Executive Summary

In this report, a detailed analysis of the lateral forces was performed in efforts to examine and test the lateral resisting framing system for the Executive Tower. This report is divided into three sections. Section one is a second look at the wind and seismic forces initially examined in technical report 1. The wind and seismic were checked, recalculated and compared using charts to find which scenarios govern over the other. The second section explains the structural systems in place and how they coordinate with each other. It was assumed, the besides the shear being the main lateral resisting system, that a study of six moment frames creating the perimeter beam and exterior columns of the building would be beneficial in studying the building. Finally a detail analysis of the frames and shear walls were preformed to check stresses, flexural and drift through the use of STAAD and RAM analysis programs.

This technical report looks in depth lateral systems and the drifts that are resulted from the combined lateral and gravity loads. Hand calculations of spot checks at critically stress locations found all columns to be able to withstand the combined loads. The check of the shears at the worse case scenario found the wall sections and reinforcement to be adequate. From the technical report, the lateral forces for the Executive Tower were checked and corrected to still be used in future examinations.

Introduction

Executive Tower is a 12 story structure consisting of primarily cast-in-place concrete. The framing system is an 8" two-way concrete flat slab with 8" drop panels at column locations and an additional 3 ½" thickened slab acting as an edge for the entire perimeter of the building.

Report Layout

- Lateral load cases and controlling conditions
- Existing lateral systems; shears walls, and moment frames
- Distribution of later loads to resisting system
- Analysis of lateral structures
- Appendix A – Seismic and Wind Calculations
- Appendix B – Load Distribution
- Appendix C – Story Drift
- Appendix D – Column Spot Check
- Appendix E – Shear Wall Spot Check

Loads

All loads are in accordance ASCE7-02 and assumes a 20 psf addition live load be assumed for partitions.

Live Loads	
Office	80 psf + 20 psf = 100 psf
Dead Loads	
Concrete	124 psf
Curtain	12 psf
Sprinkler	5 psf
MEP	5 psf
Finishes	10 psf

Lateral Load Summary

Wind

The wind analysis were taken from Technical Assignment 1 and checked to be properly used in this report using ASCE 7-02. Executive Tower is set in the middle of downtown Washington DC with several buildings of similar height near by. It was for this reason an Expose B and case 2 were used to calculate the wind pressures applied to the Executive Tower. The k_z values were found in Table 6-3 of ASCE 7-02 but were calculated using an excel spreadsheet for the mid-floor intervals using eq. 2 on Table 6-3, $k_z = 2.01(z/z_g)^{2/ά}$. In the chart below the new heights for Z can be seen along with the calculated pressures for the N-S and E-W directions. The extent of the wind analysis including Gust Factor calculations can be found in Appendix A.

z (ft)	k_z	q_z	N-S P_L (psf)	N-S P_w (psf)	N-S P_L+P_w (psf)	E-W P_L (psf)	E-W P_w (psf)	E-W P_L+P_w (psf)
153.3	1.117	19.680	-4.97	13.10	18.07	-8.39	13.27	21.66
144.1	1.097	19.335	-4.97	12.87	17.84	-8.39	13.04	21.43
128.6	1.062	18.716	-4.97	12.46	17.43	-8.39	12.62	21.01
116.6	1.033	18.199	-4.97	12.11	17.08	-8.39	12.27	20.66
105.1	1.002	17.667	-4.97	11.76	16.73	-8.39	11.91	20.30
93.58	0.970	17.091	-4.97	11.38	16.35	-8.39	11.53	19.92
82.08	0.934	16.463	-4.97	10.96	15.93	-8.39	11.10	19.49
70.58	0.895	15.768	-4.97	10.49	15.46	-8.39	10.63	19.02
59.08	0.850	14.986	-4.97	9.98	14.95	-8.39	10.11	18.50
47.58	0.799	14.088	-4.97	9.38	14.35	-8.39	9.50	17.89
36.08	0.739	13.017	-4.97	8.66	13.63	-8.39	8.78	17.17
24.58	0.662	11.665	-4.97	7.76	12.73	-8.39	7.87	16.26
9.42	0.503	8.869	-4.97	5.90	10.87	-8.39	5.98	14.37

Seismic

Like the wind analysis, the seismic forces for this report were taken from Technical Assignment 1 and recheck for accurate results. For this report, the seismic loads were found to be less than in Tech 1. The seismic loads are in accordance with ASCE 7-02. An in-depth calculation of the structures dead load for the seismic calculation is displayed in Appendix B along with hand calculations. Including the drop panels and edge beams spread over the total area of the floor. A full description of the story forces can be seen below calculated using a spreadsheet.

SEISMIC	ht (ft)	load (k)	W^*ht^k	C_{vx}		story force (k) =VCs
roof	153.33	561	218788	0.0562		26.60
pent.	134.83	1903	637237	0.1638		77.47
11	122.33	1903	567818	0.1460		69.03
10	110.83	1903	505103	0.1299		61.40
9	99.33	1903	443586	0.1140		53.93
8	87.83	1903	383378	0.0986		46.61
7	76.33	1903	324619	0.0835		39.46
6	64.83	1903	267485	0.0688		32.52
5	53.33	1903	212209	0.0546		25.80
4	41.83	1903	159116	0.0409		19.34
3	30.33	1903	108692	0.0279		13.21
2	18.83	1903	61769	0.0159		7.51
		21494	3889800			472.87

Lateral Comparisons

The story forces and wind pressures were converted into the linear pressure across the length of the building to compare which forces will govern in which directions. It was found the seismic loads controlled in the upper floors while the wind governing factors the lower floors.

Wind		Seismic		Governing		Story Forces	
E-W (pLf)	N-S (pLf)	E-W (pLf)	N-S (pLf)	E-W (pLf)	N-S (pLf)	E-W (kips)	N-S (kips)
199.3	166.2	183.4	289.1	199.3	289.1	28.9	26.6
332.2	276.5	534.3	842.0	534.3	842.0	77.5	77.5
252.1	209.1	476.1	750.3	476.1	750.3	69.0	69.0
237.6	196.5	423.5	667.4	423.5	667.4	61.4	61.4
233.9	192.7	371.9	586.1	371.9	586.1	53.9	53.9
229.0	188.0	321.4	506.6	321.4	506.6	46.6	46.6
224.2	183.2	272.2	428.9	272.2	428.9	39.5	39.5
218.8	177.8	224.3	353.5	224.3	353.5	32.5	32.5
212.7	171.9	177.9	280.4	212.7	280.4	30.8	25.8
205.7	165.0	133.4	210.3	205.7	210.3	29.8	19.3
197.4	156.8	91.1	143.6	197.4	156.8	28.6	14.4
246.5	193.1	51.8	81.6	246.5	193.1	35.7	17.8

Lateral Resisting System

Executive Tower is a twelve story office building comprised of a concrete flat slab with drop panels. The lateral resisting system consists of two components. The first is six interacting shear walls developing the elevator core in the center of the building. The shear walls are fully reinforced with #6@8” O.C. on each face and in both the vertical and horizontal directions. In the East-West direction, wall 1, 2, 3, 4 form the sides of the elevator with typical lengths of 9 ½’ and 8 ½’. Walls 5 and 6 make up the backs of the elevator and are of length 19 ½’ and 21’ supporting the North-South direction. Along with shear wall extending the full height of the building, Executive Tower’s flat slab system contains exterior columns of 20”x20” with edge beams of 11½”x5’-4”around the perimeter of the building acting as a second lateral resisting system. An edge beam of 11 ½” thick by 5 feet wide create a moment frames around the entire building aiding in torsional support. The plan view below, Figure 1, shows the location of the shear wall and the frames. The perimeter beams were divided into 5 moment frames that insist the shear walls in resisting lateral and torsional motion.

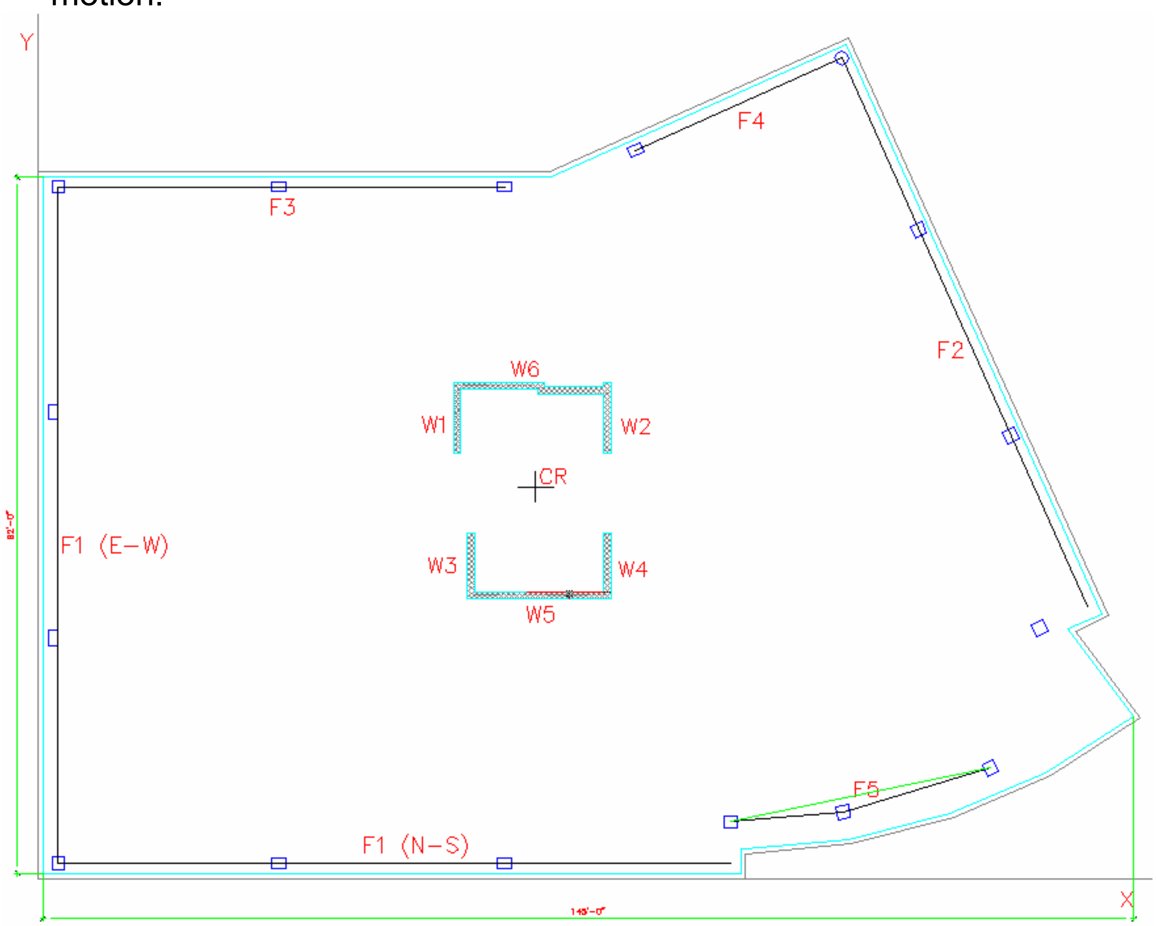


Figure 1 – view of typical floor showing
locations of shears and moment frames.

Distribution

To the start the load distribution, the continuous edge beam around the perimeter of the building was divided up into 6 frames for simplicity and model in STAAD. A 100 kip unit load was applied to each frame at each floor level. By dividing the corresponding unit by the deflection at that node resulted in the rigidity of the structure for each level. The rigidity for the shear walls were computed through spreadsheets. At frames 2, 4, and 5, the applied forces from wind and seismic were skewed from the frame directions. For this, it was assumed that a proportion of the rigidity would act in each direction equal to sine or cosine of the angle. However, this method was not used when accounting for torsion. The full rigidity was used times the perpendicular to the center of rigidity.

Below are the controlling story forces and shear forces organized by level.

story forces (kips)	E-W	N-S
roof	28.9	26.6
pent.	77.5	77.5
11	69.0	69.0
10	61.4	61.4
9	53.9	53.9
8	46.6	46.6
7	39.5	39.5
6	32.5	32.5
5	30.8	25.8
4	29.8	19.3
3	28.6	14.4
2	35.7	17.8
base	534.2	484.3

story shear (kips)	E-W	N-S
roof	28.9	26.6
pent.	106.4	104.1
11	175.4	173.1
10	236.8	234.5
9	290.7	288.4
8	337.3	335.0
7	376.8	374.5
6	409.3	407.0
5	440.1	432.8
4	469.9	452.1
3	498.5	466.5
2	534.2	484.3
base	534.2	484.3

Using the story forces, the table below calculated the overturning moments in each direction. From there results, the exterior on frame was checked to resist these added stresses including the flexural moments from the lateral forces. A detailed analysis of this spot check of a 20"x20" concrete column with 8#11 bars evenly distributed around the perimeter can be found in Appendix D. I was found the required area of steel was less than the actual steel area using the interaction diagrams on page 757 of the Design of Concrete Structures.

E-W	Story Forces (kips)	Story Height (ft)	Overturning Moment (kip-ft)
roof	28.9	153.33	4431.2
pent.	77.5	134.83	10449.3
11	69.0	122.33	8440.8
10	61.4	110.83	6805.0
9	53.9	99.33	5353.9
8	46.6	87.83	4092.9
7	39.5	76.33	3015.0
6	32.5	64.83	2107.0
5	30.8	53.33	1642.6
4	29.8	41.83	1246.5
3	28.6	30.33	867.4
2	35.7	18.83	672.2
	Over Turning Moment		49123.8

N-S	Story Forces (kips)	Story Height (ft)	Overturning Moment (kip-ft)
roof	26.6	153.33	4078.6
pent.	77.5	134.83	10449.3
11	69.0	122.33	8440.8
10	61.4	110.83	6805.0
9	53.9	99.33	5353.9
8	46.6	87.83	4092.9
7	39.5	76.33	3015.0
6	32.5	64.83	2107.0
5	25.8	53.33	1375.9
4	19.3	41.83	807.3
3	14.4	30.33	436.8
2	17.8	18.83	335.2
	Over Turning Moment		47297.6

Analysis of Deflections

For the analysis of the deflections, two programs were used to solve for these values and were checked by hand using the force over rigidity. The hand calculation for shear wall 5 found the wall to deflect 3.15" over a wall height of 153'-4". This value is equivalent to a total building deflection of L/583.8 which is within in the parameters of L/400. The maximum story drift for wall 5 was .442 which is also within the L/240 parameter. However, the building drift does not much the drift from the Ram analysis yielding a maximum drift of 5.15". It is believed that the hand calculations are more accurate. During the STAAD analysis of the moment frame, the materials were unable to be 4000 psi concrete as specified in the plan, so the frame was analyzed with 3000 psi. With the proper strength concrete in place, the frames should become more rigid and assist more in taking some of the load from the shears causing a decrease in the building drift.

Conclusion

Through this report the lateral systems were found to be sufficient to withstand the lateral loads. The spot checks of the shear wall 5 in Appendix E show the wall with the worst case load is able to satisfy the both shear and flexural. The column in Frame 1 was analyzed to using combined loads and the interaction diagram. The reinforcement in this member was found be to be adequate for combine flexure and compressive stresses.

The lateral deflections of the shear walls and frames were found to be accurate through hand calculation but will require further investigation to insure the rigidity of the moment frames are acceptable.

From the technical report, the lateral forces for the Executive Tower were checked and corrected to still be used in future examinations.

Appendix A

Seismic and Wind

Location DC

Category II site Class D

seismic group I
importance factor = 1.0

S_s = 18.0%g (Fig 9.4.1.1a - ASCE7-02)
S_1 = 6.2%g (Fig 9.4.1.1b - ASCE7-02)

$F_a = 1.6$
$F_v = 2.4$

$S_{MS} = F_a S_s$
$= (1.6)(.18)$
$= .288$

$S_{M1} = (2.4)(.062)$
$= .149$

$S_{DS} = \frac{2}{3} S_{MS} = .192$

$S_{D1} = \frac{2}{3} S_{M1} = .099$

Seismic Design Cat.
I - (Tab. 9.1.3 - ASCE7-02)
R = 3

$C_{s\,min} = .044 I S_{DS}$
$= .044(1.0)(.192) = .0084$

$C_s = \frac{S_{DS}}{(R/I)}$
$= \frac{.192}{(3/1.0)} = .064$

$C_{s\,max} = \frac{S_{D1}}{T(R/I)}$
$= \frac{.099}{1.48(3/1.0)} = .022$ ← controls

$T = C_T h_n^x$

$C_T = .016$
$h = 153.33$ ft
$x = .9$

$T = .016(153.33)^{.9}$
$T = 1.48$

Floor loads

Typ Live load

office - 80 psf + 20 psf (partition)

= 100 psf

Typ Floor Area
12 200 ft²
P = 450 ft
roof Area
3600 ft²

Dead Load estimation

- concrete flat slab w/ 8" drops + 3½" edge thickness

Total Area (ft²) of drop panels 2341.2 ft²

Total Area (ft²) of 3½" edge 1365.9 ft²

equivalent Floor Thickness

$$T = 8'' + \frac{(8'')(2341.2\,ft^2)}{12{,}200\,ft^2} + \frac{(3\tfrac{1}{2}'')(1365.9\,ft^2)}{12{,}200\,ft^2}$$

$T = 9.93''$ ⟶ Self wt = $9.93''(150\,pcf) = 124\,psf$

Curtain wall ≈ 300 plf (450') ⇒ 135^k

equivalent floor load ≈ $\frac{135^k}{12200\,ft^2}$ = 11.06 psf ⇒ 12 psf

Total Dead load

concrete	124 psf
Sprinklers	5 psf
MEP	5 psf
Finishes	10 psf
Curtain wall	12 psf
	156 psf

Total Floor Load = (156 psf)(12,200 ft²) ⇒ 1903.0^k/FL

Total roof Load = (156 psf)(3600 ft²) ⇒ 561.6^k

Total Building load = (1903^k/FL)(11 FL) + 561.6^k

= 21500^k

Wind | Appendix A

Exposure B case2

k_{zt} = 1.0 (no hills)
k_d = .85
V = 90 mph
I = 1.0

$q_z = 0.00256 k_z k_{zt} k_d V^2 I$

Period of Building

C_t = .016
h = 153.33
x = .9

$T = .016(153.33)^{.9}$
= 1.48

$f = \frac{1}{T}$
= .67 < 1.0 ∴ flexible building

Windward pressure: $P_w = q_z G C_p$ $C_p = .8$

Leeward pressure: $P_L = q_h G C_p$

for leeward
C_p = -.3 N-S
= -.5 E-W

z_g = 1200 ft
α = 7.0

$k_h = 2.01\left(\frac{153.33}{1200}\right)^{7.0} = 1.12$

$q_z = 0.00256(1.12)(1.0)(.85)(90\,mph)^2(1.0)$

q_z = 19.74

G = .839 N-S
= .843 E-W

P_L = (19.74)(.839)(-.3) = -4.97 N-S
= (19.74)(.843)(-.5) = -8.32 E-W

flexible buildings (f < 1.0 Hz) (6.5.8): G_f

Table 6-2

z_{min}	30		V	90	F 6-1
c	0.30		β	0.05	
ℓ	320		$V_z = b\,(z/33)^{\alpha} V(88/60) =$	68.7694659	
ε	0.33		$n_1 = f =$	0.67424168	
b	0.45		$\eta_h = 4.6 n_1 h/V_z =$	6.91520264	
α	0.142857143		$\eta_B = 4.6 n_1 B/V_z =$	6.53951857	
L	92		$\eta_L = 15.4 n_1 L/V_z =$	13.8908395	
B	145		$R_i = (1/\eta_i)-(1/(2\eta^2))(1-e^{-2\eta})$		
g_Q	3.4		R_h	0.13415306	
g_v	3.4		R_B	0.14122476	
$z = 0.6h =$	91.998	z>30?	R_L	0.06939862	
$L_z = \ell(z/33)^{\varepsilon} =$	450.372081	TRUE			
$I_z = c(33/z)^{1/6} =$	0.252877689	use 91.998			

$Q = (1/(1+0.63((B+h)/L_z)^{0.63}))^{1/2}$ = 0.82032966

$N_1 = n_1 L_z/V_z$ = 4.41561707

$R_n = 7.47N_1/(1+10.3N_1)^{5/3}$ = 0.05489361

$R = ((1/\beta)R_n R_h R_B(0.53+0.47R_L)^{1/2}$ = 0.10817781

$g_R = (2\ln(3600n_1))^{1/2}+0.577/(2\ln(3600n_1))^{1/2} =$ 4.094435435

$G_f = 0.925(((1+1.7I_z(g_Q{}^2Q^2+g_R{}^2{}_R{}^2)^{1/2})/(1+1.7g_vI_z)) =$ 0.831964978 flexible buildings (f < 1.0 Hz) (6.5.8): G_f

Table 6-2

z_{min}	30		V	90	F 6-1
c	0.3		β	0.05
ℓ	320		$V_z = b\,(z/33)^{\alpha} V(88/60) =$	68.7694659
ε	0.3333		$n_1 = f =$	0.67424168
b	0.45		$\eta_h = 4.6 n_1 h/V_z =$	6.91520264
α	0.142857143		$\eta_B = 4.6 n_1 B/V_z =$	4.14921178
L	145		$\eta_L = 15.4 n_1 L/V_z =$	21.8931709
B	92		$R_i = (1/\eta_i)-(1/(2\eta^2))(1-e^{-2\eta})$	
g_Q	3.4		R_h	0.13415306
g_v	3.4		R_B	0.21197404
$z = 0.6h =$	91.998	z>30?	R_L	0.04463318
$L_z = \ell(z/33)^{\varepsilon} =$	450.3566897	TRUE		
$I_z = c(33/z)^{1/6} =$	0.252877689	use 91.998		

$Q = (1/(1+0.63((B+h)/L_z)^{0.63}))^{1/2}$ = 0.83633627

$N_1 = n_1 L_z/V_z$ = 4.41546617

$R_n = 7.47N_1/(1+10.3N_1)^{5/3}$ = 0.0548948

$R = ((1/\beta)R_n R_h R_B(0.53+0.47R_L))^{1/2}$ = 0.13115631

$g_R = (2\ln(3600n_1))^{1/2}+0.577/(2\ln(3600n_1))^{1/2}$ = 4.094435435

$G_f = 0.925(((1+1.7I_z(g_Q{}^2Q^2+g_R{}^2{}_R{}^2)^{1/2})/(1+1.7g_vI_z))$ = 0.843230015

Appendix B

Load Distribution

E-W walls 176.99

Wall 1+2 $R = \frac{Et}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)}$

$$R = \frac{(3600\text{ ksi})(12")}{4\left(\frac{11.5'(12")}{9.5'(12")}\right)^3 + 3\left(\frac{11.5'(12")}{9.5'(12")}\right)} = 4027\ ^{k/in}$$

Wall 3+4 $R = \frac{(3600\text{ ksi})(12")}{4\left(\frac{11.5'(12")}{8.9(12")}\right)^3 + 3\left(\frac{11.5(12)}{8.9(12)}\right)} = 3454\ ^{k/in}$

Frame 1

from STAAD analysis

$\Delta = .003"$ from 1^k Unit Load $R = \frac{1^k}{.003"} = 333.3\ ^{k/in}$

Frame 2

$\Delta = .003"$ from 1^k Unit Load $R = \frac{1^k}{.003"} = 333.3\ ^{k/in}$

Fram 3

$\Delta = .005"$ from 1^k Unit Load $R = \frac{1^k}{.005"} = 200\ ^{k/in}$

Frame 4

$\Delta = .007"$ from 1^k Unit Load $R = \frac{1^k}{.007"} = 143\ ^{k/in}$

Frame 5

$\Delta = .004"$ from 1^k Unit Load $R = \frac{1^k}{.004"} = 250\ ^{k/in}$

N-S walls

Wall 5 $R = \frac{Et}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)}$

$$R = \frac{(3600\text{ ksi})(12'')}{4\left(\frac{11.5(12'')}{19.17(12)}\right)^3 + 3\left(\frac{11.5(12)}{19.17(12)}\right)} = 16221\ ^{k/in}$$

Wall 6 $$R = \frac{(3600\text{ ksi})(12'')}{4\left(\frac{11.5(12)}{21(12)}\right)^3 + 3\left(\frac{11.5(12)}{21(12)}\right)} = 18785\ ^{k/in}$$

Lateral	h	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	F1	F2	F2	F2	F3	F4	F4	F4	F5	F5	F5	ΣR	
Rigidity	(ft)	(E-W)	(E-W)	(E-W)	(E-W)	(N-S)	(N-S)	(E-W)		(E-W)	(N-S)	(N-S)		(E-W)	(N-S)		(E-W)	(N-S)	(E-W)	(N-S)
roof	153.3	2.6	2.6	2.1	2.1	20.9	27.4	8.9	9.2	8.4	3.7	5.9	3.3	1.3	3.0	9.8	2.0	9.6	30.0	70.5
12	133.8	3.8	3.8	3.2	3.2	31.3	41.0	10.0	10.2	9.3	4.2	6.6	3.7	1.5	3.4	11.1	2.3	10.9	37.2	97.2
11	122.3	5.0	5.0	4.1	4.1	40.8	53.5	11.1	11.5	10.5	4.7	7.4	4.2	1.7	3.8	12.7	2.6	12.4	44.3	122.6
10	110.8	6.8	6.8	5.6	5.6	54.7	71.5	12.6	13.0	11.8	5.3	8.4	4.8	2.0	4.4	14.6	3.0	14.3	54.1	158.6
9	99.33	9.4	9.4	7.7	7.7	75.5	98.7	14.5	14.9	13.6	6.1	9.7	5.6	2.3	5.1	17.1	3.6	16.7	68.2	211.9
8	87.83	13.5	13.5	11.2	11.2	108.4	141.6	17.1	17.5	15.9	7.1	11.4	6.6	2.7	6.0	20.4	4.2	20.0	89.4	294.5
7	76.33	20.6	20.6	16.9	16.9	163.4	212.8	20.7	21.0	19.2	8.6	13.8	8.1	3.3	7.4	25.3	5.3	24.7	123.5	430.7
6	64.83	33.4	33.4	27.6	27.6	262.0	340.3	26.2	26.4	24.1	10.7	17.4	10.5	4.3	9.6	32.8	6.8	32.1	183.4	672.2
5	53.33	59.6	59.6	49.2	49.2	457.3	590.7	36.2	35.6	32.5	14.5	23.9	14.8	6.0	13.5	46.7	9.7	45.7	302.0	1145.6
4	41.83	121.8	121.8	100.6	100.6	898.0	1149.3	56.8	53.7	49.0	21.9	37.0	24.0	9.8	21.9	73.2	15.2	71.6	575.6	2199.7
3	30.33	309.1	309.1	256.3	256.3	2098.3	2636.8	113.6	101.2	92.4	41.2	73.5	50.5	20.5	46.1	146.0	30.3	142.8	1387.8	5038.7
2	18.83	1164.6	1164.6	976.7	976.7	6411.7	7750.7	450.5	367.7	335.8	149.7	291.5	217.9	88.6	199.1	574.0	119.3	561.5	5276.7	15364.1

E-W	story forces (kips)	story shear (kips)	Wall 1		Wall 2		Wall 3		Wall 4		F1		F2		F4		F5	
			prop. (%)	Shear (kips)	prop. (%)	Shear (kips)	prop. (%)	Shear (kips)	prop. (%)	Shear (kips)	prop. (%)	Force (kips)	prop. (%)	Force (kips)	prop. (%)	Force (kips)	prop. (%)	Force (kips)
roof	28.9	28.9	0.085	2.5	0.085	2.5	0.070	2.0	0.070	2.0	0.297	8.57	0.197	5.68	0.045	1.29	0.068	1.96
pent.	77.5	106.4	0.104	11.0	0.104	11.0	0.085	9.1	0.085	9.1	0.269	20.84	0.178	13.76	0.040	3.14	0.062	4.81
11	69.0	175.4	0.114	20.0	0.114	20.0	0.094	16.4	0.094	16.4	0.251	17.31	0.167	11.54	0.039	2.66	0.060	4.11
10	61.4	236.8	0.125	29.6	0.125	29.6	0.103	24.4	0.103	24.4	0.233	14.30	0.155	9.53	0.036	2.22	0.056	3.44
9	53.9	290.7	0.138	40.0	0.138	40.0	0.113	32.9	0.113	32.9	0.213	11.47	0.142	7.67	0.033	1.80	0.052	2.81
8	46.6	337.3	0.152	51.1	0.152	51.1	0.125	42.1	0.125	42.1	0.191	8.92	0.128	5.94	0.030	1.40	0.047	2.21
7	39.5	376.8	0.167	62.8	0.167	62.8	0.137	51.7	0.137	51.7	0.168	6.62	0.112	4.41	0.027	1.05	0.043	1.68
6	32.5	409.3	0.182	74.6	0.182	74.6	0.150	61.5	0.150	61.5	0.143	4.64	0.095	3.08	0.023	0.76	0.037	1.21
5	30.8	440.1	0.197	86.9	0.197	86.9	0.163	71.7	0.163	71.7	0.120	3.69	0.079	2.44	0.020	0.61	0.032	0.99
4	29.8	469.9	0.212	99.4	0.212	99.4	0.175	82.1	0.175	82.1	0.099	2.94	0.064	1.92	0.017	0.51	0.026	0.79
3	28.6	498.5	0.223	111.0	0.223	111.0	0.185	92.1	0.185	92.1	0.082	2.34	0.053	1.51	0.015	0.42	0.022	0.63
2	35.7	534.2	0.221	117.9	0.221	117.9	0.185	98.9	0.185	98.9	0.085	3.05	0.055	1.97	0.017	0.60	0.023	0.81
base	534.2			117.9		117.9		98.9		98.9		104.69		69.46		16.45		25.44

N-S	story forces (kips)	story shear (kips)	Wall 5		Wall 6		F1		F2		F3		F4		F5	
			prop. (%)	Shear (kips)	prop. (%)	Shear (kips)	prop. (%)	Force (kips)	prop. (%)	Force (kips)	prop. (%)	Force (kips)	prop. (%)	Force (kips)	prop. (%)	Force (kips)
roof	26.6	26.6	0.296	7.9	0.388	10.3	0.126	3.36	0.053	1.41	0.084	2.23	0.043	1.14	0.136	3.62
pent.	77.5	104.1	0.321	33.5	0.421	43.9	0.103	7.97	0.043	3.32	0.068	5.26	0.035	2.69	0.112	8.65
11	69.0	173.1	0.333	57.6	0.436	75.5	0.091	6.25	0.038	2.62	0.060	4.17	0.031	2.16	0.101	6.99
10	61.4	234.5	0.345	80.8	0.451	105.8	0.079	4.88	0.033	2.05	0.053	3.25	0.028	1.70	0.090	5.53
9	53.9	288.4	0.356	102.8	0.466	134.4	0.068	3.69	0.029	1.54	0.046	2.47	0.024	1.30	0.079	4.26
8	46.6	335.0	0.368	123.3	0.481	161.0	0.058	2.71	0.024	1.13	0.039	1.80	0.020	0.95	0.068	3.16
7	39.5	374.5	0.379	142.0	0.494	185.1	0.048	1.90	0.020	0.79	0.032	1.27	0.017	0.68	0.057	2.27
6	32.5	407.0	0.390	158.7	0.506	206.1	0.039	1.27	0.016	0.52	0.026	0.84	0.014	0.46	0.048	1.55
5	25.8	432.8	0.399	172.8	0.516	223.2	0.032	0.82	0.013	0.33	0.021	0.54	0.012	0.30	0.040	1.03
4	19.3	452.1	0.408	184.6	0.522	236.2	0.026	0.50	0.010	0.19	0.017	0.32	0.010	0.19	0.033	0.63
3	14.4	466.5	0.416	194.3	0.523	244.1	0.023	0.32	0.008	0.12	0.015	0.21	0.009	0.13	0.028	0.41
2	17.8	484.3	0.417	202.1	0.504	244.3	0.029	0.52	0.010	0.17	0.019	0.34	0.013	0.23	0.037	0.65
base	484.3			202.1		244.3		34.18		14.19		22.69		11.95		38.74

Torsion		Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	F1 (E-W)	F1 (N-S)	F2	F3	F4	F5	ΣRx^2
Dist. from CR		10.5	9.5	8.5	9.5	14.5	13.0	63.5	50.1	60.3	40.0	35.5	49.0	
roof		27	27	20	18	198	396	565	446	554	236	117	480	137654
12		40	37	27	30	452	533	635	513	564	148	131	544	151978
11		53	48	35	39	590	695	705	574	631	168	149	622	173279
10		71	64	47	53	790	930	800	650	715	192	170	715	201025
9		99	89	66	73	1091	1284	921	746	821	224	199	838	238065
8	Rx	142	129	95	106	1567	1840	1086	875	962	264	234	1000	289764
7		216	196	144	161	2360	2767	1314	1054	1159	324	288	1240	367088
6		351	318	234	262	3787	4424	1664	1322	1454	420	373	1607	491803
5		626	566	418	467	6608	7679	2299	1781	1959	592	525	2288	723096
4		1279	1157	855	956	12977	14941	3607	2688	2956	960	852	3587	1208947
3		3246	2937	2179	2435	30320	34278	7214	5071	5577	2020	1793	7154	2530898
2		12228	11063	8302	9279	92649	100759	28607	18419	20257	8716	7735	28126	9003737

Rx/Rx^2	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	F1 (E-W)	F1 (N-S)	F2	F3	F4	F5
roof	0.000	0.000	0.000	0.000	0.001	0.003	0.004	0.003	0.004	0.002	0.001	0.003
12	0.000	0.000	0.000	0.000	0.003	0.004	0.004	0.003	0.004	0.001	0.001	0.004
11	0.000	0.000	0.000	0.000	0.003	0.004	0.004	0.003	0.004	0.001	0.001	0.004
10	0.000	0.000	0.000	0.000	0.004	0.005	0.004	0.003	0.004	0.001	0.001	0.004
9	0.000	0.000	0.000	0.000	0.005	0.005	0.004	0.003	0.003	0.001	0.001	0.004
8	0.000	0.000	0.000	0.000	0.005	0.006	0.004	0.003	0.003	0.001	0.001	0.003
7	0.001	0.001	0.000	0.000	0.006	0.008	0.004	0.003	0.003	0.001	0.001	0.003
6	0.001	0.001	0.000	0.001	0.008	0.009	0.003	0.003	0.003	0.001	0.001	0.003
5	0.001	0.001	0.001	0.001	0.009	0.011	0.003	0.002	0.003	0.001	0.001	0.003
4	0.001	0.001	0.001	0.001	0.011	0.012	0.003	0.002	0.002	0.001	0.001	0.003
3	0.001	0.001	0.001	0.001	0.012	0.014	0.003	0.002	0.002	0.001	0.001	0.003
2	0.001	0.001	0.001	0.001	0.010	0.011	0.003	0.002	0.002	0.001	0.001	0.003

Torsional Perportions			Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	F1 (E-W)	F1 (N-S)	F2	F3	F4	F5
E-W	Torsional Moment (kip-ft)	Story Moment (kip-ft)	Resultant Forces from Torsion (kips)											
roof	204	204	0.040	0.040	0.030	0.027	0.294	0.586	0.838	0.661	0.821	0.350	0.174	0.712
pent.	751	547	0.200	0.181	0.133	0.149	2.233	2.633	2.286	1.845	2.029	0.533	0.473	1.958
11	1238	487	0.378	0.342	0.252	0.281	4.214	4.966	1.982	1.613	1.774	0.472	0.419	1.748
10	1672	433	0.591	0.534	0.393	0.440	6.569	7.735	1.725	1.401	1.541	0.414	0.367	1.543
9	2052	381	0.849	0.769	0.566	0.632	9.408	11.067	1.472	1.193	1.312	0.358	0.318	1.339
8	2381	329	1.169	1.058	0.779	0.871	12.875	15.123	1.233	0.993	1.092	0.300	0.266	1.135
7	2660	279	1.566	1.417	1.044	1.167	17.106	20.050	0.999	0.801	0.881	0.246	0.218	0.942
6	2890	229	2.063	1.867	1.376	1.538	22.248	25.993	0.776	0.617	0.678	0.196	0.174	0.750
5	3107	217	2.690	2.434	1.796	2.007	28.394	32.998	0.691	0.536	0.589	0.178	0.158	0.688
4	3317	210	3.509	3.175	2.347	2.623	35.610	40.999	0.628	0.468	0.514	0.167	0.148	0.624
3	3519	202	4.514	4.084	3.030	3.386	42.162	47.666	0.576	0.405	0.445	0.161	0.143	0.571
2	3771	252	5.122	4.634	3.478	3.887	38.808	42.205	0.801	0.516	0.567	0.244	0.217	0.787
base	3771	3771	5.1	4.6	3.5	3.9	38.8	42.2	14	11	12	4	3	13

N-S	Torsional Moment (kips-ft)	Story Moment (kip-ft)	Resultant Forces from Torsion (kips)											
roof	144	144	0.028	0.028	0.021	0.019	0.207	0.413	0.590	0.465	0.578	0.246	0.122	0.501
pent.	562	419	0.149	0.135	0.100	0.111	1.671	1.970	1.749	1.411	1.552	0.408	0.362	1.498
11	935	373	0.285	0.258	0.190	0.212	3.181	3.749	1.516	1.234	1.357	0.361	0.321	1.337
10	1266	332	0.447	0.405	0.298	0.333	4.975	5.859	1.320	1.072	1.179	0.317	0.281	1.180
9	1557	291	0.645	0.583	0.429	0.480	7.139	8.398	1.126	0.913	1.004	0.274	0.243	1.024
8	1809	252	0.888	0.804	0.592	0.661	9.781	11.488	0.943	0.760	0.835	0.229	0.203	0.868
7	2022	213	1.191	1.077	0.794	0.887	13.004	15.242	0.764	0.612	0.674	0.188	0.167	0.720
6	2198	176	1.569	1.420	1.047	1.170	16.922	19.769	0.594	0.472	0.519	0.150	0.133	0.574
5	2337	139	2.024	1.831	1.351	1.510	21.358	24.821	0.443	0.343	0.377	0.114	0.101	0.441
4	2441	104	2.583	2.337	1.727	1.930	26.205	30.171	0.311	0.232	0.255	0.083	0.073	0.309
3	2519	78	3.231	2.923	2.169	2.424	30.178	34.118	0.222	0.156	0.171	0.062	0.055	0.220
2	2615	96	3.552	3.213	2.411	2.695	26.911	29.266	0.305	0.197	0.216	0.093	0.083	0.300
base	2615	2615	4	3	2	3	27	29	10	8	9	3	2	9

Total Forces (E-W)	Wall 1 Story Shear (kips)			Wall 2 Story Shear (kips)			Wall 3 Story Shear (kips)			Wall 4 Story Shear (kips)			Wall 5 Story Shear (kips)			Wall 6 Story Shear (kips)			F1 (E-W) StoryForces (kips)		
roof	2.47	-0.04	2.43	2.47	0.04	2.51	2.03	-0.03	2.00	2.03	0.03	2.06	0.00	0.29	0.29	0.00	0.59	0.59	8.57	-0.84	7.73
pent.	11.01	-0.20	10.81	11.01	0.18	11.19	9.06	-0.13	8.93	9.06	0.15	9.21	0.00	2.23	2.23	0.00	2.63	2.63	20.84	-2.29	18.56
11	19.96	-0.38	19.58	19.96	0.34	20.30	16.42	-0.25	16.17	16.42	0.28	16.70	0.00	4.21	4.21	0.00	4.97	4.97	17.31	-1.98	15.32
10	29.61	-0.59	29.02	29.61	0.53	30.15	24.37	-0.39	23.97	24.37	0.44	24.81	0.00	6.57	6.57	0.00	7.73	7.73	14.30	-1.73	12.58
9	40.03	-0.85	39.18	40.03	0.77	40.80	32.94	-0.57	32.37	32.94	0.63	33.57	0.00	9.41	9.41	0.00	11.07	11.07	11.47	-1.47	10.00
8	51.13	-1.17	49.96	51.13	1.06	52.19	42.09	-0.78	41.31	42.09	0.87	42.96	0.00	12.88	12.88	0.00	15.12	15.12	8.92	-1.23	7.68
7	62.78	-1.57	61.22	62.78	1.42	64.20	51.70	-1.04	50.65	51.70	1.17	52.86	0.00	17.11	17.11	0.00	20.05	20.05	6.62	-1.00	5.62
6	74.65	-2.06	72.58	74.65	1.87	76.51	61.50	-1.38	60.12	61.50	1.54	63.03	0.00	22.25	22.25	0.00	25.99	25.99	4.64	-0.78	3.87
5	86.90	-2.69	84.21	86.90	2.43	89.34	71.66	-1.80	69.86	71.66	2.01	73.66	0.00	28.39	28.39	0.00	33.00	33.00	3.69	-0.69	3.00
4	99.44	-3.51	95.93	99.44	3.18	102.61	82.13	-2.35	79.79	82.13	2.62	84.76	0.00	35.61	35.61	0.00	41.00	41.00	2.94	-0.63	2.31
3	111.04	-4.51	106.52	111.04	4.08	115.12	92.07	-3.03	89.04	92.07	3.39	95.46	0.00	42.16	42.16	0.00	47.67	47.67	2.34	-0.58	1.77
2	117.90	-5.12	112.78	117.90	4.63	122.53	98.88	-3.48	95.40	98.88	3.89	102.77	0.00	38.81	38.81	0.00	42.21	42.21	3.05	-0.80	2.25
base	117.90	-5.12	112.78	117.90	4.63	122.53	98.88	-3.48	95.40	98.88	3.89	102.77	0.00	38.81	38.81	0.00	42.21	42.21	104.69	-14.01	90.69

F1 (N-S) Story Force (kips)			F2 StoryForces (kips)			F3 StoryForces (kips)			F4 StoryForces (kips)			F5 StoryForces (kips)		
0.00	0.66	0.66	5.68	0.82	6.50	0.00	0.35	0.35	1.29	-0.17	1.12	1.96	0.71	2.67
0.00	1.85	1.85	13.76	2.03	15.79	0.00	0.53	0.53	3.14	-0.47	2.66	4.81	1.96	6.77
0.00	1.61	1.61	11.54	1.77	13.31	0.00	0.47	0.47	2.66	-0.42	2.24	4.11	1.75	5.86
0.00	1.40	1.40	9.53	1.54	11.08	0.00	0.41	0.41	2.22	-0.37	1.85	3.44	1.54	4.99
0.00	1.19	1.19	7.67	1.31	8.98	0.00	0.36	0.36	1.80	-0.32	1.48	2.81	1.34	4.15
0.00	0.99	0.99	5.94	1.09	7.04	0.00	0.30	0.30	1.40	-0.27	1.13	2.21	1.13	3.35
0.00	0.80	0.80	4.41	0.88	5.29	0.00	0.25	0.25	1.05	-0.22	0.83	1.68	0.94	2.62
0.00	0.62	0.62	3.08	0.68	3.76	0.00	0.20	0.20	0.76	-0.17	0.58	1.21	0.75	1.96
0.00	0.54	0.54	2.44	0.59	3.03	0.00	0.18	0.18	0.61	-0.16	0.46	0.99	0.69	1.68
0.00	0.47	0.47	1.92	0.51	2.43	0.00	0.17	0.17	0.51	-0.15	0.36	0.79	0.62	1.41
0.00	0.40	0.40	1.51	0.44	1.96	0.00	0.16	0.16	0.42	-0.14	0.28	0.63	0.57	1.20
0.00	0.52	0.52	1.97	0.57	2.54	0.00	0.24	0.24	0.60	-0.22	0.38	0.81	0.79	1.59
0.00	11.05	11.05	69.46	12.24	81.71	0.00	3.62	3.62	16.45	-3.08	13.38	25.44	12.80	38.24

Total Forces (N-S)	Wall 1 Story Shear (kips)			Wall 2 Story Shear (kips)			Wall 3 Story Shear (kips)			Wall 4 Story Shear (kips)			Wall 5 Story Shear (kips)			Wall 6 Story Shear (kips)			F1 (E-W) StoryForces (kips)		
roof	0.00	0.03	0.03	0.00	0.03	0.03	0.00	0.02	0.02	0.00	0.02	0.02	7.88	0.21	8.08	10.33	-0.41	9.92	0.00	0.59	0.59
pent.	0.00	0.15	0.15	0.00	0.14	0.14	0.00	0.10	0.10	0.00	0.11	0.11	33.47	1.67	35.14	43.86	-1.97	41.89	0.00	1.75	1.75
11	0.00	0.29	0.29	0.00	0.26	0.26	0.00	0.19	0.19	0.00	0.21	0.21	57.63	3.18	60.81	75.49	-3.75	71.74	0.00	1.52	1.52
10	0.00	0.45	0.45	0.00	0.40	0.40	0.00	0.30	0.30	0.00	0.33	0.33	80.84	4.98	85.82	105.81	-5.86	99.95	0.00	1.32	1.32
9	0.00	0.64	0.64	0.00	0.58	0.58	0.00	0.43	0.43	0.00	0.48	0.48	102.80	7.14	109.94	134.41	-8.40	126.01	0.00	1.13	1.13
8	0.00	0.89	0.89	0.00	0.80	0.80	0.00	0.59	0.59	0.00	0.66	0.66	123.34	9.78	133.12	161.04	-11.49	149.55	0.00	0.94	0.94
7	0.00	1.19	1.19	0.00	1.08	1.08	0.00	0.79	0.79	0.00	0.89	0.89	142.04	13.00	155.05	185.05	-15.24	169.81	0.00	0.76	0.76
6	0.00	1.57	1.57	0.00	1.42	1.42	0.00	1.05	1.05	0.00	1.17	1.17	158.67	16.92	175.59	206.05	-19.77	186.28	0.00	0.59	0.59
5	0.00	2.02	2.02	0.00	1.83	1.83	0.00	1.35	1.35	0.00	1.51	1.51	172.76	21.36	194.12	223.17	-24.82	198.35	0.00	0.44	0.44
4	0.00	2.58	2.58	0.00	2.34	2.34	0.00	1.73	1.73	0.00	1.93	1.93	184.57	26.21	210.78	236.21	-30.17	206.04	0.00	0.31	0.31
3	0.00	3.23	3.23	0.00	2.92	2.92	0.00	2.17	2.17	0.00	2.42	2.42	194.26	30.18	224.44	244.12	-34.12	210.00	0.00	0.22	0.22
2	0.00	3.55	3.55	0.00	3.21	3.21	0.00	2.41	2.41	0.00	2.70	2.70	202.11	26.91	229.02	244.31	-29.27	215.05	0.00	0.31	0.31
base	0.00	3.55	3.55	0.00	3.21	3.21	0.00	2.41	2.41	0.00	2.70	2.70	202.11	26.91	229.02	244.31	-29.27	215.05	0.00	9.88	9.88

F1 (N-S) Story Force (kips)			F2 StoryForces (kips)			F3 StoryForces (kips)			F4 StoryForces (kips)			F5 StoryForces (kips)		
3.36	0.47	3.82	1.41	-0.58	0.83	2.23	-0.25	1.98	1.14	-0.12	1.02	3.62	0.50	4.12
7.97	1.41	9.38	3.32	-1.55	1.77	5.26	-0.41	4.85	2.69	-0.36	2.33	8.65	1.50	10.15
6.25	1.23	7.48	2.62	-1.36	1.27	4.17	-0.36	3.80	2.16	-0.32	1.84	6.99	1.34	8.32
4.88	1.07	5.95	2.05	-1.18	0.87	3.25	-0.32	2.94	1.70	-0.28	1.42	5.53	1.18	6.71
3.69	0.91	4.60	1.54	-1.00	0.54	2.47	-0.27	2.19	1.30	-0.24	1.06	4.26	1.02	5.28
2.71	0.76	3.47	1.13	-0.84	0.29	1.80	-0.23	1.57	0.95	-0.20	0.75	3.16	0.87	4.03
1.90	0.61	2.51	0.79	-0.67	0.11	1.27	-0.19	1.08	0.68	-0.17	0.51	2.27	0.72	2.99
1.27	0.47	1.74	0.52	-0.52	0.00	0.84	-0.15	0.69	0.46	-0.13	0.33	1.55	0.57	2.12
0.82	0.34	1.16	0.33	-0.38	-0.05	0.54	-0.11	0.42	0.30	-0.10	0.20	1.03	0.44	1.47
0.50	0.23	0.73	0.19	-0.25	-0.06	0.32	-0.08	0.24	0.19	-0.07	0.12	0.63	0.31	0.94
0.32	0.16	0.48	0.12	-0.17	-0.05	0.21	-0.06	0.15	0.13	-0.06	0.08	0.41	0.22	0.63
0.52	0.20	0.72	0.17	-0.22	-0.04	0.34	-0.09	0.24	0.23	-0.08	0.15	0.65	0.30	0.95
34.18	7.87	42.04	14.19	-8.72	5.47	22.69	-2.52	20.17	11.95	-2.14	9.80	38.74	8.97	47.71

Appendix C

Story Drift

Node Displacement

Node	L/C	X-Trans (in)	Y-Trans (in)	Z-Trans (in)	Absolute (in)	X-Rotan (rad)	Y-Rotan (rad)	Z-Rotan (rad)
2	8	5.675	0.034	0.000	5.675	0.00000	0.00000	-0.00056
49	8	5.561	0.034	0.000	5.561	0.00000	0.00000	-0.00108
45	8	5.334	0.034	0.000	5.335	0.00000	0.00000	-0.00189
41	8	4.990	0.033	0.000	4.990	0.00000	0.00000	-0.00260
37	8	4.545	0.032	0.000	4.545	0.00000	0.00000	-0.00319
33	8	4.020	0.030	0.000	4.020	0.00000	0.00000	-0.00364
29	8	3.435	0.027	0.000	3.435	0.00000	0.00000	-0.00398
25	8	2.807	0.023	0.000	2.807	0.00000	0.00000	-0.00421
21	8	2.146	0.019	0.000	2.146	0.00000	0.00000	-0.00444
17	8	1.494	0.015	0.000	1.494	0.00000	0.00000	-0.00449
13	8	0.853	0.011	0.000	0.853	0.00000	0.00000	-0.00425
9	8	0.290	0.006	0.000	0.290	0.00000	0.00000	-0.00329
1	8	0.000	0.000	0.000	0.000	0.00000	0.00000	0.00000

	wall 1				wall 2				wall 3			
	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}
roof	2.4	2.6	0.317	9.359	2.5	2.6	0.327	9.798	2.0	2.1	0.317	9.410
pent.	10.8	3.8	0.662	9.042	11.2	3.8	0.686	9.471	8.9	3.2	0.665	9.093
11	19.6	5.0	0.994	8.380	20.3	5.0	1.030	8.785	16.2	4.1	0.998	8.429
10	29.0	6.8	1.198	7.386	30.1	6.8	1.244	7.755	24.0	5.6	1.203	7.431
9	39.2	9.4	1.283	6.189	40.8	9.4	1.336	6.511	32.4	7.7	1.289	6.228
8	50.0	13.5	1.260	4.905	52.2	13.5	1.317	5.175	41.3	11.2	1.267	4.939
7	61.2	20.6	1.144	3.645	64.2	20.6	1.200	3.858	50.7	16.9	1.150	3.672
6	72.6	33.4	0.953	2.501	76.5	33.4	1.005	2.659	60.1	27.6	0.959	2.521
5	84.2	59.6	0.721	1.548	89.3	59.6	0.765	1.654	69.9	49.2	0.726	1.562
4	95.9	121.8	0.477	0.827	102.6	121.8	0.511	0.889	79.8	100.6	0.482	0.836
3	106.5	309.1	0.253	0.350	115.1	309.1	0.274	0.379	89.0	256.3	0.256	0.354
2	112.8	1164.6	0.097	0.097	122.5	1164.6	0.105	0.105	95.4	976.7	0.098	0.098

	wall 4				wall 5				wall 6			
	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}	Shear	R	Δ_{story}	Δ_{Total}
roof	2.1	2.1	0.326	9.806	8.1	20.9	0.129	3.151	9.9	27.4	0.120	2.698
pent.	9.2	3.2	0.686	9.480	35.1	31.3	0.263	3.023	41.9	41.0	0.239	2.578
11	16.7	4.1	1.031	8.794	60.8	40.8	0.378	2.760	71.7	53.5	0.339	2.339
10	24.8	5.6	1.245	7.763	85.8	54.7	0.434	2.382	100.0	71.5	0.385	2.000
9	33.6	7.7	1.337	6.518	109.9	75.5	0.442	1.948	126.0	98.7	0.386	1.615
8	43.0	11.2	1.317	5.182	133.1	108.4	0.413	1.507	149.5	141.6	0.354	1.229
7	52.9	16.9	1.201	3.864	155.0	163.4	0.357	1.094	169.8	212.8	0.299	0.876
6	63.0	27.6	1.006	2.663	175.6	262.0	0.286	0.736	186.3	340.3	0.232	0.577
5	73.7	49.2	0.766	1.658	194.1	457.3	0.208	0.450	198.3	590.7	0.163	0.345
4	84.8	100.6	0.512	0.892	210.8	898.0	0.134	0.242	206.0	1149.3	0.101	0.181
3	95.5	256.3	0.275	0.380	224.4	2098.3	0.072	0.108	210.0	2636.8	0.053	0.080
2	102.8	976.7	0.105	0.105	229.0	6411.7	0.036	0.036	215.0	7750.7	0.028	0.028

RAM Advanse

File :
Units system : English
Date : 11/21/2005 3:13:01 PM

Analysis Results

Translations

	Translations [in]			Rotations [Rad]		
Node	**TX**	**TY**	**TZ**	**RX**	**RY**	**RZ**
Condition **dl=Dead load**						
3	5.15120	0.41876	0.00000	0.00000	0.00000	0.00000

Appendix D

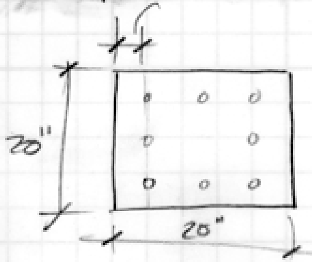
Column Spot Check

Frame F1 later checks | Appendix D

F1 col. 1

Trib Area = 1980 ft^2

$1.5'' + .705 + .25 = 2.5''$



$LL = 100\,psf$

$LL = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) > .4L$

$LL = 100\left(.25 + \frac{15}{\sqrt{1(1980)}}\right)$

$LL = 58.7\,psf$

$\gamma = \frac{20-5}{20} = .75$

using $\gamma = .70$

$DL = 150\,psf$

$P_u = 1.2(D) + 1.6(W) + L$

$P_u = \frac{(1.2(150) + 58.7)(1980\,ft^2)}{1000\,\#/k} + 1.6(39.3^k)$

$P_u = 535.5$

$M_u = 306.3$

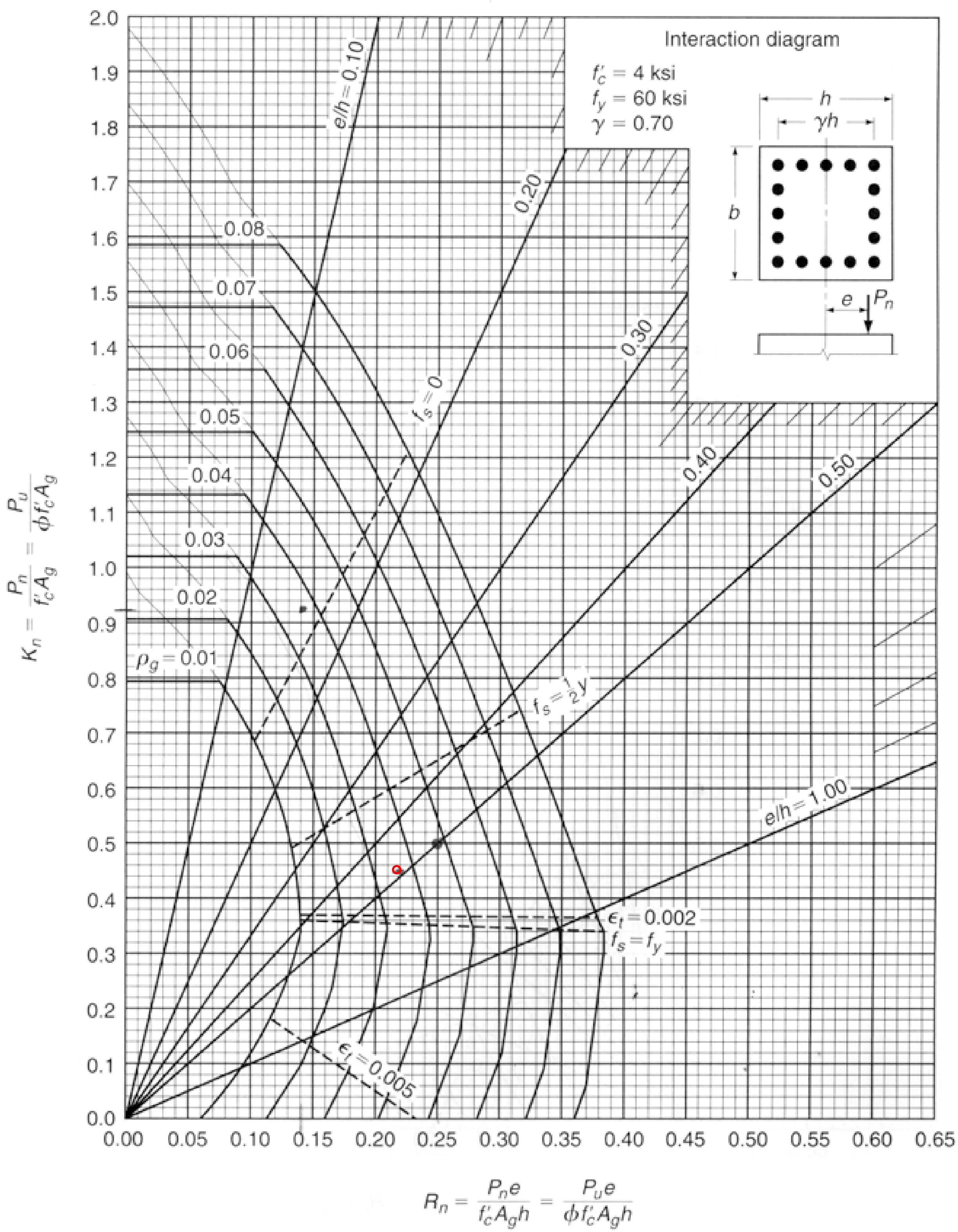
$K_n = \frac{P_u}{.65(f'_c)(A_g)} = \frac{535.5}{.65(4)(400)} = .51$

$R_n = \frac{M_u}{.65(f'_c)(A_g)(h)} = \frac{306.3(12'')}{.65(4)(400)(20)} = .18$

$\rho = .020$

$A_s = .020(400)$
$= 8\,in^2$

Actual col uses 8 #11
$A_s = 12.5\,in^2 > 8\,in^2$ ok

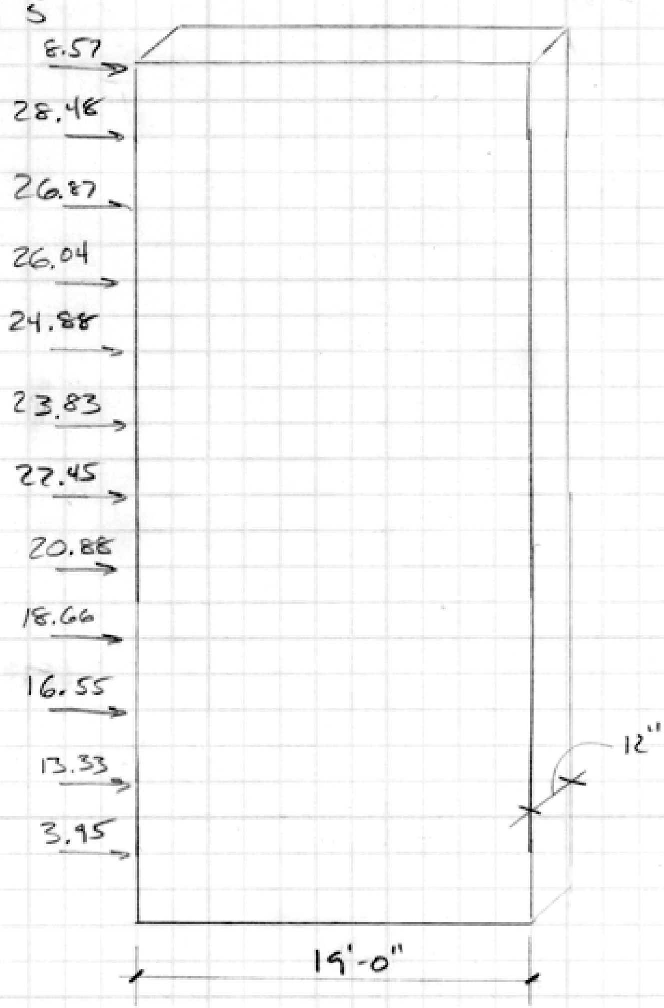

Interaction diagram
f'_c = 4 ksi
f_y = 60 ksi
γ = 0.70
h
γh
b
e
P_n
e/h = 0.10
0.20
0.30
0.40
0.50
e/h = 1.00
0.08
0.07
0.06
0.05
0.04
0.03
0.02
ρ_g = 0.01
$f_s = 0$
$f_s = \frac{1}{2} f_y$
$\epsilon_t = 0.002$
$f_s = f_y$
$\epsilon_t = 0.005$
$K_n = \frac{P_n}{f'_c A_g} = \frac{P_u}{\phi f'_c A_g}$
$R_n = \frac{P_n e}{f'_c A_g h} = \frac{P_u e}{\phi f'_c A_g h}$
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8 1.9 2.0
0.00 0.05 0.10 0.15 0.20 0.25 0.30 0.35 0.40 0.45 0.50 0.55 0.60 0.65

GRAPH A.6

Appendix E

Shear Wall Spot Check

Wall 5



from RAM Analysis

$\Delta = 5.1512''$

$\frac{L}{400} = \frac{153.33(12'')}{400} = 4.599'' < 5.1512$ no good

Shear at 2nd floor   shear design

$V_u = 1.6(234.5) = 375.2$

$\phi V_c = 2\sqrt{f'_c}\, h d$ $d = .8\, l_w$ ACI 11.10

$\phi V_c = (.75)2\sqrt{4000}\,\frac{(12'')(.8)(19')(12''/1')}{1000\ \#/k} = 207.5^k < 375.2$

must provide shear reinforcement

$\phi V_c = (.75)10\sqrt{4000}\, h d = (.75)10\,\frac{\sqrt{4000}\,(12'')(.8)(19')(12'')}{1000}$

$= 1038.2 > 375.2$ wall section ok

$\phi V_s = V_u - \phi V_c = 375.2 - 207.5 = 167.7^k$

$V_s = 223.6^k$

Horizontal reinforcement   try #6@8" $A_v = .88$

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

$V_s = \frac{(.88\,in^2)(60)(.8)(19')(12'')}{8''} = 1203.8^k > 223.6^k$

ok, reinforcement used in building is adequate

Vertical reinforcement

$S_v = 8''$

$\frac{h_w}{l_w} = \frac{153}{19} = 8.07 > 2.5$

$A_{vv} = .0025(s_v)(h)$

$A_{vv} = .0025(8'')(12'')$

$A_{vv} = .24\,in^2 < .88\,in^2$ ok

Shear Wall Design | Appendix E

Flexural design

Trib Area = (250 ft)(12) = 3000 ft^2

$LL_R = 100\left(.25 + \frac{15}{\sqrt{2(3000)}}\right)$

$= 44.4$ psf

$1.2D + 1.6W + L$

$P_u = (1.2(150\text{ psf}) + 44.4)(3000\text{ ft}^2)$

$P_u = 673.2^k$

$M_u = 1.6(47300^{1k}) = 75680^{1k}$

$A_{ST} = .88(19)\left(\frac{12''}{8''}\right) = 25.1\text{ in}^2$

$\omega = \frac{A_{ST}}{l_w h}\left(\frac{f_y}{f'_c}\right) = \frac{25.1}{(19)(12)(12)}\left(\frac{60}{4}\right) = .138$

$\alpha = \frac{P_u}{l_w h f'_c} = \frac{673.2}{(19)(12)(12)(4)} = .062$

$\frac{c}{l_w} = \frac{\omega + \alpha}{2\omega + .85\beta_1} = \frac{.138 + .062}{2(.138) + .85(.8)} = .209$

$\phi M_n = .9\left[.5 A_{ST} f_y l_w \left(1 + \frac{P_u}{A_{ST} f_y}\right)\left(1 - \frac{c}{l_w}\right)\right]$

$= .9\left[.5(25.1)(60)(19)(12)\left(1 + \frac{673.2}{25.1(60)}\right)(1 - .209)\right]$

$\phi M_n = 176856^{1k} > 75680^{1k}$ ok