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APPENDIX A

Gravity System Redesign Calculations

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SPOT CHECKS – BEAM/GIRDER DESIGN

1/2

DESIGN LOADS:

DL = 100 psf (approx. based on seismic floor wts)

SDL = 25 psf (partitions/MEP/FINISHES)

LL = 80 psf (hotel/corridors)

FLOOR SYSTEM: 8" NWC plank

Load combination used: 1.2D+1.6L

Interior Beam: (LINE C (2-3))

Factored Load: $1.2(100+25)+1.6(80) = 278$ psf

TRIB. WIDTH = $\left(\frac{31.66'+26'}{2}\right) = 28.83'$

$W_u = 278 \text{ psf}(28.83')/1000 = 8.015$ K/ft

$M_u = \frac{W_u \ell^2}{8} = \frac{8.015(13.42)^2}{8} = 180$ Kft

W14x68 $\phi M_n = 390$ Kft > M_u ✓OKAY

Uniformly Distributed Load:

$278 \text{ psf}(13.42)(28.83)/1000 = 108^K$ @ KL = 14'

W14x68 total capacity = $246^K > 108^K$ ✓OKAY

check deflection:

$\Delta_{LL} = \ell/360 = \frac{13.42(12)}{360} = 0.447$ in

$\Delta_{LL} = \frac{5\left(\frac{3.70}{12}\right)(13.42\times12)^4}{384(29000)\,I_{REQ}} = 0.447''$

$I_{REQ} = 208.3 \text{ in}^4 < 722 \text{ in}^4$ ✓ W14x68

$\Delta_{TL} < \ell/240 = \frac{13.42(12)}{240} = 0.671$ in

$\Delta_{TL} = \frac{5\left(\frac{8.014}{12}\right)(13.42\times12)^4}{384(29000)\,I_{REQ}} = 0.671$

$I_{REQ} = 300 \text{ in}^4 < 722 \text{ in}^4$ W14x68 ✓

$\boxed{\text{DESIGN OF W14x68 MEMBER CONSISTENT W/ STAAD}}$

*takes into account if any lateral loads must be applied

2/2

Exterior Girder Design:

Factored Load: $1.2D + 1.6L$

$W_u = 1.2(100+25) + 1.6(80) = 278$ psf

TRIB WIDTH $= \frac{31.66'}{2} = 15.83'$

LENGTH $= 22.2'$

$W_u = \frac{278(15.83)}{1000} = 4.401$ KIF

$M_u = \frac{4.401 \text{ kif } (22.2')^2}{8} = 271.1$ Kft

W14x90 $\phi M_N = 520$ Kft $> M_u$ ✓OKAY

UNIFORMLY DISTRIBUTED LOAD:

$\frac{278 \text{ psf } (22.2')(15.83)'}{1000} = 98^K$ @ $22' = KL$

W14x90 total capacity $= 209^K > 98^K$ ✓OKAY

check deflection:

$\Delta_{LL} = L/360 = \frac{22.2 \times 12}{360} = 0.74$ in $= \frac{5(2.03/12)(22.2 \times 12)^4}{384(29000) I_{REG}}$

$I_{REG} = 517$ in^4 < 999 in^4 ✓

W14x90

$\Delta_{TL} = L/240 = \frac{22.2 \times 12}{240} = 1.11 = \frac{5(4.401/12)(22.2 \times 12)^4}{384(29000) I_{REG}}$

$I_{REG} = 747$ in^4 < 999 in^4 ✓

W14x90

$\boxed{\text{DESIGN OF W14x90 GIRDER CONSISTENT W/ STAAD}}$

SPOT CHECKS - FLOOR SYSTEM

1/1

- HOLLOW CORE PRECAST ON NON-COMPOSITE STEEL (UNTOPPED)
  - Loads
    - $LL = 80$ (LIVE RMS/CORRID)
    - $SDL = 25$ psf (MEP/PAR)
    - $DL = 10$ psf
  - Total load $= 80 + 25 + 10 = 115$ psf
    - $f'_c = 5000$ psi
    - $f'_{ci} = 3500$ psi
    - $f_{pu} = 270{,}000$ psi
    - Max span $= 31'0"$
  - designed using 8" UNTOPPED
    - 4'x0" x 8" NWC UNTOPPED (4HC8)
  - from table in PCI design handbook:
    - 78-S carrying 118 psf @ 31' SPAN → 7 strands @ 8/16" ϕ - STRAIGHT
    - self wt. 56 psf
    - 1.1" camber erect.
    - 1.2" camber longterm

GIRDERS (FROM JUST SLAB)

Load $= 1.2(35) + 1.6(80) = 170$ psf

$$M_u = \frac{(170 \text{ psf})(13.4')(31')^2}{8} = 274^{\text{'k}}$$

THE DESIGN GIRDER W 14x90 $\phi M_n = 520^{\text{'k}}$

$\phi M_n = 520^{\text{'k}} > 274^{\text{'k}}$ ✓OKAY

$$\Delta_{LL} = \ell/360 = \frac{13.4(12)}{360} = 0.45"$$

$$0.45 = \frac{5(80)(31)(13.4)^4 \times 1728}{384(29000)(1000)I_x}$$

$I_x = 139 \text{ in}^4 < 999 \text{ in}^4$ W14x90 ✓OKAY

$$\Delta_{TL} = \frac{5(100+80+25)(31)(13.4)^4}{384(29000)(1000)(999)} \times 1728 = 0.16" < \ell/240 = 0.67"$$ ✓OKAY

SPOT CHECK - COLUMN $\frac{1}{2}$

COLUMN DESIGN (B3 COLUMN) (LEVEL 1)

TRIB AREA = $(11.9' \times 26.165') = 311.36\ ft^2$

$A_T = 311.36\ ft^2$

$A_I = 2(311.36) = 622.72\ ft^2$

$L_R = L \times (0.25 + \frac{15}{\sqrt{A_I}})$

reduction factor = 0.851

$L_R = 105\ psf\ (0.851) = 89.4\ psf$

DEADLOAD = 100 psf
LIVELOAD = 135 psf

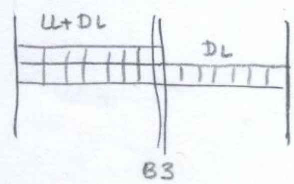
TAKE INTO ACCOUNT MOMENT OF GIRDERS

GIRDER - MOMENTS

$W_{B2\text{-}3} = 1.2(56)(13.4') + 1.6(89.5)(13.4) = 2819.4\ plf$

$FEM_{2\text{-}3} = \frac{WL^2}{12} = \frac{2.82\ (13.4)^2}{12} = 42.20\ 'k$

WORST CASE:



$W_{3\text{-}4} = 1.2(56)(10.4') = 0.70\ 'k$

$FEM_{3\text{-}4} = \frac{0.70(10.4)^2}{12} = 6.31\ 'k$

$\Delta FEM = 42.20 - 6.31 = 35.1\ 'k$

$M_u = \frac{\Delta FEM}{2} = \frac{35.1\ 'k}{2} = 17.55\ 'k$ NEGLECT

AXIAL LOAD

LL = 80 psf
DL = 100 psf

ROOF LL = 75 psf
DL = 56 psf

$W = 9[1.2(100) + 1.6(80)] + (1.2(56) + 1.6(75)) = 2419\ psf$

$P_u = \frac{2419\ psf\ (311.36)}{1000} = 753.24\ k$

- Colum load take downs give 779 k @ 1st LEVEL FOR COLUMN TO SUPPORT

KL = 18'

$P_{eff} = 1890$ FOR W 14X176

$P_{eff} = 779^k < 1890^k$ ✓ OKAY TO RESIST COLUMN LOADS

SPOT CHECK COLUMN @ 6 FLOOR where column change 2/2

(B3 COLUMN @ 6 FLOOR)

TRIB AREA = $(11.9' \times 26.165') = 311.36\ ft^2$

DEAD LOAD = 100 psf

LIVE LOAD = 80 psf

ROOF = LL = 75 psf

DL = 56 psf (plank)

AXIAL LOAD

$$W = 4[1.2(100)+1.6(80)] + (1.2(56)+1.6(75)) = 1179\ psf$$

$$P_u = \frac{1179\ psf\ (311.36)}{1000} = 367.16^k$$

° COLUMN LOAD TAKE DOWN GIVES $\underline{380}^k$ @ 6^{th} LEVEL FOR COLUMN TO SUPPORT

$KL = 10'$ $\quad P_{eff} = 1210^k$ FOR W14x99

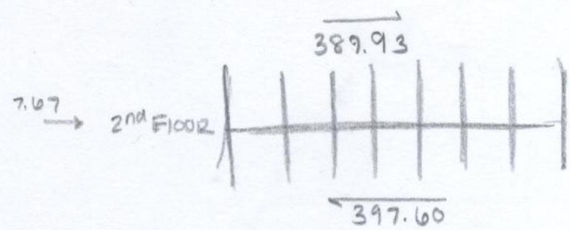
$P_{eff} = 1210 > 380^k$ ✓OKAY TO RESIST LOADS

SPOT CHECK - RESISTING MOMENT CONNECTION

1/1

BASE SHEAR = 397.60 K

$M_{per/colum.} \dfrac{187'^{K}}{7} = 26'^{K}$



THE MAXIMUM MOMENT APPLIED ON EACH COLUMN IS CALCULATED TO BE $= 26'^{K}$

THE MOMENT RESISTING CONNECTION $= 69'^{K}$

$26'^{K} < 69'^{K}$ ✓OKAY

SHEAR WALLS PROVIDE MAIN LATERAL RESISTANCE KEEPING THE MOMENT APPLIED TO EACH COLUMN

Connection Design

1/4

STEEL CONNECTION DESIGNS

Typical Moment Frame Connection

- 4 bolt unstiffened extended end plate connection

Beam: W14x68

$d = 14$ in $b_f = 10$ in $Z_x = 115$ in^3
$t_w = 0.415$ in $t_f = 0.720$ in $A_f/A_w = 1.24$

column: W14x99

$d = 14.2$ in $b_f = 14.6$ in $K = 1.38$ in
$t_w = 0.485$ in $t_f = 0.780$ in $K_1 = 1.43$ in
$T = 10$ in

beam - column flange connection $R_u = 50$ Kips
$M_u = 234$ Kips-ft

$F_y = 50$ Ksi
$F_u = 65$ Ksi
} Structural material

$F_y = 36$ Ksi
$F_u = 58$ Ksi
} connecting material

Check beam design flexural strength

$$Z_{req} = \frac{M_u \times 12}{0.9 F_y} = \frac{234 \times 12}{0.9(50)} = 62.4 \text{ in}^3$$

$Z_x = 115 \text{ in}^3 > 62.4 \text{ in}^3$ ✓ OKAY

flange force

$$P_{uf} = \frac{M_u \times 12}{(d - t_f)} = \frac{234 \times 12}{(14 - 0.720)} = 211.4^K$$

determine # bolts tension: USE 1" ϕ A325 BOLTS

$$n_{min} = \frac{P_{uf}}{\phi r_n} = \frac{211.4}{53} = 3.9 \longrightarrow 4 \text{ bolts}$$

2/4

for slip resistance:

$$n_{min} = \frac{R_u}{\phi r_n} = \frac{50^k}{19^{k}/_{bolt}} = 2.63 \rightarrow 3 \text{ bolts}$$

Min. 4 bolts @ tension
Min 2 bolts @ compression

Try $\boxed{(6)\ 1''\phi\ \text{A325 bolts}}$

check bolt shear: (N)

$\phi R_n = 6 \times 28.3 = 170^k > 50^k$ ✓ OKAY

check material bending: Try ½" - thk end plate

$\phi R_n = 2 \text{ bolts} \times 104^{k}/\text{bolt/in} \times \tfrac{1}{2} \text{ in} = 104^k > 50^k$ ✓ OKAY

- design end plate and connection to beam

try $L_e = 1\tfrac{1}{2}''$ $g = 5\tfrac{1}{2}''$ $P_f = d_b + \tfrac{1}{2} = 1\tfrac{1}{2}''$

$b_p = 2L_e + g = 2(1\tfrac{1}{2}'') + 5\tfrac{1}{2}'' = 8\tfrac{1}{2}''$

required thickness

$$M_{eu} = \frac{\alpha_m P_{uf} P_e}{4} \qquad \alpha_m = C_a C_b \left(\frac{A_f}{A_w}\right)^{1/3}\left(\frac{P_e}{d_b}\right)^{1/4}$$

$C_a = 1.45$

$C_b = \sqrt{\frac{b_f}{b_f}} = \sqrt{\frac{10}{8.5}} = 1.085$

$M_{eu} = \frac{1.57(211)(0.75)}{4}$

$M_{eu} = 62.2$ ink.

$P_e = P_f - \frac{d_b}{4} - W_t$ (assume ½" fillet)

$P_e = 1\tfrac{1}{2} - \tfrac{1}{4} - \tfrac{1}{2} = 0.75$

$\alpha_m = 1.45(1.085)(1.24)^{1/3}\left(\frac{0.75}{1}\right)^{1/4}$

$\alpha_m = 1.57$

$$t_{p_{min}} = \sqrt{\frac{4M_{eu}}{\phi F_y b_p}} = \sqrt{\frac{4(62.2)}{0.9(36)(8\tfrac{1}{2})}}$$

$t_{p_{min}} = 0.95 \longrightarrow 1$ in plate

USE 1" x 8½" end plate

3/4

check shear yield of end plate

$$\phi R_n = 2 \times \phi (0.60 F_y A_g)$$
$$= 2(0.9)(0.60)(36)(8.5 \times 1)$$
$$\phi R_n = 330.5^k > 211.4^k \quad \checkmark \text{OKAY}$$

- determined required fillet weld

Minimum size = 5/16"

$$D_{min} = \frac{0.9 F_y t_w}{2 \times 1.392} = \frac{0.9(36)(0.415)}{2 \times 1.392} = 4.83 \rightarrow 5 \text{ sixtenths}$$

USE 5/16" fillet weld on both sides of beam web.

$$l = d/2 - t_f = 14/2 - 0.720 = 6.28 \text{ in}$$

$$D_{min} = \frac{R_u}{2 \times 1.39(l)} = \frac{50}{2(1.39)(6.28)} = 2.86 \rightarrow 5 \text{ (min)}$$

USE $\boxed{5/16"}$ fillet weld on both sides of beam web below tension bolt

$$l = 2(b_f + t_f) - t_w$$
$$= 2(10 + 0.720) - 0.415 = 21.03 \text{ in}$$

$$D_{min} = \frac{P_{uf}}{1.5 \times 1.392 l} = \frac{211.4}{1.5 \times 1.392 \times 21.03} = 4.81 \rightarrow 5$$

USE $\boxed{5/16"}$ fillet welds @ beam tension flange (beam flange to end plate)
welds at compression flange may be 5/16"

check unstiff column web buckling

$$\phi R_n = 0.8(0.485)^2 \left[1 + 3\left(\frac{0.720}{14.2}\right)\left(\frac{0.485}{0.780}\right)^{1.5}\right] \sqrt{\frac{29000(50)(0.780)}{0.485}}$$

$$\phi R_n = 308.8 > 211.4^k \quad \checkmark \text{OKAY}$$

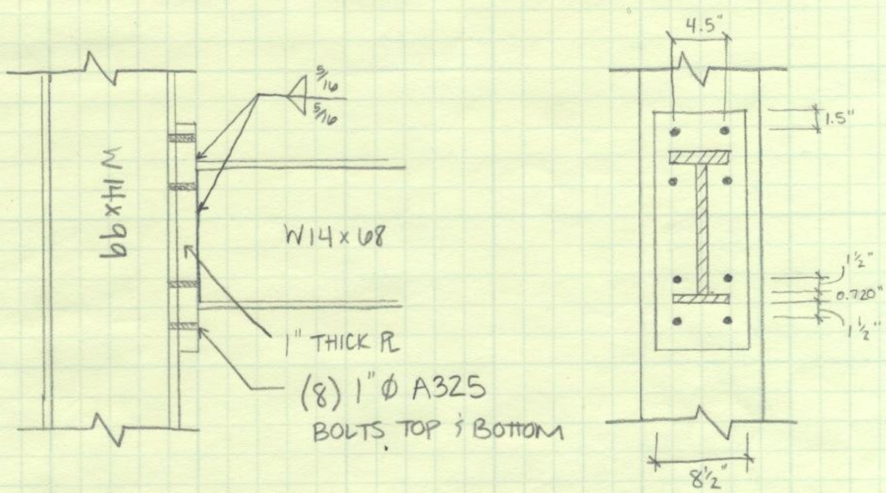
2/4

USE (6) 1" A325 bolts

USE 5/16" fillet welds on both flange and web to PL

USE 1" x 8½" Plate

Detailed Connection Below:



1/2

EXTERIOR STEEL FRAME CONNECTION

- all bolted unstiffened seated connection
- beam - column web connection

Beam: W14x90

$t_w = 0.440$ in
$d = 14$ in
$t_f = 0.710$ in
$K = 1.438$ in
$R_u = 50^K$

Column: W14x176

$t_w = 0.830$
$F_y = 50$ ksi
$F_u = 65$ ksi

USE 7/8" Ø A325-N

A36 angle material $F_y = 36$ ksi
$F_u = 58$ ksi

- seat angle and bolt design

local web yielding

$$N_{MIN} = \frac{R_u - \phi R_1}{\phi R_2} \geq K = \frac{50^k - 44.9^k}{13.7^{k}/_{in}} \geq 1.44\text{in}$$

$$= 0.372 \geq 1.44$$

$$= 1.44\text{ in}$$

web crippling

$$N/d > 0.2$$

$$N_{min} = \frac{R_u - \phi R_5}{\phi_r R_6} = \frac{50 - 51.6}{6.56} \text{ (NEG QUANTITY)}$$

$N_{min} = 1.44$ in

8" angle length - 3/4" thk - 3½in minimum leg

$\phi R_n = 97.2^k > 50^K$ ✓ OKAY

2/2

TRY L6x4x3/4 8" LONG w/ 5½-in bolt gage
For 7/8" ϕ A325-N type B (4 bolts)

$\phi R_n = 86.6^k > 50^k$ ✓OKAY

bolt bearing on angle

$\phi R_n = \phi(2.4dtF_u)$

$= 0.75(2.4 \times 7/8'' \times 3/4'' \times 58)$

$\phi R_n = 68.5^k$

BOLT BEARING NOT CRITICAL

USE (2) 7/8" ϕ A325-N bolts
to connect beam to seat angle

USE L4x4x1/4 W/ (2) 7/8" ϕ A325-N
through each leg

check supporting column

$\phi R_n = \phi(2.4dtF_u)$

$= 0.75(2.4(7/8)(0.830)65)$

$= 84.9^k$ ✓BOLT BEARING NOT CRITICAL

1/2

COLUMN BASE PLATE

$F_y = 50$ ksi
$F_y = 36$ ksi (base plate)
$f'_c = 3$ ksi
axial load $= 440^K$

W14x176 column
$d = 15.2$ in
$t_w = 0.830$ in
$b_f = 15.7$ in
$t_f = 1.31$ in

calculate req'd area

$$A_1 = \frac{P_u}{\phi_c(0.85f'_c)} = \frac{440^K}{0.6(0.85)(3)}$$

$A_1 = 287.58\ in^2$

$$\Delta = \frac{0.95d - 0.8b_f}{2} = \frac{0.95(15.2) - 0.8(15.7)}{2}$$

$\Delta = 0.94$ in

$N \approx \sqrt{A_1} + \Delta = \sqrt{287.58} + 0.94 = 17.8$ in

USE $N = 24''$ $B = 24''$

calculate thickness

$$m = \frac{N - 0.95d}{2} = \frac{24 - 0.95(15.2)}{2} = 4.78 \text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} = \frac{24 - 0.8(15.7)}{2} = 5.72 \text{ in}$$

$\phi_c P_p = 0.6(0.85 f_c A_p)$
$= 0.6(0.85 \times 3 \times 24 \times 24)$
$= 881.28^K$

$$X = \left[\frac{4db_f}{(d+b_f)^2}\right]\frac{P_u}{\phi_c P_p} = \frac{4(15.2)(15.7)}{(15.2+15.7)^2}\left(\frac{440}{882}\right)$$

$X = 0.4987$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \leq 1 = \frac{2\sqrt{0.4987}}{1+\sqrt{1-0.4987}} = 0.827 < 1 \ \checkmark$$

2/2

$$\lambda n' = \lambda \sqrt{\frac{db_f}{4}}$$

$$= (1)\sqrt{\frac{15.2 \times 15.7}{4}} = 3.86 \text{ in}$$

$$l = \max(m, n, \lambda n')$$

$$= \max(4.78,\ 5.72,\ 3.86)$$

$$l = 5.72 \text{ in}$$

$$t_{req} = l\sqrt{\frac{2P_u}{0.9F_yBN}}$$

$$= 5.72\sqrt{\frac{2(440)}{0.9(36)(24)(24)}}$$

$$t_{req} = 1.24 \text{ in}$$

USE PL 1½" x 24 x 2'-0

1/2

COLUMN SPLICE CONNECTION

d_e = W14x176

d_u = W14x99

$d_e = 15\frac{1}{4}"$

$d_u = 14\frac{1}{8}"$

$d_e = (d_u + \frac{3}{4})$ OR OVER CASE VI-C

$d_e = 14\frac{1}{8}" + \frac{3}{4}" = 14\frac{7}{8}" < d_e$ ✓OKAY

COMBO BOLTED & WELDED CONNECTION (case VI-C)

Flange Plates (from charts)

(2) WIDTH 12"

THK $\frac{3}{8}"$

LENGTH $L_u = 6\frac{1}{4}"$

$L_L = 8"$

Bolts

2 ROWS OF BOLTS

$9\frac{1}{2}"$ apart

Welds

(x) $\frac{5}{16}"$ size @ 4"

(4) $\frac{5}{16}"$ size @ 6"

FILLERS

$t = \frac{(d_p - d_u)}{2} = \frac{(15\frac{1}{4} - 14\frac{1}{8})}{2} = \frac{9}{16} - \frac{3}{16} = \frac{6}{16}"$

thk. = $\frac{3}{8}"$

weld size = B (Section J2)

length = $L_u - \frac{1}{4} = 6\frac{1}{4}" - \frac{1}{4}" = 6"$

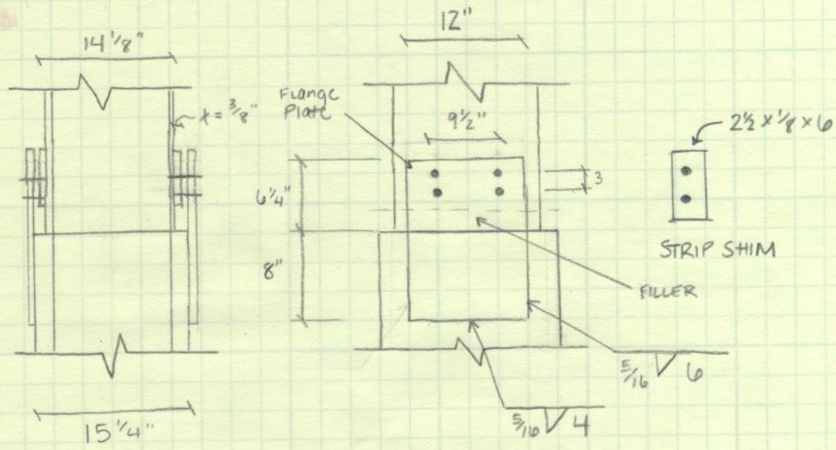
width = 6"

Shims

$2\frac{1}{2} \times \frac{1}{8} \times 6$ STRIP

DETAILED CONNECTION ON NEXT PAGE:

2/2


14 1/8"
t = 3/8"
15 1/4"
12"
Flange Plate
9 1/2"
6 1/4"
8"
3
2 1/2 x 1/8 x 6
STRIP SHIM
FILLER
5/16 6
5/16 4

APPENDIX B

Wind & Seismic Load Analysis

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Wind Loads

WIND LOADS

METHOD 2 - analytical procedure

- Determine wind variables

$V = 90$ mph
$K_d = 0.85$
$I = 1.15$
exposure = B
$K_{zt} = 1.00$

interpolate K_z:

(table 6-3)
case 2

level	height	K_z
1	0'	0
2	18'-0"	0.60
3	27'-4"	0.68
4	36'-8"	0.74
5	46'-0"	0.79
6	55'-4"	0.83
7	64'-8"	0.87
8	74'-0"	0.90
9	83'-4"	0.94
10	92'8"	0.97
roof	102'8"	1.00
high roof	112'8"	1.02

q_z (velocity pressure) $= 0.00256 \boxed{K_z} K_{zt} K_d V^2 I$
varies by level

$q_z = 0.00256 \boxed{K_z} (1.00)(0.85)(90^2)(1.15)$

example @ level 2: $q_z = 12.16$ *COMPLETED IN TABLE FOR ALL LEVELS*

q_h @ mean roof height $\bar{z} = \dfrac{102.66 + 112.66}{2} = 107.66$ ft. $= K_z = 1.01$

$\bar{z} = 0.6h = 0.6(107.66) = \boxed{64.6'} > z_{min} = 30'$ ✓

$q_h = 0.00256(1.01)(0.85)(1.00)(90^2)(1.15) = \boxed{20.47}$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{64.6}\right)^{1/6} = \boxed{0.268}$

$L_{\bar{z}} = \ell\left(\bar{z}/33\right)^{\bar{\epsilon}} = 320\left(64.6/33\right)^{1/3} = \boxed{208.81}$

2/3

WIND LOADS (cont)

$h = 107.66$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

North/South $B = 91'\text{-}0''$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{91+107.66}{208.8}\right)^{0.63}}}$$

$$\boxed{Q_{N/S} = 0.788}$$

East/West $B = 83'$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{83+107.66}{208.8}\right)^{0.63}}}$$

$$\boxed{Q_{E/W} = 0.792}$$

$$G = 0.925\left(\frac{1+1.7 g_Q I_{\bar{z}} Q}{1+1.7 g_v I_{\bar{z}}}\right)$$

g_Q / g_v should be 3.4

$$\boxed{G_{N/S} = 0.806}$$

$$\boxed{G_{E/W} = 0.808}$$

C_p - pressure coefficient

North/South
- windward = 0.8
- leeward = -0.5
- $L/B = 0.91$
- $L = 83'$ $B = 91'$

East/West
- windward = 0.8 (q_z)
- leeward = -0.3 (q_h)
- $L/B = 1.09$
- $L = 91'$ $B = 83'$

- Wind Pressure

$P_z = q_z G C_p - q_h G C_{pi}$ (windward)

$P_h = q_h G C_p - q_h G C_{pi}$ (leeward)

$GC_{pi} = +0.18$
$\phantom{GC_{pi} =} -0.18$
FOR ENCLOSED BUILDINGS

North/south

example @ level 2: $P_z = 12.16(0.806)(0.8) - 20.47(-0.18)$

$P_z = 11.53$ psf

$P_h = 20.47(0.806)(-0.5) - 20.47(0.18)$

$P_h = -11.93$ psf

3/3

WIND pressure (cont)

East/West

example @ level 2:

$P_z = 12.16(0.808)(0.8) - 20.47(-0.18)$

$P_z = 11.54$ psf

$P_h = 20.47(0.808)(-0.3) - 20.47(0.18)$

$P_h = -8.65$ psf

* wind pressures for each story calculated in table *

- Force of windward (ONLY)

  $f = B(\text{story height})\,p_z$

N/S example @ level 5: $F = 91'(9.34')(14.01) = 11.91$ K

- Force Total pressure

  N/S example @ level 5: $F = 91'(9.34')(25.94\,\text{psf}) = 22.05$ K

- Windward shear story

  N/S example @ level 9: $F = f_{\text{windward}}$ @ (PH roof + roof + 10 + 9)

  $F = (3.73 + 15.25 + 13.89 + 13.56) = 46.43$ K

- Total shear story

  N/S example @ level 10: $F = f_{\text{total}}$ @ (PH roof + roof + 10)

  $F = (6.35 + 26.11 + 24.02) = 56.48$ K

Wind Loads (North/South Direction)

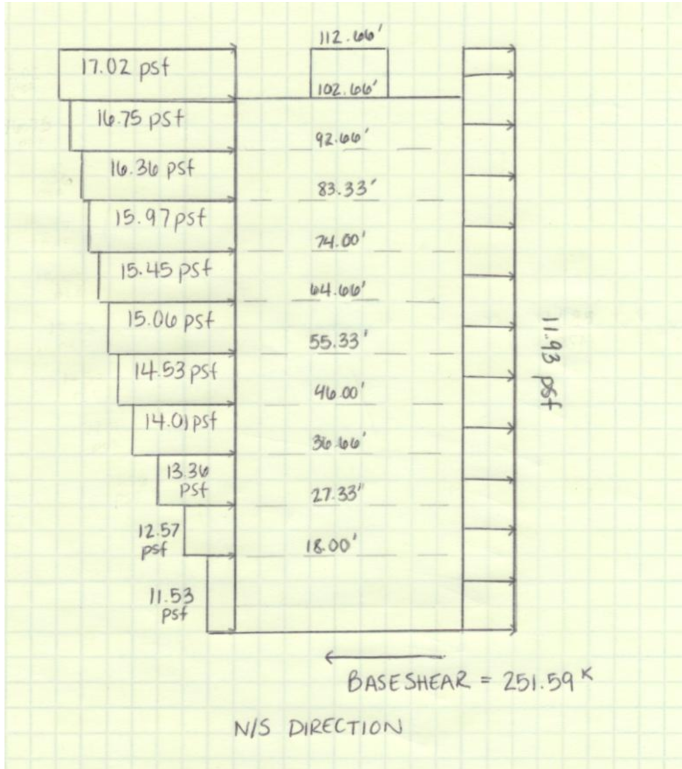
B = 91'-0" L = 83'-0"

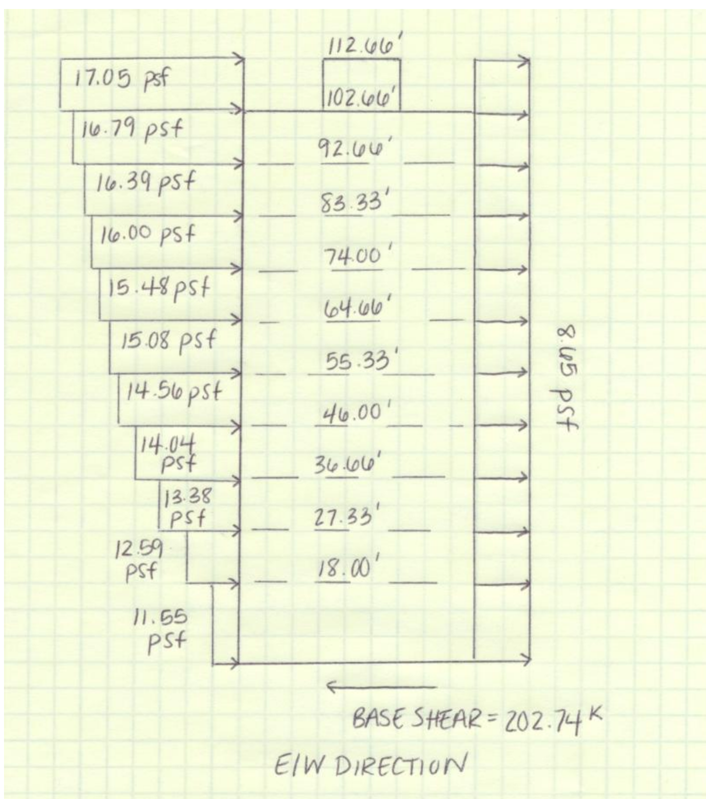
Level	Height above ground - z (ft.)	Story Height (ft.)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft-k)	Total Moment (ft-k)
					Windward	Leeward							
PH Roof	112.66	10.00	1.02	20.67	17.02	-11.93	28.95	3.73	6.35	3.73	6.35	401.92	683.81
Roof	102.66	10.00	1.00	20.27	16.75	-11.93	28.69	15.25	26.11	18.98	32.46	1488.97	2549.55
10	92.66	9.33	0.97	19.66	16.36	-11.93	28.30	13.89	24.02	32.87	56.48	1222.43	2114.02
9	83.33	9.33	0.94	19.05	15.97	-11.93	27.90	13.56	23.69	46.43	80.17	1066.63	1863.69
8	74.00	9.34	0.90	18.24	15.45	-11.93	27.38	13.13	23.27	59.56	103.45	910.26	1613.48
7	64.66	9.33	0.87	17.63	15.06	-11.93	26.99	12.78	22.91	72.34	126.36	766.88	1374.77
6	55.33	9.33	0.83	16.82	14.53	-11.93	26.47	12.34	22.47	84.68	148.83	625.13	1138.49
5	46.00	9.34	0.79	16.01	14.01	-11.93	25.94	11.91	22.05	96.59	170.88	492.13	911.35
4	36.66	9.33	0.74	15.00	13.36	-11.93	25.29	11.34	21.47	107.93	192.35	362.82	687.00
3	27.33	9.33	0.68	13.78	12.57	-11.93	24.51	10.67	20.81	118.60	213.16	241.93	471.58
2	18.00	18.00	0.60	12.16	11.53	-11.93	23.46	18.88	38.43	137.48	251.59	169.92	345.85
1	0	0	0	0	0	0	0	0	0	137.48	251.59	0	0
									Σ Windward Story Shear =	137.48	kips		
									Σ Total Story Shear =	251.59	kips		
									Σ Windward Moment =	7749.01	ft-k		
									Σ Total Moment =	13753.59	ft-k		

Wind Loads (East/West Direction)

B = 83'-0" L = 91'-0"

Level	Height above ground - z (ft.)	Story Height (ft.)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft-k)	Total Moment (ft-k)
					Windward	Leeward							
PH Roof	112.66	10.00	1.02	20.67	17.05	-8.65	25.70	4.54	6.84	4.54	6.84	488.88	736.82
Roof	102.66	10.00	1.00	20.27	16.79	-8.65	25.43	13.93	21.11	18.47	27.95	1360.70	2061.57
10	92.66	9.33	0.97	19.66	16.39	-8.65	25.04	12.70	19.39	31.17	47.34	1117.11	1706.30
9	83.33	9.33	0.94	19.05	16.00	-8.65	24.65	12.39	19.09	43.56	66.43	974.72	1501.44
8	74.00	9.34	0.90	18.24	15.48	-8.65	24.12	12.00	18.70	55.56	85.13	831.80	1296.52
7	64.66	9.33	0.87	17.63	15.08	-8.65	23.73	11.68	18.38	67.24	103.51	700.77	1102.49
6	55.33	9.33	0.83	16.82	14.56	-8.65	23.21	11.27	17.97	78.51	121.48	571.23	910.47
5	46.00	9.34	0.79	16.01	14.04	-8.65	22.68	10.88	17.58	89.39	139.06	449.69	726.72
4	36.66	9.33	0.74	15.00	13.38	-8.65	22.03	10.36	17.06	99.75	156.12	331.52	545.75
3	27.33	9.33	0.68	13.78	12.59	-8.65	21.24	9.75	16.45	109.51	172.57	221.05	372.81
2	18.00	18.00	0.60	12.16	11.55	-8.65	20.19	17.25	30.17	126.76	202.74	155.25	271.51
1	0	0	0	0	0	0	0	0	0	126.76	202.74	0	0
									Σ Windward Story Shear =	126.76	kips		
									Σ Total Story Shear =	202.74	kips		
									Σ Windward Moment =	7202.70	ft-k		
									Σ Total Moment =	11232.39	ft-k		


112.66'
17.02 psf
102.66'
16.75 psf
92.66'
16.36 psf
83.33'
15.97 psf
74.00'
15.45 psf
64.66'
15.06 psf
55.33'
14.53 psf
46.00'
14.01 psf
36.66'
13.36 psf
27.33'
12.57 psf
18.00'
11.53 psf
11.93 psf
BASE SHEAR = 251.59 K
N/S DIRECTION


112.66'
17.05 psf
102.66'
16.79 psf
92.66'
16.39 psf
83.33'
16.00 psf
74.00'
15.48 psf
64.66'
15.08 psf
55.33'
14.56 psf
46.00'
14.04 psf
36.66'
13.38 psf
27.33'
12.59 psf
18.00'
11.55 psf
8.65 psf
BASE SHEAR = 202.74 K
E/W DIRECTION

Seismic Loads

Seismic Force Resisting System: Typical Floor Weights Found

Floor 1					
Approximate Area:		7505.12	sf		
Floor to Floor Ht.		18	ft		
Steel Members:			**Superimposed:**		
Beams	W12x65	W14x68	Partitions:	15	psf
	W14x90		MEP:	10	psf
Column	W14x176		Finished:	5	psf
Weight =	**95.51**	**k**	**Weight =**	**225.15**	**k**
Slab:					
	Thickness:	8	in		
	Unit Weight:	150	pcf		
Do Not Include Slab Weight					
	Total Weight of Floor =		**320.66**	**k**	
	or		42.73	psf	

Typical Floors 2 thru 10					
Approximate Area:		7505.12	sf		
Floor to Floor Ht.		9.33	ft		
Steel Members:			**Superimposed:**		
Beams	W12x65	W14x68	Partitions:	20	psf
	W14x90		MEP:	10	psf
Column	W14x176	W14x99	Finished:	5	psf
Weight =	**95.51**	**k**	**Weight =**	**262.68**	**k**
Slab:					
	Thickness:	8	in		
	Unit Weight:	150	pcf		
	Weight =	**750.512**	**k**		
	Total Weight of Floor =		**1108.70**	**k**	
	or		147.73	psf	

1/3

SEISMIC CALCULATIONS

seismic loads

$S_{ms} = F_a F_s$

$S_{ms} = 1.6(0.125)$ $\boxed{S_{ms} = 0.2}$

$S_{DS} = \frac{2}{3}(S_{ms}) =$ $\boxed{S_{DS} = 0.133}$

$S_{m1} = F_v S_1$

$S_{m1} = 2.4(0.049)$ $\boxed{S_{m1} = 0.1176}$

$S_{d1} = \frac{2}{3}(S_{m1}) = \frac{2}{3}(0.1176) =$ $\boxed{S_{d1} = 0.0784}$

T_a (approximate fundamental period) $= C_x h_n^{x}$

$T_a = 0.02(112.66)^{0.75}$

$\boxed{T_a = 0.692 \text{ s}}$

$T = T_a(C_u) = 0.692(1.7) = \boxed{1.17 \text{ s}}$

$$C_{s\,min} = \begin{cases} \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.0784}{1.17(2/1)} = 0.0334 \\[2ex] \dfrac{S_{DS}}{R/I} = \dfrac{0.133}{(2/1)} = 0.0665 \\[2ex] \dfrac{S_{D1} T_L}{T^2(R/I)} = \dfrac{0.0784(12)}{(1.17)^2(2/1)} = 0.344 \end{cases}$$

← min controls > 0.01 ✓

$K = 0.75 + 0.5(T) = 0.75 + 0.5(1.17)$

$\boxed{K = 1.335}$

*see excel charts for floor weights determining overall building weight

Total Building weight = (Story weight) × (floor area) + ... for each floor

2/3

SEISMIC CALCULATIONS CONT

total building weight, $W_T = 11358.85^K$

base shear (V)

$V = C_S W_T = 0.034(11358.85)$

approx. $V = 386.2^K$ ✓OKAY

actual base shear calculated thru excel sheet $\boxed{V = 397.60^K}$

$W_x h_x^K$ varies @ height

example @ story 4:

$h_x = 36.66$

$W_x = 1108.7^K$

$K = 1.335$

$= 1108.7(36.66)^{1.335}$

$\boxed{= 135835}$

$\Sigma w_i h_i^k$ = sum of $W_x h_x^k$ for each floor

$\Sigma w_i h_i^k = \boxed{2725626 \text{ ft}\cdot\text{K}}$

$C_v = \dfrac{W_x h_x^k}{\Sigma w_x h_x^k}$ varies @ height

example @ story 8:

$C_v = \dfrac{346927}{2725626} = \boxed{0.127}$

Lateral Force

$F_x = C_{vx} V$

example @ story 6:

$C_{vx} = 0.086$

$V = 397.60^K$

$F_x = (0.086)(397.60^K)$

$\boxed{F_x = 34.33^K}$

3/3

SEISMIC CALCULATIONS CONT

Story shear (V_x)

V_x = lateral force (F_x) @ story + (F_x) @ all stories above

example @ story 9:

$$V_x = F_x(PH) + F_x(Roof) + F_x(10) + F_x(9)$$

$$\boxed{V_x = 202.70^K}$$

Moments @ each story due to seismic (M_x)

M_x = (trib area ht.) $\times F_x$

example @ story 9:

$$M_x = (78.665)(59.30)$$

$$\boxed{M_x = 4665 \text{ ft K}}$$

Redesigned Base Shear and Overturning Moment Distribution							
Story	h_x (ft)	Story Weight (k)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	M_x (ft-k)
PH Roof	112.66	67.67	37112	0.014	5.41	5.41	582.83
Roof	102.66	985.78	477530	0.175	69.66	75.07	6802.95
10	92.66	1108.70	468398	0.172	68.33	143.40	6012.48
9	83.33	1108.70	406522	0.149	59.30	202.70	4664.93
8	74.0	1108.70	346927	0.127	50.61	253.31	3508.65
7	64.66	1108.70	289743	0.106	42.27	295.58	2535.76
6	55.33	1108.70	235324	0.086	34.33	329.90	1739.22
5	46.0	1108.70	183906	0.067	26.83	356.73	1108.77
4	36.66	1108.70	135835	0.050	19.81	376.55	633.98
3	27.33	1108.70	91776	0.034	13.39	389.93	303.43
2	18.0	1108.70	52554	0.019	7.67	397.60	69.00
1	0	327.10	0	0	0.00	397.60	0.00
			2725626				

Total Weight =	**11358.85**	**k**
Base Shear =	**397.60**	**k**
Total Moment =	**27962.00**	**ft-k**

Existing Base Shear and Overturning Moment Distribution							
Story	h_x (ft)	Story Weight (k)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	M_x (ft-k)
PH Roof	112.66	61.34	33643	0.009	5.32	5.32	572.43
Roof	102.66	1026.15	497088	0.135	78.56	83.88	7672.36
10	92.66	1585.06	669650	0.181	105.83	189.71	9312.90
9	83.33	1585.06	581188	0.157	91.85	281.57	7225.65
8	74.0	1585.06	495988	0.134	78.39	359.95	5434.64
7	64.66	1585.06	414234	0.112	65.47	425.42	3927.71
6	55.33	1585.06	336434	0.091	53.17	478.59	2693.93
5	46.0	1585.06	262923	0.071	41.55	520.15	1717.40
4	36.66	1585.06	194198	0.053	30.69	550.84	981.98
3	27.33	1585.06	131208	0.036	20.74	571.58	470.00
2	18.0	1585.06	75134	0.020	11.87	583.45	106.87
1	0	1317.06	0	0	0.00	583.45	0.00
			3691687				

Total Weight =	**16670.13**	**k**
Base Shear =	**583.45**	**k**
Total Moment =	**40115.88**	**ft-k**

APPENDIX C

Shear Wall Design

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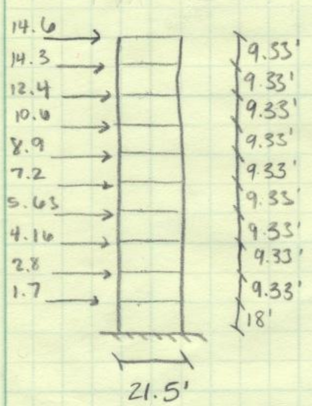
1/2

SHEAR WALL DESIGN

- seismic forces control lateral shear loads on building
- design based on seismic lateral loads distributed to each shear wall

DESIGN SHEAR WALL 1

- preliminary wall thk. = 10" (determined in prelim. shear wall thk. calcs.)
- $f'_m = 1500$ psi
- Grade 60 steel reinforcement



Normal weight concrete = 104 psf (solid grout)

$W_D = 100 \text{ psf} \times 9' = 900 \text{ plf}$

$W_L = 80 \text{ psf} \times 9' = 720 \text{ plf}$

$W_{Lr} = 75 \text{ psf} \times 9' = 675 \text{ plf}$

$$P_D = \frac{[900 + 900(9)](21.5')}{1000} + \frac{104(21.5)(102')}{100}$$

$P_D = 422$ Kips

$$P_L = \frac{10(720)(21.5')}{1000} = 155 \text{ Kips}$$

$V_E = 84$ Kips

$$P_{LR} = \frac{675(21.5)}{1000} = 15 \text{ Kips}$$

$$M_E = 14.6(102) + 14.3(92.7) + 12.4(83.34) + 10.6(74) + 8.9(64.7) + 7.2(55.4) + 5.63(46) + 4.16(36.9) + 2.8(27.36) + 1.7(18)$$

$M_E = 6126$ kft.

Factored loads applicable for load combinations are:

- $1.2D + 0.5L + 1.0E$

$P_u = [1.2 + 0.2(0.9)]422 + 0.5(155)$

$P_u = 660^K$

$V_u = 84^K$

$M_u = 6126^{Kft}$

- $0.9D + 1.0E$

$P_u = [0.9 - 0.2(0.9)](422)$

$P_u = 304^K$

$V_u = 84^K$

$M_u = 6126^{Kft}$

2/2

estimate for amount evenly distributed vertical steel req'd

$$A_{s,req'd} = \frac{2.5 M_u}{\phi f_y d} - \frac{P_u}{f_y}$$

$$= \frac{2.5(6126 \times 12)}{0.9(60)(21.5 \times 12)} - \frac{304}{60} = 8.12 \text{ in}^2$$

TRY (2) #5 bars 24" OC.

Flexural Strength w/ NO axial = $\phi M_n = 0.9\left(\frac{40840}{12}\right)$

$\phi M_n = 3063$ K-ft

Axial & Flexural Strength

$\phi P_n = 0.9(744) = 670^K$

$\phi M_n = 0.9\left(\frac{142940}{12}\right) = 10720.5$ ft K

Design wall to resist shear

$$\phi M_{n\,(P=660)} = 3063 + \left(660\left[\frac{10720 - 3063}{670 - 0}\right]\right) = 10605.72 \text{ K ft}$$

$M_n = 11784$ Kft

$V_u = 1.25(84)\left(\frac{11784}{6126}\right) = 202$ Kips

$\frac{M}{Vd} = \frac{6126}{84(258/12)} = 3.39 > 1.0$ USE 1.0

Max. shear demand on wall

$V_n < 4 A_n \sqrt{f'_m} = \frac{4(9.63 \times 244)\sqrt{1500}}{1000}$

$V_n = 385^K > 202^K$

$\frac{A_s}{S} = \frac{V_u}{\phi f_y L_w} = \frac{202}{0.8(60)(258)} = 0.0163$ in

If $S = 16"$

$A_v = 0.0163(16) = 0.27 \text{ in}^2$ $\boxed{\text{USE \#5 bars 16" OC}}$

Shear wall check 1/2

Interaction Diagram

a) Pure Flexure

$d = \frac{9.63}{2} = 4.815"$

$C = T = 0.8 f'_m ab = A_s f_y$

$a = \frac{(0.62)(60{,}000)}{0.8(1500)(16)} = 2.07"$

$M_n = A_s f_y (d - a/2) = 0.62(60000)(4.815 - \frac{2.07}{2})$

$M_n = 140678$ in lb

$\phi M_n = 0.9(140678)/1.33 = 95196$

b) Pure axial

$P_o = 0.8(0.8) f'_m (A_c - A_{st}) + A_{st} F_y$

$P_o = 0.8(0.8)1500(9.63 \times 16 - 0.62) + 0.62(60000)$

$P_o = 184521.6$ lb.

$\phi P_n/ft = 0.9(184521)/1.33 = 124864$ lb.

c) balanced pt.

$NA = 4.815\left(\frac{0.0025}{0.0025 + 0.00138}\right) = 3.102"$

$a = \beta_1 c = 2.50$

$T = A_s f_y = 0.62(60000) = 37200$

$C = 0.8 f'_m ab = 48000$ lb.

$P_n = C - T = 10800$ lb. $\phi P_n/ft = 7308$ lb/ft

$M_n = 160266$ $\phi M_n/ft = 108450$ lb/ft

2/2

P_u, M_u

$LC = 1.2D + 0.5L + 1.0E$

$M^{ecc}_{Roof} = 1239(2) = 2478$ in lb

$M^{ecc}_{Dead} = \frac{1}{2}(2478) = 1239$

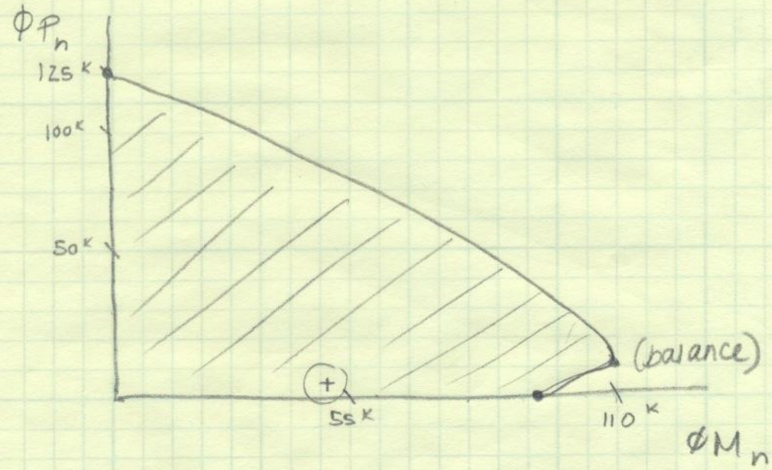
$M_E = \frac{wh^2}{8} = \frac{98(18)^2}{8} = 48060$ lbin

$M_{u\,wall} = 1.2(1239) + 1.0(48060) = 49550$ lbin/ft

$P_u = 1.2\left(1239 + 80 \times {}^{100}/_{2}\right)$

$P_u = 6289$ lb/ft wall

$+\ (49550, 6289)$ √ACCEPTABLE



Wall 3: Shear Wall Design

t_w (in) =	10
h_w (in) =	216
l_w (in) =	246
l_w (ft) =	20.5
f'm (psi) =	1500
fy (psi) =	60000
d (in) =	196.8

NWC-Solid Grout (psf) =	104
w_d (plf)=	900
w_L (plf) =	720
w_{Lr} (plf) =	675

Bar Size	Area
3	0.11
4	0.2
5	0.31
6	0.44
7	0.6
8	0.79

Required Vertical Shear Reinforcing

$\rho_{t\ min} = A_v / (s*h) = 0.0025$

$A_{v\ req'd} =$ 0.40 in^2

$A_v =$ 0.62 > 0.40 OKAY

Level	Height (ft)	F (k)	V_E (k)	Load Combination: 1.2D+0.5L+1.0E				Flexural	Axial+Flexural Strength		Vertical Reinforcement							
				P_d (k)	P_L (k)	P_u (k)	M_u (k-ft)	ΦM_n (k-ft)	ΦP_n (k)	ΦM_n (k-ft)	M_n (k-ft)	V_u (k)	< V_n (k)	A_s/s	Spacing	$A_{v,req'd}$	Design Reinf.	A_v
PH Roof	111.97	1.26	1.26	18.5	14.76	32.84	12.5512	6	31	22	26	3	296	0.000	16	0.00	(2) #5 bars	0.62
Roof	101.97	16.16	17.42	58.2	29.52	95	175	88	101	306	326	40	296	0.003	16	0.05	(2) #5 bars	0.62
10	92.64	15.85	33.27	95.2	44.28	154	485	243	168	850	885	76	296	0.006	16	0.10	(2) #5 bars	0.62
9	83.31	13.76	47.03	133.5	59.04	214	924	462	235	1618	1683	107	296	0.009	16	0.15	(2) #5 bars	0.62
8	73.98	11.74	58.77	172.0	73.8	274	1473	736	300	2577	2690	134	296	0.011	16	0.18	(2) #5 bars	0.62
7	64.65	9.81	68.57	210.9	88.56	335	2112	1056	364	3696	3879	157	296	0.013	16	0.21	(2) #5 bars	0.62
6	55.32	7.96	76.54	248.5	103.32	395	2827	1413	424	4947	5224	177	296	0.015	16	0.24	(2) #5 bars	0.62
5	45.99	6.22	82.76	286.9	118.08	455	3599	1799	484	6298	6698	193	296	0.016	16	0.26	(2) #5 bars	0.62
4	36.66	4.60	87.36	326.0	132.84	516	4415	2207	543	7726	8279	205	296	0.017	16	0.28	(2) #5 bars	0.62
3	27.33	3.11	90.47	363.6	147.6	576	5281	2641	599	9242	9977	214	296	0.018	16	0.29	(2) #5 bars	0.62
2	**18**	**1.78**	**92.25**	**420.4**	**162.36**	**661**	**6918**	**3459**	**678**	**12107**	**13213**	**220**	**296**	**0.019**	**16**	**0.30**	**(2) #5 bars**	0.62

Wall 1, 4 & 5: Shear Wall Design

t_w (in) =	10
h_w (in) =	216
l_w (in) =	258
l_w (ft) =	21.5
f'm (psi) =	1500
fy (psi) =	60000
d (in) =	206.4

NWC-Solid Grout (psf) =	104
w_d (plf)=	900
w_L (plf) =	720
w_{Lr} (plf) =	675

Bar Size	Area
3	0.11
4	0.2
5	0.31
6	0.44
7	0.6
8	0.79

Required Vertical Shear Reinforcing

$\rho_{t\,min} = A_v / (s*h) =$ 0.0025

$A_{v\,req'd} =$ 0.40 in^2

$A_v =$ 0.62 > 0.40 OKAY

Level	Height (ft)	F (k)	V_E (k)	Load Combination: 1.2D+0.5L+1.0E				Flexural	Axial+Flexural Strength		Vertical Reinforcement							
				P_d (k)	P_L (k)	P_u (k)	M_u (k-ft)	ΦM_n (k-ft)	ΦP_n (k)	ΦM_n (k-ft)	M_n (k-ft)	V_u (k)	< V_n (k)	A_s/s	Spacing	$A_{v,req'd}$	Design Reinf.	A_v
PH Roof	111.97	1.14	1.14	19.4	15.48	34.44	11.4151	6	32	20	23	3	310	0.000	16	0.00	(2) #5 bars	0.62
Roof	101.97	14.70	15.84	61.1	30.96	100	159	80	104	279	300	37	310	0.003	16	0.05	(2) #5 bars	0.62
10	92.64	14.42	30.26	99.9	46.44	161	442	221	172	773	819	70	310	0.006	16	0.09	(2) #5 bars	0.62
9	83.31	12.51	42.77	140.0	61.92	224	841	420	240	1471	1557	99	310	0.008	16	0.13	(2) #5 bars	0.62
8	73.98	10.68	53.45	180.4	77.4	288	1339	670	307	2344	2487	124	310	0.010	16	0.16	(2) #5 bars	0.62
7	64.65	8.92	62.37	221.2	92.88	352	1921	960	373	3361	3584	145	310	0.012	16	0.19	(2) #5 bars	0.62
6	55.32	7.24	69.61	260.7	108.36	414	2571	1285	435	4499	4824	163	310	0.013	16	0.21	(2) #5 bars	0.62
5	45.99	5.66	75.27	300.9	123.84	477	3273	1636	497	5728	6181	178	310	0.014	16	0.23	(2) #5 bars	0.62
4	36.66	4.18	79.45	341.9	139.32	541	4015	2008	559	7026	7634	189	310	0.015	16	0.24	(2) #5 bars	0.62
3	27.33	2.83	82.28	381.3	154.8	604	4803	2402	617	8406	9192	197	310	0.016	16	0.25	(2) #5 bars	0.62
2	**18**	**1.62**	**83.90**	**440.9**	**170.28**	**694**	**6292**	**3146**	**700**	**11011**	**12158**	**203**	**310**	**0.016**	**16**	**0.26**	**(2) #5 bars**	0.62

Wall A & D: Shear Wall Design

t_w (in) =	10
h_w (in) =	216
l_w (in) =	102
l_w (ft) =	8.5
f'm (psi) =	1500
fy (psi) =	60000
d (in) =	81.6

NWC-Solid Grout (psf) =	104
w_d (plf)=	2758
w_L (plf) =	2207
w_{Lr} (plf) =	4319

Bar Size	Area
3	0.11
4	0.2
5	0.31
6	0.44
7	0.6
8	0.79

Required Vertical Shear Reinforcing

$\rho_{t\,min} = A_v / (s*h) = 0.0025$

$A_{v\,req'd}$ = 0.40 in^2

A_v = 0.62 > 0.40 OKAY

Level	Height (ft)	F (k)	V_E (k)	Load Combination: 1.2D+0.5L+1.0E				Flexural	Axial+Flexural Strength		Vertical Reinforcement							
				P_d (k)	P_L (k)	P_u (k)	M_u (k-ft)	ΦM_n (k-ft)	ΦP_n (k)	ΦM_n (k-ft)	M_n (k-ft)	V_u (k)	< V_n (k)	A_s/s	Spacing	$A_{v,req'd}$	Design Reinf.	A_v
PH Roof	111.97	0.00	0.00	0	0	0.00	0	0	0	0	0	0	123	0.00	16	0.00	(2) #5 bars	0.62
Roof	101.97	11.28	11.28	32.283	18.7595	54	105	53	59	184	193	26	123	0.01	16	0.08	(2) #5 bars	0.62
10	92.64	11.07	22.35	63.4168	37.519	106	314	157	116	549	575	51	123	0.01	16	0.17	(2) #5 bars	0.62
9	83.31	9.61	31.96	95.081	56.2785	159	612	306	172	1071	1127	74	123	0.02	16	0.24	(2) #5 bars	0.62
8	73.98	8.20	40.16	126.8336	75.038	213	987	493	227	1727	1829	93	123	0.02	16	0.30	(2) #5 bars	0.62
7	64.65	6.85	47.01	158.763	93.7975	266	1425	712	282	2494	2660	110	123	0.02	16	0.36	(2) #5 bars	0.62
6	55.32	5.56	52.57	190.162	112.557	319	1916	958	334	3353	3603	124	123	0.03	16	0.40	(2) #5 bars	0.62
5	45.99	4.35	56.92	221.86156	131.3165	372	2447	1224	386	4283	4635	135	123	0.03	16	0.44	(2) #5 bars	0.62
4	36.66	3.21	60.12	253.844	150.076	425	3008	1504	437	5263	5737	143	123	0.03	16	0.47	(2) #5 bars	0.62
3	27.33	2.17	62.29	285.243	168.8355	478	3606	1803	486	6310	6926	150	123	0.03	16	0.49	(2) #5 bars	0.62
2	**18**	**1.24**	**63.54**	**324.598**	**187.595**	**542**	**4733**	**2366**	**545**	**8282**	**9166**	**154**	**123**	**0.03**	**16**	**0.50**	**(2) #5 bars**	0.62

Wall B & E: Shear Wall Design

t_w (in) =	10
h_w (in) =	216
l_w (in) =	96
l_w (ft) =	8
f'm (psi) =	1500
fy (psi) =	60000
d (in) =	76.8

NWC-Solid Grout (psf) =	104
w_d (plf)=	2759
w_L (plf) =	2207
w_{Lr} (plf) =	4319

Bar Size	Area
3	0.11
4	0.2
5	0.31
6	0.44
7	0.6
8	0.79

Required Vertical Shear Reinforcing

$\rho_{t\,min} = A_v / (s*h) = 0.0025$

$A_{v\,req'd} =$ 0.20 in^2

$A_v =$ 0.62 > 0.20 OKAY

Level	Height (ft)	F (k)	V_E (k)	Load Combination: 1.2D+0.5L+1.0E				Flexural	Axial+Flexural Strength		Vertical Reinforcement							
				P_d (k)	P_L (k)	P_u (k)	M_u (k-ft)	ΦM_n (k-ft)	ΦP_n (k)	ΦM_n (k-ft)	M_n (k-ft)	V_u (k)	< V_n (k)	A_s/s	Spacing	$A_{v,req'd}$	Design Reinf.	A_v
PH Roof	111.97	1.08	1.08	22.1	17.656	39.29	10.80918	5	36	19	22	3	115	0.001	8	0.00	(2) #5 bars	0.62
Roof	101.97	13.92	15.00	52.5	35.312	90	151	75	95	264	283	35	115	0.008	8	0.06	(2) #5 bars	0.62
10	92.64	13.65	28.65	81.8	52.968	139	418	209	151	732	767	66	115	0.014	8	0.11	(2) #5 bars	0.62
9	83.31	11.85	40.50	111.6	70.624	189	796	398	207	1393	1454	92	115	0.020	8	0.16	(2) #5 bars	0.62
8	73.98	10.11	50.61	141.5	88.28	239	1268	634	261	2219	2320	116	115	0.025	8	0.20	(2) #5 bars	0.62
7	64.65	8.45	59.06	171.5	105.936	290	1819	909	314	3183	3342	136	115	0.029	8	0.24	(2) #5 bars	0.62
6	55.32	6.86	65.92	201.1	123.592	339	2434	1217	365	4260	4498	152	115	0.033	8	0.26	(2) #5 bars	0.62
5	45.99	5.36	71.28	230.9	141.248	389	3099	1550	415	5424	5764	166	115	0.036	8	0.29	(2) #5 bars	0.62
4	36.66	3.96	75.23	261.0	158.904	440	3802	1901	463	6653	7122	176	115	0.038	8	0.31	(2) #5 bars	0.62
3	27.33	2.68	77.91	290.6	176.56	489	4548	2274	511	7960	8582	184	115	0.040	8	0.32	(2) #5 bars	0.62
2	**18**	**1.53**	**79.44**	**327.7**	**194.216**	**549**	**5958**	**2979**	**566**	**10426**	**11343**	**189**	**115**	**0.041**	**8**	**0.33**	**(2) #5 bars**	0.62

Wall C & F: Shear Wall Design

t_w (in) =	10
h_w (in) =	216
l_w (in) =	110
l_w (ft) =	9.1667
f'm (psi) =	1500
fy (psi) =	60000
d (in) =	88

NWC-Solid Grout (psf) =	104
w_d (plf)=	2759
w_L (plf) =	2207
w_{Lr} (plf) =	4319

Bar Size	Area
3	0.11
4	0.2
5	0.31
6	0.44
7	0.6
8	0.79

Required Vertical Shear Reinforcing

$\rho_{t\,min} = A_v / (s*h) = 0.0025$

$A_{v\,req'd} =$ 0.20 in^2

$A_v =$ 0.62 > 0.20 OKAY

Level	Height (ft)	F (k)	V_E (k)	Load Combination: 1.2D+0.5L+1.0E				Flexural	Axial+Flexural Strength		Vertical Reinforcement							
				P_d (k)	P_L (k)	P_u (k)	M_u (k-ft)	ΦM_n (k-ft)	ΦP_n (k)	ΦM_n (k-ft)	M_n (k-ft)	V_u (k)	< V_n (k)	A_s/s	Spacing	$A_{v,req'd}$	Design Reinf.	A_v
PH Roof	111.97	1.62	1.62	25.3	20.2308333	45.02	16.24082	8	42	28	33	4	132	0.001	8	0.01	(2) #5 bars	0.62
Roof	101.97	20.91	22.54	60.1	40.4616667	103	227	113	113	396	413	51	132	0.010	8	0.08	(2) #5 bars	0.62
10	92.64	20.51	43.05	93.7	60.6925	160	628	314	182	1099	1112	95	132	0.018	8	0.14	(2) #5 bars	0.62
9	83.31	17.80	60.85	127.9	80.9233333	217	1196	598	250	2093	2106	134	132	0.025	8	0.20	(2) #5 bars	0.62
8	73.98	15.19	76.04	162.1	101.154167	274	1906	953	315	3335	3361	168	132	0.032	8	0.25	(2) #5 bars	0.62
7	64.65	12.69	88.73	196.6	121.385	332	2733	1366	379	4782	4846	197	132	0.037	8	0.30	(2) #5 bars	0.62
6	55.32	10.31	99.04	230.4	141.615833	389	3658	1829	439	6401	6531	221	132	0.042	8	0.33	(2) #5 bars	0.62
5	45.99	8.05	107.09	264.6	161.846667	446	4657	2328	498	8149	8382	241	132	0.046	8	0.37	(2) #5 bars	0.62
4	36.66	5.95	113.04	299.1	182.0775	504	5712	2856	555	9997	10374	257	132	0.049	8	0.39	(2) #5 bars	0.62
3	27.33	4.02	117.06	333.0	202.308333	561	6834	3417	610	11959	12521	268	132	0.051	8	0.41	(2) #5 bars	0.62
2	**18**	**2.30**	**119.36**	**375.4**	**222.539167**	**629**	**8952**	**4476**	**674**	**15666**	**16585**	**276**	**132**	**0.052**	**8**	**0.42**	**(2) #5 bars**	0.62

APPENDIX D

Lateral Optimization Study

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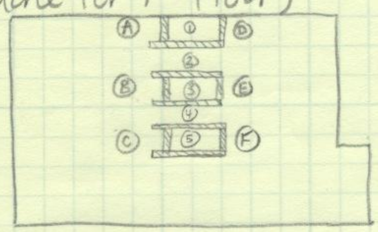
Rigidity

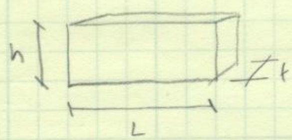
1/3

RIGIDITY CALCULATIONS

Shear wall layout:
(calculations done for 1st floor)



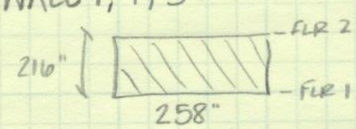
$$R = \frac{Et}{4(h/L)^3 + 3(h/L)}$$



$t = 10"$ (for all walls)
h = height from base to top of each story (varies)
L = length of wall element

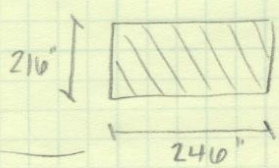
$E = 57000\sqrt{f'c}$
$E = 57000\sqrt{8000} = 5.098 \times 10^6$ psi (STORY 1-3)
$E = 57000\sqrt{5000} = 4.030 \times 10^6$ psi (STORY 4-ROOF)

WALL 1, 4, 5:



$$R_{1-1} = \frac{(5.098 \times 10^3)(10)}{4\left(\frac{216}{258}\right)^3 + 3\left(\frac{216}{258}\right)} = 10492$$

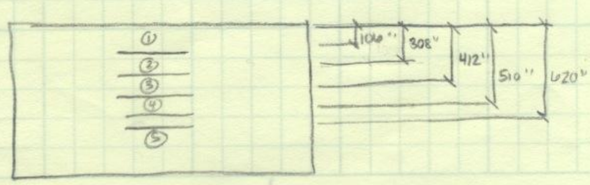
WALL 2, 3:



$$R_{2-1} = \frac{(5.098 \times 10^3)(10)}{4\left(\frac{216}{246}\right)^3 + 3\left(\frac{216}{246}\right)} = 9543$$

$$\Sigma R = R_{1-1} + R_{2-1} + R_{3-1} + R_{4-1} + R_{5-1} = \boxed{50563}$$

- CENTER OF RIGIDITY (y-coord)



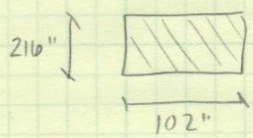
2/3

RIGIDITY CALCULATIONS CONT

$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{(10492 \times 100) + (9543 \times 308) + (9543 \times 412) + (10492 \times 510) + (10492 \times 620)}{50563}$$

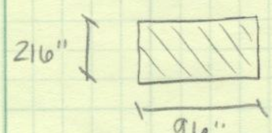
$$\boxed{y \text{ COORDINATE} = 394 \text{ in}}$$

WALL A, D:



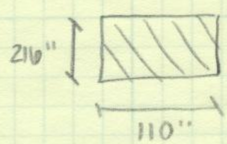
$$R_{A-1} = \frac{(5.098 \times 10^3)(10)}{4\left(\frac{216}{102}\right)^3 + 3\left(\frac{216}{102}\right)} = 1150$$

WALL B, E:



$$R_{B-1} = \frac{(5.098 \times 10^3)(10)}{4\left(\frac{216}{96}\right)^3 + 3\left(\frac{216}{96}\right)} = 975$$

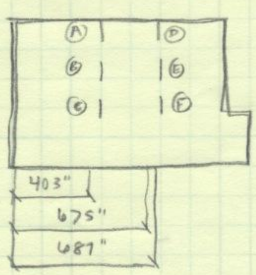
WALL C, F:



$$R_{C-1} = \frac{(5.098 \times 10^3)(10)}{4\left(\frac{216}{110}\right)^3 + 3\left(\frac{216}{110}\right)} = 1409$$

$$\Sigma R = R_{A-1} + R_{B-1} + R_{C-1} + R_{D-1} + R_{E-1} + R_{F-1} = \boxed{7067}$$

- CENTER OF RIGIDITY (X-COORD)



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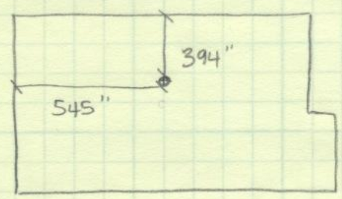
RIGIDITY CALCULATIONS CONT

$$\frac{\Sigma R \cdot d}{\Sigma R} = \frac{(1150)(403) + (975)(403) + (1409)(403) + (1150)(687) + (975)(675) + (1409)(687)}{7067}$$

$$\boxed{X \text{ COORDINATE} = 545.3''}$$

CENTER OF RIGIDITY OF FLOOR 1



° RIGIDITY CALCULATIONS FOR ALL FLOORS CAN BE FOUND IN TABLES.

CENTER OF MASS = (554.97, 504.3)

Wall Rigidity Calculation (N-S Span)									
Level	Height (in.)	Wall A ℓ = 102	Wall B ℓ = 96	Wall C ℓ = 110	Wall D ℓ = 102	Wall E ℓ = 96	Wall F ℓ = 110	Σ Rigidity	Center of Rigidity (x)
Roof	1336	0	4	6	0	4	6	19	544.6
10	1224	6	5	7	6	5	7	36	545.4
9	1112	8	6	10	8	6	10	48	545.4
8	1000	11	9	13	11	9	13	66	545.4
7	888	15	13	19	15	13	19	93	545.4
6	776	23	19	28	23	19	28	139	545.4
5	664	36	30	45	36	30	45	221	545.4
4	552	62	52	77	62	52	77	382	545.4
3	440	153	128	190	153	128	190	941	545.4
2	328	357	300	443	357	300	443	2202	545.4
1	216	1150	975	1409	1150	975	1409	7067	545.3

Wall Rigidity Calculation (E-W Span)								
Supported Floor	Height (in.)	Wall 1 ℓ = 258	Wall 2 ℓ = 246	Wall 3 ℓ = 246	Wall 4 ℓ = 258	Wall 5 ℓ = 258	Σ Rigidity	Center of Rigidity (y)
Roof	1336	0	61	61	71	71	264	471.7
10	1224	91	79	79	91	91	433	394.9
9	1112	121	105	105	121	121	573	394.9
8	1000	165	143	143	165	165	781	394.9
7	888	232	203	203	232	232	1102	394.9
6	776	342	298	298	342	342	1623	394.9
5	664	531	465	465	531	531	2522	394.8
4	552	884	776	776	884	884	4204	394.8
3	440	2043	1804	1804	2043	2043	9737	394.7
2	328	4237	3781	3781	4237	4237	20273	394.6
1	216	10492	9543	9543	10492	10492	50563	394.4

ETABS Rigidity Values		
	ETABS	
Level	x	y
Roof	598.67	406.54
10	579.52	380.00
9	579.66	381.00
8	579.53	382.00
7	579.40	383.00
6	579.24	384.00
5	579.02	385.00
4	578.70	387.00
3	578.24	389.00
2	577.57	392.00
1	577.96	393.00

Relative Stiffness

1/1

RELATIVE STIFFNESS CALCUTIONS

Relative Stiffness $= \% = \frac{R}{\Sigma R} \times 100$

WALL 1, 4, 5 : FOR FIRST FLOOR

$$\frac{R}{\Sigma R} \times 100 = \frac{10492}{50563} \times 100 = 20.8\%$$

WALL 2, 3 : 1st FLOOR

$$\frac{R}{\Sigma R} \times 100 = \frac{9543}{50563} \times 100 = 18.9\%$$

WALL A, D : 1st FLOOR

$$\frac{R}{\Sigma R} \times 100 = \frac{1150}{7067} \times 100 = 16.27\%$$

WALL B, E : 1st FLOOR

$$\frac{R}{\Sigma R} \times 100 = \frac{975}{7067} \times 100 = 13.79\%$$

WALL C, F : 1st FLOOR

$$\frac{R}{\Sigma R} \times 100 = \frac{1409}{7067} \times 100 = 19.94\%$$

- RELATIVE STIFFNESS FOR EACH WALL AT EACH LEVEL CAN BE FOUND IN THE TABLES

Torsion

1/1

TORSION CALCULATIONS

- Overall Building Torsion

in order to find $M_{t,tot}$ we need the inherent moment M_t which is due to eccentricity and the accidental moment M_{ta} which is the assumed displacement of center of mass

$M_{tot} = M_t + M_{ta}$

Factored lateral force = 1.0E (seismic controls)

E = force of total seismic @ story (found in seismic tables)

M_t = (factored lateral force) x (eccentricity)

eccentricity = center of rigidity − center of mass

example @ Story 4 in N/S direction:

$e = -0.80$

factored lateral force = 19.81^K

$M_t = 19.81(-0.80) = -15.85^{Kft}$

M_{ta} = (factored lateral force) x (5% assumed displacement each way of COM)

Center of Mass = 554.97"

% 5 displacement = 55.5" = 4.62'

factored force = 19.81^K

$M_{ta} = 19.81 \times 4.62 = 91.52^{Kft}$

$M_{tot} = M_t + M_{ta} = -15.85 + 91.52 = 75.68^{Kft.}$

overall Building torsion for each floor in each direction in tables

Shear

1/2

SHEAR CALCULATIONS

SEISMIC LOADS CONTROL

CONTROLLING COMBINATION: $0.9D + 1.0E$

Direct shear

= (factored story force) x relative stiffness

example @ story 7 in N/S direction

WALL 4: $= (50.61) \times (0.211) = 10.67^{k}$

· DIRECT shear values for each story can be found in tables

Torsional Shear

$$T = \frac{V_{tot}\, e\, d_i R_i}{J}$$

V_{tot} = story shear

e = distance from center of mass to center rigidity

d_i = distance from element to center rigidity

R_i = relative stiffness of element

J = torsional moment of inertia

example of WALL C supporting floor 3

factored story shear = 390^{k} (seismic control)

center of rigidity (x-coord) = 545.4"

center of mass = 554.97"

$e = 554.97 - 545.4 = 9.57''$

$R_i = 0.202$

location of wall C = 403" (x-COORD)

2/2

SHEAR CALCULATIONS CONT

$d_i = Wall_i - COR_i = 403'' - 545.4''$

$d_i = 142.4''$

$R_i \times d_i^2 = 0.202 \times 142.4^2 = 4097.8$

$J = 51984.7 = \Sigma R_i d_i^2$ (pulled from excel)

$$T = \frac{390(9.57'')(142.4'')(0.202)}{51984.7}$$

$\boxed{T = 2.07^K}$

calculated values for all shear walls supporting Floor 3 can be found in Tables

Original North/South Direct Shear								
Load Combination 0.9D+1.0E	Force (k)	Factored Force (k)	Distributed Force (k)					
			Wall A	Wall B	Wall C	Wall D	Wall E	Wall F
Roof	78.56	78.56	1.36	1.18	1.60	1.36	1.18	1.60
10	105.83	105.83	0.78	0.66	0.96	0.78	0.66	0.96
9	91.85	91.85	0.40	0.34	0.50	0.40	0.34	0.50
8	78.39	78.39	0.24	0.21	0.31	0.24	0.21	0.31
7	65.47	65.47	0.16	0.14	0.21	0.16	0.14	0.21
6	53.17	53.17	0.11	0.10	0.14	0.11	0.10	0.14
5	41.55	41.55	0.08	0.07	0.10	0.08	0.07	0.10
4	30.69	30.69	0.06	0.05	0.07	0.06	0.05	0.07
3	20.74	20.74	0.04	0.03	0.04	0.04	0.03	0.04
2	11.87	11.87	0.02	0.02	0.02	0.02	0.02	0.02

Original East/West Direct Shear							
Load Combination 0.9D+1.0E	Force (k)	Factored Force (k)	Distributed Force (k)				
			Wall 1	Wall 2	Wall 3	Wall 4	Wall 5
Roof	78.56	78.56	0.64	0.56	0.56	0.64	0.64
10	105.83	105.83	0.92	0.80	0.80	0.92	0.92
9	91.85	91.85	0.86	0.75	0.75	0.86	0.86
8	78.39	78.39	0.81	0.71	0.71	0.81	0.81
7	65.47	65.47	0.77	0.67	0.67	0.77	0.77
6	53.17	53.17	0.73	0.64	0.64	0.73	0.73
5	41.55	41.55	0.70	0.61	0.61	0.70	0.70
4	30.69	30.69	0.66	0.59	0.59	0.66	0.66
3	20.74	20.74	0.61	0.54	0.54	0.61	0.61
2	11.87	11.87	0.50	0.45	0.45	0.50	0.50

Torsional Shear in Shear Walls Supporting Floor 3							
		Factored Story Shear V_{tot} (k)	Relative Stiffness R_i	Distance from COM to COR e (in)	Distance from Wall i to COR d_i (in)	$(R_i)(d_i^2)$	Torsional Shear (k)
Wall	E/W	572	0.443	10.75	534.6	126608.1	2.657
Wall 1	E/W	572	0.014	10.75	433.6	2632.1	0.068
Wall 2	E/W	572	0.012	10.75	231.6	643.7	0.031
Wall 3	E/W	572	0.012	10.75	127.6	195.4	0.017
Wall 4	E/W	572	0.014	10.75	29.6	12.3	0.005
Wall 5	E/W	572	0.014	10.75	106.4	158.5	0.017
Wall	E/W	572	0.491	10.75	494.7	120161.5	2.725
Wall	N/S	572	0.483	29.6	569.5	156651.5	8.498
Wall A	N/S	572	0.002	29.6	171.5	58.8	0.011
Wall B	N/S	572	0.002	29.6	171.5	58.8	0.011
Wall C	N/S	572	0.003	29.6	171.5	88.2	0.016
Wall D	N/S	572	0.002	29.6	152.5	46.5	0.009
Wall E	N/S	572	0.002	29.6	150.5	45.3	0.009
Wall F	N/S	572	0.003	29.6	152.5	69.8	0.014
Wall	N/S	572	0.382	29.6	511.5	99943.5	6.036
Wall	N/S	572	0.122	29.6	577.5	40687.8	2.177
Torsional Moment of Inertia $J = \Sigma (R_i)(d_i^2) =$						548061.8	

Drift and Displacement

1/3

STORY DISPLACEMENT CALCULATIONS

- an approximate method used to determine Δ_{CONT} due to story shear up the building

- story drift

$$\Delta_{allowable} = 0.015 h_{sx}$$

h_{sx} = story height below story x (ASCE-7 table 12.12-1)

$$\Delta_{cont} = \Delta_{flex} + \Delta_{shear}$$

$$\Delta_{flex} = \frac{Ph^3}{3E_c I} \qquad \Delta_{shear} = \frac{1.2ph}{E_r A}$$

$$\Delta_{cont} = \frac{Ph^3}{3E_c I} + \frac{1.2ph}{E_r A}$$

$E_c = 57000\sqrt{8000} = 5.098 \times 10^3$ ksi (story 1-3)

$E_c = 57000\sqrt{5000} = 4.030 \times 10^3$ ksi (story 4-10)

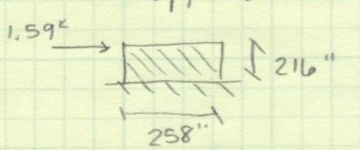
E_r = modulus of rigidity $E_r = 2.04 \times 10^3$ ksi (story 1-3)

A = (length) x (thickness) $= 1.61 \times 10^3$ ksi (story 4-10)

$$I = \frac{(\text{thickness}) \times (\text{length})^3}{12}$$

Example for WALL 5 in E/W direction

Floor 2 supported:



$A = 10 \times 258 = 2580$ in²

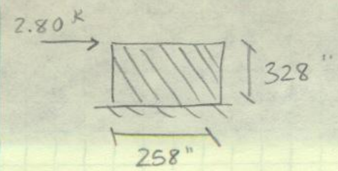
$$I = \frac{10 \times (258)^3}{12} = 14311260 \text{ in}^4$$

$$\Delta_1 = \frac{1.59(216)^3}{3(5.098 \times 10^3) I} + \frac{1.2(1.59)(216)}{2.04 \times 10^3 (2580)}$$

$$\Delta_1 = 0.00007321 + 0.0000783$$

$$\boxed{\Delta_1 = 0.0000152}$$

FLOOR 3 SUPPORTED



$A = 2580 \text{ in}^2$

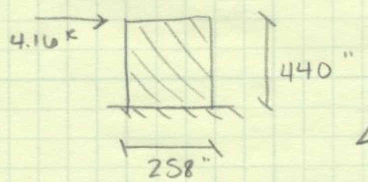
$I = 14311260 \text{ in}^4$

$$\Delta_2 = \frac{2.80(328)^3}{3(E_c)I} + \frac{1.2(2.80)(328)}{(2.04\times10^3)(2580)}$$

$$\Delta_2 = 0.000451 + 0.0000209$$

$$\boxed{\Delta_2 = 0.000660 \text{ in}}$$

FLOOR 4 SUPPORTED



$A = 2580 \text{ in}^2$

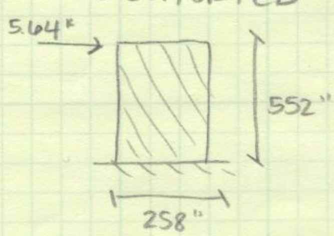
$I = 14311260 \text{ in}^4$

$$\Delta_3 = \frac{4.16(440)^3}{3E_cI} + \frac{1.2(4.16)(440)}{(2.04\times10^3)(2580)}$$

$$\Delta_3 = 0.001617 + 0.000417$$

$$\boxed{\Delta_3 = 0.002034 \text{ in}}$$

FLOOR 5 SUPPORTED



$A = 2580 \text{ in}^2$

$I = 14311260 \text{ in}^4$

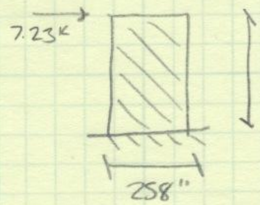
$$\Delta_4 = \frac{5.64(552)^3}{3E_cI} + \frac{1.2(5.64)(552)}{(1.61\times10^3)(2580)}$$

$$\Delta_4 = 0.005484 + 0.000900$$

$$\boxed{\Delta_4 = 0.006383 \text{ in}}$$

3/3

FLOOR 6 SUPPORTED:



$A = 2580\ in^2$

$I = 14311260\ in^4$

$$\Delta_6 = \frac{7.23(664)^3}{3(E_c)I} + \frac{1.2(7.23)(664)}{(1.61 \times 10^3)(2580)}$$

$$\Delta_6 = 0.012229 + 0.001386$$

$$\boxed{\Delta_6 = 0.013615\ in}$$

ALL DISPLACEMENTS OF WALL SUPPORTING EACH FLOOR CAN BE FOUND IN TABLES

overall wall displacement:

$$\Sigma\Delta = 0.020505 + 0.161 + 0.1192 + 0.0759 + 0.04592 + 0.02605 + 0.013615 + 0.001638 + 0.002034 + 0.000660 + 0.000152$$

$$\Sigma\Delta = 0.47413\ in$$

$$\Delta_{ALLOW} = 0.015 h_{sx} = 0.015(1225) = 18.36\ in$$

$0.47413" < 18.36"$ ✓OKAY

CALCULATED DISPLACEMENTS & DRIFTS FOR WALLS 5 & C CAN BE FOUND IN THE TABLES

Wall 5 Story Displacements											
Floor Supported	Lateral Force (k)	E_c (ksi)	E_r (ksi)	Thickness (in.)	Length (in.)	Height (in.)	Δ_{flex}	Δ_{shear}	Story Displacement (in.)	Story Drift (in.)	Allowable Story Displacement
PH Roof	1.45	4030	1610	10	258	1336	0.019947	0.000559	0.020505	0.000015	2.2
Roof	14.70	4030	1610	10	258	1224	0.155795	0.005198	0.160993	0.000132	2.2
10	14.42	4030	1610	10	258	1112	0.114566	0.004631	0.119198	0.000107	1.8
9	12.51	4030	1610	10	258	1000	0.072287	0.003613	0.075900	0.000076	1.8
8	10.67	4030	1610	10	258	888	0.043182	0.002737	0.045920	0.000052	1.8
7	8.91	4030	1610	10	258	776	0.024055	0.001997	0.026051	0.000034	1.8
6	7.23	4030	1610	10	258	664	0.012229	0.001386	0.013615	0.000021	1.8
5	5.64	4030	1610	10	258	552	0.005484	0.000900	0.006383	0.000012	1.8
4	4.16	5098	2040	10	258	440	0.001617	0.000417	0.002034	0.000005	1.8
3	2.80	5098	2040	10	258	328	0.000451	0.000209	0.000660	0.000002	1.8
2	1.59	5098	2040	10	258	216	0.000073	0.000078	0.000152	0.000001	3.24
								Total Wall Displacement (in.) =	0.471413	<	20.04 OKAY

Etabs vs. Hand Calc. Wall Displacement		
Story	Etabs Story Displacement (in.)	Approx. Hand Calc. Story Displacement (in.)
11	0.012955	0.020505
10	0.011765	0.160993
9	0.010105	0.119198
8	0.008527	0.075900
7	0.006995	0.045920
6	0.005535	0.026051
5	0.004175	0.013615
4	0.002935	0.006383
3	0.001843	0.002034
2	0.000916	0.000660
1	0.000176	0.000152
Total Wall	0.065927	0.471413

Wall C Story Displacements											
Floor Supported	Lateral Force (k)	E_c (ksi)	E_r (ksi)	Thickness (in.)	Length (in.)	Height (in.)	Δ_{flex}	Δ_{shear}	Story Displacement (in.)	Story Drift (in.)	Allowable Story Displacement
PH Roof	1.62	4030	1610	10	110	1336	0.288811	0.001470	0.290282	0.000217	2.2
Roof	14.14	4030	1610	10	110	1224	1.933288	0.011725	1.945013	0.001589	2.2
10	13.87	4030	1610	10	110	1112	1.421786	0.010447	1.432233	0.001288	1.8
9	12.03	4030	1610	10	110	1000	0.897184	0.008152	0.905336	0.000905	1.8
8	10.27	4030	1610	10	110	888	0.536029	0.006177	0.542205	0.000611	1.8
7	8.57	4030	1610	10	110	776	0.298651	0.004506	0.303158	0.000391	1.8
6	6.96	4030	1610	10	110	664	0.151869	0.003130	0.154999	0.000233	1.8
5	5.43	4030	1610	10	110	552	0.068125	0.002032	0.070157	0.000127	1.8
4	4.00	5098	2040	10	110	440	0.020103	0.000942	0.021045	0.000048	1.8
3	2.70	5098	2040	10	110	328	0.005608	0.000473	0.006081	0.000019	1.8
2	1.53	5098	2040	10	110	216	0.000909	0.000177	0.001085	0.000005	3.24
								Total Wall Displacement (in.) =	5.671593	<	20.04 OKAY

APPENDIX E

Foundation Check

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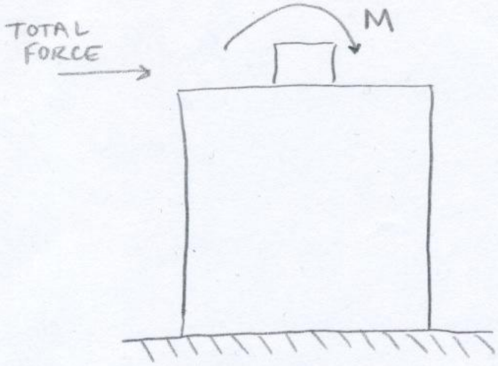
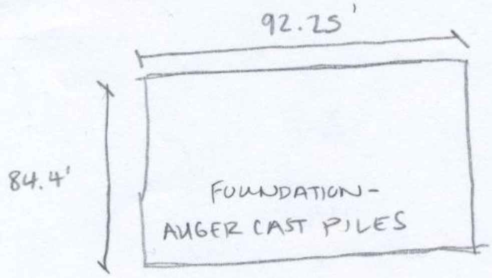
Overturning and Building Weight

OVERTURNING 1/1



- Lateral forces will create overturning moment while the gravity loads will try and resist that moment (The dead Loads)
- determine if gravity loads exceed lateral loads



- Stresses due to dead loads

$$= \frac{\text{wgt. building}}{\text{area foundation}}$$

$$= \frac{11358.85}{(84.4)(92.25)} \times 1000$$

$$= 1458.90 \text{ psf}$$

- Stresses due to lateral loads (Seismic controls)

$$= \frac{397.60 \times 1000}{(84.4)(92.25)} = 51.07 \text{ psf}$$

$$\frac{51.07}{1458.90} \times 100 = 3.4\% \text{ dead loads}$$

- GRAVITY LOADS exceed Lateral loads, therefore NO overturning done to the foundation

* FOUNDATION CAN SUPPORT NEW DESIGN

Foundation Piles

Existing Column Load (Column #10)								
Level Supported	Tributary Area (sf)	Dead Load (psf)	Live Load (psf)	Reduction LL	Dead Load (kips)	Live Load (kips)	Total Load (1.2DL+1.6L) kips	Accumulated Load (kips)
Roof	311.36	56	75	63.83	17.4	19.9	52.72	52.72
10	311.36	100	40	34.04	31.1	10.6	54.32	107.04
9	311.36	100	40	34.04	31.1	10.6	54.32	161.36
8	311.36	100	40	34.04	31.1	10.6	54.32	215.68
7	311.36	100	40	34.04	31.1	10.6	54.32	270.00
6	311.36	100	40	34.04	31.1	10.6	54.32	324.33
5	311.36	100	40	34.04	31.1	10.6	54.32	378.65
4	311.36	100	40	34.04	31.1	10.6	54.32	432.97
3	311.36	100	40	34.04	31.1	10.6	54.32	487.29
2	311.36	100	40	34.04	31.1	10.6	54.32	541.61
1	311.36	100	80	68.08	31.1	21.2	145.20	686.81

Redesigned Column Load (Column #10)								
Level Supported	Tributary Area (sf)	Dead Load (psf)	Live Load (psf)	Reduction LL	Dead Load (kips)	Live Load (kips)	Total Load (1.2D+1.6L) kips	Accumulated Load (kips)
Roof	311.36	56	75	63.83	17.4	19.9	52.72	52.72
10	311.36	70	40	34.04	21.8	10.6	43.11	95.83
9	311.36	70	40	34.04	21.8	10.6	43.11	138.94
8	311.36	70	40	34.04	21.8	10.6	43.11	182.06
7	311.36	70	40	34.04	21.8	10.6	43.11	225.17
6	311.36	70	40	34.04	21.8	10.6	43.11	268.28
5	311.36	70	40	34.04	21.8	10.6	43.11	311.39
4	311.36	70	40	34.04	21.8	10.6	43.11	354.50
3	311.36	70	40	34.04	21.8	10.6	43.11	397.62
2	311.36	70	40	34.04	21.8	10.6	43.11	440.73
1	311.36	70	80	68.08	21.8	21.2	60.07	500.80

Comparison of the number of piles to support the new design vs. the original design				
Column No.	**Total Load on Each Column (k)**	**# of piles required to support Steel System**	**# of piles used in Original Design**	**% Decrease in required # of piles**
1	132	1	4	75
2	265	2	4	50
3	246	2	Not in original	-200
4	330	2	4	50
5	390	2	4	50
6	300	2	4	50
7	135	1	4	75
8	262	2	4	50
9	547	3	4	25
10	505	3	4	25
11	664	3	4	25
12	774	4	4	0
13	580	3	4	25
14	266	2	4	50
15	286	2	4	50
16	597	3	4	25
17	553	3	4	25
18	732	4	4	0
19	853	4	4	0
20	653	3	4	26
21	392	2	4	50
22	102	1	4	75
23	195	1	4	75
24	478	3	4	25
25	643	4	4	0
26	680	4	4	0
27	318	2	4	50
Average % Decrease in # Piles Required =				**28**

APPENDIX F

Façade Study Calculations

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THERMAL GRADIENT CALCULATIONS

~~EXISTING~~

EXISTING CMU/BRICK SYSTEM:
① BRICK
② 3/4" AIR SPACE
③ MEMBRANE FLASHING
④ FULL HT. RIGID INSULATION
⑤ CMU UNIT
⑥ 5/8" GYPSOM BOARD

① BRICK:
direction of heat flow: HORIZ.
Mean temp: 0°F
temp. difference: 10°F
thk: 3/4"
E = 0.82
R = 0.11

② AIR SPACE:
direction heat flow: HORIZ.
Mean temp = 0°F
temp. difference = 20°F
thk = 3/4"
E = 0.82 R = 1.26

③ membrane flashing
R = 1.32

④ RIGID INSULATION
direction of heat flow = HORIZ.
Mean temp = 50°F
temp difference = 30°F
thk = 2½"
E = 0.82
R = 13.75

⑤ CMU block unit
R = 3.2
u = 0.314

⑥ 5/8" GYPSUM BRD.
R = 0.56

$R_o = 0.17$
$R_1 = 0.11$
$R_2 = 1.26$
$R_3 = 1.32$
$R_4 = 13.75$
$R_5 = 3.2$
$R_6 = 0.56$
$R_i = 0.68$

$\Sigma R_{0-i} = 21.05$

u value = 0.0475

$$T_x = T_o + (T_i - T_o)\frac{\Sigma R_{0-x}}{\Sigma R_{0-i}}$$

$$T_1 = 0 + (70-0)\frac{0.28}{21.05} = 0.931°F$$
$$T_2 = 70\left(\frac{1.54}{21.05}\right) = 5.12°F$$
$$T_3 = 70\left(\frac{2.86}{21.05}\right) = 9.51°F$$
$$T_4 = 70\left(\frac{16.61}{21.05}\right) = 55.24°F$$
$$T_5 = 70\left(\frac{19.81}{21.05}\right) = 65.88°F$$
$$T_6 = 70\left(\frac{20.37}{21.05}\right) = 67.74°F$$

• THERMAL GRADIENT

BTW.	ΣR_{o-x}	Temp(°F)
0-1	0.17	0
1-2	0.28	0.931
2-3	1.54	5.12
3-4	2.86	9.51
4-5	16.61	55.24
5-6	19.81	65.88
	20.37	67.74
	21.05	70°F

CURTAIN WALL SYSTEM :

① GLASS
② AIR SPACE
③ GLASS

OPTION I

(R values obtained from Manufacturer)

① GLASS PANEL
$R = 2.17$

② AIR SPACE
direction of heat flow: HORIZ.
mean temp = 0°F
temp. diff = 10°F
thk = ½"
E = 0.82
$R = 1.15$

③ GLASS PANEL
$R = 2.17$

$R_o = 0.17$
$R_1 = 2.17$
$R_2 = 1.15$
$R_3 = 2.17$
$R_i = 0.68$

$\Sigma R_{o-i} = 6.34$
↳ $U = 0.158$

• THERMAL GRADIENT

BTW	ΣR_{o-x}	Temp(°F)
0-1	0.17	0
1-2	2.34	25.84°F
2-3	3.49	38.53°F
3-i	5.66	62.49°F
	6.34	70°F

$$T_x = T_o + (T_i - T_o)\frac{\Sigma R_{o-x}}{\Sigma R_{o-i}}$$

$$T_1 = 0 + (70-0)\frac{2.34}{6.34} = 25.84°F$$

$$T_2 = 70\left(\frac{3.49}{6.34}\right) = 38.53°F$$

$$T_3 = 70\left(\frac{5.66}{6.34}\right) = 62.49°F$$

BRICK VENEER SYSTEM: OPTION 2

1. BRICK
2. 3/4" AIR SPACE
3. MEMBRANE FLASHING
4. BATT INSULATION
5. 5/8" GYPSUM BRD

(1) BRICK VENNEER
direction of heat flow: HORIZ
Mean temp = 0°F
temp diff = 10°F
thk = 3/4"
E = 0.82
R = 0.11

(2) AIR SPACE
direction heat flow: HORIZ.
Mean temp = 0°F
temp diff = 20°F
thk = 3/4"
E = 0.82
R = 1.26

(3) membrane flashing
R = 1.32

(4) BATT INSULATION
direction of heat flow: HORIZ.
Mean temp. = 50°F
temp diff = 30°F
thk = 5 5/8"
E = 0.82
R = 19

(5) 5/8" Gypsum brd
R = 0.56

$R_o = 0.17$
$R_1 = 0.11$
$R_2 = 1.26$
$R_3 = 1.32$
$R_4 = 19$
$R_5 = 0.56$
$R_i = 0.68$

$\Sigma R_{o-i} = 23.10$
$U = 0.0433$

$$T_x = T_o + (T_i - T_o)\frac{\Sigma R_{o-x}}{\Sigma R_{o-i}}$$

$$T_1 = 70\left(\frac{0.28}{23.10}\right) = 0.85°$$

$$T_2 = 70\left(\frac{1.54}{23.10}\right) = 4.67°$$

$$T_3 = 70\left(\frac{2.86}{23.10}\right) = 8.67°$$

$$T_4 = 70\left(\frac{21.86}{23.10}\right) = 66.2°$$

$$T_5 = 70\left(\frac{22.54}{23.10}\right) = 68.3°$$

• THERMAL GRADIENT

BTW	ΣR_{o-x}	Temp (°F)
0-1	0.17	0
1-2	0.28	0.848
2-3	1.54	4.67
3-4	2.86	8.67
4-5	21.86	66.2
5-i	22.54	68.3
	23.10	70°F

APPENDIX G

Construction Schedule and Cost Calculations

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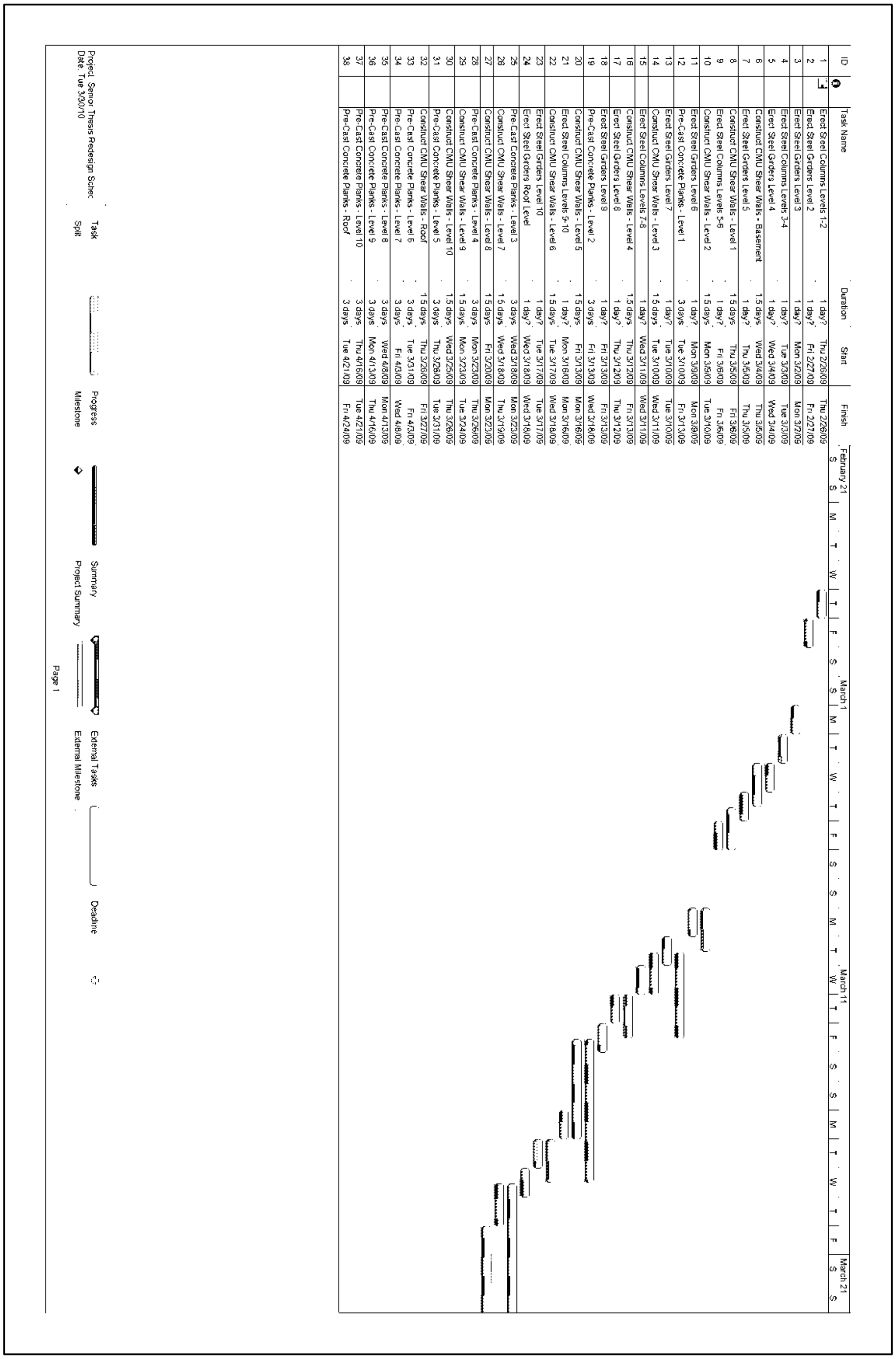
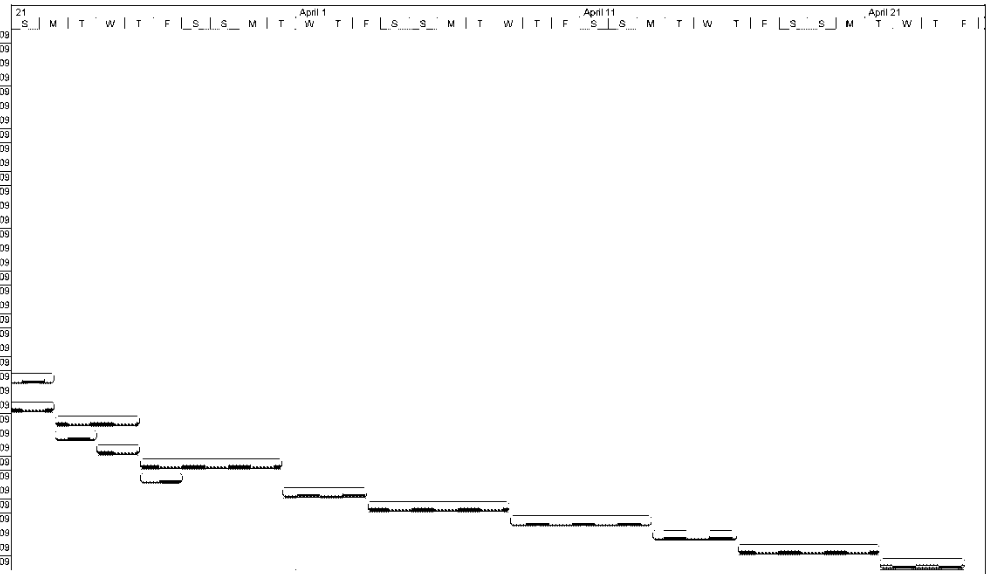
Redesign Construction Schedule



ID		Task Name	Duration	Start	Finish
1		Erect Steel Columns Levels 1-2	1 day?	Thu 2/26/09	Thu 2/26/09
2		Erect Steel Girders Level 2	1 day?	Fri 2/27/09	Fri 2/27/09
3		Erect Steel Girders Level 3	1 day?	Mon 3/2/09	Mon 3/2/09
4		Erect Steel Columns Levels 3-4	1 day?	Tue 3/3/09	Tue 3/3/09
5		Erect Steel Girders Level 4	1 day?	Wed 3/4/09	Wed 3/4/09
6		Construct CMU Shear Walls - Basement	1.5 days	Wed 3/4/09	Thu 3/5/09
7		Erect Steel Girders Level 5	1 day?	Thu 3/5/09	Thu 3/5/09
8		Construct CMU Shear Walls - Level 1	1.5 days	Thu 3/5/09	Fri 3/6/09
9		Erect Steel Columns Levels 5-6	1 day?	Fri 3/6/09	Fri 3/6/09
10		Construct CMU Shear Walls - Level 2	1.5 days	Mon 3/9/09	Tue 3/10/09
11		Erect Steel Girders Level 6	1 day?	Mon 3/9/09	Mon 3/9/09
12		Pre-Cast Concrete Planks - Level 1	3 days	Tue 3/10/09	Fri 3/13/09
13		Erect Steel Girders Level 7	1 day?	Tue 3/10/09	Tue 3/10/09
14		Construct CMU Shear Walls - Level 3	1.5 days	Tue 3/10/09	Wed 3/11/09
15		Erect Steel Columns Levels 7-8	1 day?	Wed 3/11/09	Wed 3/11/09
16		Construct CMU Shear Walls - Level 4	1.5 days	Thu 3/12/09	Fri 3/13/09
17		Erect Steel Girders Level 8	1 day?	Thu 3/12/09	Thu 3/12/09
18		Erect Steel Girders Level 9	1 day?	Fri 3/13/09	Fri 3/13/09
19		Pre-Cast Concrete Planks - Level 2	3 days	Fri 3/13/09	Wed 3/18/09
20		Construct CMU Shear Walls - Level 5	1.5 days	Fri 3/13/09	Mon 3/16/09
21		Erect Steel Columns Levels 9-10	1 day?	Mon 3/16/09	Mon 3/16/09
22		Construct CMU Shear Walls - Level 6	1.5 days	Tue 3/17/09	Wed 3/18/09
23		Erect Steel Girders Level 10	1 day?	Tue 3/17/09	Tue 3/17/09
24		Erect Steel Girders Roof Level	1 day?	Wed 3/18/09	Wed 3/18/09
25		Pre-Cast Concrete Planks - Level 3	3 days	Wed 3/18/09	Mon 3/23/09
26		Construct CMU Shear Walls - Level 7	1.5 days	Wed 3/18/09	Thu 3/19/09
27		Construct CMU Shear Walls - Level 8	1.5 days	Fri 3/20/09	Mon 3/23/09
28		Pre-Cast Concrete Planks - Level 4	3 days	Mon 3/23/09	Thu 3/26/09
29		Construct CMU Shear Walls - Level 9	1.5 days	Mon 3/23/09	Tue 3/24/09
30		Construct CMU Shear Walls - Level 10	1.5 days	Wed 3/25/09	Thu 3/26/09
31		Pre-Cast Concrete Planks - Level 5	3 days	Thu 3/26/09	Tue 3/31/09
32		Construct CMU Shear Walls - Roof	1.5 days	Thu 3/26/09	Fri 3/27/09
33		Pre-Cast Concrete Planks - Level 6	3 days	Tue 3/31/09	Fri 4/3/09
34		Pre-Cast Concrete Planks - Level 7	3 days	Fri 4/3/09	Wed 4/8/09
35		Pre-Cast Concrete Planks - Level 8	3 days	Wed 4/8/09	Mon 4/13/09
36		Pre-Cast Concrete Planks - Level 9	3 days	Mon 4/13/09	Thu 4/16/09
37		Pre-Cast Concrete Planks - Level 10	3 days	Thu 4/16/09	Tue 4/21/09
38		Pre-Cast Concrete Planks - Roof	3 days	Tue 4/21/09	Fri 4/24/09

ID	Task Name	Duration	Start	Finish
1	Erect Steel Columns Levels 1-2	1 day?	Thu 2/26/09	Thu 2/26/09
2	Erect Steel Girders Level 2	1 day?	Fri 2/27/09	Fri 2/27/09
3	Erect Steel Girders Level 3	1 day?	Mon 3/2/09	Mon 3/2/09
4	Erect Steel Columns Levels 3-4	1 day?	Tue 3/3/09	Tue 3/3/09
5	Erect Steel Girders Level 4	1 day?	Wed 3/4/09	Wed 3/4/09
6	Construct CMU Shear Walls - Basement	1.5 days	Wed 3/4/09	Thu 3/5/09
7	Erect Steel Girders Level 5	1 day?	Thu 3/5/09	Thu 3/5/09
8	Construct CMU Shear Walls - Level 1	1.5 days	Thu 3/5/09	Fri 3/6/09
9	Erect Steel Columns Levels 5-6	1 day?	Fri 3/6/09	Fri 3/6/09
10	Construct CMU Shear Walls - Level 2	1.5 days	Mon 3/9/09	Tue 3/10/09
11	Erect Steel Girders Level 6	1 day?	Mon 3/9/09	Mon 3/9/09
12	Pre-Cast Concrete Planks - Level 1	3 days	Tue 3/10/09	Fri 3/13/09
13	Erect Steel Girders Level 7	1 day?	Tue 3/10/09	Tue 3/10/09
14	Construct CMU Shear Walls - Level 3	1.5 days	Tue 3/10/09	Wed 3/11/09
15	Erect Steel Columns Levels 7-8	1 day?	Wed 3/11/09	Wed 3/11/09
16	Construct CMU Shear Walls - Level 4	1.5 days	Thu 3/12/09	Fri 3/13/09
17	Erect Steel Girders Level 8	1 day?	Thu 3/12/09	Thu 3/12/09
18	Erect Steel Girders Level 9	1 day?	Fri 3/13/09	Fri 3/13/09
19	Pre-Cast Concrete Planks - Level 2	3 days	Fri 3/13/09	Wed 3/18/09
20	Construct CMU Shear Walls - Level 5	1.5 days	Fri 3/13/09	Mon 3/16/09
21	Erect Steel Columns Levels 9-10	1 day?	Mon 3/16/09	Mon 3/16/09
22	Construct CMU Shear Walls - Level 6	1.5 days	Tue 3/17/09	Wed 3/18/09
23	Erect Steel Girders Level 10	1 day?	Tue 3/17/09	Tue 3/17/09
24	Erect Steel Girders Roof Level	1 day?	Wed 3/18/09	Wed 3/18/09
25	Pre-Cast Concrete Planks - Level 3	3 days	Wed 3/18/09	Mon 3/23/09
26	Construct CMU Shear Walls - Level 7	1.5 days	Wed 3/18/09	Thu 3/19/09
27	Construct CMU Shear Walls - Level 8	1.5 days	Fri 3/20/09	Mon 3/23/09
28	Pre-Cast Concrete Planks - Level 4	3 days	Mon 3/23/09	Thu 3/26/09
29	Construct CMU Shear Walls - Level 9	1.5 days	Mon 3/23/09	Tue 3/24/09
30	Construct CMU Shear Walls - Level 10	1.5 days	Wed 3/25/09	Thu 3/26/09
31	Pre-Cast Concrete Planks - Level 5	3 days	Thu 3/26/09	Tue 3/31/09
32	Construct CMU Shear Walls - Roof	1.5 days	Thu 3/26/09	Fri 3/27/09
33	Pre-Cast Concrete Planks - Level 6	3 days	Tue 3/31/09	Fri 4/3/09
34	Pre-Cast Concrete Planks - Level 7	3 days	Fri 4/3/09	Wed 4/8/09
35	Pre-Cast Concrete Planks - Level 8	3 days	Wed 4/8/09	Mon 4/13/09
36	Pre-Cast Concrete Planks - Level 9	3 days	Mon 4/13/09	Thu 4/16/09
37	Pre-Cast Concrete Planks - Level 10	3 days	Thu 4/16/09	Tue 4/21/09
38	Pre-Cast Concrete Planks - Roof	3 days	Tue 4/21/09	Fri 4/24/09



Project: Senior Thesis Redesign Schec
Date: Tue 3/30/10

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

Page 1

Existing Construction Schedule

Description	Early Start	Early Finish	Original Duration	Remaining Duration
DEEP FOUNDATIONS				
Mobilization of Equipment	20OCT08 A	22OCT08 A	3d	0
Site Survey & Layout	21OCT08 A	24OCT08 A	3d	0
Drilling of Auger Cast Piles	23OCT08 A	12NOV08 A	15d	0
Excavation of High Pile Caps	05NOV08 A	11NOV08 A	5d	0
Form High Caps and Install Rebar	12NOV08 A	20NOV08 A	7d	0
Mass Excavation of Basement	21NOV08 A	12JAN09 A	8d	0
Pour High Caps	21NOV08 A	24NOV08 A	2d	0
Excavate Basement Caps	03DEC08 A	12JAN09 A	4d	0
Form Basement Caps and Install Rebar	05DEC08 A	12JAN09 A	6d	0
Pour Basement Caps	15DEC08 A	13JAN09 A	2d	0
Form Gradebeams and Install Rebar	15DEC08 A	15JAN09 A	7d	0
Form Columns and Piers	24DEC08 A	22JAN09 A	5d	0
Pour Grade Beams and Piers	02JAN09 A	23JAN09 A	2d	0
Form Concrete Walls	06JAN09 A	05FEB09 A	4d	0
Ivory Block Walls	19JAN09 A	03MAR09 A	12d	0
Pour Concrete Walls	28JAN09 A	09FEB09 A	2d	0
Structural Backfill	19FEB09 A	04MAR09 A	5d	0
Prep SOG Basement	19FEB09 A	24FEB09 A	3d	0
Pour SOG Basement- North Wing	24FEB09 A	02APR09 A	1d	0
Interior Underground Sanitary & Storm	03MAR09 A	03MAR09 A	5d	0
Pour SOG Basement - East Wing	06MAR09 A	11MAR09 A	3d	0
Complete Deep Foundations & Basement Slab		12MAR09 A	0	0
STRUCTURE				
Set Plank L1	26FEB09 A	09MAR09 A	3d	0
Block Walls L1	05MAR09 A	17MAR09 A	8d	0
Prep SOG Level 1	11MAR09 A	13MAR09 A	2d	0
Pour SOG Level 1	13MAR09 A	02APR09 A	2d	0
Form Walls & Beams L1	17MAR09 A	25MAR09 A	6d	0
Pour Walls & Beams L1	23MAR09 A	25MAR09 A	2d	0
Set Precast Plank L2	25MAR09 A	02APR09 A	3d	0
Block Walls & Sigma Studs L2	30MAR09 A	09APR09 A	8d	0
Form & Pour Added Concrete Walls L2	06APR09 A	13APR09 A	5d	0
Set Precast Plank L3	16APR09 A	21APR09 A	3d	0
Block Walls & Sigma Studs L3	21APR09 A	01MAY09 A	8d	0
Form & Pour Added Concrete Walls L3	24APR09 A	01MAY09 A	5d	0
Set Precast Plank L4	06MAY09 A	07MAY09 A	3d	0
Block Walls & Sigma Studs L4	07MAY09 A	14MAY09 A	8d	0
Form & Pour Added Concrete Walls L4	08MAY09 A	14MAY09 A	5d	0
Set Precast Plank L5	15MAY09 A	19MAY09 A	3d	0
Block Walls & Sigma Studs L5	20MAY09 A	29MAY09 A	8d	0
Set Precast Plank L6	29MAY09 A	02JUN09 A	3d	0
Block Walls & Sigma Studs L6	02JUN09 A	11JUN09 A	8d	0
Set Precast Plank L7	11JUN09 A	15JUN09 A	3d	0
Block Walls & Sigma Studs L7	15JUN09 A	24JUN09 A	8d	0
Set Precast Plank L8	18JUN09 A	23JUN09 A	3d	0
Block Walls & Sigma Studs L8	23JUN09 A	29JUN09 A	8d	0
Set Precast Plank L9	29JUN09 A	06JUL09 A	5d	0
Block Walls & Sigma Studs L9	03JUL09 A	14JUL09 A	7d	0

Description	Early Start	Early Finish	Original Duration	Remaining Duration
Set Precast Plank L10	09JUL09 A	15JUL09 A	4d	0
Block Walls & Sigma Studs L10	15JUL09 A	27JUL09 A	8d	0
Set Preacast Plank - Roof Deck	21JUL09 A	27JUL09 A	4d	0
Elevator Penthouse Block Walls	27JUL09 A	06AUG09 A	4d	0
Set Plank at Elevator Penthouse	13AUG09 A	13AUG09 A	1d	0
Complete Masonry Bearing Structure		13AUG09 A	0	0

Redesigned Material Takeoffs

COLUMN TAKEOFFS

Size	#	Length (ft)	Weight (lbs)
W14x176	27	1493.64	262880.64
W14x99	27	1259.55	124695.45
	54	2753.19	387576.09

GRAVITY BEAM TAKEOFFS

Size	#	Length (ft)	Weight (lbs)
W12x65	60	855.3	55594.5
W14x68	150	2409.2	163825.6
W14x90	60	1543.5	138915
	270	4808	358335.1

Existing Material Takeoffs

BEAM TAKEOFFS

Size	#	Length (ft)
W 14x22	1	10.43
W 16x26	3	52.55
W 18x35	1	25.6
W 18x40	6	89.2
W 24x55	3	47.8
W 30x90	1	20.6
W 33x118	3	36.2
W 33x130	3	36.2
W 33x141	2	27.2
W 40x149	2	27.2
W 40x199	2	44.22
	27	417.2
Weight =	**36180 lbs**	

COLUMN TAKEOFFS

Size	#	Length (ft)	Weight (k)
W12x120	1	18	2.16
W12x96	2	36	3.46
W10x100	2	36	3.6
W12x120	5	90	10.8
W12x106	2	36	3.82
W10x112	4	48	5.38
W10x68	2	36	2.45
	18	300	31.67

Cost Estimate of Redesigned Structural System

Shearwalls	Amount	Unit	Crew	Daily Output	Days	Labor Hours/Unit	Labor Hours	Mat'l Cost/Unit	Labor Cost/Unit	Equip Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	TOTAL COST
10" CMU Block	18473	SF	D-8	320	58	0.125	2309.1	3.06	4.47	0	7.53	10.15	$ 187,500.95
Reinforcement	7	Ton	4 Rodm	5.5	1	---	---	810	420	0	1230	---	$ 8,610.00
Steel	**Amount**	**Unit**	**Crew**	**Daily Output**	**Days**	**Labor Hours/Unit**	**Labor Hours**	**Mat'l Cost/Unit**	**Labor Cost/Unit**	**Equip Cost/Unit**	**Total Cost/Unit**	**Total Cost w/ O&P**	**TOTAL COST**
Columns	2753	LF	E-2	720	3.82	0.078	214.73	145	3.26	2.18	150.44	168	$ 462,504.00
Baseplates	108	SF	E-2	60	1.80	0.061	6.588	45		0	45		$ 5,171.61
Beams	4807	LF	E-2	750	6.41	0.075	360.53	89.5	3.13	2.09	94.72	106	$ 509,542.00
Fireproofing	7559	SF	G-2	1500	5.04	900	4535.4	1	1	1.2	3.2		$ 24,188.80
Crane					76.07						300		$ 22,821.94
											Total Cost of Existing System:		**$1,197,517.36**
											Time to Construct System:		**76.07**

Cost Estimate of Existing Structural System

Shearwalls	Amount	Unit	Crew	Daily Output	Days	Labor Hours/Unit	Labor Hours	Mat'l Cost/Unit	Labor Cost/Unit	Equip Cost/Unit	Total Cost/Unit	Total Cost w/ O&P	TOTAL COST
8" CMU Block	50019	SF	D-8	395	127	0.101	5051.919	2.27	3.62	0	5.89	8	$ 400,152.00
10" CMU Block	9716	SF	D-8	320	30	0.125	1214.5	3.06	4.47	0	7.53	10.15	$ 98,617.40
Reinforcement	17	Ton	4 Rodm	5.5	3	---	---	810	420	0	1230	---	$ 20,910.00
Steel	**Amount**	**Unit**	**Crew**	**Daily Output**	**Days**	**Labor Hours/Unit**	**Labor Hours**	**Mat'l Cost/Unit**	**Labor Cost/Unit**	**Equip Cost/Unit**	**Total Cost/Unit**	**Total Cost w/ O&P**	**TOTAL COST**
Columns	300	LF	E-2	720	0.42	0.078	23.4	217	4.075	2.18	223.255	241	$ 72,300.00
Baseplates	72	SF	E-2	60	1.20	0.061	4.392	45	0	0	45	---	$ 3,447.74
Beams	417	LF	E-6	750	0.56	0.068	28.356	199.5	3.475	1.41	204.385	219	$ 91,323.00
Fireproofing	1434	SF	G-2	1500	0.96	900	860.4	1	1	1.2	3.2	---	$ 4,588.80
Crane					36.58						300		$ 10,974.62
											Total Cost of Existing System:		**$691,338.94**
											Time to Construct System:		**36.58**