



1000 CONTINENTAL SQUARE

KING OF PRUSSIA, PENNSYLVANIA

Carter Davis Hayes
Structural Option
January 13, 2008

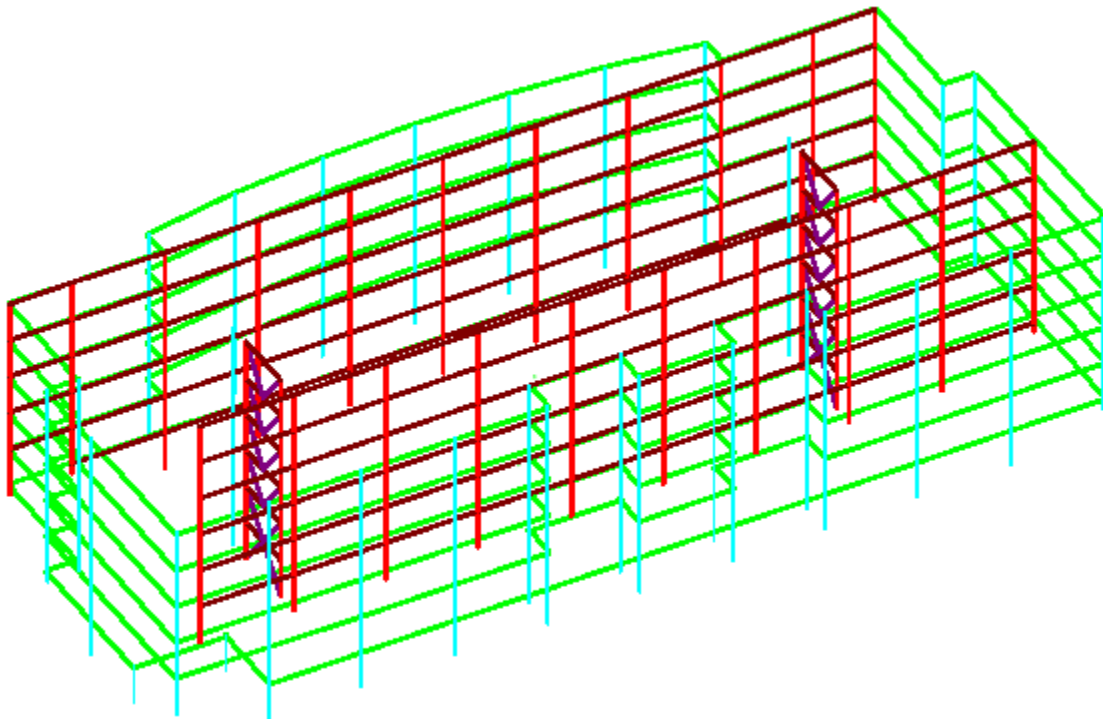
Advisor: Dr. Hanagan

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EXECUTIVE SUMMARY

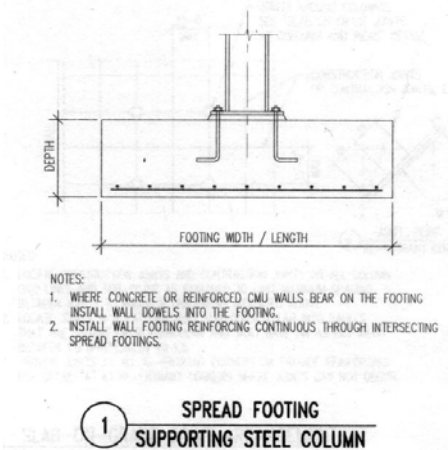
This is the third of three preliminary stages of analysis intended to impart a better understanding in each student of their individual building, and acts as an attempt to better focus research for the final thesis in the spring. This third paper focuses on the lateral force resisting systems and how lateral loads are distributed to different elements in those systems. My project is based on the design of the office building currently under construction at 1000 Continental Square in King of Prussia, PA. The building is high end office space featuring large open floor plans with uninterrupted forty foot bays along each side of the building. The site has a prominent location at the intersection of routes 202, 76, and 422, and is in close proximity to the PA Turnpike and King of Prussia Mall. A ground floor, partially below grade, serves mainly as space for mechanical systems and storage. Five floors of approximately 36,000 square feet of leasable space are located above that. The building makes use of a steel structural frame with composite metal decking and lightweight concrete slabs. Lateral loads are resisted by two moment frames along the long axis of the building and two eccentrically braced frames along the short axis.



I. STRUCTURAL SYSTEMS

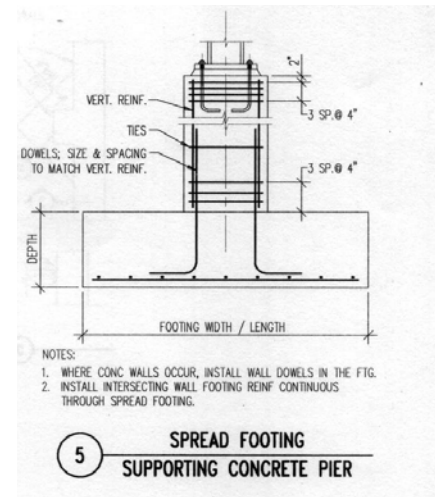
FOUNDATIONS

The foundations for 1000 Continental Square are a series of spread footings with continuous wall footings under the retaining walls located on the ground floor. The soils under the footings were found to withstand 4000 psf in most locations according to the geotechnical



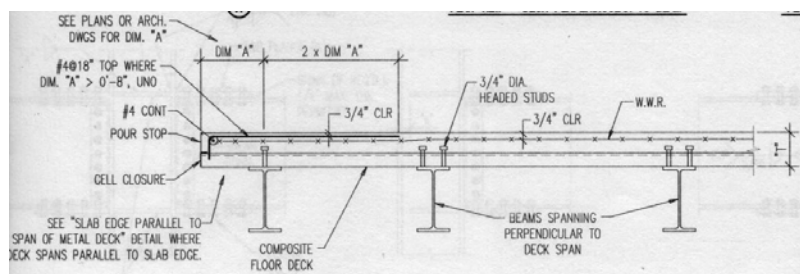
report furnished by Pennoni Associates Inc. on 24 of February 2004. Suitable bearing pressures were attained by deep dynamic compaction or partial soil exchange. Footing dimensions range from 4' x 4' x 1.5' to 20' x 20' x 4'; however, typical footings are approximately 14' x 14' x 3'. Special 55' x 18' x 3.5' spread footings are used under the braced frames. The tops of most footings are located 1.5' below grade, and minimum bearing depth is 3'. Columns either bear directly on

footings or in some atypical situations concrete piers are placed on top of the footings and columns bear on those. Footings have bottom reinforcement ranging from (7) #4's to (16) #11's with typical reinforcement being approximately (12) #9's. The continuous wall footings are integrated into the spread footings they intersect, and their reinforcement is continuous throughout. Concrete in all footings has a minimum compressive strength, $f'_c = 3000$ psi with a unit weight of 145 pcf. There is a 4" thick slab on grade which acts as the floor system for the ground floor and utilizes 4000 psi compressive strength concrete.



FLOOR FRAMING

All the floor framing above grade in the 1000 Continental Square project is 6 1/4" composite slabs. They consist of 3 1/4" lightweight concrete over 3" deep 20 gage galvanized composite floor deck. The slab is reinforced by one layer of 6 x 6 - W1.4 x W1.4 WWR, and has a weight of 115 pcf and a compressive strength of 3500 psi. This is supported by W 18 x 35's spanning 40' bays which tie into an assortment of girders spanning 30'; W 24 x 55's being the most typical. Composite action

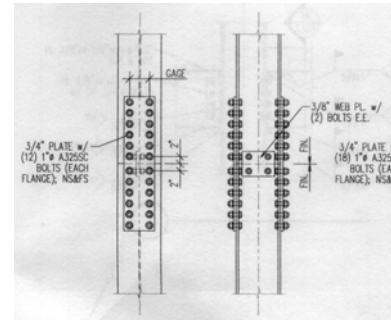


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is achieved through 6" long $\frac{3}{4}$ " diameter headed studs, approximately 34 evenly spaced per beam. The W 18's feature a typical camber of 1.5". Variations in design occur at architectural features, the elevator shafts, and intersections with the moment frames, elsewhere the system is nearly identical on all floors.

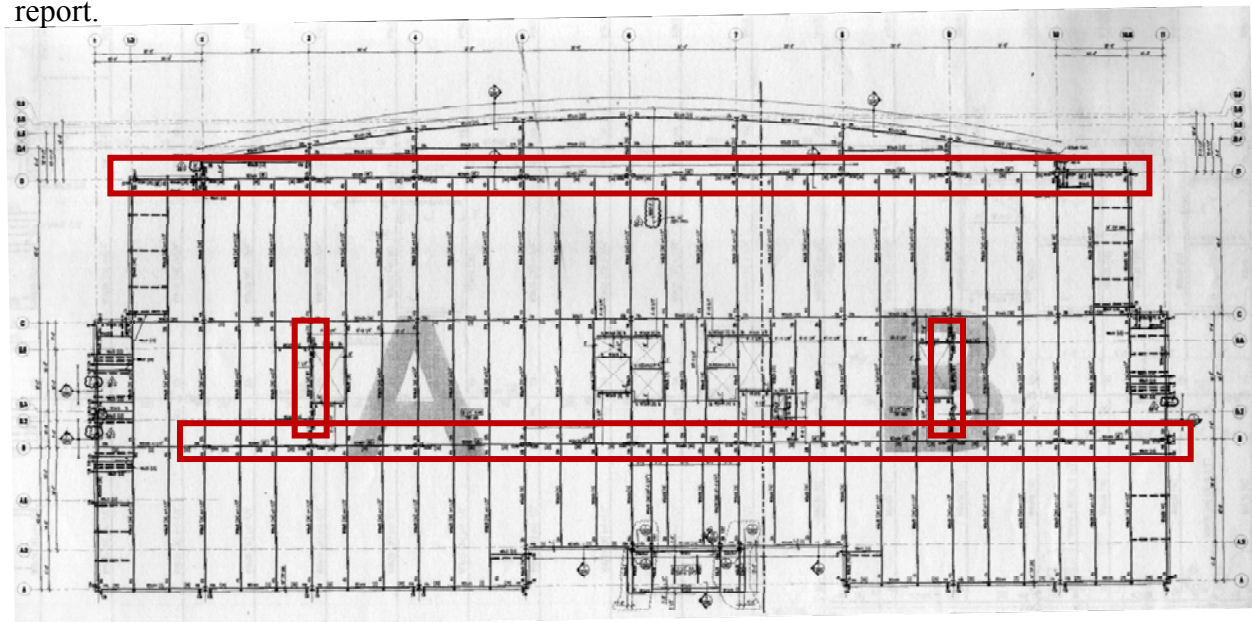
COLUMNS

The column grid for the building is laid out rectilinearly using three spans: 40', 35', 40', in the N-S direction and (10) 30' spans in the E-W, thereby creating large, uninterrupted, regular bays to simplify leasing. Column sizes vary between W 12 X 230's on the first floor of the moment frames to W 12 X 40's for gravity columns on the top floors. Splice levels are located a maximum of 4ft above the second and fourth floors. Typical columns are W 12 x 152's on the bottom floors, W 12 x 96's on the middle floors, and W 12 x 40' on the top levels. Typical columns are fixed to foundations with four $\frac{3}{4}$ " diameter anchor rods with 1' embed depths and 4" hooks.



LATERAL LOAD RESISTING SYSTEMS

1000 Continental Square is reinforced against lateral loads by different systems along its long axis (E-W) and short axis (N-S). In the E-W direction two moment frames fit into the existing grid along column lines B and D, and act over the full height of the building and effectively its full length. In the N-S direction two full height eccentrically braced frames fit off grid between lines B and C along column lines 3 and 9 to provide support for the short axis. These systems act to counter both wind and seismic forces, however wind loads were found to control the design in this situation. There are two additional types of one story braced frames used in the building to mainly support architectural elements which are not analyzed in this report.



II. CODES AND MATERIALS

CODES

Building Code:	2004 Pennsylvania Uniform Construction Code
Building Subcode:	International Building Code (IBC) 2003
Minimum Design Loads:	American Society of Civil Engineers (ASCE), 7-02
Reinforced Concrete:	American Concrete Institute (ACI), 318-02 Concrete Reinforcing Steel Institute, Manual of Standard Practice, 27 th Edition, March 2001
Precast Concrete:	Precast/Prestressed Concrete Institute (PCI), Design Handbook 5 th Edition
Steel Construction:	American Institute of Steel Construction (AISC), Manual of Steel Construction, LRFD, 3 rd Edition, 2001
Steel Decking:	Steel Deck Institute, Design Manual

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MATERIALS

Cast in place concrete (normal weight 145 pcf)

Footings		3,000 psi
Topping slabs		3,000 psi
Lightweight slabs on metal deck (115 pcf)		3,500 psi
Normal weight slabs on metal deck		3,500 psi
Slabs on grade		4,000 psi
Walls and piers		4,000 psi
Cast in Place on precast		5,000 psi
Pourable fill		1,000 psi

Precast Concrete (normal weight 145 pcf)

Structural precast		5,000 psi
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Reinforcing Steel

All types U.N.O.	ASTM A615	60,000 psi
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Structural Steel

W Shapes	ASTM A992	50,000 psi
Channels, angles, and plates	ASTM A36	36,000 psi
Round pipes	ASTM A53 E or S	35,000 psi
Square and Rectangular HSS's	ASTM A500	46,000psi

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III. DESIGN LOADS

LIVE LOADS

All floors	100 psf	Due to the open floor plan, all areas are assumed to be lobby or corridor space
Roof	20 psf	Standard flat roof loading
Snow load	21 psf	From ASCE 7-05 (see below)

$p_f = 0.7C_eC_tI_p g$		Equation 7-1
Terrain Category	B	Section 6.5.6.2
Exposure	Partially	Table 7-2 Footnote
C_e	1.0	Table 7-2
C_t	1.0	Table 7-3
I	1.0	Table 7-4
p_g	30psf	Figure 7-1

DEAD LOADS

Floor self weight	50 psf	From steel deck manufacturer's design tables
Roof self weight	5 psf	From steel deck manufacturer's design tables
Arch. Precast Panels	50 psf	Material property
Superimposed DL	30 psf	(see below)

MEP	20 psf
Ceiling Finishes	5 psf
Floor Finishes	5 psf

WIND LOADS

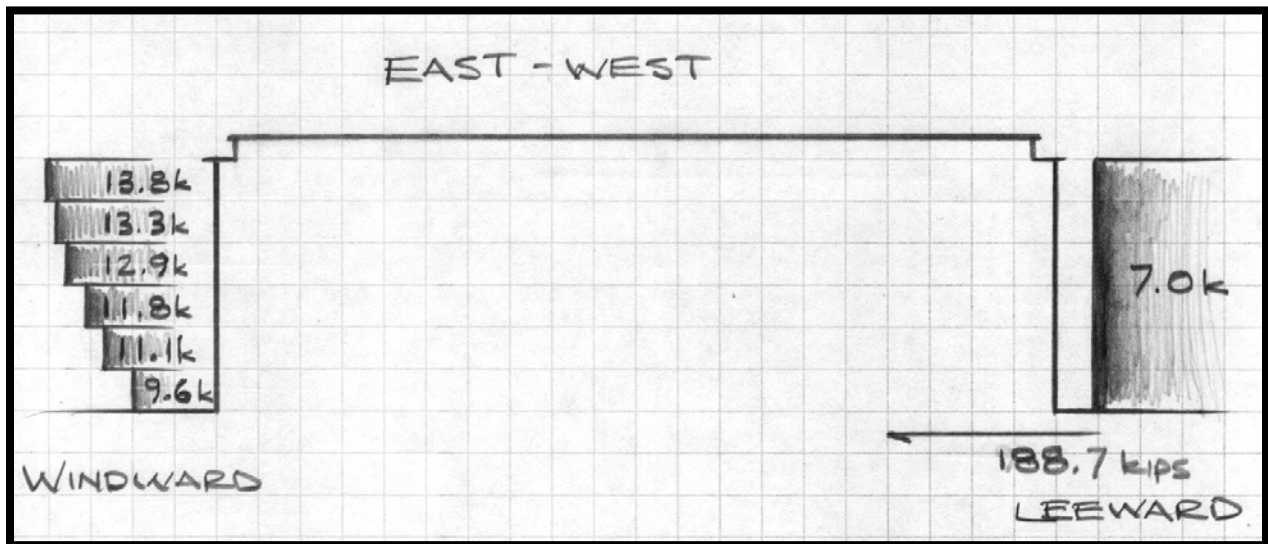
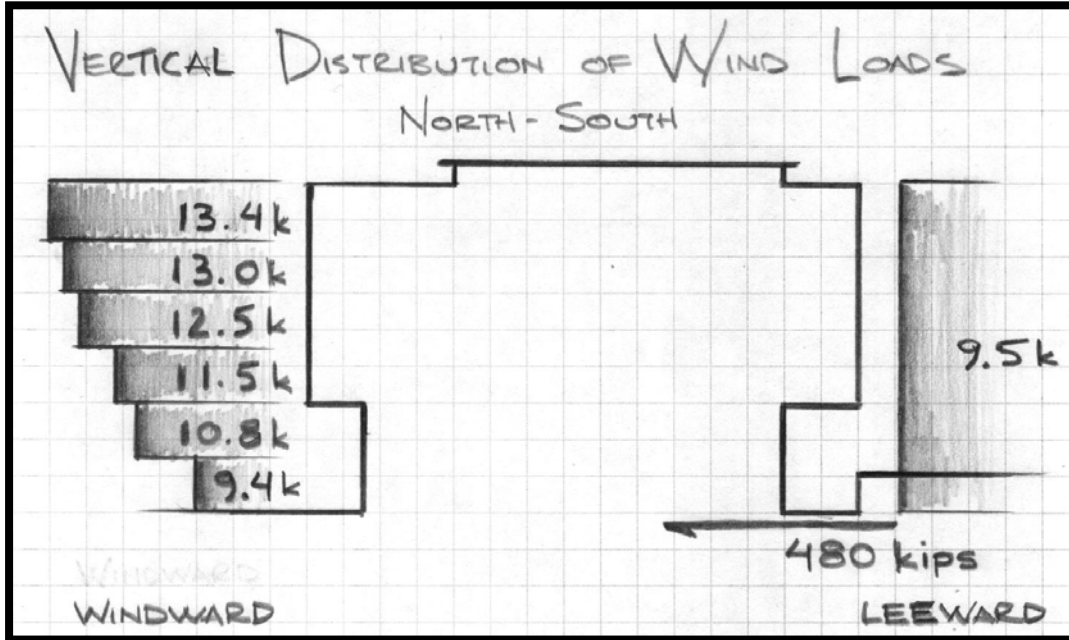
Basic Wind Speed	90 mph
Exposure Category	B
Enclosure Category	Enclosed
Wind Directionality Factor (K _d)	0.85
Importance Factor (I)	1.0
Topographic Factor (K _z t)	1.0
Gust Effect Factor (G)	0.828 (E-W) or 0.798 (N-S)
Internal Pressure Coefficient	± 0.18

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VERTICAL DISTRIBUTION OF WIND LOADS			
E-W DIRECTION			
Height (ft)	Windward	Leeward	Total (psf)
	Pressure (psf)	Pressure (psf)	
13	9.61	7.03	16.64
26	11.12	7.03	18.15
39	11.82	7.03	18.85
52	12.87	7.03	19.90
65	13.34	7.03	20.37
78	13.81	7.03	20.84
N-S DIRECTION			
Height (ft)	Windward	Leeward	Total (psf)
	Pressure (psf)	Pressure (psf)	
13	9.36	9.50	18.86
26	10.83	9.50	20.33
39	11.50	9.50	21.00
52	12.51	9.50	22.01
65	12.96	9.50	22.46
78	13.42	9.50	22.92

WIND LOAD SUMMARY		
East - West Direction	Base Shear: 188.68 kips	Overturning Moment: 7,962.16 kip-ft
North - South Direction	Base Shear: 479.33 kips	Overturning Moment: 8,805.83 kip-ft

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SEISMIC LOADS

Item	Design Value		Code Basis (ASCE 7-05)
	E-W	N-S	
Hazard Exposure Group	I		Table 1-1
Performance Category	B		Table 11.6-1,2
Importance Factor (I)	1.00		Table 11.5-1
Spectral Acceleration for Short Periods (S_S)	0.278		Figure 22-1
Spectral Acceleration for One Second Periods (S_1)	0.06		Figure 22-2
Damped Design Spec. Resp. Acc. at Short Periods (S_{DS})	0.2224		Section 11.4.4
Damped Design Spec. Resp. Acc. at One Second Periods (S_{D1})	0.068		Section 11.4.4
Seismic Response Coefficient (C_S)	0.0635	0.0278	Section 12.8.1.1
Soil Site Class	C		Section 20.3.3
Basic Structural System	Comp. Steel		
Seismic Resisting System	OSMF	CEBF	
Response Modification Factor (R)	3.5	8	Table 12.2-1
Deflection Modification Factor (C_d)	3	4	Table 12.2-1
Analysis Procedure Utilized	Equiv. Lat. Force		
Design Base Shear	420 kips		

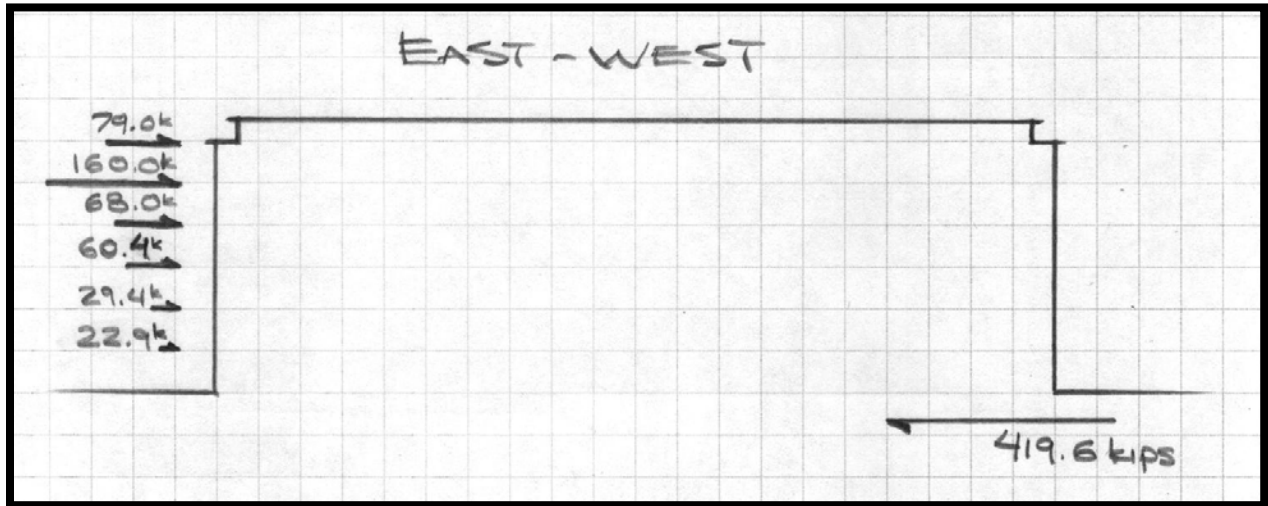
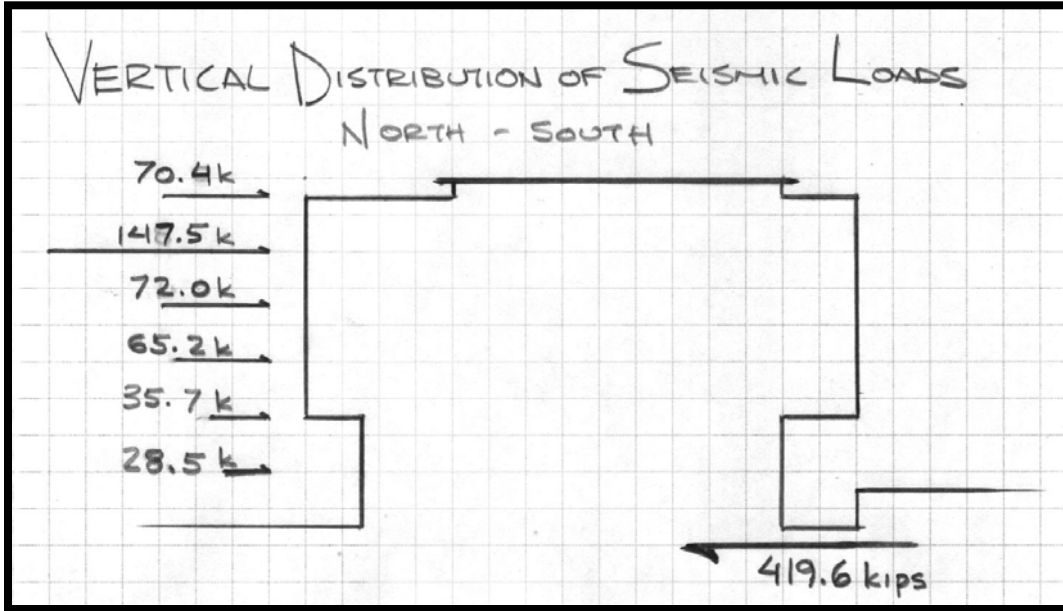
VERTICAL DISTRIBUTION OF SEISMIC FORCES

Height (ft)	E-W DIRECTION	N-S DIRECTION
	Story Shear (kips)	
0	419.60	419.60
13	396.68	390.68
26	367.24	355.00
39	306.88	289.85
52	238.90	217.87
65	79.01	70.36

SEISMIC LOAD SUMMARY

Base Shear: 419.60 kips	Overturning Moment: 42,209.27 kip-ft
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IV. LATERAL SYSTEM ANALYSIS AND DESIGN

LOAD ANALYSIS

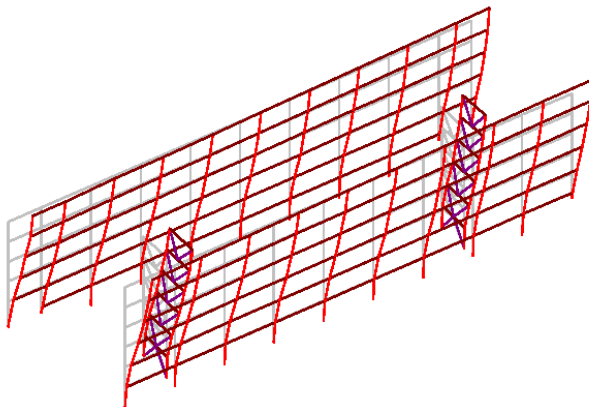
Through the load calculations above it can be determined that wind controls the lateral design along the short axis of the building and seismic controls along the long axis. Tributary area and the loadings above were used to determine lateral forces and the resulting story shears. The controlling wind forces were then applied in the north-south direction which is supported by the two braced frames. Seismic forces were applied in the east-west direction which is supported by the two moment frames. Since the braced frames are identical they both carry half the applied loads; however, loads on the moment frames were distributed based upon relative stiffness of the two individual frames. The analysis of gravity loads was complete for the largest typical bay, 30' x 40', as that is assumed to be the worst case scenario. Distributed loads were then applied to the supporting beams which run in the long direction and are supported at each end by girders, according to tributary area. The beams were assumed simply supported, then solved for their end reactions which were applied to the girders as point loads, and then subsequently to the columns by the same method.

RAM MODEL

The lateral system analysis was aided by a RAM Structural System model. For simplicity only the lateral resisting elements and floor diaphragms were modeled. The steel members were modeled as they appear on structural drawings, and the floor system was added as a 3" LOK Flooring with 3.25" of lightweight cover and 5", 3/4" diameter shear studs. Additional columns and beams were added in order to create the slab edges however these were not assigned sizes so they do not affect the lateral analysis.

DRIFT

The analysis of total building drift was completed through the use of the RAM software. I placed the controlling load cases both seismic and wind into the software in their respective directions. Then analyzed drift at each corner of the building as well as the approximate center in order to achieve both the extreme values as well as an average. The computed values for drift were then compared to the code standard for serviceability $\Delta = H/400$ which comes out to 2.34" when computed for 1000 Continental Square. The recorded values, at roof level, at all five points and in both load cases were well under this standard.



STORY DRIFT

Individual story shears were checked in respect to seismic loading. Using the same five control points, the seismic drifts at each level were compared to the allowable story drift, $\Delta = 0.020 h_{sx}$, as given in table 12.12-1 in ASCE 7-05. All story drifts fell below their respective limit values. The exact values can be seen in appendix A.2.

OVERTURNING MOMENT

Overturning moments were calculated by multiplying each seismic story force and wind load (after it had been distributed to its respective floor diaphragm through tributary area) by the height of that diaphragm. The resulting values for wind were 9439 ft-k (E-W) and 10448 ft-k (N-S), and 42209 ft-k for seismic. When compared to the moment created by the calculated seismic weight times the minimum moment arm from the center of mass to the most extreme member of that direction's respective lateral system, it is found that all values are within an acceptable range. The moment countering overturning is approximately 95000 ft-k in the north-south direction and over 2 million ft-k in the east-west direction. Obviously these moments would not actually be applied around a single point like they are assumed here but distributed throughout the structure; however, these calculations prove the weight of the building is enough to counter the overturning moment resulting from wind and seismic.

TORSION

Torsion in a building is a result of the eccentricity between the point where lateral loads are applied and the center of rigidity. This is to say the eccentricity between the center of mass and center of rigidity results in torsion from seismic loads, and similarly the eccentricity between the geometric center and center of rigidity results in torsion from wind. It can be assumed torsion has very little effect on the structure in the north-south direction because the centers of mass, rigidity, and geometry are within a foot of each other on every floor except the first and second. However, in the y direction greater eccentricities occur and thus the effect of torsional shears must be checked. This effect can be seen in the deflected shape of the lateral systems at roof level under seismic loads as shown below.



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The torsional shear calculations had to be preceded by the calculation of relative stiffness for each lateral resisting frame. This was accomplished using the RAM model by applying unit loads to each frame at each level of the structure and checking their respective deformations. Diaphragms were turned off to prevent interactions between different frames, and all stories below the one being checked were set as below ground to prevent their lateral deflection. The stiffness of each frame was determined by dividing the load by its deformation. Then these were summed for each level so the relative stiffness of each frame on each level to all the others could be determined. The results were that the brace frames generally are much stiffer than the moment frames. As expected since the bother braced frames are identical they are equally stiff and split the loads evenly between then. The moment frames on the other had are not identical but still relatively even except for the first floor where the northern moment frame is partially underground and thus takes 83 percent of the lateral loads for that floor.

RELATIVE STIFFNESS				
Floor	N-S		E-W	
	BF 1	BF 2	MF 3	MF 4
1	50.0 %	50.0 %	83.2 %	16.8 %
2	50.0 %	50.0 %	51.5 %	48.5 %
3	50.0 %	50.0 %	47.3 %	52.7 %
4	50.0 %	50.0 %	49.2 %	50.8 %
5	50.0 %	50.0 %	48.3 %	51.7 %
Roof	50.0 %	50.0 %	49.9 %	50.1 %

Once the relative stiffness of each frame is computed, torsional effects can be determined. As was stated earlier, due to its symmetry, the north-south direction is ignored. The formula for torsional shear in a direction is $F_i = VeR_iC / \sum RC^2$. Here V is the base shear in that direction, R_i is the relative stiffness of a frame, and C is the perpendicular distance to the centers of geometry or rigidity depending on whether the load is wind or seismic.

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TORSION FROM WIND											
				MF3				MF4			
Floor	V	COG, Y	e	R _i	C	RC ²	F _i	R _i	C	RC ²	F _i
1	29.85	57.00	3.59	83.2%	58.00	2797.43	0.53	16.8%	17.00	48.67	0.24
2	31.75	62.00	0.88	51.5%	53.00	1445.81	0.08	48.5%	22.00	234.88	0.23
3	33.25	62.00	0.87	47.3%	53.00	1328.17	0.07	52.7%	22.00	255.15	0.26
4	34.56	62.00	1.00	49.2%	53.00	1381.22	0.09	50.8%	22.00	246.01	0.30
5	35.36	62.00	0.83	48.3%	53.00	1358.05	0.08	51.7%	22.00	250.00	0.26
Roof	17.88	62.00	0.64	49.9%	53.00	1402.65	0.03	50.1%	22.00	242.32	0.10

TORSION FROM SEISMIC											
				MF3				MF4			
Floor	V	COR, Y	e	R _i	C	RC ²	F _i	R _i	C	RC ²	F _i
1	22.92	94.85	38.08	83.2%	20.15	337.64	4.16	16.8%	54.85	506.70	1.84
2	29.44	80.85	18.11	51.5%	34.15	600.26	2.66	48.5%	40.85	809.82	2.41
3	60.36	78.43	15.61	47.3%	36.57	632.34	4.63	52.7%	38.43	778.56	4.36
4	67.98	79.86	16.94	49.2%	35.14	607.18	5.66	50.8%	39.86	807.58	5.33
5	159.89	78.80	16.05	48.3%	36.20	633.55	12.77	51.7%	38.80	777.61	11.74
Roof	79.01	77.38	14.75	49.9%	37.62	706.70	6.22	50.1%	37.38	699.55	4.98

These results show relatively small shear effects from torsion. Wind effects are totally negligible, and in most cases seismic effects were less than 10% of the direct shear. However, the seismic effects should still be added to the direct shears in order to attain the total design shears. These are given in the table below.

DESIGN SHEAR IN EAST -WEST DIRECTION			
Floor	Direct Shear	Total MF3	Total MF4
1	22.92	23.21	5.70
2	29.44	17.82	16.70
3	60.36	33.17	36.18
4	67.98	39.08	39.88
5	159.89	90.07	94.33
Roof	79.01	45.68	44.54

V. CONCLUSIONS

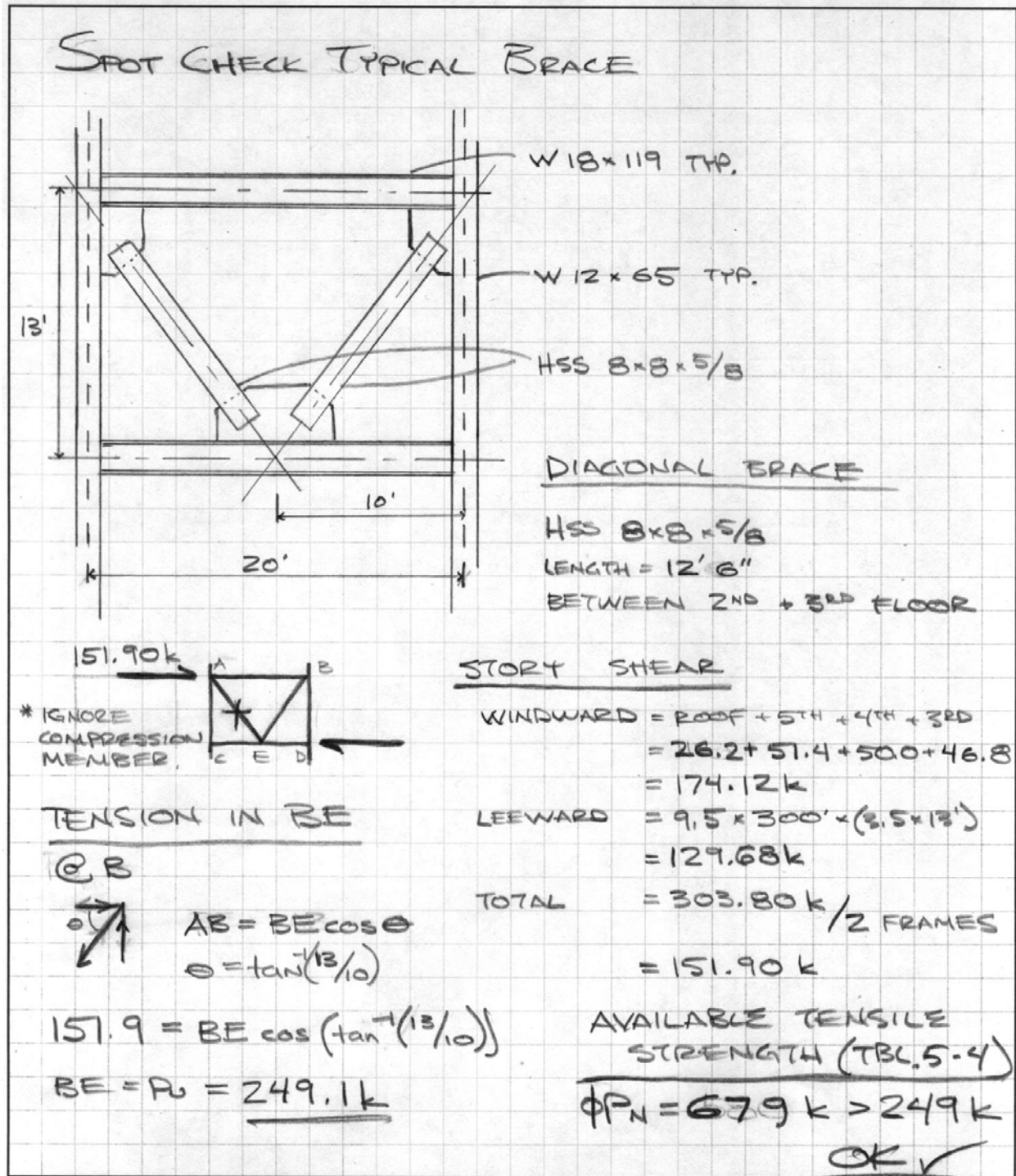
Through the analysis presented in this report the lateral force resisting system of 1000 Continental Square was investigated through a combination of hand calculations and computer modeling. The computer results were verified by comparison with common sense approximations and spot checks on typical members. The analysis was used to check both strength and serviceability, and results seem to conclude the building is well within code limits. The values found for drift, both overall and within each story, are acceptable by the standards set out in ASCE 7-05. Both overturning moments and torsional values are easily within the capacities of the structure; although, the effects of torsional shear had to be taken into consideration in the total shear in the east – west direction. Spot checks of member sizes seem to imply most members in the lateral system were sized to control deflection since their strength capacities are much higher than necessary. These oversized members could possibly be redesigned, perhaps with one of the alternate flooring systems from technical report two in order to create a more efficient use of material. This idea will be explored in my final thesis in the spring.



VII. APPENDICES

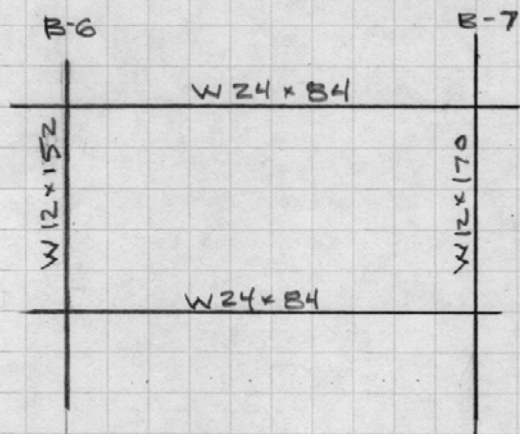
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A.1 SPOT CHECKS



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SPOT CHECK MOMENT FRAME



BETWEEN 2ND + 3RD FLOORS

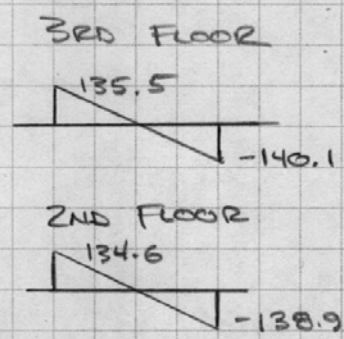
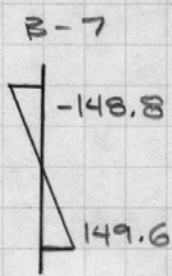
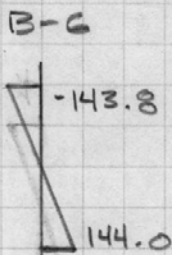
AXIAL LOAD

COLUMNS FROM TECH

$$3 \times (220 \text{ PSF} \times 37.5 \times 30) + (102 \text{ PSF} \times 37.5 \times 30) =$$

$$P_u = 860 \text{ kips}$$

BENDING MOMENTS (FROM RAM MODEL)



COMBINED STRENGTH (TBL 6-1)

W 12x152	KL = 13'	P = 0.592 x 10 ³	b _x = 0.987 x 10 ³
W 12x170	KL = 13'	P = 0.528 x 10 ³	b _x = 0.870 x 10 ³
W 24x84	KL = 30'	P = 6.11 x 10 ³	b _x = 3.11 x 10 ³

NO AXIAL → DIAPHRAGM TAKES AXIAL LOAD

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CHECK B-6

$$p \cdot P_u + b_x M_{ux} = 0.592 \times 10^{-3} (860k) + 0.987 \times 10^{-3} (144 \text{ ft}\cdot\text{k})$$
$$= 0.65 < 1.0 \quad \underline{\text{OK}} \checkmark$$

CHECK B-7

$$p \cdot P_u + b_x M_{ux} = 0.528 \times 10^{-3} (860k) + 0.870 \times 10^{-3} (149.6 \text{ ft}\cdot\text{k})$$
$$= 0.58 < 1.0 \quad \underline{\text{OK}} \checkmark$$

CHECK BEAMS

$$W 24 \times 84 \quad L_b = 10' \quad \phi M_N = 840 \text{ ft}\cdot\text{kips}$$

$$\checkmark 840 > 140.1 \quad \text{3RD FLOOR} \quad \underline{\text{OK}} \checkmark$$

$$840 > 138.9 \quad \text{2ND FLOOR} \quad \underline{\text{OK}} \checkmark$$

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A.2 DRIFTS



RAM Frame v11.0
 DataBase: 1000 Continental 3
 Building Code: IBC

Drift

01/02/08 09:13:24
 Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
 Member Force Output: At Face of Joint
 P-Delta: Yes Scale Factor: 1.00
 Diaphragm: Rigid
 Ground Level: Base

LOAD CASE DEFINITIONS:

W1 Wind N-S W_User
 E1 Seismic E-W EQ_User

RESULTS:

Location (ft): (0.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X in	Y in	X in	Y in	X	Y
Roof	W1	-0.0015	0.3745	0.0001	0.0759	0.0000	0.0005
	E1	1.8651	-0.4211	0.2172	-0.0602	0.0014	0.0004
5th Floor	W1	-0.0015	0.2986	-0.0002	0.0812	0.0000	0.0005
	E1	1.6479	-0.3610	0.3440	-0.1011	0.0022	0.0006
4th Floor	W1	-0.0013	0.2175	0.0000	0.0752	0.0000	0.0005
	E1	1.3039	-0.2599	0.4010	-0.0996	0.0026	0.0006
3rd Floor	W1	-0.0013	0.1423	-0.0000	0.0694	0.0000	0.0004
	E1	0.9029	-0.1603	0.4842	-0.0748	0.0031	0.0005
2nd Floor	W1	-0.0013	0.0729	-0.0013	0.0729	0.0000	0.0002
	E1	0.4187	-0.0855	0.4187	-0.0855	0.0013	0.0003
1st Floor	W1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Location (ft): (300.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X in	Y in	X in	Y in	X	Y
Roof	W1	-0.0015	0.3699	0.0001	0.0761	0.0000	0.0005
	E1	1.8651	0.4181	0.2172	0.0614	0.0014	0.0004

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Story	LdC	Displacement		Story Drift		Drift Ratio	
5th Floor	W1	-0.0015	0.2938	-0.0002	0.0804	0.0000	0.0005
	E1	1.6479	0.3567	0.3440	0.1001	0.0022	0.0006
4th Floor	W1	-0.0013	0.2134	0.0000	0.0752	0.0000	0.0005
	E1	1.3039	0.2566	0.4010	0.0995	0.0026	0.0006
3rd Floor	W1	-0.0013	0.1383	-0.0000	0.0692	0.0000	0.0004
	E1	0.9029	0.1571	0.4842	0.0748	0.0031	0.0005
2nd Floor	W1	-0.0013	0.0690	-0.0002	0.0565	0.0000	0.0004
	E1	0.4187	0.0823	0.3454	0.0636	0.0022	0.0004
1st Floor	W1	-0.0011	0.0126	-0.0011	0.0126	0.0000	0.0001
	E1	0.0734	0.0187	0.0734	0.0187	0.0005	0.0001

Location (ft): (10.000, 115.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X in	Y in	X in	Y in	X	Y
Roof	W1	0.0003	0.3744	-0.0000	0.0759	0.0000	0.0005
	E1	1.5433	-0.3932	0.1706	-0.0561	0.0011	0.0004
5th Floor	W1	0.0003	0.2985	0.0001	0.0811	0.0000	0.0005
	E1	1.3728	-0.3371	0.2669	-0.0944	0.0017	0.0006
4th Floor	W1	0.0003	0.2174	0.0000	0.0752	0.0000	0.0005
	E1	1.1059	-0.2427	0.3246	-0.0930	0.0021	0.0006
3rd Floor	W1	0.0003	0.1422	0.0000	0.0694	0.0000	0.0004
	E1	0.7812	-0.1497	0.4268	-0.0698	0.0027	0.0004
2nd Floor	W1	0.0002	0.0728	-0.0000	0.0569	0.0000	0.0004
	E1	0.3544	-0.0799	0.2966	-0.0594	0.0019	0.0004
1st Floor	W1	0.0002	0.0159	0.0002	0.0159	0.0000	0.0001
	E1	0.0578	-0.0206	0.0578	-0.0206	0.0004	0.0001

Location (ft): (290.000, 115.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X in	Y in	X in	Y in	X	Y
Roof	W1	0.0003	0.3700	-0.0000	0.0761	0.0000	0.0005
	E1	1.5433	0.3901	0.1706	0.0574	0.0011	0.0004

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Story	LdC	Displacement		Story Drift		Drift Ratio	
5th Floor	W1	0.0003	0.2940	0.0001	0.0804	0.0000	0.0005
	E1	1.3728	0.3328	0.2669	0.0934	0.0017	0.0006
4th Floor	W1	0.0003	0.2136	0.0000	0.0752	0.0000	0.0005
	E1	1.1059	0.2394	0.3246	0.0929	0.0021	0.0006
3rd Floor	W1	0.0003	0.1384	0.0000	0.0692	0.0000	0.0004
	E1	0.7812	0.1465	0.4268	0.0698	0.0027	0.0004
2nd Floor	W1	0.0002	0.0692	-0.0000	0.0565	0.0000	0.0004
	E1	0.3544	0.0767	0.2966	0.0594	0.0019	0.0004
1st Floor	W1	0.0002	0.0127	0.0002	0.0127	0.0000	0.0001
	E1	0.0578	0.0173	0.0578	0.0173	0.0004	0.0001

TECHNICAL REPORT 3



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Drift

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CRITERIA:

Rigid End Zones: Ignore Effects
 Member Force Output: At Face of Joint
 P-Delta: Yes Scale Factor: 1.00
 Diaphragm: Rigid
 Ground Level: Base

LOAD CASE DEFINITIONS:

W1 Wind N-S W_User
 E1 Seismic E-W EQ_User

RESULTS:

Location (ft): (150.000, 60.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X in	Y in	X in	Y in	X	Y
Roof	W1	-0.0005	0.3722	0.0000	0.0760	0.0000	0.0005
	E1	1.6972	-0.0015	0.1929	0.0006	0.0012	0.0000
5th Floor	W1	-0.0006	0.2962	-0.0001	0.0808	0.0000	0.0005
	E1	1.5043	-0.0021	0.3038	-0.0005	0.0019	0.0000
4th Floor	W1	-0.0005	0.2155	0.0000	0.0752	0.0000	0.0005
	E1	1.2006	-0.0017	0.3611	-0.0001	0.0023	0.0000
3rd Floor	W1	-0.0005	0.1403	-0.0000	0.0693	0.0000	0.0004
	E1	0.8394	-0.0016	0.4543	0.0000	0.0029	0.0000
2nd Floor	W1	-0.0005	0.0710	-0.0001	0.0567	0.0000	0.0004
	E1	0.3852	-0.0016	0.3199	0.0000	0.0021	0.0000
1st Floor	W1	-0.0004	0.0143	-0.0004	0.0143	0.0000	0.0001
	E1	0.0653	-0.0016	0.0653	-0.0016	0.0004	0.0000

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A.3 CENTERS

CENTER OF MASS			
Floor	X Value	Y Value	Height
1	153.22	58.58	13
2	150.10	62.87	26
3	150.21	62.84	39
4	150.38	62.92	52
5	150.36	62.75	65
Roof	150.12	62.63	78
Total	150.7317	62.09833	45.5

CENTER OF RIGIDITY			
Floor	X Value	Y Value	Height
1	164.82	94.85	13
2	152.27	80.85	26
3	150.97	78.43	39
4	150.65	79.86	52
5	150.53	78.80	65
Roof	150.33	77.38	78
Total	153.2617	81.695	45.5

CENTER OF GEOMETRY			
Floor	X Value	Y Value	Height
1	150	57	13
2	150	62	26
3	150	62	39
4	150	62	52
5	150	62	65
Roof	150	62	78
Total	150	61.16667	45.5

A.4 DEFLECTIONS UNDER UNIT LOADING

DEFLECTION				
Floor	BF 1	BF 2	MF 3	MF 4
1	0.00052	0.00052	0.00032	0.00158
2	0.00083	0.00083	0.00198	0.00210
3	0.00097	0.00097	0.00291	0.00261
4	0.00113	0.00113	0.00276	0.00267
5	0.00171	0.00171	0.00328	0.00307
Roof	0.00243	0.00243	0.00381	0.00380