LOCKWOOD PLACE
BALTIMORE, MARYLAND

TECHNICAL REPORT
1

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Executive Summary

Lockwood Place in Baltimore, Maryland is a thirteen story mixed-use development building utilized primarily 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.

The goal of this report is to discuss structural concepts and existing conditions in accordance with building codes and industry standards. Criteria on which decisions are based are loading requirements and design concepts. Analysis was done on building framing systems including, but not limited to: general floor framing, structural slabs, and lateral resisting system.

The primary loads in the building were determined through the Maryland Building Code Performance Standards and the 1996 BOCA Building Code. Updated building codes were use for analysis purposes. All gravity loads required by code were consistent with the loads employed in design. After analyzing wind and seismic loads, it was determined that wind is the governing lateral force in both north/south and east/west directions. Maximum pressure at the peak height of the building was found to be 25 pounds per square foot.

The structural system consists of a composite steel floor and combined moment and eccentric brace lateral frames. Framing systems are verified to be reasonable sizes through spot checks of typical members. While differences may occur in calculations, justification was made through the use of current codes opposed to the codes used during the design of the building. Also, other design criteria that were not addressed in this report may have governed the sizing of the members. These criteria could include vibrations or connection feasibility. Calculations and general assumptions are included in an Appendix of this report.

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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.

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. All superstructure concrete has a minimum compressive design strength of 3500psi. Concrete and masonry reinforcement strength requirement for foundations and walls conforms to ASTM A615, Grade 60 and splice laps fit class 'B.' Intermediate rail support framing for the elevator area is comprised of HSS6x4x1/4 tubes. MEP systems are run through the structural framing system. Four inch thick equipment pads are provided for large MEP equipment. Reinforcement details for holes existing in the beams and girders are provided in accordance with an AISC Design guide. A two hour fire rating is provided for all floor slabs, beams, girders, columns, roofs, and vertical trusses.

Ground level

At ground level, the floor system is slab-on-grade construction made of 5" thick concrete, reinforced with 6x6-W1.4xW1.4 W.W.F. The slab sits on 3" thick compacted fine granular fill, a vapor barrier, and 4" compacted granular base. Minimum concrete compressive strength for the floor is 3500psi.

A loading dock is located at the north-eastern corner of the building. Designed for much greater loads than that of a typical floor, the slab is 8" thick slab-on-grade concrete reinforced with #5@12"o.c. each way at the top and bottom of the slab. The slab sits on 4" compacted granular base. Minimum concrete compressive strength for the floor is 3500psi.

Machine Room

Machine room floor framing is similar with the typical superstructure floor system. Additionally, elevated equipment framing is provided by HSS posts and W12 beams varying in size.

Parking Garage Ramp

The parking garage ramp runs along the north face of the building from the ground level to the second floor. It allows access from the second floor of Lockwood Place to the second level of the adjacent parking garage. The ramp is on grade for a length of 36'-8."

This portion of the slab is 6” thick slab-on-grade concrete, reinforced with #3@12” each way or 4x4-W4.0xW4.0 W.W.F. sitting on 4” compacted granular based.

Extending to the second floor, the sloped portion of the ramp is 5” thick normal weight concrete topping on 3”x20ga. galvanized composite metal deck. The slab is reinforced each way with #4 bars at 12” on center at the bottom. Steel structural member framing consists of W24x94s that span the length of the ramp. During construction, temporary shoring was provided at the deck mid-span for the entire slope.

At the second level, the garage ramp is 4-1/2” normal weight concrete topping on 3”x20ga. galvanized composite metal deck. The slab is reinforced with 6x6 W2.9xW2.9 W.W.F. drape. An ice and snow melting cable is embedded in the slab for this portion of the ramp. Studs are ¾” diameter x 5-1/2” long headed shear studs. All garage ramp concrete has a minimum compressive strength of 3500psi.

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 down 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 both 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 W14x19 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 frame 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.

Foundation

Being 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. To accommodate for this bearing capacity, the foundation system is made of drilled caissons. Caisson shaft diameters range from 2'-6" to 6'-0." Typically, they extend a minimum if 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.

Special Conditions

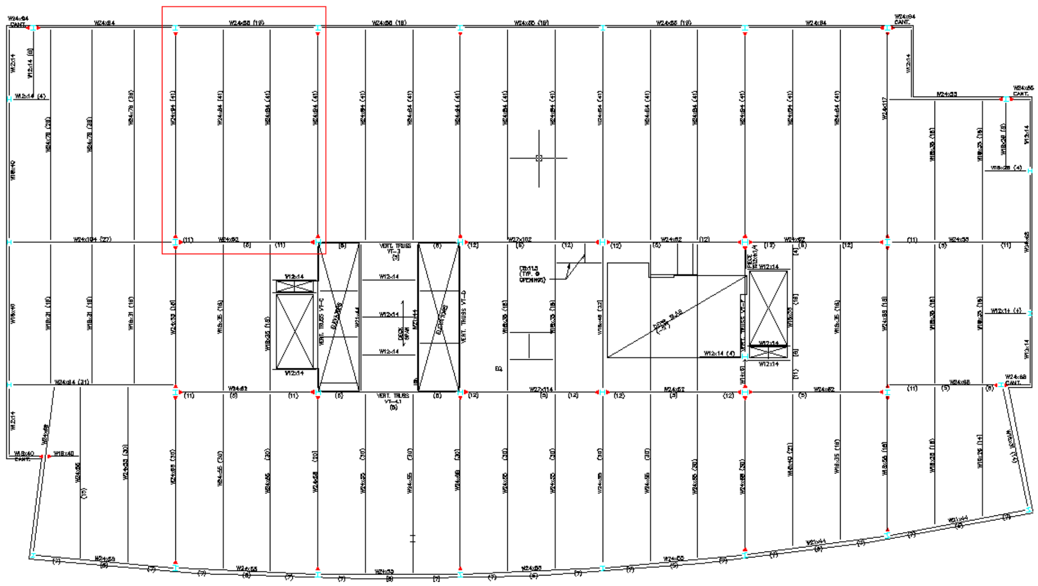
There are several interesting design aspects to Lockwood Place that are not included in this report, but will be addressed in the future. These aspects include, but are not limited to detailed dunnage framing; screen wall framing at level twelve; wind loads distributed on walls; uplift forces on roof framing; and detailed canopy framing. Snow drift load calculations for differing roof heights are considered, while snow drift on rooftop mechanical equipment are omitted for this report.

Although an elevator shaft and lobby attached to the adjacent parking garage were also designed with the Lockwood Place building, analysis of its structural components are not in the scope of this report.

FRAMING PLANS & ELEVATIONS

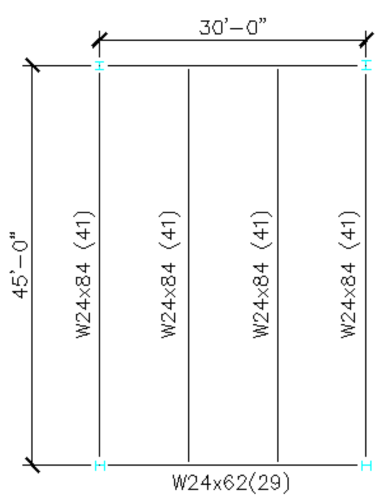
Typical Floor Framing Plan

Typical Framing Plan (levels 5-11) involves long spans and open areas, providing space flexibility for tenants. Because of setbacks and balconies, the footprints and framing of the first through fourth floors, the twelfth floor, and the penthouse vary from what is shown below. A typical bay is highlighted and shown below.



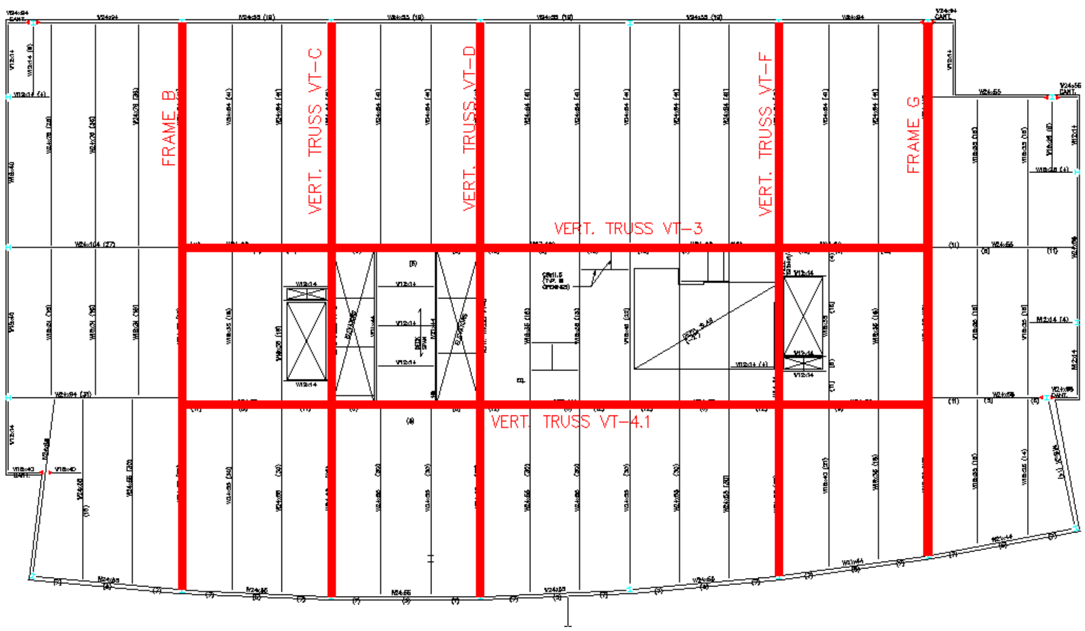
Typical Bay Framing

The typical bay size shown is 30’-0” x 45’-0” and consists of composite steel and concrete. Beams are spaced 10’-0” on center and connect to the girders through simple shear connections. The location of the bay is highlighted above.

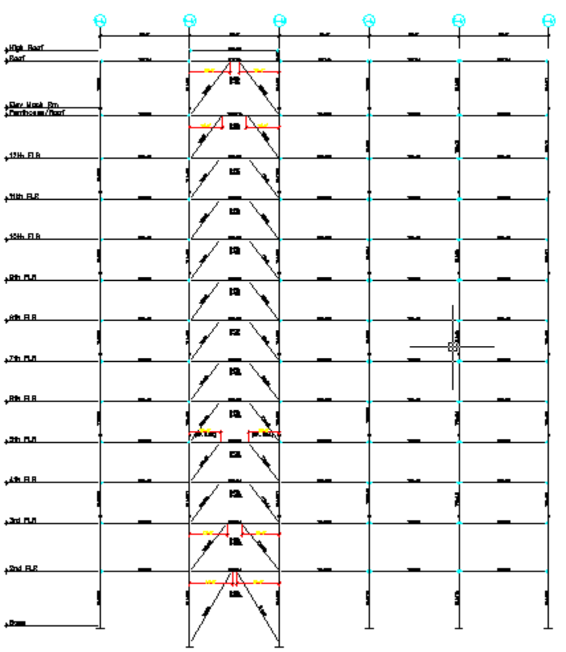


Lateral System Plan

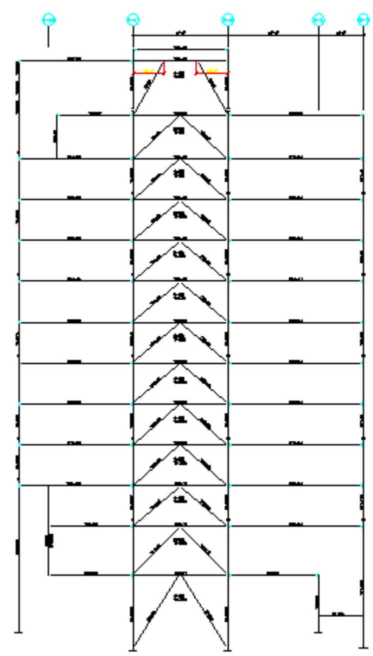
Lateral loads are resisted through a combination of eccentric bracing and moment frames. This combination runs in both east/west and north/south directions.



Elevation of Lateral Load Resisting Frames



VERTICAL TRUSS VT–4.1



VERTICAL TRUSS VT–D

CODES & DESIGN STANDARDS

Applied in original design:

- BOCA Building Code 1996 Edition (Building Officials and Code Administrators)
- 1997 Baltimore Building Code
- American Society for Testing Materials (ASTM)
- American Concrete Institute (ACI)
  ACI 530.1 Specifications for Reinforced Masonry Structures
  ACI 301, ACI 318, ACI 315 Specifications for Reinforced Concrete Structures
- American Society for Civil Engineers (ASCE)
  ASCE-6, Specification for Reinforced Masonry Structures
- American Welding Society (AWS)
  AWS D1.1, Specifications for Structural Steel Welds
- American Institute for Steel Construction (AISC)
  Specifications for Structural Steel Design, ASD

Substituted for thesis analysis:

- American Society for Civil Engineers (ASCE-7-05)
  Design Code for Minimum Design Loads
- American Institute of Steel Construction (AISC)
  Specifications for Structural Steel Design – Unified Version, 2005
- American Concrete Institute
  Specification for Reinforced Steel and Masonry Structures, 2005

Material Strength Requirement Summary:

Structural Steel
- Wide flange, square or rectangular tubing……………………………….50ksi
- Plates used as doubler plates, base plates, or stiffeners…………………50ksi
- Channels, angles, bars, plates……………………………………………36ksi

Concrete
- Composite concrete……………………………………………………..3500psi
- Foundation walls, piers, grade beams…………………………………..4000psi
- Caissons………………………………………………………………... 4500psi
- Slab-on-grade …………………………………………………….......3500psi
- Reinforcing……………………………………………………………….. 60ksi

Masonry
- Prism Strength……………………………………………………………..1500psi
- Reinforcing…………………………………………………………...60ksi

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 wall estimated 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. Snow drift calculations can be found in Appendix A. Calculations for louvered screen walls were omitted due to a lack of information on size and exact location.

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

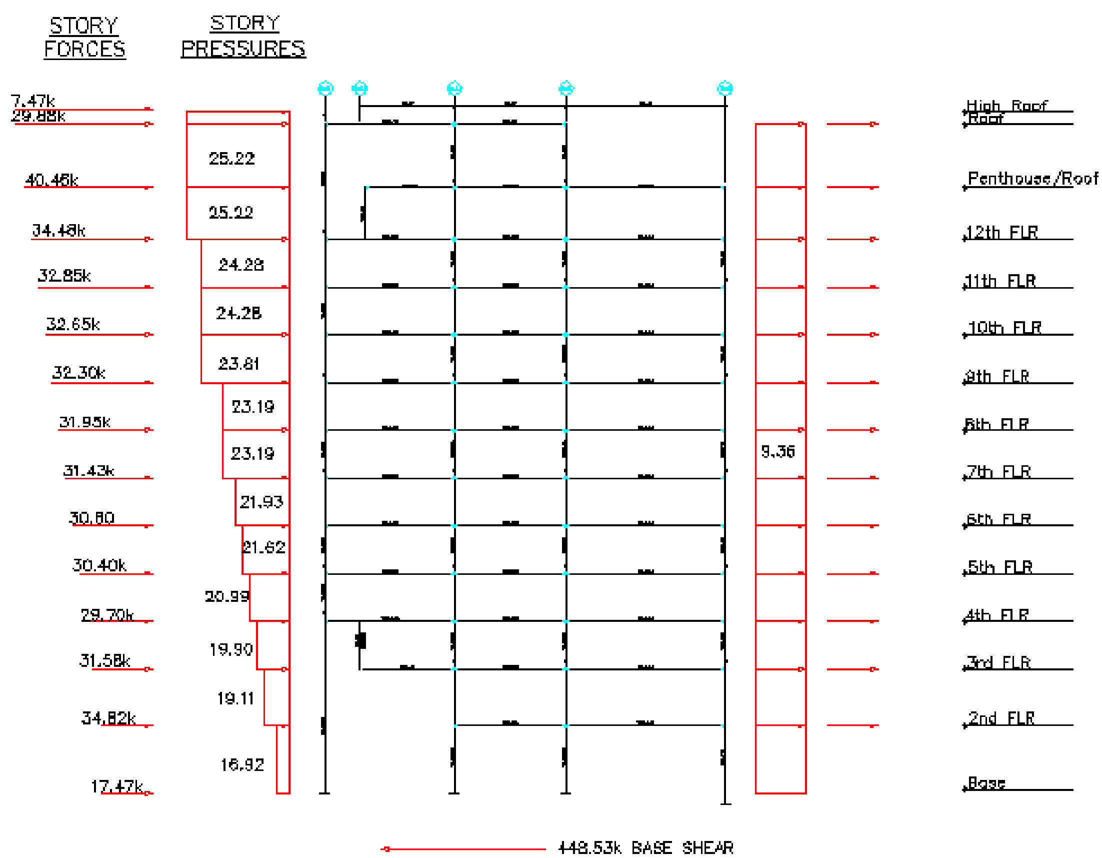
Lateral Loads

Wind Load

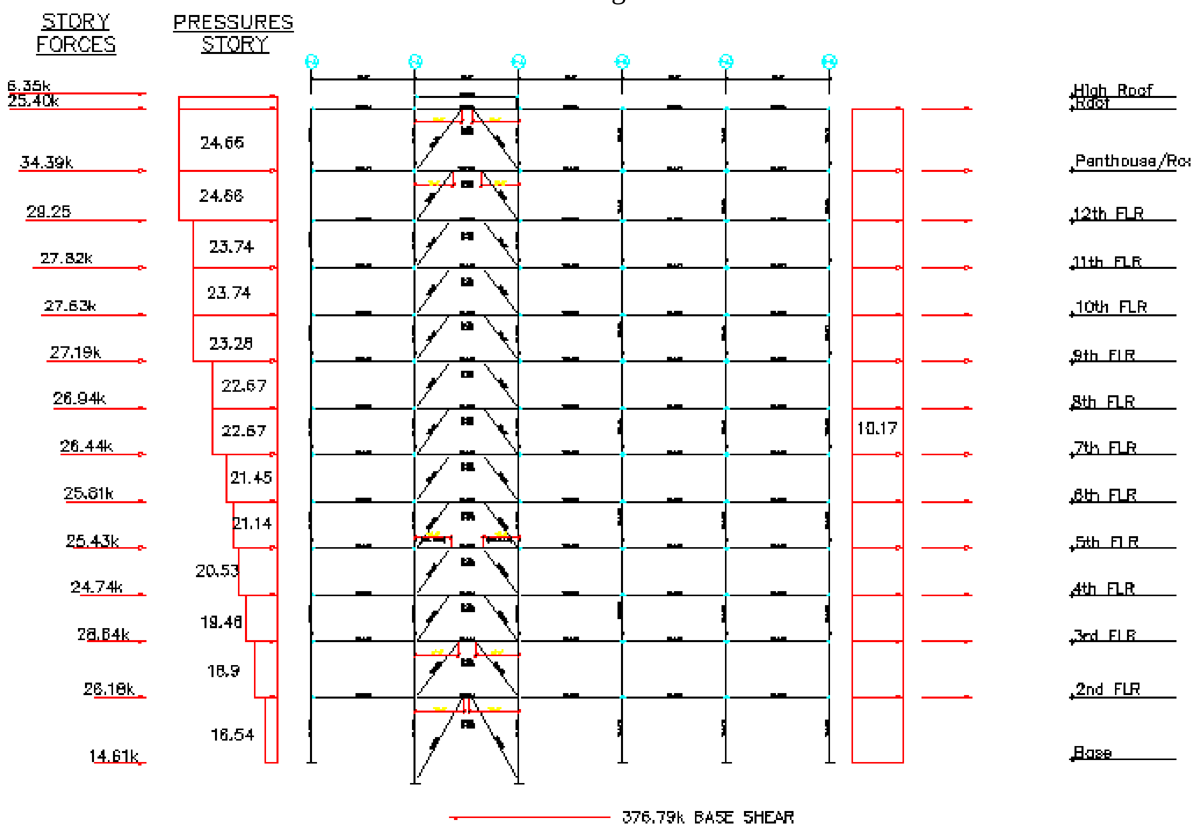
Determination of wind and loading was carried out in accordance with Section 6 of ASCE7-05. All factors were based on location and geometry of the building. Standardization of the curved façade was assumed. The north/south dimension of the building was taken from the largest dimension in the curve. This section is a summary of the results. Detailed calculations can be found in Appendix A.

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

Floor	Height Above Ground(ft.)	Floor Height (ft.)	Forces (k)		Story Shears (k)	
			North/South	East/West	North/South	East/West
1	0	18	17.87	14.61	448.53	376.79
2	18	16	34.82	28.64	430.66	362.19
3	34	13.5	31.58	26.18	395.84	333.55
4	47.5	13.5	29.70	24.74	364.26	307.37
5	61	13.5	30.40	25.43	334.56	282.63
6	74.5	13.5	30.79	25.81	304.16	257.20
7	88	13.5	31.43	26.44	273.37	231.40
8	101.5	13.5	31.95	26.94	241.94	204.96
9	115	13.5	32.20	27.19	209.99	178.02
10	128.5	13.5	32.65	27.63	177.79	150.83
11	142	13.5	32.85	27.82	145.14	123.21
12	155.5	14.5	34.48	29.25	112.29	95.39
Penthouse	170	18	40.46	34.39	77.82	66.14
Low Roof	188	6	29.88	25.40	37.35	31.75
High Roof	194		7.47	6.35	7.47	6.35



FRAME B

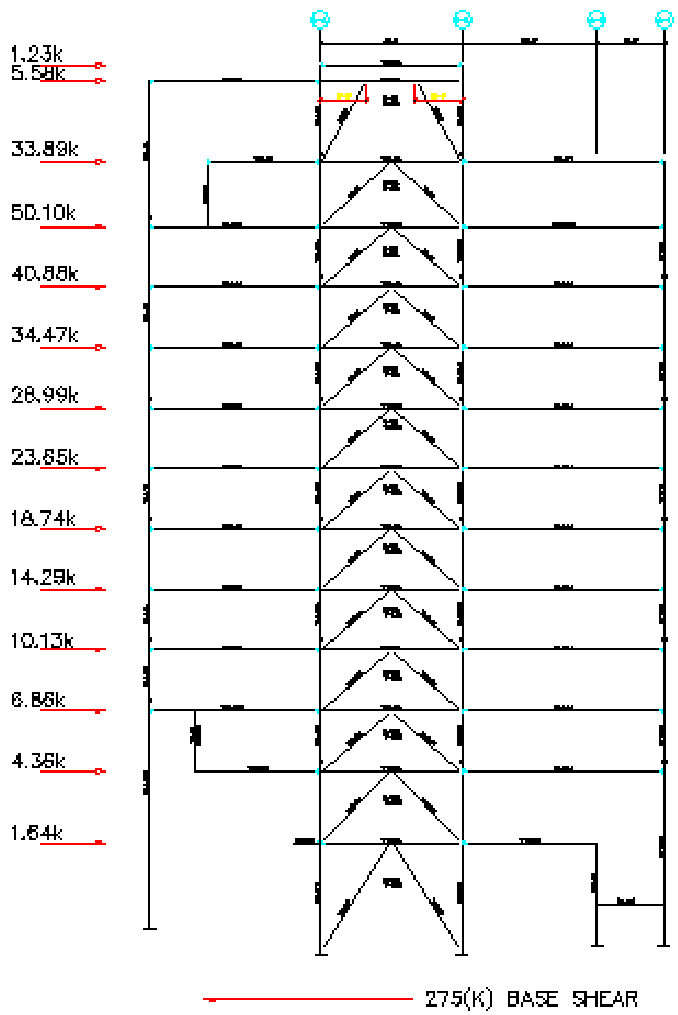


VERTICAL TRUSS VT-3

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. This section is a summary of the results. Detailed calculations can be found in Appendix A.

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		275(k)
Overturning Moment		34683.09(ft*k)



VERTICAL TRUSS VT-C

PRELIMINARY DESIGN ANALYSIS

Gravity Load Spot Checks

Composite Beam

Preliminary analysis for a typical composite beam was done for an existing W24x84 that spans 45' and is spaced 10' on center. The existing beam is called to have (41) ¾ diameter x 5-1/2" long studs. The beams are connected to the girders by a simple shear connection. Through an analysis of the beam, this design is determined to have much higher capacity than necessary to support the given loads. To achieve reactions specified on the beams, it is necessary to use a 50psf live load in calculation as opposed to the 100psf specified on the drawings. The 50psf is the minimum live load for an office required by code. The beam exceeds live load deflection requirements for a noncomposite beam by 28%.

A second beam with a smaller span was analyzed for comparison to the large size of the first beam. The second beam is called out as a W18x35 with (18) ¾" x 5-1/2" shear studs. This beam is also connected to the girders through simple shear connections. Through analysis, it is determined that this beam has much greater capacity than required to support the given loads. Live load deflection requirements for a noncomposite beam are precisely met.

Largely over sizing of the beams may be due to vibration requirements for an office space. The floor system would have been adjusted for noncomposite action to help satisfy these requirements. ASD design is more conservative for composite beam verses LRFD design. While the original floor was designed to an ASD standard, the analysis was calculated with an LRFD standard. Another possible explanation for beam over sizing could be due to several holes reinforced with A992 steel placed sporadically throughout the beam for MEP system runs. To verify factors controlling the design, a further analysis will need to be completed in the future. All beam calculations can be found in Appendix A.

Composite Girder

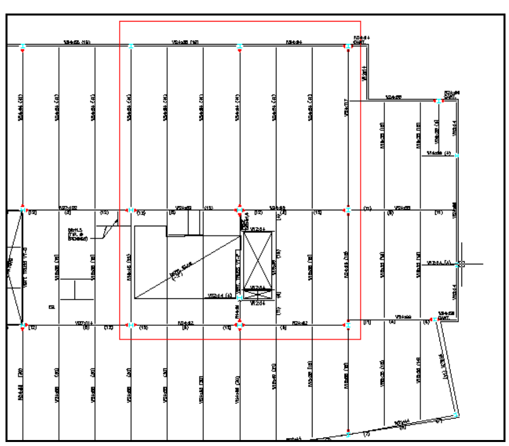
Preliminary analysis for a typical composite girder was done for an existing W24x62 that spans 30' and has a tributary area of approximately 1150 square feet. The existing girder is called to have (12) ¾ diameter x 5-1/2" long studs. The girder to column connection on the girder analyzed was modeled as fixed at both ends. The fixed end connections allow the girder to become part of the lateral system. Through an analysis of the girder and given gravity loads, required size for the girder was determined to be smaller than the girder provided. Again, the increase in size could be due to vibration requirements or extra strength for holes throughout the length of the girder. Further analysis will need to be completed in the future. Calculations for this beam can be found in Appendix A.

Column

A preliminary analysis for a typical column was taken from a column located on the first floor of the retail space. This specific column is part of the moment frame. The size is W14x211 and strength is 50ksi. This column extends two stories and is considered braced at the floor levels. Two different load combinations calculated during analysis are: 1.2Dead+1.6Live and 1.2Dead+0.5Live+1.6Wind. Live loads for calculations required by code are used as 50psf for office space and 100psf for retail space. To achieve reactions specified on the second floor beams of the retail space, it needed to use 50psf live load in the retail space. The member sizes and reactions are the same for retail and office spaces; although it is important to keep in mind the members have higher capacity than needed for required design loads of a retail space. The possibility of drafting error exists when copying reactions onto drawings. For analysis purposes, live loads were calculated to meet the code requirements.

When analyzing the load combination of 1.2Dead+1.6Live(reduced), the requirements for column interaction equation were not met. Assumptions made during the process include fifty percent of the moment from each girder being transferred to the column. To achieve an exact percentage, a full moment distribution will need to be performed on the entire frame. Another difference in calculations may be due to a difference in live load calculation for retail space. When using 50psf live load, column interaction requirements are met.

The second load combination analysis, 1.2Dead+0.5Live(reduced)+1.5Wind, satisfies the column interaction requirements. An assumption made during the calculation process is that the eccentric braces in the vertical truss will resist 80% of the lateral load. To verify this assumption, the wind frame was modeled in SAP2000 with only wind loads factored at 1.0. Results were similar to the assumption made. This load combination developed loads well under the capacity of the column. It was determined 1.2Dead+0.5Live+1.5Wind does not control in the east/west direction. However, eccentric braces frame into the column in the north/south direction. The north/south direction will need to be further analyzed in the future to accurately determine the loads governing the column design. Also in the future, second order effects on the column will need to be considered. Detailed calculations for this column can be found in Appendix A.



Lateral Load Spot Checks

The vertical truss VT-3 on gridline 3 was chosen for the lateral load spot check. This frame runs in the east/ west direction and extends from ground level to roof level in each bay. With the exception of one bay framed with eccentric braces, VT-3 is a moment frame truss. Wind loads were the controlling lateral load, therefore unfactored wind loads were distributed on the frame according to tributary area. Further analysis will need to be completed in the future to verify the relative stiffness approximation of the frame.

Total deflection resulted in 6.23" at the top corner of the truss, opposite the wind application side. To adjust for reasonable serviceability, a reduction of 85% was applied to result in a deflection of 5.3." This deflection satisfies H/400 requirements for deflection of the building (5.82").

Three different load combinations were analyzed aside from 1.0*Wind to determine stresses in the truss members. These load cases are as follows:
1.4*D
1.2*D+1.6*L
1.2*D+0.5L+1.6W
General strength requirements were met for the members in the frame. Axial deformations along the column line E-3 resulted in a total of 2.5" at the top of the frame under gravity loading. Further calculations will need to be performed in the future to verify that cause of the deflections is due to modeling error and not over loading of the columns.

SAP2000 was used for the analysis. Result printouts are available upon request. Calculation of unfactored gravity point loads applied to the frame from girders can be found in Appendix A.

ANALYSIS & CONCLUSIONS

Due to Lockwood Place's location and primary use being retail and corporate, specific building codes govern the loading and design of the building. The primary building codes followed in the design are the 1996 BOCA Code and the 1997 Baltimore Building Code. Various other standards apply such as ASCE-7.

Gravity forces

Dead loads in the building are comprised of, but not limited to concrete slab weight, MEP systems, metal deck, paving, insulation, and lighting. Live loads in the building vary according to occupancy. Although the live loads are specified as 100psf for retail space design, a live load of 50psf is more compatible with reactions found on the members in existing drawings. Snow loads were calculated as 22.5psf, matching the load specified on the drawings. Because of multiple set backs and roofs on the building, five different drift load cases were analyzed. The greatest drift pressure to be found was approximately 50psf.

Lateral forces

Wind pressures against all sides of the building for windward and leeward winds have been determined according to an updated ASCE-7 code rather than when the building was designed. Wind loads were found according to a Main Wind Resisting System (MWRS). The building was analyzed as flexible and fully enclosed. Baltimore sits on the dividing line between a design wind speed of 90mph and 100mph. The greater wind speed was used in calculations for conservative purposes. Despite new codes, wind pressures were determined to be similar to that of the original. To verify similarities, unfactored wind pressures were applied to a single lateral frame based on tributary area and reasonable deflections were calculated for the building.

Seismic loads were calculated according to updated code ASCE-7-05. There is no existing information on the drawings concerning seismic loading. These calculations may not have been required by code at the time, given Baltimore is located in a low seismic region. Values for seismic calculations were obtained by government hazard maps. The seismic resisting system was analyzed as a 'steel system not specifically detailed for seismic resistance.' The response modification factor (R) was used as 3 and the period as 1.768. Building weight was found by calculating the dead weight of each floor including: superimposed dead weight, self weight, and permanent equipment. A total base shear was found as approximately 275(k), allowing wind to control the lateral forces on the building.

Frame Member Analysis

After analyzing two typical beam sizes and a typical girder size, all were determined to be oversized. Reasonable causes of this include additional size for vibration criteria and additional capacity to account for holes that are located at various points throughout the beams. Although the floor is a composite system, beams and girders met live load deflection criteria for noncomposite members.

After analyzing a typical column for two different load combinations, the column proved to be reasonably acceptable. Although the first load case of 1.2D+1.6L failed by a small percentage in the interaction equation, several variables based on assumptions may contribute to the margin of error. A major assumption that could have been a factor is the percentage of moment that the column takes from the girders it supports. To determine an accurate percentage, moment distribution will need to be computed. The second load case of 1.2*D+0.5L+1.6W was under the requirements for the interaction equation. Assumptions that may vary the results include: the percentage of moment the column takes from the girders, distribution of wind loads according to tributary area, and percentage of stiffness the column analyzed has relative to the other columns its frame. The wind load was analyzed only in the east/west direction. To accurately determine the loads governing the design of the column, lateral forces in the north/south direction will also need to be examined.

The analysis computer model of a vertical truss, VT-3, developed in SAP2000, analyzed stresses and deflections in the lateral truss members. After analyzing four different load combinations, deflections at the top of the building and stresses throughout the frame proved to be acceptable. Under a load of 0.85*Wind, total deflections are approximately 5.3” at the top west corner of the building. Result printouts of this model are available upon request.

APPENDIX A
CALCULATIONS

Seismic Load

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 sits 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 from 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**

Wind Loads

$Ta= 0.02*194^{0.75}= 1.04> 1.0$. 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;
$G_f=0.925*(1+1.7*Iz*(g_Q{}^2Q^2+g_R{}^2R^2)^{1/2})/(1+1.7g_VIz)$
$g_Q = g_V=3.4$
$g_R = (2\ln(3000*).96)^{1/2} + 0.577/(2\ln(3000*0.96)^{1/2}= 0.4136$
$N_1= 0.96*(760.9)/135 = 5.41$
$Lz= 650*(116.4/33)^{1/8} = 760.9$
$Vz= 0.8*(116.4/33)^{1/9}*100*(88/60) = 135$
$Iz= 0.15*(10/116.4)^{1/6}=0.11$
$Rn= 7.47*5.41/(1+10.3*5.41)^{5/3}= 0.048$
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
$({}^1/_{0.5}*.145*0.048*(.13)*(0.53+0.47*0.04))^{1/2}= 0.22$E/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
Leewad	-35.03	Leewad	-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	17.87	14.61	448.53	376.79
2	18	16	34.82	28.64	430.66	362.19
3	34	13.5	31.58	26.18	395.84	333.55
4	47.5	13.5	29.70	24.74	364.26	307.37
5	61	13.5	30.40	25.43	334.56	282.63
6	74.5	13.5	30.79	25.81	304.16	257.20
7	88	13.5	31.43	26.44	273.37	231.40
8	101.5	13.5	31.95	26.94	241.94	204.96
9	115	13.5	32.20	27.19	209.99	178.02
10	128.5	13.5	32.65	27.63	177.79	150.83
11	142	13.5	32.85	27.82	145.14	123.21
12	155.5	14.5	34.48	29.25	112.29	95.39
Penthouse	170	18	40.46	34.39	77.82	66.14
Low Roof	188	6	29.88	25.40	37.35	31.75
High Roof	194		7.47	6.35	7.47	6.35

*Tributary Width of VT-3 is 60.75'

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

Snow Drift Loads

Drift Load Calculation Guide:

1. E/W 12th level to penthouse elevation difference
2. E/W penthouse to machine room elevation difference
3. N/S penthouse to machine room elevation difference
4. N/S 12th floor to machine room elevation difference
5. N/S 12th floor to penthouse elevation difference

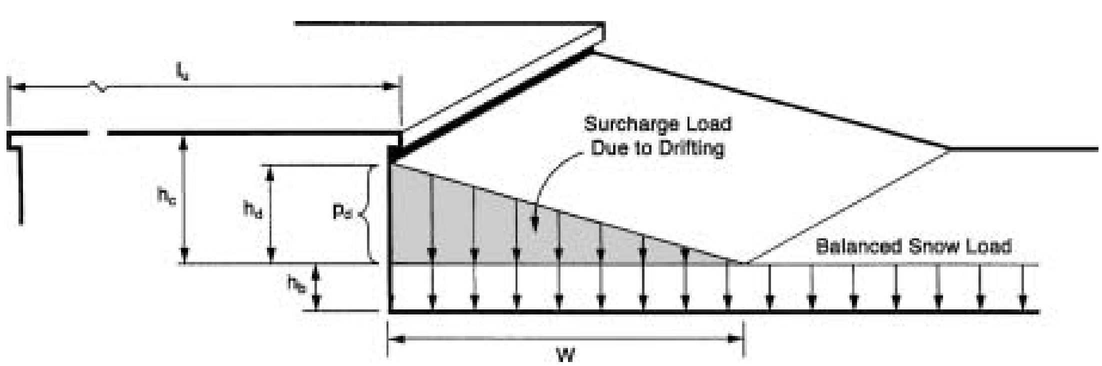
Drift #1	
Roof Length	
Upper	47.67
Lower	12
Wall Height	21.5
hd leeward (ft)	2.29
hd windward (ft)	0.67
hd(ft)	2.29
gamma	17.25
hb (ft)	1.30
w	9.17
pd	39.54

Drift #2	
Roof Length	
Upper	73.9
Lower	47.67
Wall Height	7.5
hd leeward (ft)	2.89
hd windward (ft)	1.72
hd(ft)	2.89
gamma	17.25
hb (ft)	1.30
w	11.56
pd	49.84

Drift #3	
Roof Length	
Upper	31.5
Lower	29.5
Wall Height	7.5
hd leeward (ft)	1.80
hd windward (ft)	1.30
hd(ft)	1.80
gamma	17.25
hb (ft)	1.30
w	7.21
pd	31.10

Drift #4	
Roof Length	
Upper	31.5
Lower	45
Wall Height	25.5
hd leeward (ft)	1.80
hd windward (ft)	1.67
hd(ft)	1.80
gamma	17.25
hb (ft)	1.30
w	7.21
pd	31.10

Drift #5	
Roof Length	
Upper	72
Lower	45
Wall Height	20
hd leeward (ft)	2.85
hd windward (ft)	1.67
hd(ft)	2.85
gamma	17.25
hb (ft)	1.30
w	11.40
pd	49.18



Composite Deck Spot Check

w= (1.2*56 +1.6*100) + 1.2*(3/4)/12*(115) = 235.8psf
-consider 3/4" topping during construction
Uniform live load service capacity= 300psf>235.8psf……………………OK
United Steel Deck Catalog:
Max unshored for 3 spans= 11.21 for 6.5" total slab thickness> 10'…… OK

Beam Spot Check

Beam 1: W24x84 (41)
f'c= 3,500psi
b_{eff}= 10*12= 120" ← controls
45*12/4= 135"
W=(1.2*56+1.6*50)=147.2psf
w= 147.2*10/1000= 1.472 (k/ft)
Mu= 1.472*45^2/8= 372.6 (k*ft)
Vu= 1.472*45/2= 33.3 (k)
SigmaQn= (41)*19.15=785.15← shear stud capacity
a= 785.15/(.85*3.5*120)= 2.20"
Y_2= 6.25-1.20= 5.15"
Phi-Mn=1408(ft*k)>> 372.6 (ft*k)….. OK

Construction Loading:
Delta Dead= ($^5/_{384}$)*45^4*1728*(0.46)/(29000*I); I_{needed}= 1463.5 in^4 < I_{actual}
Delta Live= ($^5/_{384}$)*45^4*1728*(0.2)/(29000*I_{LB}); I_{needed}= 424.3in^4 < I_{actual}
Mu= [1.2*(.46)+1.6*(.20)]*45^2/8=221< 503 (ft*k)….OK
Live Load deflection:
Delta/360= 45*12/360= **1.5"**
Delta actual= ($^5/_{384}$)*45^4*1728*(0.8)/(29000*2370)= 1.07" <1.5" … OK
Phi-Mn with no composite action is 840(ft*k)

Beam 2: W18x35 (18)
f'c= 3,500psi
b_{eff}= 10*12= 120"
30*12/4= 90" ← controls
W=(1.2*56+1.6*50)=147.2psf
w= 147.2*10/1000= 1.472 (k/ft)
Mu= 1.472*30^2/8= 165.6 (k*ft)
Vu= 1.472*30/2= 22 (k)
SigmaQn= (18)*19.15=345← shear stud capacity
a= 345/(.85*3.5*90)= 1.29"
Y_2= 6.25-1.29/2= 5.6"
Phi-Mn=482.3(ft*k)>> 165.6 (ft*k)….. OK

Delta/360= 30*12/360= **1.0"**
Delta actual= $(^5/_{384})*30^4*1728*(0.8)/(29000*510)$= 0.99" < 1.0" … OK
Phi-Mn with no composite action is 249 (ft*k)

*Construction dead and live loads are unfactored for deflection purposes.
*****Hand calculations are available upon request.

Girder Spot Check

Girder connections are fixed at both ends:
f'c= 3,500psi
b_{eff}= 30*12/4= 90" ← controls
Tributary width= (45/2+31.5/2)= 38.25'
P= (45/2*10+31.5/2*10)*147.2= 56 (k)
Mu(-)= $(56*10^2*20)/30^2 + (56*20^2*10)/30^2$= 373.3 (ft*k)
Mu(+)= 56*10-373.3= 186.7 (ft*k)
Solved by method of super position
373.3 (ft*k) controls maximum moment in the beam.
Vu= 56*2/2= 56 (k)
Number of Studs= 12*19.15=230← shear stud capacity
Check a= (230)/(.85*3.5*90)= 0.86"
Y_2= 6.25-0.86/2= 5.82"
Phi-Mn=827(ft*k) at Y_2= 5.5" >>373.3(ft*k)

Construction Loading:
w= 20*1.6+56*1.2=95.6psf
P= (45/2*10+31.5/2*10)*95.6/1000= 36.6(k)
Mu(-)= $(36.6*10^2*20)/30^2 + (36.6*20^2*10)/30^2$= 244 (ft*k)<574 (ft*k)…OK
Mu(+)= 36.6*10-244= 122 (ft*k)
Live Load Deflection:
Delta Actual=$23*10^2*15^2*(3*20*30-3*20*25-20*15)/(6*29000*30^3*1550)*2$=0.16"
Delta/360= 30*12/360= 1.0">0.16"
Phi-Mn with no composite action is 574(ft*k)

*Deflections are calculated as a result of superposition.
*Detailed hand calculations are available upon request.

Column Spot Check

Column Size: W14x211
G_{BOT}= 10
G_{TOP}= $\frac{2660/(18*12) + 2660/(16*12)}{1830/(30*12)+1550/(12*30)}$= 2.79
KxLx= 2.12*18/1.61= 23.7’ >18’(KyLy) ………..23.7’ controls
Dead Load= 63psf retail, Live Load= 100psf retail
*will assume no live load reduction because of possible public assembly
Load Combination: 1.2*D+1.6*L

FEM from girder: (1.2*63+1.6*100)*10*30.75/1000= 72.45 (k)
Negative FEM by method of superposition= 483(ft*k)
*Assume half goes into column from each girder
Interaction Equation: $\frac{1507.73}{1930.0}$+ $\frac{8*483}{9*650}$= 1.08 > 1.0 ……. NOT OK

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)	Factored Load(k)
2	922.5	63	100	100	150.37	217.34
3 to 12	1147.5	56	50	20	872.10	1138.32
Penthouse						
Machine	236.5	56	125	125	42.81	63.19
Tenant	236.5	56	100	40	22.70	31.03
Exterior	337.5	14	31.1	31.1	15.22	22.46
Roof						
High	236.5	14	22.5	22.5	8.63	12.49
Low	236.5	14	50	50	15.14	22.89
				Total	**1126.97**	**1507.73**

Column Loads (E-3) Load Combination 1.2D+0.5L						
Level	AT(sq.ft.)	Dead Load (psf)	Live Load (psf)	LL Reduction	Total Load (k)	Factored Load(k)
2	922.5	63	100	100	150.37	115.87
3 to 12	1147.5	56	50	20	872.10	885.87
Penthouse						
Machine	236.5	56	125	125	42.81	30.67
Tenant	236.5	56	100	40	22.70	20.62
Exterior	337.5	14	31.1	31.1	15.22	10.92
Roof						
High	236.5	14	22.5	22.5	8.63	6.63
Low	236.5	14	50	50	15.14	9.89
				Total	**1126.97**	**1080.47**

Load Combination: 1.2*D+0.5*L+1.6*W

P= (1.2*63+0.5*100)*10*30.75/1000= 25 (k)

FEM= $25*10*20^2/30^2+25*20*10^2/30^2$= 167 (ft*k)

*Assume columns takes half of the moment from each girder

Story Shear at second level= 362.2 (k)

2 parts of 20% of lateral load= 24 (k)

1.6W= 1.6*24*(18/2)= 345.5 (ft*k)

Interaction Equation:

$\frac{1080.47}{1930.00} + \frac{8*512.3}{9*1460}$= 0.87 <1.0 ……………..OK

Vertical Truss spot check

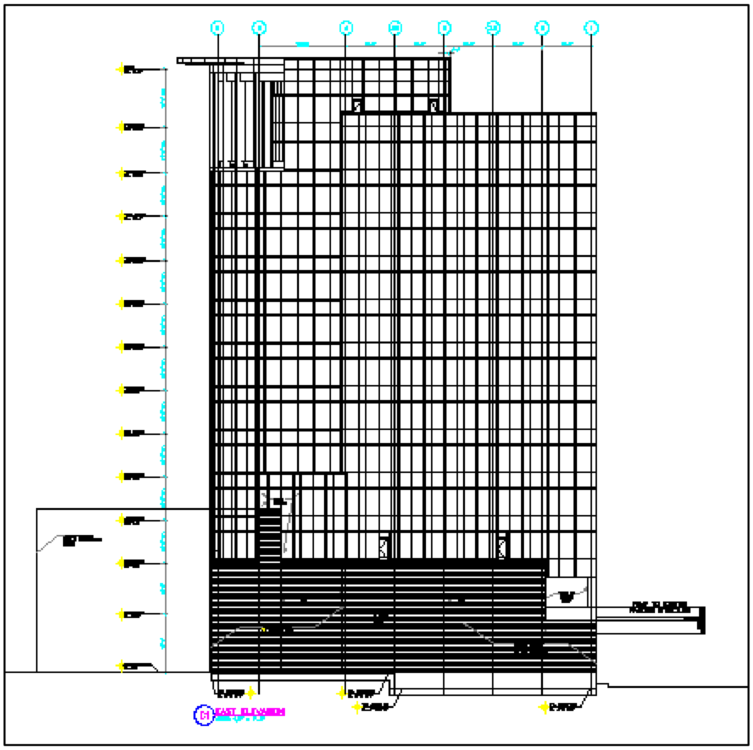
2nd floor point loads:
Dead: 63*(31.5/2+30/2)*10= 19.4 (k)
Live: 100*(31.5/2+30/2)*10= 30.8 (k)

3rd – 12th floor point loads:
Dead: 56*(31.5/2+45/2)*10= 21.4 (k)
Live: 50*(31.5/2+45/2)*10= 19.1(k)

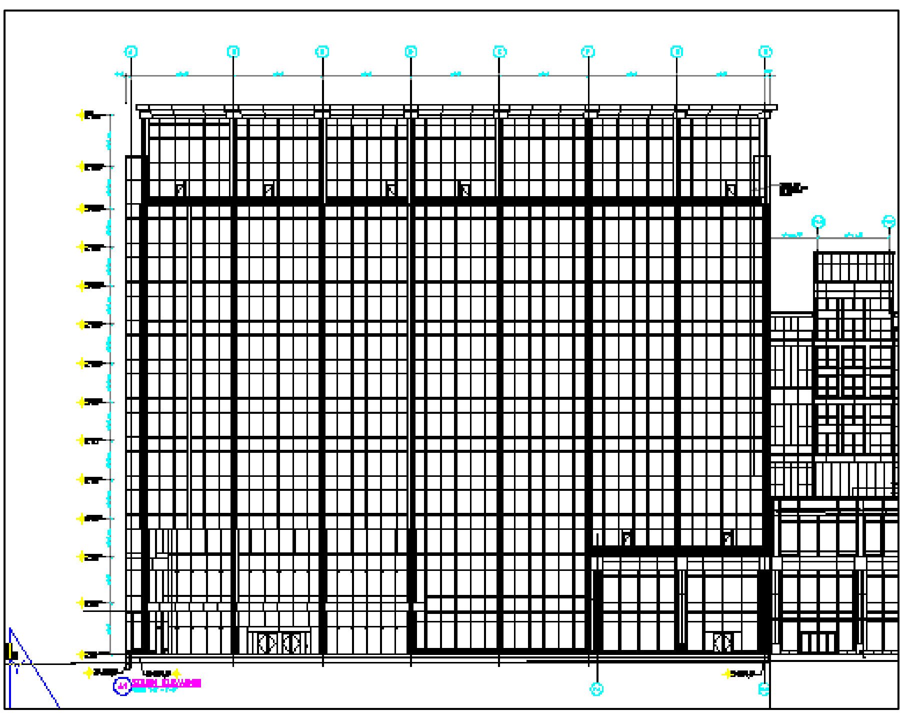
Penthouse Loads:
Distributive:
Dead: 14*45/2= 0.315klf
Live: 20*45/2= 0.225klf
Point:
Dead: 56*(31.5/2)*8=8.82 (k)
Live: 50*(31.5/2)*8= 7.88 (k)
Machine: 125*(31.5/2)*10= 19.7 (k)

Roof:
Distributive:
Dead: 14*31.5/2=0.22klf
Live: 20*31.5=0.315klf

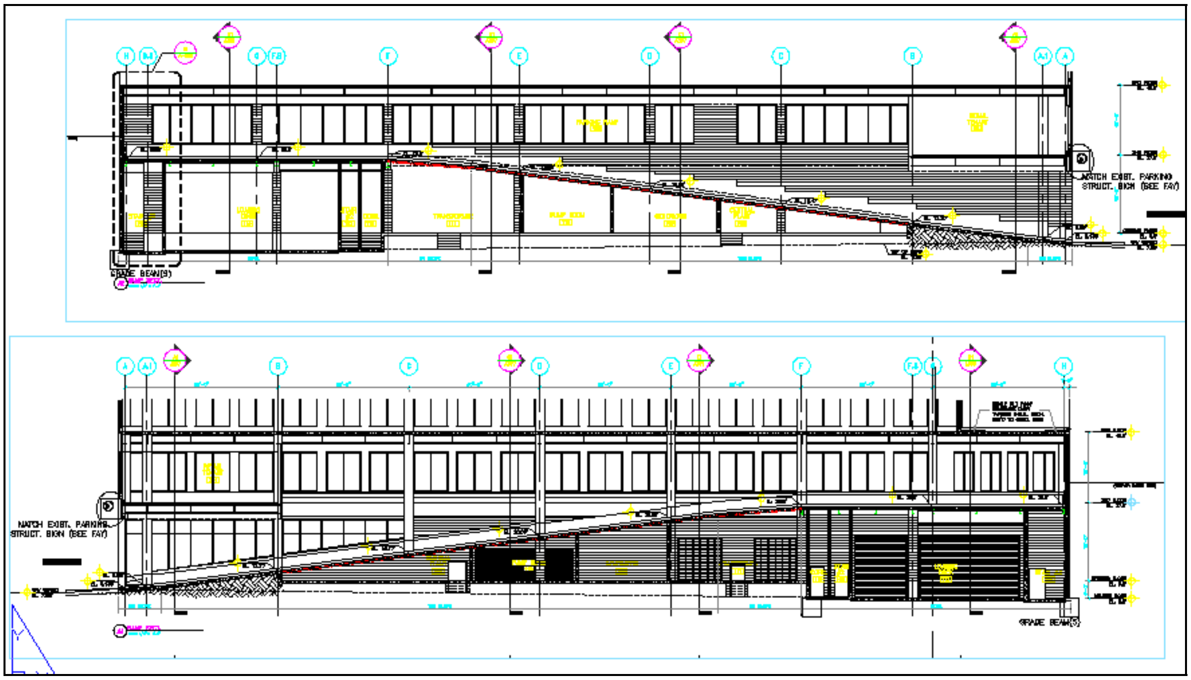
APPENDIX B
LOCKWOOD PLACE



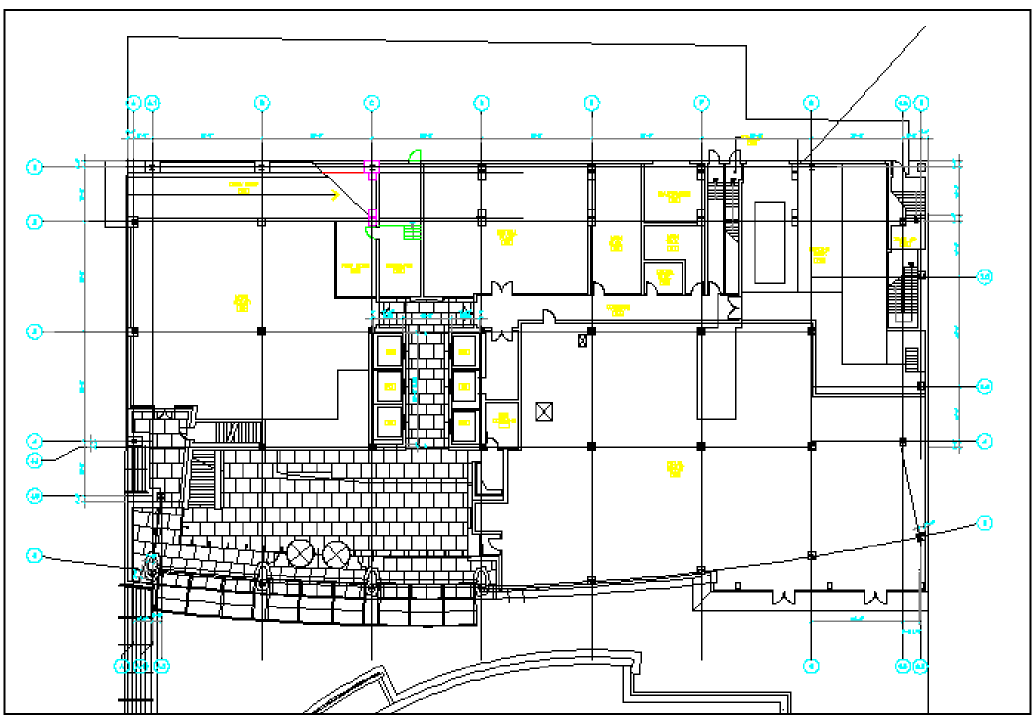
North / South Elevation



East / West Elevation



Rear Parking Garage Ramp Elevation



First Floor Layout