November 28, 2007

LOCKWOOD PLACE
BALTIMORE, MD

[TECHNICAL ASSIGNMENT 3]

Monica Steckroth
Structural Option
Faculty Consultant: Dr. Linda Hanagan

Executive Summary

Lockwood Place in Baltimore, Maryland is a thirteen story, one hundred and ninety four foot, mixed-use development building utilized essentially for retail and corporate businesses. The building enclosure is primarily made of steel with a glass curtain wall façade. Directly adjacent to the building sits a covered mall area and a parking garage. The parking garage connects to the second level of Lockwood Place through a corridor and lobby. Gravity framing consists of a composite steel system and lateral framing is comprised of both eccentric braces and moment frames.

The goal of this report is to conduct an in-depth study of Lockwood Place's lateral load resisting system. The study was completed through various hand calculations that were verified by computer modeling in SAP2000 and RAM Structural Systems. Evaluation of the lateral system was determined through story shear distributions, drift analysis, and simple member spot checks.

The controlling load case was determined to be wind forces in both the north/south and east/west directions. Each floor was assumed to be a rigid diaphragm that distributed story shears to each frame according to relative stiffness. Eccentric braces provided greater stiffness than moment frames. Uneven spacing of eccentric braces in the north/south direction created the center of rigidity to be greatly offset from the center of mass. Torsional shears became a significant factor in the north/south direction. Frames in the east/west are evenly spaced and relatively close to the center of rigidity, eliminating significant torsional effects.

Drift for the entire building was determined to be within limits. Wind controlled drifts with total drifts of 2.99" in the north/south direction and 2.63" in the east/west direction. Limitations were evaluated as H/400 for wind and 0.002hx for seismic.

Simple spot checks were used to evaluate the strength of lateral system members. All members were determined acceptable. While a column proved to have adequate capacity, an eccentric brace and beam had more capacity than required to support the given load. Serviceability (drift) controlled the design in the eccentric brace, as is expected in midrise buildings.

TABLE OF CONTENTS

INTRODUCTION

As an expansion to the corporate/entertainment district of Baltimore's Inner Harbor, the Lockwood Place Office Building is located directly across from the National Aquarium. The building has a curved glass, curtain wall façade and abuts a covered mall area and an adjacent parking garage. It is comprised of thirteen floors and over 300,000 square feet of floor space.

At ground level, a visitor is welcomed by a grand lobby entrance. At the second level, a visitor has direct access to the adjacent parking garage. At the third level, tenants have the option to utilize two balcony spaces. Each floor is designed with large bay sizes, allowing for open floor plans. The spaces on the first two floors, occupied by retail tenants, rise to a combined height of 34 feet. The third through the twelfth floors are occupied by corporate tenants and each floor height is 13'-6". A penthouse is constructed on the thirteenth floor. The floor height is 18' and it sets back slightly from the rest of the building. Lockwood Place is designed to accommodate a range of tenants' needs, while providing a sleek exterior look with each story consisting of full height glass and large spans.

This report is an analysis and confirmation design study of Lockwood Place's lateral systems. Evaluation of the lateral system is done through drift analysis, story shear distribution and torsional effects, overturning moment, and member strength checks. Loads are obtained from Technical Assignment 1. Strength checks are in accordance with ASCE7-05 load combinations. Analysis was performed through comparison of models developed in RAM Structural Systems and SAP with hand calculations.

STRUCTURAL SYSTEM OVERVIEW

Floor System

500 East Pratt Street has a typical superstructure floor framing system made of composite steel beams and girders. The slab is 3-1/4” light weight concrete topping on 3”x20gage galvanized metal deck. For composite beam action, ¾” diameter by 5-1/2” long headed shear studs are used, conforming to ASTM A108, Grades 1010 through 1020. Typical bay sizes are 30’-0” x 30’-0” and 45’-0” x 30’-0.” Infill beams are spaced 10’-0” on center, framing into a typical girder size of W24x62. All steel conforms to ASTM A572, Grade 50, unless otherwise noted on the drawings. MEP systems are run through the structural framing system. Holes created in the beams and girders from the MEP systems are reinforced according to AISC Design Guide 2. A two hour fire rating is provided for all floor slabs, beams, girders, columns, roofs, and vertical trusses. For a more detailed description of atypical floor systems, please refer to Technical Assignment 1.

Roof System

At the penthouse level of Lockwood Place, the building steps back creating a high roof and a low roof. A third roof, the highest point of the building, is created by an extended machine room ceiling located at the penthouse level. The roof on the penthouse is sloped slightly downward into the machine room wall. While the framing of the penthouse floor is consistent with the typical building superstructure system, infill beam sizes are reduced due to smaller bay widths. All three roof systems are 1-1/2”x20ga. galvanized type ‘B’ metal deck. Infill beams are located at 6’ on center. Beam sizes range from W10x12 to W24x76 depending on their location.

Exterior slabs that are located at level twelve are 4-1/2” normal weight concrete topping on 3”x20gage galvanized composite metal deck. The slabs are reinforced with 6x6-W2.9xW2.9 W.W.F. Waterproofing is required for all exterior slabs.

A screen wall is located on level twelve to disguise mechanical equipment. A canopy extends over a balcony on the twelfth floor. The canopy is also made of 1-1/2”x20gage galvanized type ‘B’ metal deck.

Lateral System

Lockwood Place’s lateral system is comprised of moment frames and eccentric braced frames. Moment frames run both east/west and north/south directions. Eccentric braced frames are located around the elevators/elevator lobby. Sizes of the braces range from W14x90 at the base of the building to W8x31 at the top of the building and are pinned connections. Lateral loads were distributed based on the rigidity of each frame. Columns that have eccentric braces framed into them are designed to be fixed to their supports at the base of the building. All other columns are designed to have pinned bases. View lateral system plan and elevations in Figure 1 and Figure 2 below.

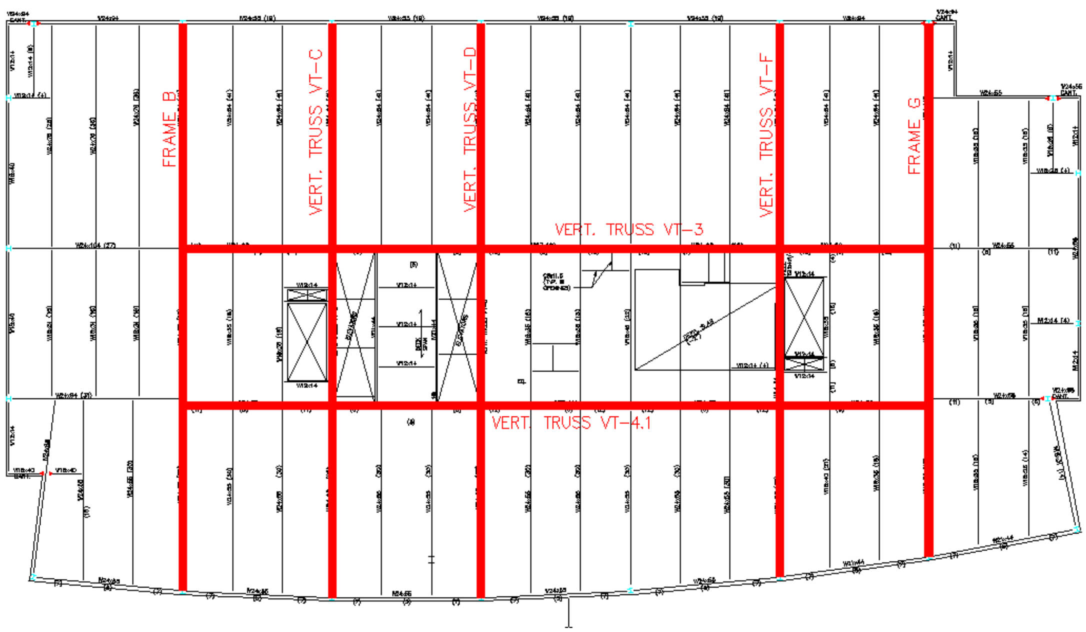


Figure 1. Lateral System Plan

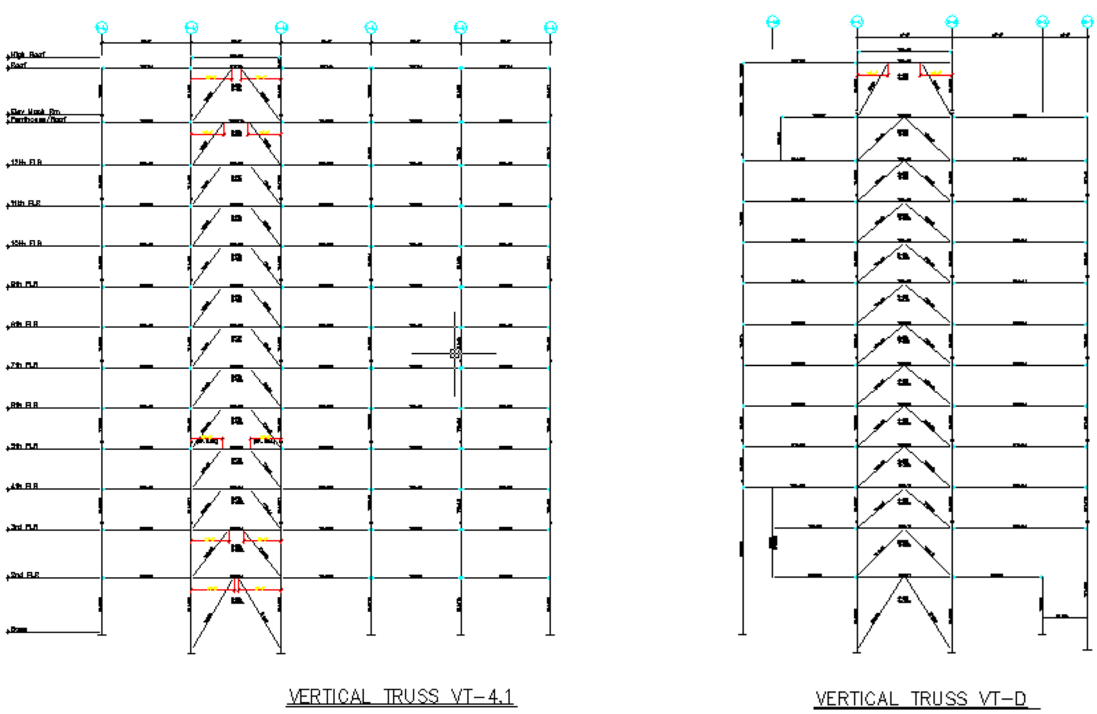


Figure 2. Lateral System Elevation

Foundation

Located along Baltimore's Inner Harbor, Lockwood Place's soils consist of existing man-made fill. The maximum soil bearing pressure for spread footings is 1000psf. The foundation system is made of drilled caissons to accommodate for this bearing capacity. Caisson shaft diameters range from 2'-6" to 6'-0." Typically, they extend a minimum of 1'-0" into Gneiss bedrock and have a minimum concrete compressive stress of 4500psi.

Grade beams travel between pile caps and have a minimum concrete compressive strength of 4000psi. Each grade beam ranges in size from 18"x24" to 24"x42" and is reinforced with top and bottom bars.

CODES & LOAD COMBINATIONS

Codes utilized in this report:

- Design Standards
  American Society for Civil Engineers (ASCE7-05)
  Design Code for Minimum Design Loads

- Structural Steel
  American Institute of Steel Construction (AISC)
  ASD Specifications for Structural Steel Design – Unified Version, 2005

- Structural Concrete
  American Concrete Institute
  Specification for Reinforced Concrete and Masonry Structures, 2005

ASD Load Combinations applied:

Dead + Live
Dead + Wind
Dead + (0.75Wind or 0.7Earthquake) + 0.75Live + 0.75(Snow or Roof Live)
0.6Dead +Wind
0.6Dead + 0.7Earthquake

*Fluid pressure, earth pressure, self-straining, and rain load are omitted from the equations.

BUILDING LOAD SUMMARY

Gravity Loads

The loads for Lockwood Place are presented in an abbreviated form below. The loads are accumulated from The Maryland Building Code Performance Standard. Design loads from the engineer of record and those of the building code are shown in comparison.

Dead Load

DEAD LOAD (psf)							
Location/Loading	Office	Lobby/ Corridor	Machine Room	Retail	1st Floor Lobby	Balconies	Roof
Concrete Slab	46	46	46	63	63	63	-
Metal Deck	2	2	2	-	-	2	2
Pavers/ W.P.	-	-	-	-	-	2	2
M/E/C/L	8	8	8	-	-	8	8
Roofing	-	-	-	-	-	2	2
Insulation	-	-	-	-	-	2	2
Total Dead Load	56	56	56	63	88	115	14

Live Load

LIVE LOAD (psf)		
Location	Design Load	Minimum Required
Office	100	50 for offices only
Lobby/Corridor	100	100 first level, 80 above first level
Machine Room	125	125
Retail	100	100 first level, 75 above first level
1st Floor Lobby	100	100
Balconies	100	100 exterior
Roof	30	20 assuming no reduction

It is a conservative assumption to use an unreduced roof live load. Given that the front of the building is a curved radius, there is great variation in tributary areas among roof members. In many cases in the southern half of the building, the tributary area is too small to be reduced. To simplify the design, no live loads were reduced on the roof.

Wall Load
The building exterior is made of metal faced composite wall panels glazed into a glass curtain wall system. The estimated wall weight is 25psf. This weight is used to determine the building's seismic base shear.

Snow Load

General Information		
Ground Snow Load	P_g	25psf
Exposure Factor	C_e	0.9
Thermal Factor	C_t	1.0
Importance Factor	I_s	1.0
Minimum Flat Roof Snow Load	P_f	**22.5psf**

Multiple step backs among the roofs set precedence for snow drift overloading. Detailed snow drift calculations are not in the scope of this report. To view these calculations refer to Technical Assignment 1.

Retaining Wall Parameters
Equivalent At-Rest Earth Pressure…………………60pcf
Equivalent Active Earth Pressure………………….. 45pcf
Equivalent Passive Earth Pressure…………………275pcf
Bulk Density (Wet)…………………………………120pcd
Angle of Internal Friction (Original)………..........16 degrees

Lateral Loads

Wind Load

Determination of wind and loading was carried out in accordance with Section 6 of ASCE7-05. All factors were based on location and geometry of the building. Standardization of the curved façade was assumed. The north/south dimension of the building was taken from the largest dimension in the curve. A table is provided to summarize values used in calculations. Calculations can be found in the Appendix.

General Information	
Building Category	II
Importance Factor, I	1.0
Exposure Category	D
Kd	0.85
Topographic Factor, kzt	1.0
V (mph)	100
Period (T)	1.04
Gust Effect Factor	0.90/0.88
Cp Windward	0.80
Building Height, hn	194
X	0.75
Frequency, n1	0.96
North/South Length	118.6
East/West Length	218.3
Enclosure Classification	Fully Enclosed

Seismic Loads

Determination of seismic loading was carried out in accordance with Section 9 of ASCE7-05. The geotechnical report was not available for this report. Lockwood Place was assumed to fall into Site Class B. This data was obtained through government earthquake hazard maps. These maps can be found at http://earthquake.usgs.gov/research/hazmaps/design. The weight of the building is based on the structural framing and additional dead loads of the building. The table below summarizes information used in calculation. Calculations can be found in the Appendix.

General Information		
Occupancy Type		II
Seismic Use Group		I
Site Class		B
Seismic Design Category		A
Short Period Spectral Response	Ss	0.170
Spectral Response at 1 Second	S1	0.051
Maximum Short Period Spectral Response	Sms	0.170
Maximum Spectral Response at 1 Second	Sm1	0.051
Design Short Period Spectral Response	SDS	0.113
Design Spectral Response at 1 Second	SD1	0.034
Response Modification Coefficient	R	3
Seismic Response Coefficient	Cs	0.01
Effective Period	T	1.04
Height Above Grade	hn	194

LATERAL LOAD DISTRIBUTION & ANALYSIS

Story Shear & Overturning Moment

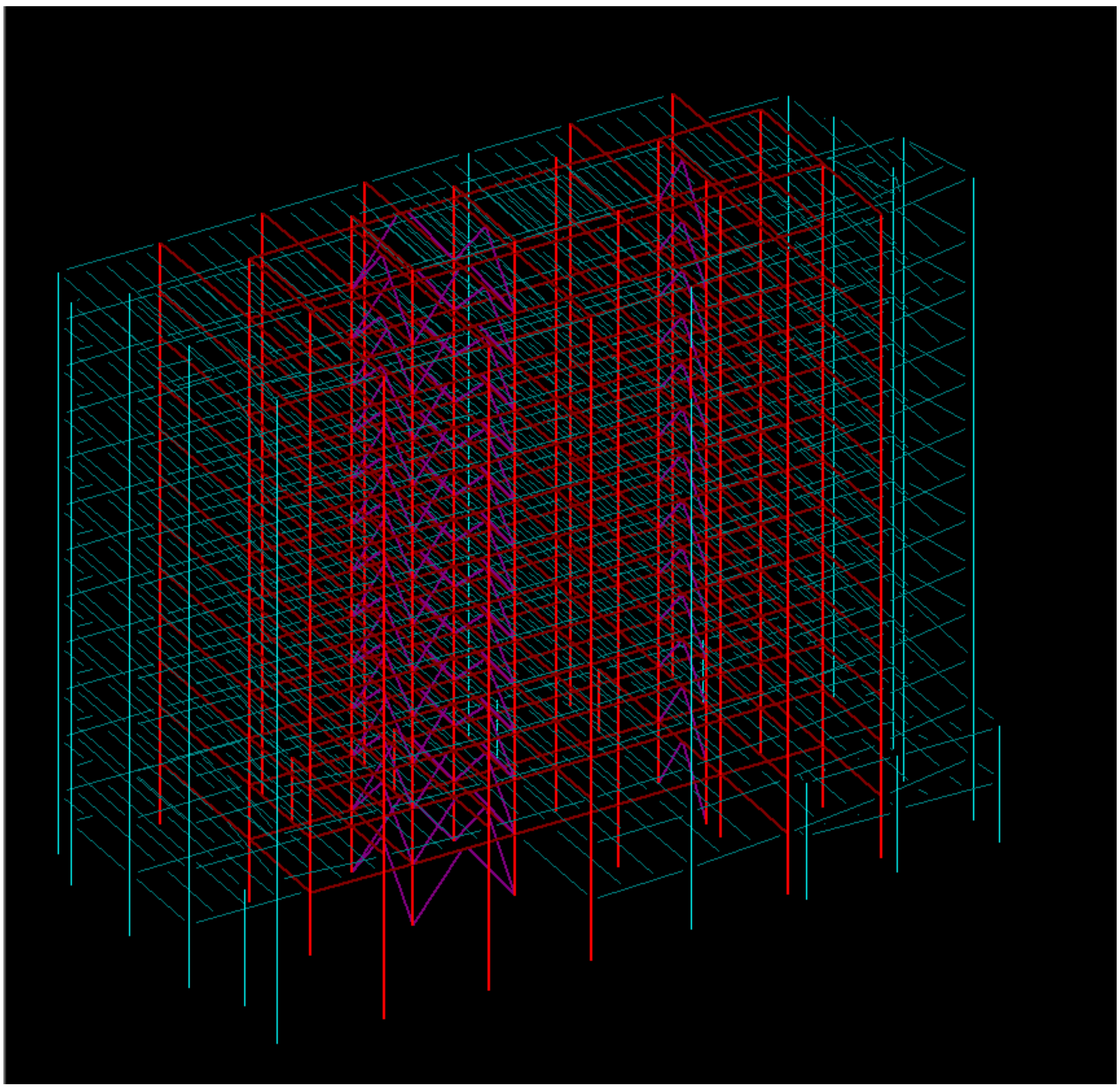
Lateral Loads are accumulated from Technical Assignment 1. Wind story shears are based on differing wind pressures at each story level. Seismic story shears are based on the height and weight of each level. A summary of story shears is provided below. Calculations can be found in the Appendix. Based on the story shears below, wind is determined to be the governing lateral force in both north/south and east/west directions.

Story Shear			
	Wind		Seismic
Level	North/South	East/West	
2	1547.75	736.04	275.27
3	1422.6	707.5	274.04
4	1309.12	651.56	268.46
5	1202.39	600.43	234.57
6	1093.12	552.1	184.47
7	982.47	502.43	143.59
8	869.51	452.01	108.84
9	754.69	400.38	79.85
10	638.96	347.75	56.2
11	521.61	294.64	37.46
12	403.57	240.67	23.17
Penthouse	279.66	186.33	12.86
Low Roof	134.24	129.19	6
High Roof	26.85	62.01	1.64
Base Shear	1547.75k	736.04k	275.27k
Overturning Moment	851,614.2'k	370,300.0'k	34,683.1'k

Center of Mass

To determine the center of mass at each floor level, a three dimensional model was developed in RAM Structural System. Both floor openings and the curvature of the building face were accounted for in the model. The center of mass for each floor level is summarized in the table below. The distances are taken from building line intersection of A.1 and 1.

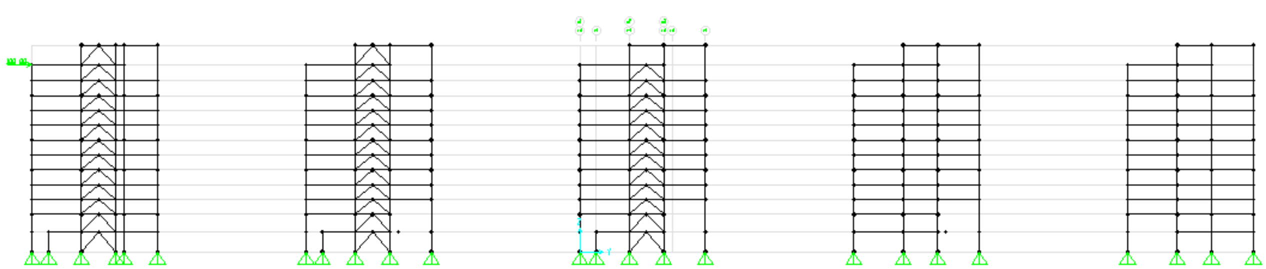
Center of Mass		
Level	COMx (ft.)	COMy (ft.)
2	114.0	56.0
3	116.0	50.7
4	105.0	55.9
5 to 11	105.0	55.8
12	105.0	55.8
Penthouse	108.3	58.0
Low Roof	101.0	80.4



Center of Rigidity

All floors in Lockwood Place are assumed to have a rigid diaphragm. Story shears become distributed according to relative stiffness. A SAP model was developed to determine the relative stiffness of each lateral load resisting frame. A unit load was applied at each floor level, in two dimensional frames aligned with one another. Relative stiffness was determined by summing the shear forces in the members in each frame at each level and dividing those shear forces by the total unit load applied to that level. The center of rigidity was then determined for each level. The results are displayed in the chart below. Calculations are found in the Appendix. The distances are taken from building line intersection of A.1 and 1.

Center of Rigidity		
Level	X (ft.)	Y (ft.)
2	116.56	60.93
3	116.62	60.93
4	117.18	60.93
5	117.27	60.93
6	117.18	60.93
7	117.17	60.93
8	117.27	60.93
9	117.32	60.93
10	117.37	60.93
11	117.39	60.93
12	117.44	60.93
Penthouse	117.44	60.93
Roof	117.39	60.93



North/South Frames - Unit Load at Penthouse Level

Torsion

In addition to calculation of direct shear on each frame, torsional effects were considered in this report. Total shear was determined through the addition of direct and torsional shear. Direct shear forces were calculated from the story shears and relative frame stiffness previously discussed. After reviewing the results located in the tables below, consideration of torsion becomes significant in the north/south direction. Torsional shear contributed 5-10% of total shear at the base of the frames and up to 40% of total shear near the peak height of the frames. In the east/west direction torsion was not as significant due to evenly spaced frames, relatively small eccentricities between the center of mass and center of rigidity, and a shorter building width. Detailed calculations for torsion can be found in the Appendix.

Direct Shear (kip)							
	North/South					East/West	
Level	Frame B	VT-C	VT-D	VT-F	Frame G	VT-3	VT-4.1
2	71.82	453.18	453.18	528.56	41.02	363.75	372.29
3	66.01	415.97	415.97	482.12	40.83	349.65	357.85
4	62.05	383.05	383.05	442.22	38.75	322.00	329.56
5	57.11	351.82	351.46	405.93	36.07	296.73	303.70
6	51.70	319.63	319.63	368.93	33.23	272.85	279.25
7	46.57	288.06	287.27	331.39	29.18	248.30	254.13
8	41.30	254.33	254.24	293.29	26.35	223.38	228.63
9	35.92	220.75	220.75	254.48	22.79	197.87	202.51
10	30.48	186.83	186.90	215.39	19.36	171.86	175.89
11	24.88	152.52	152.52	175.78	15.91	145.61	149.03
12	19.29	118.00	118.00	135.96	12.31	118.94	121.73
Penthouse	13.34	81.74	81.66	94.19	8.73	92.08	94.25
Roof	6.40	39.25	39.25	45.21	4.12	63.85	65.34

Torsional Shear (kip)							
	North/South					East/West	
Level	Frame B	VT-C	VT-D	VT-F	Frame G	VT-3	VT-4.1
2	7.52	30.01	12.55	-26.09	-3.60	8.14	-8.14
3	1.68	6.71	2.81	-5.78	-0.87	15.63	-15.63
4	30.23	118.48	50.37	-99.12	-15.58	13.41	-25.44
5	28.79	112.68	47.96	-93.87	-14.97	6.48	-6.48
6	25.86	101.49	43.14	-84.91	-13.72	5.95	-5.95
7	23.25	91.33	38.71	-76.18	-12.03	5.50	-5.50
8	20.80	81.39	34.67	-67.73	-10.92	4.96	-4.96
9	18.19	71.04	30.30	-58.99	-9.49	4.31	-4.31
10	15.50	60.41	25.81	-50.03	-8.08	3.78	-3.78
11	12.64	49.27	21.06	-40.78	-6.63	3.28	-3.28
12	9.83	38.25	16.37	-31.57	-5.14	2.75	-2.75
Penthouse	4.98	19.40	8.29	-16.01	-2.67	1.28	-1.28
Roof	4.27	16.63	7.11	-13.75	-2.25	-6.13	6.13

Total Shear (kip)							
	North/South					East/West	
Level	Frame B	VT-C	VT-D	VT-F	Frame G	VT-3	VT-4.1
2	79.34	483.19	465.73	502.46	37.41	371.89	364.15
3	67.69	422.68	418.78	476.34	39.96	365.28	342.22
4	92.28	501.53	433.42	343.10	23.17	335.41	304.12
5	85.91	464.50	399.41	312.06	21.10	303.21	297.22
6	77.56	421.12	362.77	284.02	19.52	278.80	273.30
7	69.82	379.39	325.98	255.21	17.15	253.81	248.62
8	62.10	335.72	288.91	225.55	15.42	228.35	223.66
9	54.11	291.79	251.05	195.49	13.30	202.17	198.21
10	45.98	247.24	212.71	165.36	11.28	175.64	172.11
11	37.52	201.79	173.57	135.01	9.28	148.89	145.75
12	29.12	156.26	134.37	104.39	7.17	121.69	118.98
Penthouse	18.32	101.14	89.95	78.18	6.06	93.36	92.97
Roof	10.67	55.88	46.36	31.46	1.87	57.72	71.47

Drift Analysis

For serviceability, building drift is limited by certain code criteria. Drift limits were evaluated at each story for wind and seismic loading and compared to actual deflections produced from the loads applied to the lateral force resisting system. To determine actual deflections, a three dimensional SAP model was developed. The lateral frames were modeled parallel to each in their existing geometry. Story shears were applied to the center of mass at each level in the north/south and east/west direction. The results of the story drifts are summarized in the tables below.

Seismic Drift				
Story	Story Height (ft.)	North/South SAP Drift (in.)	East/West SAP Drift (in.)	Code Allowable 0.020 h_{sx} (in.)
2	18	0.02	0.05	0.36
3	34	0.06	0.12	0.68
4	47.5	0.10	0.21	0.95
5	61	0.15	0.29	1.22
6	74.5	0.20	0.40	1.49
7	88	0.25	0.51	1.76
8	101.5	0.31	0.63	2.03
9	115	0.37	0.78	2.30
10	128.5	0.43	0.90	2.57
11	142	0.49	0.99	2.84
12	155.5	0.54	1.07	3.11
Penthouse	170	0.59	1.12	3.40
Low Roof	188	0.65	1.16	3.76

Wind Drift				
Story	Story Height (ft.)	North/South SAP Drift (in.)	East/West SAP Drift (in.)	Code Allowable H/400(in.)
2	18	0.16	0.14	0.54
3	34	0.35	0.33	1.02
4	47.5	0.53	0.53	1.43
5	61	0.72	0.74	1.83
6	74.5	0.94	0.97	2.24
7	88	1.17	1.20	2.64
8	101.5	1.41	1.45	3.05
9	115	1.66	1.74	3.45
10	128.5	1.92	1.97	3.86
11	142	2.18	2.17	4.26
12	155.5	2.42	2.34	4.67
Penthouse	170	2.67	2.49	5.10
Low Roof	188	2.99	2.63	5.64

Discussion

The logical path for lateral loading is from the curtain wall directly into the lateral frames. Load is distributed by the rigid diaphragm at each level according to relative frame stiffness. In spite of even spacing, eccentric braced frames provide a much higher relative stiffness. Both VT-3 and VT-4.1 in the east/west direction have eccentric braces and prove to have fairly equal stiffness. In the north/south direction VT-C, VT-D, and VT-F have eccentric braces while Frame B and Frame G do not. A weak link is created in Frame B and Frame G through the lack of braces. Relative stiffness of these frames is much lower than that of the others in the same direction and in turn a much lower load is shared.

The percentage of shear in each frame created by torsion increases as each frame's distance from the center of rigidity increases. Frame B has the largest percentage of shear created by torsion (up to 40%) and is farthest from the center of rigidity. VT-D has the smallest percentage of shear created by torsion (up to 20%) and is closest to the center of rigidity. Both frames in the east/west direction have minimal torsion effects due to even spacing and close proximity to the center of rigidity.

A higher relative stiffness in frames with braces creates higher loads in members connected to the braces. The higher loads will be transferred through the columns into the foundations, resulting in larger caisson diameters under the eccentric braced frames.

Overturning moment creates uplift forces in caissons transferred through the columns. A maximum uplift force of 1000kips is permitted within each caisson. To ensure that this criterion was met, base reactions were examined when the building was subjected to pure wind forces. Column F3 was required to support 1044kips of uplift force, however when applying load combinations the column is only required to support 303.6kips of uplift force, which is well under the 1000kip limit. The controlling load combination was Dead + Wind.

MEMBER STRENGTH CHECK

Existing Building Models

SAP2000 - Wind loads used in the evaluation of member strength were developed through a three dimensional SAP2000 building model. The building model was assembled with applied wind loads at each level's center of mass. Each member was assigned according to existing floor plans. The loads were distributed within the frame according to stiffness.

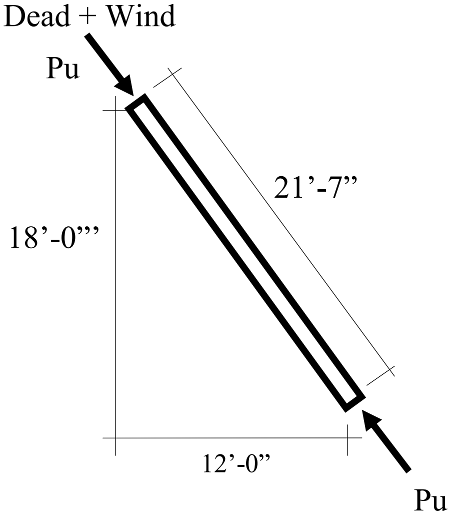
RAM Structural System - Gravity loads were developed through hand calculations and verified with a 3D RAM Structural System model. Each floor was assembled accurately including slab openings, load variation among floors, and balconies.

Lateral Eccentric Brace

Location: Between F3 & F3.8, Base Level

Member Size: W12x106

Loading:
Controlling load case= Dead + Wind
Puwind=327.34k
Pudead=4k



Member Properties:

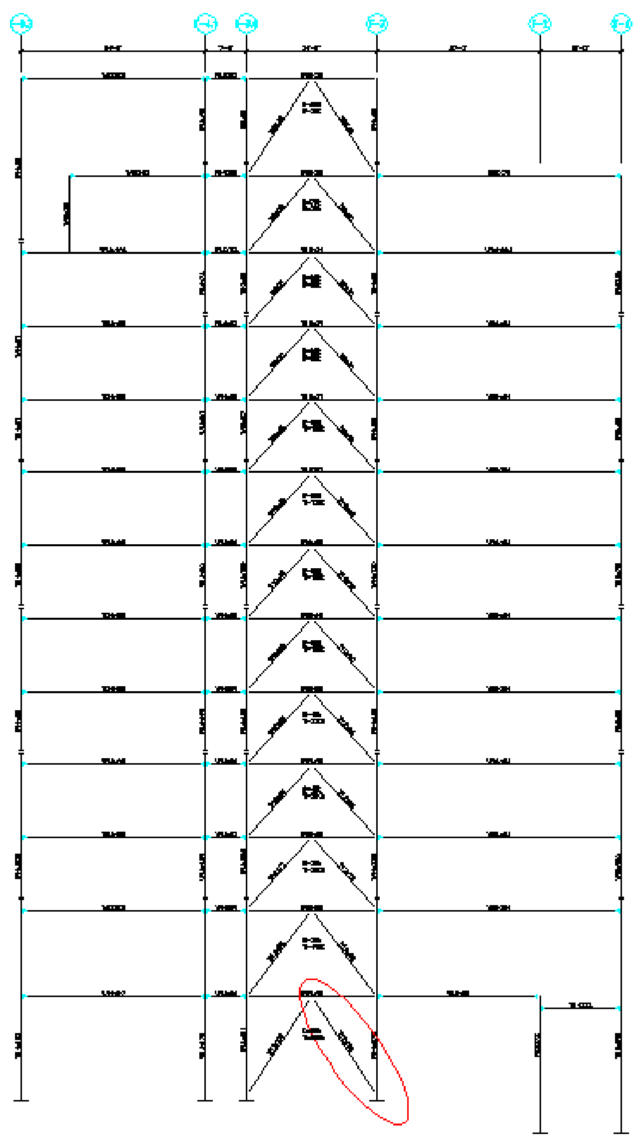
Fy= 50ksi	ry= 3.11in.	bf/2tw= 6.17
Fu= 65ksi	rx= 5.47in.	h/tw= 15.19
L= 21.6ft.	Ag= 31.2 in^2	

Check Compact Section:
$\lambda r = 0.56\sqrt{(E/Fy)} = 13.5 > 6.17$ ≈ Flanges are not slender.
$\lambda r = 1.46\sqrt{(E/Fy)} = 35.9 > 15.19$ ≈ Web is not slender.

Check Buckling:
KL/rx= 47.4
KL/ry= 83.3 ← Controls < 200ft. ≈ OK
λ=KL/rπ*√(Fy/E)= 1.10 < 1.5 ≈ OK

Check Strength:
P/Ω= 551kip > 331.4 kip ≈ OK

A large variation between the capacity of the brace and applied load exists. In midrise construction, drift controls over strength. The larger brace may be due in part to limitations on building drift. Detailed hand calculations for this load case and alternative load cases are available upon request. Brace location can be found in the diagram below.



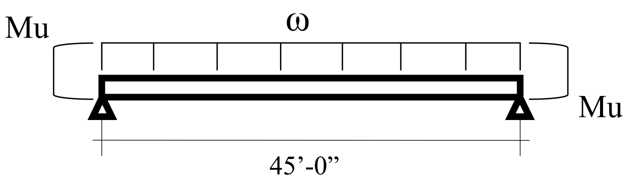
VERTICAL TRUSS VT-F

Lateral Beam

Location: VT-F, between column line 1 & 2, Level 7

Member Size: W24x68, 45ft. span

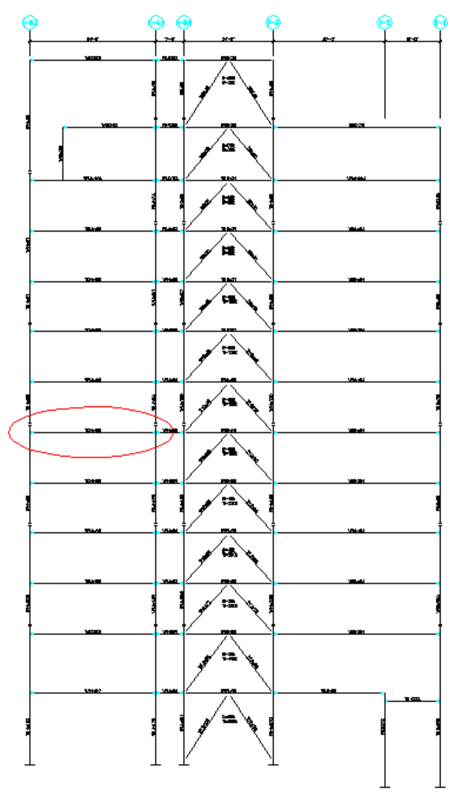
Loading:
Controlling load case= Dead + 0.75Live + 0.75Wind



Mu_{Wind}= 30.15 ft.k
Mu_{Live}= 94.5 ft.k
Mu_{Dead}= 168.75 ft.k
Vu_{Wind}= 1.34 k
Vu_{Live}= 22.5 k
Vu_{Dead}= 12.6 k
ω_{Dead}= 0.56 k/ft.
ω_{Live}= 1k/ft.

Mn/Ω= 442 ft.k > 234.68 ft.k ≈ OK
V/Ω= 197 k > 30.48 ≈ OK

Determined by the excess capacity of the members, beams were sized according to serviceability (drift). Hand calculations and computer model outputs are found in the Appendix. Detailed hand calculations for this beam are available upon request. The location of the beam can be viewed in the diagram below.



VERTICAL TRUSS VT-F

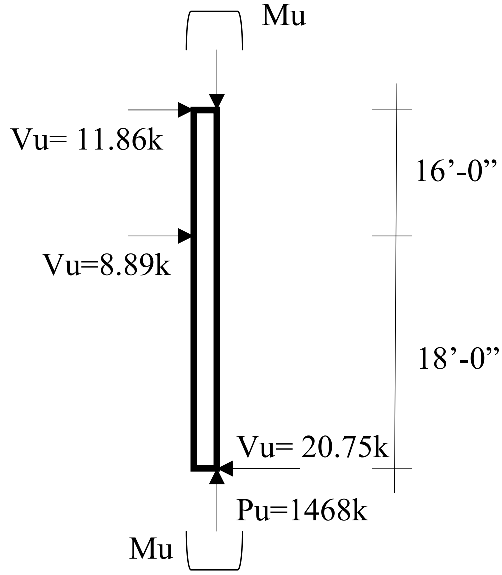
Lateral Column

Location: E3, moment frame in east/west direction

Member Size: W14x211

Loading:
Controlling load case= Dead+0.75Wind +0.75Live

Mu_{Wind}=166ft.k
Pu_{Wind}=13.6k
Pu_{Dead}=849.1k
Pu_{Live}=619.5k
Vu_{Wind}=20.75k



Member Properties:

- Braced by diaphragm at 18'
- Resists moment in east/west direction

Fy= 50ksi	ry= 4.07in.	bf/2tw= 5.06	Zx=390in.^{3}
Fu= 65ksi	rx= 6.55in.	h/tw= 11.6	Zy=198in.^{3}
L= 24ft.	Ag= 62.0 in^2		Ix=2660in.^{4}

Check Compact Section:
$\lambda r= 0.56\sqrt{(E/Fy)}= 13.5 > 5.06$ ≈ Flanges are not slender.
$\lambda r= 1.46\sqrt{(E/Fy)}= 35.9 > 11.6$ ≈ Web is not slender.

Check Buckling:
KL/(rx/ry)= 1.54*18/1.61= 17.72'
KLy= 18 ft. ← Controls
$\lambda=KL/r\pi*\sqrt{(Fy/E)}= 0.7< 1.5$ ≈ OK
*Base connection is modeled as fixed.

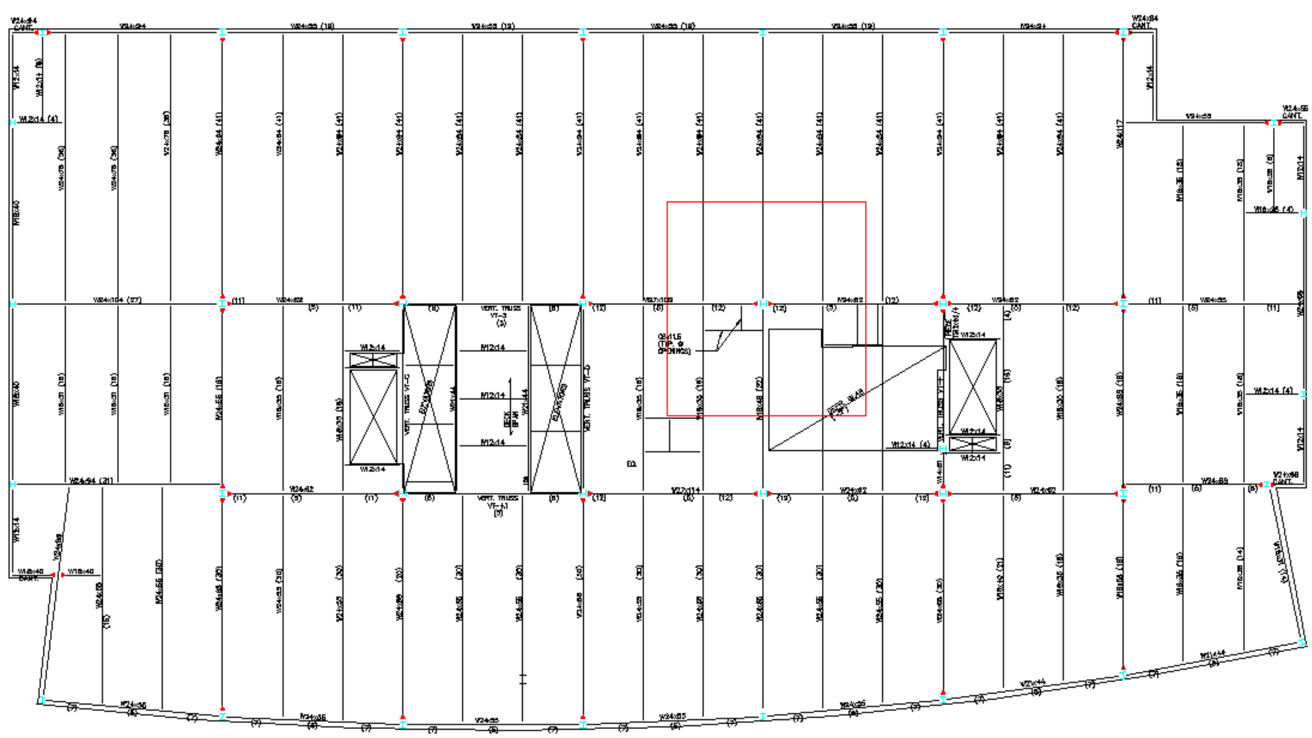
Check Shear:
$Vn/\Omega= 308 >20.75$ ≈ OK

Check Compression:
Peff= 1313.73+ 13.6(.75) = 1323.93k
P/Ω= 1510k > 1323.93k ≈ OK
Interaction Equation:
$\beta_1 = \beta_2 = 1.0$
Cm= 0.39
α= 1.6 (ASD)
Lp= 14.4'; Use beam tables for unbraced moment.
Mr=124.5k Pr= 1323.93k

$P_u/(P/\Omega)$= 1323.93/1510= 0.877 > 0.2

$$\frac{1323.93}{1510} + \frac{8}{9}\,\frac{124.5}{973} = 0.99 \quad < 1.0 \approx \text{OK}$$

Hand calculation of gravity loads are within 7.5% of RAM gravity loads (1468k vs. 1377k). Variation between the two calculations is attributed to omission of member self-weight in hand calculations. RAM loads were deemed more accurate and used in the strength check calculations. Output data from structural programs can be obtained from the index. Calculations for different load cases are available upon request. The location of the column can be viewed in the diagram below.



ANALYSIS & CONCLUSIONS

A complete analysis was done on the existing lateral force resisting system of Lockwood Place. The system is composed of moment frames acting with eccentric braces. After accumulating all lateral loads, story shears were applied to each level to evaluate strength and serviceability of the system. Hand calculations were compared to computer model results to verify accuracy.

The lateral system design is controlled by wind forces over seismic forces. This result was expected considering the height and location of the building. Drift for wind and seismic were within the code limitations, being H/400 and 0.020 h_{sx} respectively. Drift was also controlled by wind forces with a total drift of 2.63" in the east/west direction and 2.99" in the north/south direction. Caisson requirements for uplift and size variation due to overturning moment were satisfied.

Story shear forces are distributed through frames according to relative stiffness. Each floor acts as a rigid diaphragm when distributing forces to the frames. Torsional shear did not become a significant factor in the east/west direction because of symmetry and a close proximity of the frames to the center of rigidity (approximately 15'). In the north/south direction torsional shears became a significant factor, accounting for up to fifty percent of total shear distributed to Frame B.

Eccentric braced frames have greater stiffness than a simple moment frame. These frames resist greater loads because of greater stiffness. The larger loads are transferred into the foundations. Caisson diameters under eccentric braced frames are larger and penetrate deeper into bedrock than caissons supporting simple moment frames to accommodate the loads.

A strength check of a lateral brace, column, and beam provided justification that lateral members are acceptable to resist lateral loads. Drift controlled the design of lateral members, as is expected in midrise buildings. Increased stiffness of eccentric braces and beams assists in satisfying serviceability requirements.

APPENDIX
CALCULATIONS

Wind Calculations

Ta= $0.02*194^{0.75}$= 1.04> 1.0. FLEXIBLE
K_{ZT}=1.0
Kd=0.85
Exposure Category D
I=1.0
P=q*G*C_P
C_P = 0.8 windward; C_P = -0.5 NS leeward; C_P = -0.33 EW leeward; C_P = -0.7 sidewall;
G_f=$0.925*(1+1.7*Iz*(g_Q{}^2Q^2+g_R{}^2R^2)^{1/2})/(1+1.7g_VIz)$
g_Q =g_V=3.4
g_R = $(2\ln(3000*).96)^{1/2}\boldsymbol{+}0.577/(2\ln(3000*0.96)^{1/2}$= 0.4136
N_1= 0.96*(760.9)/135 = 5.41
Lz= $650*(116.4/33)^{1/8}$ = 760.9
Vz= $0.8*(116.4/33)^{1/9}*100*(88/60)$ = 135
Iz= $0.15*(10/116.4)^{1/6}$=0.11
Rn= $7.47*5.41/(1+10.3*5.41)^{5/3}$= 0.048
Rh: n= 4.6*0.96*194/135=6.35

$$Rh= \frac{1}{6.35} - \frac{1}{2*6.35^2}*(1-e^{-2*(6.35)})= 0.145$$

R_B: n= 4.6*0.96*(118.33)/125= 3.87 N/S
n= 4.6*0.96*(218.67)/125= 7.14 E/W

$$R_B= \frac{1}{3.87} - \frac{1}{2*3.87^2}*(1-e^{-2*(3.87)})= 0.22 \text{ N/S}$$

$$R_B= \frac{1}{7.14} - \frac{1}{2*7.14^2}*(1-e^{-2*(7.14)})= 0.13 \text{ E/W}$$

R_L: n= 15.4*0.96*(118.33)/135= 12.96 N/S
n= 15.4*0.96*(118.33)/135= 23.90 E/W

$$R_L= \frac{1}{12.9} - \frac{1}{2*12.9^2}*(1-e^{-2*(12.96)})= 0.074 \text{ N/S}$$

$$R_L= \frac{1}{23.9} - \frac{1}{2*23.9^2}*(1-e^{-2*(23.9)})= 0.04 \text{ E/W}$$

R= $({}^{1}/_{0.5}*.145*0.048*(.22)*(0.53+0.47*0.074))^{1/2}$= 0.29 N/S
$({}^{1}/_{0.5}*.145*0.048*(.13)*(0.53+0.47*0.04))^{1/2}$= 0.22E/W

Q= $(1/(1+0.63*((L+194)/760.9)^{0.63}))^{1/2}$= 0.86 NS
0.83 EW

Gf: 0.90 North/South
0.88 East/West

*Hand calculations are available upon request.

General Information	
Building Category	II
Importance Factor, I	1.0
Exposure Category	D
kd	0.85
Topographic Factor, kzt	1.0
V (mph)	100
Period (T)	1.04
Gust Effect Factor	0.85
Cp	0.80
Building Height, hn	194
x	0.75
Frequency, n1	0.96
North/South Length	118.6
East/West Length	218.3
Enclosure Classification	Fully Enclosed

Parapets E/W			
GCpn		GCpn	
Windward	1.5	Windward	1.5
Leeward	-1.0	Leeward	-1.0
qp	35.03	qp	35.03
Pp (psf)		Pp (psf)	
Windward	52.55	Windward	52.55
Leeward	-35.03	Leeward	-35.03

Floor	Height Above Ground(ft.)	Floor Height (ft.)	Forces (k)		Story Shears	
			North/South	East/West	North/South	East/West
1	0	18	64.23	28.54	1611.98	736.04
2	18	16	125.15	55.94	1547.75	707.50
3	34	13.5	113.48	51.13	1422.60	651.56
4	47.5	13.5	106.73	48.33	1309.12	600.43
5	61	13.5	109.27	49.68	1202.39	552.10
6	74.5	13.5	110.65	50.41	1093.12	502.43
7	88	13.5	112.96	51.64	982.47	452.01
8	101.5	13.5	114.81	52.62	869.51	400.38
9	115	13.5	115.73	53.11	754.69	347.75
10	128.5	13.5	117.35	53.97	638.96	294.64
11	142	13.5	118.04	54.34	521.61	240.67
12	155.5	14.5	123.90	57.14	403.57	186.33
Penthouse	170	18	145.42	67.18	279.66	129.19
Low Roof	188	6	107.39	49.61	134.24	62.01
High Roof	194		26.85	12.40	26.85	12.40

Floor	Height Above Ground(ft.)	Floor Height (ft.)	Kz	qz
1	0	18		
2	18	16	1.08	23.50
3	34	13.5	1.22	26.55
4	47.5	13.5	1.27	27.64
5	61	13.5	1.34	29.16
6	74.5	13.5	1.38	30.03
7	88	13.5	1.40	30.46
8	101.5	13.5	1.48	32.20
9	115	13.5	1.48	32.20
10	128.5	13.5	1.52	33.08
11	142	13.5	1.55	33.73
12	155.5	14.5	1.55	33.73
Penthouse	170	18	1.61	35.03
Low Roof	188	6	1.61	35.03
High Roof	194		1.61	35.03

North/South Windward	North/South Leeward	North/South Side Wall	East/West Windward	East/West Leeward	East/West Side Wall
16.92	-15.77	-22.07	16.54	-10.17	-21.58
19.11	-15.77	-22.07	18.69	-10.17	-21.58
19.90	-15.77	-22.07	19.46	-10.17	-21.58
20.99	-15.77	-22.07	20.53	-10.17	-21.58
21.62	-15.77	-22.07	21.14	-10.17	-21.58
21.93	-15.77	-22.07	21.45	-10.17	-21.58
23.19	-15.77	-22.07	22.67	-10.17	-21.58
23.19	-15.77	-22.07	22.67	-10.17	-21.58
23.81	-15.77	-22.07	23.28	-10.17	-21.58
24.28	-15.77	-22.07	23.74	-10.17	-21.58
24.28	-15.77	-22.07	23.74	-10.17	-21.58
25.22	-15.77	-22.07	24.66	-10.17	-21.58
25.22	-15.77	-22.07	24.66	-10.17	-21.58
25.22	-15.77	-22.07	24.66	-10.17	-21.58

Seismic Calculations

Base Shear
Seismic Use Group: II
Importance Factor: 1.0
Mapped Spectral Response Acceleration:
S_S= 0.170g
S_1= 0.051g
Site Class Factors: (Site Class B)
Fa= 1.0
Fv= 1.0
$S_{MS}= S_S *Fa= 0.170g$
$S_{M1}= S_1 *Fv= 0.051g$
$S_{DS}= {}^2/_3 * S_{MS}= 0.113g$
$S_{D1}= {}^2/_3 * S_{M1}= 0.034g$
Seismic Design Category A
$T_a= C_t * hn^x= 0.02*(194)^{.75}= 1.04$
(Other frame system chosen due to duel systems)
$T=T_a*C_u= 1.04*1.7 =1.768$
(C_u from table 12.8-1)
C_s= $S_{DS} /(R/I)= 0.113/3= .037$
$S_{D1}/[T*(R/I)]= 0.034/(1.768*3) = 0.006$
$S_{D1}*T_L/[T^2*(R/I)]= 0.034*6/(1.768^2*3)= 0.004$

Controlling C_s= 0.01 (minimum required by code)

*T_L, the long-period transition period is chosen as 8 seconds. Lockwood Place sites sit directly on division line. Neither six second nor eight second periods control. The value can be found in ASCE-7-05, Figure 22-15.
*The response modification coefficient is chosen for a ‘steel systems not specifically detailed for seismic resistance’ system and conforms to requirements ASCE-7-05.

General Information		
Occupancy Type		II
Seismic Use Group		I
Site Class		B
Seismic Design Category		A
Short Period Spectral Response	Ss	0.170
Spectral Response at 1 Second	S1	0.051
Maximum Short Period Spectral Response	Sms	0.170
Maximum Spectral Response at 1 Second	Sm1	0.051
Design Short Period Spectral Response	SDS	0.113
Design Spectral Response at 1 Second	SD1	0.034
Response Modification Coefficient	R	3
Seismic Response Coefficient	Cs	0.01
Effective Period	T	
Height Above Grade	hn	194
Base Shear		275k
Overturning Moment		34,683.09(ft*k)

Base Shear & Overturning Moment							
Level	h (ft)	Total Weight (kip)	k	hxkWx	Cvx	Fx	Moment (ft-kip)
High Roof	194	37.6	1.63	201685.24	0.0045	1.23	0
Low Roof	188	179.2	1.63	912463.08	0.0203	5.58	7.39
Penthouse	170	1283.4	1.63	5546372.12	0.1231	33.89	129.95
12	155.5	2193.8	1.63	8198144.97	0.1820	50.10	720.13
11	142	2075.9	1.63	6690274.88	0.1485	40.88	1915.31
10	128.5	2075.9	1.63	5684942.26	0.1262	34.74	3723.81
9	115	2075.9	1.63	4744073.84	0.1053	28.99	5970.43
8	101.5	2075.9	1.63	3870380.02	0.0859	23.65	8608.55
7	88	2075.9	1.63	3067049.01	0.0681	18.74	11565.97
6	74.5	2075.9	1.63	2337914.83	0.0519	14.29	14776.41
5	61	2075.9	1.63	1687724.64	0.0375	10.31	18179.72
4	47.5	2075.9	1.63	1122598.72	0.0249	6.86	21722.28
3	34	2277.1	1.63	713997.27	0.0159	4.36	25596.54
2	18	2406.7	1.63	267619.95	0.0059	1.64	29735.59
1	0	2423.8	1.63				34683.09
			Sum=	45045240.84		Base Shear	Overturning Moment
					TOTAL	**275.27**	**34683.09**

Location	Area	Load (psf)	Weight (kip)
Level 1			
Retail	22002	63	1386.1
Lobby	2000	88	176.0
Curtain Wall	10800	25	270.0
Masonry Wall	1800	62	111.6
Level 2			
Retail	24923	63	1570.1
Curtain Wall	9576	25	239.4
Masonry Wall	1592	62	98.7
Level 3			
Office	23555	56	1319.1
Curtain Wall	9054	25	226.4
Balcony	2266	115	260.6
Level 4-11			
Office	24486	56	10969.7
Curtain Wall	8600	25	1720.0
Level 12			
Office	21600	56	1209.6
Curtain Wall	8812	25	220.3
Balcony	2886	115	331.9
Penthouse			
Office	12800	56	716.8
Balcony	733	115	84.3
Curtain Wall	9054	25	226.4
Roof	8800	14	123.2
Low Roof			
Surface	12800	14	179.2
High Roof			
Surface	2688	14	37.6
Super Imposed Dead/Steel Structure			
	302348	20	6050.0
TOTAL BUILDING WEIGHT			**27527.0k**

Torsion Calculations

Relative Rigidity							
Level	Frame B	VT-C	VT-D	VT-F	Frame G	VT-3	VT-4.1
2	0.0464	0.2928	0.2928	0.3415	0.0265	0.4942	0.5058
3	0.0464	0.2924	0.2924	0.3389	0.0287	0.4942	0.5058
4	0.0474	0.2926	0.2926	0.3378	0.0296	0.4942	0.5058
5	0.0475	0.2926	0.2923	0.3376	0.0300	0.4942	0.5058
6	0.0473	0.2924	0.2924	0.3375	0.0304	0.4942	0.5058
7	0.0474	0.2932	0.2924	0.3373	0.0297	0.4942	0.5058
8	0.0475	0.2925	0.2924	0.3373	0.0303	0.4942	0.5058
9	0.0476	0.2925	0.2925	0.3372	0.0302	0.4942	0.5058
10	0.0477	0.2924	0.2925	0.3371	0.0303	0.4942	0.5058
11	0.0477	0.2924	0.2924	0.337	0.0305	0.4942	0.5058
12	0.0478	0.2924	0.2924	0.3369	0.0305	0.4942	0.5058
Penthouse	0.0477	0.2923	0.292	0.3368	0.0312	0.4942	0.5058
Roof	0.0477	0.2924	0.2924	0.3368	0.0307	0.4942	0.5058
Distance X_R	35	65	95	155	185	0	0
Distance Y_R	0	0	0	0	0	45	76.5

Stiffness K=P/Δ							
	North/South					East/West	
Level	Frame B	VT-C	VT-D	VT-F	Frame G	VT-3	VT-4.1
2	441.90	2788.57	2788.57	3252.38	252.38	2671.35	2734.05
3	187.10	1179.03	1179.03	1366.53	115.73	1093.36	1119.03
4	115.89	715.40	715.40	825.92	72.37	636.86	1236.67
5	78.77	485.24	484.74	559.87	49.75	437.73	448.01
6	56.11	346.86	346.86	400.36	36.06	312.78	320.13
7	42.40	262.25	261.54	301.70	26.57	240.02	245.65
8	32.76	201.72	201.66	232.62	20.90	185.65	190.01
9	26.10	160.36	160.36	184.87	16.56	144.38	147.77
10	21.19	129.90	129.94	149.76	13.46	118.43	121.21
11	17.45	106.95	106.95	123.26	11.16	99.90	102.24
12	14.60	89.34	89.34	102.93	9.32	85.18	87.18
Penthouse	11.63	71.29	71.22	82.15	7.61	73.59	75.31
Roof	9.68	59.31	59.31	68.32	6.23	62.78	64.25

Torsional Rigidity $J= \Sigma K*di^2$	
Level	J
2	18977503.65
3	8036956.684
4	5026021.505
5	3319273.163
6	2374014.218
7	1791459.928
8	1382685.497
9	1097617.021
10	890448.2773
11	734610.4938
12	614660.6584
PH	493975.322
Roof	411221.9754

Torsional Shear= V*e*d*K/J

Where:
V = story shear
e = eccentricity between COM &COR
d = distance from COM to frame
K= stiffness
J = torsional rigidity

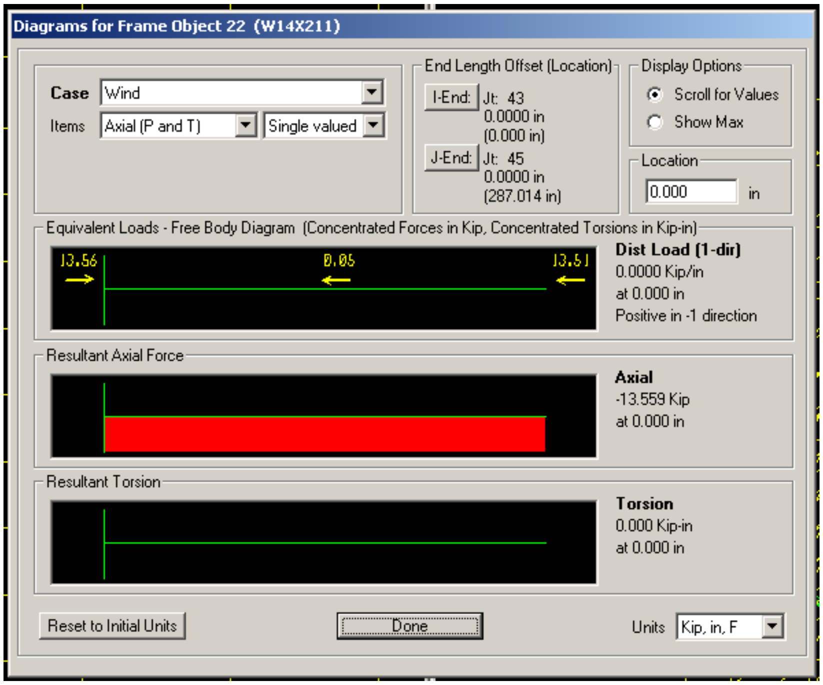
Gravity Loads

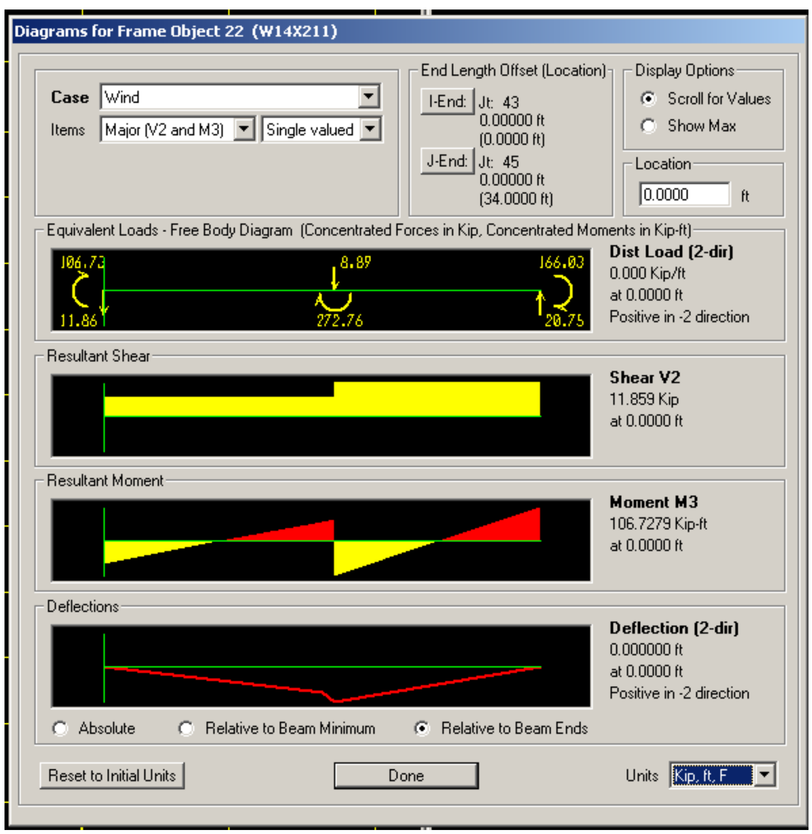
Column Loads (E-3)-Load Combination 1.2Dead+1.6Live					
Level	AT(sq.ft.)	Dead Load (psf)	Live Load (psf)	LL Reduction	Total Load (k)
2	922.5	63	100	100	150.37
3 to 12	1147.5	56	100	40	1101.60
Penthouse					
Machine	236.5	56	125	125	42.81
Tenant	236.5	56	100	40	22.70
Exterior	337.5	14	22.5	22.5	12.32
Roof					
High	236.5	14	22.5	22.5	8.63
Low	236.5	14	22.5	22.5	8.63
				Total	**1377.06**

*Total reducible area of live load is 0.4LL for office tenant spaces.

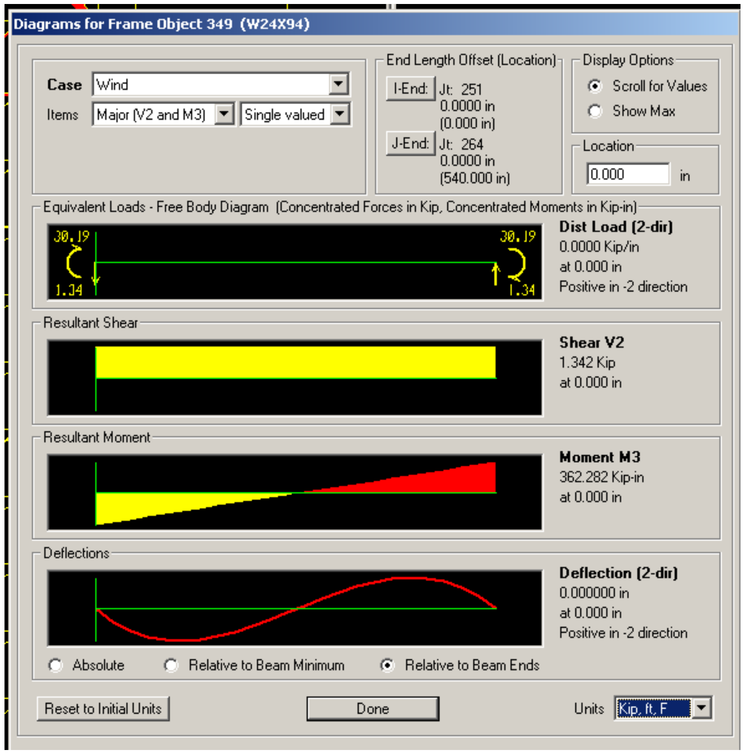
SAP Outputs

Column subjected to 1.0Wind

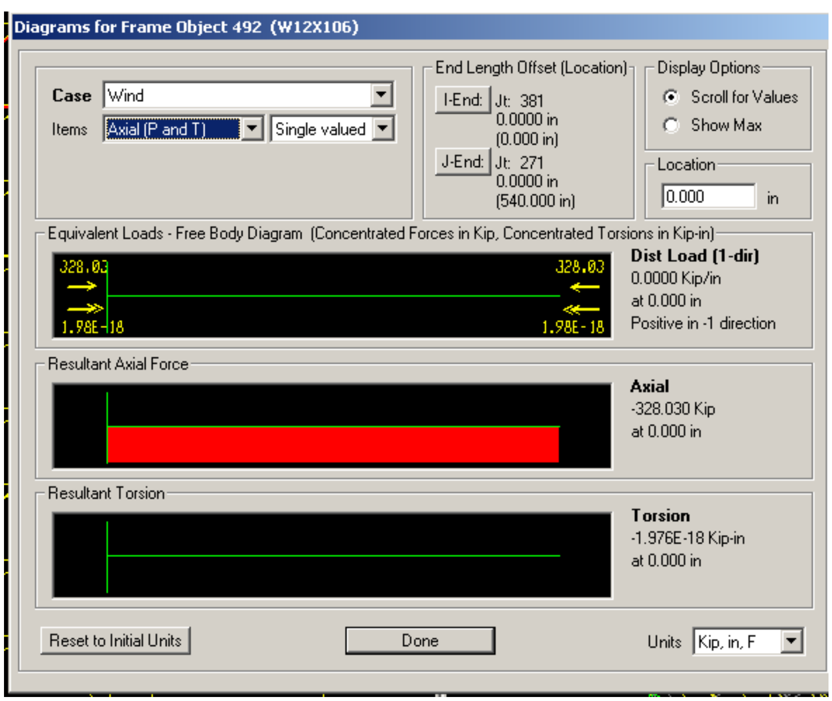




Beam subjected to 1.0Wind



Brace subjected to 1.0Wind



RAM Structural System Outputs

Column Gravity Loading

Column Line E - 3

Level	Col#	Height	Dead	Self	+Live	-Live	MinTot	MaxTot
high roof	6	3.50	5.3	0.2	6.5	0.0	5.4	11.9
roof	6	18.00	21.4	0.9	14.1	0.0	22.4	36.4
penthouse	14	14.50	63.3	1.5	76.2	0.0	64.8	141.0
12	14	13.50	135.1	2.1	121.3	0.0	137.2	258.5
11	14	13.50	204.9	3.0	160.0	0.0	207.9	367.9
10	14	13.50	274.6	4.0	205.9	0.0	278.6	484.5
9	14	13.50	344.4	5.2	251.8	0.0	349.6	601.4
8	14	13.50	414.2	6.4	297.7	0.0	420.6	718.3
7	14	13.50	483.9	8.0	343.6	0.0	492.0	835.6
6	14	13.50	553.7	9.6	389.5	0.0	563.4	952.9
5	14	13.50	623.5	11.6	435.4	0.0	635.1	1070.5
4th	13	13.50	695.3	13.6	481.3	0.0	708.8	1190.2
3	17	16.00	767.1	16.7	527.2	0.0	783.7	1311.0
retail	17	18.00	829.0	20.1	619.5	0.0	849.1	1468.6