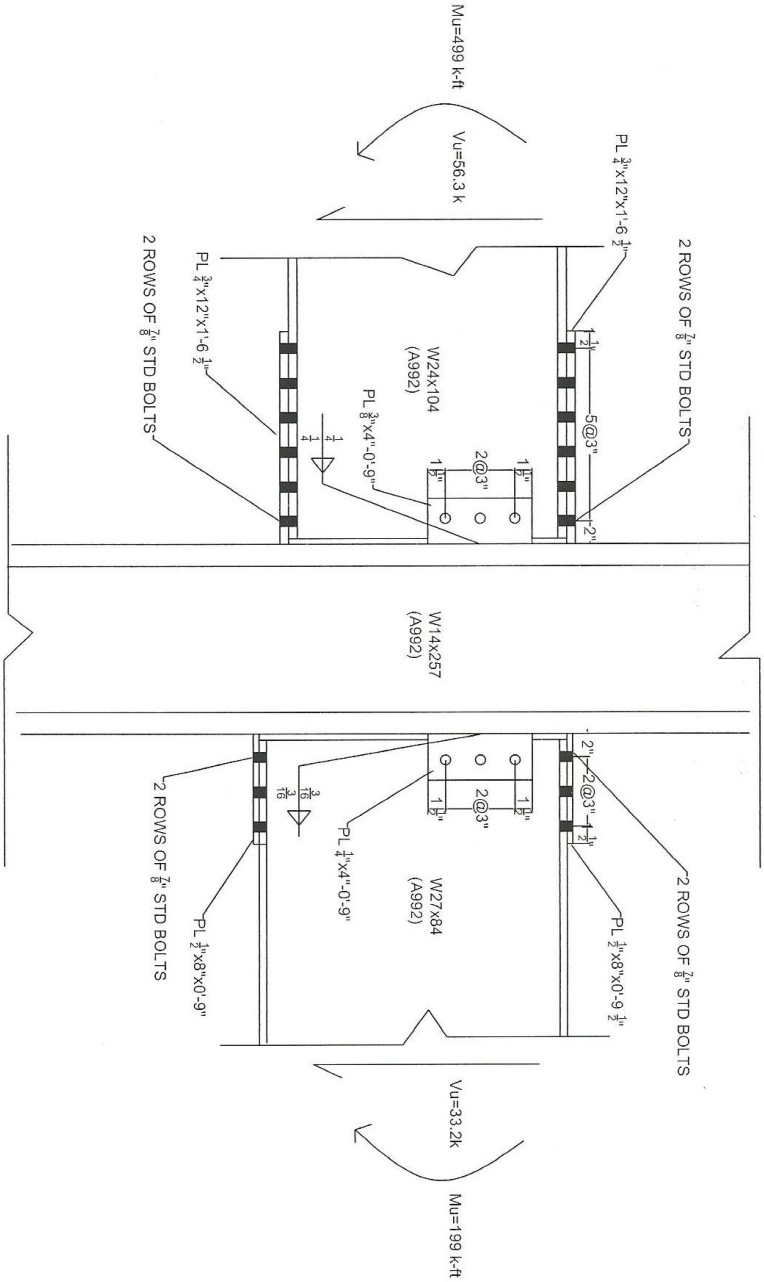
column web buckling

$$\phi R_n = \frac{0.9(24)t_{wc}\sqrt{EF_{yc}}}{h/t_{wc}} = \frac{0.9(24)(1.18)^2\sqrt{29000(50)}}{9.41} = 3848.7\ K > C_{u_{max}} = 241\ K$$

∴ do not need full depth stiffeners for this limit state

the W14x257 column subjected to the design loads do not require stiffeners

- refer to autoCAD drawing for final design



TYPICAL FLANGE PLATE BOLTED/WEB BOLTED MOMENT CONNECTION

Appendix E: Construction Management Breadth Analysis

Detailed Superstructure Cost

STRUCTURAL STEEL												
GRAVITY BEAMS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 2700	W16x26	L.F.	174	$20.50	$3,567.00	$4.50	$783.00	$2.49	$433.26	28.5	$136,323
35'-0"	05 12 23.75 3500	W18x40	L.F.	84	$55.00	$4,620.00	$4.07	$341.88	$1.69	$141.96	35	$178,634
30'-0"	05 12 23.75 1300	W12x19 *	L.F.	36	$26.13	$940.50	$3.07	$110.52	$1.70	$61.20	30	$33,367
	05 12 23.75 1300	W12x19	L.F.	72	$26.13	$1,881.36	$3.07	$221.04	$1.70	$122.40	4.5	$10,012
	05 12 23.75 1300	W12x19	L.F.	24	$26.13	$627.12	$3.07	$73.68	$1.70	$40.80	14	$10,382
	05 12 23.75 2900	W16x31	L.F.	120	$42.50	$5,100.00	$3.00	$360.00	$1.66	$199.20	30	$169,776
19'-4"	05 12 23.75 1300	W12x19	L.F.	48	$26.13	$1,254.24	$3.07	$147.36	$1.70	$81.60	19.33	$28,670
25'-0"	05 12 23.75 1300	W14x22*	L.F.	108	$30.50	$3,294.00	$3.07	$331.56	$1.70	$183.60	25	$95,229
17'-6"	05 12 23.75 0620	W10x15	L.F.	72	$20.50	$1,476.00	$4.50	$324.00	$2.49	$179.28	17.5	$34,637
	05 12 23.75 1300	W12x19	L.F.	24	$26.13	$627.12	$3.07	$73.68	$1.70	$40.80	17.5	$12,978
42'-6"	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	36	$28,382
	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	31.33	$24,701
38'-0"	05 12 23.75 3300	W18x35	L.F.	12	$48.00	$576.00	$4.07	$48.84	$1.69	$20.28	33	$21,289
	05 12 23.75 2700	W16x26	L.F.	12	$36.00	$432.00	$2.07	$24.84	$1.50	$18.00	28.4	$13,485
	05 12 23.75 1300	W14x22	L.F.	12	$30.50	$366.00	$3.07	$36.84	$1.70	$20.40	23.5	$9,946
24'-4"	05 12 23.75 2700	W16x26	L.F.	12	$36.00	$432.00	$2.07	$24.84	$1.50	$18.00	24.33	$11,553
	05 12 23.75 1300	W12x22	L.F.	24	$30.50	$732.00	$3.07	$73.68	$1.70	$40.80	24.33	$20,595
16'-6"	05 12 23.75 0620	W10x15	L.F.	96	$20.50	$1,968.00	$4.50	$432.00	$2.49	$239.04	16.5	$43,544
31'-0"	05 12 23.75 2700	W14x26	L.F.	12	$36.00	$432.00	$2.73	$32.76	$1.51	$18.12	31	$14,969
	05 12 23.75 2700	W16x26	L.F.	72	$36.00	$2,592.00	$2.07	$149.04	$1.50	$108.00	31	$88,320
	05 12 23.75 2900	W16x31	L.F.	84	$42.50	$3,570.00	$3.00	$252.00	$1.66	$139.44	31	$122,805
										Total Gravity Beam Cost		$1,109,598

GRAVITY GIRDERS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
40'-0"	05 12 23.75 5500	W24x76	L.F.	11	$105.00	$1,155.00	$3.52	$38.72	$1.46	$16.06	40	$48,391
	05 12 23.75 4900	W24x55	L.F.	11	$75.50	$830.50	$3.52	$38.72	$1.46	$16.06	40	$35,411
28'-6"	05 12 23.75 4300	W21x50	L.F.	12	$69.00	$828.00	$3.67	$44.04	$1.52	$18.24	28.5	$25,373
	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	28.5	$22,469
30'-0"	05 12 23.75 4300	W21x50	L.F.	12	$69.00	$828.00	$3.67	$44.04	$1.52	$18.24	30	$26,708
	05 12 23.75 4100	W21x44	L.F.	12	$60.50	$726.00	$3.67	$44.04	$1.53	$18.36	30	$23,652
	05 12 23.75 2700	W16x26	L.F.	24	$36.00	$864.00	$2.07	$49.68	$1.50	$36.00	30	$28,490
	05 12 23.75 3500	W18x40	L.F.	12	$55.00	$660.00	$4.07	$48.84	$1.69	$20.28	30	$21,874
	05 12 23.75 1300	W14x22*	L.F.	36	$30.50	$1,098.00	$3.07	$110.52	$1.70	$61.20	30	$38,092
	05 12 23.75 1300	W12x19 *	L.F.	24	$26.13	$627.00	$3.07	$73.68	$1.70	$40.80	30	$22,244
	05 12 23.75 4900	W24x55	L.F.	12	$75.50	$906.00	$3.52	$42.24	$1.46	$17.52	30	$28,973
41'-0"	05 12 23.75 5800	W27x84	L.F.	60	$116.00	$6,960.00	$3.28	$196.80	$1.36	$81.60	41	$296,774
	05 12 23.75 5300	W27x68	L.F.	26	$93.50	$2,431.00	$3.52	$91.52	$1.46	$37.96	41	$104,980
24'-0"	05 12 23.75 2900	W16x31	L.F.	25	$42.50	$1,062.50	$3.00	$75.00	$1.66	$41.50	24	$28,296
30'-0"	05 12 23.75 4100	W21x44	L.F.	24	$60.50	$1,452.00	$3.67	$88.08	$1.53	$36.72	30	$47,304
35'-0"	05 12 23.75 5800	W27x84	L.F.	12	$116.00	$1,392.00	$3.28	$39.36	$1.36	$16.32	35	$50,669
44'-0"	05 12 23.75 5500	W24x76	L.F.	12	$105.00	$1,260.00	$3.52	$42.24	$1.46	$17.52	44	$58,069
										Total Gravity Girder Cost		$907,770

MOMENT FRAME A.1 BEAMS/GIRDERS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 4300	W21x48*	L.F.	1	$66.00	$66.00	$3.67	$3.67	$1.52	$1.52	28.5	$2,029
	05 12 23.75 5940	W27x129*	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	28.5	$10,384
	05 12 23.75 5940	W27X146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	28.5	$23,461
	05 12 23.75 5960	W27X161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	28.5	$6,435
	05 12 23.75 5960	W27X178*	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	28.5	$7,112
	05 12 23.75 5960	W27X194*	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	28.5	$7,739
	05 12 23.75 6100	W30x90*	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	28.5	$7,316
17'-6"	05 12 23.75 2700	W16x26	L.F.	10	$36.00	$360.00	$2.70	$27.00	$1.50	$15.00	17.5	$7,035
	05 12 23.75 5940	W27x129	L.F.	8	$177.38	$1,419.00	$3.39	$27.12	$1.41	$11.28	17.5	$25,505
	05 12 23.75 5940	W27X146	L.F.	10	$201.00	$2,010.00	$3.39	$33.90	$1.41	$14.10	17.5	$36,015
	05 12 23.75 5960	W27X161	L.F.	8	$221.00	$1,768.00	$3.39	$27.12	$1.41	$11.28	17.5	$31,612
	05 12 23.75 5960	W27X178	L.F.	4	$244.75	$979.00	$3.39	$13.56	$1.41	$5.64	17.5	$17,469
	05 12 23.75 5960	W27X194	L.F.	15	$266.75	$4,001.25	$3.39	$50.85	$1.41	$21.15	17.5	$71,282
	05 12 23.75 6100	W30x90	L.F.	5	$123.75	$618.75	$3.25	$16.25	$1.35	$6.75	17.5	$11,231
25'-0"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	25	$3,016
	05 12 23.75 5940	W27x129	L.F.	6	$177.38	$1,064.25	$3.39	$20.34	$1.41	$8.46	25	$27,326
	05 12 23.75 5940	W27x146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	25	$25,725
	05 12 23.75 5960	W27x161	L.F.	5	$42.50	$212.50	$3.00	$15.00	$1.66	$8.30	25	$5,895
	05 12 23.75 5960	W27x178	L.F.	2	$244.75	$489.50	$3.39	$6.78	$1.41	$2.82	25	$12,478
	05 12 23.75 5960	W27x194	L.F.	4	$266.75	$1,067.00	$3.39	$13.56	$1.41	$5.64	25	$27,155
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	25	$3,209
30'-0"	05 12 23.75 4500	W21x62	L.F.	2	$85.50	$171.00	$3.77	$7.54	$1.56	$3.12	30	$5,450
	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	30	$7,238
	05 12 23.75 5940	W27x129	L.F.	7	$177.38	$1,241.63	$3.39	$23.73	$1.41	$9.87	30	$38,257
	05 12 23.75 5940	W27x146	L.F.	15	$201.00	$3,015.00	$3.39	$50.85	$1.41	$21.15	30	$92,610
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	30	$13,548
	05 12 23.75 5960	W27x178	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	30	$7,487
	05 12 23.75 5960	W27x194	L.F.	3	$266.75	$800.25	$3.39	$10.17	$1.41	$4.23	30	$24,440
	05 12 23.75 6100	W30x99	L.F.	4	$136.00	$544.00	$3.25	$13.00	$1.35	$5.40	30	$16,872
								Total Moment Frame A.1 Beam/Girder Cost				$575,329

MOMENT FRAME B BEAMS/GIRDERS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 4300	W21x48*	L.F.	1	$66.00	$66.00	$3.67	$3.67	$1.52	$1.52	28.5	$2,029
	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	28.5	$3,438
	05 12 23.75 5940	W27x129*	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	28.5	$10,384
	05 12 23.75 5940	W27X146	L.F.	3	$201.00	$603.00	$3.39	$10.17	$1.41	$4.23	28.5	$17,596
	05 12 23.75 5960	W27X161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	28.5	$12,871
	05 12 23.75 5960	W27X194*	L.F.	2	$266.75	$533.50	$3.39	$6.78	$1.41	$2.82	28.5	$15,478
	05 12 23.75 6100	W30x90*	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	28.5	$3,658
35'-0"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	35	$4,222
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	35	$12,752
	05 12 23.75 5940	W27x146	L.F.	6	$201.00	$1,206.00	$3.39	$20.34	$1.41	$8.46	35	$43,218
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	35	$9,504
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	35	$4,492
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.39	$3.39	$1.35	$1.35	35	$4,926
19'-4"	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	19.33	$10,564
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	19.33	$15,912
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	19.33	$8,729
	05 12 23.75 5960	W27X178*	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	19.33	$4,824
	05 12 23.75 5960	W27x194	L.F.	10	$266.75	$2,667.50	$3.39	$33.90	$1.41	$14.10	19.33	$52,491
	05 12 23.75 6100	W30x90	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	19.33	$4,962
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.39	$6.78	$1.35	$2.70	19.33	$5,441
10'-8"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	10.67	$2,574
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	10.67	$5,831
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	10.67	$8,784
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	10.67	$4,819
	05 12 23.75 5960	W27x194	L.F.	5	$266.75	$1,333.75	$3.39	$16.95	$1.41	$7.05	10.67	$14,487
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	10.67	$1,369
	05 12 23.75 6100	W30x99	L.F.	9	$136.00	$1,224.00	$3.39	$30.51	$1.35	$12.15	10.67	$13,515
42'-6"	05 12 23.75 5740	W24x104	L.F.	1	$143.00	$143.00	$3.72	$3.72	$1.54	$1.54	42.5	$6,301
	05 12 23.75 5760	W24x117	L.F.	2	$161.00	$322.00	$3.72	$7.44	$1.54	$3.08	42.5	$14,132
	05 12 23.75 5780	W24x146	L.F.	2	$201.00	$402.00	$3.72	$7.44	$1.54	$3.08	42.5	$17,532
	05 12 23.75 5940	W27x129	L.F.	1	$177.38	$177.38	$3.39	$3.39	$1.41	$1.41	42.5	$7,742
	05 12 23.75 5940	W27x146	L.F.	12	$201.00	$2,412.00	$3.39	$40.68	$1.41	$16.92	42.5	$104,958
	05 12 23.75 5960	W27x161	L.F.	4	$221.00	$884.00	$3.39	$13.56	$1.41	$5.64	42.5	$38,386
	05 12 23.75 5960	W27x178	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	42.5	$10,606
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	42.5	$11,541
30'-0"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	30	$7,238
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	30	$10,931
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	30	$24,696
	05 12 23.75 5960	W27x161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	30	$6,774
	05 12 23.75 5960	W27x178	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	30	$7,487
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	30	$8,147
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	30	$3,851
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.39	$3.39	$1.35	$1.35	30	$4,222
								Total Moment Frame B Beam/Girder Cost				$583,416

MOMENT FRAME C BEAMS/GIRDERS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 4300	W21x48*	L.F.	1	$66.00	$66.00	$3.67	$3.67	$1.52	$1.52	28.5	$2,029
	05 12 23.75 5940	W27x129*	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	28.5	$10,384
	05 12 23.75 5940	W27X146	L.F.	3	$201.00	$603.00	$3.39	$10.17	$1.41	$4.23	28.5	$17,596
	05 12 23.75 5960	W27X161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	28.5	$6,435
	05 12 23.75 5960	W27X178*	L.F.	1	$244.75	$244.75	$3.39	$3.39	$1.41	$1.41	28.5	$7,112
	05 12 23.75 5960	W27X194*	L.F.	2	$266.75	$533.50	$3.39	$6.78	$1.41	$2.82	28.5	$15,478
	05 12 23.75 6100	W30x90*	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	28.5	$3,658
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.25	$3.25	$1.35	$1.35	28.5	$4,007
35'-0"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	35	$4,222
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	35	$19,128
	05 12 23.75 5940	W27x146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	35	$36,015
	05 12 23.75 5960	W27x161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	35	$7,903
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	35	$9,504
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	35	$4,492
19'-4"	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	19.33	$2,332
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	19.33	$10,564
	05 12 23.75 5940	W27x146	L.F.	6	$201.00	$1,206.00	$3.39	$20.34	$1.41	$8.46	19.33	$23,869
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	19.33	$8,729
	05 12 23.75 5960	W27X178	L.F.	2	$244.75	$489.50	$3.39	$6.78	$1.41	$2.82	19.33	$9,648
	05 12 23.75 5960	W27x194	L.F.	7	$266.75	$1,867.25	$3.39	$23.73	$1.41	$9.87	19.33	$36,743
	05 12 23.75 6100	W30x90	L.F.	1	$123.75	$123.75	$3.25	$3.25	$1.35	$1.35	19.33	$2,481
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.39	$6.78	$1.35	$2.70	19.33	$5,441
10'-8"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	10.67	$2,574
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	10.67	$3,888
	05 12 23.75 5940	W27x146	L.F.	3	$201.00	$603.00	$3.39	$10.17	$1.41	$4.23	10.67	$6,588
	05 12 23.75 5960	W27x161	L.F.	1	$221.00	$221.00	$3.39	$3.39	$1.41	$1.41	10.67	$2,409
	05 12 23.75 5960	W27x194	L.F.	12	$266.75	$3,201.00	$3.39	$40.68	$1.41	$16.92	10.67	$34,769
	05 12 23.75 6100	W30x90	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	10.67	$2,739
	05 12 23.75 6100	W30x99	L.F.	4	$136.00	$544.00	$3.39	$13.56	$1.35	$5.40	10.67	$6,007
42'-6"	05 12 23.75 5760	W24x117	L.F.	2	$161.00	$322.00	$3.72	$7.44	$1.54	$3.08	42.5	$14,132
	05 12 23.75 5780	W24x131*	L.F.	1	$180.13	$180.13	$3.72	$3.72	$1.54	$1.54	42.5	$7,879
	05 12 23.75 5780	W24x146	L.F.	3	$201.00	$603.00	$3.72	$11.16	$1.54	$4.62	42.5	$26,298
	05 12 23.75 5940	W27x146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	42.5	$43,733
	05 12 23.75 5960	W27x194	L.F.	1	$266.75	$266.75	$3.39	$3.39	$1.41	$1.41	42.5	$11,541
30'-0"	05 12 23.75 5800	W27x84	L.F.	2	$116.00	$232.00	$3.28	$6.56	$1.36	$2.72	30	$7,238
	05 12 23.75 5940	W27x129	L.F.	2	$177.38	$354.75	$3.39	$6.78	$1.41	$2.82	30	$10,931
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	30	$24,696
	05 12 23.75 5960	W27x161	L.F.	2	$221.00	$442.00	$3.39	$6.78	$1.41	$2.82	30	$13,548
	05 12 23.75 5960	W27x194	L.F.	2	$266.75	$533.50	$3.39	$6.78	$1.41	$2.82	30	$16,293
	05 12 23.75 6100	W30x99	L.F.	1	$136.00	$136.00	$3.39	$3.39	$1.35	$1.35	30	$4,222
								Total Moment Frame C Beam/Girder Cost				$487,257

MOMENT FRAME E BEAMS/GIRDERS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
28'-6"	05 12 23.75 3900	W21x55*	L.F.	1	$75.50	$75.50	$4.28	$4.28	$1.77	$1.77	28.5	$2,324
	05 12 23.75 5800	W27x84	L.F.	1	$116.00	$116.00	$3.28	$3.28	$1.36	$1.36	28.5	$3,438
	05 12 23.75 5940	W27x129*	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	28.5	$15,576
	05 12 23.75 5940	W27X146	L.F.	5	$201.00	$1,005.00	$3.39	$16.95	$1.41	$7.05	28.5	$29,327
	05 12 23.75 5960	W27X161	L.F.	3	$221.00	$663.00	$3.39	$10.17	$1.41	$4.23	28.5	$19,306
	05 12 23.75 5960	W27X178*	L.F.	3	$244.75	$734.25	$3.39	$10.17	$1.41	$4.23	28.5	$21,337
	05 12 23.75 5960	W27X194*	L.F.	4	$266.75	$1,067.00	$3.39	$13.56	$1.41	$5.64	28.5	$30,957
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.25	$6.50	$1.35	$2.70	28.5	$8,014
								Total Moment Frame E Beam/Girder Cost				$130,278

MOMENT FRAME 1 BEAMS/GIRDERS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
41'-0"	05 12 23.75 5740	W24x104	L.F.	11	$143.00	$1,573.00	$3.72	$40.92	$1.54	$16.94	41	$66,865
	05 12 23.75 5780	W24x131*	L.F.	1	$180.13	$180.13	$3.72	$3.72	$1.54	$1.54	41	$7,601
30'-0"	05 12 23.75 3900	W21x55*	L.F.	1	$75.50	$75.50	$4.28	$4.28	$1.77	$1.77	30	$2,447
	05 12 23.75 5800	W27x84	L.F.	11	$116.00	$1,276.00	$3.28	$36.08	$1.36	$14.96	30	$39,811
28'-6"	05 12 23.75 5800	W27x84	L.F.	12	$116.00	$1,392.00	$3.28	$39.36	$1.36	$16.32	28.5	$41,259
40'-0"	05 12 23.75 5740	W24x104	L.F.	11	$143.00	$1,573.00	$3.72	$40.92	$1.54	$16.94	40	$65,234
										Total Gravity Girder Cost		$223,217

MOMENT FRAME 1' BEAMS/GIRDERS												
Bay	Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
35'-0"	05 12 23.75 5800	W27x84	L.F.	3	$116.00	$348.00	$3.28	$9.84	$1.36	$4.08	35	$12,667
	05 12 23.75 5940	W27x129*	L.F.	7	$177.38	$1,241.63	$3.39	$23.73	$1.41	$9.87	35	$44,633
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.39	$6.78	$1.35	$2.70	35	$9,852
30'-0"	05 12 23.75 5800	W27x84	L.F.	5	$116.00	$580.00	$3.28	$16.40	$1.36	$6.80	30	$18,096
	05 12 23.75 5940	W27x129	L.F.	14	$177.38	$2,483.25	$3.39	$47.46	$1.41	$19.74	30	$76,514
	05 12 23.75 6100	W30x90*	L.F.	2	$123.75	$247.50	$3.25	$6.50	$1.35	$2.70	30	$7,701
	05 12 23.75 6100	W30x99	L.F.	3	$136.00	$408.00	$3.25	$9.75	$1.35	$4.05	30	$12,654
24'-0"	05 12 23.75 5800	W27x84	L.F.	3	$116.00	$348.00	$3.28	$9.84	$1.36	$4.08	24	$8,686
	05 12 23.75 5940	W27x129	L.F.	3	$177.38	$532.13	$3.39	$10.17	$1.41	$4.23	24	$13,117
	05 12 23.75 5940	W27x146	L.F.	4	$201.00	$804.00	$3.39	$13.56	$1.41	$5.64	24	$19,757
	05 12 23.75 6100	W30x99	L.F.	2	$136.00	$272.00	$3.25	$6.50	$1.35	$2.70	24	$6,749
										Total Gravity Girder Cost		$230,425

GRAVITY COLUMNS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
05 12 23.75 2320	W14x43	L.F.	23	$59.00	$1,357.00	$3.34	$76.82	$1.85	$42.55	30	$44,291
05 12 23.17 7350	W14x48*	L.F.	4	$66.00	$264.00	$2.75	$11.00	$1.52	$6.08	30	$8,432
05 12 23.75 2340	W14x53	L.F.	5	$73.00	$365.00	$3.38	$16.90	$1.87	$9.35	30	$11,738
05 12 23.17 7350	W14x61*	L.F.	16	$83.88	$1,342.00	$2.75	$44.00	$1.52	$24.32	30	$42,310
05 12 23.17 7350	W14x68*	L.F.	6	$93.50	$561.00	$2.75	$16.50	$1.52	$9.12	30	$17,599
05 12 23.17 7350	W14x74	L.F.	2	$102.00	$204.00	$2.75	$5.50	$1.52	$3.04	30	$6,376
05 12 23.17 7400	W14x82*	L.F.	5	$112.75	$563.75	$2.82	$14.10	$1.56	$7.80	30	$17,570
05 12 23.75 2380	W14x90	L.F.	8	$124.00	$992.00	$3.65	$29.20	$2.02	$16.16	30	$31,121
05 12 23.75 6100	W14x99	L.F.	4	$136.00	$544.00	$3.25	$13.00	$1.35	$5.40	30	$16,872
05 12 23.17 7400	W14x109*	L.F.	4	$149.88	$599.50	$2.82	$11.28	$1.56	$6.24	30	$18,511
05 12 23.17 7400	W14x120	L.F.	2	$165.00	$330.00	$2.82	$5.64	$1.56	$3.12	30	$10,163
05 12 23.17 7450	W14x132*	L.F.	4	$181.50	$726.00	$2.96	$11.84	$1.64	$6.56	30	$22,332
05 12 23.17 7450	W14x145*	L.F.	3	$199.38	$598.13	$2.96	$8.88	$1.64	$4.92	30	$18,358
05 12 23.17 7450	W14x159*	L.F.	2	$218.63	$437.25	$2.96	$5.92	$1.64	$3.28	30	$13,394
05 12 23.17 7450	W14x193*	L.F.	1	$265.38	$265.38	$2.96	$2.96	$1.64	$1.64	30	$8,099
									Total Gravity Column Cost		$287,164

MOMENT FRAME COLUMNS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
05 12 23.17 7350	W14x43*	L.F.	1	$59.13	$59.13	$2.75	$2.75	$1.52	$1.52	30	$1,902
05 12 23.17 7450	W14x193*	L.F.	53	$265.38	$14,065	$2.96	$156.88	$1.64	$86.92	30	$429,260
05 12 23.17 7450	W14x211*	L.F.	19	$290.13	$5,512.38	$2.96	$56.24	$1.64	$31.16	30	$167,993
05 12 23.17 7450	W14x233*	L.F.	23	$320.38	$7,368.63	$2.96	$68.08	$1.64	$37.72	30	$224,233
05 12 23.17 7450	W14x257*	L.F.	54	$353.38	$19,082	$2.96	$159.84	$1.64	$88.56	30	$579,920
05 12 23.17 7450	W14x283*	L.F.	10	$389.13	$3,891.25	$2.96	$29.60	$1.64	$16.40	30	$118,118
05 12 23.17 7450	W14x311*	L.F.	46	$427.63	$19,671	$2.96	$136.16	$1.64	$75.44	30	$596,471
05 12 23.17 7450	W14x342*	L.F.	6	$470.25	$2,821.50	$2.96	$17.76	$1.64	$9.84	30	$85,473
05 12 23.17 7450	W14x370*	L.F.	2	$508.75	$1,017.50	$2.96	$5.92	$1.64	$3.28	30	$30,801
05 12 23.17 7450	W14x500*	L.F.	1	$687.50	$687.50	$2.96	$2.96	$1.64	$1.64	30	$20,763
05 12 23.17 7450	W14x550*	L.F.	4	$756.25	$3,025.00	$2.96	$11.84	$1.64	$6.56	30	$91,302
05 12 23.17 7450	W14x605*	L.F.	2	$831.88	$1,663.75	$2.96	$5.92	$1.64	$3.28	30	$50,189
05 12 23.17 7450	W14x665*	L.F.	8	$914.38	$7,315.00	$2.96	$23.68	$1.64	$13.12	30	$220,554
05 12 23.17 7450	W14x730*	L.F.	11	$1,003.75	$11,041	$2.96	$32.56	$1.64	$18.04	30	$332,756
								Total Moment Frame Column Cost			$2,350,577

BRACES											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Span (ft)	Total
05 12 23.75 0120	W6x15	L.F.	5	$20.50	$102.50	$4.50	$22.50	$2.49	$12.45	33.54	$4,610
05 12 23.75 0370	W8x28	L.F.	1	$38.50	$39	$4.91	$4.91	$2.72	$2.72	33.54	$1,547
05 12 23.75 0500	W8x31	L.F.	6	$42.50	$255.00	$4.91	$29.46	$2.72	$16.32	33.54	$10,088
05 12 23.75 0520	W8x35	L.F.	1	$48.00	$48.00	$4.91	$4.91	$2.72	$2.72	33.54	$1,866
05 12 23.75 0900	W10x39*	L.F.	2	$53.63	$107	$4.91	$9.82	$2.72	$5.44	33.54	$4,109
05 12 23.75 0900	W10x45*	L.F.	3	$61.88	$185.63	$4.91	$14.73	$2.72	$8.16	33.54	$6,994
05 12 23.75 0900	W10x49	L.F.	6	$67.50	$405	$4.91	$29.46	$2.72	$16.32	35.25	$15,890
05 12 23.75 0900	W10x77*	L.F.	4	$105.88	$423.50	$4.91	$19.64	$2.72	$10.88	33.54	$15,228
05 12 23.75 1520	W12x35	L.F.	1	$48.00	$48.00	$3.34	$3.34	$1.85	$1.85	33.54	$1,784
05 12 23.75 1520	W12x35	L.F.	1	$48.00	$48.00	$3.34	$3.34	$1.85	$1.85	35.25	$1,875
05 12 23.75 2340	W12x53	L.F.	3	$73.00	$219.00	$3.38	$10.14	$1.87	$5.61	33.54	$7,874
05 12 23.75 1580	W12x58	L.F.	5	$80.00	$400.00	$3.60	$18.00	$2.00	$10.00	33.54	$14,355
05 12 23.75 3920	W12x65	L.F.	3	$89.50	$268.50	$4.34	$13.02	$1.80	$5.40	33.54	$9,623
05 12 23.75 3920	W12x65	L.F.	1	$89.50	$90	$4.34	$4.34	$1.80	$1.80	35.25	$3,371
05 12 23.75 1700	W12x72	L.F.	1	$99.00	$99.00	$4.22	$4.22	$2.34	$2.34	33.54	$3,540
05 12 23.75 1740	W12x87	L.F.	2	$120.00	$240.00	$4.22	$8.44	$2.34	$4.68	33.54	$8,490
05 12 23.75 1740	W12x96*	L.F.	2	$132.00	$264.00	$4.22	$8.44	$2.34	$4.68	33.54	$9,295
05 12 23.75 3980	W12x106	L.F.	2	$60.50	$121.00	$3.67	$7.34	$1.52	$3.04	33.54	$4,406
05 12 23.75 2500	W12x120	L.F.	4	$165.00	$660.00	$3.75	$15.00	$2.08	$8.32	33.54	$22,919
05 12 23.75 1740	W12x136*	L.F.	4	$187.00	$748.00	$4.22	$16.88	$2.34	$9.36	33.54	$25,968
05 12 23.75 1740	W12x152*	L.F.	2	$209.00	$418.00	$4.22	$8.44	$2.34	$4.68	33.54	$14,460
05 12 23.75 7600	W12x170	L.F.	2	$234.00	$468.00	$3.39	$6.78	$1.41	$2.82	33.54	$16,019
05 12 23.75 1740	W12x190*	L.F.	1	$261.25	$261.25	$4.22	$4.22	$2.34	$2.34	33.54	$8,982
05 12 23.75 1740	W12x210*	L.F.	4	$288.75	$1,155.00	$4.22	$16.88	$2.34	$9.36	33.54	$39,619
05 12 23.75 1740	W12x230*	L.F.	1	$316.25	$316.25	$4.22	$4.22	$2.34	$2.34	33.54	$10,827
05 12 23.75 1740	W12x252*	L.F.	3	$346.50	$1,039.50	$4.22	$12.66	$2.34	$7.02	33.54	$35,525
05 12 23.75 1740	W12x279*	L.F.	2	$383.63	$767.25	$4.22	$8.44	$2.34	$4.68	33.54	$26,174
05 12 23.75 1740	W12x305*	L.F.	1	$419.38	$419.38	$4.22	$4.22	$2.34	$2.34	33.54	$14,286
05 12 23.75 1740	W12x336*	L.F.	23	$462.00	$10,626	$4.22	$97.06	$2.34	$53.82	33.54	$361,457
05 12 23.75 2500	W14x193*	L.F.	7	$265.38	$1,857.63	$3.75	$26.25	$2.08	$14.56	33.54	$63,674
									Total Brace Cost		$764,853

*- the section was unavailable in the RS means, therefore the next higher section was used as a reference. To find the material cost, the sections unit weight was multiplied by $1.375/lb

COLUMN BASE PLATE CONNECTIONS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0500	1"x20" plate**	S.F.	55	$51.00	$2,805.00	-	-	-	-	0.139	$389.58
05 05 23.25 1380	A490-7/8" Φ bolt**	Ea.	220	$16.35	$3,597.00	$4.39	$965.80	-	-	-	$4,562.80
								Total Column Base Plate Connections			$4,952

** Design assumption

COLUMN SPLICE CONNECTIONS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0450	3/4"x12" plate**	S.F.	550	$38.50	$21,175.00	-	-	-	-	0.0625	$1,323.44
05 05 23.25 1380	A490-7/8" Φ bolt**	Ea.	6600	$16.35	$107,910.00	$4.39	$28,974.00	-	-	-	$136,884.00
								Total Column Splice Connections			$138,207

** Design assumption

Braced Frame Connections											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0300	3/8"x6"plate	S.F.	132	$19.15	$2,527.80	-	-	-	-	0.0156	$39
05 05 23.25 0220	A325-7/8" Φ bolt x 3" long	Ea.	792	$3.20	$2,534	$3.43	$2,717	-	-	-	$5,251
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Weight (lb)	Total
05 12 23.40 0400	L4x4x1/4 angle	Lb.	5730	$0.73	$4,182.90	$2.73	$15,643	$0.28	$1,604.40	6.6	$141,439
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Length (ft)	Total
05 05 21.90 1500	1/4" thick weld	L.F.	132	$0.63	$83	$8.20	$1,082	$2.43	$320.76	0.708	$1,053
								Total Skewed Shear Cost			$147,783

ORTHOGONAL SHEAR CONNECTIONS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Weight (lb)	Total
05 12 23.40 0400	L4x4x1/4 angle	Lb.	5730	$0.73	$4,182.90	$2.73	$15,643	$0.28	$1,604.40	6.6	$141,439
05 05 23.25 0220	A325-3/4" Φ bolt	Ea.	17190	$3.20	$55,008	$3.43	$58,962	-	-	-	$113,970
								Total Orthogonal Shear Cost			$255,409

SKEWED SHEAR CONNECTIONS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0300	3/8"x6"plate	S.F.	132	$19.15	$2,527.80	-	-	-	-	0.0156	$39
05 05 23.25 0220	A325-3/4" Φ bolt x 3" long	Ea.	792	$3.20	$2,534	$3.43	$2,717	-	-	-	$5,251
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Length (ft)	Total
05 05 21.90 1500	1/4" thick weld	L.F.	132	$0.63	$83	$8.20	$1,082	$2.43	$320.76	0.708	$1,053
05 05 21.90 1800	3/8" thick weld	L.F.	132	$1.05	$139	$13.70	$1,808	$4.05	$534.60	0.708	$1,758
								Total Skewed Shear Cost			$8,101

MOMENT CONNECTIONS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 12 23.65 0450	3/4"x12"plate	S.F.	1980	$38.50	$76,230.00	-	-	-	-	0.0625	$4,764
05 12 23.65 0300	3/8"x4"plate	S.F.	990	$19.15	$18,958.50	-	-	-	-	0.0104	$197
05 05 23.25 0365	A325-7/8" Φ bolt x 3"long	Ea.	26730	$4.41	$117,879	$3.59	$95,961	-	-	-	$213,840
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Length (ft)	Total
05 05 21.90 1500	1/4" thick weld	L.F.	1980	$0.63	$1,247	$8.20	$16,236	$2.43	$4,811.40	0.750	$16,721
								Total Moment Connection Cost			$235,523

STEEL FLOOR DECK											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
05 31 13.50 5800	3" deep, 20 gauge	S.F.	1	$2.01	$2.01	$0.53	$0.53	$0.04	$0.04	381965	$985,470
								Total Steel Floor Deck Cost			$985,470

SHEAR STUDS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	-	Total
05 05 23.85 0900	3/4" diameter stud, 6" long	Ea.	25296	$0.77	$19,477.92	$0.89	$22,513	$0.43	$10,877.28	-	$52,869
								Total Shear Stud Cost			$52,869

SPRAYED CEMENTITIOUS FIREPROOFING											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
07 81 16.10 0700	flat decking, beams and columns	S.F.	1	$0.59	$0.59	$0.81	$0.81	$0.12	$0.12	381965	$580,587
								Total Fireproofing Cost			$580,587

SHEAR STUDS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	-	Total
05 05 23.85 0900	3/4" diameter stud, 6" long	Ea.	25296	$0.77	$19,477.92	$0.89	$22,513	$0.43	$10,877.28	-	$52,869
								Total Shear Stud Cost			$52,869

SPRAYED CEMENTITIOUS FIREPROOFING											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Area (ft^2)	Total
07 81 16.10 0700	flat decking, beams and columns	S.F.	1	$0.59	$0.59	$0.81	$0.81	$0.12	$0.12	381965	$580,587
								Total Fireproofing Cost			$580,587

ELEVATED SLABS											
Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip. Unit Cost	Equip. Cost	Volume (y^3)	Total
03 31 05.35 0300	Normal wt. concrete ready mix, 4000 psi	C.Y.	1	$103.00	$103.00	-	-	-	-	14146.9	$1,457,126
03 31 05.70 1500	elevated slab, 6-10" thick pumped with crane	C.Y.	1	-	-	$15.10	$15.10	$4.82	$4.82	14146.9	$281,805
03 31 05.70	high rise, for more than 5 stories, pumped, add per story	C.Y.	1	-	-	$1.15	$1.15	$0.37	$0.37	14146.9	$21,503
								Total Elevated Slab Cost			$1,760,434

SYSTEM	COST
B-4 SOG	$400,000
Building Foundations (footings & strap beams)	$725,000
Lower level (B-4 to 1st flr) foundation walls	$1,200,000.00
Columns and elevated decks (B-4 to 1st flr)	$3,140,000.00
Misc. subcontractor costs (submittals, gen. conditions, tower crane, etc.)	$250,000.00

Note: Refer to final report for total superstructure cost

Appendix F: Acoustics and Lighting Breadth Analyses

Lighting Analysis

PENTRON® T5 FLUORESCENT LAMPS

PENTRON® T5 lamps are designed to operate on dedicated electronic programmed rapid start (also know as programmed start) ballasts only. These lamps are globally standardized and are designed to operate with their peak light output at 35°C (95°F) ambient temperature. For comparison purposes and to accommodate existing lamp measurement standards, ratings are given at both 25°C (77°F) and 35°C (95°F). The new lamp dimensions allow for innovative fixture designs and improved fixture performance

PENTRON® High Performance T5 Lamps

Nominal Wattage	Bulb	Nominal Length (in)	MOL (in)	Base	Product Number	Ordering Abbreviation	Pkg Qty	Avg Rated Life @3hrs/start (@12hrs/start)	CCT (K)	CRI	Approx Lumens Initial @25°C/77°F (@35°C/95°F)	Approx Lumens Mean @25°C/77°F (@35°C/95°F)	Symbols & Footnotes
28	T5	48	45.8	Mini Bi-pin	20868	FP28/830/ECO	40	20000	3000	85	2600 2900	2418 2697	31 33 34 49 74,76
					20901	FP28/835/ECO	40	20000	3500	85	2600 2900	2418 2697	31 33 34 49 74,76
					20902	FP28/841/ECO	40	20000	4100	85	2600 2900	2418 2697	31 33 34 49 74,76
					22203	FP28/850/ECO	40	20000	5000	85	2545 2840	2367 2641	31 33 34 49 74,76
					20990	FP28/865/ECO	40	20000	6500	85	2400 2750	2232 2558	31 33 34 49 74,76
					20977	FP28RED 40/CS 1/SKU	40	20000			2100		15 31 33 34 48 74
					20978	FP28GREEN 40/CS 1/SKU	40	20000			3500		15 31 33 34 48 74
					20986	FP28BLUE 40/CS 1/SKU	40	20000			700		15 31 33 34 48 74
14	T5	24	22.2	Mini Bi-pin	20907	FP14/830/ECO	40	20000	3000	85	1200 1350	1116 1256	31 33 34 49 74,76
					20908	FP14/835/ECO	40	20000	3500	85	1200 1350	1116 1256	31 33 34 49 74,76
					20914	FP14/841/ECO	40	20000	4100	85	1200 1350	1116 1256	31 33 34 49 74,76
					20988	FP14/865/ECO	40	20000	6500	85	1100 1300	1045 1209	31 33 34 49 74,76
21	T5	36	34	Mini Bi-pin	20919	FP21/830/ECO	40	20000	3000	85	1900 2100	1767 1953	31 33 34 49 74,76
					20921	FP21/835/ECO	40	20000	3500	85	1900 2100	1767 1953	31 33 34 49 74,76
					20924	FP21/841/ECO	40	20000	4100	85	1900 2100	1767 1953	31 33 34 49 74,76
					20989	FP21/865/ECO	40	20000	6500	85	1750 2000	1662 1860	31 33 34 49 74,76
35	T5	60	57.6	Mini Bi-pin	20925	FP35/830/ECO	40	20000	3000	85	3300 3650	3069 3394	31 33 34 49 74,76
					20926	FP35/835/ECO	40	20000	3500	85	3300 3650	3069 3394	31 33 34 49 74,76
					20927	FP35/841/ECO	40	20000	4100	85	3300 3650	3069 3394	31 33 34 49 74,76

PENTRON® PREMIER™ High Performance T5 Lamps

Nominal Wattage	Bulb	Nominal Length (in)	MOL (in)	Base	Product Number	Ordering Abbreviation	Pkg Qty	Avg Rated Life @3hrs/start (@12hrs/start)	CCT (K)	CRI	Approx Lumens Initial @25°C/77°F (@35°C/95°F)	Approx Lumens Mean @25°C/77°F (@35°C/95°F)	Symbols & Footnotes
28	T5	48	45.8	Mini Bi-pin	20948	FP28/830PM/ECO	40	20000	3000	85	2730 3050	2594 2898	31 33 34 49 74,76
					20943	FP28/835PM/ECO	40	20000	3500	85	2730 3050	2594 2898	31 33 34 49 74,76
					20944	FP28/841PM/ECO	40	20000	4100	85	2730 3050	2594 2898	31 33 34 49 74,76

For more complete product information visit www.sylvania.com

Symbols/Footnotes on page 160-165

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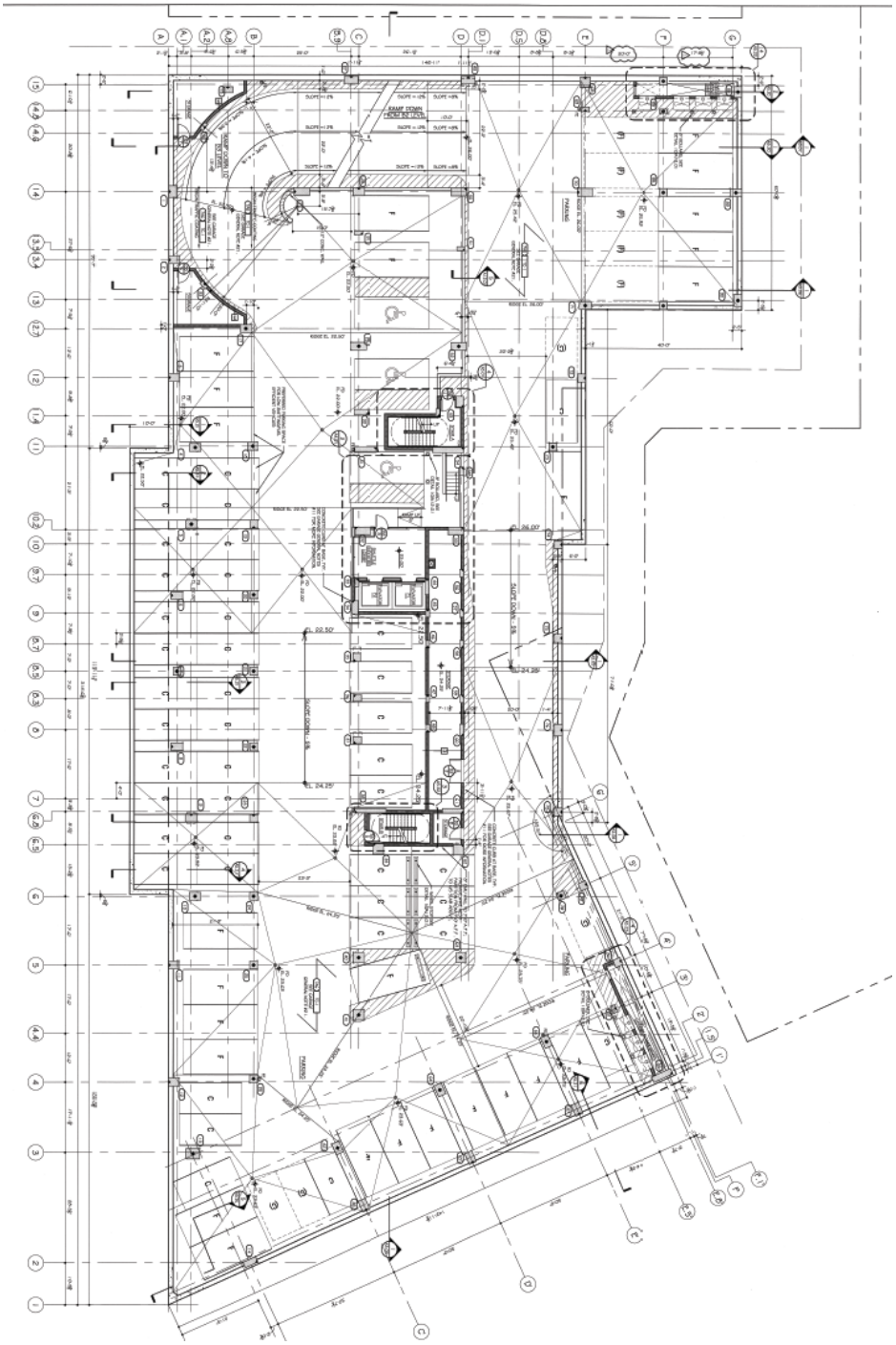
FLUORESCENT

The luminaire/Lamp catalog sheet with selected lamp highlighted can be seen in yellow

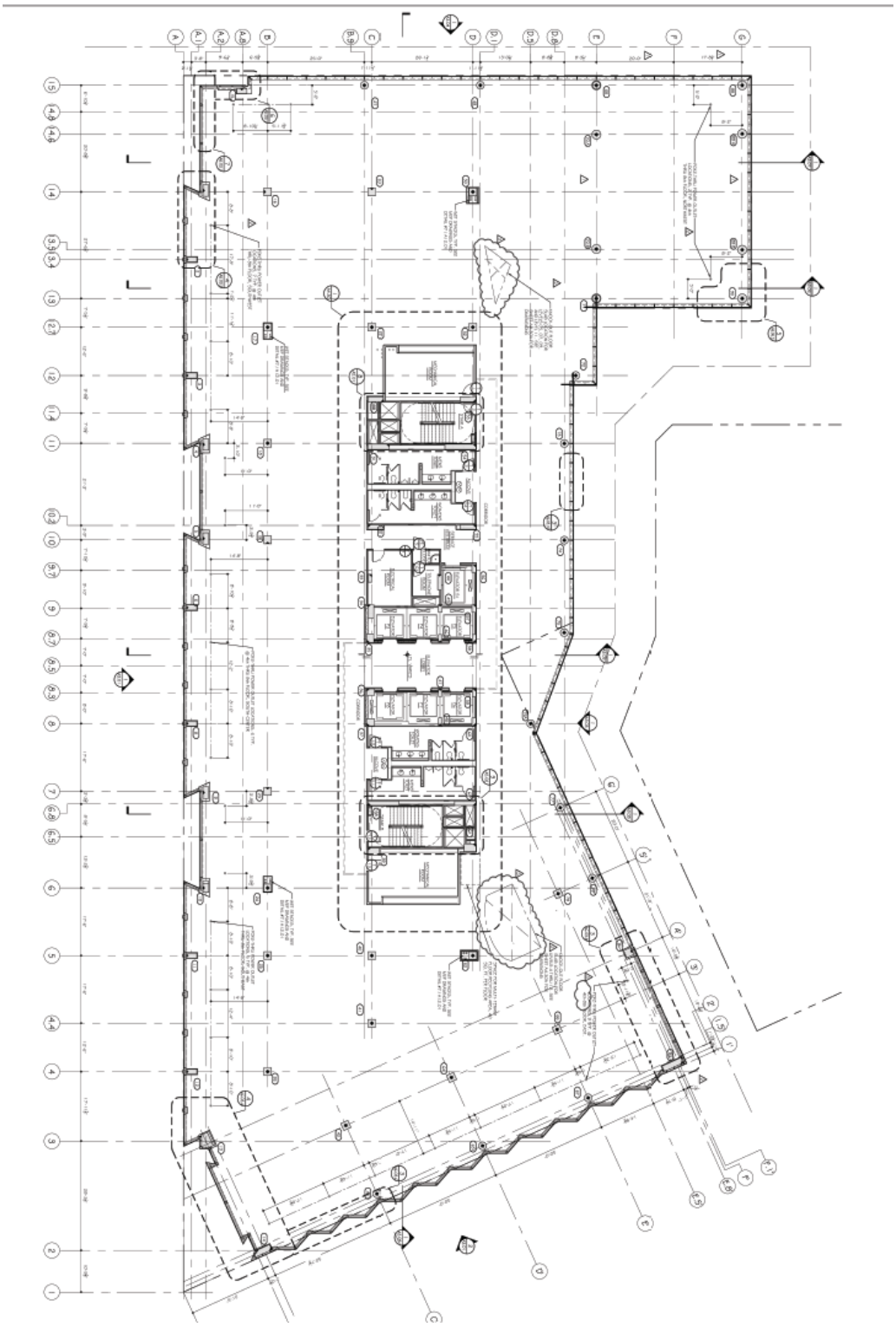
Appendix G: Typical Floor Plans

Existing System Typical Floor Plans and Building Sections



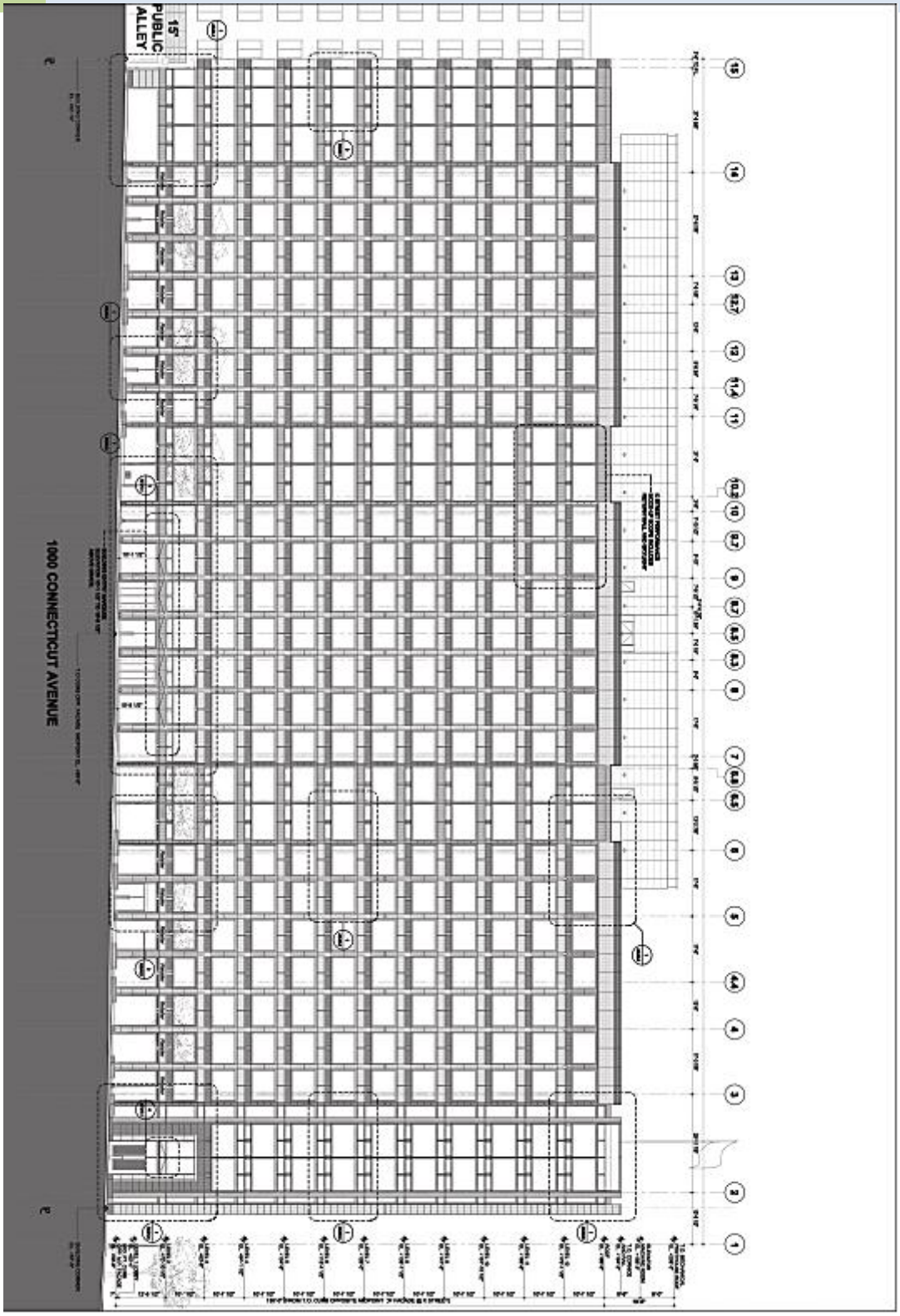
Typical underground parking plan rotated 90 degrees CW

Image obtained from Sheet A2.03 from existing Architectural drawings



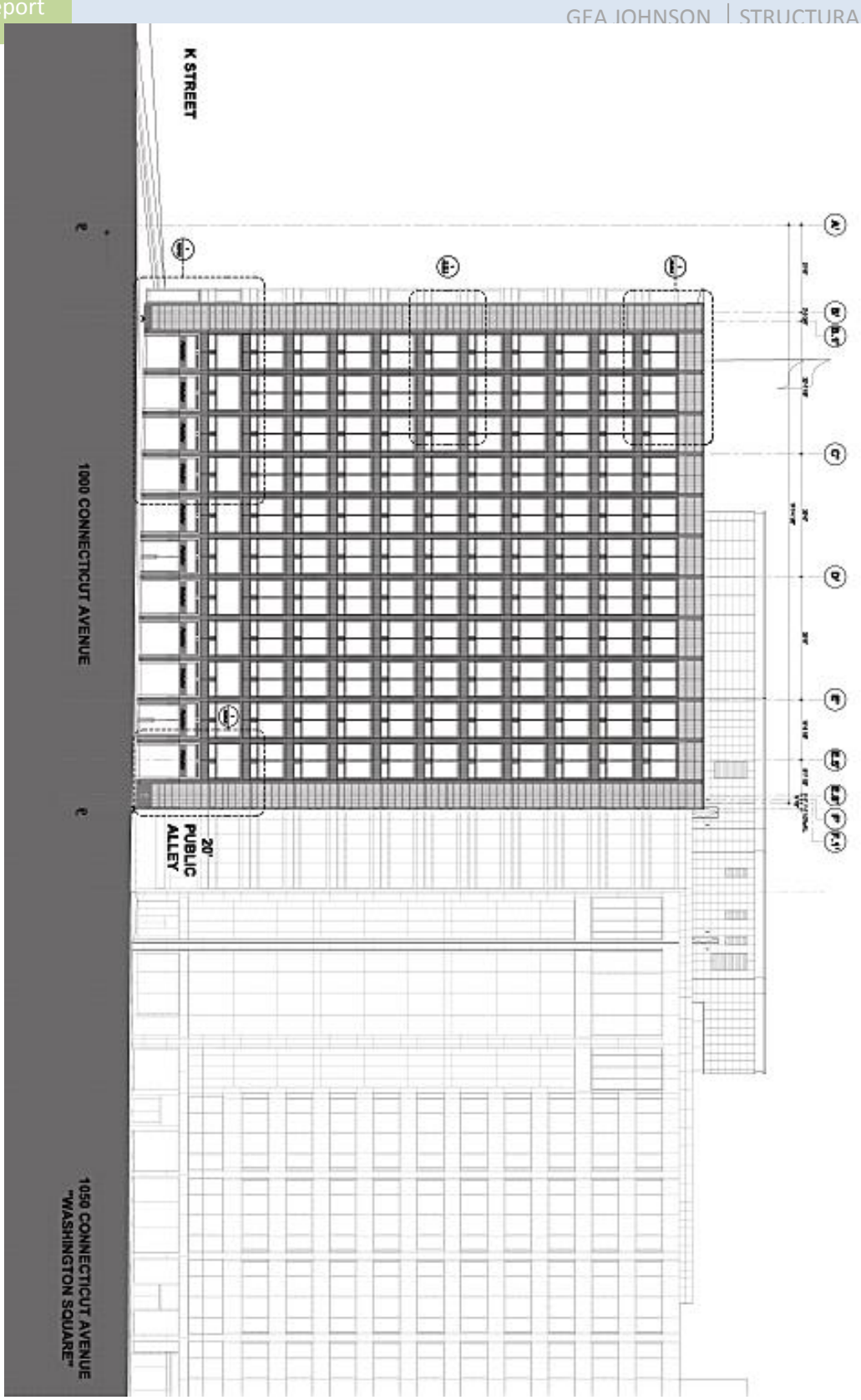
Typical Floor plan oriented 90 degrees CW

Image obtained from Sheet A2.08 from existing Architectural drawings



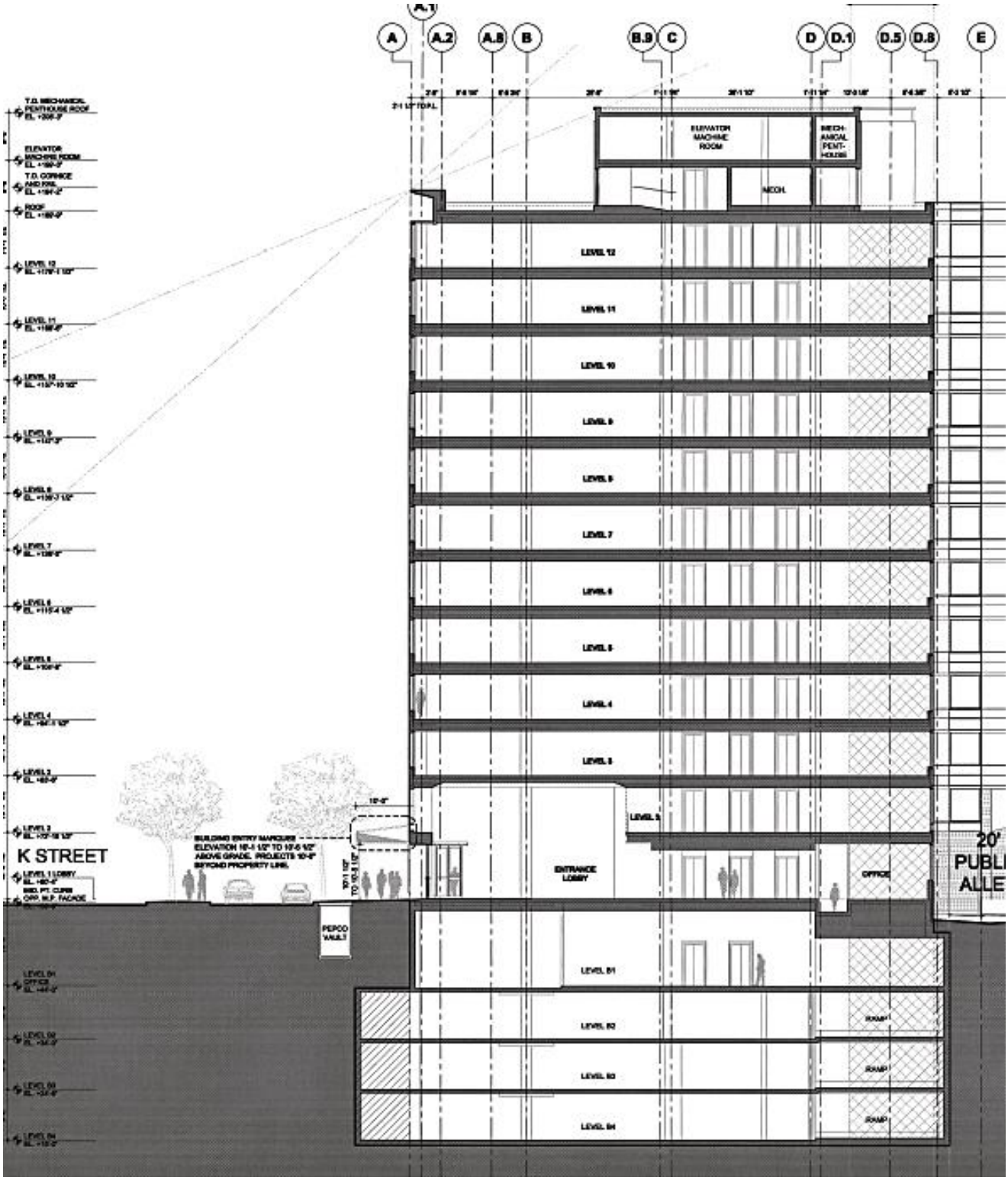
Elevation K Street oriented 90 degrees CW

Image obtained from Sheet A3.01 from existing Architectural drawings



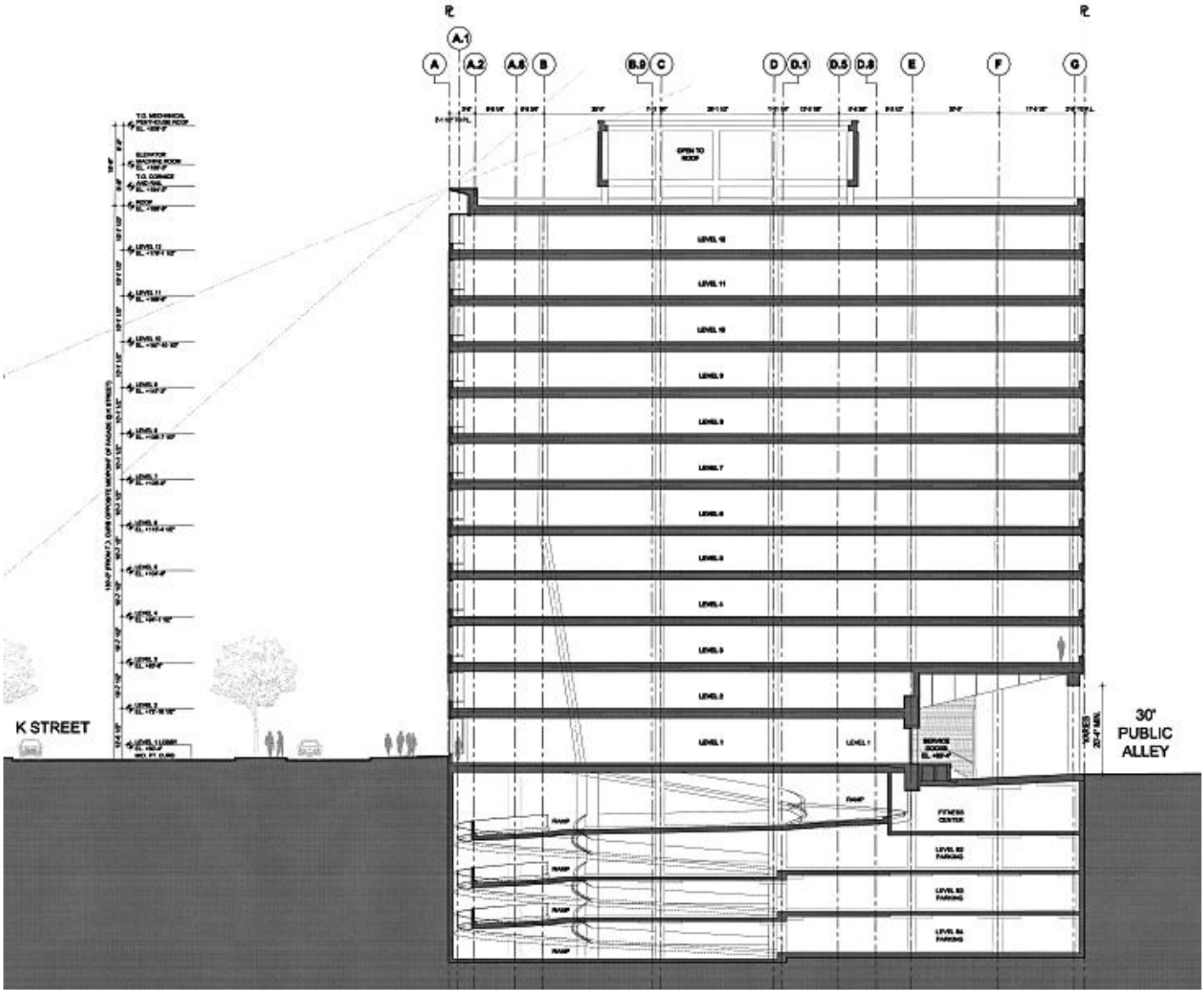
Elevation Connecticut Avenue oriented 90 degrees CW

Image obtained from Sheet A3.02 from existing Architectural drawings



Building section A

Image obtained from Sheet A3.05 from existing Architectural drawings



Building section B

Image obtained from Sheet A3.06 from existing Architectural drawings