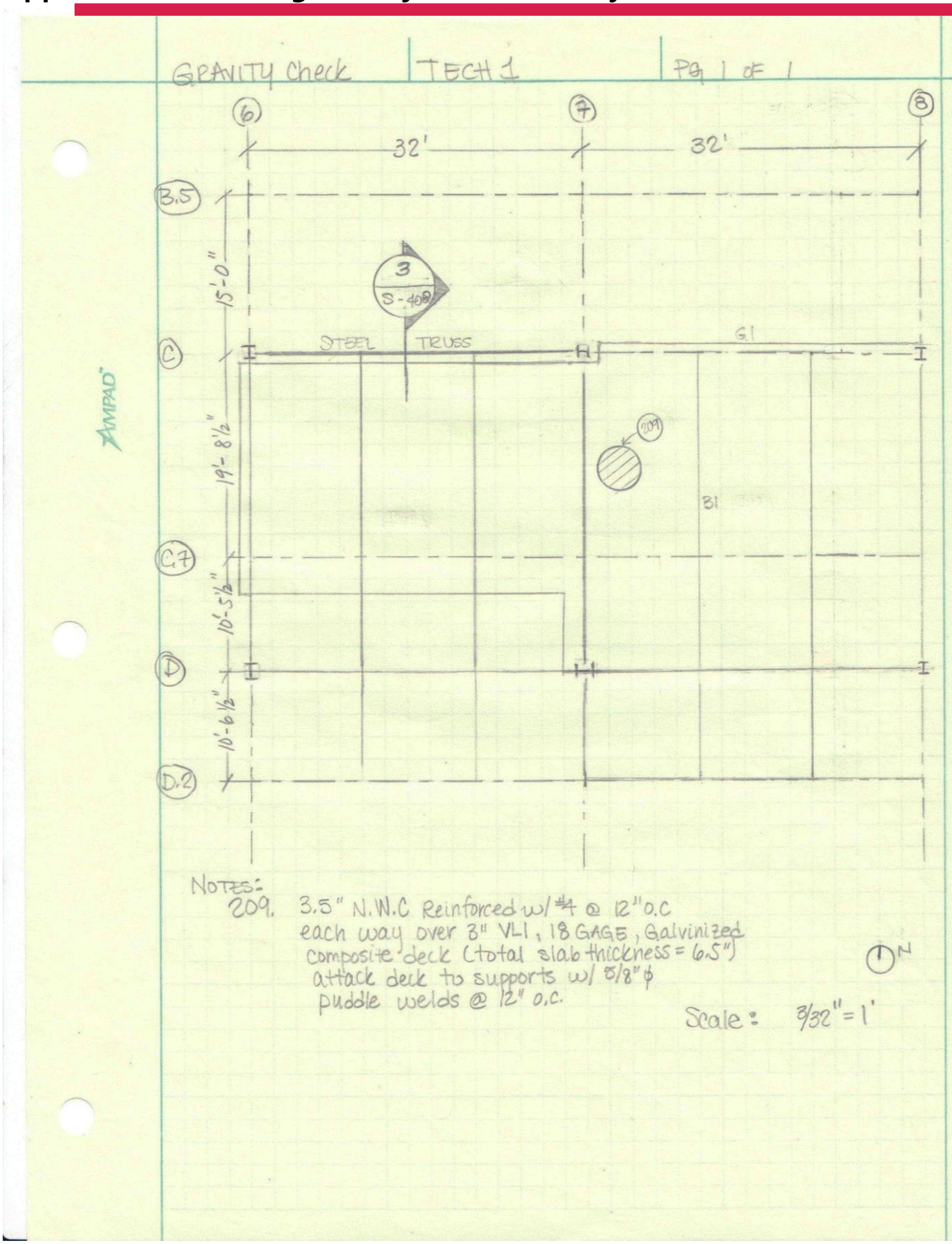
Appendix A – Existing Gravity and Lateral System Checks

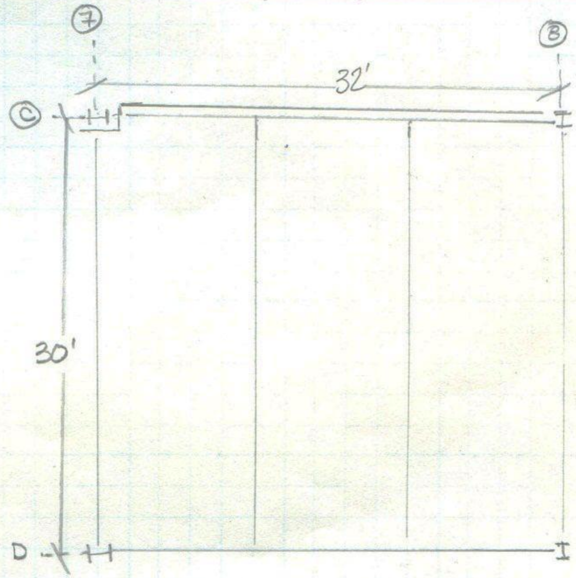


GRAVITY Check - SLAB | TECH 1 | PG 1 OF 1

Floor LOAD

Dead: Slab + deck

Live load: LL = 80 psf (corridors)
LL = 80 psf (classroom)
160 psf



Use: SDI No. 31
Unshored
3 span 10'-6" span
Composite deck
3 VLI t = 3.5
Normal weight concrete

① Maximum clear span

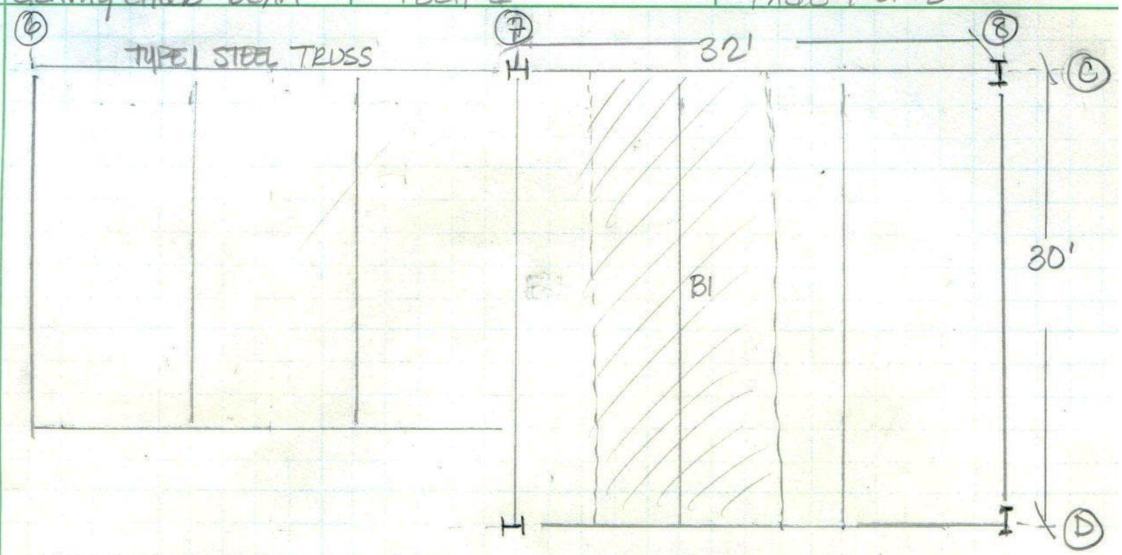
6.5" total thickness

Max span = 13'-10" > 10'-6" span ✓ok

② Reinforced concrete allowable loads
in Superimposed live load 229 psf ≥ 180 psf ✓ok
req'd = 229 psf for 18 gage
Slab information
6.5 depth → Use 6x6 - W2.1 x W2.1

Use 3 VLI w/ 3.5" concrete
reinforced with WWF 6x6 - W2.1 x W2.1

GRAVITY CHECK - BEAM | TECH 1 | PAGE 1 OF 2



ASTM A992 GRADE 50 WIDE FLANGE BEAM

3 span deck for unshored construction
Composite deck
max spacing = 12'-0" > 10'-8" ✓

Classroom, Corridor loading LL = 100 psf
1. self wt = 10 psf
2. Superimposed dead wt = 10 psf
3. slab NWC $t = 3.5"$
3VLI $d = 6.5"$
slab wt = 63 psf

$L = \max \left| \begin{array}{l} 80 \\ 70+50 \end{array} \right. = 80$ psf

$A_T = 30'(10'8") = 320$
$K_L = 2$ (ASCE 7-05, Table 4-2)

$L = 100 \times \max \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(320)}} = 0.842 \end{array} \right. = 0.842(80) = 67.36 \text{ ft}^2$ ∴ reduction ✓

$2(320) > 400 \text{ ft}^2$ ✓

LOADING:
DL = slab + deck + supt bm = 63 + 10 + 10 = 83 psf
$W_u = 1.2(83) + 1.6(67) = 206.8$ psf
$w_u = 207 \text{ psf}(10'\text{-}8") = 2205 \text{ plf} = 2.21$ klf

$V_u = \frac{w_u l}{2} = \frac{2.21 \text{ klf}(30')}{2} = 33.15$ kip

$M_u = \frac{w_u l^2}{8} = \frac{2.21 \text{ klf}(30')^2}{8} = 248.6$ k-ft

GRAVITY CHECK-BEAM | TECH I | PAGE 2 OF 2

- beams spaced @ 10'-8" O.C. w/ steel weight allowance (0p
- beam simply supported with deck on flange ∴ $L_b = 0$

→ Using AISC Steel Manual, 13th edition

- (Zx table 3-2) Using LRFD

$M_u \le \phi_b M_{px}$

W18x40 $\phi_b M_{px} = 294$ kip-ft $> M_u = 248.6$ k-ft

$V_u \le \phi_v V_{nx}$

W18x40 $\phi_v V_{nx} = 169$ k $> V_u = 33.15$ k

- $\Delta_{LL} \le L/360$ for W18x40 $I_x = 612 in^4$ $w_{LL} = 0.0842 ksf(10'-8") = 0.899k$

$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x} = \frac{5(0.714)(30')^4(1728)}{384(29000)(612)} = 0.923"$

$L/360 = \frac{30(12)}{360} = 1" > \Delta_{LL}$ ∴ ok ✓

- $\Delta_{TL} \le L/240$ for W18x40 $w_{TL} = (0.0842 + 0.083)(10'-8") = 1.783$ klf

$\Delta_{TL} = \frac{5 w_{TL} L^4}{384 E I_x} = \frac{5(1.59)(30')^4(1728)}{384(29000)(612)} = 1.642"$

$L/240 = \frac{30(12)}{240} = 1.5" < \Delta_{TL}$ ∴ DOES NOT WORK

$w_{bm} = 4.4\left(\frac{31}{10'-8"}\right)$

DID NOT WORK: TRY W21x44, $I_x = 843 in^4$

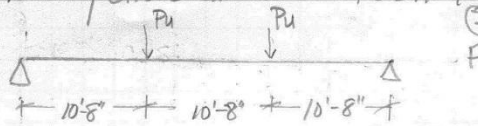
$\phi_b M_{px} = 358$ k-ft $> M_u$
$\phi V V_{nx} = 215^k > V_u$

$\Delta_{LL} = \frac{5(0.714)(30)^4(1728)}{384(29000)(843)} = 0.532" < L/360 = 1"$ ∴ ok ✓

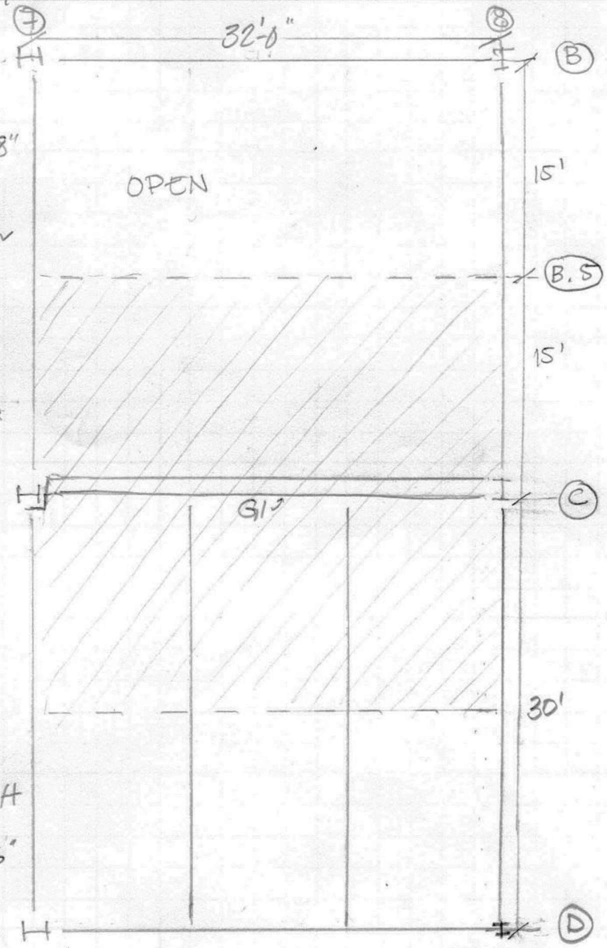
$\Delta_{TL} = \frac{5(1.59)(30)^4(1728)}{384(29000)(612)} = 1.18" < L/240 = 1.5"$ ∴ ok ✓

There may be the discrepancy because they used such a higher classroom load than ASCE 7-05. Therefore, they may have used judgement to use the W18x40 instead.

GRAVITY CHECK-GIRDER | TECH 1 | PAGE 1 OF 2



$\therefore L_b = 10'\text{-}8"$



$A_T = 30(32) = 960\ SF$

$K_{LL} = 2$

$A = K_{LL}A_T = 2(960) = 1920\ SF > 400SF$ ✓

$L = L_0\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = L_0\left(0.25 + \frac{15}{\sqrt{1920}}\right)$

$= L_0(0.5925)$

LL = 100 psf corridors/classroom

LL on girder = $0.5925(100psf)(30')/2$ *

$= 0.89\ klf/2 =$

DL on girder = slab + deck + sup + girder

$= (63/2) + 10 + 10$

$= 51.5 psf (30')$

$= 1.545\ klf$

DL from W18 = $0.04(30) = 1.2k$

$P_u = [1.2(1.55) + 1.6(0.89)](10'\text{-}8")$

$+ 1.2(1.2)$

$= 36.47^k$

$V_u = P_u = 36.47\ k$

$M_u = \frac{P_uL}{3} = \frac{36.5(32')}{3} = 583.5^k ft$

$\Delta_{LL} = \frac{P_uL^3}{28EI} \le \frac{L}{360} = \frac{32(12)}{360} = 1.066"$

$P_{uLL} = 0.89(10.66) = 9.49^k$

$\Delta_{TL} = \frac{P_uL^3}{28EI} \le \frac{L}{240} = \frac{32(12)}{240} = 1.6"$

$P_{u\,TL} = (1.55 + .89)(10.66) + 1.2$

$= 27.2\ k$

NOTE: Live load and slab+deck weight divided by 2 because they only load girder from C-D.

Using Z tables (Table 3-2 AISC Steel Manual) LRFD

$M_u \le \phi_b M_{px}$

W24x68 $\phi_b M_{px} = 664$ kip-ft $> M_u = 583.5^k ft$ ✓ok

$V_u \le \phi_v V_{nx}$

W24x68 $\phi_v V_{nx} = 295\ k > V_u = 36.47\ k$ ✓ok

$I_x = 1830 in^4$

$\Delta_{LL} = \frac{9.49(32^3)(1728)}{28(29000)(1830)} = 0.362" < L/360 = 1.066"$ ✓ok

$\Delta_{TL} = \frac{27.2(32^3)(1728)}{28(29000)(1830)} = 1.037" < L/240 = 1.6"$ ✓ok

OR CAN USE W21x68 but W24x68 provides a deeper connection to the truss?

The design used a W24x68.

GRAVITY CHECK - GIRDER | TECH 1 | PAGE 2 OF 2

Flange local buckling
$b_f/2t_f = 7.66$ (AISC Table 1-1)
$\lambda_p = 0.38\sqrt{29000/50}$ (Table B4.1)
$= 9.15$
$\lambda_p > b_f/2t_f$ Yes, flange compact

Web local buckling
$h/t_w = 52.0$ (AISC Table 1-1)
$\lambda_p = 3.76\sqrt{E/F_y}$ (Table B4.1)
$= 3.76\sqrt{29000/50}$
$= 90.6$
$\lambda_p > h/t_w$ Yes, web compact

Shear capacity
$V_u = W_u L/2 = 36.5k$
$d = 23.7 in$
$t_w = 0.45 in$

$A_w = d \cdot t_w = 10.67 in^2$ $\phi_{vn} = 1.0$
$\phi V_n = \phi_{vn}(0.6 F_{yw}) A_w = 320.1k$

$\phi V_n > V_u$ ok ✓

$\boxed{\text{Use } W24x68}$

Appendix B – Building Weight Calculations

Level	Area (SF)	Beams (kip)	Columns (kip)	Floor (kip)	Superimposed (kip)	Walls (kip)	Total Floor Weight (kip)	Weight/Area (psf)
Stair 3	380	0	0	26	11	0	37	98
High Roof	12,071	43	11	821	145	0	1,021	85
Low Roof	13,748	65	25	1,551	905	4	2,551	186
Level 4	24,275	153	24	1,551	905	4	2,638	109
Level 3	13,392	169	24	922	561	4	1,681	125
Level 2	25,867	203	33	1,790	1,028	5	3,057	118
Level 1	23,434	154	25	1,609	951	5	2,744	117
						Total Weight (kip)	**13,729**	

Building Weight

Stair 3

Approx. Area	=	380	SF				
Ht.	=	2	ft	Total Weight	=	37.06	k

Walls				**Superimposed**				**Floor**				**Beams**		
Height	=	0	ft	Partitions	=	0	psf	3VLI Deck	=	68	psf	Length (ft)	Joist	Weight (lb)
SW1, Length	=	0	ft	Misc.	=	10	psf	**Weight**	=	**25.83864**	**k**	11.5	10K1	57.5
SW2, Length	=	0	ft	Finishes	=	0	psf					11.5	10K1	57.5
SW3, Length	=	0	ft	Roof	=	20	psf					11.5	10K1	57.5
SW4, Length	=	0	ft	**Weight**	=	**10.8798**	**k**					11.5	10K1	57.5
SW5, Length	=	0	ft									11.5	10K1	57.5
SW6, Length	=	0	ft									11.5	10K1	57.5
SW7, Length	=	0	ft										**Total Weight (k) =**	**0.345**
SW8, Length	=	0	ft											
SW9, Length	=	32	ft											
SW10, Length	=	32	ft											
SW11, Length	=	12.33	ft											
SW12, Length	=	12.33	ft											
SW13, Length	=	0	ft											
Unit Wt.	=	145	pcf											
Weight	=	**0.00**	**k**											

High Roof			
Approx. Area	=	12,071	SF
Ht.	=	7	ft
Total Weight	=	1021.31	k

Walls			
Height	=	2	ft
SW1, Length	=	0	ft
SW2, Length	=	0	ft
SW3, Length	=	0	ft
SW4, Length	=	0	ft
SW5, Length	=	0	ft
SW6, Length	=	0	ft
SW7, Length	=	0	ft
SW8, Length	=	0	ft
SW9, Length	=	32	ft
SW10, Length	=	32	ft
SW11, Length	=	12.33	ft
SW12, Length	=	12.33	ft
SW13, Length	=	0	ft
Unit Wt.	=	145	pcf
Weight	**=**	**0.20**	**k**

Superimposed			
Mechanical	=	0	psf
Misc.	=	10	psf
Finishes	=	0	psf
Roof	=	20	psf
Weight	**=**	**145.492023**	**k**

Floor			
3VLI Deck	=	68	psf
Weight	**=**	**820.8418**	**k**

Columns		
Height	Shape	Weight
7	HSS6X6X1/8	68.95
7	W10X33	231
7	W10X33	231
7	W10X33	231
7	W10X33	231
7	W10X33	231
7	W10X39	273
7	W10X45	315
7	W12X136	952
7	W12X40	280
7	W12X40	280
7	W12X53	371
7	W12X79	553
7	W12X87	609
7	W14X145	1015
7	W14X159	1113
7	W14X159	1113
7	W14X211	1477
7	W14X211	1477
7	W14X43	301
	Total Weight (k) =	**11.35295**

Beams		
Length (ft)	Beam Size	Weight
9.33	W10X12	111.96
9.33	W10X12	111.96
20.54	W12X14	287.56
7.58	W12X14	106.12
7.58	W12X14	106.12
7.58	W12X14	106.12
30	W12X16	480
30	W12X16	480
30	W14X22	660
30	W14X22	660
30	W14X22	660
30	W14X22	660
23.67	W14X34	804.78
23.67	W14X34	804.78
23.67	W14X43	1017.81
30	W16X26	780
19.54	W16X26	508.04
30	W16X36	1080
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
32	W18X40	1280
31.5	W21X50	1575
32	W21X50	1600
32	W21X50	1600
32	W21X50	1600
32	W21X50	1600
20.04	24K7	202.404
30	24K7	303
30	24K7	303
30	24K7	303
30	24K7	303
29.5	24K7	297.95
29.5	24K7	297.95
29.5	24K7	297.95
1080	24K7	10908
	Total Weight (k) =	43.416504

Note: Low Roof, Level 4, Level 3, Level 2, and Level 1 detailed building weight calculations are available upon request.

Difference in Dead Load from Calculated to RAM Model					
Level	**Area (SF)**	**Calculated Floor Weight (kip)**	**RAM Model, Floor Weight (kip)**	**% Difference**	**Floor Weight Used for ETABS and Seismic Calcs (k)**
Stair 3	380	37	14.2	-161%	14
High Roof	12,071	1,021	511	-100%	511.27
Low Roof	13,748	2,544	668	-281%	667.55
Level 4	24,275	2,638	3,870	32%	3869.78
Level 3	13,392	1,681	1,473	-14%	1473.26
Level 2	25,867	3,057	2,823	-8%	2823.35
Level 1	23,434	2,744	2,979	8%	2979.48

Appendix C – Wind Load Calculations

ASCE 7-05 Chapter 6 Method 2

Wind Load Design Criteria	
Basic Wind Speed	90 MPH
Wind Importance Factor	IW = 1.15
Building Category	III
Exposure	C
Internal Pressure Coefficient , GCpi	GCPI = 0.18
Apply Directionality Factor	Kd = 0.85
Topography Factor	Kzt = 1.00
Mean Roof Height (ft): Top Story Height + Parapet =	71.83
Fundamental Frequency, n1 = 75/H = 1.044 >1	Structure is Rigid

Main Wind Force Resisting System – Method 2	All Heights
Figure 6-6 (con't) External Pressure Coefficients, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings	

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
200 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

External Pressure Coefficients (Cp)				
Wall Pressure Coefficients (C_p)				
Surface	L/B (X)	L/B (Y)	Cp (X)	Cp (Y)
Windward Wall	All Values	All Values	0.8	0.8
Side Wall	All Values	All Values	-0.7	-0.7
Leeward Wall				
Stair 3	2.60	0.39	-0.270	-0.5
High Roof	3.87	0.26	-0.207	-0.5
Low Roof	1.97	0.51	-0.306	-0.5
Level 4	2.04	0.49	-0.298	-0.5
Level 3	2.04	0.49	-0.298	-0.5
Level 2	2.13	0.47	-0.293	-0.5
Level 1	2.13	0.47	-0.293	-0.5
Base	2.13	0.47	-0.293	-0.5
Roof Pressure Coefficients (Cp)				
h/L	X:	0.281	Y:	0.599
			Cp (X)	Cp (Y)
Roof - 0 to h/2			-0.900	-0.979
Roof - h/2 to h			-0.900	-0.861
Roof - h to 2h			-0.500	-0.539
Roof - > 2h			-0.300	-0.379

Importance Factor, I (Wind Loads)

Table 6-1

Category	Non-Hurricane Prone Regions and Hurricane Prone Regions with V = 85-100 mph and Alaska	Hurricane Prone Regions with V > 100 mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

Terrain Exposure Constants

Table 6-2

Exposure	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (ft)	$\bar{\epsilon}$	z_{min} (ft)*
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

*z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is greater of 0.6*h* or z_{min}. For buildings with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min}.

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right) \quad (6\text{-}4)$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \quad (6\text{-}5)$$

In SI: $I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6}$

where $I_{\bar{z}}$ = the intensity of turbulence at height $\bar{z}$ where $\bar{z}$ = the equivalent height of the structure defined as $0.6h$, but not less than z_{min} for all building heights h. z_{min} and c are listed for each exposure in Table 6-2; g_Q and g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad (6\text{-}6)$$

where B, h are defined in Section 6.3; and $L_{\bar{z}}$ = the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \quad (6\text{-}7)$$

Gust Effect Factor (Gf)		
Variable	N-S Wind (Y)	E-W Wind (X)
I (Table 6-1)	1.15	1.15
c (Table 6-2)	0.2	0.2
g_Q	3.4	3.4
g_v	3.4	3.4
z_{mean}	43.10	43.10
$I_{z,\ mean}$	0.191	0.191
$L_{z,\ mean}$	527.43	527.42
Q		
Stair 3	0.963	0.970
High Roof	0.902	0.954
Low Roof	0.901	0.935
Level 4	0.899	0.935
Level 3	0.899	0.935
Level 2	0.896	0.935
Level 1	0.896	0.935
Base	0.896	0.935
G		
Stair 3	0.907	0.910
High Roof	0.877	0.903
Low Roof	0.877	0.893
Level 4	0.876	0.893
Level 3	0.876	0.893
Level 2	0.874	0.893
Level 1	0.874	0.893
Base	0.874	0.893

	E-W Wind (X)		N-S Wind (Y)	
PEARL HALL	B (ft)	L (ft)	B (ft)	L (ft)
Stair 3	12.33	32	32	12.33
High Roof	60	232.08	232.08	60
Low Roof	120	236.34	236.34	120
Level 4	120	244.67	244.67	120
Level 3	120	244.67	244.67	120
Level 2	120	256	256	120
Level 1	120	256	256	120
Base	120	256	256	120

6.5.10 Velocity Pressure. Velocity pressure, q_z, evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \text{ (lb/ft}^2\text{)} \quad (6\text{-}15)$$

[In SI: $q_z = 0.613 K_z K_{zt} K_d V^2 I$ (N/m²); V in m/s]

Velocity Pressure Coefficients (Kz) and Velocity			
Level	Elevation (ft)	Kz	q_z(psf)
Stair 3	71.83	1.177	23.86
High Roof	69.83	1.169	23.70
Low Roof	62.83	1.141	23.13
Level 4	50.50	1.092	22.13
Level 3	38.50	1.031	20.90
Level 2	26.50	0.952	19.30
Level 1	13.50	0.850	17.23
Base	0.00	0.850	17.23

Wind Pressures E-W Direction (X)							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressures (psf)		Net Pressures (psf)	
				(+)(GCpi)	(-)(GCpi)	(+)(GCpi)	(-)(GCpi)
Windward Walls	Stair 3	71.83	17.4	4.30	-4.30	21.7	13.1
	High Roof	69.83	17.1	4.27	-4.27	21.4	12.8
	Low Roof	62.83	16.5	4.16	-4.16	20.7	12.4
	Level 4	50.50	15.8	3.98	-3.98	19.8	11.8
	Level 3	38.50	14.9	3.76	-3.76	18.7	11.2
	Level 2	26.50	13.8	3.47	-3.47	17.3	10.3
	Level 1	13.50	12.3	3.10	-3.10	15.4	9.2
	Base	0.00	12.3	3.10	-3.10	15.4	9.2
Leeward Walls	Stair 3	71.83	-5.9	4.30	-4.30	-1.6	-10.2
	High Roof	69.83	-4.4	4.27	-4.27	-0.2	-8.7
	Low Roof	62.83	-6.3	4.16	-4.16	-2.2	-10.5
	Level 4	50.50	-5.9	3.98	-3.98	-1.9	-9.9
	Level 3	38.50	-5.6	3.76	-3.76	-1.8	-9.3
	Level 2	26.50	-5.1	3.47	-3.47	-1.6	-8.5
	Level 1	13.50	-4.5	3.10	-3.10	-1.4	-7.6
	Base	0.00	-4.5	3.10	-3.10	-1.4	-7.6
Side Walls	All	All	-0.7	4.30	-4.30	3.6	-5.0
Roof - 0 to h/2		0 to 35.92	-0.9	4.30	-4.30	3.4	-5.2
Roof - h/2 to h		35.92 to 71.83	-0.9	4.30	-4.30	3.4	-5.2
Roof - h to 2h		71.83 to 143.66	-0.5	4.30	-4.30	3.8	-4.8
Roof - > 2h		>143.66	-0.3	4.30	-4.30	4.0	-4.6

Wind Pressures N-S Direction (Y)							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressures (psf)		Net Pressures (psf)	
				(+)(GCpi)	(-)(GCpi)	(+)(GCpi)	(-)(GCpi)
Windward Walls	Stair 3	71.83	17.3	4.30	-4.30	21.6	13.0
	High Roof	69.83	16.6	4.27	-4.27	20.9	12.4
	Low Roof	62.83	16.2	4.16	-4.16	20.4	12.1
	Level 4	50.50	15.5	3.98	-3.98	19.5	11.5
	Level 3	38.50	14.6	3.76	-3.76	18.4	10.9
	Level 2	26.50	13.5	3.47	-3.47	17.0	10.0
	Level 1	13.50	12.1	3.10	-3.10	15.2	8.9
	Base	0.00	12.1	3.10	-3.10	15.2	8.9
Leeward Walls	Stair 3	71.83	-10.8	4.30	-4.30	-6.5	-15.1
	High Roof	69.83	-10.4	4.27	-4.27	-6.1	-14.7
	Low Roof	62.83	-10.1	4.16	-4.16	-6.0	-14.3
	Level 4	50.50	-9.7	3.98	-3.98	-5.7	-13.7
	Level 3	38.50	-9.2	3.76	-3.76	-5.4	-12.9
	Level 2	26.50	-8.4	3.47	-3.47	-5.0	-11.9
	Level 1	13.50	-7.5	3.10	-3.10	-4.4	-10.6
	Base	0.00	-7.5	3.10	-3.10	-4.4	-10.6
Side Walls	All	All	-0.7	4.30	-4.30	3.6	-5.0
Roof - 0 to h/2		0 to 35.92	-1.0	4.30	-4.30	3.3	-5.3
Roof - h/2 to h		35.92 to 71.83	-0.9	4.30	-4.30	3.4	-5.2
Roof - h to 2h		71.83 to 143.66	-0.5	4.30	-4.30	3.8	-4.8
Roof - > 2h		>143.66	-0.4	4.30	-4.30	3.9	-4.7

Windward Forces in E-W Direction (Cases 1-IV)

Wind Forces E-W Direction (WIND 1X)

Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.3	0.3	20.0
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	7.0	7.3	507.0
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	26.0	33.2	2138.1
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	32.5	65.8	3781.7
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	30.4	96.2	4951.8
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	29.5	125.6	5732.5
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	28.3	153.9	6115.0
Base	120	-	0.00	-	0.0	6.8	810.0	13.6	167.6	6115.0
									Total Base Shear (k) =	168
									Total Overturning Moment (k-ft) =	6115

Wind Forces E-W Direction (WIND 2X)

Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	M_T (+e_x)	M_T (-e_x)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.2	0.2	15.0	4.5	-4.5
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	5.2	5.4	380.2	3922.0	-3922.0
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	19.5	24.9	1603.6	42053.8	-42053.8
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	24.4	49.3	2836.3	52727.5	-52727.5
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	22.8	72.1	3713.9	49235.2	-49235.2
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	22.1	94.2	4299.4	47725.4	-47725.4
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	21.2	115.5	4586.3	45896.5	-45896.5
Base	120	-	0.00	-	0.0	6.8	810.0	10.2	125.7	4586.3	22081.4	-22081.4
									Total Base Shear (k) =	126		
									Total Overturning Moment (k-ft) =	4586		

Wind Forces E-W Direction (WIND 3X)

Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.2	0.2	15.0
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	5.2	5.4	380.2
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	19.5	24.9	1603.6
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	24.4	49.3	2836.3
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	22.8	72.1	3713.9
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	22.1	94.2	4299.4
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	21.2	115.5	4586.3
Base	120	-	0.00	-	0.0	6.8	810.0	10.2	125.7	4586.3
									Total Base Shear (k) =	126
									Total Overturning Moment (k-ft) =	4586

Wind Forces E-W Direction (WIND 4X)

Floor	B_x (ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	$M_{T\,(-e_x,-e_y)}$	$M_{T\,(+e_x,-e_y)}$
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	12	2	71.83	1.0	12.0	-	0.0	0.2	0.2	11.3	-81.3	-74.5
High Roof	70.7083	7	69.83	3.5	247.5	1.0	70.7	3.9	4.1	285.4	-106932.4	-101044.1
Low Roof	120	12.333	62.83	6.2	740.0	3.5	420.0	14.6	18.7	1203.7	-316459.7	-253322.9
Level 4	120	12	50.50	6.0	720.0	6.2	740.0	18.3	37.0	2129.1	-420910.0	-341748.4
Level 3	120	12	38.50	6.0	720.0	6.0	720.0	17.1	54.1	2787.9	-400563.6	-326645.1
Level 2	120	13	26.50	6.5	780.0	6.0	720.0	16.6	70.7	3227.4	-431119.3	-359467.5
Level 1	120	13.5	13.50	6.8	810.0	6.5	780.0	16.0	86.7	3442.7	-423713.3	-354807.3
Base	120	-	0.00	-	0.0	6.8	810.0	7.7	94.3	3442.7	-203853.5	-170702.0
									Total Base Shear (k) =	94		
									Total Overturning Moment (k-ft) =	3443		

Windward Forces in N-S Direction (Cases 1-IV)

Wind Forces N-S Direction (WIND 1Y)

Floor	B_y(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.9	0.9	64.7
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	22.9	23.8	1661.2
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	60.4	84.2	5456.0
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	75.4	159.6	9265.2
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	71.9	231.5	12034.2
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	71.4	302.9	13926.9
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	70.3	373.3	14876.4
Base	256	-	0.00	-	0.0	6.8	1728.0	33.8	407.1	14876.4
									Total Base Shear (k) =	407
									Total Overturning Moment (k-ft) =	14876

Wind Forces N-S Direction (WIND2Y)

Floor	By(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	M_T (+e_y)	M_T (-e_y)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.7	0.7	48.5	103.7	-103.7
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	17.1	17.8	1245.9	138527.8	-138527.8
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	45.3	63.1	4092.0	379517.7	-379517.7
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	56.6	119.7	6948.9	507987.4	-507987.4
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	53.9	173.6	9025.7	484375.2	-484375.2
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	53.6	227.2	10445.2	526589.7	-526589.7
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	52.7	279.9	11157.3	518552.8	-518552.8
Base	256	-	0.00	-	0.0	6.8	1728.0	25.4	305.3	11157.3	249481.9	-249481.9
									Total Base Shear (k) =	305		
									Total Overturning Moment (k-ft) =	11157		

Wind Forces N-S Direction (WIND3Y)

Floor	By(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)			
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.7	0.7	48.5
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	17.1	17.8	1245.9
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	45.3	63.1	4092.0
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	56.6	119.7	6948.9
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	53.9	173.6	9025.7
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	53.6	227.2	10445.2
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	52.7	279.9	11157.3
Base	256	-	0.00	-	0.0	6.8	1728.0	25.4	305.3	11157.3
									Total Base Shear (k) =	305
									Total Overturning Moment (k-ft) =	11157

Wind Forces N-S Direction (WIND4Y)

Floor	By(ft)	Height (ft)	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)	$M_{T\,(+e_x,+e_y)}$	$M_{T\,(-e_x,+e_y)}$
				Height (ft)	Area (ft2)	Height (ft)	Area (ft2)					
Stair 3	32	2	71.83	1.0	32.0	-	0.0	0.5	0.5	36.4	81.3	74.5
High Roof	232.08	7	69.83	3.5	812.3	1.0	32.0	12.9	13.4	935.2	106932.4	101044.1
Low Roof	236.34	12.333	62.83	6.2	1457.4	3.5	812.3	34.0	47.4	3071.7	316459.7	253322.9
Level 4	244.67	12	50.50	6.0	1468.0	6.2	1457.4	42.5	89.8	5216.3	420910.0	341748.4
Level 3	244.67	12	38.50	6.0	1468.0	6.0	1468.0	40.5	130.3	6775.3	400563.6	326645.1
Level 2	256	13	26.50	6.5	1664.0	6.0	1468.0	40.2	170.6	7840.9	431119.3	-431119.3
Level 1	256	13.5	13.50	6.8	1728.0	6.5	1664.0	39.6	210.1	8375.4	423713.3	-423713.3
Base	256	-	0.00	-	0.0	6.8	1728.0	19.1	229.2	8375.4	203853.5	-203853.5
									Total Base Shear (k) =	229		
									Total Overturning Moment (k-ft) =	8375		

Approximate Fundamental Frequency. To estimate the dynamic response of structures, knowledge of the fundamental frequency (lowest natural frequency) of the structure is essential. This value would also assist in determining if the dynamic response estimates are necessary. Most computer codes used in the analysis of structures would provide estimates of the natural frequencies of the structure being analyzed. However, for the preliminary design stages some empirical relationships for building period T_a ($T_a = 1/n_1$) are available in the earthquake chapters of ASCE 7. However, it is noteworthy that these expressions are based on recommendations for earthquake design with inherent bias toward higher estimates of fundamental frequencies [Refs. C6-48, C6-49]. For wind design applications these values may be unconservative because an estimated frequency higher than the actual frequency would yield lower values of the gust effect factor and concomitantly a lower design wind pressure. However, [Refs. C6-48, C6-49] also cite lower bound estimates of frequency that are more suited for use in wind applications. These expressions are

For steel Moment-Resisting-Frames MRFs

$$n_1 = 22.2/H^{0.8} \qquad \text{(C6-14)}$$

For concrete MRFs: $n_1 = 43.5/H^{0.9}$ (C6-15)

For concrete shearwall systems:

$$n_1 = 385(C_w)^{0.5}/H \qquad \text{(C6-16)}$$

where

$$c_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{H}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

n_1 = building natural frequency (Hz)
H = building height (ft)
n = number of shear walls in the building effective in resisting lateral forces in the direction under consideration
A_B = base area of the structure (ft^2)
A_i = area of shear wall (ft^2)
D_i = length of shear wall "i" (ft)
h_i = height of shear wall "i" (ft)

Observation from wind tunnel testing of buildings where frequency is calculated using analysis software reveals the following expression for frequency, applicable to all buildings in steel or concrete:

$n_1 = 100/H$ (ft) average value (C6-17)

$n_1 = 75/H$ (ft) lower bound value (C6-18)

Appendix D – Seismic Load Calculations

ASCE 7-05 Equivalent Lateral Force Method (Special Reinforced Concrete Shear Walls)

Seismic Load Design Criteria	ASCE 7-05		
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
Ss	0.564	g	*§11.4.1, Fig. 22-1*
S1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
Fa	1.349		*Table 11.4-1*
Fv	2.120		*Table 11.4-2*
SMS	0.761	g	*EQ. 11.4-1*
SM1	0.360	g	*EQ. 11.4-2*
SDS	0.507	g	*EQ. 11.4-3*
SD1	0.240	g	*EQ. 11.4-4*

Period Determination				
C_t		0.020		*Table 12.8-2*
x		0.750		*Table 12.8-2*
TL		6.000	sec	*Fig. 22-15*
Cu		1.46		*Table 12.8-1*
$T_a = C_t h_n^x$		0.493	sec	*EQ. 12.8-7*
$C_w = \frac{100}{A_B}\sum_{i=1}^{x}\left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$				*EQ. 12.8-10*
	X	0.11		
	Y	0.10		
$T_a = \frac{0.0019}{\sqrt{C_w}} h_n$	X	0.420	sec	*EQ. 12.8-9*
	Y	0.430	sec	
Tx		0.295	sec	*ETABS*
Ty		0.5243	sec	ETABS

Calculation of Seismic Response Coefficient Special Reinforced Concrete Shear Walls				
R (Special reinforced concrete shear walls)		6.000		*Table 12.2-1*
$C_s = \frac{S_{DS}}{(R/I)}$		0.106		*EQ. 12.8-2*
$C_s = \frac{S_{D1}}{T(R/I)}, T \leq T_L$	X	0.119		*EQ. 12.8-3*
	Y	0.116		
$C_s \geq 0.01$		ok		*EQ. 12.8-5*
$C_s =$	X	0.106		*ETABS*
	Y	0.096		ETABS
k	X	1.00		*§12.8.3*
k	Y	1.01		*§12.8.3*
Base shear, $V = C_s W$		1764.0	kip	*EQ. 12.8-1*
Base shear, $V = C_s W$		1593.7	kip	*EQ. 12.8-1*

Calculation of Cw

for the calculation of the approximate fundamental period, Ta for concrete shear wall structures

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Direction	hi	Di	Ai	CW/(100/AB)
1	12	34.00	90	62.83	X	62.83	34.00	34.00	11.59
2	12	34.00	90	69.83	X	69.83	34.00	34.00	7.99
3	12	33.00	0	69.83	Y	69.83	33.00	33.00	7.40
4	12	33.00	0	62.83	Y	62.83	33.00	33.00	10.76
5	12	23.17	0	69.83	Y	69.83	23.17	23.17	2.87
6	12	23.17	0	69.83	Y	69.83	23.17	23.17	2.87
7	12	10.33	90	68.83	X	68.83	10.33	10.33	0.30
8	12	10.33	90	69.83	X	69.83	10.33	10.33	0.28
9	12	32.00	90	71.83	X	71.83	32.00	32.00	6.18
10	12	32.00	90	71.83	X	71.83	32.00	32.00	6.18
11	18	12.33	0	71.83	Y	71.83	12.33	18.50	0.63
12	18	12.33	0	71.83	Y	71.83	12.33	18.50	0.63
13	24	23.17	0	69.83	Y	69.83	23.17	46.34	5.74
AB =	30720	$C_w = \frac{100}{A_B}\sum_{i=1}^{x}\left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$						Shear Walls 1-13	
hn=	71.83							CW - X =	0.106
								CW - Y =	0.101

Exisiting Special Reinforced Shear Walls - Seismic Forces (E-W Dirction, X)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	31.0	71.83	2226.3	0.00	6.7	7	481
High Roof	736.3	69.83	51414.1	0.09	154.8	161	11291
Low Roof	1208.6	62.83	75936.6	0.13	228.6	390	25655
Level 4	4373.9	50.50	220881.7	0.38	665.0	1055	59238
Level 3	2046.4	38.50	78787.6	0.13	237.2	1292	68371
Level 2	3438.7	26.50	91125.2	0.16	274.4	1567	75641
Level 1	4853.5	13.50	65522.8	0.11	197.3	1764	78304
Base	-	0.00	0.0	0.00	0.0	1764	78304
		$\sum w_x h_x^k$	585894.3	1.0	Total Building Weight, k =		16,688
k = 1.000						Base Shear,k =	1,764
T = 0.295						Total Moment,k-ft =	78,304

Exisiting Special Reinforced Shear Walls - Seismic Forces (N-S Dirction, Y)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	31.0	71.83	2345.0	0.00	6.1	6	438
High Roof	736.3	69.83	54136.2	0.09	140.8	147	10270
Low Roof	1208.6	62.83	79854.4	0.13	207.7	355	23318
Level 4	4373.9	50.50	231661.9	0.38	602.5	957	53743
Level 3	2046.4	38.50	82360.9	0.13	214.2	1171	61990
Level 2	3438.7	26.50	94826.8	0.15	246.6	1418	68525
Level 1	4853.5	13.50	67628.0	0.11	175.9	1594	70900
Base	-	0.00	0.0	0.00	0.0	1594	70900
		$\sum w_x h_x^k$	612813.1	1.0	Total Building Weight, k =		16,688
k = 1.012						Base Shear,k =	1,594
T = 0.524						Total Moment,k-ft =	70,900

% Difference of ETABS from Hand Calculations			
Level	**Hand Calculated Fx (k) = V*C_{vx}**	**ETABS Fx (k)**	**% Difference**
Stair 3	6.7	6.7	0.04%
High Roof	154.8	154.7	0.04%
Low Roof	228.6	228.5	0.04%
Level 4	665.0	664.7	0.04%
Level 3	237.2	237.1	0.04%
Level 2	274.4	274.2	0.04%
Level 1	197.3	197.2	0.04%
Base Shear	1,764.0	1,763.2	0.04%
< 10%, therefore can use ETABS Calculated Seismic Forces			

% Difference of ETABS from Hand Calculations			
Level	**Hand Calculated Fy (k) = V*C_{vx}**	**ETABS Fy (k)**	**% Difference**
Stair 3	6.1	6.1	-0.02%
High Roof	140.8	140.76	0.02%
Low Roof	207.7	207.63	0.02%
Level 4	602.5	602.34	0.02%
Level 3	214.2	214.15	0.02%
Level 2	246.6	246.56	0.02%
Level 1	175.9	175.84	0.02%
Base Shear	1593.7	1593.37	0.02%
< 10%, therefore can use ETABS Calculated Seismic Forces			

ASCE 7-05 Equivalent Lateral Force Method (Special Reinforced Concrete Shear Walls)

Seismic Load Design Criteria			ASCE 7-05
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
S_S	0.564	g	*§11.4.1, Fig. 22-1*
S_1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
F_a	1.349		*Table 11.4-1*
F_v	2.120		*Table 11.4-2*
S_{MS}	0.761	g	*EQ. 11.4-1*
S_{M1}	0.360	g	*EQ. 11.4-2*
S_{DS}	0.507	g	*EQ. 11.4-3*
S_{D1}	0.240	g	*EQ. 11.4-4*

Calculation of Seismic Response Coefficient Special Reinforced Concrete Shear Walls (Design #2)				
R (Special reinforced concrete shear walls)		**6.000**		***Table 12.2-1***
Tx		0.211	s	*ETABS*
Ty		0.4105	s	ETABS
$C_s = \frac{S_{DS}}{(R/I)}$		**0.106**		***EQ. 12.8-2***
$C_s = \frac{S_{D1}}{T(R/I)}, T \le T_L$	X	0.237		*EQ. 12.8-3*
	Y	0.122		
$C_s \geq 0.01$		**ok**		***EQ. 12.8-5***
$C_s =$	X	0.106		*ETABS*
	Y	0.106		*ETABS*
k	**X**	**1.00**		***§12.8.3***
k	**Y**	**0.96**		***§12.8.3***
Base shear, $V = C_s * W$		1792.3	kip	*EQ. 12.8-1*
Base shear, $V = C_s * W$		1792.3	kip	*EQ. 12.8-1*

Modified Special Reinforced Shear Walls - Seismic Forces (E-W Dirction, X)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = $V*C_{vx}$	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	32.0	71.83	2298.2	0.00	6.9	7	497
High Roof	749.6	69.83	52342.2	0.09	157.6	165	11502
Low Roof	1207.9	62.83	75895.3	0.13	228.5	393	25860
Level 4	4469.1	50.50	225692.0	0.38	679.6	1073	60178
Level 3	2071.6	38.50	79756.4	0.13	240.1	1313	69423
Level 2	3494.0	26.50	92591.4	0.16	278.8	1592	76811
Level 1	4939.4	13.50	66682.0	0.11	200.8	1792	79522
Base	-	0.00	0.0	0.00	0.0	1792	79522
		$\Sigma w_x h_x^k$	595257.5	1.0	Total Building Weight, k =		16,964
k = 1.000						Base Shear,k =	1,792
T = 0.211						Total Moment,k-ft =	79,522

Modified Special Reinforced Shear Walls - Seismic Forces (N-S Dirction, Y)

Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = $V*C_{vx}$	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	32.0	71.83	2298.2	0.00	6.9	7	497
High Roof	749.6	69.83	52342.2	0.09	157.6	165	11502
Low Roof	1207.9	62.83	75895.3	0.13	228.5	393	25860
Level 4	4469.1	50.50	225692.0	0.38	679.6	1073	60178
Level 3	2071.6	38.50	79756.4	0.13	240.1	1313	69423
Level 2	3494.0	26.50	92591.4	0.16	278.8	1592	76811
Level 1	4939.4	13.50	66682.0	0.11	200.8	1792	79522
Base	-	0.00	0.0	0.00	0.0	1792	79522
		$\Sigma w_x h_x^k$	595257.5	1.0	Total Building Weight, k =		16,964
k = 1.000						Base Shear,k =	1,792
T = 0.411						Total Moment,k-ft =	79,522

ASCE 7-05 Equivalent Lateral Force Method (Ordinary Steel Concentric Brace Frames)

Seismic Load Design Criteria	ASCE 7-05		
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
S_S	0.564	g	*§11.4.1, Fig. 22-1*
S_1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
F_a	1.349		*Table 11.4-1*
F_v	2.120		*Table 11.4-2*
S_{MS}	0.761	g	*EQ. 11.4-1*
S_{M1}	0.360	g	*EQ. 11.4-2*
S_{DS}	0.507	g	*EQ. 11.4-3*
S_{D1}	0.240	g	*EQ. 11.4-4*

Calculation of Seismic Response Coefficient Special Concentric Brace Frames				
$T_a = C_t h_n^x$		0.493	sec	*EQ. 12.8-7*
Tx		0.720	s	*ETABS*
Ty		0.720	s	ETABS
C_t		0.020		*Table 12.8-2*
x		0.750		*Table 12.8-2*
R (Special Steel Concentric Brace Frames)		6.000		*Table 12.2-1*
$C_s = \frac{S_{DS}}{(R/I)}$		0.106		*EQ. 12.8-2*
$C_s = \frac{S_{D1}}{T(R/I)}, T \le T_L$	X	0.069		*EQ. 12.8-3*
	Y	0.069		
$C_s \geq 0.01$		ok		*EQ. 12.8-5*
$C_s =$	X	0.069		*ETABS*
	Y	0.069		ETABS
k	X	1.11		*§12.8.3*
k	Y	1.11		*§12.8.3*
Base shear, $V = C_s W$		853.2	kip	*EQ. 12.8-1*
Base shear, $V = C_s W$		853.2	kip	*EQ. 12.8-1*

SCBF - Seismic Forces (E-W Dirction, X)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = $V*C_{vx}$	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14.2	71.83	1633.7	0.00	2.1	2	149
High Roof	511	69.83	57004.1	0.08	72.4	74	5206
Low Roof	668	62.83	66192.5	0.10	84.1	159	10489
Level 4	3,870	50.50	301078.9	0.45	382.5	541	29803
Level 3	1,473	38.50	84812.1	0.13	107.7	649	33951
Level 2	2,823	26.50	107362.4	0.16	136.4	785	37565
Level 1	2,979	13.50	53584.2	0.08	68.1	853	38484
Base	-	0.00	0.0	0.00	0.0	853	38484
		$\Sigma w_x h_x^k$	671667.9	1.0	Total Building Weight, k =		12,339
k = 1.110						Base Shear,k =	853
T = 0.720						Total Moment,k-ft =	38,484

SCBF - Seismic Forces (N-S Dirction, Y)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = $V*C_{vx}$	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14.2	71.83	1633.7	0.00	2.1	2	149
High Roof	511	69.83	57004.1	0.08	72.4	74	5206
Low Roof	668	62.83	66192.5	0.10	84.1	159	10489
Level 4	3,870	50.50	301078.9	0.45	382.5	541	29803
Level 3	1,473	38.50	84812.1	0.13	107.7	649	33951
Level 2	2,823	26.50	107362.4	0.16	136.4	785	37565
Level 1	2,979	13.50	53584.2	0.08	68.1	853	38484
Base	-	0.00	0.0	0.00	0.0	853	38484
		$\Sigma w_x h_x^k$	671667.9	1.0	Total Building Weight, k =		12,339
k = 1.110						Base Shear,k =	853
T = 0.720						Total Moment,k-ft =	38,484

ASCE 7-05 Equivalent Lateral Force Method (Special Steel Moment Frames)

Seismic Load Design Criteria	ASCE 7-05		
Building Height (h), ft	71.830		
Occupancy Category	III		*Table 1-1*
S_S	0.564	g	*§11.4.1, Fig. 22-1*
S_1	0.170	g	*§11.4.1, Fig.22-2*
Importance Factor	1.250		*Table 11.5-1*
Soil Site Class	D		*§11.4.2*
Seismic Design Category	D		*Table 11.6-1*
F_a	1.349		*Table 11.4-1*
F_v	2.120		*Table 11.4-2*
S_{MS}	0.761	g	*EQ. 11.4-1*
S_{M1}	0.360	g	*EQ. 11.4-2*
S_{DS}	0.507	g	*EQ. 11.4-3*
S_{D1}	0.240	g	*EQ. 11.4-4*

Calculation of Seismic Response Coefficient Special Steel Moment Frames				
$T_a = C_t h_n^x$		**0.855**	**sec**	*EQ. 12.8-7*
Tx		0.789	s	*ETABS*
Ty		1.0142	s	ETABS
C_t		**0.028**		***Table 12.8-2***
x		0.800		*Table 12.8-2*
R (Special Steel Moment Frames)		**8.000**		***Table 12.2-1***
$C_s = \frac{S_{DS}}{(R/I)}$		0.079		*EQ. 12.8-2*
$C_s = \frac{S_{D1}}{T(R/I)}, T \leq T_L$	**X**	**0.048**		*EQ. 12.8-3*
	Y	**0.037**		
$C_s \geq 0.01$		ok		*EQ. 12.8-5*
$C_s =$	**X**	**0.048**		***ETABS***
	Y	**0.037**		***ETABS***
k	X	1.14		*§12.8.3*
k	Y	1.26		*§12.8.3*
Base shear, Vx = C_s*W		**584.6**	**kip**	*EQ. 12.8-1*
Base shear, Vy = C_s*W		**454.5**	**kip**	*EQ. 12.8-1*

SMF - Seismic Forces (E-W Dirction, X)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14	71.83	1909.1	0.00	1.5	1	105
High Roof	517	69.83	66638.0	0.09	51.1	53	3671
Low Roof	633	62.83	72253.2	0.09	55.4	108	7150
Level 4	3854	50.50	342792.7	0.45	262.7	371	20416
Level 3	1479	38.50	96415.2	0.13	73.9	444	23260
Level 2	3025	26.50	128629.9	0.17	98.6	543	25873
Level 1	2757	13.50	54189.2	0.07	41.5	585	26433
Base	-	0.00	0.0	0.00	0.0	585	26433
		$\Sigma w_x h_x^k$	762827.4	1.0	Total Building Weight, k =		12,280
k = 1.144						Base Shear,k =	585
T = 0.789						Total Moment,k-ft =	26,433

SMF - Seismic Forces (N-S Dirction, Y)							
Level	Story Weight, wx (k)	Story Height, hx (ft)	$w_x h_x^k$	C_{vx}	Fx (k) = V*C_{vx}	Story Shear (k) = Vx=Σfi	Overturning Moment (k-ft)
Stair 3	14	71.83	3091.9	0.00	1.2	1	87
High Roof	517	69.83	107579.6	0.09	42.0	43	3021
Low Roof	633	62.83	115263.2	0.10	45.0	88	5849
Level 4	3854	50.50	533534.9	0.46	208.4	297	16372
Level 3	1479	38.50	145541.0	0.13	56.8	353	18560
Level 2	3025	26.50	186159.0	0.16	72.7	426	20487
Level 1	2757	13.50	72679.9	0.06	28.4	455	20870
Base	-	0.00	0.0	0.00	0.0	455	20870
		$\Sigma w_x h_x^k$	1163849.5	1.0	Total Building Weight, k =		12,280
k = 1.257						Base Shear,k =	455
T = 1.014						Total Moment,k-ft =	20,870

ASCE 7-05 Chapter 12: Horizontal Irregularities

Special Reinf. Shear WallAmplification Factor, Ao in the E-W Direction								
Story	δx	δxpe	δavg	δmax	Ax	Ax (used)	% torsion Δ	Torsion Irreg.
Stair 3	0.17	0.14	0.17	0.31	2.4	2.4	1.9	Irregular, 1a
HGH ROOF	0.23	0.22	0.23	0.45	2.7	2.7	2.0	Irregular, 1a
LOW ROOF	0.22	0.21	0.22	0.42	2.6	2.6	1.9	Irregular, 1a
STORY4	0.17	0.16	0.17	0.34	2.6	2.6	2.0	Irregular, 1a
STORY3	0.12	0.11	0.12	0.23	2.6	2.6	2.0	Irregular, 1a
STORY2	0.07	0.07	0.07	0.13	2.6	2.6	1.9	Irregular, 1a
STORY1	0.01	0.01	0.01	0.02	2.7	2.7	2.0	Irregular, 1a

Special Reinf. Shear Wall,Amplification Factor, Ao in the N-S Direction								
Story	δy	δype	δavg	δmax	Ax	Ax (used)	% torsion Δ	Torsion Irreg.
Stair 3	0.63	0.69	0.63	1.31	3.1	3.1	2.1	Irregular, 1a
HGH ROOF	0.72	0.80	0.72	1.52	3.1	3.1	2.1	Irregular, 1a
LOW ROOF	0.13	0.16	0.13	0.29	3.2	3.2	2.2	Irregular, 1a
STORY4	0.39	0.43	0.39	0.82	3.1	3.1	2.1	Irregular, 1a
STORY3	0.25	0.28	0.25	0.53	3.1	3.1	2.1	Irregular, 1a
STORY2	0.04	0.05	0.04	0.09	3.3	3.3	2.2	Irregular, 1a
STORY1	0.02	0.02	0.02	0.03	3.0	3.0	2.1	Irregular, 1a

Modified Special Reinf. Shear Wall, Amplification Factor, Ao in the E-W Direction								
Story	δx	δxpe	δavg	δmax	Ax	Ax (used)	% torsion Δ	Torsion Irreg.
Stair 3	0.17	0.17	0.17	0.34	2.8	2.8	2.0	Irregular, 1a
HGH ROOF	0.26	0.24	0.26	0.50	2.6	2.6	1.9	Irregular, 1a
LOW ROOF	0.24	0.22	0.24	0.46	2.6	2.6	1.9	Irregular, 1a
STORY4	0.17	0.16	0.17	0.34	2.6	2.6	1.9	Irregular, 1a
STORY3	0.11	0.10	0.11	0.21	2.6	2.6	1.9	Irregular, 1a
STORY2	0.06	0.05	0.06	0.11	2.6	2.6	1.9	Irregular, 1a
STORY1	0.00	0.00	0.00	0.01	2.7	2.7	2.0	Irregular, 1a

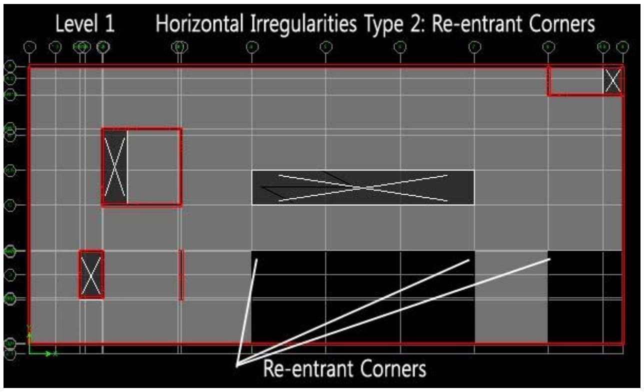
Modified Special Reinf. Shear Wall, Amplification Factor, Ao in the N-S Direction								
Story	δy	δype	δavg	δmax	Ax	Ax (used)	% torsion Δ	Torsion Irreg.
Stair 3	0.56	0.61	0.58	0.61	0.8	1.0	1.0	Good
HGH ROOF	0.63	0.69	0.66	0.69	0.8	1.0	1.0	Good
LOW ROOF	0.47	0.52	0.49	0.52	0.8	1.0	1.1	Good
STORY4	0.34	0.37	0.36	0.37	0.8	1.0	1.0	Good
STORY3	0.21	0.24	0.22	0.24	0.8	1.0	1.0	Good
STORY2	0.10	0.11	0.11	0.11	0.8	1.0	1.0	Good
STORY1	0.01	0.01	0.01	0.01	0.7	1.0	1.0	Good

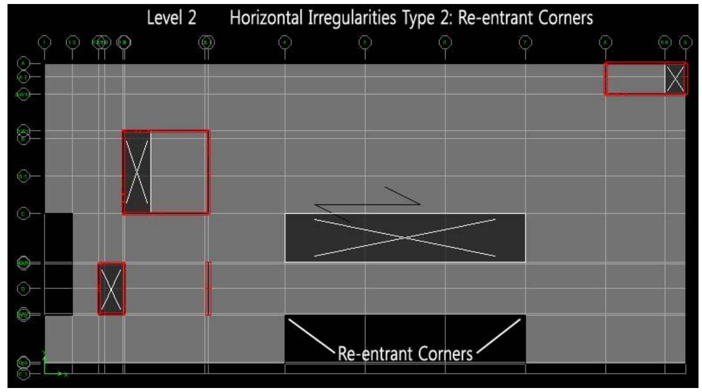
SCBF, Amplification Factor, Ao in the E-W Direction							
Story	δx	δxpe	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.14	0.14	0.14	0.27	2.8	2.0	Irregular, 1a
HGH ROOF	0.20	0.19	0.20	0.39	2.6	1.9	Irregular, 1a
LOW ROOF	0.19	0.18	0.19	0.37	2.6	1.9	Irregular, 1a
STORY4	0.16	0.15	0.16	0.31	2.6	1.9	Irregular, 1a
STORY3	0.11	0.10	0.11	0.21	2.6	1.9	Irregular, 1a
STORY2	0.06	0.06	0.06	0.12	2.6	1.9	Irregular, 1a
STORY1	0.00	0.00	0.00	0.01	2.7	2.0	Irregular, 1a

SCBF, Amplification Factor, Ao in the N-S Direction							
Story	δy	δype	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.48	0.54	0.48	1.03	3.1	2.1	Irregular, 1a
HGH ROOF	0.82	0.92	0.82	1.74	3.1	2.1	Irregular, 1a
LOW ROOF	0.43	0.48	0.43	0.91	3.1	2.1	Irregular, 1a
STORY4	0.32	0.36	0.32	0.69	3.1	2.1	Irregular, 1a
STORY3	0.21	0.24	0.21	0.46	3.1	2.1	Irregular, 1a
STORY2	0.11	0.12	0.11	0.23	3.1	2.1	Irregular, 1a
STORY1	0.00	0.00	0.00	0.00	2.9	2.0	Irregular, 1a

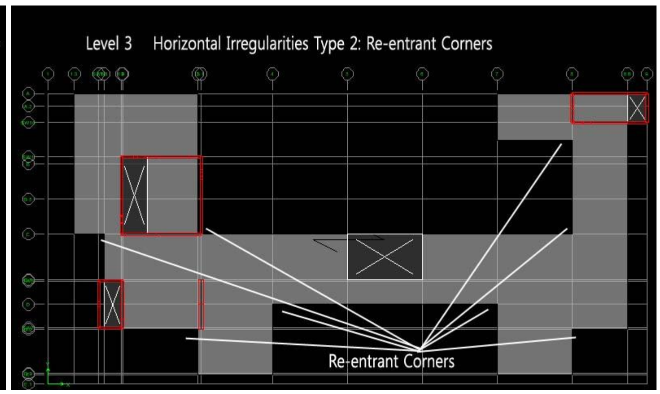
SMF, Amplification Factor, Ao in the E-W Direction							
Story	δx	δxpe	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.28	0.29	0.28	0.57	2.8	2.0	Irregular, 1a
HGH ROOF	0.43	0.41	0.43	0.83	2.6	1.9	Irregular, 1a
LOW ROOF	0.50	0.48	0.50	0.98	2.7	2.0	Irregular, 1a
STORY4	0.34	0.33	0.34	0.67	2.6	1.9	Irregular, 1a
STORY3	0.25	0.23	0.25	0.48	2.6	1.9	Irregular, 1a
STORY2	0.14	0.14	0.14	0.28	2.6	1.9	Irregular, 1a
STORY1	0.00	0.00	0.00	0.01	2.7	2.0	Irregular, 1a

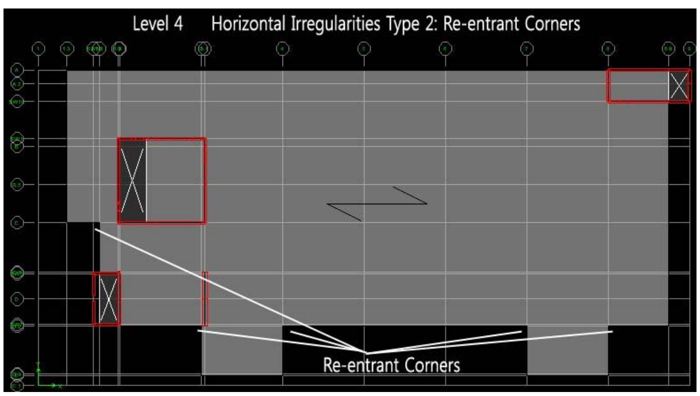
SMF, Amplification Factor, Ao in the N-S Direction							
Story	δy	δype	δavg	δmax	Ax	% torsion Δ	Torsion Irreg.
Stair 3	0.87	0.99	0.87	1.86	3.2	2.1	Irregular, 1a
HGH ROOF	1.41	1.59	1.41	3.01	3.1	2.1	Irregular, 1a
LOW ROOF	0.80	0.90	0.80	1.70	3.2	2.1	Irregular, 1a
STORY4	0.65	0.73	0.65	1.38	3.1	2.1	Irregular, 1a
STORY3	0.48	0.54	0.48	1.02	3.2	2.1	Irregular, 1a
STORY2	0.31	0.35	0.31	0.66	3.2	2.1	Irregular, 1a
STORY1	0.00	0.01	0.00	0.01	2.9	2.1	Irregular, 1a

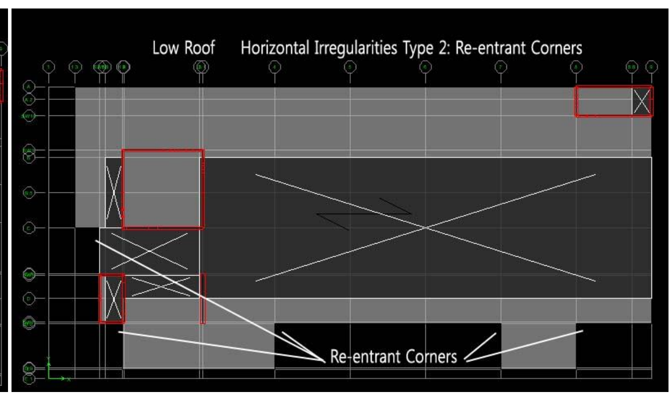

Level 1 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners

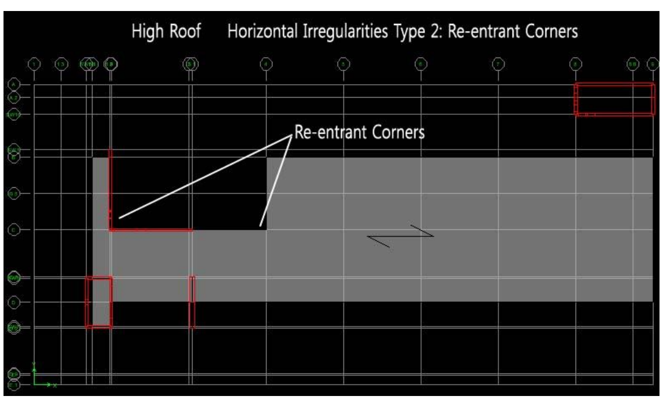

Level 2 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners

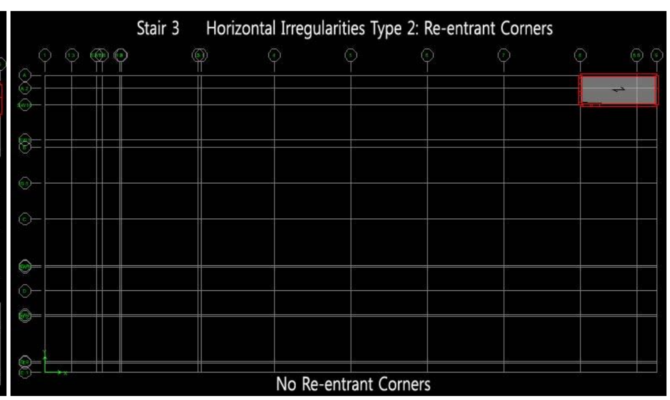

Level 2 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Level 3 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Level 4 Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Low Roof Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


High Roof Horizontal Irregularities Type 2: Re-entrant Corners
Re-entrant Corners


Stair 3 Horizontal Irregularities Type 2: Re-entrant Corners
No Re-entrant Corners

HORIZONTAL STRUCTURAL IRREGULARITIES Type 3: Diaphragm Discontinuity Irregularity				
Story	Total Area (SF)	Area w/o Openings (SF)	% Open	ASCE 7-05 TABLE 12.3-1
Stair 3	380	380	0%	Ok
High Roof	16410.377	12,071	26%	Ok
Low Roof	29360.4	13,748	53%	Not Ok
Level 4	29360.4	24,275	17%	Ok
Level 3	29360.4	13,392	54%	Not Ok
Level 2	30720	25,867	16%	Ok
Level 1	30720	23,434	24%	Ok

ASCE 7-05 Chapter 12: Vertical Irregularities

Special Reinforced Shear Wall - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	1.9E-03	1.3E-03	1.5E-03	--	ok
High Roof	3.8E-04	2.7E-04	3.1E-04	--	ok
Low Roof	2.1E-04	1.5E-04	1.7E-04	--	ok
Level 4	2.8E-04	2.0E-04	2.3E-04	6.6E-04	not ok
Level 3	3.3E-04	2.3E-04	2.6E-04	2.3E-04	ok
Level 2	3.0E-04	2.1E-04	2.4E-04	2.2E-04	ok
Level 1	5.0E-05	3.5E-05	4.0E-05	2.4E-04	not ok

Special Reinforced Shear Wall - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	6.1E-03	4.3E-03	4.9E-03	--	ok
High Roof	1.8E-03	1.3E-03	1.5E-03	--	ok
Low Roof	3.7E-04	2.6E-04	2.9E-04	--	ok
Level 4	5.9E-04	4.1E-04	4.7E-04	2.2E-03	not ok
Level 3	5.1E-04	3.6E-04	4.1E-04	7.5E-04	not ok
Level 2	4.2E-04	3.0E-04	3.4E-04	3.9E-04	ok
Level 1	8.1E-05	5.7E-05	6.5E-05	4.1E-04	not ok

Modified Special Reinforced Shear Wall - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	2.7E-03	1.9E-03	2.2E-03	--	ok
High Roof	5.5E-04	3.9E-04	4.4E-04	--	ok
Low Roof	2.9E-04	2.0E-04	2.3E-04	--	ok
Level 4	3.1E-04	2.2E-04	2.5E-04	2.8E-03	not ok
Level 3	3.4E-04	2.4E-04	2.7E-04	9.2E-04	not ok
Level 2	2.5E-04	1.7E-04	2.0E-04	7.5E-04	not ok
Level 1	2.5E-05	1.7E-05	2.0E-05	7.2E-04	not ok

Modified Special Reinforced Shear Wall - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	1.8E-03	1.2E-03	1.4E-03	--	ok
High Roof	1.5E-03	1.0E-03	1.2E-03	--	ok
Low Roof	6.4E-04	4.5E-04	5.1E-04	--	ok
Level 4	7.5E-04	5.3E-04	6.0E-04	1.3E-03	not ok
Level 3	6.5E-04	4.6E-04	5.2E-04	9.5E-04	not ok
Level 2	4.7E-04	3.3E-04	3.8E-04	6.8E-04	not ok
Level 1	6.0E-05	4.2E-05	4.8E-05	6.3E-04	not ok

SCBF - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next	Soft Story Status
Stair 3	4.4E-03	3.1E-03	3.5E-03	--	ok
High Roof	6.6E-04	4.7E-04	5.3E-04	--	ok
Low Roof	2.7E-04	1.9E-04	2.2E-04	--	ok
Level 4	6.1E-04	4.3E-04	4.9E-04	4.3E-03	not ok
Level 3	5.5E-04	3.9E-04	4.4E-04	1.2E-03	not ok
Level 2	7.1E-04	5.0E-04	5.7E-04	1.2E-03	not ok
Level 1	2.0E-05	1.4E-05	1.6E-05	1.5E-03	not ok

SCBF - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	2.4E-04	1.7E-04	1.9E-04	--	ok
High Roof	4.8E-03	3.4E-03	3.9E-03	--	ok
Low Roof	2.5E-04	1.7E-04	2.0E-04	--	ok
Level 4	7.8E-04	5.5E-04	6.3E-04	1.8E-03	not ok
Level 3	7.6E-04	5.3E-04	6.1E-04	2.0E-03	not ok
Level 2	1.0E-03	7.2E-04	8.2E-04	6.0E-04	ok
Level 1	2.8E-05	1.9E-05	2.2E-05	8.6E-04	not ok

SMF - Vertical Irregularity 1a X Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	7.1E-05	5.0E-05	5.7E-05	--	ok
High Roof	3.0E-05	2.1E-05	2.4E-05	--	ok
Low Roof	3.4E-05	2.4E-05	2.7E-05	--	ok
Level 4	5.6E-05	3.9E-05	4.5E-05	1.1E-04	not ok
Level 3	7.8E-05	5.5E-05	6.3E-05	9.6E-05	not ok
Level 2	7.2E-05	5.0E-05	5.7E-05	1.3E-04	not ok
Level 1	1.2E-06	8.6E-07	9.9E-07	1.7E-04	not ok

SMF - Vertical Irregularity 1a Y Direction					
Story	ETABS Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Stair 3	3.5E-03	2.5E-03	2.8E-03	--	ok
High Roof	5.6E-05	3.9E-05	4.5E-05	--	ok
Low Roof	7.8E-05	5.4E-05	6.2E-05	--	ok
Level 4	2.7E-04	1.9E-04	2.2E-04	1.2E-03	not ok
Level 3	1.9E-04	1.3E-04	1.5E-04	1.3E-04	ok
Level 2	4.0E-04	2.8E-04	3.2E-04	1.8E-04	ok
Level 1	8.9E-05	6.2E-05	7.1E-05	2.8E-04	not ok

Appendix E – Lateral Force Resisting System Design Checks-Existing System

Level 1 Shear Wall Data*

Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ (in)**	I =Ri/∑Ri
1	12	34.00	90	13.50	1.00	13.50	34.00	459.00	31043.54	3.33E-05	2.97%
2	12	34.00	90	13.50	1.00	13.50	34.00	459.00	31043.54	3.33E-05	2.97%
3	12	33.00	0	13.50	33.00	445.50	1.00	13.50	29801.04	3.47E-05	2.85%
4	12	33.00	0	13.50	33.00	445.50	1.00	13.50	29801.04	3.47E-05	2.85%
5	12	23.17	0	13.50	23.17	312.80	1.00	13.50	17523.79	5.87E-05	1.67%
6	12	23.17	0	13.50	23.17	312.80	1.00	13.50	17523.79	5.87E-05	1.67%
7	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
8	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
9	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.73%
10	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.73%
11	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.74%
12	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.74%
13	24	21.17	0	13.50	21.17	285.80	2.00	27.00	30112.66	3.41E-05	2.88%
14	12	120.00	0	13.50	120.00	1620.00	1.00	13.50	131256.84	7.94E-06	12.54%
15	12	107.67	0	13.50	107.67	1453.55	1.00	13.50	117279.46	8.88E-06	11.20%
16	12	256.00	90	13.50	1.00	13.50	256.00	3456.00	283841.76	3.67E-06	27.12%
17	12	224.00	90	13.50	1.00	13.50	224.00	3024.00	248069.26	4.20E-06	23.70%
* Assume that the general area of wall is rectangular yet has openings								∑Ri =	1046672.10		100.00%
** Using a 1k load applied at the top of each LFRS system											

Center of Rigidity

X Direction	k_{ix} (k/ft)	x_i (ft)	$k_{ix}\ x_i$
SW3	357612.53	31.50	11264794.82
SW4	357612.53	65.50	23423620.98
SW5	210285.51	21.67	4556186.11
SW6	210285.51	32.00	6729136.41
SW11	40905.82	224.00	9162903.93
SW12	40905.82	256.00	10471890.20
SW13	342672.27	65.50	22445033.57
SW14	131256.84	0.00	0.00
SW15	117279.46	256.00	30023541.88
	∑ 1808816.31		118077107.89
		x (ft) = $\sum k_{ix}\ x_i / k_{ix}$ =	**65.28**
Y Direction	**k_{iy} (k/ft)**	**y_i (ft)**	**$k_{iy}\ y_i$**
SW1	372522.46	97.00	36134678.63
SW2	372522.46	64.00	23841437.45
SW7	357612.53	24.00	8582700.82
SW8	357612.53	44.00	15734951.50
SW9	210285.51	124.00	26075403.57
SW10	210285.51	111.67	23482583.20
SW16	283841.76	4.00	1135367.03
SW17	248069.26	124.00	30760587.93
	∑ 2412752.03		165747710.13
		y (ft) = $\sum k_{iy}\ y_i / k_{iy}$ =	**68.70**

Existing Special Reinforced Concrete Shear Wall Design Check

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY		**Wall 13**			**X-Direction**					
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi	Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	3718	k	at BASE
REBAR YIELD STRESS	f_y	=	60	ksi	FACTORED BASE MOMENT LOAD	M_u	=	46338	ft-k	
HEIGHT OF WALL	H	=	376.0	in	FACTORED BASE SHEAR LOAD	V_u	=	330	k	
LENGTH OF SHEAR WALL	L	=	254.0	in						
THICKNESS OF WALL	t	=	24	in	**THE WALL DESIGN IS ADEQUATE.**					
	Acv	=	6096.96	in^2						
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			771.2	Need 1						

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	330.4	kip
Vn = Vc + Vs	d	=	203.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3084.8	kip
	ΦVn	=	2313.6	kip
ACI 318-08 § 21.9.4	Vn	≤	53429.2	kip
$V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	617.0	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	385.6	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	231.4	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	-176.4	kip	
S = (Av*fy*d)/Vs	Ag	=	6096.96	in^2	
	0.0025*Ag	=	15.2	in^2	
TRY #8	Abar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 84.68	S	=	8.00	in	**USE**
S ≤ 3t = 72					
S ≤ 18" **Governs**	# bars required	=	19		
ρt = Av/(S*t)	ρt	=	0.0794	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.1186	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 84.68	S	=	6	in	
S ≤ 3t = 72					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	11		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.4799	ρl≥ρt	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	11	#11	@	6	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	182.91	in	
Mu = Φ Mn = Φ As*fy*j*d	As	=	56.30	in^2	
	a	=	41.40	in	
jd = d - (a/2)	jd	=	182.53	in	
	As	=	56.41	in^2	

TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	36		
Check Capacity:	a	=	41.48	in	
C=T 0.85*f'c*a*b =As*fy	c	=	48.80	in	
c = a/0.85	εt	=	0.01	>0.0025	**OK**
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)			Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0794	>	$(\rho t)_{min.}$ =	0.0025	**OK**	
$\rho l_{provd.}$ = 0.1186	>	$(\rho l)_{min.}$ =	0.0025	**OK**	

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	36	#11	@	6	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 40072 kips > V_u = 330 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 556,057 kip-ft > M_u = 46,338 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

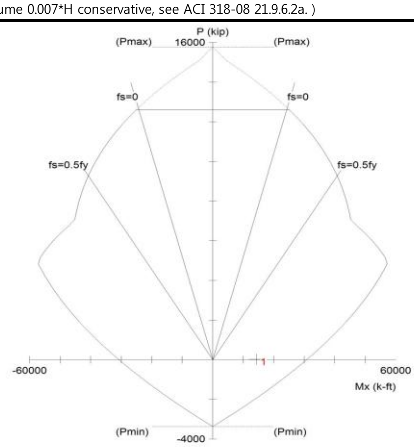
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 60.49 in. **No Boundary Element Needed**

where c = 49 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Appendix F – Lateral Force Resisting System Design Checks-System #1

Modified Special Reinforced Shear Wall Shear Wall Data* - Level 1											
Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ(in)**	I =Ri/∑Ri
1	18	34.00	90	13.50	1.50	20.25	34.00	459.00	46565.31	2.22E-05	4.50%
2	18	34.00	90	13.50	1.50	20.25	34.00	459.00	46565.31	2.22E-05	4.50%
5	18	23.17	0	13.50	23.17	312.80	1.50	20.25	26285.69	3.91E-05	2.54%
6	18	23.17	0	13.50	23.17	312.80	1.50	20.25	26285.69	3.91E-05	2.54%
7	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
8	12	10.33	90	13.50	1.00	13.50	10.33	139.46	3408.82	2.97E-04	0.33%
9	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.76%
10	12	32.00	90	13.50	1.00	13.50	32.00	432.00	28556.02	3.62E-05	2.76%
11	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.75%
12	18	12.33	0	13.50	12.33	166.46	1.50	20.25	7722.84	1.32E-04	0.75%
13	24	21.17	0	13.50	21.17	285.80	2.00	27.00	30112.66	3.41E-05	2.91%
14	12	120.00	0	13.50	120.00	1620.00	1.00	13.50	131256.84	7.94E-06	12.67%
15	12	107.67	0	13.50	107.67	1453.55	1.00	13.50	117279.46	8.88E-06	11.32%
16	12	256.00	90	13.50	1.00	13.50	256.00	3456.00	283841.76	3.67E-06	27.41%
17	12	224.00	90	13.50	1.00	13.50	224.00	3024.00	248069.26	4.20E-06	23.95%
* Assume that the general area of wall is rectangular yet has openings								∑Ri =	1035637.35		100.00%
** Using a 1k load applied at the top of each LFRS system											

Modified Special Reinforced Shear Wall Shear Wall Data* - Level 2											
Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ(in)**	I =Ri/∑Ri
1	18	34.00	90	13.00	1.50	19.50	34.00	442.00	48996.34	2.11E-05	18.12%
2	18	34.00	90	13.00	1.50	19.50	34.00	442.00	48996.34	2.11E-05	18.12%
5	18	23.17	0	13.00	23.17	301.21	1.50	19.50	27947.89	3.68E-05	10.33%
6	18	23.17	0	13.00	23.17	301.21	1.50	19.50	27947.89	3.68E-05	10.33%
7	12	10.33	90	13.00	1.00	13.00	10.33	134.29	3730.75	2.72E-04	1.38%
8	12	10.33	90	13.00	1.00	13.00	10.33	134.29	3730.75	2.72E-04	1.38%
9	12	32.00	90	13.00	1.00	13.00	32.00	416.00	30088.01	3.44E-05	11.13%
10	12	32.00	90	13.00	1.00	13.00	32.00	416.00	30088.01	3.44E-05	11.13%
11	18	12.33	0	13.00	12.33	160.29	1.50	19.50	8401.54	1.21E-04	3.11%
12	18	12.33	0	13.00	12.33	160.29	1.50	19.50	8401.54	1.21E-04	3.11%
13	24	21.17	0	13.00	21.17	275.21	2.00	26.00	32113.73	3.20E-05	11.87%
* Assume that the general area of wall in rectangular yet has openings								∑Ri =	270442.77		100.00%
** Using a 1k load applied at the top of each LFRS system											

Modified Special Reinforced Shear Wall Shear Wall Data* - Level 3											
Shear Wall Number	thickness (in)	b, Length (ft)	Angle with NS-axis (Y) (deg)	h, Height (ft)	Length in NS-Dir (ft)	Area in NS- Dir. (ft2)	Length in EW-Dir (ft)	Area in EW-dir (ft2)	k cantilever (k/in)	Δ(in)**	I =Ri/∑Ri
1	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
2	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
5	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
6	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
7	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
8	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
9	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
10	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
11	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
12	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
13	24	21.17	0	12.00	21.17	254.04	2.00	24.00	36654.01	2.81E-05	12.02%
* Assume that the general area of wall in rectangular yet has openings								∑Ri =	304977.93		100.00%
** Using a 1k load applied at the top of each LFRS system											

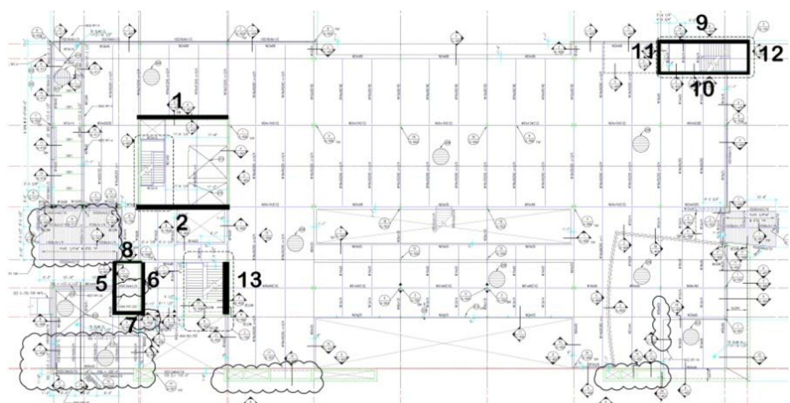
Modified Special Reinforced Shear Wall Shear Wall Data* - Level 4											
Shear Wall Number	**thickness (in)**	**b, Length (ft)**	**Angle with NS-axis (Y) (deg)**	**h, Height (ft)**	**Length in NS-Dir (ft)**	**Area in NS- Dir. (ft2)**	**Length in EW-Dir (ft)**	**Area in EW-dir (ft2)**	**k cantilever (k/in)**	**∆(in)****	**I =Ri/∑Ri**
1	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
2	18	34.00	90	12.00	1.50	18.00	34.00	408.00	54438.94	1.90E-05	17.85%
5	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
6	18	23.17	0	12.00	23.17	278.04	1.50	18.00	31703.97	3.25E-05	10.40%
7	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
8	12	10.33	90	12.00	1.00	12.00	10.33	123.96	4499.25	2.26E-04	1.48%
9	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
10	12	32.00	90	12.00	1.00	12.00	32.00	384.00	33520.27	3.09E-05	10.99%
11	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
12	18	12.33	0	12.00	12.33	147.96	1.50	18.00	9999.54	1.02E-04	3.28%
13	24	21.17	0	12.00	21.17	254.04	2.00	24.00	36654.01	2.81E-05	12.02%
* Assume that the general area of wall in rectangular yet has openings								**∑Ri =**	**304977.93**		**100.00%**
** Using a 1k load applied at the top of each LFRS system											

Modified Special Reinforced Shear Wall Shear Wall Data* - Low Roof											
Shear Wall Number	**thickness (in)**	**b, Length (ft)**	**Angle with NS-axis (Y) (deg)**	**h, Height (ft)**	**Length in NS-Dir (ft)**	**Area in NS- Dir. (ft2)**	**Length in EW-Dir (ft)**	**Area in EW-dir (ft2)**	**k cantilever (k/in)**	**∆(in)****	**I =Ri/∑Ri**
1	18	34.00	90	12.33	1.50	18.50	34.00	419.32	52532.93	1.97E-05	17.94%
2	18	34.00	90	12.33	1.50	18.50	34.00	419.32	52532.93	1.97E-05	17.94%
5	18	23.17	0	12.33	23.17	285.76	1.50	18.50	30383.76	3.39E-05	10.38%
6	18	23.17	0	12.33	23.17	285.76	1.50	18.50	30383.76	3.39E-05	10.38%
7	12	10.33	90	12.33	1.00	12.33	10.33	127.40	4222.80	2.40E-04	1.44%
8	12	10.33	90	12.33	1.00	12.33	10.33	127.40	4222.80	2.40E-04	1.44%
9	12	32.00	90	12.33	1.00	12.33	32.00	394.66	32317.96	3.20E-05	11.04%
10	12	32.00	90	12.33	1.00	12.33	32.00	394.66	32317.96	3.20E-05	11.04%
11	18	12.33	0	12.33	12.33	152.07	1.50	18.50	9428.09	1.08E-04	3.22%
12	18	12.33	0	12.33	12.33	152.07	1.50	18.50	9428.09	1.08E-04	3.22%
13	24	21.17	0	12.33	21.17	261.09	2.00	24.67	35055.51	2.94E-05	11.97%
* Assume that the general area of wall in rectangular yet has openings								**∑Ri =**	**292826.61**		**100.00%**
** Using a 1k load applied at the top of each LFRS system											

Modified Special Reinforced Shear Wall Shear Wall Data* - High Roof											
Shear Wall Number	**thickness (in)**	**b, Length (ft)**	**Angle with NS-axis (Y) (deg)**	**h, Height (ft)**	**Length in NS-Dir (ft)**	**Area in NS- Dir. (ft2)**	**Length in EW-Dir (ft)**	**Area in EW-dir (ft2)**	**k cantilever (k/in)**	**∆(in)****	**I =Ri/∑Ri**
2	18	34.00	90	7.00	1.50	10.50	34.00	238.00	103386.85	1.01E-05	19.68%
5	18	23.17	0	7.00	23.17	162.19	1.50	10.50	66208.30	1.57E-05	12.60%
6	18	23.17	0	7.00	23.17	162.19	1.50	10.50	66208.30	1.57E-05	12.60%
7	12	10.33	90	7.00	1.00	7.00	10.33	72.31	13537.42	7.58E-05	2.58%
8	12	10.33	90	7.00	1.00	7.00	10.33	72.31	13537.42	7.58E-05	2.58%
9	12	32.00	90	7.00	1.00	7.00	32.00	224.00	64408.42	1.61E-05	12.26%
10	12	32.00	90	7.00	1.00	7.00	32.00	224.00	64408.42	1.61E-05	12.26%
11	18	12.33	0	7.00	12.33	86.31	1.50	10.50	27421.50	3.75E-05	5.22%
12	18	12.33	0	7.00	12.33	86.31	1.50	10.50	27421.50	3.75E-05	5.22%
13	24	21.17	0	7.00	21.17	148.19	2.00	14.00	78900.71	1.31E-05	15.02%
* Assume that the general area of wall is rectangular								**∑Ri =**	**525438.84**		**100.00%**
** Using a 1k load applied at the top of each LFRS system											

Modified Special Reinforced Shear Wall Shear Wall Data* - Stair 3											
Shear Wall Number	**thickness (in)**	**b, Length (ft)**	**Angle with NS-axis (Y) (deg)**	**h, Height (ft)**	**Length in NS-Dir (ft)**	**Area in NS- Dir. (ft2)**	**Length in EW-Dir (ft)**	**Area in EW-dir (ft2)**	**k cantilever (k/in)**	**∆(in)****	**I =Ri/∑Ri**
9	12	32.00	90	2.00	1.00	2.00	32.00	64.00	239119.06	4.36E-06	32.04%
10	12	32.00	90	2.00	1.00	2.00	32.00	64.00	239119.06	4.36E-06	32.04%
11	18	12.33	0	2.00	12.33	24.66	1.50	3.00	134053.03	7.77E-06	17.96%
12	18	12.33	0	2.00	12.33	24.66	1.50	3.00	134053.03	7.77E-06	17.96%
* Assume that the general area of wall in rectangular yet has openings								**∑Ri =**	**746344.18**		**100.00%**
** Using a 1k load applied at the top of each LFRS system											

WALL	Height (ft)	Length (ft)
Wall 1_a	24.33	34.00
Wall 1_b	38.50	34.00
Wall 2_a	31.33	34.00
Wall 2_b	38.50	34.00
Wall 3_a	31.33	34.00
Wall 3_b	38.50	34.00
Wall 4_a	24.33	34.00
Wall 4_b	38.50	34.00
Wall 5_a	31.33	20.00
Wall 5_b	38.50	20.00
Wall 6_a	31.33	20.00
Wall 6_b	38.50	20.00
Wall 7_c	31.33	10.33
Wall 7_d	38.50	10.33
Wall 8_c	31.33	10.33
Wall 8_d	38.50	10.33
Wall 9_c	33.33	34.00
Wall 9_d	38.50	34.00
Wall 10_c	33.33	34.00
Wall 10_d	38.50	34.00
Wall 11_e	33.33	12.33
Wall 11_f	38.50	12.33
Wall 12_e	33.33	12.33
Wall 12_f	38.50	12.33
Wall 13_g	31.33	21.17
Wall 13_h	38.50	21.17



Type	Thickness (in)	Vertical Spacing (in)	Vertical Bar Size	Vertical Bar Diameter (in)	Vertical Bar Weight (plf)	Horizontal Spacing (in)	Horizontal Bar Size	Horizontal Bar Diameter	Horizontal Bar Weight (plf)
a	12	12	6	0.75	1.502	12	6	0.75	1.502
b	12	6	11	1.41	5.313	8	8	1	2.67
c	12	12	6	0.75	1.502	12	6	0.75	1.502
d	12	8	11	1.41	5.313	8	8	1	2.67
e	18	12	6	0.75	1.502	12	6	0.75	1.502
f	18	6	11	1.41	5.313	8	8	1	2.67
g	18	6	11	1.41	5.313	8	8	1	2.67
h	18	6	11	1.41	5.313	8	8	1	2.67

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 1 a** — **X-Direction**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi	Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	2201	k	at BASE
REBAR YIELD STRESS	f_y	=	60	ksi	FACTORED BASE MOMENT LOAD	M_u	=	968	ft-k	
HEIGHT OF WALL	H	=	292.0	in	FACTORED BASE SHEAR LOAD	V_u	=	205	k	
LENGTH OF SHEAR WALL	L	=	408.0	in						
THICKNESS OF WALL	t	=	18	in	**THE WALL DESIGN IS ADEQUATE.**					
	Acv	=	7344	in^2						
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 1						

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	204.7	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	124863.3	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			**OK**	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu		1/2ΦVc	=	278.7	kip
				Reinf. According to Ch 14	
Vs = Vu/(0.75) - Vc		Vs	=	-	kip
S = (Av*fy*d)/Vs		Ag	=	7344	in^2
		0.0025*Ag	=	18.4	in^2
TRY	#8	Abar	=	0.79	in^2
Max. Spacing	S ≤ L/3 = 136	S	=	8.00	in **USE**
	S ≤ 3t = 54				
	S ≤ 18" **Governs**	# bars required	=	40	
ρt = Av/(S*t)		ρt	=	0.2167	>0.0025 **OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)		ρl	=	0.40781	>0.0025 **OK**
Max. Spacing	S ≤ L/3 = 136	S	=	6	in
	S ≤ 3t = 54				
	S ≤ 18" **Governs**				
TRY	#11	A/bar	=	1.56	in^2
		# bars required	=	28	
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
		h/l	=	0.7156	ρl≥ρt
		ρl≥ρt	is	**OK**	

WALL DIST. HORIZ. REINF.	40	#8	@	8 " O.C.
WALL DIST. VERT. REINF.	28	#11	@	6 " O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d					**TRY** #6	A/bar	=	0.44	in^2
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in		# bars required	=	1	
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.73	in^2	**Check Capacity:**	a	=	0.65	in
	a	=	0.72	in	C=T 0.85*f'c*a*b =As*fy	c	=	0.76	in
jd = d - (a/2)	jd	=	326.04	in	c = a/0.85	εt	=	1.61	>0.0025 **OK**
	As	=	0.66	in^2	εu = 0.003 dt = L-3"				
					εt = εu*((dt-c)/c)			Wall 1	

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.2167	>	$(\rho t)_{min.}$ =	0.0025	**OK**	**WALL DIST. HORIZ. REINF.** 40 #8 @ 8 " O.C.
$\rho l_{provd.}$ =	0.4078	>	$(\rho l)_{min.}$ =	0.0025	**OK**	**WALL DIST. VERT. REINF.** 24 #11 @ 6 " O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 93647 kips > $V_{u=}$ 205 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 11,618 kip-ft > $M_{u=}$ 968 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

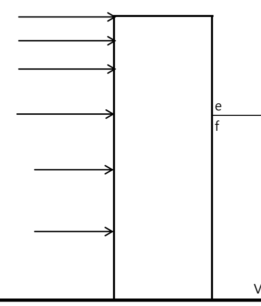
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 1 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.0 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 1 b**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	7344	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	3752	k
FACTORED MOMENT LOAD	M_u	=	1051	ft-k
FACTORED SHEAR LOAD	V_u	=	196	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	195.5	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	124863.3	kip
$V_n \le A_{cv}(\alpha c*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip
			Reinf. According to Ch 14	
Vs = Vu/(0.75) - Vc	Vs	=	-	kip
S = (Av*fy*d)/Vs	Ag	=	7344	in^2
	0.0025*Ag	=	18.4	in^2
TRY #8	A/bar	=	0.79	in^2
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in **USE**
S ≤ 3t = 54	Av	=	7344.00	
S ≤ 18" **Governs**	# bars required	=	40	
ρt = Av/(S*t)	ρt	=	0.2167	>0.0025 **OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.36318	>0.0025 **OK**
Max. Spacing S ≤ L/3 = 136	S	=	6	in
S ≤ 3t = 54				
S ≤ 18" **Governs**				
TRY #11	A/bar	=	1.56	in^2
	# bars required	=	25	

ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)

h/l	=	1.1324	ρl≥ρt
ρl≥ρt	is	**OK**	

WALL DIST. HORIZ. REINF.	40	#8	@	8 " O.C.
WALL DIST. VERT. REINF.	25	#11	@	6 " O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.80	in^2
	a	=	0.78	in
jd = d - (a/2)	jd	=	326.01	in
	As	=	0.72	in^2

	A/bar	=	1.56	in^2
	# bars required	=	0	
TRY #11				
Check Capacity:	a	=	0.70	in
C=T 0.85*f'c*a*b =As*fy	c	=	0.83	in
c = a/0.85	εt	=	1.48	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\, d_u)$ for ACI 21.9.6.2 apply c < 0.83 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 6.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.2167 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.3632 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 2786.9 kips > V_u = 195.5 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 12,612 kip-ft > M_u = 1,051 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00 " O.C.
WALL DIST. VERT. REINF.	28	#11	@	6 " O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 2 a**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	292.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	7344	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	2527	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	3371	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	1266	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	1265.6	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	85557.6	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	944.3	kip	
S = (Av*fy*d)/Vs	Ag	=	7344	in^2	
	0.0025*Ag	=	18.4	in^2	
TRY #8	Abar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	USE
S ≤ 3t = 54					
S ≤ 18" Governs	# bars required	=	23		
ρt = Av/(S*t)	ρt	=	0.1275	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.23903	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	17		

ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)

h/l	=	0.7156	ρl≥ρt
ρl≥ρt	is	OK	

WALL DIST. HORIZ. REINF.	23	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	17	#11	@	6	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	2.55	in^2
	a	=	2.50	in
jd = d - (a/2)	jd	=	325.15	in
	As	=	2.30	in^2

TRY #11	A/bar	=	1.56	in^2
	# bars required	=	1	
Check Capacity:	a	=	2.26	in
C=T 0.85*f'c*a*b =As*fy	c	=	2.66	in
c = a/0.85	εt	=	0.46	>0.0025 OK
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.1275	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.2390	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	23	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	24	#11	@	6	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 64168 kips > $V_{u=}$ 1266 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 40,452 kip-ft > $M_{u=}$ 3,371 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

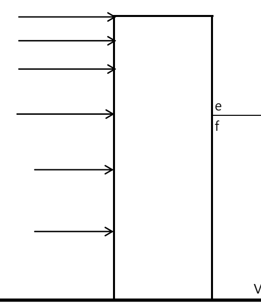
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 3 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.0 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Wall 2 b

INPUT DATA & DESIGN SUMMARY				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	7344	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			929.0	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	4123.0	k
FACTORED MOMENT LOAD	M_u	=	1268.0	ft-k
FACTORED SHEAR LOAD	V_u	=	114.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	114.0	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3715.8	kip
	ΦVn	=	2786.9	kip
ACI 318-08 § 21.9.4	Vn	≤	124863.3	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	743.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	464.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	278.7	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	7344	in^2	
	0.0025*Ag	=	18.4	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	USE
S ≤ 3t = 54	Av	=	7344.00		
S ≤ 18" Governs	# bars required	=	40		
ρt = Av/(S*t)	ρt	=	0.2167	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.36318	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	25		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.1324	ρl ≥ ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	25	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.96	in^2
	a	=	0.94	in
jd = d - (a/2)	jd	=	325.93	in
	As	=	0.86	in^2

	A/bar	=	1.56	in^2
	# bars required	=	1	
TRY #11				
Check Capacity:	a	=	0.85	in
C=T 0.85*f'c*a*b =As*fy	c	=	1.00	in
c = a/0.85	εt	=	1.22	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L^*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 1.00 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 23.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.2167 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.3632 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 2786.9 kips > V_u = 114.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 15,216 kip-ft > M_u = 1,268 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 5 a**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	4320	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			546.4	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	897	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	9415	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	353	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	353.0	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2185.8	kip
	ΦVn	=	1639.3	kip
ACI 318-08 § 21.9.4	Vn	≤	30240.0	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	437.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	273.2	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	163.9	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	33.5	kip	
S = (Av*fy*d)/Vs	Ag	=	4320	in^2	
	0.0025*Ag	=	10.8	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 80	S	=	12.00	in	USE
S ≤ 3t = 54					
S ≤ 18" Governs	# bars required	=	25		
ρt = Av/(S*t)	ρt	=	0.0500	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.07217	>0.0025	OK
Max. Spacing S ≤ L/3 = 80	S	=	12	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	35		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.5665	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	25	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	35	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	12.11	in^2
	a	=	11.87	in
jd = d - (a/2)	jd	=	186.06	in
	As	=	11.24	in^2

TRY #8	A/bar	=	0.79	in^2	
	# bars required	=	14		
Check Capacity:	a	=	11.02	in	
C=T 0.85*f'c*a*b =As*fy	c	=	12.97	in	
c = a/0.85	εt	=	0.05	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)			Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.0500	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$ =	0.0722	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	25	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 22680 kips > V_u = 353 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 112,980 kip-ft > M_u = 9,415 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

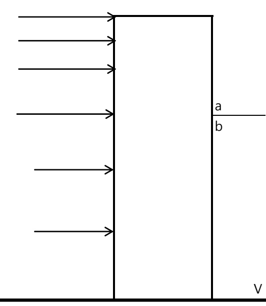
CHECK BOUNDARY ZONE REQUIREMENTS

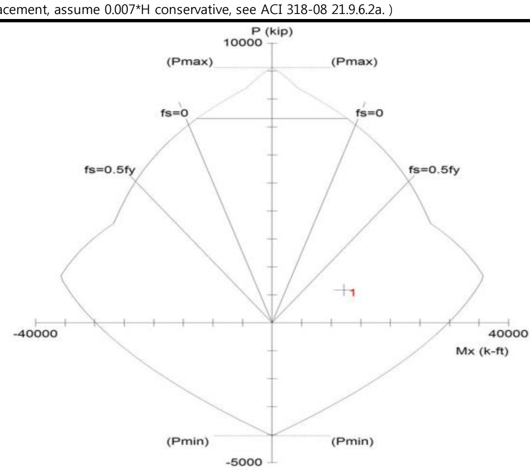
AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 57.14 in. **No Boundary Element Needed**

where c = 13 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)





Wall 5 b

INPUT DATA & DESIGN SUMMARY				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	4320	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			546.4	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	463.0	k
FACTORED MOMENT LOAD	M_u	=	17087.8	ft-k
FACTORED SHEAR LOAD	V_u	=	405.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	405.0	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2185.8	kip
	ΦVn	=	1639.3	kip
ACI 318-08 § 21.9.4	Vn	≤	36720.0	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	437.2	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	273.2	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	163.9	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	102.8	kip	
S = (Av*fy*d)/Vs	Ag	=	4320	in^2	
	0.0025*Ag	=	10.8	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 80	S	=	8.00	in	USE
S ≤ 3t = 54	Av	=	4320.00		
S ≤ 18" Governs	# bars required	=	14		
ρt = Av/(S*t)	ρt	=	0.0750	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.09584	>0.0025	OK
Max. Spacing S ≤ L/3 = 80	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	7		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.9250	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	14	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	7	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	21.98	in^2
	a	=	21.54	in
jd = d - (a/2)	jd	=	181.23	in
	As	=	20.95	in^2

	A/bar	=	1.56	in^2
TRY #11	# bars required	=	13	
Check Capacity:	a	=	20.54	in
C=T 0.85*f'c*a*b =As*fy	c	=	24.17	in
c = a/0.85	εt	=	0.03	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 24.17 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 65.9 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0750 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0958 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1639.3 kips > V_u = 405.0 **OK**

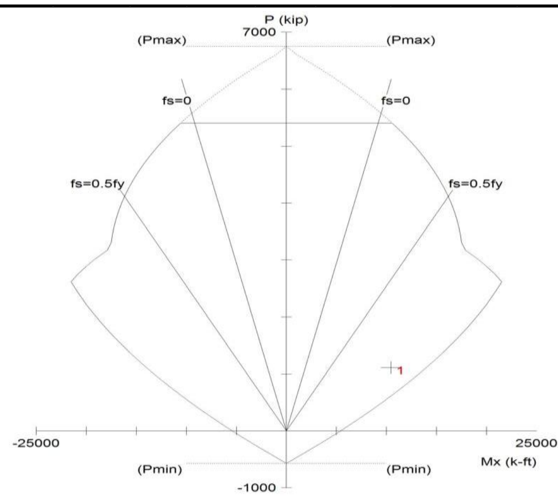
CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 205,054 kip-ft > M_u = 17,088 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	14	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	16	#11	@	6	" O.C.



Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 6 a**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	2880	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			364.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1111	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	10463	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	291	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	291.0	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1457.2	kip
	ΦVn	=	1092.9	kip
ACI 318-08 § 21.9.4	Vn	≤	20160.0	kip
$V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	291.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	182.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	109.3	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	96.6	kip	
S = (Av*fy*d)/Vs	Ag	=	2880	in^2	
	0.0025*Ag	=	7.2	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 80	S	=	12.00	in	**USE**
S ≤ 3t = 36					
S ≤ 18" **Governs**	# bars required	=	16		
ρt = Av/(S*t)	ρt	=	0.0500	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.07217	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 80	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	24		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.5665	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	16	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	13.46	in^2
	a	=	19.79	in
jd = d - (a/2)	jd	=	182.11	in
	As	=	12.77	in^2

TRY #7	A/bar	=	0.6	in^2	
	# bars required	=	21		
Check Capacity:	a	=	18.78	in	
C=T 0.85*f'c*a*b =As*fy	c	=	22.09	in	
c = a/0.85	εt	=	0.03	>0.0025	**OK**
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)					

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0500	>	$(\rho t)_{min.}$ =	0.0025	**OK**	
$\rho l_{provd.}$ = 0.0722	>	$(\rho l)_{min.}$ =	0.0025	**OK**	

WALL DIST. HORIZ. REINF.	16	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c \sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 15120 kips > $V_{u=}$ 291 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 125,555 kip-ft > $M_{u=}$ 10,463 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

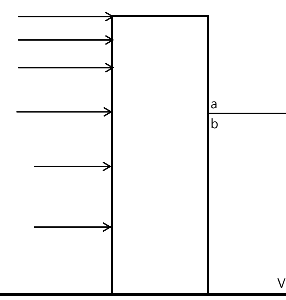
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 57.14 in. **No Boundary Element Needed**

where c = 22 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 6b**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	240.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	2880	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			364.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1165.1	k
FACTORED MOMENT LOAD	M_u	=	10224.2	ft-k
FACTORED SHEAR LOAD	V_u	=	101.2	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	101.2	kip
Vn = Vc + Vs	d	=	192.0	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1457.2	kip
	ΦVn	=	1092.9	kip
ACI 318-08 § 21.9.4	Vn	≤	67689.0	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	291.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	182.1	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	109.3	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	2880	in^2	
	0.0025*Ag	=	7.2	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 80	S	=	8.00	in	USE
S ≤ 3t = 36	Av	=	2880.00		
S ≤ 18" Governs	# bars required	=	40		
ρt = Av/(S*t)	ρt	=	0.3251	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.41779	>0.0025	OK
Max. Spacing S ≤ L/3 = 80	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	19		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.9250	ρl≥ρt	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	19	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	172.80	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	13.15	in^2
	a	=	19.34	in
jd = d - (a/2)	jd	=	182.33	in
	As	=	12.46	in^2

	A/bar	=	0.79	in^2
	# bars required	=	16	
TRY #8				
Check Capacity:	a	=	18.33	in
C=T 0.85*f'c*a*b =As*fy	c	=	21.56	in
c = a/0.85	εt	=	0.03	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 57.14 in. **No Boundary Element Needed**

where c = 22 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 3.2 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.3251	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$ =	0.4178	>	$(\rho l)_{min.}$ =	0.0025	OK

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 1092.9 kips > V_u = 101.2 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 122,690 kip-ft > M_u = 10,224 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	48	#8	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 7 c**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	479	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	1723	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	516	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	516.0	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	8258.1	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	537.4	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	12.00	in	USE
S ≤ 3t = 36					
S ≤ 18" Governs	# bars required	=	8		
ρt = Av/(S*t)	ρt	=	0.0258	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.0283	>0.0025	OK
Max. Spacing S ≤ L/3 = 41.332	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" Governs					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)	h/l	=	3.0320	FALSE	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	9	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	4.29	in^2
	a	=	6.31	in
jd = d - (a/2)	jd	=	96.04	in
	As	=	3.99	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
Check Capacity:	a	=	5.86	in	
C=T 0.85*f'c*a*b =As*fy	c	=	6.90	in	
c = a/0.85	εt	=	0.05	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)			Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.0258	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.0283	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	12	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 6194 kips > $V_{u\,=}$ 516 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 20,676 kip-ft > $M_{u\,=}$ 1,723 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

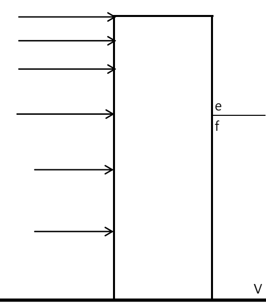
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 29.52 in. **No Boundary Element Needed**

where c = 7 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Wall 7 d

INPUT DATA & DESIGN SUMMARY				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	860.6	k
FACTORED MOMENT LOAD	M_u	=	1283.0	ft-k
FACTORED SHEAR LOAD	V_u	=	186.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	186.0	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	9411.2	kip
$V_n \le A_{cv}(\alpha c*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	97.4	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	8.00	in	USE
S ≤ 3t = 36	Av	=	1487.95		
S ≤ 18" Governs	# bars required	=	5		
ρt = Av/(S*t)	ρt	=	0.0387	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.05167	>0.0025	OK
Max. Spacing S ≤ L/3 = 41.332	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" Governs					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	2		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.7259	FALSE	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	5	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	2	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	3.19	in^2
	a	=	4.70	in
jd = d - (a/2)	jd	=	96.85	in
	As	=	2.94	in^2

	A/bar	=	1.56	in^2
	# bars required	=	2	
TRY #11				
Check Capacity:	a	=	4.33	in
C=T 0.85*f'c*a*b =As*fy	c	=	5.09	in
c = a/0.85	εt	=	0.07	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)			Wall 1	

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 29.52 in. **No Boundary Element Needed**

where c = 5 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 3.2 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0387 > $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0517 > $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 564.6 kips > V_u = 186.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

Φ M_n = 15,396 kip-ft > M_u = 1,283 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	5	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	14	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 8 c**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	376.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	438	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	1724	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	517	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	516.7	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	8258.1	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	538.4	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	12.00	in	USE
S ≤ 3t = 36					
S ≤ 18" Governs	# bars required	=	8		
ρt = Av/(S*t)	ρt	=	0.0258	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.0283	>0.0025	OK
Max. Spacing S ≤ L/3 = 41.332	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" Governs					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.0320	FALSE	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	9	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	4.29	in^2
	a	=	6.31	in
jd = d - (a/2)	jd	=	96.04	in
	As	=	3.99	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
Check Capacity:	a	=	5.87	in	
C=T 0.85*f'c*a*b =As*fy	c	=	6.90	in	
c = a/0.85	εt	=	0.05	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)			Wall 1		

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.0258	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$ =	0.0283	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	8	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	12	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 6194 kips > $V_{u\,=}$ 517 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$\Phi\, M_n$ = 20,684 kip-ft > $M_{u\,=}$ 1,724 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

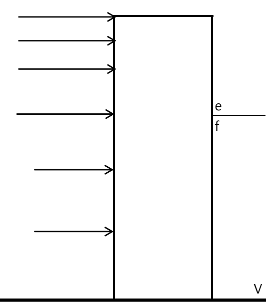
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 29.52 in. **No Boundary Element Needed**

where c = 7 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Wall 8 d

INPUT DATA & DESIGN SUMMARY				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	124.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	1487.952	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			188.2	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	171.4	k
FACTORED MOMENT LOAD	M_u	=	1283.0	ft-k
FACTORED SHEAR LOAD	V_u	=	186.4	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	186.4	kip
Vn = Vc + Vs	d	=	99.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	752.9	kip
	ΦVn	=	564.6	kip
ACI 318-08 § 21.9.4	Vn	≤	9411.2	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	150.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	94.1	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	56.5	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	98.0	kip	
S = (Av*fy*d)/Vs	Ag	=	1487.952	in^2	
	0.0025*Ag	=	3.7	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 41.332	S	=	8.00	in	**USE**
S ≤ 3t = 36	Av	=	1487.95		
S ≤ 18" **Governs**	# bars required	=	5		
ρt = Av/(S*t)	ρt	=	0.0387	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.0412	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 41.332	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	2		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.7259	FALSE	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	5	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	2	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	89.28	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	3.19	in^2
	a	=	4.70	in
jd = d - (a/2)	jd	=	96.85	in
	As	=	2.94	in^2

	A/bar	=	1.56	in^2
TRY #11	# bars required	=	2	
Check Capacity:	a	=	4.33	in
C=T 0.85*f'c*a*b =As*fy	c	=	5.09	in
c = a/0.85	εt	=	0.07	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)		Wall 1		

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\, d_u)$ for ACI 21.9.6.2 apply c < 5.09 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 12.1 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0387	>	$(\rho t)_{min.}$ =	0.0025	**OK**
$\rho l_{provd.}$ = 0.0412	>	$(\rho l)_{min.}$ =	0.0025	**OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 564.6 kips > V_u = 186.4 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 15,396 kip-ft > M_u = 1,283 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	5	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	12	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY **Wall 9 c** **X-Direction**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi	Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1225	k	at BASE
REBAR YIELD STRESS	f_y	=	60	ksi	FACTORED BASE MOMENT LOAD	M_u	=	635	ft-k	
HEIGHT OF WALL	H	=	400.0	in	FACTORED BASE SHEAR LOAD	V_u	=	72	k	
LENGTH OF SHEAR WALL	L	=	408.0	in						
THICKNESS OF WALL	t	=	12	in	**THE WALL DESIGN IS ADEQUATE.**					
	Acv	=	4896	in^2						
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1						

1. Check Permitted Shear Strength
ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	71.8	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	32748.8	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			**OK**	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	4896	in^2	
	0.0025*Ag	=	12.2	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	12.00	in	**USE**
S ≤ 3t = 36					
S ≤ 18" **Governs**	# bars required	=	15		
ρt = Av/(S*t)	ρt	=	0.0448	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.07697	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 136	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	25		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	0.9803	ρl≥ρt	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	25	#6	@	12	" O.C.

5. Design for Flexture
Assume Tension-controlled section, Φ = 0.9
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in	
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.48	in^2	
	a	=	0.71	in	
jd = d - (a/2)	jd	=	326.05	in	
	As	=	0.43	in^2	

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	1		
Check Capacity:	a	=	0.64	in	
C=T 0.85*f'c*a*b =As*fy	c	=	0.75	in	
c = a/0.85	εt	=	1.63	>0.0025	**OK**
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)		Wall 1			

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.0448	>	$(\rho t)_{min.}$ =	0.0025	**OK**
$\rho l_{provd.}$ =	0.0770	>	$(\rho l)_{min.}$ =	0.0025	**OK**

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	28	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 24562 kips > $V_{u=}$ 72 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 7,620 kip-ft > $M_{u=}$ 635 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

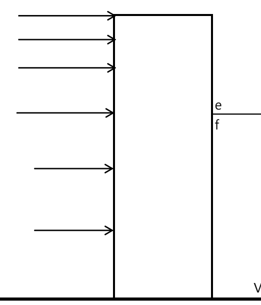
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 1 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY **Wall 9 d**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	4896	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	2774.9	k
FACTORED MOMENT LOAD	M_u	=	26.1	ft-k
FACTORED SHEAR LOAD	V_u	=	14.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	14.0	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	115071.3	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip	
			Reinf. According to Ch 14		
Vs = Vu/(0.75) - Vc	Vs	=	-	kip	
S = (Av*fy*d)/Vs	Ag	=	4896	in^2	
	0.0025*Ag	=	12.2	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	8.00	in	**USE**
S ≤ 3t = 36	Av	=	4896.00		
S ≤ 18" **Governs**	# bars required	=	40		
ρt = Av/(S*t)	ρt	=	0.3251	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.54562	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 136	S	=	6	in	
S ≤ 3t = 36					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	25		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.1324	ρl≥ρt	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	25	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.02	in^2
	a	=	0.03	in
jd = d - (a/2)	jd	=	326.39	in
	As	=	0.02	in^2

	A/bar	=	1.56	in^2
	# bars required	=	0	
TRY #11				
Check Capacity:	a	=	0.03	in
C=T 0.85*f'c*a*b =As*fy	c	=	0.03	in
c = a/0.85	εt	=	39.81	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)		Wall 1		

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L^*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 0.03 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 4.4 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.3251	>	$(\rho t)_{min.}$ =	0.0025	**OK**
$\rho l_{provd.}$ =	0.5456	>	$(\rho l)_{min.}$ =	0.0025	**OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1857.9 kips > $V_{u=}$ 14.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 313 kip-ft > $M_{u=}$ 26 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY **Wall 10 c**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	400.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	4896	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	974	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	6062	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	565	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	565.0	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	44553.6	kip
$V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	185.8	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	257.9	kip	
S = (Av*fy*d)/Vs	Ag	=	4896	in^2	
	0.0025*Ag	=	12.2	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 136	S	=	12.00	in	USE
S ≤ 3t = 36					
S ≤ 18" Governs	# bars required	=	28		
ρt = Av/(S*t)	ρt	=	0.0850	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.1100	>0.0025	OK
Max. Spacing S ≤ L/3 = 136	S	=	12	in	
S ≤ 3t = 36					
S ≤ 18" Governs					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	36		

ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)

h/l	=	0.9803	ρl≥ρt
ρl≥ρt	is	OK	

WALL DIST. HORIZ. REINF.	28	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	36	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	4.59	in^2
	a	=	6.74	in
jd = d - (a/2)	jd	=	323.03	in
	As	=	4.17	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	9		
Check Capacity:	a	=	6.13	in	
C=T 0.85*f'c*a*b =As*fy	c	=	7.21	in	
c = a/0.85	εt	=	0.17	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)					

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho_{t\,provd.}$ =	0.0850	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho_{l\,provd.}$ =	0.1100	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	28	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	36	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}$ (αc*√f_c' + $\rho_t f_y$) αc = 2 (conservative) 33415 kips > V_u = 565 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 72,739 kip-ft > M_u = 6,062 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

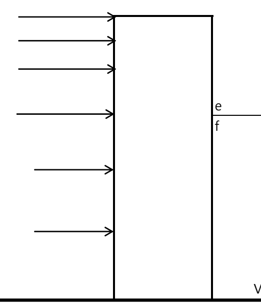
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 97.14 in. **No Boundary Element Needed**

where c = 7 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



Wall 10 d

INPUT DATA & DESIGN SUMMARY				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	408.0	in
THICKNESS OF WALL	t	=	12	in
	Acv	=	4896	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			619.3	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	467	k
FACTORED MOMENT LOAD	M_u	=	6729	ft-k
FACTORED SHEAR LOAD	V_u	=	81	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	80.6	kip
Vn = Vc + Vs	d	=	326.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	2477.2	kip
	ΦVn	=	1857.9	kip
ACI 318-08 § 21.9.4	Vn	≤	115071.3	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	495.4	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	309.7	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu		1/2ΦVc	=	185.8	kip
				Reinf. According to Ch 14	
Vs = Vu/(0.75) - Vc		Vs	=	-	kip
S = (Av*fy*d)/Vs		Ag	=	4896	in^2
		0.0025*Ag	=	12.2	in^2
TRY	#8	A/bar	=	0.79	in^2
Max. Spacing S ≤ L/3 =	136	S	=	8.00	in **USE**
S ≤ 3t =	36	Av	=	4896.00	
S ≤ 18"	**Governs**	# bars required	=	40	
ρt = Av/(S*t)		ρt	=	0.3251	>0.0025 **OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)		ρl	=	0.54562	>0.0025 **OK**
Max. Spacing S ≤ L/3 =	136	S	=	6	in
S ≤ 3t =	36				
S ≤ 18"	**Governs**				
TRY	#11	A/bar	=	1.56	in^2
		# bars required	=	25	
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
		h/l	=	1.1324	ρl≥ρt
		ρl≥ρt	is	**OK**	

WALL DIST. HORIZ. REINF.	40	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	25	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	293.76	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	5.09	in^2
	a	=	7.49	in
jd = d - (a/2)	jd	=	322.66	in
	As	=	4.63	in^2

	A/bar	=	1.56	in^2
	# bars required	=	3	
TRY #11				
Check Capacity:	a	=	6.81	in
C=T 0.85*f'c*a*b =As*fy	c	=	8.02	in
c = a/0.85	εt	=	0.15	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)		Wall 1		

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 8.02 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 42.4 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.3251 **>** $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.5456 **>** $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1857.9 kips **>** $V_{u\ =}$ 80.6 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 80,742 kip-ft **>** $M_{u\ =}$ 6,729 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	40	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	28	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 11 e**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi	
REBAR YIELD STRESS	f_y	=	60	ksi	
HEIGHT OF WALL	H	=	400.0	in	
LENGTH OF SHEAR WALL	L	=	168.0	in	
THICKNESS OF WALL	t	=	18	in	
	Acv	=	3024	in^2	
ACI 318-08 § 21.9.2.2, IF Vu ≥2*Acv*√(f'c) ; need at least two curtains (rows) =			382.5	Need 2	

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1063	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	5498	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	2113	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	2113.0	kip
Vn = Vc + Vs	d	=	134.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1530.0	kip
	ΦVn	=	1147.5	kip
ACI 318-08 § 21.9.4	Vn	≤	18446.4	kip
$V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	306.0	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	191.3	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	114.8	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	2511.3	kip	
S = (Av*fy*d)/Vs	Ag	=	3024	in^2	
	0.0025*Ag	=	7.6	in^2	
TRY #6	Abar	=	0.44	in^2	
Max. Spacing S ≤ L/3 = 56	S	=	12.00	in	USE
S ≤ 3t = 54					
S ≤ 18" Governs	# bars required	=	17		
ρt = Av/(S*t)	ρt	=	0.0350	>0.0025	OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.03694	>0.0025	OK
Max. Spacing S ≤ L/3 = 56	S	=	12	in	
S ≤ 3t = 54					
S ≤ 18" Governs					
TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	18		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	2.3807	FALSE	
	ρl≥ρt	is	OK		

WALL DIST. HORIZ. REINF.	17	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	18	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	120.96	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	10.10	in^2
	a	=	9.90	in
jd = d - (a/2)	jd	=	129.45	in
	As	=	9.44	in^2

TRY #6	A/bar	=	0.44	in^2	
	# bars required	=	21		
Check Capacity:	a	=	9.25	in	
C=T 0.85*f'c*a*b =As*fy	c	=	10.89	in	
c = a/0.85	εt	=	0.04	>0.0025	OK
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)					

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.0350	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.0369	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	17	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	24	#6	@	12	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 13835 kips > V_u = 2113 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 65,974 kip-ft > M_u = 5,498 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

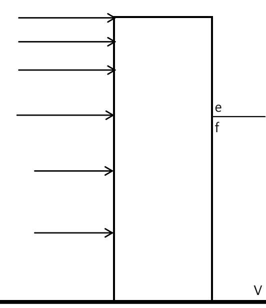
CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 40.00 in. **No Boundary Element Needed**

where c = 11 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)



INPUT DATA & DESIGN SUMMARY — **Wall 11 f**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	148.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	2663.928	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			337.0	Need 1

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	202.8	k
FACTORED MOMENT LOAD	M_u	=	443.4	ft-k
FACTORED SHEAR LOAD	V_u	=	151.5	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	151.5	kip
Vn = Vc + Vs	d	=	118.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1347.9	kip
	ΦVn	=	1010.9	kip
ACI 318-08 § 21.9.4	Vn	≤	18047.9	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	269.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	168.5	According to Ch.14

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	101.1	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	-67.6	kip	
S = (Av*fy*d)/Vs	Ag	=	2663.928	in^2	
	0.0025*Ag	=	6.7	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 49.332	S	=	8.00	in	**USE**
S ≤ 3t = 54	Av	=	2663.93		
S ≤ 18" **Governs**	# bars required	=	8		
ρt = Av/(S*t)	ρt	=	0.0462	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.03265	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 49.332	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	2		
ACI 318-08 § 21.9.4.4, IF hw/lw≤ 2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.1217	FALSE	
	ρl≥ρt	is	**NOT OK**		

WALL DIST. HORIZ. REINF.	8	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	2	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	106.56	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	0.92	in^2
	a	=	0.91	in
jd = d - (a/2)	jd	=	117.94	in
	As	=	0.84	in^2

	A/bar	=	1.56	in^2
	# bars required	=	1	
TRY #11				
Check Capacity:	a	=	0.82	in
C=T 0.85*f'c*a*b =As*fy	c	=	0.96	in
c = a/0.85	εt	=	0.46	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)				

Wall 1

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 0.96 in. **No Boundary Element Needed**

where c = 0 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 38.5 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0462 **>** $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0326 **>** $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1010.9 kips **>** $V_{u\,=}$ 151.5 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 5,321 kip-ft **>** $M_{u\,=}$ 443 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	8	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	14	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY — **Wall 12 e**

CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL	H	=	400.0	in
LENGTH OF SHEAR WALL	L	=	148.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	2663.9928	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			337.0	Need 2

X-Direction

Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	1131	k	at BASE
FACTORED BASE MOMENT LOAD	M_u	=	5471	ft-k	
FACTORED BASE SHEAR LOAD	V_u	=	2105	k	

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	2105.0	kip
Vn = Vc + Vs	d	=	118.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1347.9	kip
	ΦVn	=	1010.9	kip
ACI 318-08 § 21.9.4	Vn	≤	15584.3	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	269.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	168.5	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	101.1	kip
			According to 11.9.9	
Vs = Vu/(0.75) - Vc	Vs	=	2537.1	kip
S = (Av*fy*d)/Vs	Ag	=	2663.9928	in^2
	0.0025*Ag	=	6.7	in^2
TRY #6	Abar	=	0.44	in^2
Max. Spacing S ≤ L/3 = 49.3332	S	=	12.00	in USE
S ≤ 3t = 54				
S ≤ 18" Governs	# bars required	=	15	
ρt = Av/(S*t)	ρt	=	0.0308	>0.0025 OK

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.06167	>0.0025 OK
Max. Spacing S ≤ L/3 = 49.3332	S	=	12	in
S ≤ 3t = 54				
S ≤ 18" Governs				
TRY #6	A/bar	=	0.44	in^2
	# bars required	=	30	
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)	h/l	=	2.7024	FALSE
	ρl≥ρt	is	OK	

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	30	#6	@	12	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	106.56	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	11.41	in^2
	a	=	11.19	in
jd = d - (a/2)	jd	=	112.81	in
	As	=	10.78	in^2

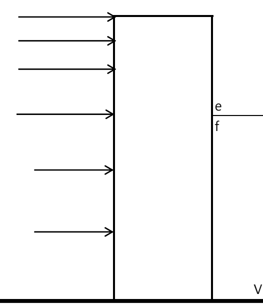
TRY #6	A/bar	=	0.44	in^2
	# bars required	=	24	
Check Capacity:	a	=	10.57	in
C=T 0.85*f'c*a*b =As*fy	c	=	12.43	in
c = a/0.85	εt	=	0.03	>0.0025 OK
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)				

Wall 1

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$	=	0.0308	>	$(\rho t)_{min.}$ =	0.0025	OK
$\rho l_{provd.}$	=	0.0617	>	$(\rho l)_{min.}$ =	0.0025	OK

WALL DIST. HORIZ. REINF.	15	#6	@	12	" O.C.
WALL DIST. VERT. REINF.	32	#6	@	12	" O.C.



CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 11688 kips > V_{u} = 2105 OK

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 65,652 kip-ft > M_u = 5,471 OK

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 35.24 in. **No Boundary Element Needed**

where c = 12 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.8 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

Wall 12 f

INPUT DATA & DESIGN SUMMARY				
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
HEIGHT OF WALL —	H	=	462.0	in
LENGTH OF SHEAR WALL	L	=	148.0	in
THICKNESS OF WALL	t	=	18	in
	Acv	=	2663.9928	in^2
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			337.0	Need 1

X-Direction				
Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	703.0	k
FACTORED MOMENT LOAD	M_u	=	2887.0	ft-k
FACTORED SHEAR LOAD	V_u	=	187.0	k

THE WALL DESIGN IS ADEQUATE.

1. Check Permitted Shear Strength

ACI 318-08 § 11.9				
ΦVn ≥ Vu	Vu	=	187.0	kip
Vn = Vc + Vs	d	=	118.4	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	1347.9	kip
	ΦVn	=	1010.9	kip
ACI 318-08 § 21.9.4	Vn	≤	18048.5	kip
$V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			OK	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	269.6	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	168.5	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	101.1	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	-20.2	kip	
S = (Av*fy*d)/Vs	Ag	=	2663.9928	in^2	
	0.0025*Ag	=	6.7	in^2	
TRY #8	A/bar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 49.3332	S	=	8.00	in	**USE**
S ≤ 3t = 54	Av	=	2663.99		
S ≤ 18" **Governs**	# bars required	=	8		
ρt = Av/(S*t)	ρt	=	0.0462	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.06167	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 49.3332	S	=	6	in	
S ≤ 3t = 54					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	4		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	3.1216	FALSE	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	8	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	4	#11	@	6	" O.C.

FALSE

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9				
Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d				
C=T 0.85*f'c*a*b =As*fy	jd	=	106.56	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	6.02	in^2
	a	=	5.90	in
jd = d - (a/2)	jd	=	115.45	in
	As	=	5.56	in^2

	A/bar	=	1.56	in^2
TRY #11	# bars required	=	4	
Check Capacity:	a	=	5.45	in
C=T 0.85*f'c*a*b =As*fy	c	=	6.41	in
c = a/0.85	εt	=	0.07	>0.0025
εu = 0.003 dt = L-3"				
εt = εu*((dt-c)/c)		Wall 1		

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

$c < (L*H) / (600\ d_u)$ for ACI 21.9.6.2 apply c < 6.41 in. **Boundary Element Needed**

where c = 6 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 38.3 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ = 0.0462 **>** $(\rho t)_{min.}$ = 0.0025 **OK**

$\rho l_{provd.}$ = 0.0617 **>** $(\rho l)_{min.}$ = 0.0025 **OK**

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv}(\alpha c\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 1010.9 kips **>** $V_{u\,=}$ 187.0 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 34,644 kip-ft **>** $M_{u\,=}$ 2,887 **OK**

where Φ= Min{0.9, Max[0.65 + (e_t - 0.002)(250/3) , 0.65]} = 0.900 (ACI 318-08 Fig. R9.3.2)

WALL DIST. HORIZ. REINF.	8	#8	@	8.00	" O.C.
WALL DIST. VERT. REINF.	14	#11	@	6	" O.C.

Special Reinforced Concrete Shear Wall Design Based on ACI 318-08 Ch. 21.9

INPUT DATA & DESIGN SUMMARY					Wall 13	X-Direction					
CONCRETE STRENGTH (ACI 318 5.1.1)	f_c'	=	4	ksi		Load Combo: 1.2 D + 1.0L +1.0E	Pu	=	3716	k	at BASE
REBAR YIELD STRESS	f_y	=	60	ksi		FACTORED BASE MOMENT LOAD	M_u	=	41262	ft-k	
HEIGHT OF WALL	H	=	376.0	in		FACTORED BASE SHEAR LOAD	V_u	=	1317	k	
LENGTH OF SHEAR WALL	L	=	254.0	in							
THICKNESS OF WALL	t	=	24	in		**THE WALL DESIGN IS ADEQUATE.**					
	Acv	=	6096.96	in^2							
ACI 318-08 § 21.9.2.2, IF Vu ≥ 2*Acv*√(f'c) ; need at least two curtains (rows) =			771.2	Need 2							

1. Check Permitted Shear Strength

ACI 318-08 § 11.9

ΦVn ≥ Vu	Vu	=	1317.0	kip
Vn = Vc + Vs	d	=	203.2	in
Vn ≤ 10*t*d*√(f'c) d=0.8*L	Vn	=	3084.8	kip
	ΦVn	=	2313.6	kip
ACI 318-08 § 21.9.4	Vn	≤	53429.2	kip
$V_n \le A_{cv} (\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative)			**OK**	

2. Shear Strength Provided by Vc

Vc ≤ 2*λ*t*d*√(f'c) λ = 1.0 (for N.W.C)	Vc	=	617.0	kip
Note: If Vu≤Acv*√(f'c) can choose ρt, ρl according to Ch.14		=	385.6	FALSE

3. Required Horizontal Shear Reinforcement

1/2ΦVc < Vu	1/2ΦVc	=	231.4	kip	
			According to 11.9.9		
Vs = Vu/(0.75) - Vc	Vs	=	1139.0	kip	
S = (Av*fy*d)/Vs	Ag	=	6096.96	in^2	
	0.0025*Ag	=	15.2	in^2	
TRY #8	Abar	=	0.79	in^2	
Max. Spacing S ≤ L/3 = 84.68	S	=	8.00	in	**USE**
S ≤ 3t = 72					
S ≤ 18" **Governs**	# bars required	=	19		
ρt = Av/(S*t)	ρt	=	0.0794	>0.0025	**OK**

4. Required Vertical Shear Reinforcement

ρl = Av/S*h ≥ 0.0025 +0.5 (2.5 - h/L)*(ρt-0.0025)	ρl	=	0.1186	>0.0025	**OK**
Max. Spacing S ≤ L/3 = 84.68	S	=	6	in	
S ≤ 3t = 72					
S ≤ 18" **Governs**					
TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	11		
ACI 318-08 § 21.9.4.4, IF hw/lw≤2 ; need reinf. In two directions (ρl≥ρt)					
	h/l	=	1.4799	ρl≥ρt	
	ρl≥ρt	is	**OK**		

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	11	#11	@	6	" O.C.

5. Design for Flexture

Assume Tension-controlled section, Φ = 0.9

Mn = As*fy*(d-(a/2))= As*fy*j jd = 0.9*d

C=T 0.85*f'c*a*b =As*fy	jd	=	182.91	in
Mu = Φ Mn = Φ As*fy*j*d	As	=	50.13	in^2
	a	=	36.86	in
jd = d - (a/2)	jd	=	184.80	in
	As	=	49.62	in^2

TRY #11	A/bar	=	1.56	in^2	
	# bars required	=	32		
Check Capacity:	a	=	36.48	in	
C=T 0.85*f'c*a*b =As*fy	c	=	42.92	in	
c = a/0.85	εt	=	0.01	>0.0025	**OK**
εu = 0.003 dt = L-3"					
εt = εu*((dt-c)/c)		Wall 1			

CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-08 14.3, 21.9.2)

$\rho t_{provd.}$ =	0.0794	>	$(\rho t)_{min.}$ =	0.0025	**OK**	
$\rho l_{provd.}$ =	0.1186	>	$(\rho l)_{min.}$ =	0.0025	**OK**	

WALL DIST. HORIZ. REINF.	19	#8	@	8	" O.C.
WALL DIST. VERT. REINF.	32	#11	@	6	" O.C.

CHECK SHEAR CAPACITY (ACI 318-08 11.2 & 21.9.4)

$\Phi V_n \le A_{cv} (\alpha c^*\sqrt{f_c'} + \rho_t f_y)$ αc = 2 (conservative) 40072 kips > V_u = 1317 **OK**

CHECK FLEXURAL & AXIAL CAPACITY

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

ΦM_n = 495,139 kip-ft > M_u = 41,262 **OK**

where Φ= 0.900 (ACI 318-08 Fig. R9.3.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-05 21.9.6.2, 21.9.6.3, and 21.9.6.5(a) PROVIDED THAT

c < (L*H) / (600 d_u) for ACI 21.9.6.2 apply c < 60.49 in. **No Boundary Element Needed**

where c = 43 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

d_u = 2.6 in. (design displacement, assume 0.007*H conservative, see ACI 318-08 21.9.6.2a.)

Appendix G – Lateral Force Resisting System Design Checks-System #2

Special Moment Frame - Level 2					
Moment Frame #	kx cantilever (k/in)	D(in)**	ky cantilever (k/in)	Ix =Ri/∑Ri	I y=Ri/∑Ri
1	125.00	0.0		17.54%	
2	72.57	0.0		10.18%	
3		0.0	72.57		14.44%
5		0.0	100.00		19.91%
6		0.0	100.00		19.91%
7	125.00	0.0		17.54%	
8	125.00	0.0		17.54%	
9	41.67	0.0		5.85%	
10	41.67	0.0		5.85%	
11		0.0	71.43		14.22%
12		0.0	71.43		14.22%
14		0.0	43.48		8.65%
15		0.0	43.48		8.65%
16	90.91	0.0		12.76%	
17	90.91	0.0		12.76%	
∑Ri =	**712.72**		**502.38**	**100.00%**	**100.00%**

Center of Rigidity Special Moment Frame - Level 2			
X Direction	**k_{ix} (k/ft)**	**x_i (ft)**	**k_{ix} x_i**
SW3	870.83	31.50	27431.06
SW14	521.74	224.00	116869.57
SW5	1200.00	21.67	26000.00
SW6	1200.00	32.00	38400.00
SW11	857.14	224.00	192000.00
SW12	857.14	256.00	219428.57
SW15	500.00	247.67	123835.00
∑	6006.85		743964.20
		x (ft) = $\sum k_{ix} x_i / k_{ix}$ =	**123.85**
Y Direction	**k_{iy} (k/ft)**	**y_i (ft)**	**k_{iy} y_i**
SW1	1500.00	97.00	145500.00
SW2	870.83	64.00	55732.95
SW7	1500.00	24.00	36000.00
SW8	1500.00	44.00	66000.00
SW16	1090.91	64.00	69818.18
SW17	1090.91	94.00	102545.45
SW9	500.00	124.00	62000.00
SW10	500.00	111.67	55835.00
∑	8552.65		593431.58
		y (ft) = $\sum k_{iy} y_i / k_{iy}$ =	**69.39**

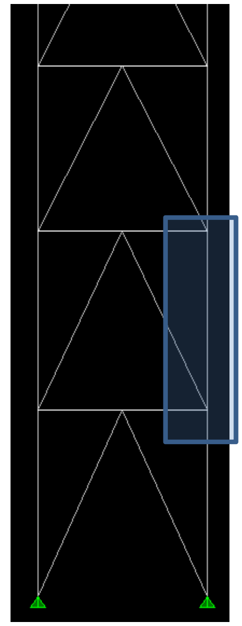
Center of Rigidity Special Concentric Braced Frame - Level 2			
X Direction	**k_{ix} (k/ft)**	**x_i (ft)**	**k_{ix} x_i**
SW3	1333.33	31.50	42000.00
SW4	1333.33	65.50	87333.33
SW5	1714.29	21.67	37142.86
SW6	1714.29	32.00	54857.14
SW11	2000.00	224.00	448000.00
SW12	2000.00	256.00	512000.00
SW13	1500.00	65.50	98250.00
SW14	1575082.14	0.00	0.00
SW15	1407353.53	256.00	360282502.54
∑ 2994030.90			361562085.87
		x (ft) = $\sum k_{ix} x_i / k_{ix}$ =	**120.76**
Y Direction	**k_{iy} (k/ft)**	**y_i (ft)**	**k_{iy} y_i**
SW1	1277.96	97.00	123961.66
SW2	1333.33	64.00	85333.33
SW7	1333.33	24.00	32000.00
SW8	1333.33	44.00	58666.67
SW9	1714.29	124.00	212571.43
SW10	1714.29	111.67	191434.29
SW16	283841.76	4.00	1135367.03
SW17	248069.26	124.00	30760587.93
∑ 540617.54			32599922.34
		y (ft) = $\sum k_{iy} y_i / k_{iy}$ =	**60.30**

Special Concentric Braced Frame - Level 2					
Moment Frame #	kx cantilever (k/in)	∆(in)	ky cantilever (k/in)	Ix =Ri/∑Ri	Iy =Ri/∑Ri
1	53.56	0.02		13.30%	
2	55.56	0.02		13.79%	
3		0.02	55.56		11.78%
4		0.02	55.56		11.78%
5		0.01	71.43		15.15%
6	71.43	0.01		17.73%	
7	83.33	0.01		20.69%	
8	83.33	0.01		20.69%	
9	55.56	0.02		13.79%	
10		0.02	55.56		11.78%
11		0.01	83.33		17.68%
12		0.01	83.33		17.68%
13		0.02	66.67		14.14%
∑Ri =	**402.77**		**471.43**	**100.00%**	**100.00%**

Frame 11 for SCBF Design - Column Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	**W14x730**			r =	4.69	in	Table 1-1
COLUMN YIELD STRESS		F_y =	50.0 ksi	ΦPn =	8810	k	Table 4-1
HEIGHT		H =	13.0 ft				
AXIAL LOAD, Factored, Pu		P =	2600.4 kips				

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)



$$\frac{P_u}{\phi P_n} > 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

Pu/ΦPn = 0.30

$$\frac{P_u}{\phi P_n} \le 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

0.36 < 1.0 **OK**

Where KL_x = 13.0 ft

$(KL / r)_{max}$ = 33.3 < EI

Pu = 2600.4 kips

Mu_x = 135.6 ft-kips

Mu_y = 144.9 ft-kips

ΦP_c = 8810.0 kips, (AISC 360-05 Chapter E) Table 4-1

\> Pu **OK**

ΦMn_x = 6230.0 ft-kips, (AISC 360-05 Chapter F) Table 3-2

\> Mu_x **OK**

ΦMn_y = 3060.0 ft-kips, (AISC 360-05 Chapter F) Table 3-4

\> Mu_y **OK**

CHECK LATERAL DEFLECTION

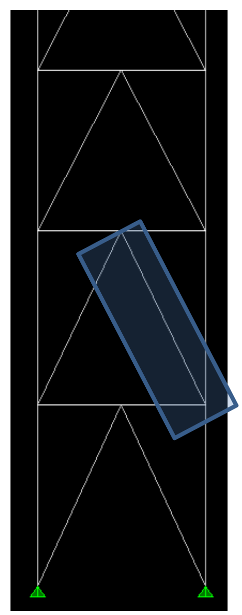
D_{max} = 0.01 in

< L / 240 = 0.05 in **OK**

Frame 11 for SCBF Design - Brace Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	**W12x252**			r =	3.34	in	Table 1-1
COLUMN YIELD STRESS		F_y =	50.0 ksi	ΦPn =	2770	k	Table 4-1
HEIGHT		H =	14.4 ft				
AXIAL LOAD, Factored, Pu		P =	664.0 kips				

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)



$$\frac{P_u}{\phi P_n} > 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

Pu/ΦPn = 0.24

$$\frac{P_u}{\phi P_n} \le 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

0.25 < 1.0 **OK**

Where KL_x = 14.4 ft, for x-x axial load.

$(KL / r)_{max}$ = 51.7 < 200

Pu = 664.0 kips

Mu_x = 0.0 ft-kips

Mu_y = 9.1 ft-kips

ΦP_c = 2770.0 kips, (AISC 360-05 Chapter E) Table 4-1

\> Pu **OK**

ΦMn_x = 1610.0 ft-kips, (AISC 360-05 Chapter F) Table 3-2

\> Mu_x **OK**

ΦMn_y = 735.0 ft-kips, (AISC 360-05 Chapter F) Table 3-4

\> Mu_y **OK**

CHECK LATERAL DEFLECTION

D_{max} = 0.09 in

< L / 240 = 0.06 in **NOT OK**

Where E_s = 29000 ksi

I_x = 272 in^4

I_y = 93.4 in^4

Frame 11 for SMF Design - Column Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	**W14x730**			r =	4.69	in	Table 1-1
COLUMN YIELD STRESS	F_y =	50.0	ksi	ΦPn =	8810	k	Table 4-1
HEIGHT	H =	13.0	ft				
AXIAL LOAD, Factored, Pu	P =	570.7	kips				

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)

$$\frac{P_u}{\phi P_n} \ge 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

Pu/ΦPn = 0.06

$$\frac{P_u}{\phi P_n} < 0.2 \Rightarrow \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

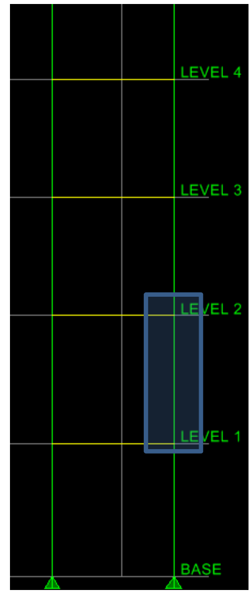
0.15 < 1.0 **OK**

Where KL_x =	13.0	ft		
$(KL / r)_{max}$ =	33.3	<	200	
Pu =	570.7	kips		
Mu_x =	501.8	ft-kips		
Mu_y =	46.8	ft-kips		
ΦP_c =	8810.0	kips, (AISC 360-05 Chapter E)	Table 4-1	
	>	Pu	**OK**	
ΦMn_x =	6230.0	ft-kips, (AISC 360-05 Chapter F)	Table 3-2	
	>	Mu_x	**OK**	
ΦMn_y =	3060.0	ft-kips, (AISC 360-05 Chapter F)	Table 3-4	
	>	Mu_y	**OK**	

CHECK LATERAL DEFLECTION

D_{max} = 0.01 in

< L / 240 = 0.65 in **OK**



Frame 11 for SMF Design - Beam Check on AISC Manual 13th Edition (AISC 360-05)

COLUMN SECTION	**W24x335**			r =	3.34	in	Table 1-1
COLUMN YIELD STRESS	F_y =	50.0	ksi	ΦPn =	2770	k	Table 4-1
HEIGHT	L =	12.3	ft				
AXIAL LOAD, Factored, Pu	P =	0.0	kips				

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05)

$$\frac{P_u}{\phi P_n} \ge 0.2 \Rightarrow \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

Pu/ΦPn = 0.00

$$\frac{P_u}{\phi P_n} < 0.2 \Rightarrow \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

0.22 < 1.0 **OK**

Where KL_x =	12.3	ft, for x-x axial load.			
$(KL / r)_{max}$ =	44.3	< 200			
Pu =	0.0	kips			
Mu_x =	948.0	ft-kips	Vu_x =	0.0	ft-kips
Mu_y =	0.0	ft-kips	Vu_y =	247.7	ft-kips
ΦP_c =	2770.0	kips, (AISC 360-05 Chapter E)	Table 4-1		
	>	Pu	**OK**		
ΦMn_x =	3830.0	ft-kips, (AISC 360-05 Chapter F)	Table 3-2		
	>	Mu_x	**OK**		
ΦMn_y =	893.0	ft-kips, (AISC 360-05 Chapter F)	Table 3-4		
	>	Mu_y	**OK**		

CHECK LATERAL DEFLECTION

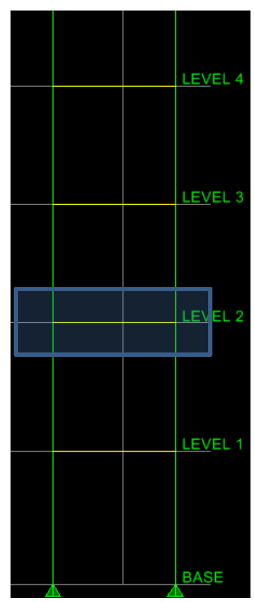
D_{max} = 0.12 in

< L / 240 = 0.62 in **OK**

CHECK SHEAR CAPACITY ABOUT MAJOR AXIS (AISC 360-05 Chapter G2 or G5)

$V_{allowable}$ = ΦV_n = 1140.0 kips Table 3-2

> V_{Max} **OK**



Special Moment Frames Seismic Provisions , AISC 341-05 9.6

Column-Beam Moment Ratio $\left(\frac{\sum M_{pc}}{\sum M_{pb}}\right) > 1.0$ =

	Mx	My
	1.6	3.4
	OK	**OK**

Appendix H –Cost Analysis for Lateral Force Resisting Systems

RS Means 2007

Exisiting Special Reinforced Shear Walls - Concrete Material and Labor Take-Off

Item	Length (ft)	Width (ft)	Height (ft)	Volume (cf)	Add 10% for waste	Material Unit Cost*	Material Cost [27 cf per cy]	Labor Unit Cost ($/CF) **	Labor Cost	Wall Finish Unit Cost (Mataterial and Labor, $/SF)***	Wall Finish Cost	Total Cost
Wall 1	34.00	1.00	62.83	2136.22	2349.84	$108/CY	$ 8,544.88	$ 0.92	$ 536,874.81	$ 0.33	$ 1,409.91	$ 546,829.93
Wall 2	34.00	1.00	69.83	2374.22	2611.64	$108/CY	$ 9,496.88	$ 0.92	$ 663,167.13	$ 0.33	$ 1,566.99	$ 674,231.33
Wall 3	33.00	1.00	69.83	2304.39	2534.83	$108/CY	$ 9,217.56	$ 0.92	$ 643,662.21	$ 0.33	$ 1,520.90	$ 654,401.00
Wall 4	33.00	1.00	62.83	2073.39	2280.73	$108/CY	$ 8,293.56	$ 0.92	$ 521,084.37	$ 0.33	$ 1,368.44	$ 530,746.70
Wall 5	20.00	1.00	69.83	1396.60	1536.26	$108/CY	$ 5,586.40	$ 0.92	$ 390,098.31	$ 0.33	$ 921.76	$ 396,606.80
Wall 6	20.00	1.00	69.83	1396.60	1536.26	$108/CY	$ 5,586.40	$ 0.92	$ 390,098.31	$ 0.33	$ 921.76	$ 396,606.80
Wall 7	10.33	1.00	69.83	721.57	793.73	$108/CY	$ 2,886.30	$ 0.92	$ 201,550.14	$ 0.33	$ 476.24	$ 204,913.01
Wall 8	10.33	1.00	69.83	721.55	793.71	$108/CY	$ 2,886.21	$ 0.92	$ 201,544.29	$ 0.33	$ 476.23	$ 204,907.06
Wall 9	32.00	1.00	71.83	2298.56	2528.42	$108/CY	$ 9,194.24	$ 0.92	$ 660,422.26	$ 0.33	$ 1,517.05	$ 671,133.88
Wall 10	32.00	1.00	71.83	2298.56	2528.42	$108/CY	$ 9,194.24	$ 0.92	$ 660,422.26	$ 0.33	$ 1,517.05	$ 671,133.88
Wall 11	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 12	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 13	20.00	2.00	69.83	2793.20	3072.52	$108/CY	$ 11,172.80	$ 0.92	$ 780,196.62	$ 0.33	$ 921.76	$ 792,291.51
											Total Concrete Cost	**$5,726,922**

* Normal Weight Concrete, Ready Mix 4000psi (Agilia, Self Consolidating Concrete)

** Placing of concrete (Walls, pumped) for Labor and Equipment

*** Wall finish

RS Means 2007 COSTS - EXISTING SPECIAL REINFORCED SHEAR WALLS REBAR

Type	Thickness (in)	Vertical Spacing (in)	Vertical Bar Size	Vertical Bar Diameter (in)	Vertical Bar Weight (plf)	Horizontal Spacing (in)	Horizontal Bar Size	Horizontal Bar Diameter (in)	Bar Weight (plf)
a	12	12	6	0.75	1.502	12	6	0.75	1.502
b	12	6	11	1.41	5.313	8	8	1	2.67
c	12	12	6	0.75	1.502	12	6	0.75	1.502
d	12	8	11	1.41	5.313	8	8	1	2.67
e	18	12	6	0.75	1.502	12	6	0.75	1.502
f	18	6	11	1.41	5.313	8	8	1	2.67
g	18	6	11	1.41	5.313	8	8	1	2.67
h	18	6	11	1.41	5.313	8	8	1	2.67

WALL	Height (ft)	# Veritical bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length (ft)	Total weight (pounds)	Add 10% for waste and lap	Length (ft)	# Horizontal bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length	Total weight (pounds)	Add 10% for waste and lap
Wall 1_a	24.33	24.33	25	23.83	603.61	906.63	997.29	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 1_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 2_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 2_b	38.50	77.00	78	38.00	2964.00	4179.24	4597.16	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 3_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 3_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 4_a	24.33	24.33	25	23.83	603.61	906.63	997.29	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 4_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 5_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 5_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 6_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 6_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 7_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 7_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.24	433.18	476.50
Wall 8_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 8_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.15	432.93	476.22
Wall 9_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 9_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 10_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 10_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 11_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.77	236.97	260.67
Wall 11_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.74	616.07	677.68
Wall 12_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.78	236.98	260.68
Wall 12_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.75	616.10	677.71
Wall 13_g	31.33	62.66	64	30.83	1962.64	10427.49	11470.24	21.17	31.76	33	20.67	677.05	1016.92	1118.62
Wall 13_h	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	21.17	31.76	33	20.67	677.05	1807.71	1988.48

Existing Special Reinforced Shear Walls - Reinforcing Bars Material and Labor Cost

ITEM	Daily Output (tons)	Labor (Hours)	Material Cost	Labor Cost	Work Hours/ton	Material (tons)	Material Cost	Labor Cost
Walls, Rebar #3 to #7	3	10.67	$850/ton	$440/ton	3.56	16.59	$ 14,105.68	$ 7,301.76
Walls, Rebar #8 to #18	4	8.00	$850/ton	$330/ton	2.00	107.30	$ 91,205.32	$ 35,409.13
						Total Reinforcing Bar Cost		$ 148,021.89

RS Means 2007

Modified Special Reinforced Shear Walls - Concrete Material and Labor Take-Off

Item	Length (ft)	Width (ft)	Height (ft)	Volume (cf)	Add 10% for waste	Material Unit Cost*	Material Cost [27 cf per cy]	Labor Unit Cost ($/CF) **	Labor Cost	Wall Finish Unit Cost (Mataterial and Labor, $/SF)***	Wall Finish Cost	Total Cost
Wall 1	20.00	1.50	62.83	1884.90	2073.39	$108/CY	$ 7,539.60	$ 0.92	$ 473,713.07	$ 0.33	$ 829.36	$ 482,082.35
Wall 2	34.00	1.50	69.83	3561.33	3917.46	$108/CY	$ 14,245.32	$ 0.92	$ 994,750.70	$ 0.33	$ 1,566.99	$1,010,563.33
Wall 5	20.00	1.50	69.83	2094.90	2304.39	$108/CY	$ 8,379.60	$ 0.92	$ 585,147.47	$ 0.33	$ 921.76	$ 594,449.15
Wall 6	20.00	1.50	69.83	2094.90	2304.39	$108/CY	$ 8,379.60	$ 0.92	$ 585,147.47	$ 0.33	$ 921.76	$ 594,449.15
Wall 7	10.33	1.00	69.83	721.57	793.73	$108/CY	$ 2,886.30	$ 0.92	$ 201,550.14	$ 0.33	$ 476.24	$ 204,913.01
Wall 8	10.33	1.00	69.83	721.55	793.71	$108/CY	$ 2,886.21	$ 0.92	$ 201,544.29	$ 0.33	$ 476.23	$ 204,907.06
Wall 9	34.00	1.00	71.83	2442.22	2686.44	$108/CY	$ 9,768.88	$ 0.92	$ 701,698.65	$ 0.33	$ 1,611.87	$ 713,079.73
Wall 10	34.00	1.00	71.83	2442.22	2686.44	$108/CY	$ 9,768.88	$ 0.92	$ 701,698.65	$ 0.33	$ 1,611.87	$ 713,079.73
Wall 11	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 12	12.33	1.50	71.83	1328.85	1461.74	$108/CY	$ 5,315.41	$ 0.92	$ 381,805.59	$ 0.33	$ 584.69	$ 387,706.02
Wall 13	20.00	2.00	69.83	2793.20	3072.52	$108/CY	$ 11,172.80	$ 0.92	$ 780,196.62	$ 0.33	$ 921.76	$ 792,291.51
											Total Concrete Cost	**$5,292,936**

* Normal Weight Concrete, Ready Mix 4000psi (Agilia, Self Consolidating Concrete)

** Placing of concrete (Walls, pumped) for Labor and Equipment

*** Wall finish

****Agilia, Self-Consolidating Concrete from Lafarge Conrete

RS Means 2007 COSTS - MODIFIED SPECIAL REINFORCED SHEAR WALLS REBAR

Type	Thickness (in)	Vertical Spacing (in)	Vertical Bar Size	Vertical Bar Diameter (in)	Vertical Bar Weight (plf)	Horizontal Spacing (in)	Horizontal Bar Size	Horizontal Bar Diameter (in)	Horizontal Bar Weight
a	12	12	6	0.75	1.502	12	6	0.75	1.502
b	12	6	11	1.41	5.313	8	8	1	2.67
c	12	12	6	0.75	1.502	12	6	0.75	1.502
d	12	8	11	1.41	5.313	8	8	1	2.67
e	18	12	6	0.75	1.502	12	6	0.75	1.502
f	18	6	11	1.41	5.313	8	8	1	2.67
g	24	6	11	1.41	5.313	8	8	1	2.67
h	24	6	11	1.41	5.313	8	8	1	2.67

WALL	Height (ft)	# Veritical bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length (ft)	Total weight (pounds)	Add 10% for waste and lap	Length (ft)	# Horizontal bar spaces	# bars	Bar Length (ft) 3" cover	Total bar length	Total weight (pounds)	Add 10% for waste and lap
Wall 1_f	24.33	48.66	50	23.83	1183.40	1777.46	1955.21	34.00	51.00	52	33.50	1742.00	2616.48	2878.13
Wall 1_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 2_f	31.33	62.66	64	30.83	1962.64	2947.88	3242.67	34.00	51.00	52	33.50	1742.00	2616.48	2878.13
Wall 2_f	38.50	77.00	78	38.00	2964.00	4179.24	4597.16	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 5_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 5_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 6_a	31.33	31.33	32	30.83	996.73	1497.09	1646.80	20.00	20.00	21	19.50	409.50	615.07	676.58
Wall 6_b	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	20.00	30.00	31	19.50	604.50	1614.02	1775.42
Wall 7_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 7_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.24	433.18	476.50
Wall 8_c	31.33	31.33	32	30.83	996.73	1497.09	1646.80	10.33	10.33	11	9.83	111.44	167.38	184.12
Wall 8_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	10.33	15.50	16	9.83	162.15	432.93	476.22
Wall 9_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 9_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 10_c	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	34.00	34.00	35	33.50	1172.50	1761.10	1937.20
Wall 10_d	38.50	57.75	59	38.00	2232.50	11861.27	13047.40	34.00	51.00	52	33.50	1742.00	4651.14	5116.25
Wall 11_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.77	236.97	260.67
Wall 11_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.74	616.07	677.68
Wall 12_e	33.33	33.33	34	32.83	1127.05	1692.83	1862.12	12.33	12.33	13	11.83	157.78	236.98	260.68
Wall 12_f	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	12.33	18.50	19	11.83	230.75	616.10	677.71
Wall 13_g	31.33	62.66	64	30.83	1962.64	10427.49	11470.24	21.17	31.76	33	20.67	677.05	1016.92	1118.62
Wall 13_h	38.50	77.00	78	38.00	2964.00	15747.73	17322.51	21.17	31.76	33	20.67	677.05	1807.71	1988.48

Modified Special Reinforced Shear Walls - Reinforcing Bars Material and Labor Cost

ITEM	Daily Output (tons)	Labor (Hours)	Material Cost	Labor Cost	Work Hours/ton	Material (tons)	Material Cost	Labor Cost
Walls, Rebar #3 to #7	3	10.67	$850/ton	$440/ton	3.56	15.55	$ 13,220.46	$ 6,843.53
Walls, Rebar #8 to #18	4	8.00	$850/ton	$330/ton	2.00	84.86	$ 72,132.38	$ 28,004.34
						Total Reinforcing Bar Cost		$ 120,200.71

RS MEANS 2007 COSTS - SPECIAL MOMENT FRAMES				
FRAME	LEVEL	BEAM	LENGTH (ft)	Weight (lb)
1	LR	W24x370	34	62900.0
2	HR	W24x370	34	75480.0
3	HR	W24x370	33	73260.0
5	HR	W24x370	20	44400.0
6	HR	W24x370	20	44400.0
7	HR	W24x335	10.333	20769.3
8	HR	W18x158	10.333	9795.7
9	STAIR3	W18x158	32	30336.0
10	STAIR3	W18x158	32	30336.0
11	STAIR3	W18x158	12.333	11691.7
12	STAIR3	W18x175	12.333	12949.7
15	HR	W24x370	30	66600.0
16	HR	W24x370	23.667	52540.7
17	HR	W24x370	23.667	52540.7
			Total Weight (tons)	294

RS MEANS 2007 COSTS - SPECIAL MOMENT FRAMES			
Grid Line	Column	LENGTH (ft)	Weight (lb)
8-A	W14x730	71.83	52435.9
9-A	W14x730	71.83	52435.9
8-SW10	W14x730	71.83	52435.9
9-SW10	W14x730	71.83	52435.9
8-B	W14x730	69.93	51048.9
8.8-B	W14x730	69.93	51048.9
8-C	W14x730	69.93	51048.9
8.8-C	W14x730	69.93	51048.9
3.1-SW1	W14x730	62.83	45865.9
3.1-C	W14x730	62.83	45865.9
1.9-SW1	W14x730	69.93	51048.9
1.9-C	W14x730	69.93	51048.9
1.9-SW8	W14x730	69.93	51048.9
1.9-D2	W14x730	69.93	51048.9
SW5-SW8	W14x730	69.93	51048.9
SW5-D2	W14x730	69.93	51048.9
		Total (tons) =	406

Special Moment Frame					
Iteam	Tonnage of Steel	Material($/ton)	Labor ($/ton)	Equipment ($/ton)	Total
Beams	294	2050	225	115	$ 702,660
Columns	406	2050	225	115	$ 970,297

Connection Fabrication				
#MF's	# of Connections	Fabrication Time (hrs)	Cost ($/Fabr.hr)	Total
15	95	4.8 Ea.	45	$ 20,520

Connection Installation			
Installation Time (days)	Installation Time (hrs)	Cost ($/Labor hr)	Total
15	95	4.8 Ea.	$ 3,240

Appendix I – Mechanical Breadth Cost Analysis

Electricity Costs (PNM)		Summer	Winter
Utility		June-Aug	Sept-May
Electricity consumption/KW		$9.56	$8.19
Electricity demand per month/KWh		$0.0821025	$0.064170

Gas Costs (New Mexico Gas Company)	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
Gas distribution/ therm	$ 0.49100	$ 0.53090	$ 0.47790	$0.50080	$ 0.46870	$ 0.52350	$ 0.56990	$ 0.53470	$ 0.49170	$ 0.53750	$ 0.49010	$ 0.45170

VRE 3-54 Glazing

Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,051	1,819	2,170	1,955	3,576	4,815	5,946	4,755	3,458	2,271	2,022	1,981	
Consumption ($)	$ 131.61	$ 116.72	$ 139.25	$ 125.45	$ 229.47	$ 395.32	$ 488.18	$ 390.40	$ 221.90	$ 145.73	$ 129.75	$ 127.12	
Peak(KW)	10	11	11	11	17	19	19	17	16	11	11	10	
Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$90.09	$90.09	$81.90	
Gas(therms)	1217	672	498	60	1	0	0	0	1	71	629	1041	
Gas Dist ($)	$597.55	$356.76	$237.99	$30.05	$0.47	$0.00	$0.00	$0.00	$0.49	$38.16	$308.27	$470.22	
Total Elec. Cons ($)	$ 131.61	$ 116.72	$ 139.25	$ 125.45	$ 229.47	$ 395.32	$ 488.18	$ 390.40	$ 221.90	$ 145.73	$ 129.75	$ 127.12	
Total Elec. Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$90.09	$90.09	$81.90	
Total gas dist ($)	$597.55	$356.76	$237.99	$30.05	$0.47	$0.00	$0.00	$0.00	$0.49	$38.16	$308.27	$470.22	TOTAL
Total Elect. Costs	$ 213.51	$ 206.81	$ 229.34	$ 215.54	$ 368.70	$ 576.96	$ 669.82	$ 552.92	$ 352.94	$ 235.82	$ 219.84	$ 209.02	**$ 4,051.23**
Total Gas Costs	$597.55	$356.76	$237.99	$30.05	$0.47	$0.00	$0.00	$0.00	$0.49	$38.16	$308.27	$470.22	**$ 2,039.97**
Total	$ 811.06	$ 563.58	$ 467.33	$ 245.59	$ 369.17	$ 576.96	$ 669.82	$ 552.92	$ 353.43	$ 273.98	$ 528.11	$ 679.24	**$ 6,091.20**

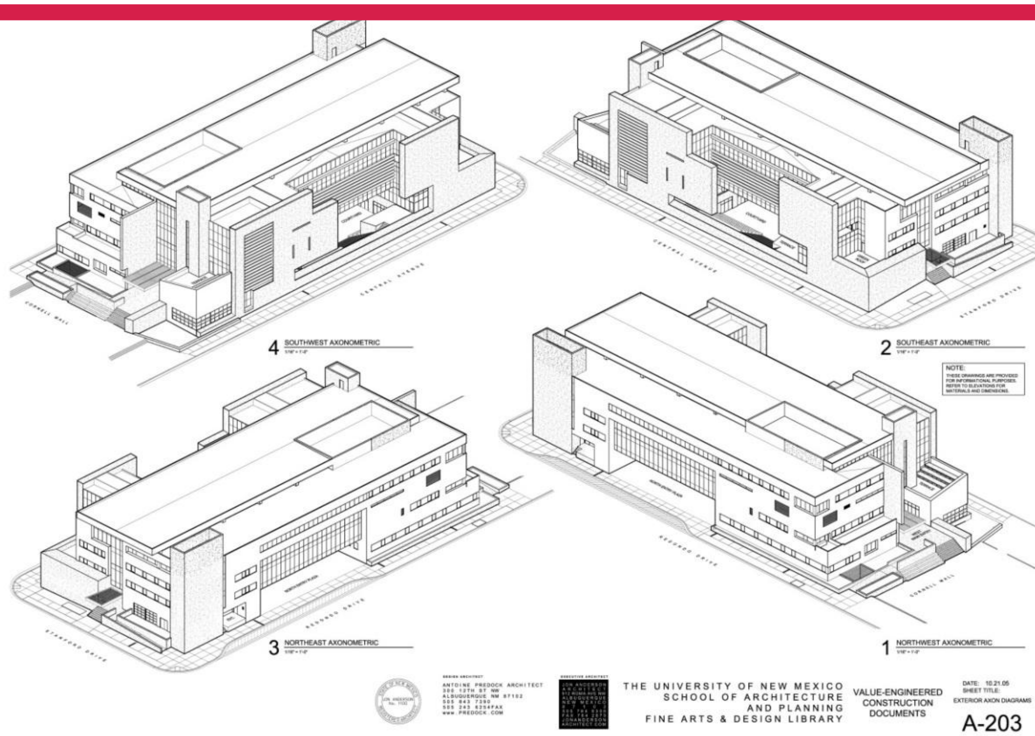
VNE 1-30 Glazing

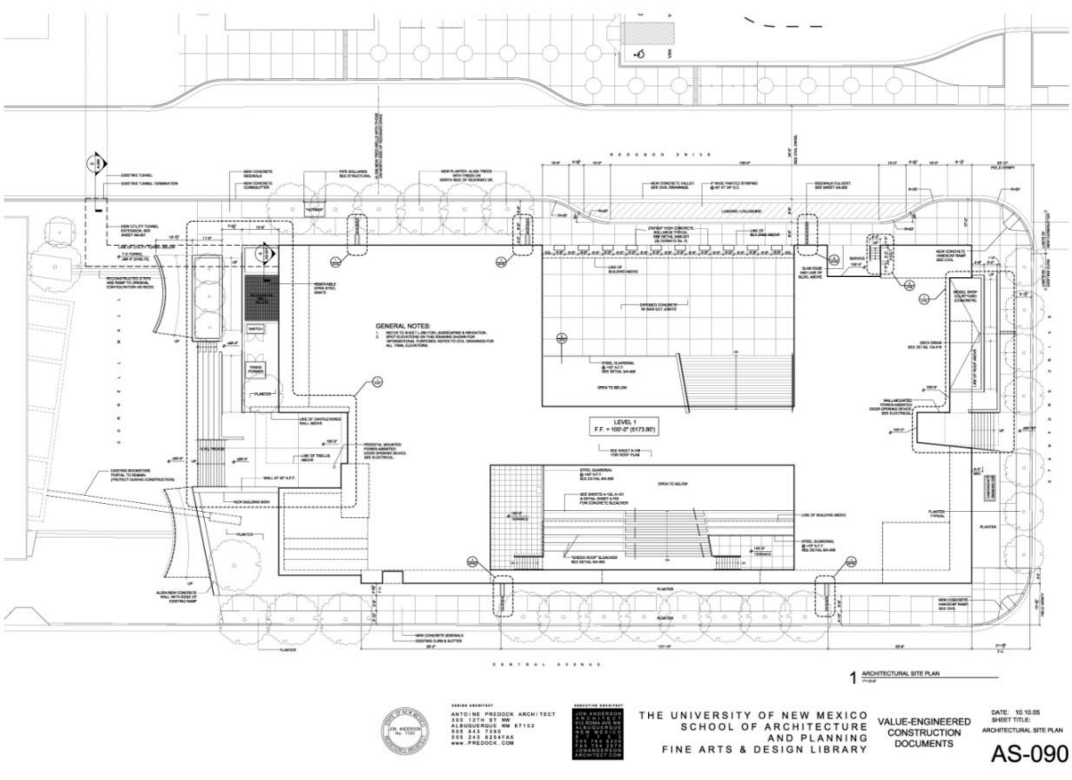
Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,037	1,805	2,152	1,973	3,618	4,776	5,936	4,751	3,476	2,280	2,011	1,963	
Consumption ($)	$ 130.71	$ 115.83	$ 138.09	$ 126.61	$ 232.17	$ 392.12	$ 487.36	$ 390.07	$ 223.05	$ 146.31	$ 129.05	$ 125.96	
Peak(KW)	10	11	11	11	17	19	19	17	16	12	11	10	
Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$98.28	$90.09	$81.90	
Gas(therms)	1139	623	451	50	1	0	0	0	1	61	586	974	
Gas Dist ($)	$559.25	$330.75	$215.53	$25.04	$0.47	$0.00	$0.00	$0.00	$0.49	$32.79	$287.20	$439.96	
Total Elec. Cons ($)	$ 130.71	$ 115.83	$ 138.09	$ 126.61	$ 232.17	$ 392.12	$ 487.36	$ 390.07	$ 223.05	$ 146.31	$ 129.05	$ 125.96	
Total Elec. Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$181.64	$162.52	$131.04	$98.28	$90.09	$81.90	
Total gas dist ($)	$559.25	$330.75	$215.53	$25.04	$0.47	$0.00	$0.00	$0.00	$0.49	$32.79	$287.20	$439.96	TOTAL
Total Elect. Costs	$ 212.61	$ 205.92	$ 228.18	$ 216.70	$ 371.40	$ 573.76	$ 669.00	$ 552.59	$ 354.09	$ 244.59	$ 219.14	$ 207.86	**$ 4,055.84**
Total Gas Costs	$559.25	$330.75	$215.53	$25.04	$0.47	$0.00	$0.00	$0.00	$0.49	$32.79	$287.20	$439.96	**$ 1,891.47**
Total	$ 771.86	$ 536.67	$ 443.72	$ 241.74	$ 371.86	$ 573.76	$ 669.00	$ 552.59	$ 354.59	$ 277.37	$ 506.33	$ 647.82	**$ 5,947.31**

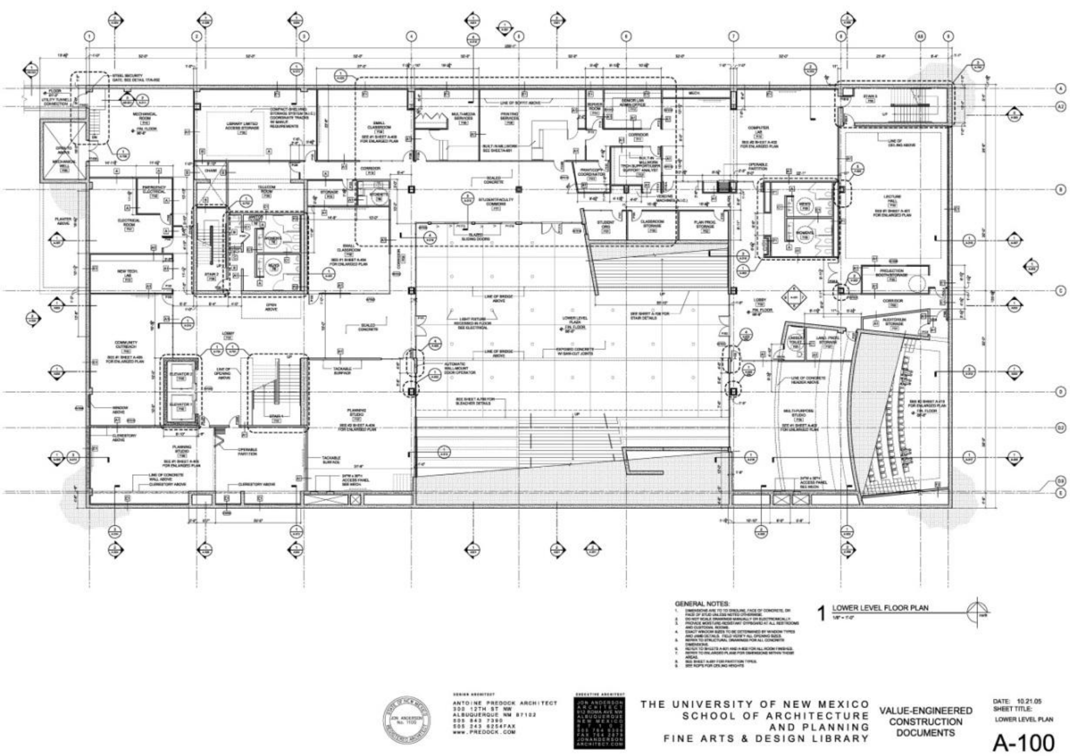
VRE1-63 Glazing

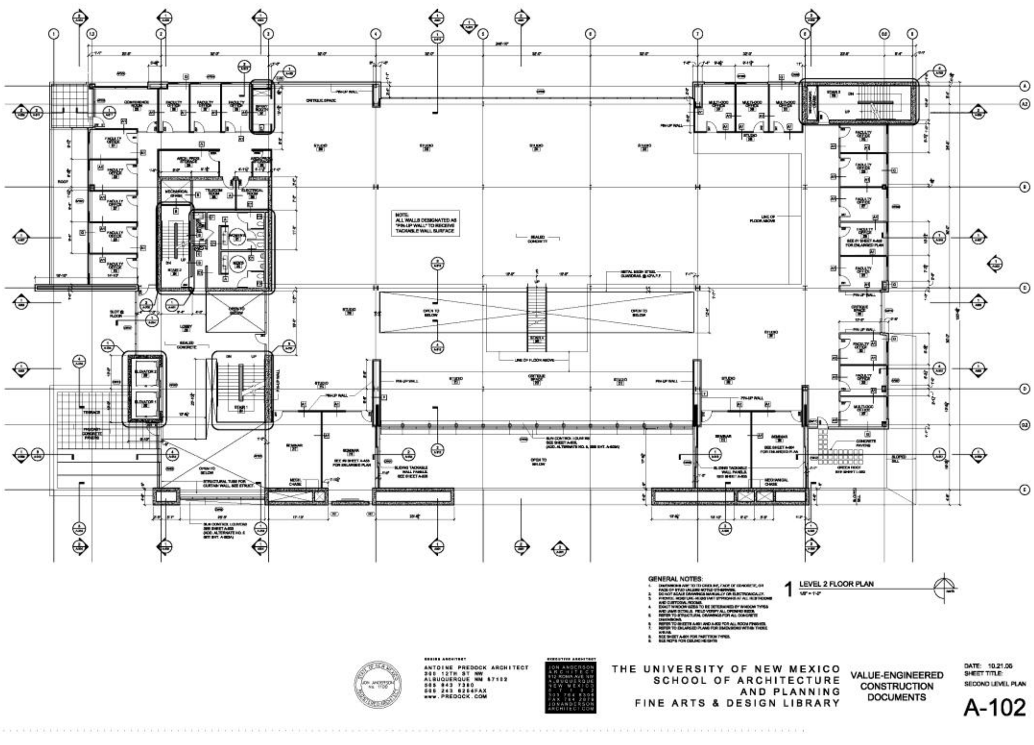
Monthly Costs	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Elec(KWh)	2,014	1,799	2,143	1,970	3,643	4,774	5,906	4,749	3,500	2,292	1,999	1,940	
Consumption ($)	$ 129.24	$ 115.44	$ 137.52	$ 126.41	$ 233.77	$ 391.96	$ 484.90	$ 389.90	$ 224.59	$ 147.08	$ 128.28	$ 124.49	
Peak(KW)	10	11	11	11	17	19	18	17	16	12	11	10	
Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$172.08	$162.52	$131.04	$98.28	$90.09	$81.90	
Gas(therms)	1082	587	418	42	1	0	0	0	1	53	555	925	
Gas Dist ($)	$531.26	$311.64	$199.76	$21.03	$0.47	$0.00	$0.00	$0.00	$0.49	$28.49	$272.01	$417.82	
Total Elec. Cons ($)	$ 129.24	$ 115.44	$ 137.52	$ 126.41	$ 233.77	$ 391.96	$ 484.90	$ 389.90	$ 224.59	$ 147.08	$ 128.28	$ 124.49	
Total Elec. Demand ($)	$81.90	$90.09	$90.09	$90.09	$139.23	$181.64	$172.08	$162.52	$131.04	$98.28	$90.09	$81.90	
Total gas dist ($)	$531.26	$311.64	$199.76	$21.03	$0.47	$0.00	$0.00	$0.00	$0.49	$28.49	$272.01	$417.82	TOTAL
Total Elect. Costs	$ 211.14	$ 205.53	$ 227.61	$ 216.50	$ 373.00	$ 573.60	$ 656.98	$ 552.42	$ 355.63	$ 245.36	$ 218.37	$ 206.39	**$ 4,042.52**
Total Gas Costs	$531.26	$311.64	$199.76	$21.03	$0.47	$0.00	$0.00	$0.00	$0.49	$28.49	$272.01	$417.82	**$ 1,782.97**
Total	$ 742.40	$ 517.17	$ 427.37	$ 237.54	$ 373.47	$ 573.60	$ 656.98	$ 552.42	$ 356.13	$ 273.84	$ 490.37	$ 624.21	**$ 5,825.49**

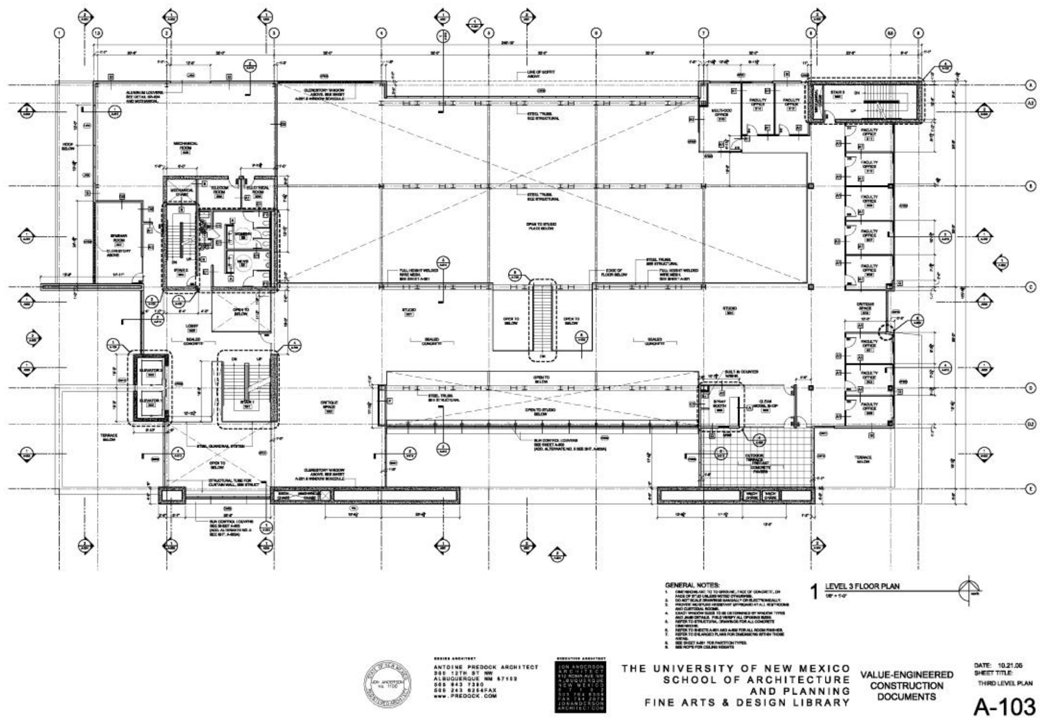
Appendix J –Construction Documents

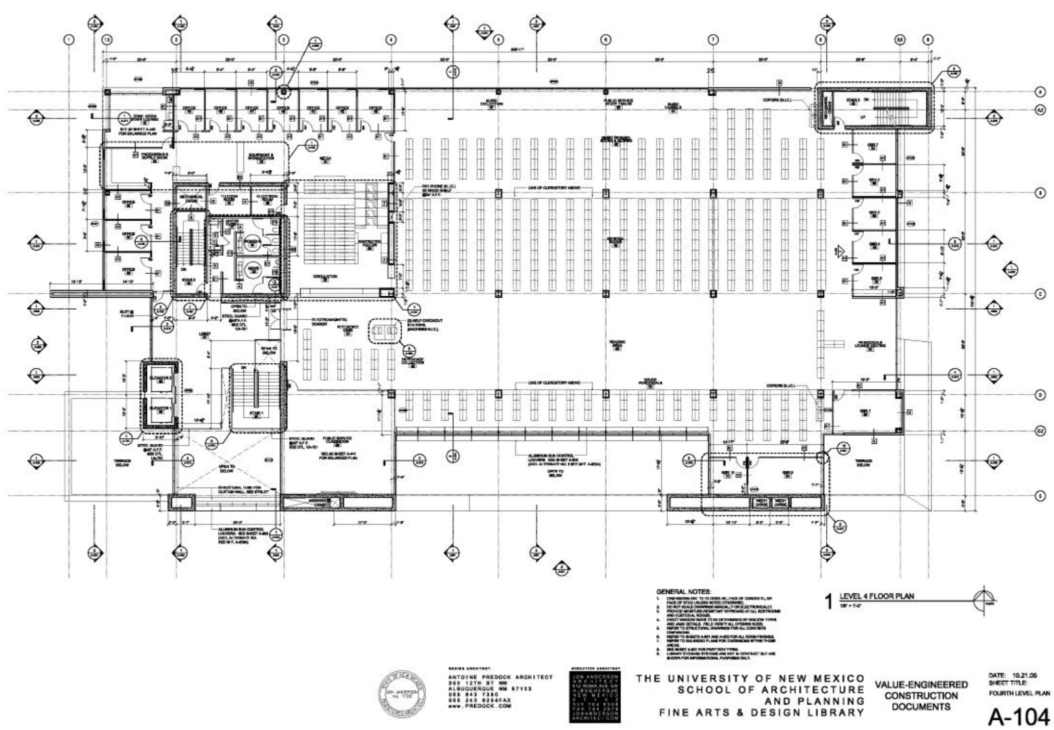


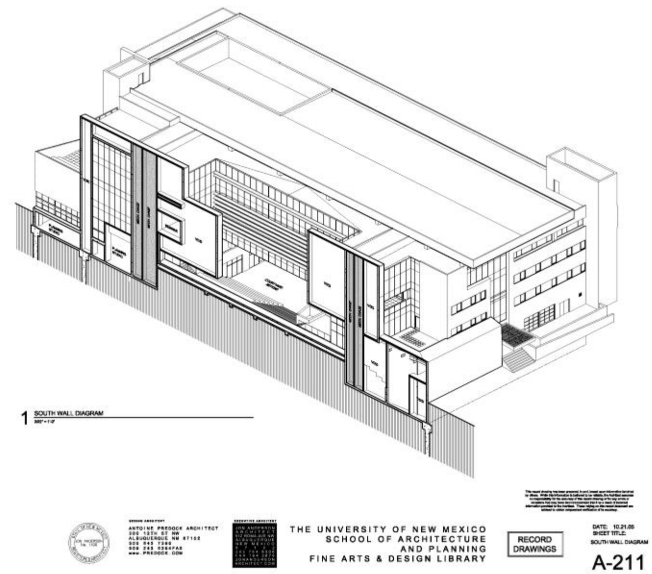


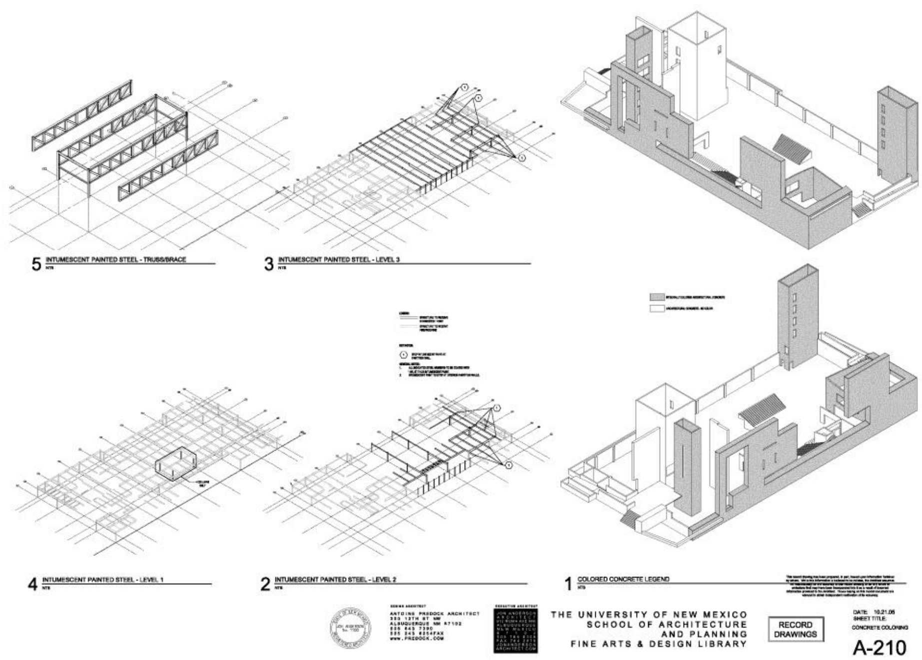

1 LOWER LEVEL FLOOR PLAN
GENERAL NOTES:
ANTOINE PREDOCK ARCHITECT
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-100

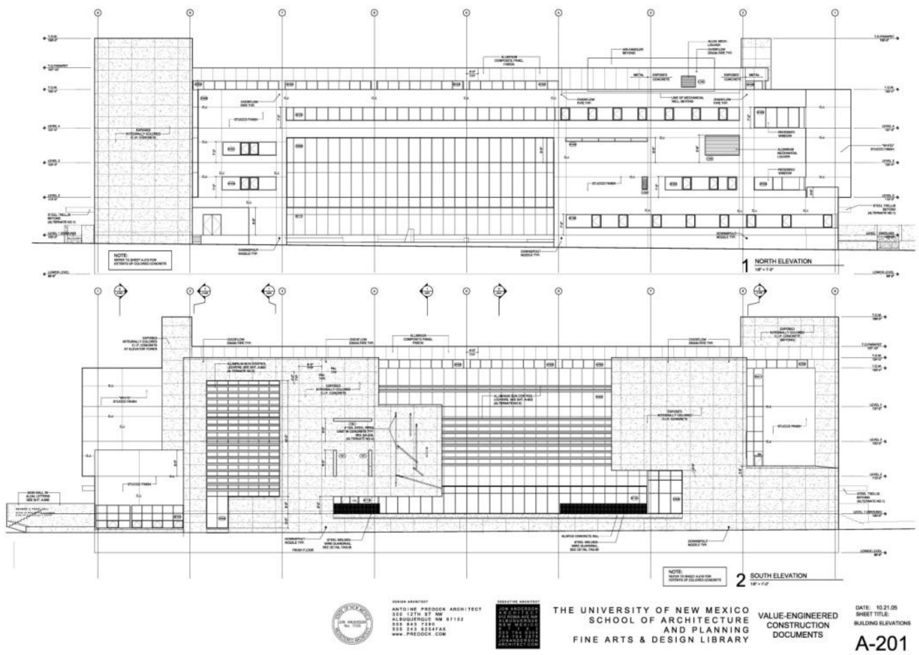

1 LEVEL 2 FLOOR PLAN
GENERAL NOTES:
ANTOINE PREDOCK ARCHITECT
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-102

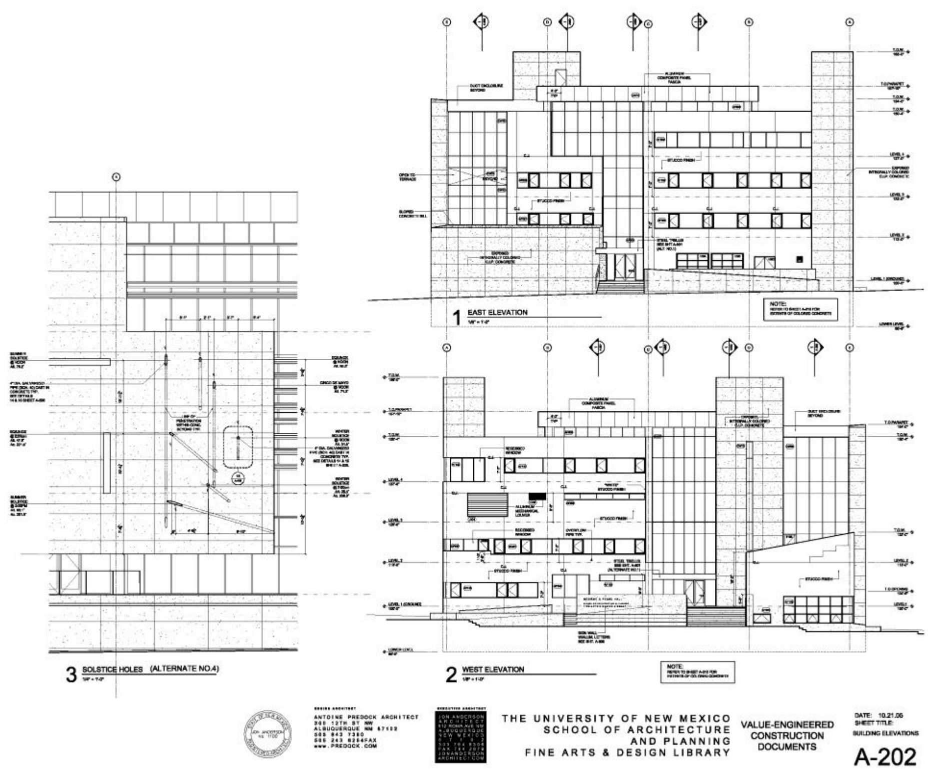

GENERAL NOTES:
1 LEVEL 3 FLOOR PLAN
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
THIRD LEVEL PLAN
A-103

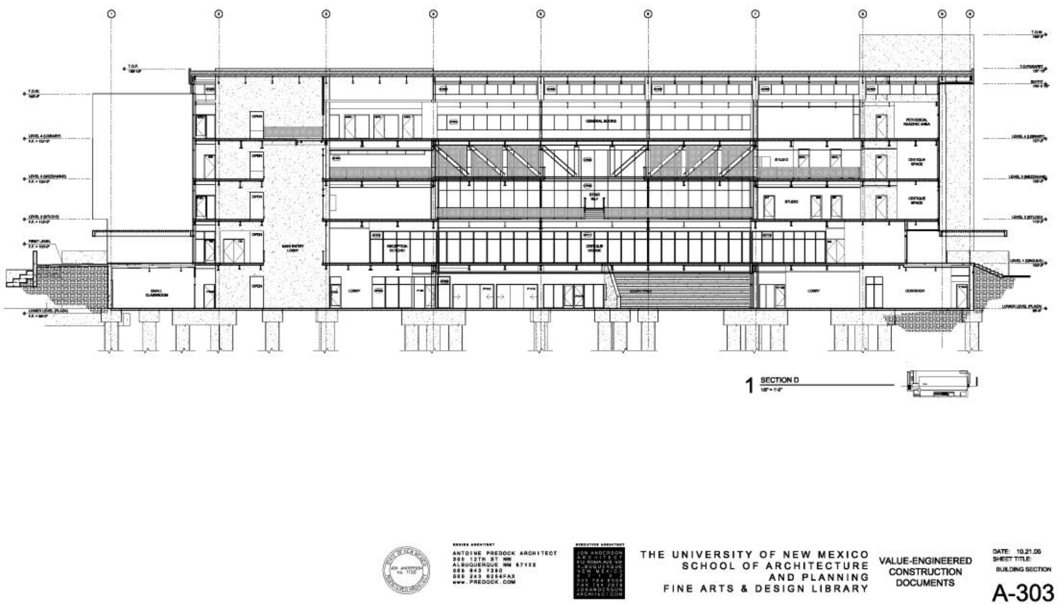

GENERAL NOTES:
1 LEVEL 4 FLOOR PLAN
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
FOURTH LEVEL PLAN
A-104

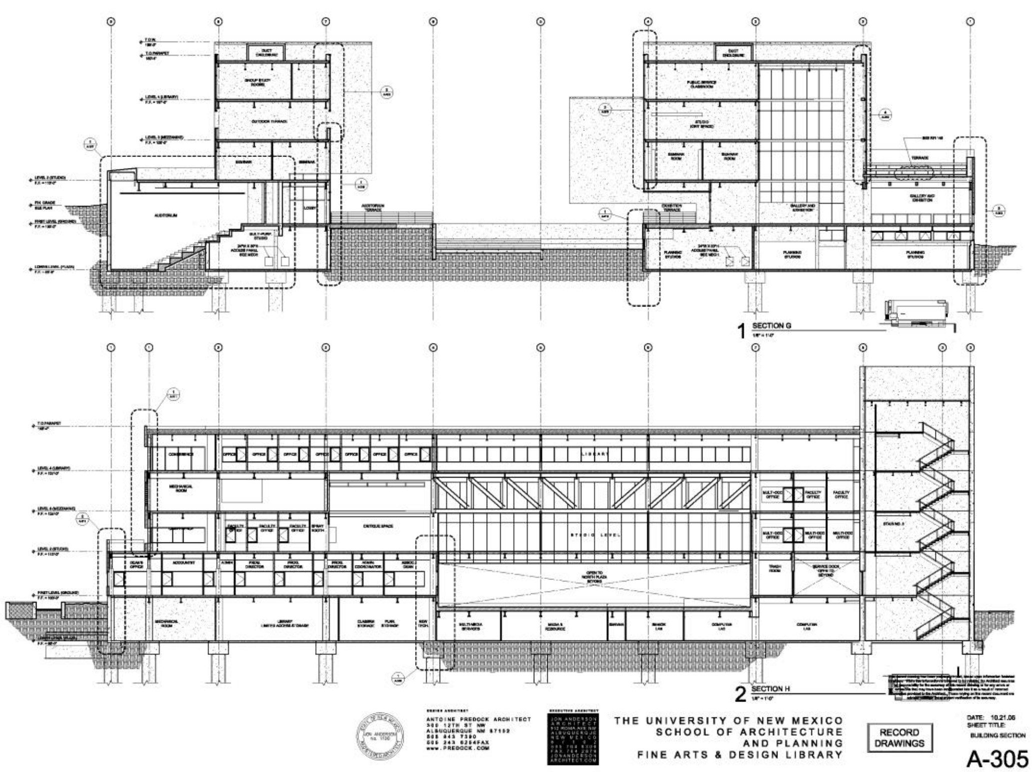

1 SOUTH WALL DIAGRAM
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
RECORD DRAWINGS
SOUTH WALL DIAGRAM
A-211


5 INTUMESCENT PAINTED STEEL - TRUSS/BRACE
3 INTUMESCENT PAINTED STEEL - LEVEL 3
4 INTUMESCENT PAINTED STEEL - LEVEL 1
2 INTUMESCENT PAINTED STEEL - LEVEL 2
1 COLORED CONCRETE LEGEND
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
RECORD DRAWINGS
A-210


1 NORTH ELEVATION
2 SOUTH ELEVATION
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-201


1 EAST ELEVATION
3 SOLSTICE HOLES (ALTERNATE NO.4)
2 WEST ELEVATION
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-202


1 SECTION D
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
VALUE-ENGINEERED
CONSTRUCTION
DOCUMENTS
A-303


1 SECTION G
2 SECTION H
THE UNIVERSITY OF NEW MEXICO
SCHOOL OF ARCHITECTURE
AND PLANNING
FINE ARTS & DESIGN LIBRARY
RECORD
DRAWINGS
A-305