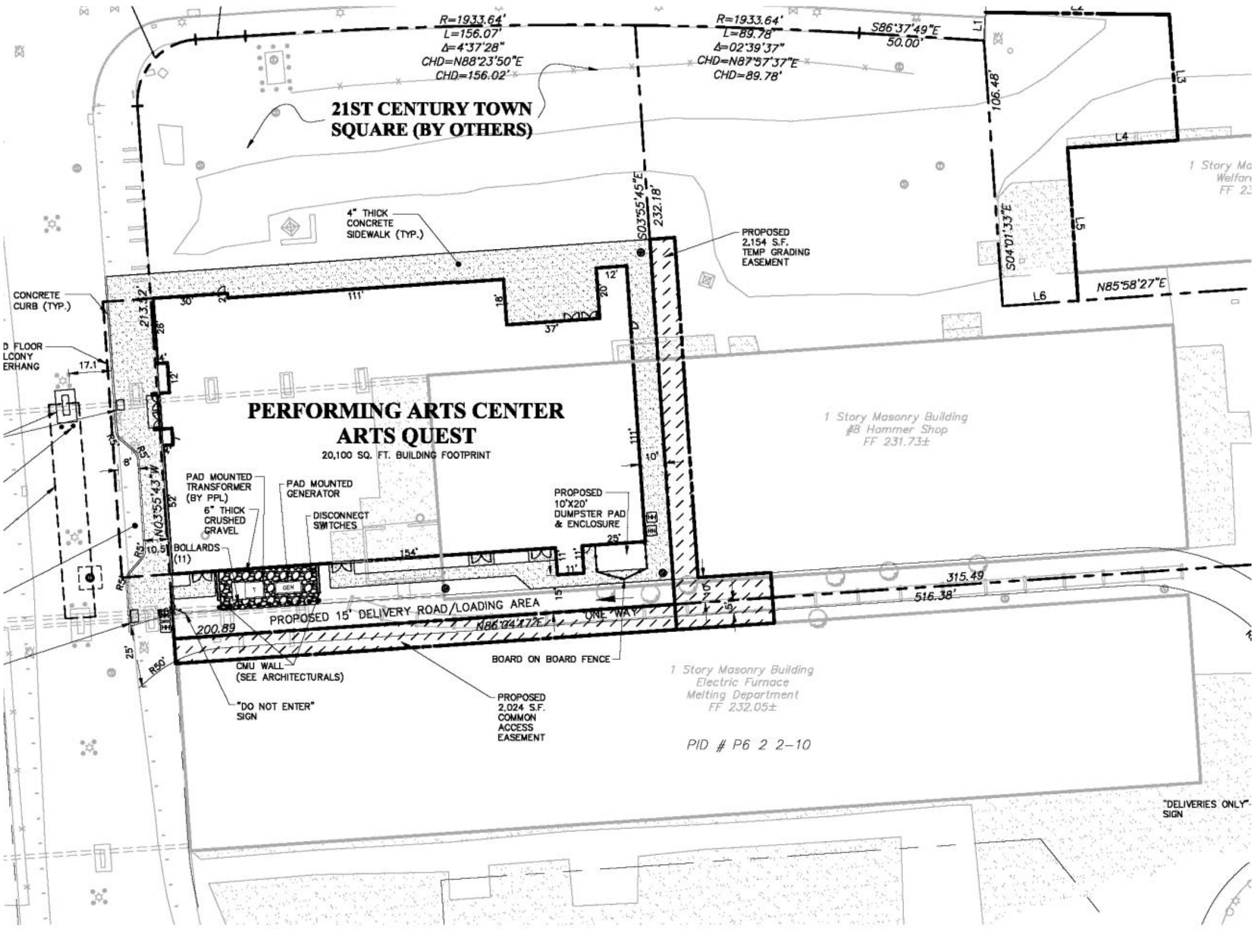
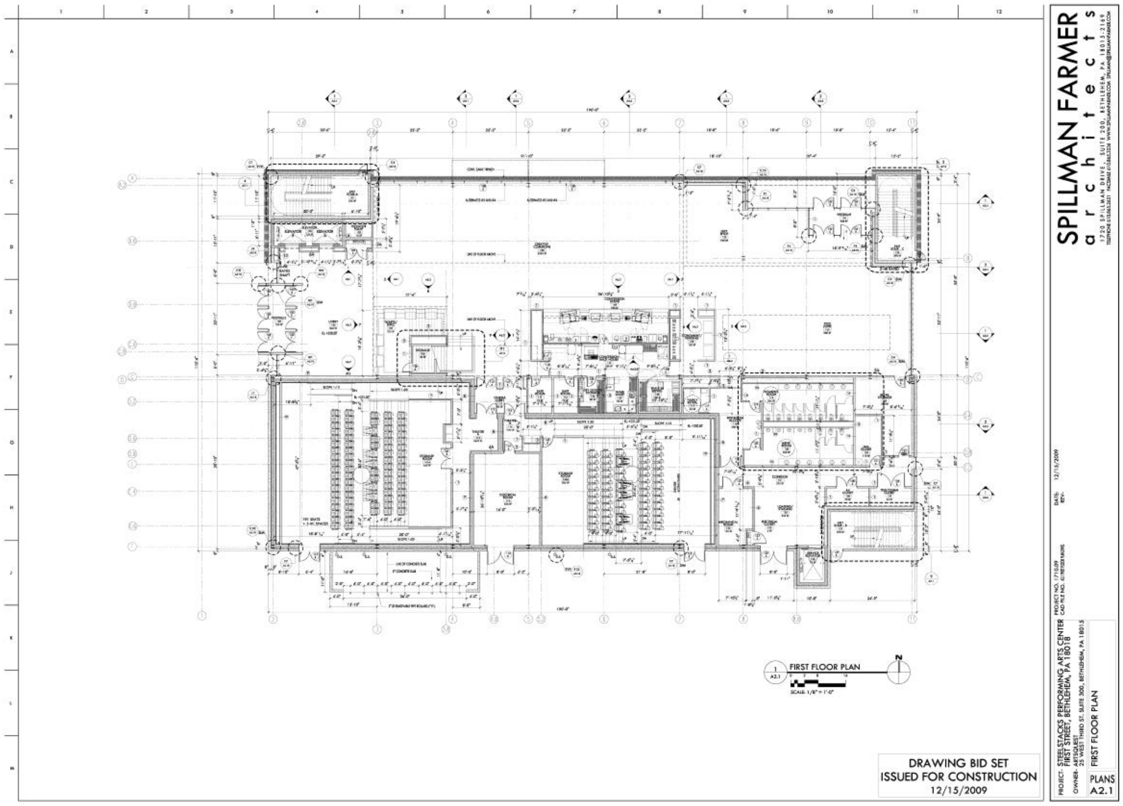
Appendix A Structural System Overview

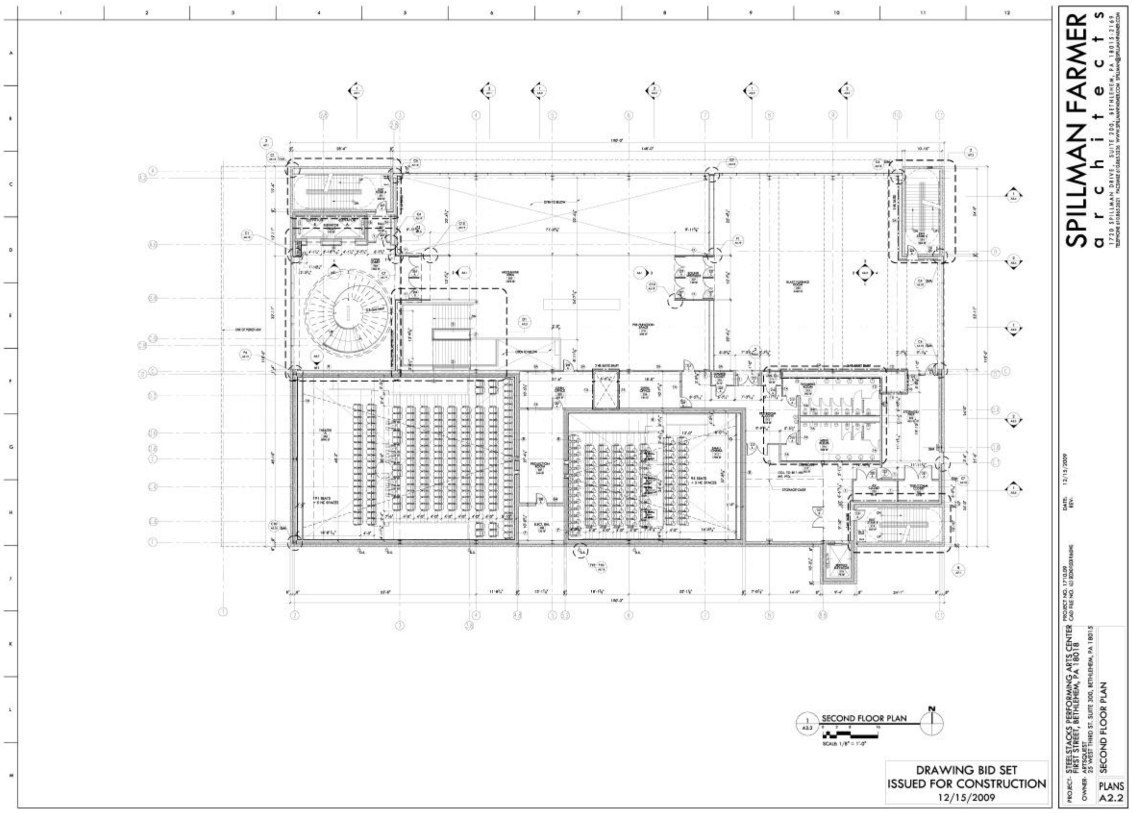
Site Plan Detail

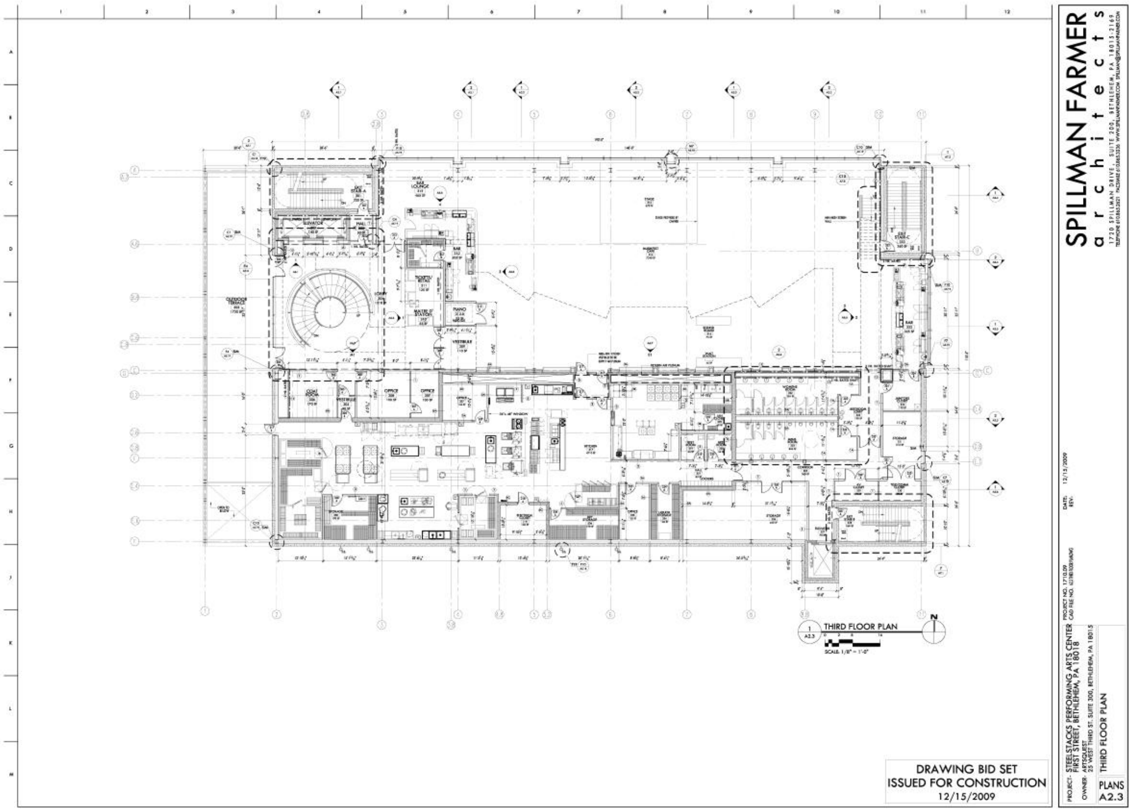
The location of the existing site at onset of project with current location overlaid.

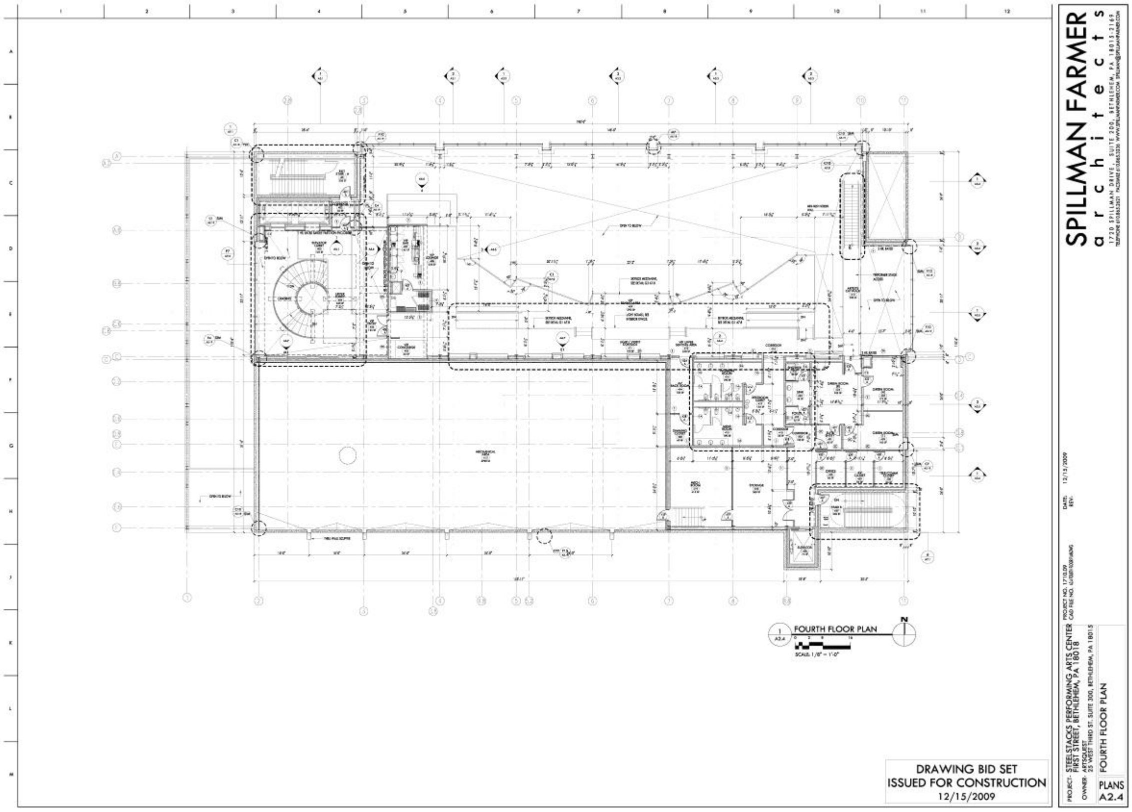


Architectural Floor Plans

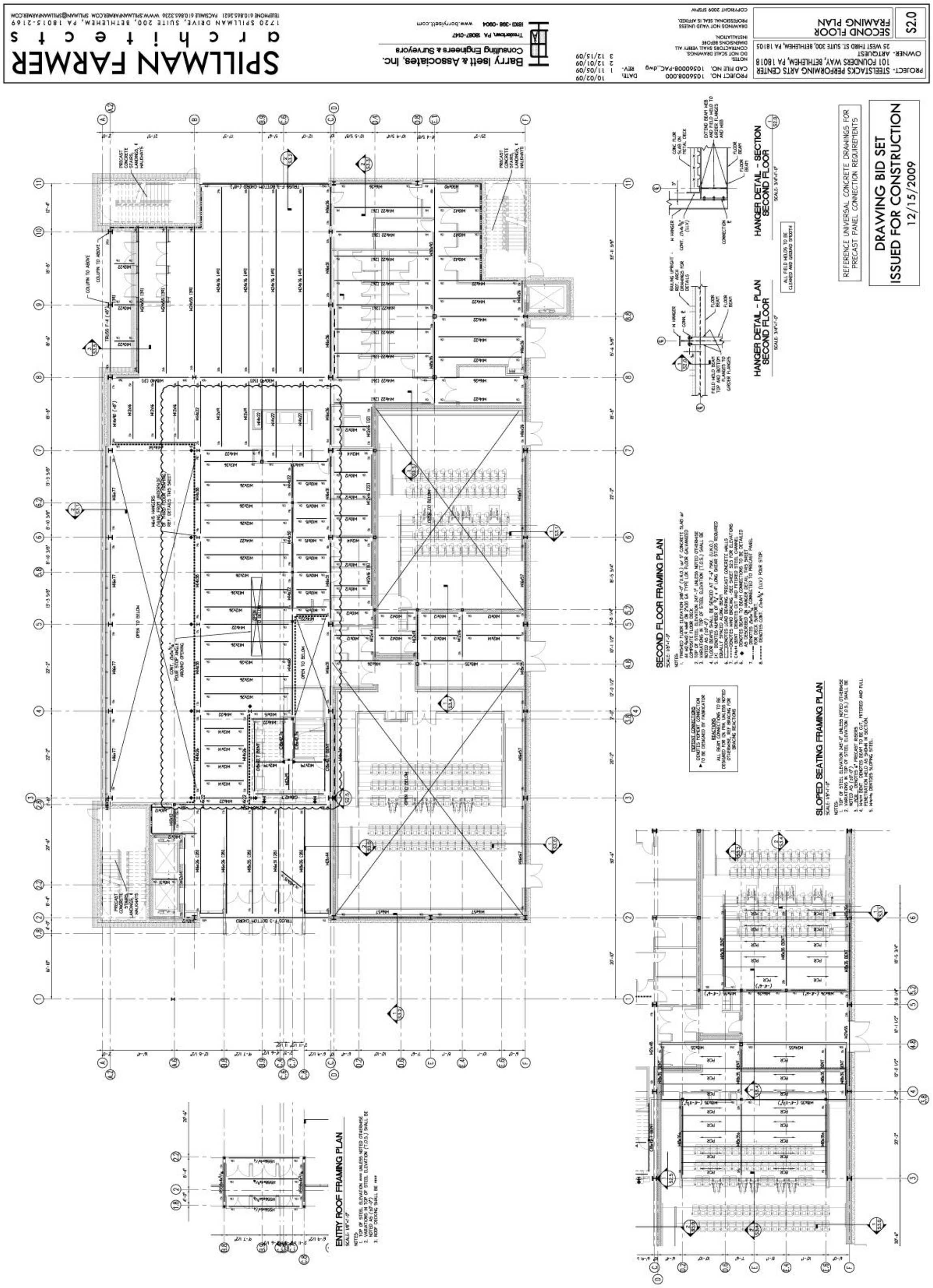




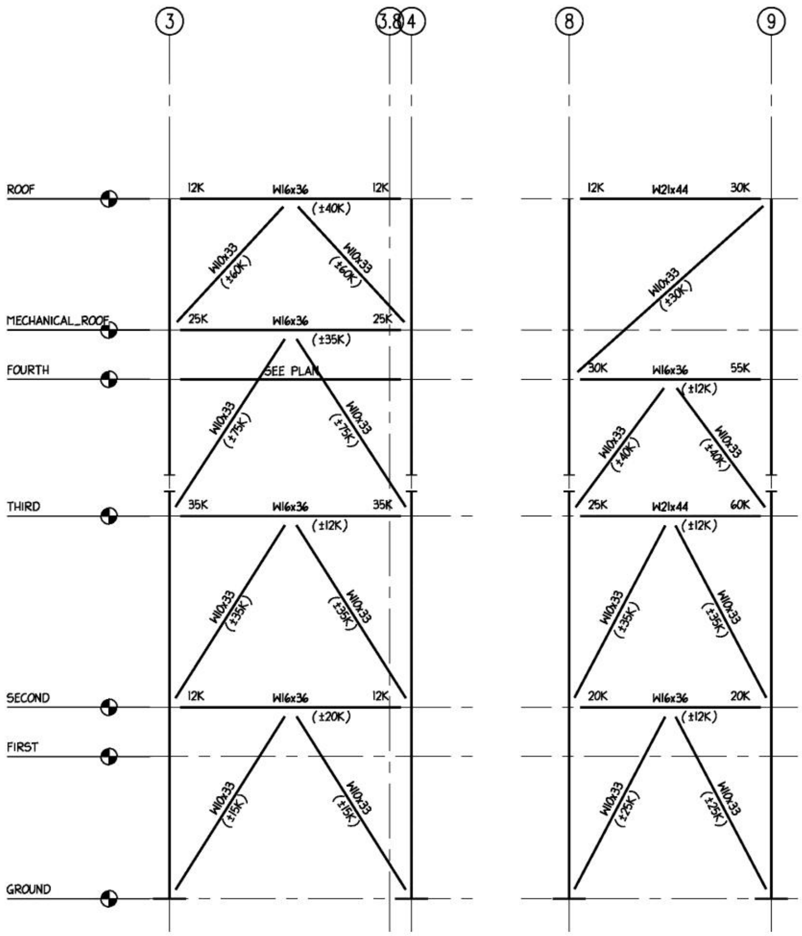

THIRD FLOOR PLAN
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009
SPILLMAN FARMER
architects
THIRD FLOOR PLAN
PLANS
A2.3


FOURTH FLOOR PLAN
DRAWING BID SET
ISSUED FOR CONSTRUCTION
12/15/2009
SPILLMAN FARMER
architects
FOURTH FLOOR PLAN
PLANS
A2.4

Structural Floor Plans



Lateral System



1 **ELEVATION AT LINE C**
SCALE: 1/8"=1'-0"

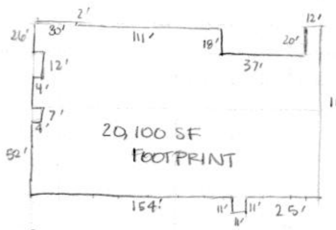
NOTES:
1. CONNECTIONS TO BE DESIGNED FOR FORCES INDICATED BY FABRICATORS ENGINEER.
2. (##k) DENOTES AXIAL FORCE IN MEMBER
   (+) TENSION
   (-) COMPRESSION
3. ##k DENOTES VERTICAL REACTION ON END OF BEAM

Appendix B Existing: Wind Calculations

Sarah Bednarcik | WIND LOAD CALCS | THESIS Sept 2012 | 1

USING: Chapter 6: Wind Loads ASCE 7-05
Method 2:

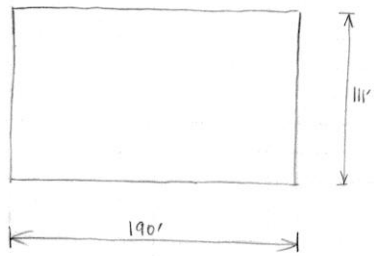
Bldg layout:



all units in feet

ROOF height = 64'0"

Simplified:



Lower roof height at 51'6" is surrounded by parapet, which goes to 64'0" elevation. Because this space is about 25% of the total roof sq. ft. To simplify this calculation, this is considerd at elevation 64'0". Thus, the roof is a flat roof at 64'0" with no parapet.

∴ mean roof height, $h = 64'$

For a N-S wind:
$L = 111$ ft
$B = 190$

For an E-W wind:
$L = 190$ ft
$B = 111$ ft

6.5.3 Design (Values) Procedure

1. Wind Speed $V = 90$ mph (Figure 6-1)
   Directionality Factor $K_d = .85$ (Table 6-4)
2. Importance factor, $I = 1.15$ (Table 6-1)
   Category III (by Table 1-1)
3. Exposure Category:
   by Section 6.5.6. – Surface Roughness B. → Exposure B
   Velocity Pressure Exp. Coeff (Table 6-3)
   Case 2, Exp B.
   interpolate $z = 60$ | .85 ; $z = 70$ | .89 → at $z = 64'$ $K_z = .87$
   other K_z values on excel.
4. Topographic factor, K_{zt}
   by Section 6.5.7 $K_{zt} = 1.0$
5. Gust effect factor G or G_f
   by Section 6.5.8
   rigid or flexible.
   $n_1 = \frac{100}{64} = 1.56$ C6-17
   $n_1 = \frac{75}{64} = 1.17$ C6-18
   ∴ treat as a rigid system. (follow §6.5.8.2)

RIGID

2/2

$$G = .925\left(\frac{1+1.7\, g_Q I_z Q}{1+1.7\, g_v I_z}\right)$$

$g_Q = g_v = 3.4$

from Table 6-2 $\alpha = 7.0$ $z_g = 1200$ $\hat{a} = 1/4.0$ $\hat{b} = 0.45$
$c = 0.3$ $\ell = 320$ $\ell = 320$ $\bar{\epsilon} = 1/3.0$

$I_z = c\left(\frac{33}{z}\right)^{1/6} = 0.30\left(\frac{33}{38.4}\right)^{1/6} = .293$ $\bar{z}_{max} = 6(64') = 38.4'$, 30'

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_z}\right)^{.63}}} = \sqrt{\frac{1}{1+.63\left(\frac{B+64}{336.6}\right)^{.63}}}$$

N-S. $B = 190$ ft | $B = 111$ ft **E-W**

$\therefore Q = .877$ | $\therefore Q = .910$

$$G = .925\left(\frac{1+1.7(3.4)(.293)(.877)}{1+1.7(3.4)(.293)}\right) = .853 \qquad = .925\left(\frac{1+1.7(3.4)(.293)(.910)}{1+1.7(3.4)(.293)}\right) = .873$$

6. enclosure - Fully enclosed

7. internal pressure, $GC_{pi} = \pm 0.18$ by Fig. 6-5

8. ext. pressure coefs.
Fig 6-6 walls: windward $C_p = .80$
leeward:
N/S $L/B = 111/190 = .584$ ⟶ $C_p = -.5$
E-W $L/B = 190/111 = 1.71$ ⟶ $C_p = -.36$ (interpolated)
Side $C_p = -.70$
roof:

θ=0° N-S				θ=0 E-W
.5 < h/L < 1.0	−1.0	0 to h/2	−.9	h/L ≤ .0.5
∴ interpolate	−0.8	h/2 to h	−.9	↓
	−0.5	h to 2h	−.5	Roof Area = 190×111 > 1000 SF
	N/A	>2h	−.3	∴ Reduction factor = .8

9. velocity pressure q_z, q_h. §6.5.10
at h = 64':
$q_z = .00256\, K_z K_{zt} K_d V^2 I = 17.63$ – others on excel spreadsheet

10. MWFRS.
$p = q\,GC_p - q_i(GC_{pi})$
pressure:
$p = q_h G C_p - q_z G C_{pi}$ where $q = q_z$ for windward. All else $q = q_h$.

See excel for further calcs.

Overturning Moment/Base Shear North-South Direction

	Location	Height	Area Below (ft^2)	Area Above (ft^2)	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Windward	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Leeward	Factored Story Shear (k)	Overturning Moment (k-ft)	Factored Story Load
Windward Wall	Roof	64	1662.5	0	8.45	8.86	22.48	-10.69	-10.69	-28.44	50.93	3259	50.93
	Floor 4	46.5	1187.5	1662.5	7.43	8.45	36.61	-10.69	-10.69	-48.76	136.29	3969	85.37
	Floor 3	35	1662.5	1187.5	6.11	7.43	30.38	-10.69	-10.69	-48.76	215.43	2770	79.14
	Floor 2	17.5	1662.5	1662.5	5.80	6.11	31.69	-10.69	-10.69	-56.88	304.00	1550	88.58
	Ground	0	0	1662.5	0	5.80	15.44	0	-10.69	-28.44	347.89	0	43.88
	Width (ft)	190				Factored Total Base Shear (k):	347.89				Total Overturning Moment (k-ft):	11549	

Overturning Moment/Base Shear East-West Direction

	Location	Height	Area Below (ft^2)	Area Above (ft^2)	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Windward	Pressure Below (psf)	Pressure Above (psf)	Factored Story Load (k) Leeward	Factored Story Shear (k)	Overturning Moment (k-ft)	Factored Story Load
Windward Wall	Roof	64	971.25	0	8.72	9.14	13.55	-8.71	-8.71	-13.54	27.09	1734	27.09
	Floor 4	46.5	638.25	971.25	7.67	8.72	21.39	-8.71	-8.71	-22.44	70.92	2038	43.83
	Floor 3	35	971.25	638.25	6.30	7.67	17.63	-8.71	-8.71	-22.44	110.99	1402	40.07
	Floor 2	17.5	971.25	971.25	5.21	6.30	17.89	-8.71	-8.71	-27.08	155.97	787	44.97
	Ground	0	0	971.25	0	5.21	8.09	0	-8.71	-13.54	177.60	0	21.64
	Width (ft)	111				Factored Total Base Shear (k):	177.60				Total Overturning Moment (k-ft):	5962	

Appendix C Existing: Seismic Calculations

S Bednarcik | Seismic Calcs | TB

Ch 11 §11.4 Seismic Design Values — USING ASCE 7-05.

$S_s = 0.263g$
$S_1 = 0.062g$ — From Geotech Report

Site Class D
SDC C
Occ. Cate. IV — §11.4.2

Structure
- eccentrically braced steel frame in E-W
- Shear walls in N-S

by table 11.4-1 $F_a = 1.6$
table 11.4-2 $F_v = 2.4$

$S_{MS} = F_a \cdot S_s = 1.6(0.263) = .4208$ → $S_{DS} = \frac{2}{3} S_{MS} = .281$
$S_{M1} = F_v S_1 = 2.4(0.062) = .0148$ → $S_{D1} = \frac{2}{3} S_{M1} = .099$

$S_a = S_{DS}(0.4 + 0.6\,T/T_0)$
$T_0 = 0.2\, S_{D1}/S_{DS} = .071$
$T_S = S_{D1}/S_{DS} = .356$

$T_L = 6$ by Fig 22-15

Finding T_1 fundamental period of bldg:

$T_a = C_t h_n^x = 0.03(64)^{.75} = .6788$ by T. 12.8-2

$C_u = 1.7$ T. 12.8-1

$T \le C_u T_a = 1.7(.6788) = 1.15$ $< T_L$ ✓ ok

by 12.8.2, allowed to use T_a.

$C_s = \frac{S_{DS}}{R/I} = \frac{.281}{3/1.5} = .140$ ← compare to engr's value = .139 ✓ GOOD.

$C_s = .139 \le \frac{S_{D1}}{T(R/I)} = \frac{.099}{1.15(2)} = .042$ by § 12.8.1-1 (12.8-3) can use $C_s = .042$

$V_s = C_s \cdot W$

Calculate Bldg Weight

SBednarcik | Seismic calcs | 13

Building weight §12.8.3

Floor 2,4
5" slab & deck
· slab 50 PSF
· MEP 7 PSF
· framing 10 PSF | 67 PSF

Floor 3
8" slab & deck
· slab 81.5 PSF
· MEP 7 PSF
· framing 10 PSF | 105 PSF

Roof
Lt wt conc 5"
· snow 30 PSF
· slab 40 PSF
· MEP 10 PSF | 80 PSF

Mech Roof
5" slab & deck
· slab 50 PSF
· MEP* 20 PSF
· framing 10 PSF | 80 PSF

*takes into account rooftop units

PCF from AISC Table 17-13

Materials
cmu wall 8" thick·(8/12) 137 PCF = 92 PSF

Precast panels 8" thick·(8/12) 150 PCF = 100 PSF

Curtain wall system glass w/ alum. mullions. = 20 PSF as per specs & manuf.

Mechanical System
RTU 2400, 1000, 1350, 10550, 10625, 1400 | 27.3 k
ERU 1056, 1536, 1732, 4285 | 8.6 K

total = 35.9 k

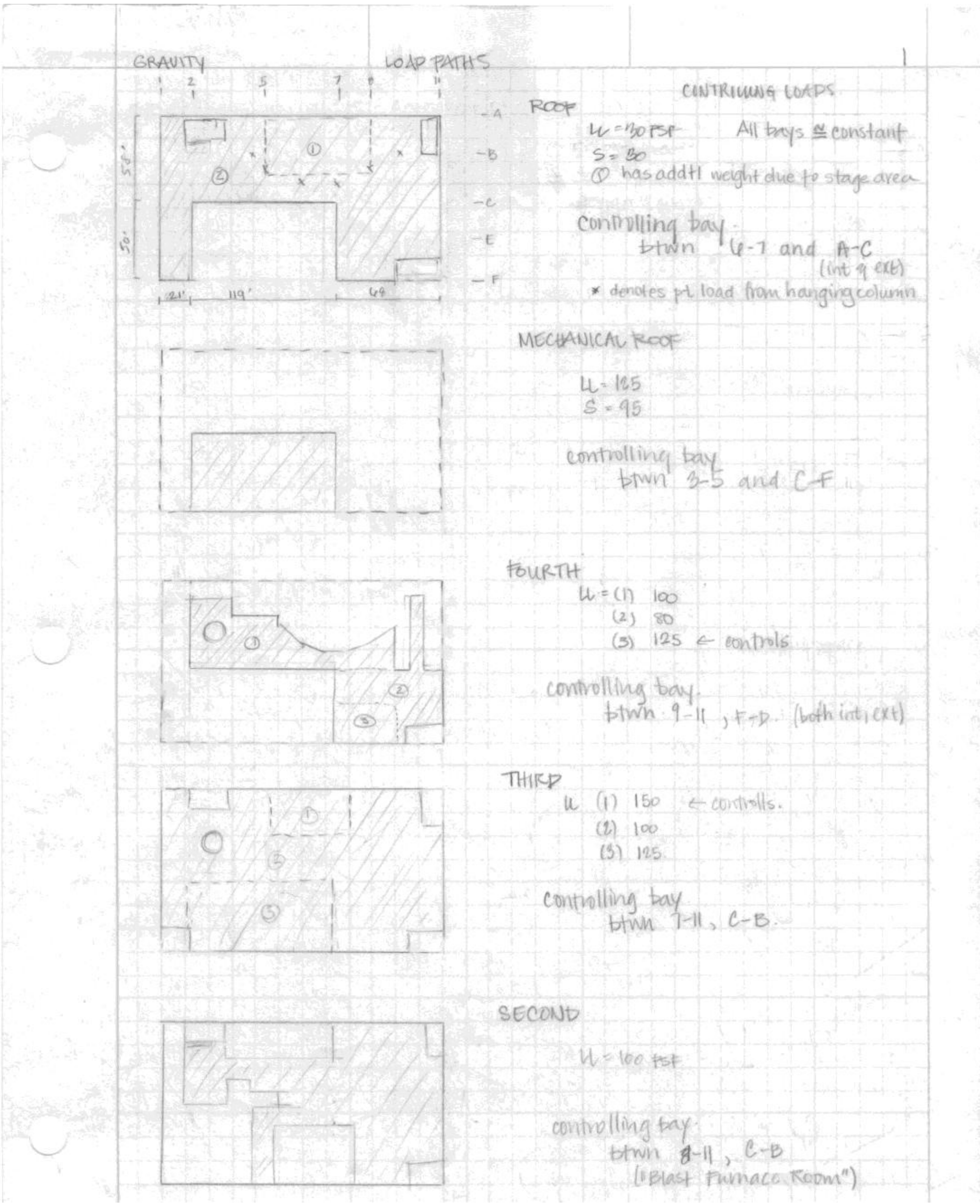
See excel for further calcs

Weight of Building	Area	PSF	Load (lbs)	Story Weight (lbs)
CMU	4310	91	392210	
Curtain Wall	2160	20	43200	
Concrete Panels	9610	100	961000	
Floor 2	12090	67	810030	2206440
CMU	9140	91	831740	
Curtain Wall	2160	20	43200	
Concrete Panels	9610	100	961000	
Floor 3	21060	105	2211300	4047240
CMU	5920	91	538720	
Curtain Wall	2300	20	46000	
Concrete Panels	6030	100	603000	
Floor 4	21060	67	1411020	2598740
Mechnical (RTU)			35934	35934
CMU	4520	91	411320	
Curtain Wall	3500	20	70000	
Concrete Panels	8530	100	853000	
Roof	17460	80	1396800	2731120
Columns	1870	70	130900	130900
			Total Weight (lbs)	11750374
			(k)	11750

Appendix D Gravity Redesign

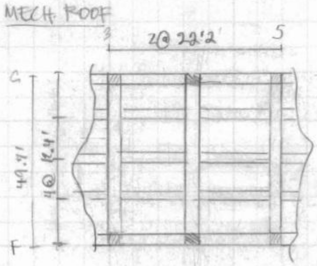
RC System

Hand Calculations:



2

MECH ROOF



W_L = 125 PSF

f'_c = 4000 psi
f_y = 60 ksi

Minimum slab thickness §9.5(a) ACI 318-11

int. bay $\frac{l}{28} = \frac{12.4' \times 12}{28} = 5.3''$

ext. bay. $\frac{l}{24} = \frac{12.4 \times 12}{24} = 6.2''$

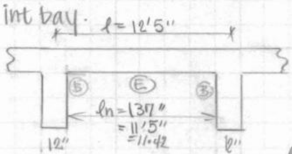
use a slab thickness of 7"

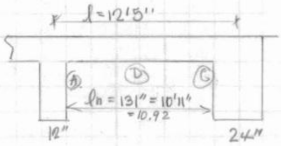
$W_D = (7/12'')(150\ PCF) = 87.5\ PSF + 20$ superimposed $= 107.5\ PSF$
W_L = 125 PSF (Mech)

$W_u = 1.2(107.5) + 1.6(125) = 329\ PSF$

Slab design:
Assume tension controlled section $\phi = 0.9$
Beam width 12" (strip)
$W_L = 125 < 3(W_D) = 3(107.5) = 323$ PSF ✓OK CAN USE ACI M. coeff.

int bay.





l_n avg = 11'2 = 11.17'

by §8.3

Ⓐ First int sup.
$\frac{-W_u l_n^2}{10} = \frac{-(329)(10.92)^2}{10} = -3923$ lb·ft/(ft width) ← controls

Ⓑ Other int sup.
$\frac{-W_u l_n^2}{11} = \frac{-(329)(11.42)^2}{11} = -3900$ lb·ft/ft width

Ⓒ ext sup.
$\frac{-W_u l_n^2}{24} = \frac{-(329)(10.92)^2}{24} = -1635$ lb·ft/ft width

Ⓓ Positive Mom. ext span
$\frac{+W_u l_n^2}{11} = \frac{329(10.92)^2}{11} = +3900$ lb ft/ ft width

Ⓔ Pos. Mom int span
$\frac{+W_u l_n^2}{16} = \frac{(329)(11.42)^2}{16} = +2682$ lb·ft / ft width

3

Reinforcement Assume $jd = 0.9d$ for int support

$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{M}{0.9 f_y jd} = \frac{-3923 \text{ lb ft} \times 12''}{0.9(60\,000)(0.9)(7.0)} = .14 \text{ in}^2/\text{ft width}$

where $d = h - c_c - d_b/2 = 8 - 3/4 - .5/2 = 7.0''$

assume #4 bars $d_b = .5$ $A_b = 0.20 \text{ in}^2$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(.14)(60)}{.85(4)(12'')} = .206$

$\therefore A_{s\,provd} = .20 \text{ in}^2 > A_{s\,req'd} = \frac{-3923 \times 12}{0.9(60000)(7 - .206/2)} = .13 \text{ in}^2$ ✓ok

$\rho_{req'd} = \frac{A_s}{bd} = \frac{.13 \text{ in}^2}{(12'')(7)} = .0015$

by S & T § 7.12. $\rho_{min} = .0018$ ← controls

Shear Check:

$V_u = \frac{1.15\, w_u l_n}{2} = \frac{1.15(329)(10.92)}{2} = 2066$ lb/ft of slab — ext face of 1st support

$V_u = \frac{w_u l_n}{2} = \frac{329(10.92)}{2} = 1796$ lb/ft width — other supports

$\phi V_c = \phi 2 \sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{4000}\; 12(7) = 7969$ lb/ft $\gg V_u$ ✓ok

Design Reinf

$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4000}}{60\,000} 12 \cdot 7 = .27$ — §10.5.1 & .4

$A_{s\,min}$ (transverse) $= \rho \cdot bd = .0018(12)(7) = .15$ — §7.12

$A_{s\,min} = 200\, b_w d / f_y = .28 \text{ in}^2/\text{ft width}$

Spacing — §10.6

S+T S_{min} { $5h = 5(8) = 40$; 18 ← controls }

$S_{max} = \min$ { $3h = 24$; 18 ← controls }

$S_{max} = \min$ { $15\left(\frac{40\,000}{f_s}\right) - 2.5c_c = 13.125$; $12\left(\frac{40\,000}{f_s}\right) = 12''$ } — $f_s = \frac{2}{3} f_y = 40$ ksi

> here 8" slab
> #4 bars @ 8" o.c.
> #4 bars @ 12" o.c. transverse reinf.

	1st int sup
l_n	10.92
M_u	-3923
$A_{s\,req'd}$	.13
$A_{s\,min}$	.27
	.28
(trans)	.17
bars	#4s @ 8"
$A_{s\,prov'd}$	.30 in²

others shown on following excel with calcs done using same method.

Slab Spreadsheet:

	Mechanical Roof			Roof			Fourth Floor	
h of slab		8	h of slab		6	h of slab		6
db	#4	0.5	db	#4	0.50	db	#5	0.5
Ab		0.2	Ab		0.20	Ab		0.2
d		7	d		5	d		5

Input	Equation/ Referenc	Mechanical Roof					Roof					Fourth Floor						
		Exterior Support	Exterior Span	Interior Support	Interior Span	Other Int Support	Exterior Support	Exterior Span	Interior Support	Interior Span	Other Int Support	Exterior Support	Exterior Span	Interior Support	Interior Span	Other Int Support	Exterior Support	Exterior Span
		Neg M	Pos M	Neg M	Pos M	Neg M	Neg M	Pos M	Neg M	Pos M	Neg M	Neg M	Pos M	Neg M	Pos M	Neg M	Neg M	Pos M
Minimum Slab thickness		6.2			5.3		5.82			4.99		3.3		4.33			5.05	
Used Slab Thickness		8					6					6						
ℓ		12.4	12.4	12.4	12.4	12.4	11.64	11.64	11.64	11.64	11.64	6.6	6.6	10.1	10.1	10.1	10.1	10.1
ℓ_n		10.92	10.92	10.92	11.42	11.42	10.14	10.14	10.14	10.64	10.64	5.1	5.1	8.6	8.6	8.6	8.6	8.6
Wll	psf	125					30					125						
Wu		329					138					290						
Moment Coeff		- 1/24	+ 1/11	-1/10	+1/16	-1/11	- 1/24	+ 1/11	-1/10	+1/16	-1/11	- 1/24	+ 1/11	-1/10	+1/16	-1/11	- 1/24	+ 1/11
		0.04	0.09	0.10	0.06	0.09	0.04	0.09	0.10	0.06	0.09	0.04	0.09	0.10	0.06	0.09	0.04	0.09
Mu	lb- ft	1634.7	3566.6	3923.2	2681.7	3900.6	591.2	1289.9	1418.9	976.4	1420.3	314.3	685.7	2144.8	1340.5	1949.9	893.7	1949.9
Intermediates																		
As		0.058	0.126	0.138	0.095	0.138	0.029	0.064	0.070	0.048	0.070	0.016	0.034	0.106	0.066	0.096	0.044	0.096
a		0.085	0.185	0.204	0.139	0.202	0.043	0.094	0.103	0.071	0.103	0.023	0.050	0.156	0.097	0.142	0.065	0.142
As req'd		0.052	0.115	0.126	0.086	0.126	0.026	0.058	0.064	0.044	0.064	0.014	0.031	0.097	0.060	0.088	0.040	0.088
Shear																		
Vu (ext supports)		2066	--	--	--	--	805	--	--	--	--	850	--	--	--	--	1434	--
(other)		--	1796	1796	1879	1879	--	700	700	734	734	--	740	1247	1247	1247	--	1247
ϕVc		7969	7969	7969	7969	7969	5692	5692	5692	5692	5692	5692	5692	5692	5692	5692	5692	5692
OK?		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Requirements																		
Asmin		0.27	0.27	0.27	0.27	0.27	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19
		0.28	0.28	0.28	0.28	0.28	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
S&T		0.15	0.15	0.15	0.15	0.15	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11
Spacing																		
Smax		18					18					18						
Smax		12					12					12						
S&T min		18					18					18						
Asreq'd		0.28	0.28	0.28	0.28	0.28	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
Bars				#4 @ 8					#4 @ 12									
As Prov'd (in2)				0.30					0.20									
Tranverse				#4 @ 12					#4 @ 12	<< match mech roof for construction ease								
As Prov'd (in2)				0.20					0.20									

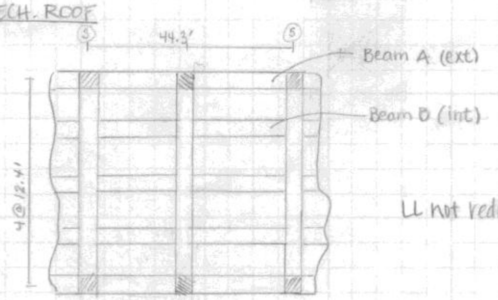
db	#5	0.5
Ab		0.2
d		5

db	#4	0.50
Ab		0.20
d		4

					Third Floor					Second Floor									
Interior Support	Interior Span	Other Int Support	Exterior Support	Exterior Span	Exterior Support	Exterior Span	Interior Support	Interior Span	Other Int Support	Exterior Support	Exterior Span	Interior Support	Interior Span	Other Int Support	Interior Span	Other Int Support	Interior Support	Exterior Support	Exterior Span
Neg M	Pos M	Neg M	Neg M	Pos M	Neg M	Pos M	Neg M	Pos M	Neg M	Neg M	Pos M	Neg M	Pos M	Neg M	Pos M	Neg M	Neg M	Neg M	Pos M
4.33			4.25		5.8			4.97		4.35			3.73		3.09			3.6	
					6					5									
10.1	8.5	8.5	8.5	8.5	11.6	11.6	11.6	11.6	11.6	8.7	8.7	8.7	8.7	8.7	7.2	7.2	7.2	7.2	7.2
8.6	7.5	7.5	7.0	7.0	10.1	10.1	10.1	10.6	10.6	7.2	7.2	7.2	7.7	7.7	6.2	6.2	5.7	5.7	5.7
					150					100									
					330					235									
-1/10	+1/16	-1/11	- 1/24	+ 1/11	- 1/24	+ 1/11	-1/10	+1/16	-1/11	- 1/24	+ 1/11	-1/10	+1/16	-1/11	+1/16	-1/11	-1/10	- 1/24	+ 1/11
0.10	0.06	0.09	0.04	0.09	0.04	0.09	0.10	0.06	0.09	0.04	0.09	0.10	0.06	0.09	0.06	0.09	0.10	0.04	0.09
2144.8	1019.5	1483.0	592.1	1291.8	1402.6	3060.3	3366.3	2317.4	3370.8	507.6	1107.5	1218.2	870.8	1266.7	564.6	821.2	763.5	318.1	694.1
0.106	0.050	0.073	0.029	0.064	0.069	0.151	0.166	0.114	0.166	0.031	0.068	0.075	0.054	0.078	0.035	0.051	0.047	0.020	0.043
0.156	0.074	0.108	0.043	0.094	0.102	0.222	0.244	0.168	0.245	0.046	0.101	0.111	0.079	0.115	0.051	0.075	0.069	0.029	0.063
0.097	0.046	0.067	0.026	0.058	0.063	0.139	0.153	0.105	0.154	0.028	0.062	0.069	0.049	0.071	0.032	0.046	0.043	0.018	0.039
--	--	--	1167	--	1916	--	--	--	--	973	--	--	--	--	--	--	--	770	--
1247	1088	1088	--	1015	--	1667	1667	1749	1749	--	846	846	905	905	729	729	670	--	670
5692	5692	5692	5692	5692	5692	5692	5692	5692	5692	4554	4554	4554	4554	4554	4554	4554	4554	4554	4554
OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16
0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
					18					15								18	
					12					12								12	
					18					18								18	
0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16
	#4 @ 12						#4 @ 12					#4 @ 12							
	0.20						0.20					0.20							
	#4 @ 12						#4 @ 12					#4 @ 12							
	0.20						0.20					0.20							

GRAVITY - BEAMS 6

MECH. ROOF



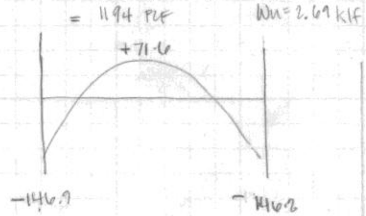
Assumptions
b = 24"
b = 12"
h_{slab} = 8"
LL not reducible

GRAVITY - BMS 6

Beam A: ext bm b = 24"

h = l/21 = 22.2/21 = 1.06 ≅ 14" try deeper for deflections

LL = 125 × 12.4/2 = 775 PLF
DL = Slab + Self wt + Superimp.
= 8/12 × 150 × 12.4/2 + 24/12 × 18/12 × 150 + 20 × 12.4/2
= 1194 PLF
Wu = 2.69 klf



Midspan try #5s
d = h − cc − db/2
= 26 − 1.5 − 0.625/2
= 24.19

As = Mu/4d = 71.6/4(24.19) = .74 in²

try (7) #5s As = 2.17
a = 2.17 (60) / .85 (4) (24) = 1.60
c = a/β = 1.88
εs = εu (d − c)/c = 0.04 ✓ok ∴ φ = 0.9

φMn = As fy (d − a/2)
= φ 2.17 (60) (24.19 − 1.60/2)
= 228.4 kft

φMn = 228.4 > Mu = 71.6 kft ✓ok

Asmin = max { 3√f'c bwd/fy = 1.83 ; 200 bw d/fy = 1.93

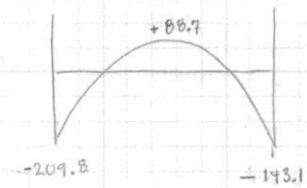
∴ (7) #5s ok

Beam B: Int bm

l = 22.2

= 1550 PLF
= 8/12 × 150 × 12.4 + 2 × 18/12 × 150 + 20 × 12.4
= 1938 plf

Wu = 1.2D + 1.6L
= 4.84 klf



VALUES CONFIRMED VIA STAAD

Midspan try #5s
d = h − Cc − db/2
= 24.19

As = 88.7/4(24.19) = 0.92

try (7) #5s As = 2.17
a = 1.60
c = 1.88
εs = 0.04 ✓ok ∴ φ = 0.9

φMn = 0.9 (2.17) (60) (24.19 − 1.60/2)
= 228.4 kft

φMn = 228.4 > Mu = 88.7

Asmin = { 1.83 ; 1.93 ✓ok

7

@ Supports: Left = Right

Mu = 146.9

As = Mu/4d = 146.9/4(24.19) = 1.53

try (4) #7s

As = 2.40

a = Asfy/.85f'cb = 1.76

→ c = 2.08

εs = 0.03

∴ φ = 0.9

φMn = φAsfy (d − a/2) = 250.3

φMn = 250.3 > Mu = 146.9 kft

✓ok

Vertical Shear

Vn = Wuln/2 = 2.69 (21.2)/2 = 28.51

φVn = 0.75 (2√4000 (24.06) + 2 (.2)(60^k)(24.06)/12)

= 74.6k

φVn = 74.6 K > Vu = 28.5

✓ok

using 2#3s @ 12" o.c.

@ Supports: Left

Mu = 209.8

As = 209.8/4(24.19) = 2.19 in²

try (3) #8s As = 2.37 in²

a = 2.37(60)/.85 (4)(24) = 1.61

∴ c = 1.89 → εs = .04 φ0.9

φMn = 0.9(2.37)(60)(24.0 − 1.61/2) = 247.4

φMn = 247.4 > Mu = 209.8 kft

✓ok

@ Sup. : Right

Mu = 143.1

As req'd = 143.1/2(24.19) = 1.49

try (2) #8s. As = 1.58 in²

a = 1.16 → c = 1.37

εs = 0.05 φ = 0.9

φMn = 0.9 (1.58)(60)(24 − 1.16/2) = 166.5

φMn = 166.2 > Mu = 143.1 ftk

✓ok

Vertical Shear

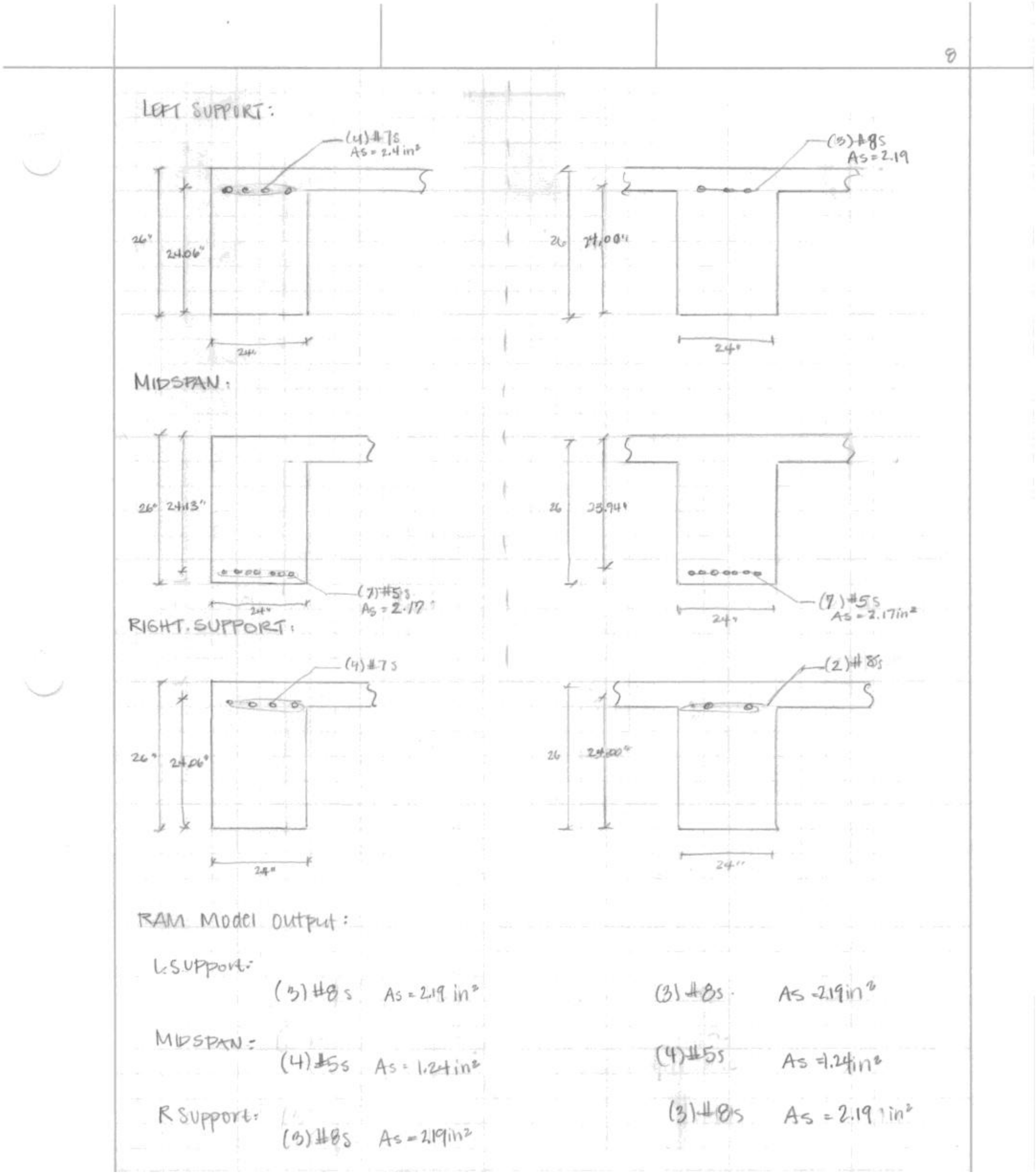
Vu = 4.84 × 21.2/2 = 51.30

φVn = 0.75 (2√4000 · 24 (24.06) + .4 (60^k)(24)(12))

= 90.64 k > 51.3 k

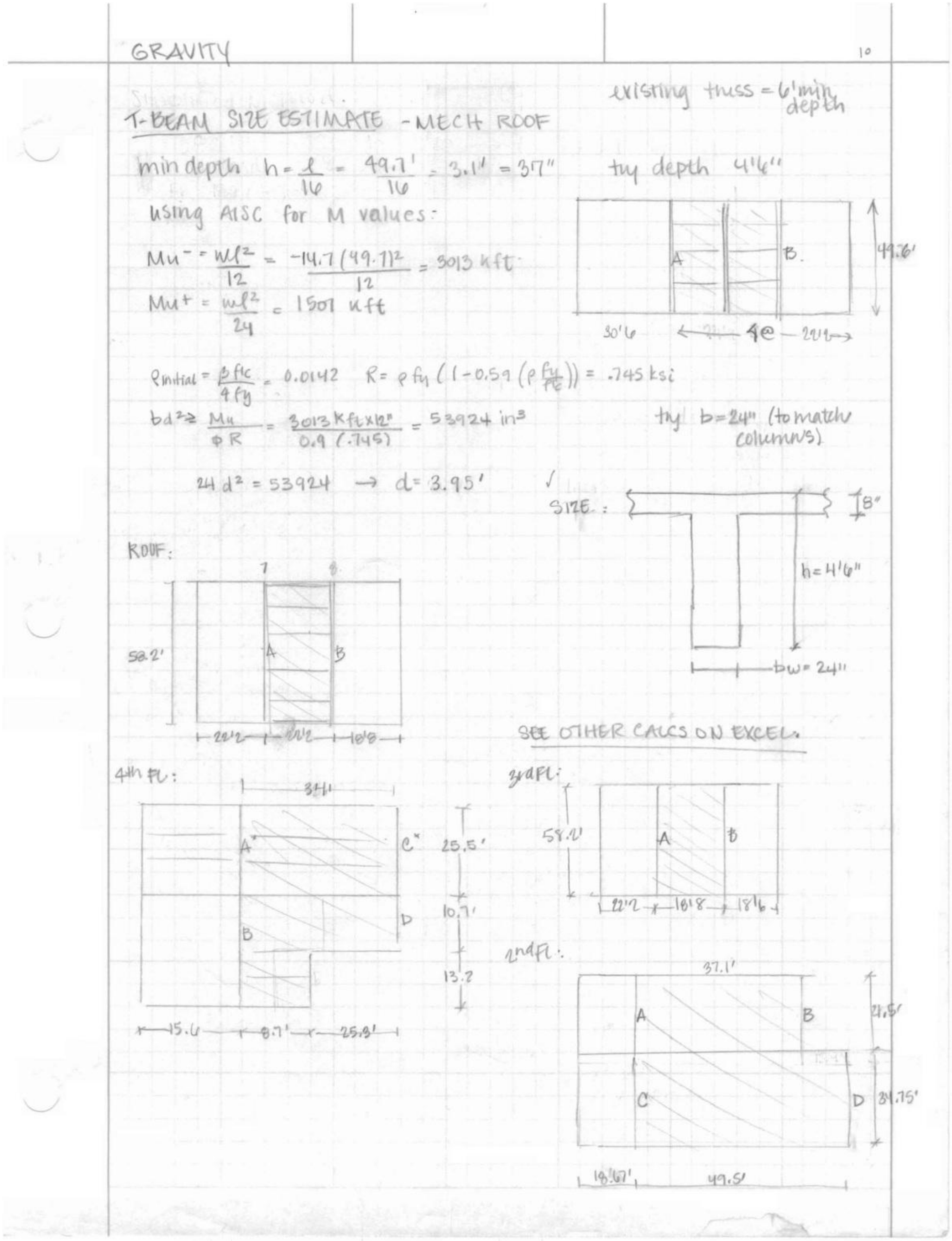
✓ok

using 2#3s @ 12" o.c.


LEFT SUPPORT:
(4) #7s
As = 2.4 in²
(3) #8s
As = 2.19
MIDSPAN:
(7) #5s
As = 2.17
(7) #5s
As = 2.17in²
RIGHT SUPPORT:
(4) #7s
(2) #8s
RAM Model Output:
L.Support:
(3) #8s As = 2.19 in²
(3) #8s As = 2.19in²
MIDSPAN:
(4) #5s As = 1.24 in²
(4) #5s As = 1.24in²
R Support:
(3) #8s As = 2.19in²
(3) #8s As = 2.19 in²

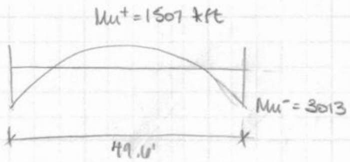
Excel Spreadsheet:

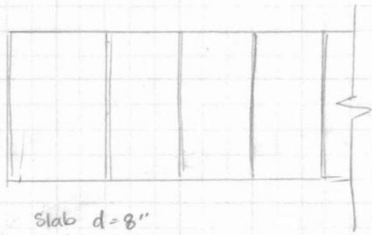
		Mechanical Roof		Roof		Fourth Floor				Third Floor		Second Floor	
Input	**Units**	**Beam A (exterior)**	**Beam B (interior)**	**Beam A (exterior)**	**Beam B (interior)**	**Beam A (exterior)**	**Beam B (interior)**	**Beam C (exterior)**	**Beam E (exterior)**	**Beam A (exterior)**	**Beam B (interior)**	**Beam B (interior)**	**Beam C (interior)**
h slab		8	8	6	6	6	6	6	6	6	6	5	5
b		24.0	26.0	24.0	24.0	24.0	24.0	20.0	20.0	20.0	20.0	24.0	30.0
hmin=ℓ/21	in	12.11	12.11	12.11	12.11	18.29	18.29	13.31	4.40	10.11	10.11	20.63	27.71
Used beam depth	in	26.0	30.0	14.0	20.0	26.0	26.0	24.0	12.0	24.0	24.0	26.0	34.0
ℓ	ft	22.2	22.2	22.2	22.2	33	33	24.3	8.7	18.7	18.7	37.1	49.5
ℓ_n	ft	21.2	21.2	21.2	21.2	32	32	23.3	7.7	17.7	17.7	36.1	48.5
trib width	ft	6.2	12.4	5.82	11.64	6.375	11.425	11.05	6.6	5.8	11.6	10.8	11.6
Wu	plf	2690	4840	2690	4840	2690	2690	2690	2690	2690	1869.92	4840	4840
Mu- (Left)	k-ft	146.9	209.8	74.13	51.55	132.55	249.99	168.59	5.02	55.48	76.78	284.48	695.78
Mu+	k- ft	71.60	88.70	36.37	17.94	131.63	301.86	137.87	5.77	22.38	36.61	200.71	581.51
Mu- (Right)	k- ft	146.20	143.10	70.92	52.48	180.94	217.20	124.64	6.95	46.55	132.19	132.91	186.33
Midspan (Bottom)													
Trial Bar Size		# 5	# 5	# 5	# 5	# 7	# 7	# 7	# 5	# 6	# 6	# 9	# 10
db		0.625	0.625	0.625	0.625	0.875	0.875	0.875	0.625	0.750	0.750	1.128	1.270
Ab		0.310	0.310	0.310	0.310	0.600	0.600	0.600	0.310	0.440	0.440	1.000	1.270
d		24.19	28.19	12.19	18.19	24.06	24.06	22.06	10.19	22.13	22.13	23.94	31.87
As req'd		0.74	0.79	0.75	0.25	1.37	3.14	1.56	0.14	0.25	0.41	2.10	4.56
Min # of bars		3	2	3	2	3	2	3	3	3	2	2	3
Max # of bars		11	3	11	4	11	4	11	11	11	4	4	9
# of bars		7	7	6	3	4	6	3	3	4	4	4	6
As prov'd		2.17	2.17	1.86	0.93	2.40	3.60	1.80	0.93	1.76	1.76	4.00	7.62
a		1.60	1.47	1.37	0.68	1.76	2.65	1.59	0.82	1.55	1.55	2.94	4.48
c		1.88	1.73	1.61	0.80	2.08	3.11	1.87	0.97	1.83	1.83	3.46	5.27
εs		0.04	0.05	0.02	0.06	0.03	0.02	0.03	0.03	0.03	0.03	0.02	0.02
ϕMn	k-ft	228.40	268.06	96.29	74.68	250.35	368.37	172.27	40.92	169.08	169.08	404.38	1015.80
CHECK		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Asmin		1.84	2.32	0.92	1.38	1.83	1.83	1.40	0.64	1.40	1.40	1.82	3.02
Asmin		1.94	2.44	0.98	1.46	1.93	1.93	1.47	0.68	1.48	1.48	1.91	3.19
Supports (Top) LEFT													
Trial Bar Size		# 7	# 8	# 7	#6	# 6	# 8	# 7	# 5	# 7	# 7	# 7	# 7
db		0.875	1.000	0.875	0.750	0.750	0.750	0.875	0.625	0.875	0.875	0.875	0.875
Ab		0.600	0.790	0.600	0.440	0.440	0.440	0.600	0.310	0.600	0.600	0.600	0.600
d		24.06	28.00	12.06	18.13	24.13	24.13	22.06	10.19	22.06	22.06	24.06	32.06
As req'd		1.53	1.87	1.47	0.72	1.88	2.25	1.41	0.17	0.53	1.50	1.38	1.45
Min # of bars		3	2	3	2	3	2	3	3	3	2	2	3
Max # of bars		11	3	11	4	11	4	11	11	11	4	4	11
# of bars		4	3	3	4	5	5	3	3	3	4	4	5
As prov'd		2.40	2.37	1.80	1.76	2.20	2.20	1.80	0.93	1.80	2.40	2.40	3.00
a		1.76	1.27	1.32	0.53	1.62	1.62	1.59	0.15	1.59	1.32	1.02	1.76
c		2.08	1.50	1.56	0.63	1.90	1.90	1.87	0.18	1.87	1.55	1.19	2.08
εs		0.03	0.05	0.02	0.08	0.04	0.04	0.03	0.17	0.03	0.04	0.06	0.04
ϕMn		250.3	291.8	92.3	141.4	230.8	230.8	172.3	42.3	172.3	231.1	254.4	420.9
CHECK		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Asmin		1.83	2.30	0.92	1.38	1.83	1.83	1.40	0.64	1.40	1.40	1.83	3.04
Asmin		1.93	2.43	0.97	1.45	1.93	1.93	1.47	0.68	1.47	1.47	1.93	3.21
Supports (Top) RIGHT													
Trial Bar Size		# 7	# 8	# 7	#6	# 6	# 6	# 7	# 5	# 7	# 7	# 7	# 7
db		0.875	1.000	0.875	0.750	0.750	0.750	0.875	0.625	0.875	0.875	0.875	0.875
Ab		0.600	0.790	0.600	0.440	0.440	0.440	0.600	0.310	0.600	0.600	0.600	0.600
d		24.06	28.00	12.06	18.13	24.13	24.13	22.06	10.19	22.06	22.06	24.06	32.06
As req'd		1.52	1.28	1.47	0.72	1.88	2.25	1.41	0.17	0.53	1.50	1.38	1.45
Min # of bars		3	2	3	2	3	2	3	3	3	2	2	3
Max # of bars		11	3	11	4	11	4	11	11	11	4	4	11
# of bars		4	2	4	4	4	5	3	3	3	4	4	5
As prov'd		2.40	1.58	2.40	1.76	1.76	2.20	1.80	0.93	1.80	2.40	2.40	3.00
a		1.76	1.07	8.65	6.39	10.35	19.41	10.59	1.00	10.59	13.22	12.18	17.65
c		2.08	1.26	10.17	7.51	12.18	22.84	12.46	1.18	12.46	15.55	14.33	20.76
εs		0.03	0.06	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00
ϕMn		250.3	195.3	83.6	118.3	150.1	142.7	135.8	40.5	135.8	166.9	194.1	313.7
CHECK		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Asmin		0.23	0.18	0.11	0.11	0.23	0.15	0.21	0.10	0.21	0.14	0.15	0.30
Asmin		0.24	0.19	0.12	0.12	0.24	0.16	0.22	0.10	0.22	0.15	0.16	0.32
Vertical Shear													
Vu	(k)	28.51	51.30	28.51	51.30	43.04	43.04	31.34	10.36	23.81	16.55	87.36	117.37
ϕVc		74.64	92.16	37.42	68.46	74.64	74.64	60.06	27.73	60.06	60.06	126.31	186.28
CHECK		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Results													
Bottom bars @ Midspan (+)		7 # 5	7 # 5	6 # 5	3 # 5	4 # 7	6 # 7	3 # 7	3 # 5	4 # 6	4 # 6	4 # 9	6 # 10
Top bars @ LEFT Support (-)		4 # 7	3 # 8	3 # 7	4 #6	5 #6	5 # 8	3 # 7	3 # 5	3 # 7	4 # 7	4 # 7	5 # 7
Top bars @ RIGHT Support (-)		4 # 7	2 # 8										
Shear Reinforcing		# 3 @ 12	# 3 @ 12	# 3 @ 12	# 3 @ 12	# 3 @ 12	# 3 @ 12	# 3 @ 12	# 3 @ 12	# 3 @ 12	# 3 @ 12	# 4 @ 12	# 4 @ 12
b		24.0	26.0	24.0	24.0	24.0	24.0	20.0	20.0	20.0	20.0	24.0	30.0
d req'd		24.19	28.19	12.19	18.19	24.13	24.13	22.06	10.19	22.13	22.13	24.06	32.06
h		26	26	14	14	20	20	20	20	20	20	22	28
Shear Reinforcing			2 # 3s @ 12"		2 # 4s @ 12"								
		Mechanical Roof		Roof		Fourth Floor				Third Floor			


GRAVITY
10
T-BEAM SIZE ESTIMATE - MECH ROOF
existing truss = 6' min depth
min depth h = l/16 = 49.7'/16 = 3.1' = 37"
try depth 4'6"
using AISC for M values:
Mu⁻ = wl²/12 = -14.7(49.7)²/12 = 3013 kft
Mu⁺ = wl²/24 = 1507 kft
ρinitial = βf'c/4fy = 0.0142 R = ρfy(1-0.59(ρfy/f'c)) = .745 ksi
bd² ≥ Mu/φR = 3013kft×12"/0.9(.745) = 53924 in³
try b=24" (to match columns)
24 d² = 53924 → d = 3.95'
SIZE:
8"
h=4'6"
bw=24"
ROOF:
4th FL:
SEE OTHER CALCS ON EXCEL.
3rd FL:
2nd FL:

GRAVITY – GIRDERS

MECH RF:





∴ size estimated (see previous) at t = 24" b = 4'6"

Wt self effects:

$W_{s.w.} = (24/12)(4.5 - 8/12) \times 150 = 1150$ PLF

$\omega_u = 14.7 + 1.2(1150)/1000 = 16.1$ ∴ $M_u^- = 3314$ kft, $M_u^+ = 1657$

req'd steel:

$A_s = \frac{M_u}{4d} = \frac{3314}{4(4.5' \times 12")} = 15.3$ in² (using h as estimate) try 2 layers of (7) #10's each layer

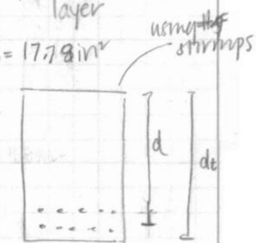
$A_s = 17.78$ in²

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{17.78 \cdot 60}{.85(4)(24)} = 13.07$ → c = 15.38

$\phi M_n = \phi A_s f_y (d - a/2)$
$= (0.9)\,17.78\,(60)\,(49.97 - 13.07/2) = 41703$ kin
$= 3475$ kft

$\phi M_n = 3475$ kft > $M_u = 3314$ kft

Midspan ok



$d_t = 4.5 \times 12 - 1.5 - 5/8 - 1.27/2 = 51.24"$

$d = 4.5 \times 12 - 1.5 - 5/8 - 1.27 \times 1.5 = 49.97"$

ck ductility

$\varepsilon_s = .003(49.97 - 15.38)/15.38 = .0067 > .005$ ok

Ends:

$A_s = 7.65$ in² try 1 layer (7) #10s → $A_{sp} = 8.89$ in²

$a = 6.54$ → $c = 7.69$ → $\varepsilon_s = .0169 > .005$ ok

$\phi M_n = 0.9(8.89)\,60\,(51.24 - 6.54/2) = 23029$ kin
$= 1919$ kft > $M_u = 1657$ ✓ ok

2

Ck min:

$A_{s,min} = 200\,(24)(49.97)/60 = 4.0\,in^2$ ok for both

$V_u = \frac{w_u l_n}{2} = \frac{16.1\,klf\,(49.6' - 2)}{2} = 383\,k$

$\phi V_n = 0.75\left(\frac{2\sqrt{4000} \times 24\,(49.97)}{1000} + V_s\right) \geq 383\,k$

$113.8\,k + V_s \geq 383\,k \rightarrow V_s > 270\,k < 8\sqrt{f'_c}\,b_w d$ ✓ok

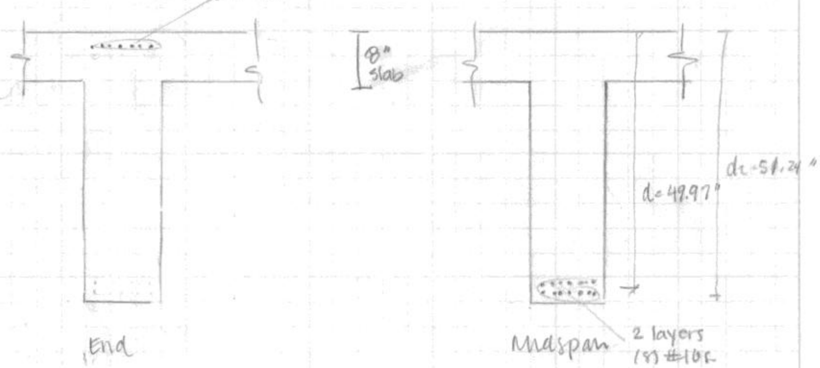
$V_s = A_v f_{yt} \frac{d}{s}$

$s = A_v f_{yt} \frac{d}{V_s} = \frac{0.31(60)(49.97)}{270} = 3.4"$ (use #5s) → use 3" spacing

start w/ #5s @ 3" spacing typ.

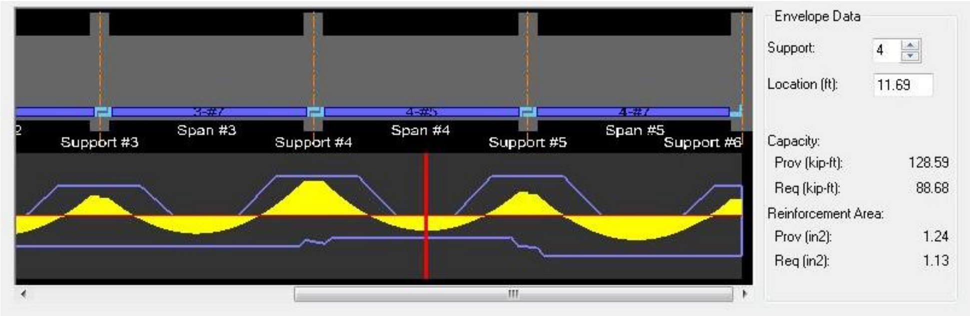
check: $113.8 + 309.8 = 423.6 > 383\,k$ ✓ok

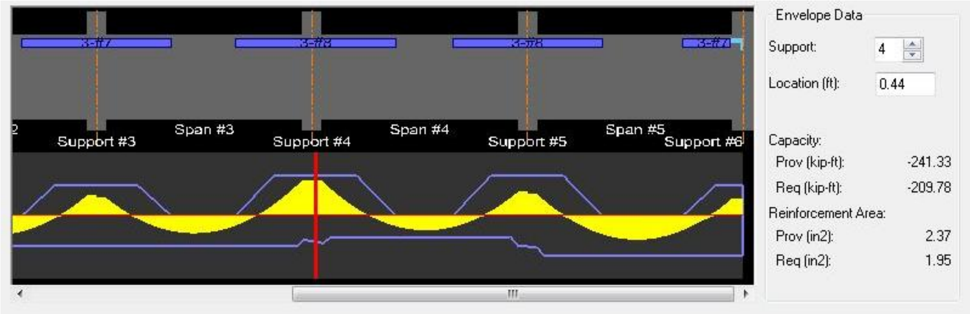
Design:

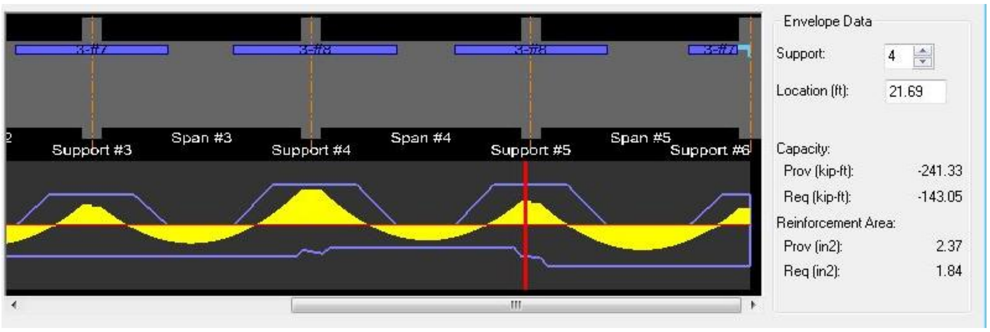


comparable to RAM design output

Interior Beam E-5 to E-6





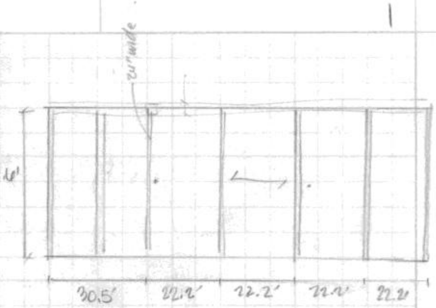


Prestressed System:

1

GRAVITY - P.T. DESIGN

MECH ROOF SYSTEM:



AS previously, FOR GIRDER

$M_u^- = 3013$ k ft

$M_u^+ = 1507$ k ft

FOR SLAB

$M_u^- = \frac{-w l_n^2}{11} = \frac{-329\,(22.2 - 24/12)^2}{11} = -12.2$ k ft

$M_u^+ = \frac{w_u l_n^2}{16} = \frac{+329\,(22.2 - 24/12)^2}{16} = +8.4$ k ft

compr stresses - / tension

$\sigma_{ci} = 0.6 f'_{ci}$ initial — σ_{ti} $3\sqrt{f'c}$

$\sigma_{csus} = 0.45 f'_{ci}$ sus — $\sigma_{tsus} = 6\sqrt{f'c}$

$\sigma_{tot} = 0.6 f'_{ci}$ — $\sigma_{tot} = 6\sqrt{f'c}$

see ADAPT-PTVS 8 output →

load	Deflection (")
DL	+.21
SDL	+.04
LL	+.26
PT	-0.19
sus LL	+.09

deflection check:

by inspection, passes

total load: $\frac{l}{240} = \frac{22.2 \times 12}{240} = 1.11" \geq .21 + .04 + .26 - .19 = .32"$

live load: $\frac{l}{360} = \frac{22.2 \times 12}{360} = 0.74" \geq 0.26"$

sust. load: $\frac{l}{480} = 0.56" \geq .21 + .04 + .09 - .19 = .15"$

GOOD FOR SERVICEABILITY & STRENGTH ✓

2

GIRDER:

$M_u^- = 3013$ k ft

$M_u^+ = 1507$ k ft

$l = 49.6$

span, b = 24"

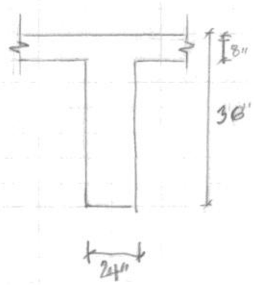
trib w = 22.2' total

$h = 3.5' \times 12 = 42"$

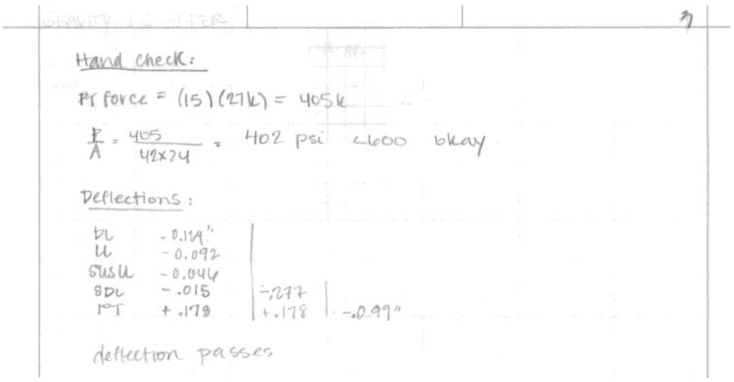
by §8.12 $\leq \frac{1}{4}(l) = 12.4' = 148.8"$

$f'_c = 5000$ ∴ $E = 4030e3$

$f'_{ci} = 4000$



deflection passes by inspection


Hand check:

PT force = (15)(27k) = 405k

P/A = 405 / 42x24 = 402 psi < 600 okay

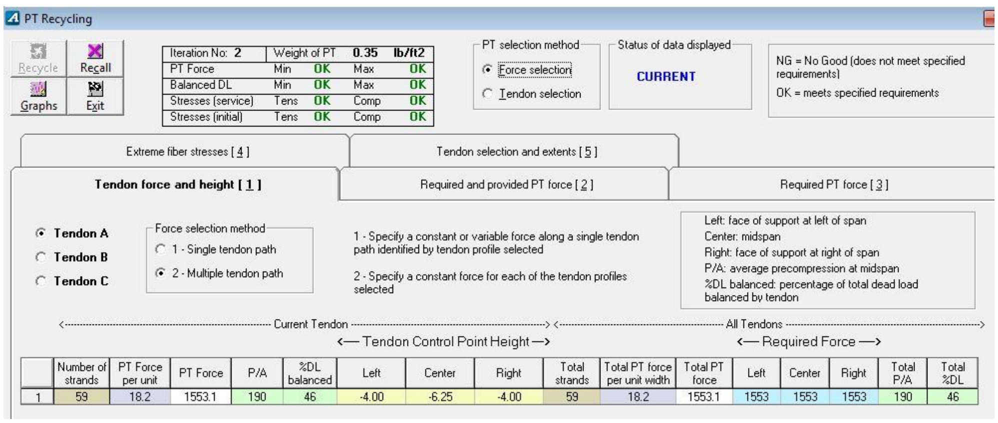
Deflections:

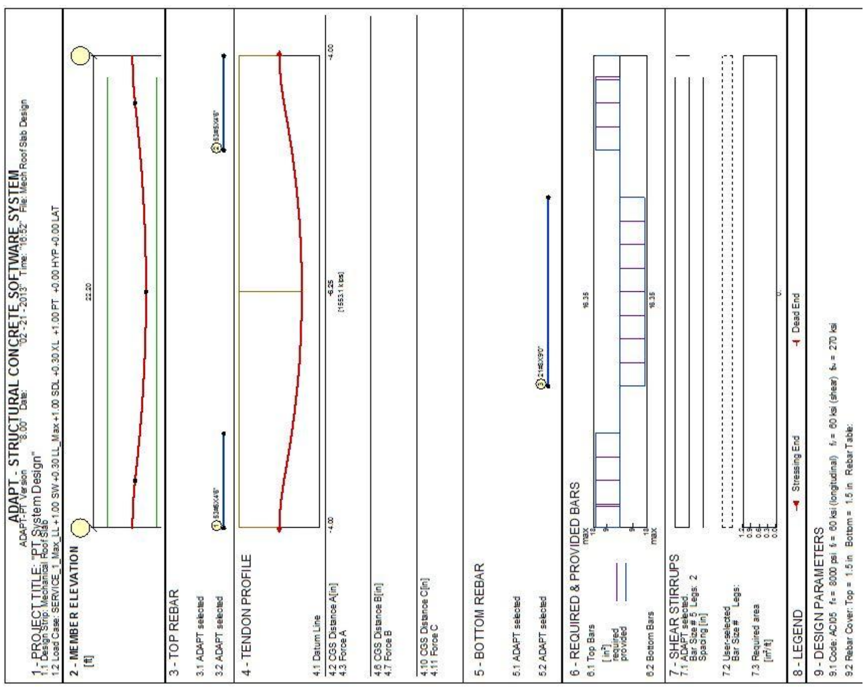
DL -0.119"
LL -0.092
sus LL -0.046
SDL -.015
PT +.179

-.277
+.178
-0.99"

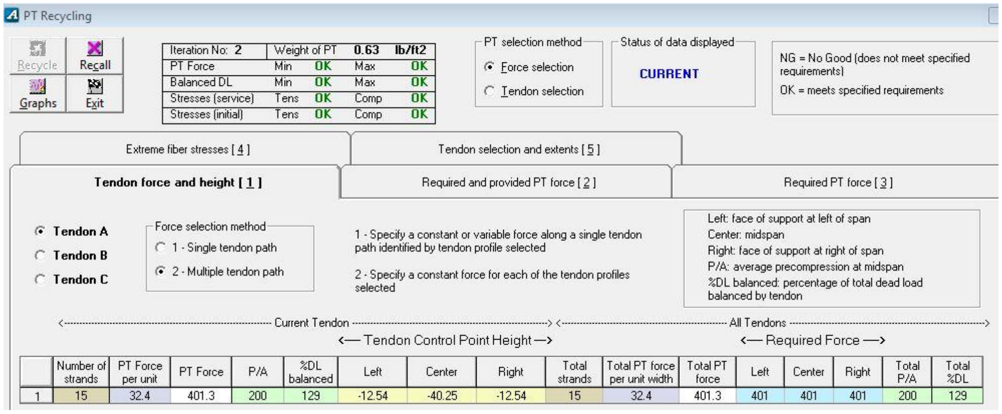
deflection passes

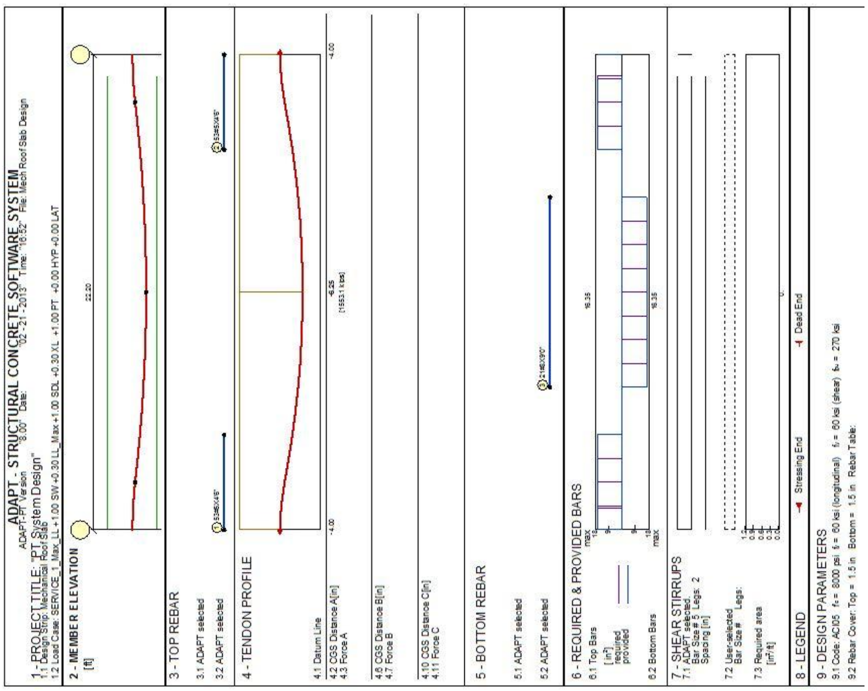
Slab





Beams:


PT Recycling
Recycle
Recall
Graphs
Exit
Iteration No: 2
Weight of PT 0.63 lb/ft2
PT Force Min OK Max OK
Balanced DL Min OK Max OK
Stresses (service) Tens OK Comp OK
Stresses (initial) Tens OK Comp OK
PT selection method
Force selection
Tendon selection
Status of data displayed
CURRENT
NG = No Good (does not meet specified requirements)
OK = meets specified requirements
Extreme fiber stresses [4]
Tendon selection and extents [5]
Tendon force and height [1]
Required and provided PT force [2]
Required PT force [3]
Tendon A
Tendon B
Tendon C
Force selection method
1 - Single tendon path
2 - Multiple tendon path
1 - Specify a constant or variable force along a single tendon path identified by tendon profile selected
2 - Specify a constant force for each of the tendon profiles selected
Left: face of support at left of span
Center: midspan
Right: face of support at right of span
P/A: average precompression at midspan
%DL balanced: percentage of total dead load balanced by tendon
Current Tendon
All Tendons
Tendon Control Point Height
Required Force
Number of strands
PT Force per unit
PT Force
P/A
%DL balanced
Left
Center
Right
Total strands
Total PT force per unit width
Total PT force
Left
Center
Right
Total P/A
Total %DL
1 15 32.4 401.3 200 129 -12.54 -40.25 -12.54 15 32.4 401.3 401 401 401 200 129


ADAPT - STRUCTURAL CONCRETE SOFTWARE SYSTEM
1 - PROJECT TITLE: "PT System Design"
2 - MEMBER ELEVATION
3 - TOP REBAR
4 - TENDON PROFILE
5 - BOTTOM REBAR
6 - REQUIRED & PROVIDED BARS
7 - SHEAR STIRRUPS
8 - LEGEND
9 - DESIGN PARAMETERS

Comparison

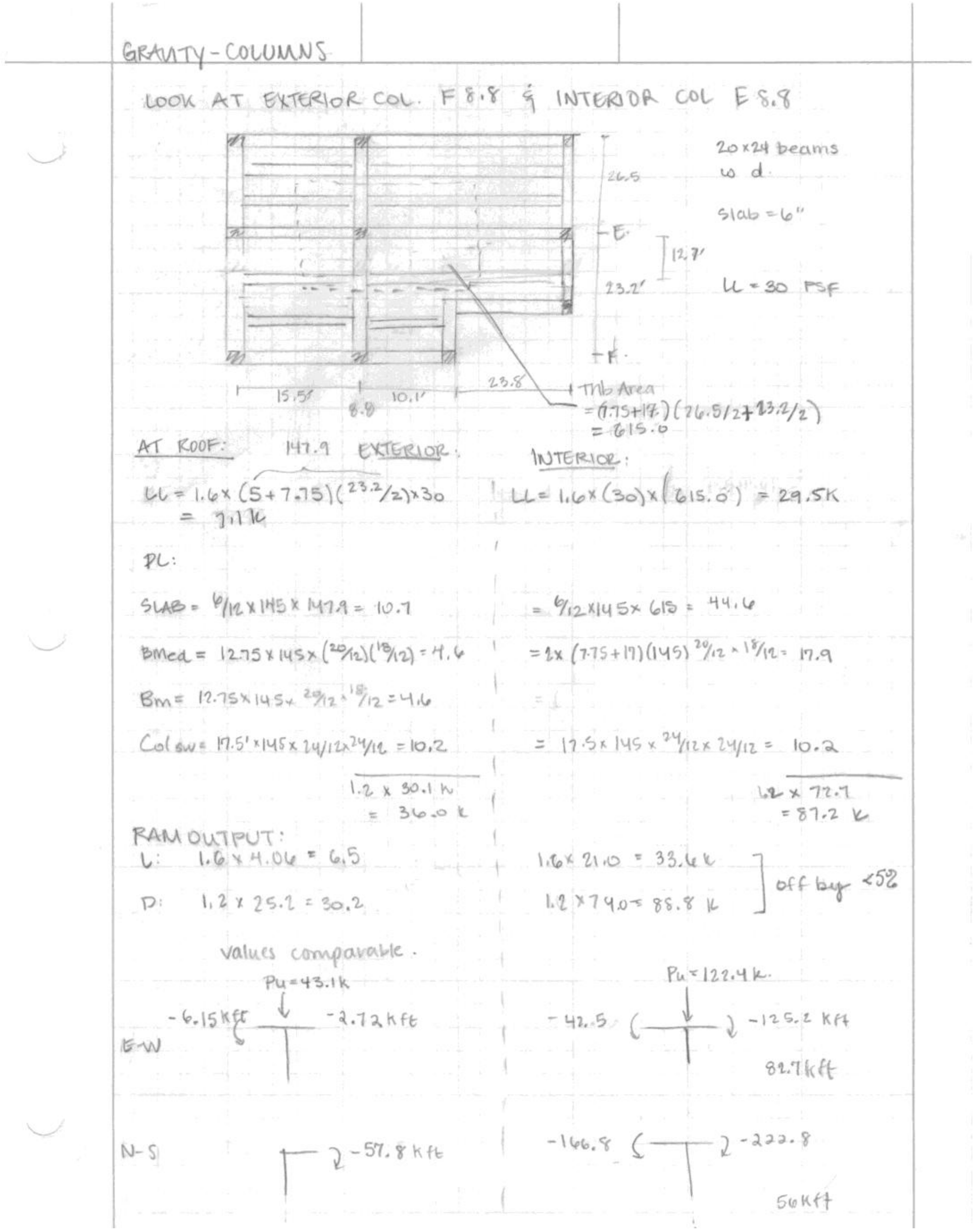
Reinforced Concrete One Way Slab & Beam					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
Forms in place, flat plate to 15' high, 4 uses	1655.68	S.F.	1.56	6.30	7.86
Forms in place, interior beam, 24", 4 uses	877.9	SFCA	0.26	2.55	2.59
Reinforcing in place, elevated slabs #4 to #7	3.1	Ton	1.49	0.77	2.26
Reinforcing in place, elevated beams #10	1.0	Ton	0.45	0.27	0.72
Reinforcing in place, elevated beams #4 to #7	0.5	Ton	0.22	0.22	0.45
Concrete ready mix, regular weight, 4000 psi	70.02	CY	3.30	0.00	3.30
Place and vibrate concrete, elevated slab 6" to 10" thick, pump	70.02	CY	0.00	0.64	0.64
Cure with sprayed membrane curing compound	25.34	C.S.F.	0.09	0.07	0.15
	Total SF	2186.80		Total ($/sf)	17.96

Prestressed One-Way Slab & Beam					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
Forms in place, flat plate to 15' high, 4 uses	2007.9	S.F.	1.89	7.64	8.75
Forms in place, interior beam, 24", 4 uses	240.2	SFCA	0.07	0.70	0.71
Reinforcing in place, elevated slabs #4 to #7	1.6	Ton	0.78	0.40	1.18
Reinforcing in place, elevated beams #4 to #7	0.7	Ton	0.32	0.32	0.65
Concrete ready mix, regular weight, 5000 psi	65.0	CY	3.24	0.00	3.24
Place and vibrate concrete, elevated slab 6" to 10" thick, pump	65.0	CY	0.00	0.59	0.59
Cure with sprayed membrane curing compound	20.1	C.S.F.	0.07	0.05	0.12
Pre-Stressing Tendons	3530.38	Lb.	1.02	2.00	4.41
	Total SF	2186.80		Total ($/sf)	19.64

Existing - Composite Steel					
System Components	Quantity	Unit	Cost per SF ($)		
			Material	Installation	Total
W24x55	37.2	LF	1.14	0.08	1.23
W24w76	198.0	LF	8.40	0.45	8.85
W16x31	49.5	LF	0.86	0.11	0.97
W30x90	22.4	LF	1.24	0.05	1.29
Welded Shear Connectors 3/4" diameter 3-7/8" long	240.5	Ea.	0.12	0.14	0.26
Metal decking, non cellular composite, galv. 2" deep, 20 gauge	2215.1	S.F.	1.83	0.47	2.30
Sheet metal edge closure form, 12" w/2 bends, 18 ga, galv	188.5	L.F.	0.09	0.09	0.17
Welded wire fabric rolls, 6 x 6 - W1.4xW1.4 (10x10), 21 lb/csf	22.2	C.S.F.	0.14	0.23	0.36
Concrete ready mix, normal weight, 3000 psi	20.5	CY	0.95	0.00	0.95
Place and vibrate concrete, elevated slab less than 6", pumped	20.5	CY	0.00	0.21	0.21
Curing with spread membrane curing compound	22.2	C.S.F.	0.07	0.06	0.13
Sprayed mineral fiber/cement for fireproof, 1" thick on beams	2215.1	S.F.	0.53	0.68	1.21
	Total SF	2215.13		Total ($/sf)	17.93

Columns & Walls:

Hand calculations:



2

TRIAL SIZE = 24 x 24

EXTERIOR: N-S direction l_u = 16.5' unsupported

$e = \frac{M_u}{P_u} = \frac{57.8 \times 12}{43.1} = 16''$ → ck slenderness

$\frac{l_u}{h} = \frac{16.5 \times 12}{24} = 8.3 \leq 8.4$ ✓ok

$\frac{e}{h} = \frac{16}{24} = .67$ bars in 2 faces

$\therefore \gamma = [24 - 2(2.5)] / 24 = .79$ → Fig A-9b $\frac{\phi P_n}{bh} = 6.9 \Rightarrow b = h = 6.9$ ✓ok

Fig A-9b

$\rho_g = .003$ $e/h \approx .65$

$\frac{\phi P_n}{bh} = \frac{43.1}{24^2} = .07$

$\frac{\phi M_n}{bh^2} = \frac{57.8 \times 12}{24^3} = .05$

$\rho_g = 1\%$ ⟶

Fig A-9c

$\rho_g = .003$ $e/h \approx .65$

$\frac{\phi P_n}{bh} = 0.07$

$\frac{\phi M_n}{bh^2} = 0.05$

ACI §10.9 ρ req'd > 1%

$A_s = \rho_g A_c = 0.01 (24)^2 = 5.76 in^2$ → use (6) #9s $A_s = 6.0 in^2$ ✓ok

TIES: #4s @ $S_{max} = \min \begin{cases} 16 d_{be} = 16(1.128) = 16'' \\ 48 d_{bt} = 48(0.5) = 24'' \\ .5(24'') = 12'' \end{cases}$ @ 12" o.c.

INTERIOR

$e = \frac{82.7 \times 12}{122.4} = 8.1''$

slenderness still not okay.

$\frac{e}{h} = \frac{8.1}{24} = .34$

$\therefore \gamma = [24 - 2(2.5)] / 24 = .79$ Fig A-9b $\frac{\phi P_n}{bh} = 1.6 \rightarrow b = h = 9.0''$ ✓ok

Fig A-9b

$\rho_g = .03$ $e/h \approx .34$

$\frac{\phi P_n}{b \cdot h} = \frac{122.4}{24^2} = .21$

$\frac{\phi M_n}{bh^2} = .07$

⇓

$\rho_g = 1\%$

Fig A-9c

$\rho_g = 0.03$ $e/h = .34$

$\frac{\phi P_n}{bh} = .21$

$\frac{\phi M_n}{bh^2} = .07$

$\rho_g = 1\%$ → use $A_s = 6 in^2$ ✓ok > $A_{s req'd}$

TIES: same req'mnts as above

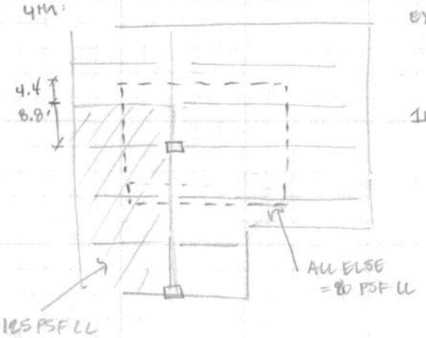
use #4s @ 12" o.c.

3

Design these 2 columns at bottom floor: Full col ht = 64'

		Roof	4th	3rd	2nd	
Ext	DL	21.7	→			+ col SW
	LL	30		100	100	(PSF)
Int	DL	75 k	→			+ col SW
	LL	30		100	100	

4th:



EXT:
LL = 125 PSF × (7.75 × 23.2/2) × 1.6
+ 80 × 5.× 23.2/2 × 1.6 = 18.0

INT:
LL = (80 × (23.2/2 + 26.5/2) × 17
(+ 80 × (4.4 × 7.75) + 125 (8.8 × 7.75)) × 1.6
= 72.0 K

∴ Load on Col:

EXT.

DL = 21.7 × 4 + 64 × 145 × 4 = 145.5 k (38.4 × 1.2)
+ superimp.
LL = (230) 1.6 (147.9) + 18
= 72.4

Pu = 217.9 k

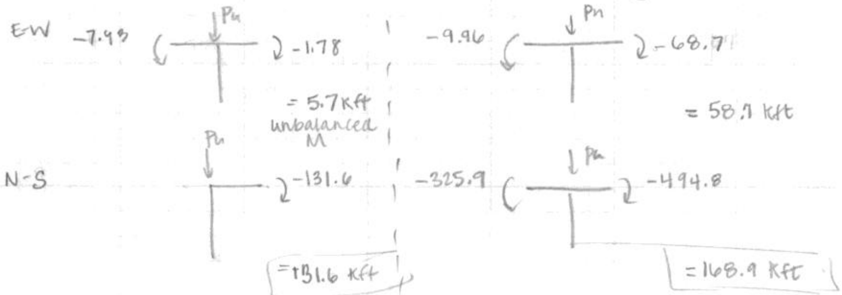
INT.

= 75.2 × 4 + 38.4 × 1.2 = 400 K
+ super.
= (230) × 1.6 (615) + 72 = 298.3

Pu = 698.3 k

} off of RAM Model < 10%

REASON FOR NOT SAME AS RAM –
– AREAS OVER GENERALIZED (LARGER) IN HAND CALCS.
– DL REDUNDANT IN RAM MODEL
{ LL MORE CONS. BY HAND,
DL BY RAM CALCULATIONS }.



4

DESIGN:

EXTERIOR

$e = \frac{M_u}{P_u} = \frac{131.6 \times 12}{217.9} = 7.2"$

$\frac{e}{h} = \frac{7.2}{24} = .3$

$\therefore \gamma = .79$ Fig A-9b = $\frac{\phi P_n}{bh} = 1.8 \rightarrow b = h = 11"$

Fig A-9b

$\rho_g = 0.03$ $e/h = .3$

$\frac{\phi P_n}{b \cdot h} = \frac{217.9}{24^2} = .38$

$\frac{\phi M_n}{b \cdot h^2} = \frac{131.6 \times 12}{24^3} = .11$

$\therefore \rho = 1\%$

Fig A-9c

$\frac{\phi P_n}{bh} = .38$

$\frac{\phi M_n}{bh^2} = .11$

$\rho = 1\%$

$\therefore$ use $A_s = \rho_g A_g = 5.76 in^2 \longrightarrow$ use (6) #9s $A_s = 6 in^2$

TIES SAME AS ABOVE.

INTERIOR

$e = \frac{M_u}{P_u} = \frac{168.9 \times 12}{698.3} = 2.9"$

$e/h = 2.9/24 = .1$

$\therefore \gamma = .79$ Fig A-9b $\frac{\phi P_n}{bh} = 3.2 \Rightarrow b = h = 14.8"$

Fig A-9b

$\frac{\phi P_n}{bh} = \frac{698.3}{24^2} = 1.21$

$\frac{\phi M_n}{bh^2} = \frac{168.9 \times 12}{24^3} = .15$

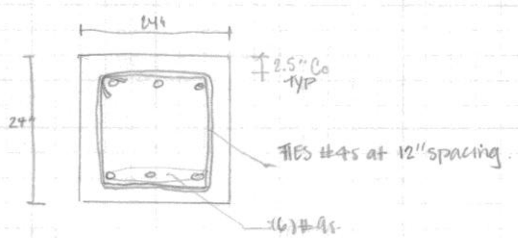
$\rho = 1\%$

Fig A-9c

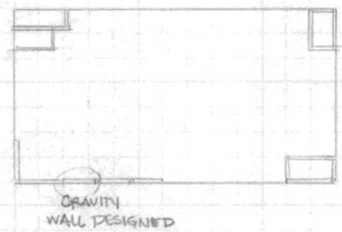
$\frac{\phi P_n}{bh} = 1.21$

$\frac{\phi M_n}{bh^2} = .15$

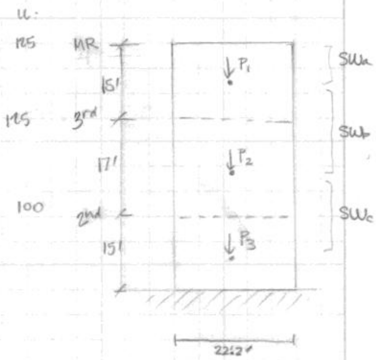
$\therefore$ use $A_s = \rho_g A_g = 5.76 in^2$ req'd $\rightarrow$ use (6) #9s



GRAVITY - WALLS
(BEARING WALLS)



LOAD ON WALL:



SLAB MR (trib width)
$= 150 \times \frac{9}{12} \times 22.2' \times 12.5/2 = 13.9\ k$

$S.W.a = 15'/2 \times 22.2' \times 8/12 \times 150 = 16.7\ k$

$S.W.b = (15'/2 + 17/2) \times 22.2 \times 8/12 \times 150 = 35.5\ k$

$S.W.c = (15/2 + 17/2) \times 22.2 \times 8/12 \times 150 = 35.5\ k$

$\text{Slab 3rd} = 150 \times \frac{6}{12} \times 22.2 \times \frac{8.3'}{2} = 6.9\ k$

$\text{Slab 2nd} = 150 \times \frac{5}{12} \times 22.2 \times 16.6'/2 = 11.5\ k$

$P_1 = 13.9 + 16.7 = 30.6\ k$

$P_2 = P_1 + 6.9 + 35.5 = 73\ k$

$P_3 = P_2 + 11.5 + 35.5 = 120\ k$

Liveload:

$P_1 = 125\ PSF \times 22.2 \times 12.5/2 = 17.3\ k$

$P_2 = 125 \times 22.2 \times 8.3/2 = 11.5\ k$

$P_3 = 100 \times 22.2 \times 16.6/2 = 23.0\ k$

§14.5.3.1

$t_{min} = \max \begin{cases} 4" \\ \min \begin{cases} h/25 = 17 \times 12/25 = 8.16 \\ \ell/25 = 22.2/25 = \end{cases} \end{cases}$ → use a 10" t wall

$\phi P_n > P_u$

$\phi P_n = 0.55\ \phi\ f'_c A_g \left[1 - \left(\frac{k\ell_c}{32h}\right)^2\right]$

$k = 0.8$

$\phi = 0.65$ §9.3.2

$f'_c = 4000$

$h = 10"$

$\ell_c = 17' \times 12"$

$A_g = 10" \times 12"$ (per ft length)

2.

$\phi P_n = 0.55\,(0.65)\,4000\,(10 \times 12)\left[1 - \left(\frac{0.8\,(17 \times 12)}{32 \times 10}\right)^2\right]$

$= 171\,600\,(.594) = 101\,860 = 101.9$ k/ft

$P_u = [1.2\,(30.6 + 73 + 120) + 1.6\,(17.3 + 11.5 + 23)]\,/\,22.2$

$= \frac{351.2}{22.2} = 15.8$ k/ft

$\phi P_n > P_u$ ✓ therefore ok

§14.3.2,.3 Min Reinf:

vert. reinf:

(a) $A_s \geq 0.0012 A_g$ where $\geq$ #5s, $f_y = 60$ ksi

$A_s \geq .0012\,(10 \times 12)$ ft length

$= .144$ in²/ft — use #4s @ 16" o.c

spacing, max = min { 18" ; 3t = 3(10) = 30" }

$A_{s\,prov'd} = .20\,\text{in}^2 \left(\frac{12}{16}\right) = .15$ in²/ft $> .144$ in²/ft ✓ok

horiz reinf

same max spacing

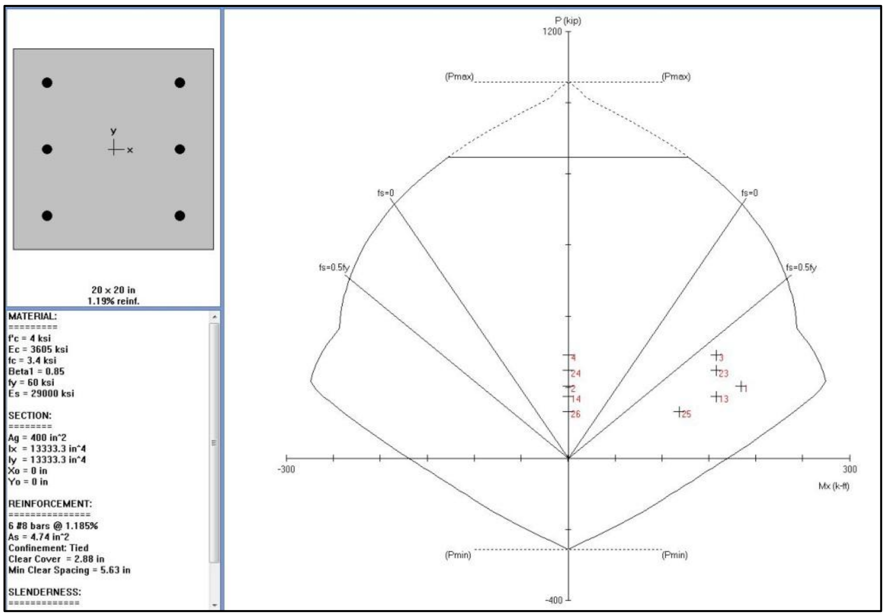
$A_s \geq .0025\,A_g$ where $\geq$ #5s, $f_y = 60$ ksi

$= .0025\,(10 \times 12) = .30$ in²/ft — use #5s @ 12" o.c.

$A_{s\,prov'd} = .31$ in²/ft $> .30$ in²/ft ✓ok

§14.3.6

$A_{st} = \frac{.15\,\text{in}^2/\text{ft}}{10 \times 12\ \text{in}^2/\text{ft}} = .00125 < .01\,A_g$ ∴ no ties req'd



Foundations

FOUNDATIONS 1

SOIL BEARING CAPACITY · 3000 PSF = q_a

COLUMN FOUNDATION

try $f'c$ = 4000 psi

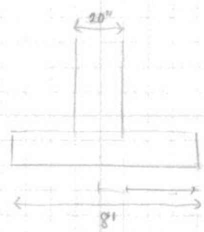
of COL F8.8 – (F9) Square footing

LOAD: $P_u = 1.2(150.44) + 1.6(43.56) = 250.22\ k$

L = B

sizing: $\frac{P}{A} \le q_a$ $\frac{150.44+43.56}{B^2} \le 3000$

$B \ge 8.0'$



design:

$q = \frac{250.2\ k}{8\times8\ ft^2} = 3.9\ \frac{k}{ft^2} \times \frac{1}{144} = 27.1\ psi$

per 1 ft width = 3.9 klf = w

$M = wl^2/2 = (3.9)(\frac{1}{2}(8 - 20/12))^2/2 = 19.55\ k\text{-}ft$

$\phi V_c = \phi 4\sqrt{f'c} = 0.75(4)\sqrt{4000} = 189.7\ psi = .190\ ksi$

$d^2(V_c + q/4) + d(V_c + q/2)w + -\frac{q}{4}(BL - w^2) = 0$

$d^2(190 + \frac{27.1}{4}) + d(190 + 27.1/2)20" - 27.1/4(8^2 144 - 20^2) = 0$

$197 d^2 + 4071 d = 59728$

$d \ge 9.9$

use #5s $d_b = 5/8$

$h = d + c_c + d_b$

$= 9.9 + 3 + 5/8 = 13.5"$ → use 14" depth (1'2")

$\therefore d = 14 - 3 - 5/8 = 10.375"$

$b_o = 2(20) + 2(8\times12) = 232"$

$\therefore V_c \le \min \begin{cases} 0.75(4)\sqrt{4000} \cdot 232" \cdot 10.375 = 457 \\ 0.75(2+2)\sqrt{4000} \cdot 232" \cdot 10.375 = ✓ \\ (\frac{40(10.375)}{232} + 2)\sqrt{4000} \cdot 232 \cdot 10.375 = 577 \end{cases}$ > ϕV_c above ok.

Design for flexure:

$l = 4' - 10/12 = 3.17$

$M = \frac{wl^2}{2} = \frac{(3.9\ k/ft)(3.17)^2}{2} = 19.6\ k\,ft$

$a = \frac{A_s f_y}{0.85 f'c\, b} = \frac{A_s\, 60}{0.85(4)(12)} = 1.48 A_s$

$M_u \le \phi M_n = \phi A_s f_y (d - a/2) = 0.9 A_s\, 60(10.375 - 1.48A_s/2)$

$19.6\times12 = 235.2 = 560.25 A_s - 39.96 A_s^2$

$A_s \ge .43\ in^2$

Footings 2

try #5s $A_{s1} = .31 \frac{in^2}{ft} \left(\frac{12"}{8"}\right) = .47\ in^2/ft$ width

∴ #5s @ 8"

ck ductility:

$c = a/\beta \Rightarrow a = 1.48(.47) = 3.15 \rightarrow c = 3.70"$

$\varepsilon_s = .003(10.375 - 3.70)/3.70 = .0054 > .005 \therefore \phi = 0.9$ ✓ok

uses (12) #5s

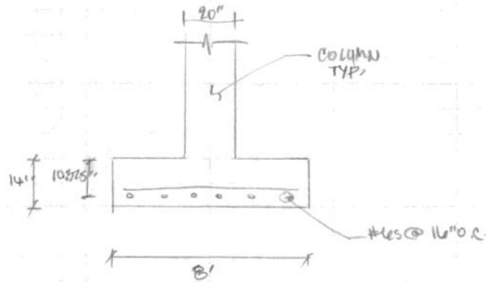
$A_s = .44\ in^2$ per 1 #6

or, can use (8) #6s @ 16" o.c.

Bearing ck:

$\phi B_n = 0.65(0.85)\ 4\ ksi\ 20 \times 20 = 884\ k > P_u = 250.2\ k$ ✓ok

∴ Col Footing at 8'x8' footing with #6s @ 16" o.c both direction



Fndtns   3

Strip footing

wall along column line F.

use 12" CMU wall

Size by ASD:

$P = 12.4$ klf

$\frac{12.4 \times 1}{1' \times B} \le 3 \rightarrow B = 4.1'$ [4.5' wide]

$P_{LL} = 2.3$ klf
$P_{DL} = 10.1$ klf
$P_u = 18.9$ klf

$l = 4.5/2 - 5/12 = 1.83'$

by LRFD:

$P_u = 18.9$ klf

$\therefore q = \frac{18.9}{4.5} = 4.2$ k

$M_u = q l^2/2 = 4.2(1.83)^2/2 = 7.03$ k ft / ft width

$\phi M_n = 0.55(5\sqrt{4000})(12h^2/6) \ge 7.03$ k·ft $\times 1000 \times 12"$

$h \ge 15.6 \rightarrow 16"$ depth

ck. shear:

@ quarter pt. $l = 1.83 + 2.5/12 = 2.04'$

$\phi V_n = 0.55(\frac{4}{3})\sqrt{4000} \cdot 10" \cdot 16 \ge q \times l = 4.2 \times 1000$ k $\times 2.04$

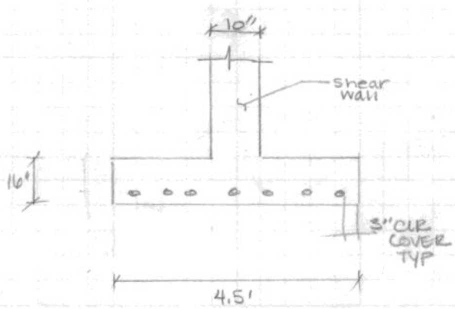
$7420.8 \ge 8568$ lbs.

try $d = 20"$ $\phi V_n = 9276$ lbs ≥ 8568 lbs   ✓ ok

T&S reinf by ACI code Ch7

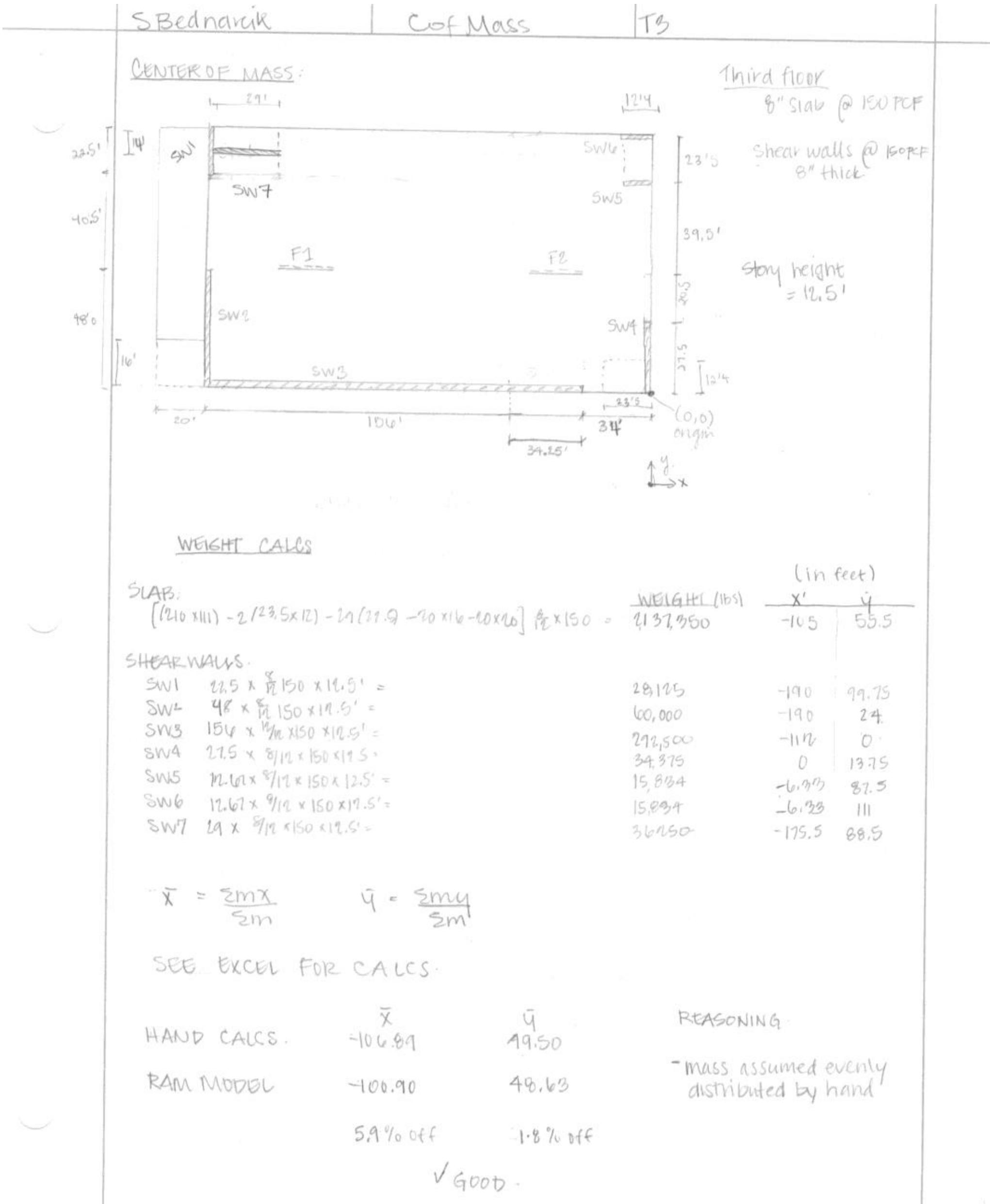
$\rho_{min\,T\&S} = .0018 = \frac{A_s}{4.5' \times 20" \times 12} \rightarrow A_s = 1.94$ in²

use (7) #5s spaced evenly



Appendix E RAM Model & Building Properties

Center of Mass & Center of Rigidity



Center of Mass Hand Calculations - 3rd Floor					
System	**Mass**	**x (ft)**	**y (ft)**	**m*x**	**m*y**
Slab	2137350	-105	55.5	-224421750	118622925
SW1	28125	-190	99.75	-5343750	2805468.75
SW2	60000	-190	24	-11400000	1440000
SW3	292500	-112	0	-32760000	0
SW4	34375	0	13.75	0	472656.25
SW5	15834	6.33	87.5	100229.22	1385475
SW6	15834	6.33	111	100229.22	1757574
SW7	36250	-175.5	88.5	-6361875	3208125
Sums	2620268			-280086916.6	129692224
		xbar=	-106.89		
		ybar=	49.50		

Periods of Vibration	
Tx=	0.8072
Ty=	1.1262
Ttors=	0.9004

Load Transfer

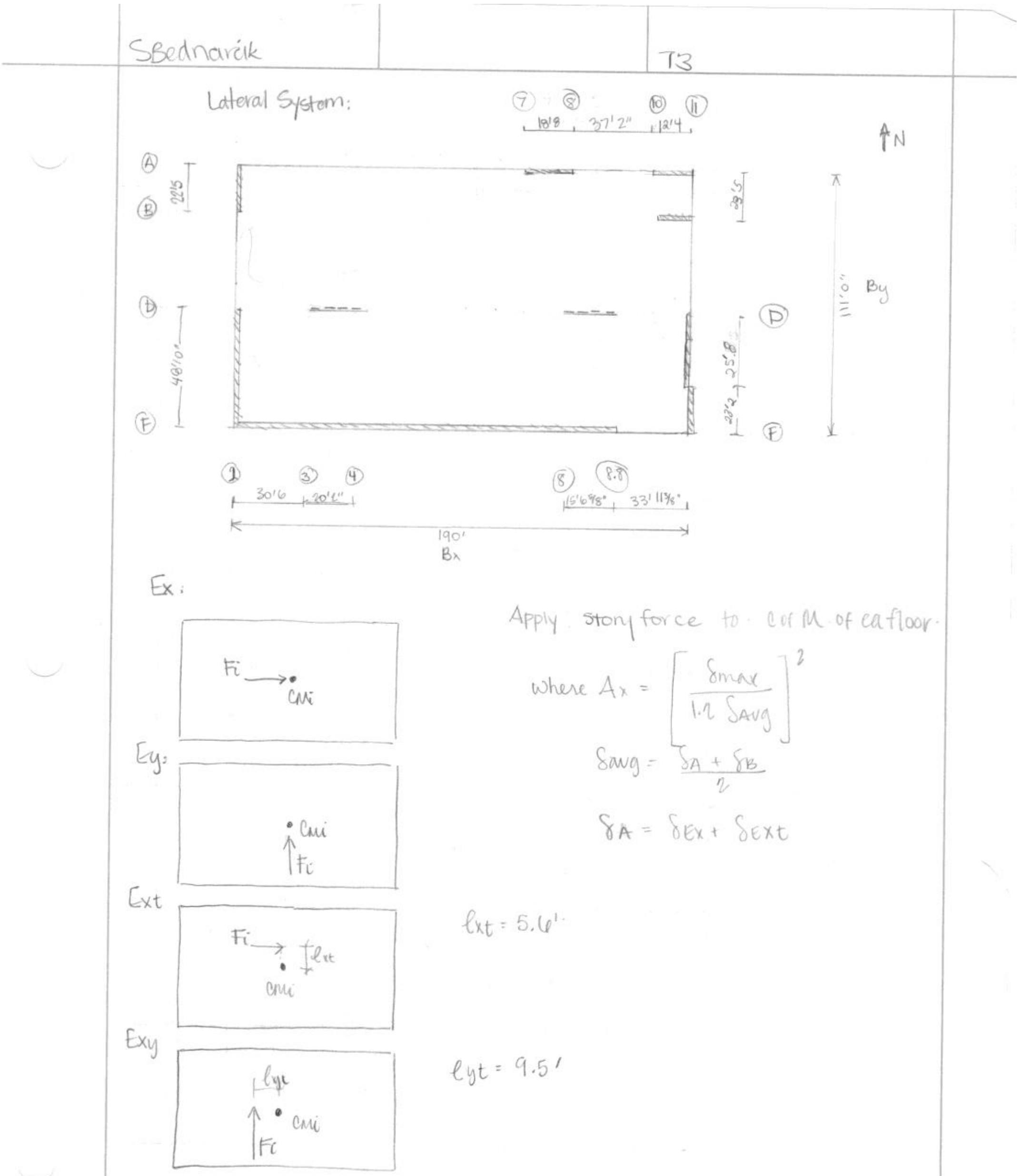
Level	Direction	Frame	Horiz Force (k)	%V	Sum Check	% Error	Stiffness, X	Stiffness, Y
Roof	x	SW3	334.08	35.60			0.68	
		SW5	30.23	3.22			0.05	
		SW6	33.09	3.53			0.05	
		SW7	494.05	52.64			0.79	
		F1	43.86	4.67			0.07	
		F2	3.19	0.34	938.50	6.15	0.01	
	y	SW1	668.06	66.81				1.00
		SW4	316.26	31.63	1000.01	0.00		0.47
Mech Roof	x	SW3	352.47	37.99			0.72	
		SW5	30.23	3.26			0.06	
		SW6	33.09	3.57			0.07	
		SW7	491.03	52.92			1.00	
		F1	20.96	2.26			0.04	
		F2	0.07	0.01	927.85	7.21	0.00	
	y	SW1	668.05	66.80				1.00
		SW2	69.43	6.94				0.10
		SW4	316.26	31.62	1000.04	0.00		0.47
4th	x	SW3	406.43	41.98			0.91	
		SW5	30.23	3.12			0.07	
		SW6	33.09	3.42			0.07	
		SW7	447.56	46.23			1.00	
		F1	22.20	2.29			0.05	
		F2	28.70	2.96	968.21	3.18	0.06	
	y	SW1	814.09	80.96				1.00
		SW2	69.43	6.90				0.09
		SW4	108.61	10.80	1005.53	-0.55		0.13
3rd	x	SW3	494.32	49.90			1.00	
		SW5	-12.29	-1.24			-0.02	
		SW6	-6.10	-0.62			-0.01	
		SW7	476.62	48.11			0.96	
		F1	19.63	1.98			0.04	
		F2	18.40	1.86	990.59	0.94	0.04	
	y	SW1	149.66	14.97				0.20
		SW2	731.65	73.17				1.00
		SW4	118.17	11.82	1000.00	0.00		0.16
2nd	x	SW3	563.76	56.38			1.00	
		SW5	50.66	5.07			0.18	
		SW6	61.63	6.16			0.08	
		SW7	279.52	27.95			0.38	
		F1	61.41	6.14			0.08	
		F2	50.15	5.02	1067.13	-6.71	0.07	
	y	SW1	46.44	4.64				0.06
		SW2	731.65	73.17				1.00
		SW4	231.31	23.13	1006.33	-0.63		0.32
Ground	x	SW3	563.76	56.38			1.00	
		SW5	50.66	5.07			0.09	
		SW6	61.63	6.16			0.11	
		SW7	279.52	27.95			0.50	
		F1	33.61	3.36			0.06	
		F2	31.38	3.14	1020.55	-2.06	0.06	
	y	SW1	40.00	4.00				0.05
		SW2	731.65	73.17				1.00
		SW4	231.31	23.13	1001.58	-0.16		0.32
			V (total story shear)=			1000	k	Acting at Roof

Relative Stiffness check and torsional shear calculations:

Relative Stiffnesses			
Floor	**Shear Wall**	**R. S. X**	**R. S. Y**
Roof	SW1	0	0.26
	SW2	0.09	0.25
	SW3	-0.03	0.12
	SW4	-0.05	0.36
	SW5	0.61	0
	SW6	0.4	0
	SW7	0	0
Mech Rf	SW1	0	0.27
	SW2	0.06	0.26
	SW3	-0.03	0.12
	SW4	-0.05	0.36
	SW5	0.69	-0.01
	SW6	0.4	0
	SW7	-0.07	0.01
Fourth	SW1	0.19	0.26
	SW2	0.06	0.26
	SW3	-0.03	0.12
	SW4	-0.23	0.37
	SW5	0.25	0.02
	SW6	0.13	0.01
	SW7	0.64	-0.03
Third	SW1	0.11	0.26
	SW2	0.12	0.26
	SW3	-0.06	0.13
	SW4	-0.18	0.36
	SW5	0.31	0.01
	SW6	0.15	0.01
	SW7	0.57	-0.02
Second	SW1	0.1	0.29
	SW2	0.1	0.24
	SW3	-0.06	0.16
	SW4	-0.14	0.32
	SW5	0.31	0.01
	SW6	0.26	0.01
	SW7	0.45	-0.02

Roof	Shear Wall	Direct Shear	Torsional Shear								Total Shear		% off
		x	ki	F	5%	M	di	$k_i*d_i^2$	$\sum k_i*d_i^2$	$F_i=M_i*k_i*d_i/(\sum k_i*d_i^2)$	Hand Calcs	RAM Output	
	SW1	0	0.26	232.54	9.5	2209.13	-90.84	2145.50		-5.83	-5.83	-5.55	-5.03
	SW2	20.9286	0.25	232.54	9.5	2209.13	90.84	2062.98		5.60	26.53	26.61	0.29
	SW3	-6.9762	0.12	232.54	9.5	2209.13	86.86	905.36		2.57	-4.40	-7.01	37.18
	SW4	-11.627	0.36	232.54	9.5	2209.13	99.16	3539.77		8.81	-2.82	-12.99	78.32
Input:	SW5	141.8494	0.61	232.54	5.55	1290.60	17.16	179.62		1.51	143.36	151.9	5.62
Load	SW6	93.016	0.4	232.54	5.55	1290.60	17.16	117.79		0.99	94.01	98.33	4.40
232.54	SW7	0	0	232.54	5.55	1290.60	66.83	0.00	8951	0.00	0.00	0	0.00

Torsion



	Calculation of Amplification Factor:									
X Direction		δ1			δ2					Irregularity Type
	Level	δ1 (Ex)	δ1 (Ext)	δ1	δ2 (Ex)	δ2 (Ext)	δ2	δavg	Ax	by Table 12.3-1
	Roof	0.189	0.206	0.395	0.188	0.206	0.394	0.395	1.0	1a
	Mech Roof	0.141	0.154	0.295	0.141	0.154	0.295	0.295	1.0	1a
	4th	0.124	0.136	0.260	0.124	0.135	0.259	0.260	1.0	1a
	3rd	0.079	0.083	0.162	0.079	0.086	0.165	0.164	1.0	1a
	2nd	0.027	0.030	0.057	0.027	0.030	0.057	0.057	1.0	1a
Y Direction		δ1			δ2					Irregularity Type
	Level	δ1 (Ey)	δ1 (Eyt)	δ1	δ2 (Ey)	δ2 (Eyt)	δ2	δavg	Ay	by Table 12.3-1
	Roof	0.345	0.337	0.682	0.843	0.900	1.743	1.212	1.4	1b
	Mech Roof	0.250	0.245	0.495	0.622	0.664	1.286	0.891	1.4	1b
	4th	0.215	0.211	0.426	0.542	0.579	1.121	0.773	1.5	1b
	3rd	0.124	0.121	0.245	0.334	0.357	0.691	0.468	1.5	1b
	2nd	0.035	0.034	0.069	0.104	0.111	0.215	0.142	1.6	1b

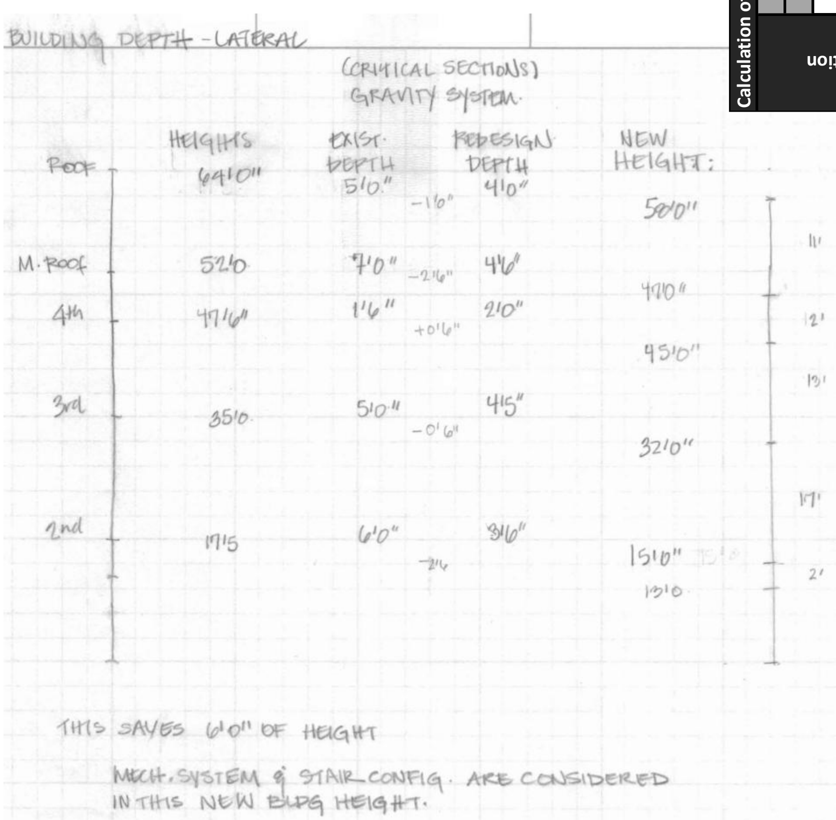
Appendix F Lateral Redesign

Applied Building Loads

Weight of Building	Area	PSF	Load (lbs)	Story Weight (lbs)
CMU	4310	91	392210	
Curtain Wall	2160	20	43200	
Concrete Shear Walls	9610	100	961000	
Floor 2	12090	67	810030	2206.44
CMU	9140	91	831740	
Curtain Wall	2160	20	43200	
Concrete Shear Walls	9610	100	961000	
Floor 3	21060	105	2211300	4047240
CMU	5920	91	538720	
Curtain Wall	2300	20	46000	
Concrete Shear Walls	6030	100	603000	
Floor 4	21060	67	1411020	2598740
Mechnical (RTU)			35934	35934
CMU	4520	91	411320	
Curtain Wall	3500	20	70000	
Concrete Shear Walls	8530	100	853000	
Roof	17460	80	1396800	2731120
Columns	1870	70	130900	130900
		Total Weight (lbs)		9546140.44
			(k)	9546

Calculation of Amplification Factor:

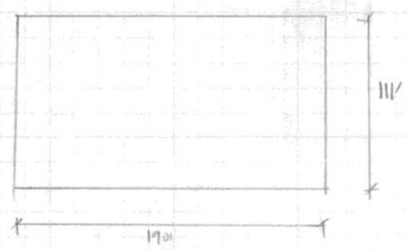
	δ1			δ2					
	Δ1	δ1 (Ex+)	δ1 (Ex-)	δ2	Δ2	δavg	δmax	Ax= (δmax/(1.2*δavg))^2	Irregularity Type by Table 12.3-1
X Direction	0.312	0.758	0.766	0.766	0.167	0.813	0.861	N/A	N/A
	0.011	0.599	0.593	0.599	-0.005	0.574	0.599	N/A	N/A
	0.174	0.604	0.585	0.604	0.285	0.571	0.604	N/A	N/A
	0.254	0.315	0.319	0.319	0.210	0.341	0.363	N/A	N/A
	0.109	0.109	0.105	0.109	0.109	0.109	0.109	N/A	N/A
	Δ1	δ1 (Ey+)	δ1 (Ey-)	δ2	Δ2	δavg	δmax	Ay= (δmax/(1.2*δavg))^2	Irregularity Type by Table 12.3-1
Y Direction	-0.048	0.374	0.452	0.452	0.103	0.334	0.452	1.3	1a
	0.109	0.289	0.348	0.348	0.025	0.307	0.348	N/A	N/A
	0.053	0.266	0.323	0.323	0.123	0.240	0.323	1.3	1a
	0.069	0.165	0.200	0.200	0.137	0.152	0.200	1.2	1a
	0.034	0.052	0.063	0.063	0.063	0.049	0.063	1.2	1a



LATERAL – WIND BEDNARCIK 1

USING ASCE 7-05 Ch.6, Method 2:

BLDG LAYOUT:



New roof height

Elevation = 58'

Assumed flat roof
∴ mean roof ht, h = 58'

FOR A N-S WIND:
L = 111 ft
B = 190 ft

E-W
L = 190 ft
B = 111 ft

§6.5.3 Design Procedures

1. Wind speed V = 90 mph (Fig 6-1)
Directionality Factor, K_d = 0.85 (Table 6-4)
2. Importance Factor I = 1.15 (Tab 6-1)
Category III (Tab 1-1)
3. Exposure Category
§ 6.5.6 Exposure B
Vel. Pressure Exp Coeff. (Table 6-3)
Case 2, Exp B
Interpolate z = 50 | .81 → at z = 58' K_z = .84
z = 60 | .85
Other K_z values on Excel
4. Topographic Factor, K_{zt}
§6.5.7 K_{zt} = 1.0
5. Gust Effect Factor, G or G_f §6.5.8
rigid or flexible:
$n_1 = \frac{100}{58} = 1.72$ (C 6-17)
$n_1 = \frac{75}{58} = 1.29$ ∴ rigid building (C6-18)
n_1 = natural bldg freq. → use §6.5.8.1

$$G = .925\left(\frac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_v I_z}\right)$$

$g_Q = g_v = 3.4$

from table 6-2 α = 7.0 z_g = 1200 $\hat{a}$ = 1/4.0 $\hat{b}$ = 0.45
c = 0.3 ℓ = 320 $\bar{\epsilon}$ = 1/3.0

$I_z = c\left(\frac{33}{z}\right)^{1/6} = 0.30\left(\frac{33}{34.8}\right)^{1/6} = .297$ $z = \max\begin{cases} .6(58) = 34.8 \\ 30 \end{cases}$

2

N-S | E-W

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{.63}}}$ $\therefore L_z = \ell\left(\frac{z}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{58}{33}\right)^{1/3.0} = 386.18$

$Q_{NS} = .82$ | $Q_{EW} = .85$

$\therefore$

$G = .925\left(\frac{(1+1.7(3.4)(386.18)(.82))}{1+1.7(3.4)(386.18)}\right)$ | $G = .925\left(\frac{(1+1.7(3.4)(386.18)(.85))}{1+1.7(3.4)(386.18)}\right)$

$= .76$ | $= .79$

6. Fully enclosed.
7. Internal Pressure $GC_{pi} = \pm 0.18$ (Fig 6-5)
8. Ext Pressure Coeff

$L/B = .58$ | $= 1.71$

Walls:

	Cp:	
0.8	(q_z) Windward	0.8
−0.5	(q_h) Leeward	−0.34
−0.7	(q_h) Side	−0.7

Roof Pressures: (q_h)
$\theta = 0°$

$h/L = 58/111 = .52$ | $h/L = 58/160 = .36$

Interpolated	−.88, −0.18	0 to h/2 = 29	−0.9, −0.18
	−0.89, −0.18	h/2 to h = 29 to 58	−0.9, −0.18
	−.51, −0.18	h to 2h = 58 to 116	−0.5, −0.18
	−0.32, −0.18	>2h = >116	−0.3, −0.18

roof area = 190x111 ≥ 1000 SF. $\therefore$ Reduction factor = 0.8
9. Velocity Pressure q_z, q_h §6.5.10
at h=58'

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$ §6.5.4.4 $K_d = 0.85$
$= 0.00256(0.85)(0.84)(90^2)(1.15)$ §6.5.6.6 $K_z = 0.84$ * varies per height
$= 17.08$ §6.5.7.2 $K_{zt} = 1.0$

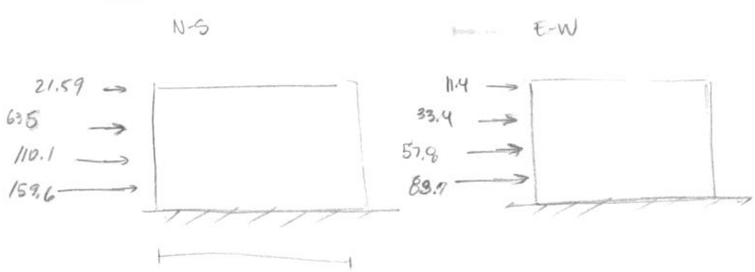
* others on excel spreadsheet

10. Design wind load §6.5.12.2
MWFRS
$P = qGC_p - q_i(GC_{pi})$ ($q_i = q_h$)

$q = q_z$ windward
q_h leeward, side, roof
$q_i = q_h$ all

* see excel for further calcs.

WIND:



Look at critical sections:

LATERAL – SEISMIC

USING ASCE 7-05 §11.4
FROM GEOTECH REPORT
$S_S = 0.26$
$S_1 = 0.06$
Site class D

$F_a = 1.6$ (Table 11.4-1) $F_v = 2.4$ (Table 11.4-2)

$S_{MS} = F_a S_S = 1.6(0.26) = .416 \longrightarrow S_{DS} = \frac{2}{3} S_{MS} = .279$
$S_{M1} = F_v S_1 = 2.4(0.06) = .144 \qquad S_{D1} = \frac{2}{3} S_{M1} = .096$

$S_a = S_{DS}(0.4 + 0.6\ T/T_0)$
$T_0 = 0.2\ S_{D1}/S_{DS} = 0.2(0.096)/.277 = 0.0693$
$T_S = S_{D1}/S_{DS} = 0.096/0.277 = .347$

FINDING T: Fund. period of bldg. §12.8.2

$T \le C_u T_a$
$T_a = C_t h_n^x = (0.02)(58)^{.75} = .420$
$C_u = 1.7$ (Table 12.8-2)
$\rightarrow T = 1.7(.420) = .715$

$\therefore S_a = .277(0.4 + 0.6(.7145/0.0693) = 1.8244$

$C_S = \frac{S_{DS}}{R/I} = \frac{.279}{4/1.5} = .1039$ where $R = 4$ Table 2.1, $I = 1.25$ Table 11.5-1 Occ Cat III

$T_L = 6$ (Fig 22-15) $T < T_L$ $\therefore C_S = .1039 \le \frac{S_{D1}}{T(R/I)} = \frac{.096}{.715(4/1.25)} = .042$

by §12.8.1-1 (12.8-1-3) $\ge .01$
$C_S = .1039$
$\therefore$ can be reduced to
by §12.8.1.1 $= 0.042$

values confirmed with
"Loads & Applied Forces"

§10.10.4

walls	uncracked	$0.70 I_g$
	cracked	$0.35 I_g$

2

Building Weight: Example

Roof: slab 75 PSF
mech 7 PSF
6" ← framing 75 PSF
walls = 498 k
= 2958 k

Area = 111 x 205 − (30 + 22x4) 50 = 16055 ft²

walls: [7 x 30.5' + 2 x 23.5 + 50 + 40 + 4 x 12] x 150 x 10/12 x 7.5' = 498 k

In comparison,

Hand Calc = 3144 k   RAM = 3167 k

off by 0.8 % okay ∴ RAM will calc other loadings correctly

§12.8.3

$F_x = C_{vx} V$ {force per level} where w_x = weight at level
h_x = height from base to level

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$

k = 1   T < 0.5
T
k = 2   T > 2.5

building period T = .715 ⟶ k = 1.11

Story shear = $V_x = \sum F_i$ $V = C_s W$

§12.8.4 Torsion

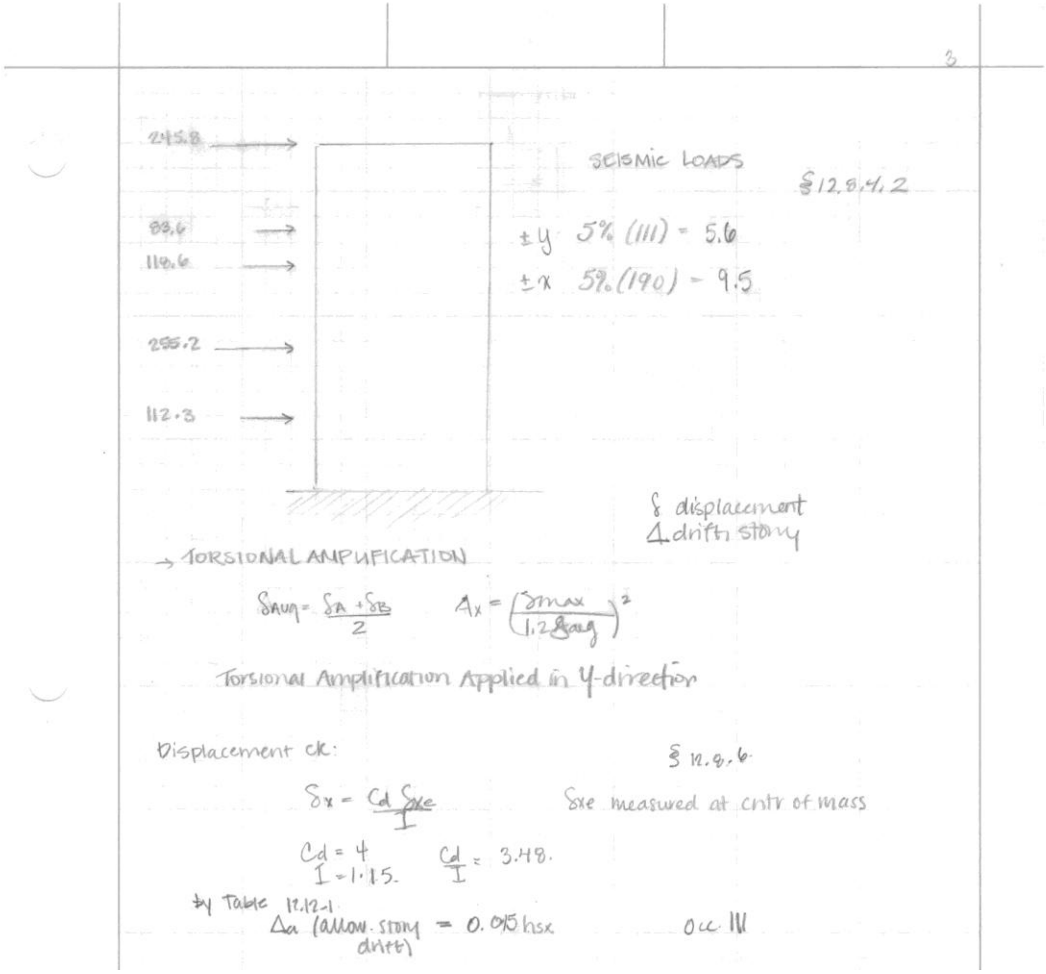
Amplification − 4.3 $A_x = \left(\frac{\delta_{max}}{1.2\,\delta_{avg}}\right)^2$

using nodes at:

Roof, 3rd	NE	(−10.0, 108.85)
	SW	(−187.3, 0)
Mech Rf	SE	(−68.2, 0.2)
	NW	(−187.3, 48.9)
4th, 2nd	SE	(0, 10.55)
	NW	(−188.3, 109.2)

using interstory drift

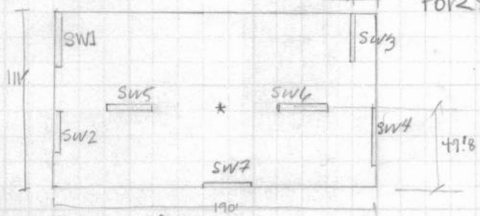
Design Results



4

SHEAR WALLS:

CONFIRMING OUTPUT FOR SHEAR WALL LOADS



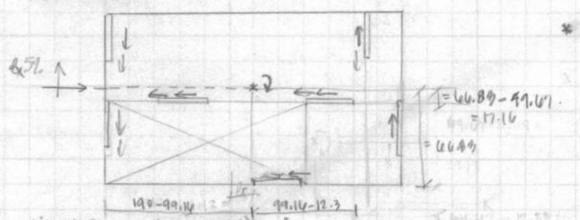
Shear wall: relative stiffness
· Apply 1000 k at cntr of mass (-99.16, 64.83, 58')
→ see excel spreadsheet for further calcs

FORCE DISTRIBUTION PER FLOOR PER SHEAR WALL
(EXAMPLE – ROOF)

X-DIRECTION:

LOAD = 245.76 k

* C of M = (-99.16, 66.83)



± 5% width

Direct Shear (per excel)

SW5	149.9
SW6	98.3
SW7	0

C of R.

CONFIRMING WITH LOAD CASE E1.X+ IN RAM.

Torsional Shear:

Frame SW:	k_i (stiffness)	F	5%	M	d_i	$k_i d_i^2$	$F_i = \frac{M k_i d_i}{\Sigma k_i d^2}$
1	0.26	245.80	9.5'		90.84	see excel	
2	0.25				90.84		
3	0.12				86.84		
4	0.36			$= F \times 5\%$	99.16		
5	0.61		5.65'		17.16		
6	0.40				17.16		
7	0				66.83		
					Σ =		

under 6%. CONSIDERED ACCEPTABLE. ✓

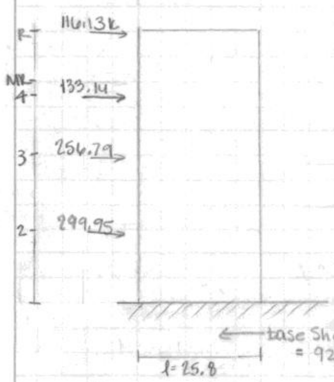
∴ CAN USE LOADS ON EACH WALL AS OUTPUT

5

CONSIDER SHEAR WALL 1: -Not req'd to be designed by Seismic provisions at SDC by §21.1.1.5, .7

wall thickness = 8" = h.

See excel for distribution calcs.



a) Slender or short

$\frac{h_w}{l_w} = \frac{58}{25.8} = 2.25$ ∴ < 3 > 2 ∴ design by Ch 10 & 11 ACI

See excel for deflection check

See Ch 11 - walls - shear load

see ch 14 - walls - of ACI 318-11 axial load

Mov = 25,443.4 kft = Mu

f'c = 4000 ksi

§11.9.1 Shear Str.

d = 0.8 l_w = 0.8 (25.8) = 20.64' x 12" = 247.7" §11.9.

Vc = 3.3λ √f'c h·d + $\frac{N_u d}{4 l_w}$ where Nu = + 1.2(186.37) + 1.6(164.46) = 1206.78k

= min { = 3.3√4000 (8)(247.7) + $\frac{1206.78 \cdot 247.7}{4(25.8 \times 12")}$ = 654 903 lbs = 655 kips

= $\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{25443 \times}{922} - \frac{25.8 \times 12}{2}$ = -127.3 < 0 ∴ doesn't apply.

φVc = 0.75 (655) = 491.3 ≯ Vu ∴ req'd Vs reinf.

§11.9.8 Vu = 922.1k > 0.5 φVc = 0.5 (.75)(655) = 245.6k

§11.9.9

Vs = $\frac{A_v f_y d}{s} = \frac{A_v}{s}$ (60 000)(20.64) = $\frac{A_v}{s}$ 1.238E6

by eq. 11-1 and 11-2

Vs ≥ $\left(\frac{V_u}{\phi} - V_c\right) = \left(\frac{922.1}{.75} - 655\right)$ = 574.5

$\frac{A_v}{s}$ (1.238E6) ≥ 574.5 x 1000 ⟶ $\frac{A_v}{s}$ ≥ .464

Smax horiz = min { lw/5 = 25.8 x 12/5 = 61.9"; 3h = 3(8) = 24; 18 ⟵ controlling

Smax vert = min { lw/3; 3h = 24; 18 ⟵ controls

6

h = 8"

min reinf:

horiz. $\rho_t = \frac{A_v}{h \cdot S_t} = .125 \frac{A_v}{S_t} > .0025$

vert. $\rho_\ell = \frac{A_v}{h \cdot S_\ell} = .125 \frac{A_v}{S_\ell} > \max \begin{cases} .0025 \\ .0025 + .5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_t - .0025) \end{cases}$

use min $(.125)\frac{2(.2)}{12" \text{ o.c.}} = .004 > .0025$ ✓ for ρ_t

$\rho_\ell = \frac{.125(2)(.31)}{12"} = .006 > \begin{cases} .0025 \\ .0025 + .5\left(2.5 - \frac{58 \times 12}{25.8 \times 12}\right)(.004 - .0025) \end{cases}$

$= .0027$

✓OKAY by min

req'd for str:

$A_v = \frac{s}{f_y \cdot d}(574.5)$ try 10" o.c.

$= \frac{10"(574.5)}{60 \times 247.7} = .387 \text{ in}^2$ → use (2) #4's @ $A_s = .40 \text{ in}^2$

# of layers req'd:

2 if $V_u > 2A_{cv}\sqrt{f'_c} = 2(8" \times 25.8 \times 12)\sqrt{4000} = 313.2$ k ✓ 2 layers

ck:

$\phi V_n \geq V_u$

$\phi(V_c + V_s) = .75\left(655 + \frac{.4}{10"}(60) \cdot 247.7"\right) = 937.k > 922k$

∴ use: (2) #4s @ 10" o.c. in 2 layers horiz. reinf. (shear)

Moment Strength

$M_u = 25\,443$ kft

$N_u = 1206.8$ k

following procedure in Ch18 of Wight & McGregor in allowance of ACI 318-11

ck: $\phi M_n \geq M_u$

$M_n = T\left(d - \frac{a}{2}\right) + N_u\left(\frac{l_w - a}{2}\right)$

recall:

$l_w = 25.8 \times 12 = 309.6$

$a = \frac{T + N_u}{.85 f'_c b}$

$T = A_s f_y = A_s 60^k$

$d = l_w - (3" + 3 \times 6) = 288.6"$ (steel estimate)

$b = 8"$

use #5 bars

$A_b = .31 \text{ in}^2$

6@6'

7

$a = \frac{60 A_s + 1206.8}{.85(4)(8)} = \frac{60 A_s + 1206.8}{27.2} = 2.21 A_s + 44.4$ using $f'_c = 4$ ksi, $\beta = 0.85$

$\therefore M_n = 60 A_s \left(288.6'' - \frac{1}{2}(2.21 A_s + 44.4)\right) + 1206.8 \left(309.6'' - (2.21 A_s + 44.4)\right)\frac{1}{2}$

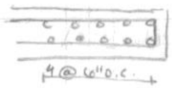
$= -66.3 A_s^2 + 14650.5 A_s + 160\,022$

ϕM_n where $\phi = 0.9$ for flexure $M_u = 25\,443$ k·ft x12"

→ solving for $A_{s\,req'd} = 10.41$ in²

use (10) #10s A_s provid = 10x1.27 = 12.7 in²

this provides enough str for tension reinf

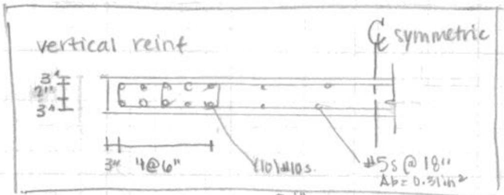


confirm: $\phi M_n > M_u$

$0.9\left[60(12.7)(294.6 - 72.4/2) + 1206.8\left(\frac{309.6 - 72.4}{2}\right)\right]$ $\therefore d = 294.6$

$a = \frac{60(12.7) + 1206.8}{27.2} = 72.4$

= 306 k·in > 25 443 kft x12 = 205 kin √okay



min sp btwn bars = max {1", db = 1.27} √ ok

$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} b_w \cdot d = \frac{3\sqrt{4000}}{60\,000} \cdot 8 \cdot 294.6 = 7.45 < 12.7$ in² ok √.

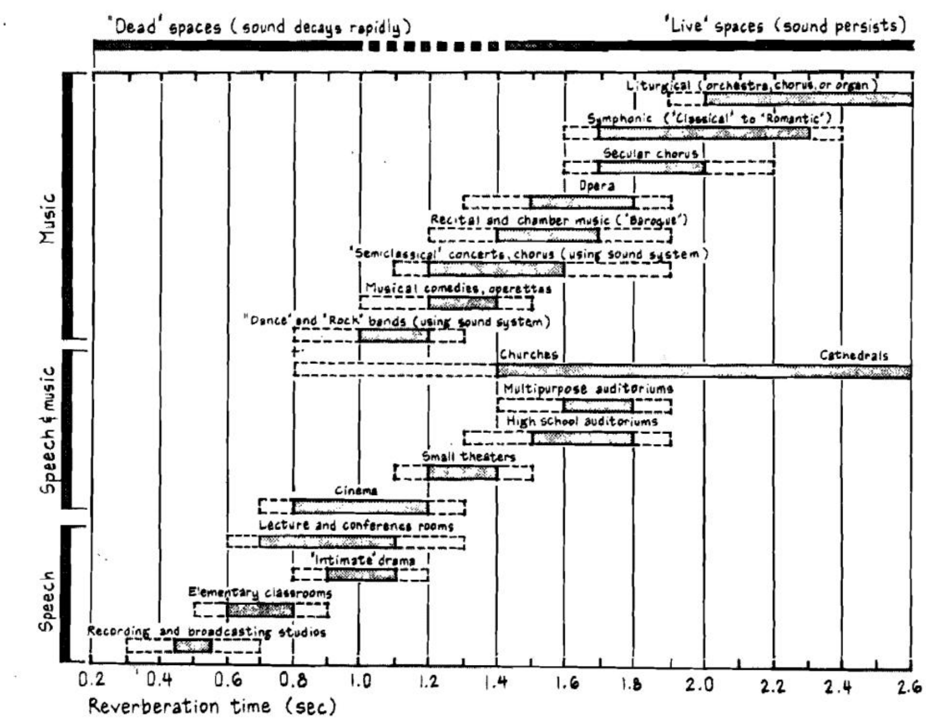
$\rho_{st} = .0018 = \frac{A_s}{A_c}$ → $A_s = (.0018)(6'' \times 25.8 \times 12) = 3.34$ in² or .13 in²/ft length

§7.12

∴ #5s @ 18" ok for S+T.

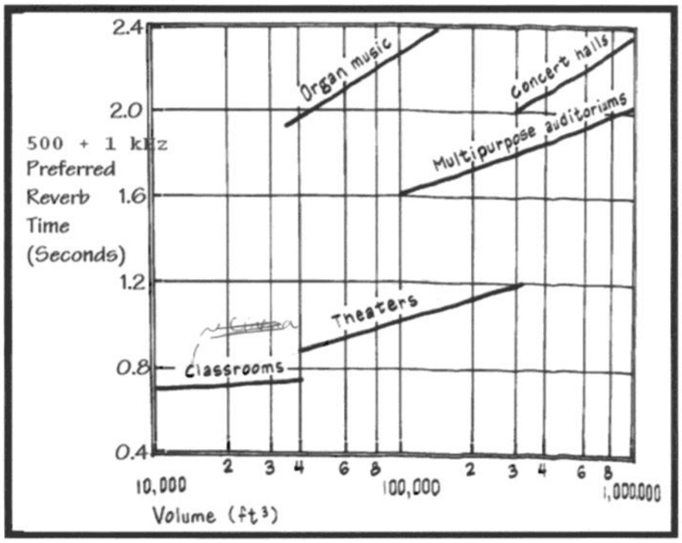
Appendix G Breadth – Acoustics

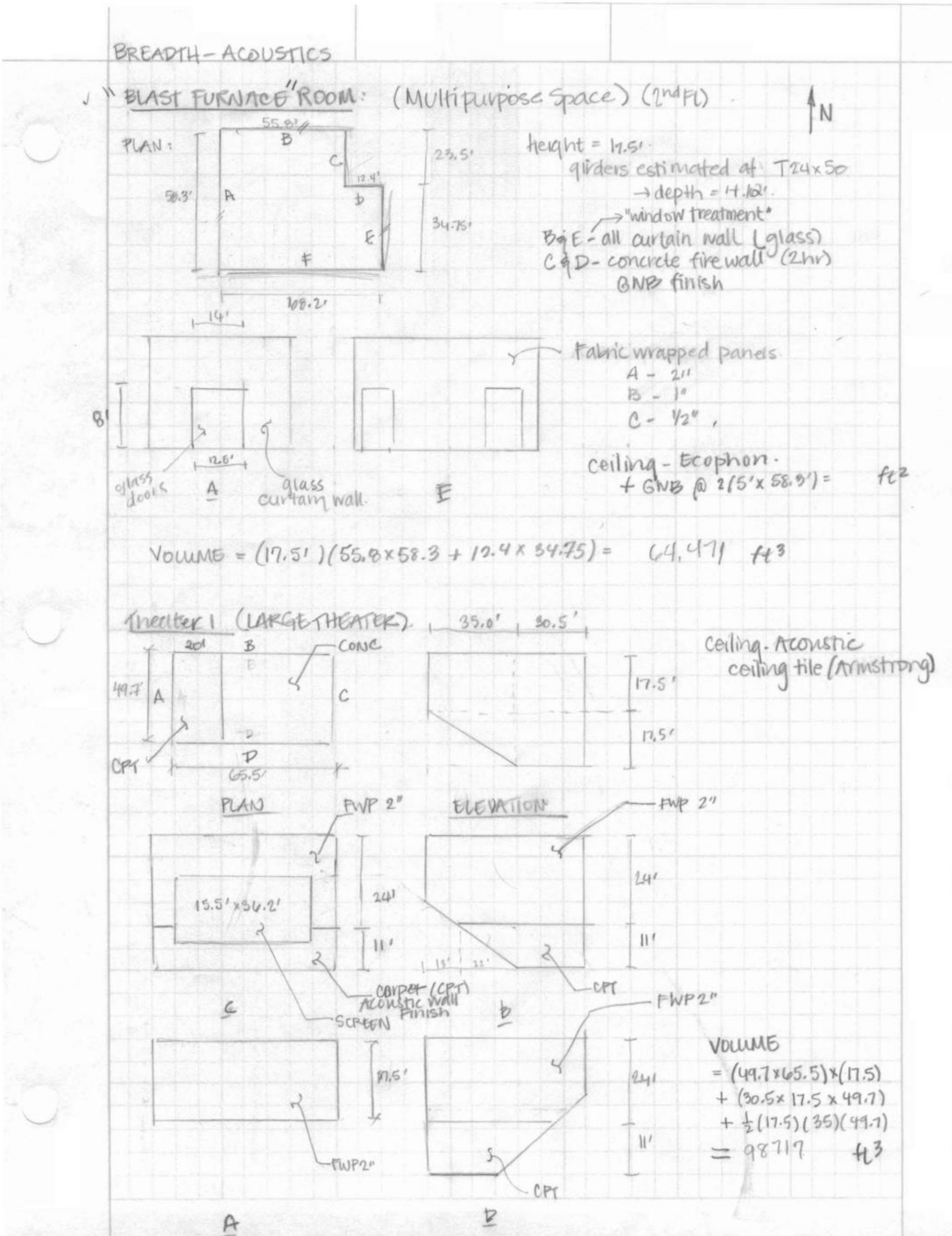
Reverberation Times

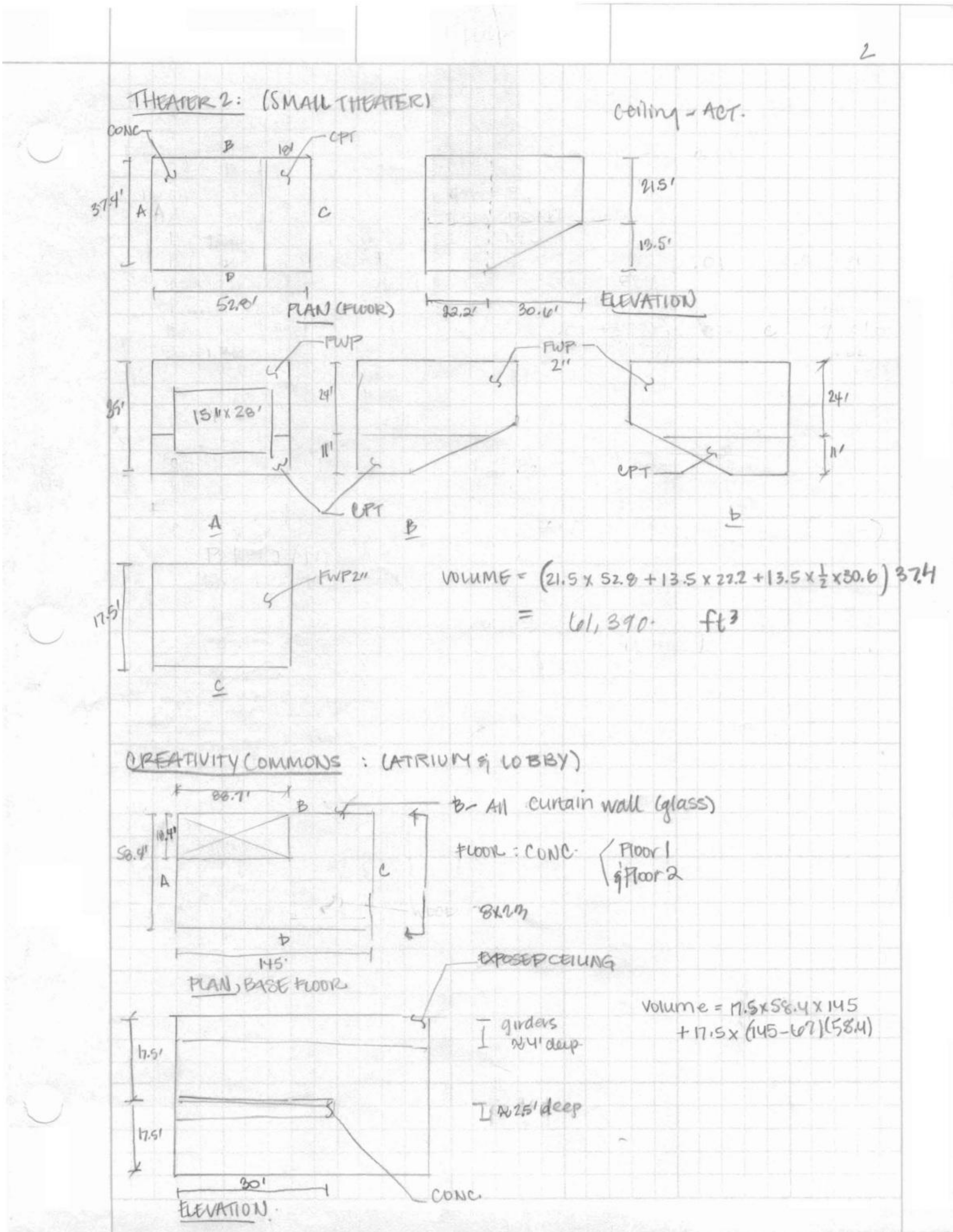


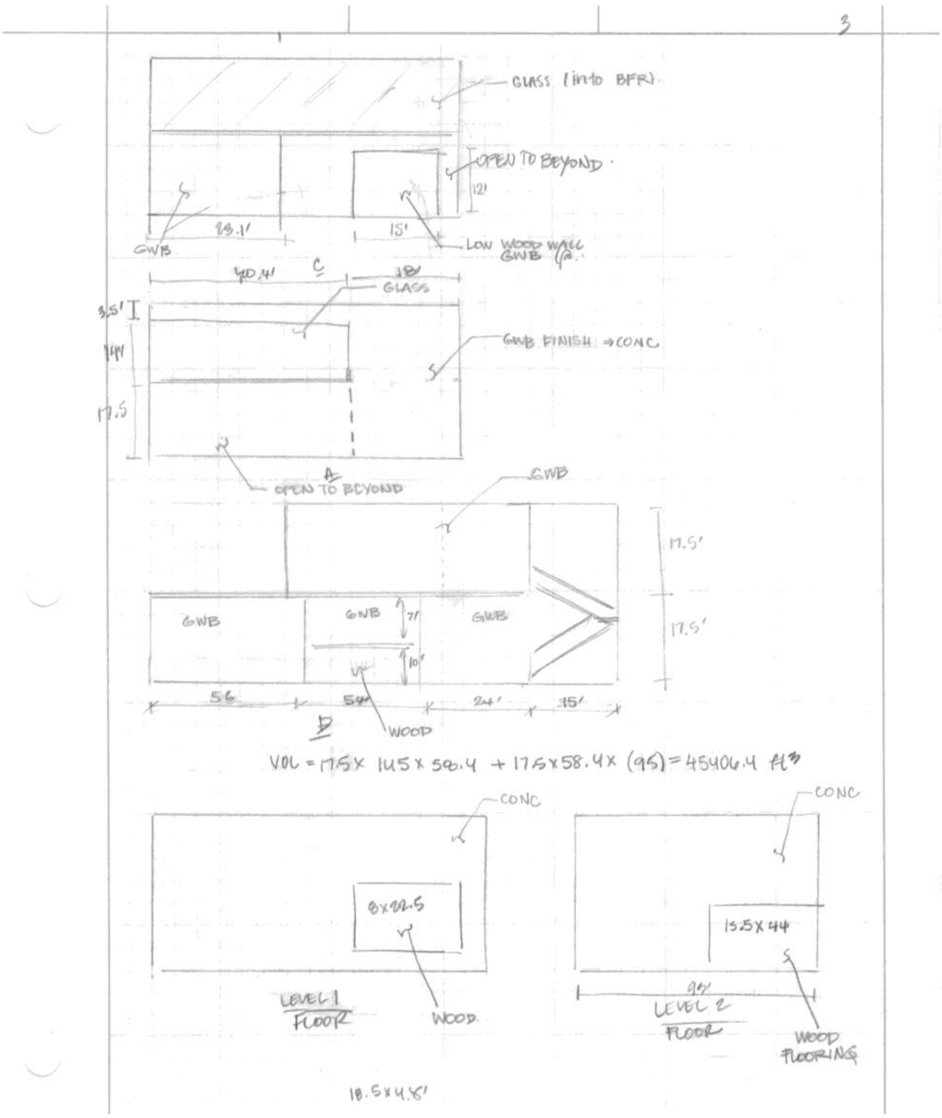
Note: Long reverberation times degrade speech perception of hearing-impaired persons far more than normal-hearing persons. For hearing-impaired and elderly listeners, reverberation times should be well below most of the values in the graph (e.g., < 0.5 s for satisfactory speech perception).

- Optimum T_{60} at 500 Hz and 1 kHz for different room usages (empirically derived):




BREADTH – ACOUSTICS
"BLAST FURNACE" ROOM: (Multipurpose Space) (2nd Fl)
N
PLAN:
height = 17.5'
girders estimated at T24x50
→ depth = 4.42'
"window treatment"
B & E – all curtain wall (glass)
C & D – concrete fire wall (2hr)
GWB finish
Fabric wrapped panels
A – 2"
B – 1"
C – 1/2"
glass doors
glass curtain wall
ceiling – Ecophon
VOLUME = (17.5')(55.8x58.3 + 12.4 x 34.75) = 64,471 ft³
Theater I (LARGE THEATER)
CONC
Ceiling – Acoustic ceiling tile (Armstrong)
PLAN
FWP 2"
ELEVATION
FWP 2"
carpet (CPT)
Acoustic wall finish
SCREEN
CPT
FWP2"
VOLUME
= (49.7x65.5)x(17.5)
+ (30.5x 17.5 x 49.7)
+ ½(17.5)(35)(49.7)
= 98717 ft³
FWP2"
CPT


2
THEATER 2: (SMALL THEATER)
Ceiling - ACT.
CONC
CPT
PLAN (FLOOR)
ELEVATION
FWP
CPT
VOLUME = (21.5 x 52.8 + 13.5 x 22.2 + 13.5 x ½ x 30.6) 37.4
= 61,390 ft³
CREATIVITY COMMONS : (ATRIUM & LOBBY)
B - All curtain wall (glass)
FLOOR : CONC.
Floor 1
& Floor 2
PLAN, BASE FLOOR
EXPOSED CEILING
girders
volume = 17.5 x 58.4 x 145
+ 17.5 x (145 - 67)(58.4)
ELEVATION
CONC.



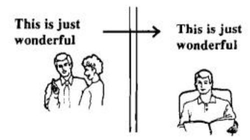
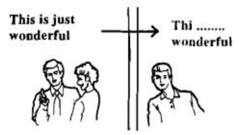
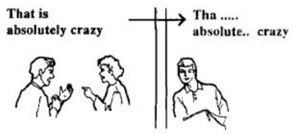
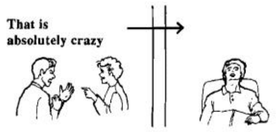
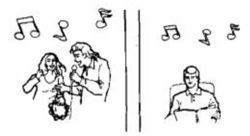
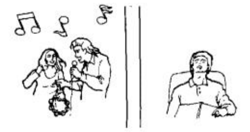
Reverberation Time Calculator

Volume (ft^3)	V =	64471.00
Total Surface Area (ft^2)	S_{total} =	11493.03

Surface Description	Surface Area S (ft^2)	Material Description	Sound Absorption Coefficient (α) Frequency (Hz) 125	250	500	1000	2000	4000	S*α (sabins) Frequency (Hz) 125	250	500	1000	2000	4000
Wall A	1020.25	Glass curtain wall	0.35	0.25	0.18	0.12	0.07	0.04	357.09	255.06	183.65	122.43	71.42	40.81
Wall B	976.50	Glass curtain wall	0.35	0.25	0.18	0.12	0.07	0.04	341.78	244.13	175.77	117.18	68.36	39.06
Wall C	411.25	GWB - PT	0.29	0.10	0.05	0.04	0.07	0.09	119.26	41.13	20.56	16.45	28.79	37.01
Wall D	217.00	GWB - PT	0.29	0.10	0.05	0.04	0.07	0.09	62.93	21.70	10.85	8.68	15.19	19.53
Wall E	608.13	Glass curtain wall	0.35	0.25	0.18	0.12	0.07	0.04	212.84	152.03	109.46	72.98	42.57	24.33
Wall F	103.89	Acoustic paneling A - 2"	0.37	0.95	0.99	0.99	0.99	0.99	38.44	98.70	102.85	102.85	102.85	102.85
Wall F	83.00	Acoustic paneling B - 1"	0.17	0.41	0.93	0.99	0.99	0.99	14.11	34.03	77.19	82.17	82.17	82.17
Wall F	85.00	Acoustic paneling C - 1/2"	0.12	0.14	0.45	0.90	0.99	0.99	10.20	11.90	38.25	76.50	84.15	84.15
Wall F	195.30	Acoustic paneling D - Perforated Metal	0.30	0.40	0.50	0.50	0.50	0.40	58.59	78.12	97.65	97.65	97.65	78.12
Wall F	314.01	GWB - PT	0.29	0.10	0.05	0.04	0.07	0.09	91.06	31.40	15.70	12.56	21.98	28.26
Wall F	116.20	Doors	0.29	0.10	0.05	0.04	0.07	0.09	33.70	11.62	5.81	4.65	8.13	10.46
Ceiling	583.00	GWB	0.29	0.10	0.05	0.04	0.07	0.09	169.07	58.30	29.15	23.32	40.81	52.47
Ceiling	3098.25	Acoustic Ceiling paneling	0.50	0.85	0.95	0.85	0.95	0.85	1549.13	2633.51	2943.34	2633.51	2943.34	2633.51
Floor	3681.25	Carpet	0.10	0.15	0.25	0.30	0.30	0.30	368.13	552.19	920.31	1104.38	1104.38	1104.38
								Σ(S*α)	3426.32	4223.81	4730.54	4475.30	4711.78	4337.11
								Avg. α	0.30	0.37	0.41	0.39	0.41	0.38
								Air Attenuation, m (ft^{-1})	0.00	0.00	1.83E-04	3.26E-04	7.86E-04	2.56E-03
							Sabine Reverberation Time (s)	**RT =**	0.92	0.75	0.66	0.69	0.64	0.63
							Norris-Eyring Reverberation Time (s)	**RT =**	0.78	0.60	0.51	0.55	0.50	0.52
							Calculated Reverberation Time (s)	**RT =**	**0.78**	**0.60**	**0.51**	**0.55**	**0.50**	**0.52**

Sound Transmission Criteria

Table 6.1 Subjective Perception of STC Values*

STC	FSTC	Subjective description	
30	22 - 25	 This is just wonderful → This is just wonderful	Most sentences clearly understood.
40	32 - 35	 This is just wonderful → Thi .... wonderful	Speech can be heard with some effort. Individual words and occasional phrases heard.
50	42 - 45	 That is absolutely crazy → Tha ..... absolute.. crazy	Loud speech can be heard with some effort. Music easily heard.
60	52 - 55	 That is absolutely crazy →	Loud speech essentially inaudible. Music heard faintly; bass note disturbing.
70	62 - 65		Loud music heard faintly, which could be a problem if the adjoining space is highly sensitive to sound intrusion, such as a recording studio, concert hall, etc.
75 and above			Most noises effectively blocked.

* This table assumes a reasonably quiet background noise level in the receiving room — NC 35 or less. See Chapter 8 for NC values.

Cost Comparison

	Label	Material	Crew	Output	Labor-hrs	unit	Material	Labor	Equipment	Total		Unit	Into S.F.	Total Cost per S.F.	Total Cost per System
1) EXISTING	300 4000 psi	concrete slab				CY	102			102		8" x1'x1'	0.02	2.52	
	5300	metal decking, 20 ga	E-4	3600	0.006	S.F.	2.01	0.45	0.04	2.5		1	1.00	2.50	
	1550	6-10" thick, pumped	c-7	110	0.655	CY		25.5	10.95	36.45		8"x1'x1'	0.02	0.90	
	1102	W12x16	e-2	880	0.064	L.F.	23	3.12	1.73	27.85		1	0.14	3.98	
		Welded shear connectors 3/4" diameters				S.F.	0.12	0.16		0.26			1.00	0.26	
															10.16
3) REDESIGN		6" conc slab				CY	102			102		6"	0.02	1.89	
	1550	6-10" thick, pumped	c-7	110	0.655	CY		25.5	10.95	36.45			0.02	0.90	
	650	formwork 4 use, girders & beams	c-2	310	0.155	SFCA	0.97	6.75		7.72	in2	336	0.19	1.50	
	2250	forms in place, elevated slabs	c-2	480	0.1	S.F.	2.22	4.37		6.59		1	1.00	6.59	
															10.88
2) EXISTING		conc slab				CY	102			102		8"	0.02	2.52	
	5300	metal decking, 20 ga	E-4	3600	0.006	S.F.	2.01	0.45	0.04	2.5			1.00	2.50	
	500	1" deflection spring isolators				EA.	34			34			0.06	2.13	
	2100	r-11 batt insulation	1 carp	700	0.011	S.F.	0.38	0.51		0.89			1.00	0.89	
	3000	1 layer 5/8" gypsum board	2 carp	1800	0.009	S.F.	0.27	0.4		0.67			1.00	0.67	
	3000	1 layer 5/8" gypsum board	2 carp	1800	0.009	S.F.	0.27	0.4		0.67			1.00	0.67	
	3000	+1 layer 5/8" gypsum board	2 carp	1800	0.009	S.F.	0.27	0.4		0.67			1.00	0.67	
	1550	6-10" thick, pumped	c-7	110	0.655	CY		25.5	10.95	36.45		8"	0.02	0.90	
	1102	W12x16	e-2	880	0.064	l.f.	23	3.12	1.73	27.85		1	0.14	3.98	
		Welded shear connectors 3/4" diameters				S.F.	0.12	0.16		0.26			1.00	0.26	
															15.18
4) REDESIGN	80	3/8" plywood	2 carp	1860	0.009	S.F.	0.81	0.39		1.2			1.00	1.20	
	3000	1 layer 5/8" gypsum board	2 carp	1800	0.009	S.F.	0.27	0.4		0.67			1.00	0.67	
	3000	1 layer 5/8" gypsum board	2 carp	1800	0.009	S.F.	0.27	0.4		0.67			1.00	0.67	
		3/4" plywood	2 carp	1300	0.012	S.F. Flr	1.24	0.55		1.79			1.00	1.79	
		6" conc				CY	102			102		6"	0.02	1.89	
	1550	6-10" thick, pumped	c-7	110	0.655	CY		25.5	10.95	36.45		6"	0.02	0.90	
	650	formwork 4 use, girders & beams	c-2	310	0.155	SFCA	0.97	6.75		7.72		336	0.19	1.50	
	2250	forms in place, elevated slabs	c-2	480	0.1	S.F.	2.22	4.37		6.59		1	1.00	6.59	
															15.21