Technical Report III
Lateral System Analysis and Confirmation of Design

Life Sciences Building
The Pennsylvania State University, University Park, Pennsylvania

Executive Summary|

This report is an in – depth study of the lateral force resisting systems for the Life Sciences Building at The Pennsylvania State University – University Park Campus, University Park, Pennsylvania. The building was designed from 1999 and completed in 2004. The building is 'L' shaped, 6 floors (97') tall, and 154,000 GSF with a mechanical penthouse and has concrete floors with a steel frame using composite floor deck, composite beams and composite girders.

The gravity framing system consists of concrete slabs on composite steel deck. The composite steel deck is supported by composite steel beams and girders which frame into steel columns.

The building lateral system consists of moment resisting frames, concentrically braced frames, eccentrically braced frames, and frames that are hybrid combinations of moment and braced frames. In the east – west direction there are three moment frames and three hybrid frames that are combinations of moment and eccentrically braced frames. In the north – south direction there are three concentrically braced frames, two eccentrically braced frames, and two hybrid moment / concentrically braced frames.

For this report wind and seismic loads were calculated by hand. Then an ETABS model was created that models only the building lateral system with all elements connected by a rigid diaphragm. The wind and seismic loads were compared and wind was found to control. The wind loads were resolved into story forces and applied to the diaphragms at the center of pressure and an analysis was run in ETABS. From the ETABS model distribution to the frames was found, as well as deflections, and member loads. Distributions were verified to be correct, deflections were within code limits, and spot checks ensured that critical members did not fail under lateral loading.

Table of Contents

Building Description|

The Life Sciences Building at The Pennsylvania State University, University Park Campus, University Park, Pennsylvania is a six story steel frame structure that is roughly shaped like an "L". The longer leg of the "L" runs in an east – west direction across the northern edge of the site. The shorter leg of the "L" runs north – south along the west central portion of the site. There is also an attached mechanical vault structure at the end of the long leg of the "L" and a two level above grade connection that ties into the knuckle of the "L".

The building can be conveniently broken down into three sections. The first section – referred to herein as "the long leg of the 'L'" – is the part of the building running east – west along the northern edge of the site occurring to the east of column line C. The long leg of the 'L' contains the bulk of the labs, offices and classrooms. The second section – referred to herein as "the knuckle" – is the part of the building that runs east – west along the northern edge of the site and occurs to the west of column line C. "The knuckle" is also the part of the building where the above grade connection to the Chemistry Building ties into the Life Sciences Building. The third and final section – referred to herein as "the short leg of the 'L'" – is the part of the building that runs north – south along the west central portion of the site and ties into the knuckle at its northern end.

Other notable features of the Life Sciences Building include the two story above grade connection to the adjacent Chemistry Building which occurs on the third and fourth floors. A one level mechanical vault was constructed along with the building at its lowest level and is located on the top of the long leg of the "L" (far east side of building). This mechanical vault is constructed entirely of reinforced concrete and its roof is used as a loading dock / truck parking area for the Life Sciences Building. A greenhouse is located on the top of the short leg of the "L". The greenhouse is located on the fourth floor which is also the rooftop of the short leg of the "L" (southernmost portion of building).

Floors of the Life Sciences Building will be referred to in this and all subsequent reports by using the following convention:

B	Basement	1150'-0"
V	Vault	1156'-6" **
G	Ground Floor	1166'-8"
1	First Floor	1180'-8"
2	Second Floor	1194'-8"
3	Third Floor	1208'-8"
4	Fourth Floor	1222'-8"
P	Penthouse	1236'-8"
R	Roof	1263'-0"

** mechanical vault area attached to and constructed with Life Sciences Building which is located adjacent to main structure with a roof used as a loading dock area.

Building Diagram

Existing Structural System Summary|

Foundation|

The Life Sciences Building uses a combination of several foundation types to adapt to several different base slab elevations and varying subsurface conditions.

The vault area of the building is built on a continuous reinforced concrete mat foundation. Columns and walls of the vault will bear on thickened portions of the mat foundation. The mat foundation will have a thickness of 2'-0" and be reinforced with #6 and #7 bars at 12" o.c. The bearing capacity of the soil underneath the mat foundation is 2 ksf for exterior walls and 2.5 ksf for columns.

The foundation of the long leg of the "L" will consist primarily of reinforced concrete spread footings. The maximum allowed bearing pressure on the soil underneath the spread footings is 6 ksf. Underneath walls the foundation ranges from 1'-6" to 2'-3" thick and from 5'-6" to 10'-2" wide. To support columns the spread footings range from 1'-7" to 4'-0" thick and from 5'-6" to 17'-4" wide.

To support the rest of the building, including the knuckle and short leg of the "L", footings are supported on driven steel H – piles. The soil bearing capacity is considered to be 6 ksi on the gross section area of the steel H – pile (and the skin friction value is currently unknown). The piles used are HP10x57 and HP12x74 sections with allowable working loads of 100 k and 130 k respectively. Piles are driven in groups to an average depth of 25' and capped. Piles are driven vertically in the center of pile caps and battered outward on the perimeter of pile caps on a 1:6 (H:V) batter. The piles are arranged in groups of 2,3,4,5,6,8,11, and 16. The pile caps are reinforced concrete and range in thickness from 3'-0" to 5'-0" deep. Grade beams span between pile caps to support the exterior walls.

Floor Framing|

The typical basement slab on grade is 6" of 4000 psi concrete on 6" of PennDOT 2A aggregate reinforced with WWF6x6 – W4xW4. The typical ground level slab on grade is 5" of 4000 psi concrete reinforced with WWF6x6 – W2.9x2.9. The typical floor deck is composite 18 gage, 2" thick fluted with 4-1/2" of concrete cover for a total thickness of 6-1/2". The concrete is normal weight, 4000 psi with one layer of WWF4x4 – W5.5xW5.5. All beams and girders are composite steel wide flange sections using 5" long, ¾" diameter shear studs welded directly to the beam. The shear studs have a shear transfer capacity of 13.3 k/stud.

The basement level of the Life Sciences Building only occurs underneath the long leg of the "L". The basement level of the long leg of the "L" and ground floor level of the short leg of the "L" and knuckle are slabs on grade. Slabs on grade in the basement are typically 6" concrete reinforced with one layer of welded wire fabric. Slabs on grade at ground level are typically 5" thick.

Existing Structural System Summary (continued)|

Beginning with the ground floor level of the long leg of the "L" the floor framing system takes on a typical layout. This framing system is typical and occurs on the ground through fourth floors. The typical floor deck is composite 18 gage, 2" thick fluted with 4-1/2" of concrete cover for a total thickness of 6-1/2". The concrete is normal weight, 4000 psi with one layer of WWF4x4 – W5.5xW5.5. Infill beams for the ground through fourth floors are typically composite W16x26 (spaced 8'-0" o.c.) and composite W16x31 (spaced 8'-8" o.c.) with a built in camber and span of 31'-0". The girders supporting the W16x26 infill beams are composite W24x68 and span 31'-0". The girders supporting the W16x26 infill beams are composite W30x99 and span 41'-0".

The knuckle floor framing system starts with a typical slab on grade on the first floor. The framing for the second through fourth floors consists of the typical composite floor system bearing on W21x44 composite beams. Due to the knuckle not being square the span of the W21x44 beams ranges from roughly 34' to 38' and their spacing is between 8' and 9'.

The framing of the short leg of the "L" is typical on the second through fourth floors, but becomes quite complex on the ground floor to accommodate an auditorium with a sloped floor. The floor framing system for the second through fourth floors of the short leg consists of the typical composite floor system bearing on composite W14x22 infill beams. The W14x22 infill beams are spaced at 8'-8" o.c. and span 20'-8". They are supported by W21x57 composite girders which span 26'-0". Each girder supports two infill beams at third points.

The mechanical penthouse level occurs at the top of the long leg of the "L". The penthouse houses air handlers and various other pieces of mechanical and electrical equipment. The penthouse was designed for comparatively heavy live and dead loads so the beams and girders are much larger than the typical floor framing for the long leg of the "L". The penthouse floor structure begins with the typical composite floor deck and slab that can be found throughout the rest of the building. This slab bears into various W18 infill beams ranging from composite W18x40 to W18x97 (used to frame around openings in the slab). The most typical infill beams are W18x46 and W18x50 but larger sizes are also common where slab openings exist or support structures for the mechanical equipment bear down on the infill beam. The typical span of the beams and girders is 31'. The girders are most typically composite steel W33x141 and W33x201.

Existing Structural System Summary (continued)|

Roof Framing|

The typical roof deck is 20 gage, 1-1/2" deep, wide rib steel roof decking. The roof consists of low roofs that are framed as part of the mechanical penthouse floor system. From the low roof, set back in from the building perimeter, a sharply angled roof / wall goes up to form the enclosure of the mechanical penthouse. On the top of the space created by the angled roof / walls there is another flat roof to completely enclose the mechanical penthouse. As stated previously the low roof is framed as part of the mechanical penthouse floor system. The sharply angled roof is framed by noncomposite W18x60 girders running at an angle that is more vertical than horizontal. These girders run from the low roof to the top of the mechanical penthouse enclosure and act as beams / columns by forming the walls and supporting the higher flat roof. The girders are spaced at 31'-0". W12x26 infill beams then span horizontally in between the W18x60 girders. The infill beams span the entire 31'-0" space between the girders and are spaced with three equal spaces measured from the low flat roof to the top of the high flat roof. Finally, the top of the mechanical penthouse covered by the high flat roof is framed by W16x40, W16x31, and W16x26 beams in various configurations that allow large openings for the vents that ventilate the laboratories. The flat roofs are both covered with the typical roof deck. The sloped roof / walls are covered with plywood and light gauge steel framing.

Lateral System|

The lateral force resisting system (and system of columns) is made up of a combination of braced and moment resisting frames. Due to the complex geometry of the footprint of the building; numerous lateral force resisting systems are located throughout the structure. The building is shaped roughly like an "L" with the long side of the "L" running east to west. A steel moment resisting frame runs along each of the long exterior walls of the building in the east – west direction. Additionally in the east – west direction are three combined moment / braced frames located internally in the short leg of the "L". One moment frame runs east –west on the end of the short leg of the "L". Two smaller moment frames also run east – west to support a section of the building that is isolated due to an expansion joint (isolated section not considered in this report). The total number of frames providing lateral support to the building in the east – west direction is eight.

In the north – south direction, three braced frames located inside the long leg of the "L" provide lateral support. Also, on the east end of the long leg of the "L" a braced frame provides north – south lateral support. In the short leg of the "L" one moment frame runs along each exterior wall. Additionally, in the north – south direction, a braced frame located at the outside corner where the long and short legs of the "L" meet provides additional lateral support. Finally, two braced frames provide north – south lateral load resistance to the portion of the building that is isolated due to an expansion joint. The total number of frames providing lateral support to the building in the north – south direction is nine.

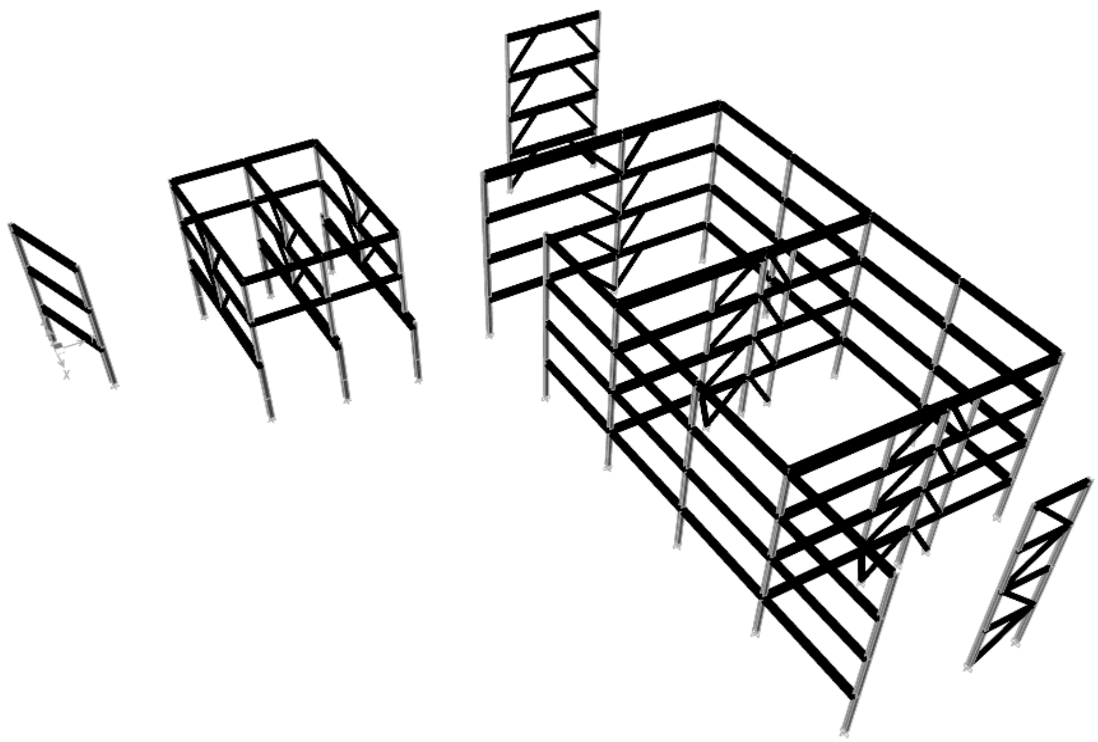
Existing Structural System Summary (continued)|

Columns|

The system of columns and lateral force resisting system is designed so that very few columns aren't involved in a moment frame or braced frame. Most column loading depends on many more factors than just the accumulation of gravity loads. The columns range in size from W10 up to W14. The weights generally vary from 33 lbs/ft to 311 lbs/ft. Estimated column loads vary from 60 k to 1100 k, with the vast majority of column loads in the range of 200 k to 800 k.

Detailed Lateral System Description and Diagrams|

Lateral System Diagram (from ETABS)|



Detailed Lateral System Description and Diagrams|

Detailed Lateral System Description|

The building lateral system consists of moment resisting frames, concentrically braced frames, eccentrically braced frames, and frames that are hybrid combinations of moment and braced frames. In the east – west direction there are three moment frames, and three hybrid frames that are combinations of moment and eccentrically braced frames. In the north – south direction there are three concentrically braced frames, two eccentrically braced frames, and two hybrid moment / concentrically braced frames. The system is further complicated by the fact that although most of the frames are on two orthogonal axes – there are three lateral resisting frames that are rotated at various angles from the orthogonal axes due to architectural constraints. Nearly all of the lateral force resisting frames are tied into frames in the orthogonal direction with moment connections, further complicating the analysis of the system by hand. The lateral frame illustrations shown in this section have their bases at the first floor level.

East – West Direction Lateral System Description|

The lateral system in the east – west direction as stated above consists of three moment frames and three hybrid frames that combine both moment resisting and concentrically braced elements. Two of the moment frames, Moment Frame 1 and Moment Frame 4 occur along the exterior wall of the long leg of the “L”. Moment Frame 1 and Moment Frame 4 are illustrated below:

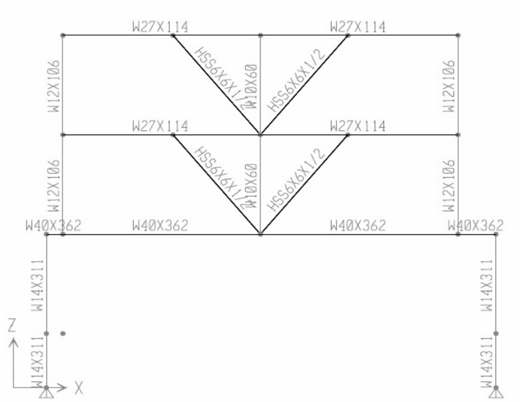
Moment Frame 1

Moment Frame 4

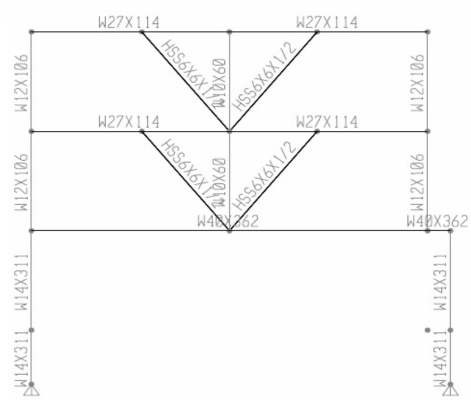
Detailed Lateral System Description and Diagrams (continued)|

East – West Direction Lateral System Description (continued)|

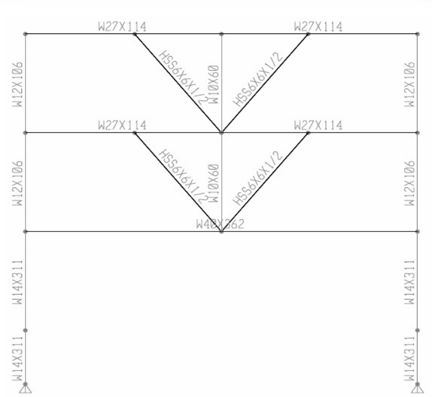
The only other moment frame running in the east – west direction is Moment Frame 9. It runs angled at twenty degrees from the east – west axis at the very end of the short leg of the "L". It is illustrated at the far left of the three dimensional view of the lateral force resisting system shown at the beginning of this section of the report. The three hybrid moment / eccentrically braced frames that take east – west lateral loading are all very similar with slight changes in each frame to adapt to the architectural restrictions of the building. Views of all three hybrid frames are shown below – note the large clear span moment frame on the lowest level. The clear span is needed to allow for an auditorium to be located on the first floor of the building, convenient to the major entrances and exits. These three hybrid frames occur in the short leg of the "L".



Hybrid Frame 5.3



Hybrid Frame 6

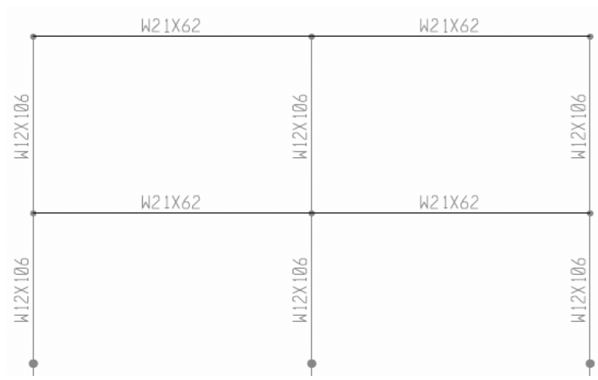


Hybrid Frame 7

Detailed Lateral System Description and Diagrams (continued)

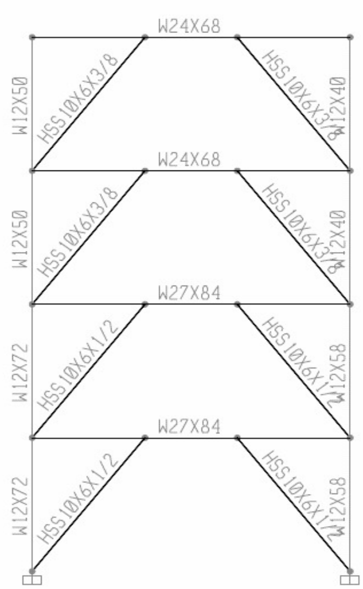
North – South Direction Lateral System Description

In the north – south direction lateral forces are resisted by a total of seven frames. Two of these frames – Hybrid Frame C.2 and Hybrid Frame D.8 – are hybrid frames combining concentrically braced elements with moment resisting elements. These two hybrid frames occur on the outside walls of the short leg of the "L". The upper two stories of each three story frame is illustrated below (bracing in the lowest level of the frame is not visible due to the limitations of ETABS).



Hybrid Frame C.2
Hybrid Frame D.8 (identical)
(first floor braced level not shown)

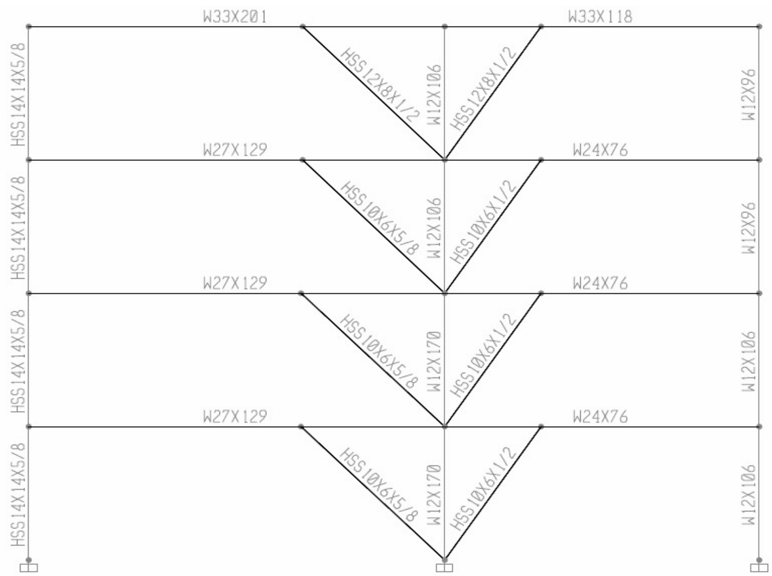
Two eccentrically braced frames are utilized to resist lateral forces on the building in the north – south direction. They are located in the "knuckle" where the long leg and short leg of the "L" meet. The first eccentrically braced frame – Braced Frame C – is one bay wide and is located at the outside corner of the "L". The second eccentrically braced frame – Braced Frame E – is located internally where the long and short legs meet and is roughly in line with the exterior wall of the short leg of the "L". The eccentrically braced moment frames that occur in the "knuckle" of the "L" are shown below.



Braced Frame C

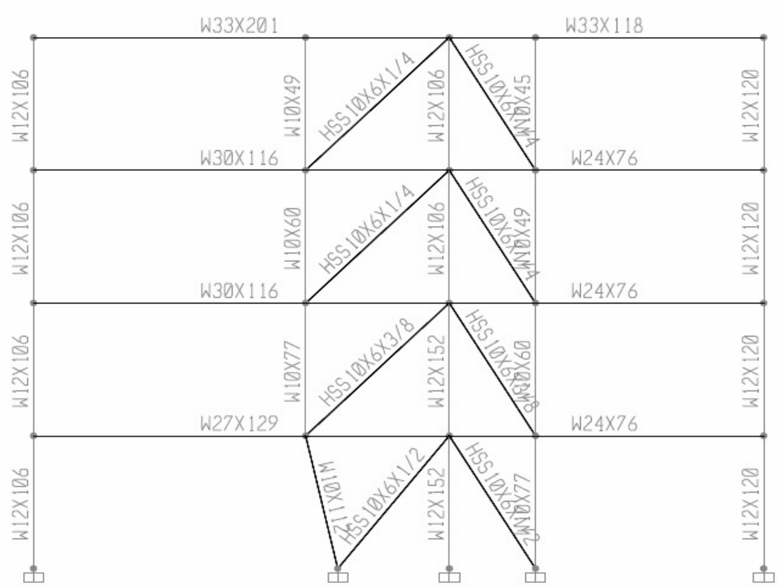
Detailed Lateral System Description and Diagrams (continued)

North – South Direction Lateral System Description (continued)



Braced Frame E

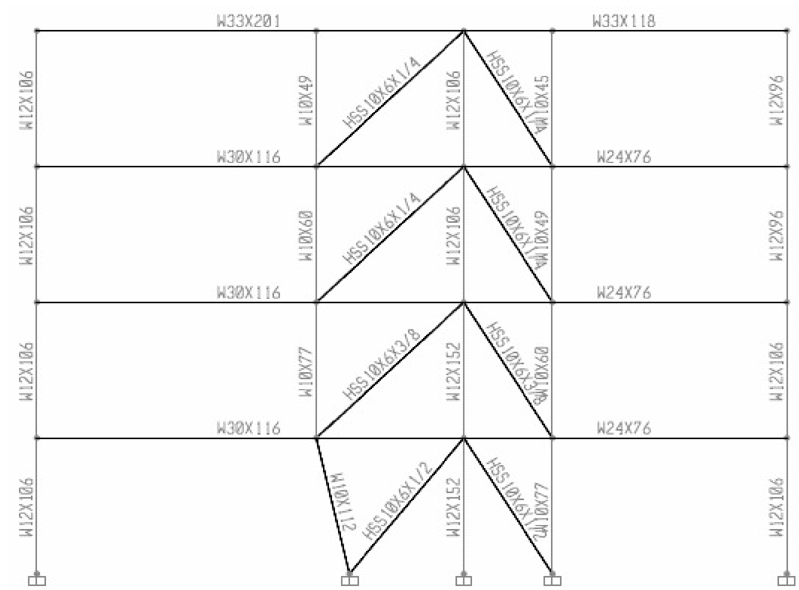
Finally, three concentrically braced frames provide lateral force resistance within the long leg of the "L". Two north – south braced frames – Braced Frame G and Braced Frame J – span the entire orthogonal distance between east – west Moment Frame 1 and Moment Frame 4. The last lateral force resisting frame in the north – south direction – Braced Frame K – is located at the far east end of the building at the end of long leg of the "L". These three frames are illustrated below.



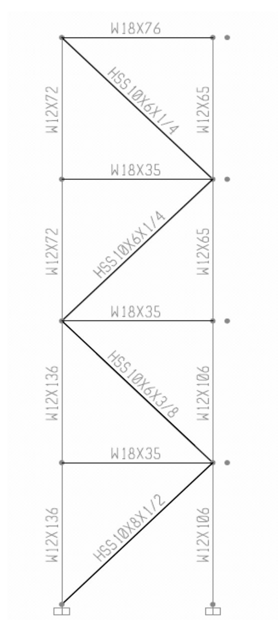
Braced Frame G

Detailed Lateral System Description and Diagrams (continued)

North – South Direction Lateral System Description (continued)



Braced Frame J



Braced Frame K

ETABS Modeling|

ETABS Modeling Simplifications|

The lateral system of the Life Sciences Building is complex to say the least. Several simplifying assumptions were made to enable the modeling of the lateral system (only the lateral system, no gravity elements) in ETABS. The simplifications made in modeling the real structure in ETABS will be listed in the following paragraphs.

The concrete slabs on composite steel deck were modeled as rigid diaphragms throughout the building that tie all of the lateral frames together. Because the first floor is at grade on one side it is assumed that lateral forces distributed to the first floor do not enter the building lateral system but instead are transferred through the diaphragm directly to the foundation. As a result of this the building model's base is located at the first floor and the floors are modeled up to the penthouse level. All of the columns that extend continuously through the first floor and have their bases at some lower level were modeled with fixed base constraints on the first floor. All of the columns that have their bases on the first floor were modeled as pinned, consistent with the actual design of the building. Wind loads (found to control over seismic) were calculated by hand and applied to the rigid diaphragm on each floor at a point corresponding to their center of pressure.

Modeling Procedure|

The wind and seismic loading calculated for Technical Report I was checked and confirmed then used as the loading for this report. Wind load was found to control by doing a hand calculation analysis of the story forces, base shears, and overturning moments due to seismic and wind loading in both the north – south and east – west directions. The frame of the building was created in ETABS using the simplifications discussed in the previous section of this report. To analyze the effects of wind loading on the lateral force resisting system two models were created, one with wind forces applied in the north – south direction and the other with wind forces applied in the east – west direction. The wind loads were calculated by hand and placed into ETABS as unfactored point loads on the rigid diaphragms at locations corresponding to their centers of pressure on the façade of the building. Finally, an analysis of the building was run for both models and the results of this analysis are the basis of this report.

It is recognized that the model of the building in ETABS can be refined to more accurately represent the actual structure of the Life Sciences Building. The model will be developed further as my work with this project continues.

Material Strength|

The following material strengths were assumed in the analysis of the building lateral force resisting system for Technical Assignment III unless otherwise noted in individual calculations:

Reinforced Concrete|

Compressive Strength

f'_c = 4000 psi

Reinforcement Bars (ASTM A615 Grade 60)

f_y = 60000 psi

Welded Wire Fabric (ASTM A185)

f_y = 70000 psi

Structural Steel|

Beams, Columns, Other Framing Members = ASTM A572 Gr. 50

F_y = 50 ksi F_u = 65 ksi

Plates, Bars, Angles = ASTM A36

F_y = 36 ksi F_u = 58 ksi

Structural Tubing = ASTM A500 Gr. B

F_y = 42 ksi F_u = 58 ksi

Structural Pipe = ASTM A501

F_y = 36 ksi F_u = 58 ksi

All bolts will be ¾" ASTM A325N (threads included)

V_n = 15.9 k / bolt

Shear Studs will be ¾" diameter 5" long

V_n = 13.3 k / stud

Steel Deck|

Roof Deck

F_y = 33 ksi

Composite Floor Deck

F_y = 40 ksi

Building Codes – Original Design|

The Life Sciences Building was designed, along with the connected Chemistry Building, in the late 1990s – early 2000s. The programs began development in September 1999. Construction of the Chemistry Building finished up in September 2003. The notice to proceed for construction of the Life Sciences Building was issued in July 2002 and the building was occupied in September 2004. When the Life Sciences Building was originally designed it used the most current building codes at the time:

Building Code / Loading|
- Building Officials and Code Administrators
  - BOCA 1996
- Pennsylvania Department of Labor and Industry
  - PA L&I Title 34 1996
- American Society of Civil Engineers
  - ASCE 7

Reinforced Concrete|
- American Concrete Institute
  - ACI 318 – 95

Structural Steel|
- American Institute of Steel Construction
  - AISC – Codes and Specifications (most current at the time of design)

Cold Formed Steel Decking|
- Steel Deck Institute
  - SDI – Steel Deck Design Manual (most current at the time of design)

Building Codes – Technical Report III|

In the reanalysis of this building the most current building codes at this time will be used. The following codes will be used extensively in the reanalysis and design of the Life Sciences Building:

Building Code / Loading|
- International Code Council
  - IBC 2006
- American Society of Civil Engineers
  - ASCE 7 – 05

Reinforced Concrete|
- American Concrete Institute
  - ACI 318 – 05

Structural Steel|
- American Institute of Steel Construction
  - AISC – 13th Edition Steel Manual

Cold Formed Steel Decking|
- Steel Deck Institute
  - SDI – Steel Deck Institute Design Manual for Composite, Form, and Roof Decks

Live Load|

Live loads used were given on the drawings for the original design. Live loads were compared with recommended values from IBC 2006 and ASCE 7 – 05 for reanalysis. Loads that were higher than recommended values from IBC 2006 and ASCE 7 – 05 were left unchanged from the original design as a conservative assumption. Several loads were specified by the user. The following lists the live load assumptions that were used in the original design – which are also the live loads I will be using in my calculations:

Assembly Areas|	
Fixed Seats	60 PSF
Lobbies / Moveable Seats	100 PSF
Corridors|	
All Floors	100 PSF
Classrooms, Labs, Offices|	
Reducible Live Load	80 PSF
Partition Load	20 PSF **
Electrical / Mechanical Rooms|	
	200 PSF *
Stairs / Landings|	
	100 PSF
Storage Areas|	
Light Storage	125 PSF *
File Areas	User Defined
Special Storage	User Defined

* Indicates that load is non-reducible because it is a heavy live load according to IBC 2006 and ASCE 7 – 05 (S.4.8.2).
** Indicates that load is non-reducible because it is a partition load which will constantly be applied to the structure.

Dead Load|

Dead loads will be taken as the self weights of the building materials as given by their manufacturers and standard practice.

The partition load allowance – as shown in the live load section of this report – will be added to classroom, lab and office areas but was taken as part of the live load for most calculations. However, the partition load and also the live load for mechanical rooms was conservatively assumed to act as dead load for the seismic load calculations.

Additional superimposed dead loads will be added to the classroom, lab and office areas, as well as added to the structures that are directly above mechanical and electrical rooms. The values used for these superimposed dead loads follow:

Classrooms, Labs, Offices|

Collateral Dead Load	10 PSF

Electrical / Mechanical Rooms|

Collateral Dead Load	30 PSF

Snow Load|

Snow loads will be considered in this initial analysis (Technical Assignment III) to a point. In later, more refined, analyses the effects of snow drifting may be considered using ASCE 7 – 05. These simplifying assumptions are made because a significant majority of the building has a flat roof with no obstructions. However, special considerations may have to be made eventually for the small areas of flat roof that are enclosed by parapet walls and a steeply sloped roof / wall for the mechanical penthouse.

I also feel that I can neglect snow loads on the roof in my initial analysis because the roof structure that would be experiencing the snow loads under drifting conditions is also used as the framing for the mechanical penthouse – which was designed for live loads of 200 PSF. The mechanical penthouse floor also serves as the roof for a small portion of the building. The mechanical penthouse floor / roof is a flat plate that extends to the top of the brick perimeter wall of the building. Inside the parapet formed by the brick perimeter wall there is a small section of roof before the steeply sloped roof / wall that encloses the mechanical penthouse. It is in this depression where snow drifts are likely to form in the perimeter around the building. The roof in this depression is also designed for 200 PSF live load because it serves as the mechanical penthouse floor on the other side of the sloped roof / wall. I am extremely confident that no snow drift will ever exceed the 200 PSF that the flat roof subject to drifting was designed for.

Snow Load|

(State College IBC Amendment) 30 PSF (ignoring drifting)

Wind Load|

Wind load was initially calculated in Technical Assignment I and verified in Technical Assignment III using the analytical procedure as prescribed in Section 6.1.2 of ASCE 7 – 05. The wind load calculations are outlined below.

Building Type|	Enclosed (S.6.2 & S.6.5.9) Regular Shaped (S.6.2) Rigid Structure (S.6.2) Approximate Fundamental Period (S.12.8.2.1) T_a = .615 f = 1.626
Basic Wind Speed|	State College, PA V = 90 MPH (S.6.5.4 & Fig. 6-1) Building MWFRS K_d = .85 (S.6.5.4.4 & Table 6-4)
Importance Factor|	More than 300 people congregate in one area College building with capacity greater than 500 people Occupancy Category III (Table 1-1) I = 1.15 (Table 6-1)
Exposure Category|	Surface Roughness B (S.6.5.6.2) Exposure B (S.6.5.6.3) Exposure B, Case II (Table 6-2) α = 7.0 (Table 6-2) Z_g = 1200 ft (Table 6-2) $K_z = 2.01\ (z/z_g)^{2/\alpha}$ [h > 15'] $K_z = 2.01\ (15/z_g)^{2/\alpha}$ [h < 15']
Topographic Factor|	No adjustments needed (S.6.5.7) K_{zt} = 1.0
Gust Effect Factor|	Conservative estimate given (S.6.5.8) G = .85
Internal Pressure Coefficient|	Enclosed building (S.6.5.11 & Fig. 6-5) GC_{pi} = +/- .18
External Pressure Coefficients|	(shown in spreadsheet, next page)

Wind Load (Continued)

ASCE 7 Wind Pressure Calculations

East - West Wind Loading

height range	h (max)	qz	G	Cp	Windward Wall Pressure (PSF)
83.5 '- 97'	97	19.857	0.85	0.80	13.50
63' - 83.5'	83.5	19.025	0.85	0.80	12.94
49' - 63'	63	17.554	0.85	0.80	11.94
35' - 49'	49	16.337	0.85	0.80	11.11
21' - 35'	35	14.840	0.85	0.80	10.09
7' - 21'	21	12.825	0.85	0.80	8.72
0' - 7'	7	11.649	0.85	0.80	7.92

h	qz	G	Cp	Leeward Wall Pressure (PSF)
-	19.857	0.85	-0.428	-7.22

North - South Wind Loading

height range	h (max)	qz	G	Cp	Windward Wall Pressure (PSF)
83.5 '- 97'	97	19.857	0.85	0.80	13.50
63' - 83.5'	83.5	19.025	0.85	0.80	12.94
49' - 63'	63	17.554	0.85	0.80	11.94
35' - 49'	49	16.337	0.85	0.80	11.11
21' - 35'	35	14.840	0.85	0.80	10.09
7' - 21'	21	12.825	0.85	0.80	8.72
0' - 7'	7	11.649	0.85	0.80	7.92

h	qz	G	Cp	Leeward Wall Pressure (PSF)
-	19.857	0.85	-0.5	-8.44

Wind Load (Continued)

Total Wind Pressure Applied to Building

East - West Wind Loading

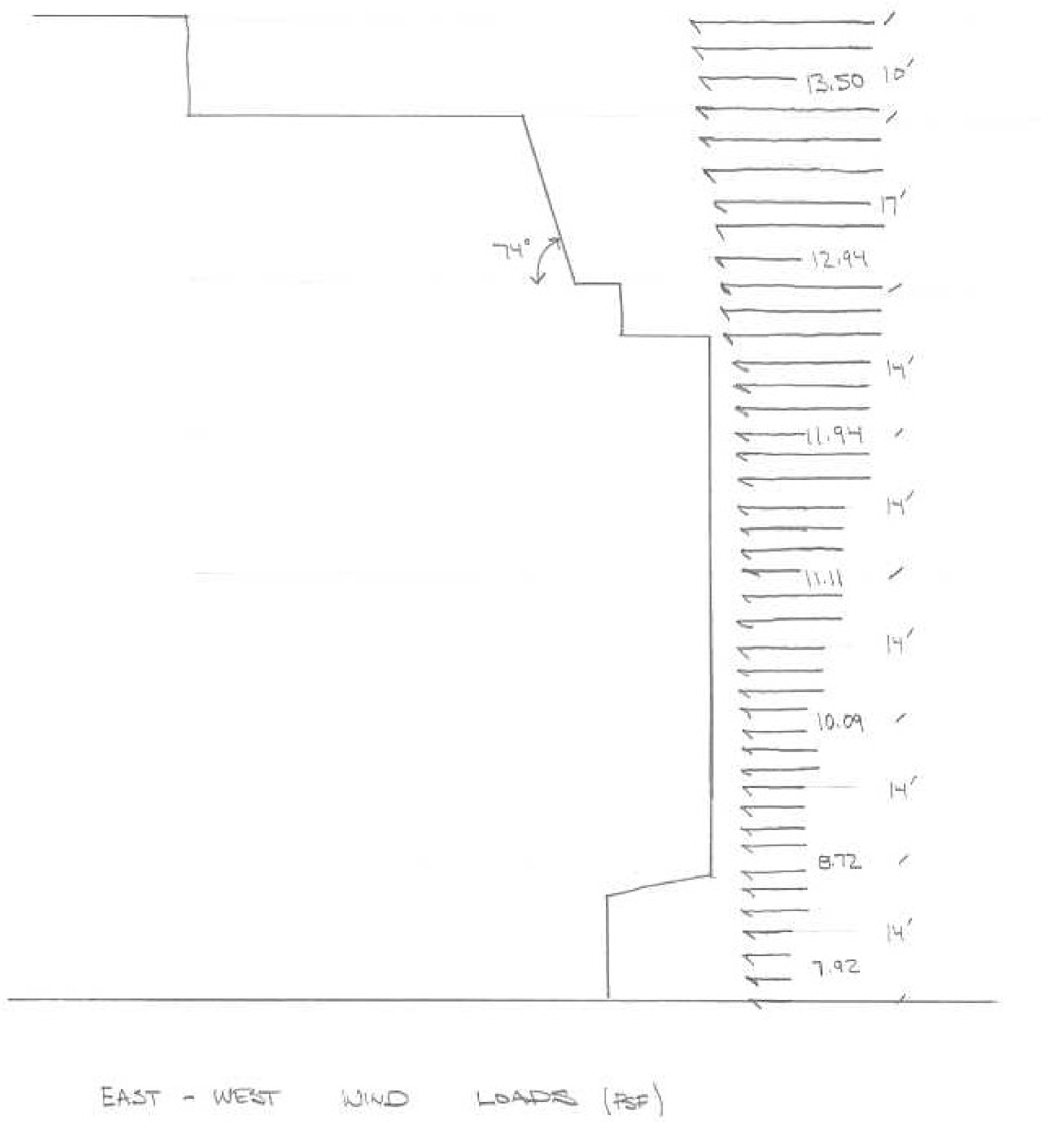
Height Range	Max. Height	Windward Pressure	Leeward Pressure	Total Pressure
83.5 '- 97'	97'	13.50	7.22	**20.72**
63' - 83.5'	83.5'	12.94	7.22	**20.16**
49' - 63'	63'	11.94	7.22	**19.16**
35' - 49'	49'	11.11	7.22	**18.33**
21' - 35'	35'	10.09	7.22	**17.31**
7' - 21'	21'	8.72	7.22	**15.94**
0' - 7'	7'	7.92	7.22	**15.14**

North - South Wind Loading

Height Range	Max. Height	Windward Pressure	Leeward Pressure	Total Pressure
83.5 '- 97'	97'	13.50	8.44	**21.94**
63' - 83.5'	83.5'	12.94	8.44	**21.38**
49' - 63'	63'	11.94	8.44	**20.38**
35' - 49'	49'	11.11	8.44	**19.55**
21' - 35'	35'	10.09	8.44	**18.53**
7' - 21'	21'	8.72	8.44	**17.16**
0' - 7'	7'	7.92	8.44	**16.36**

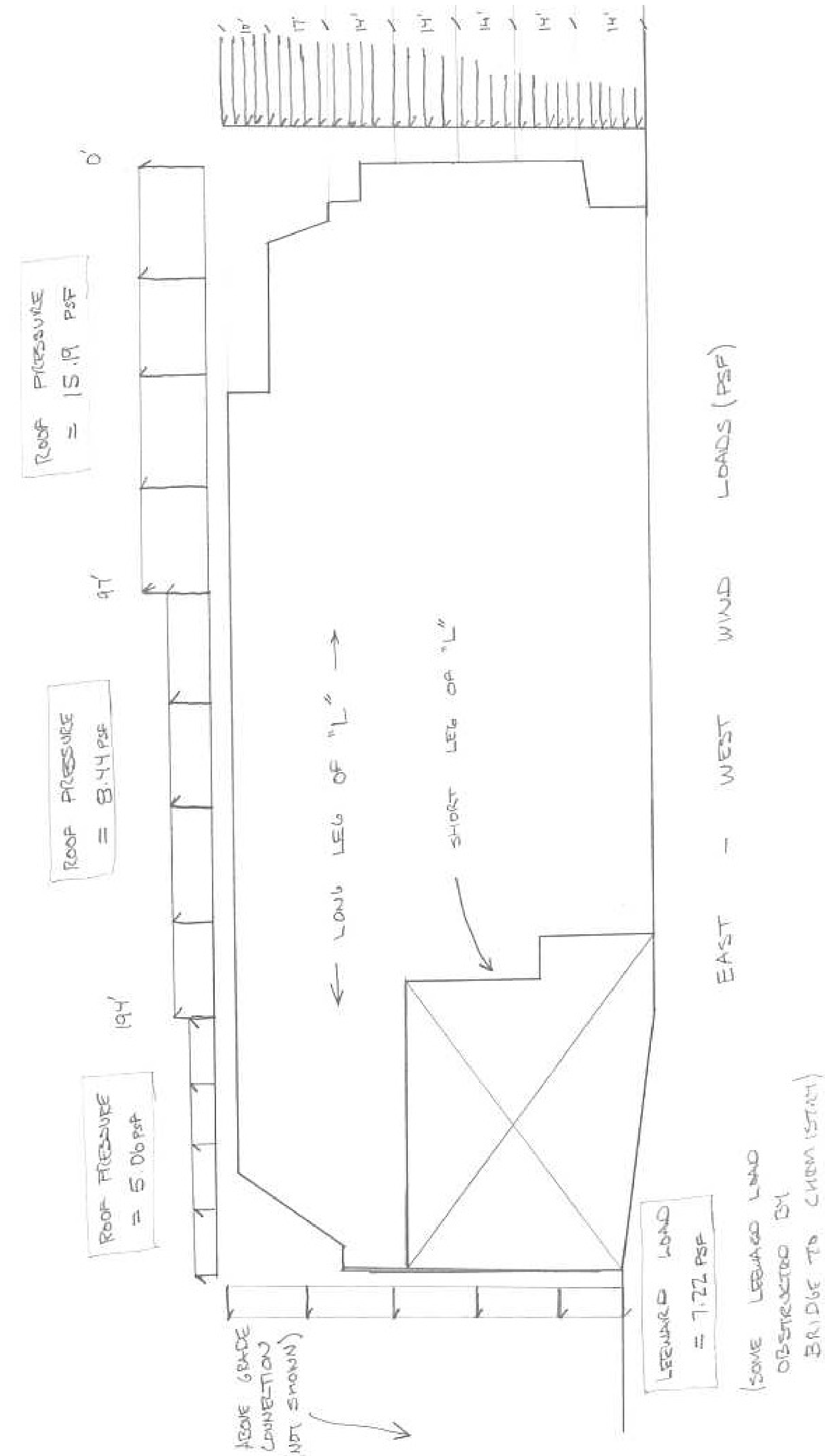
Wind Load (Continued)

East – West Wind Load Diagram - Detail



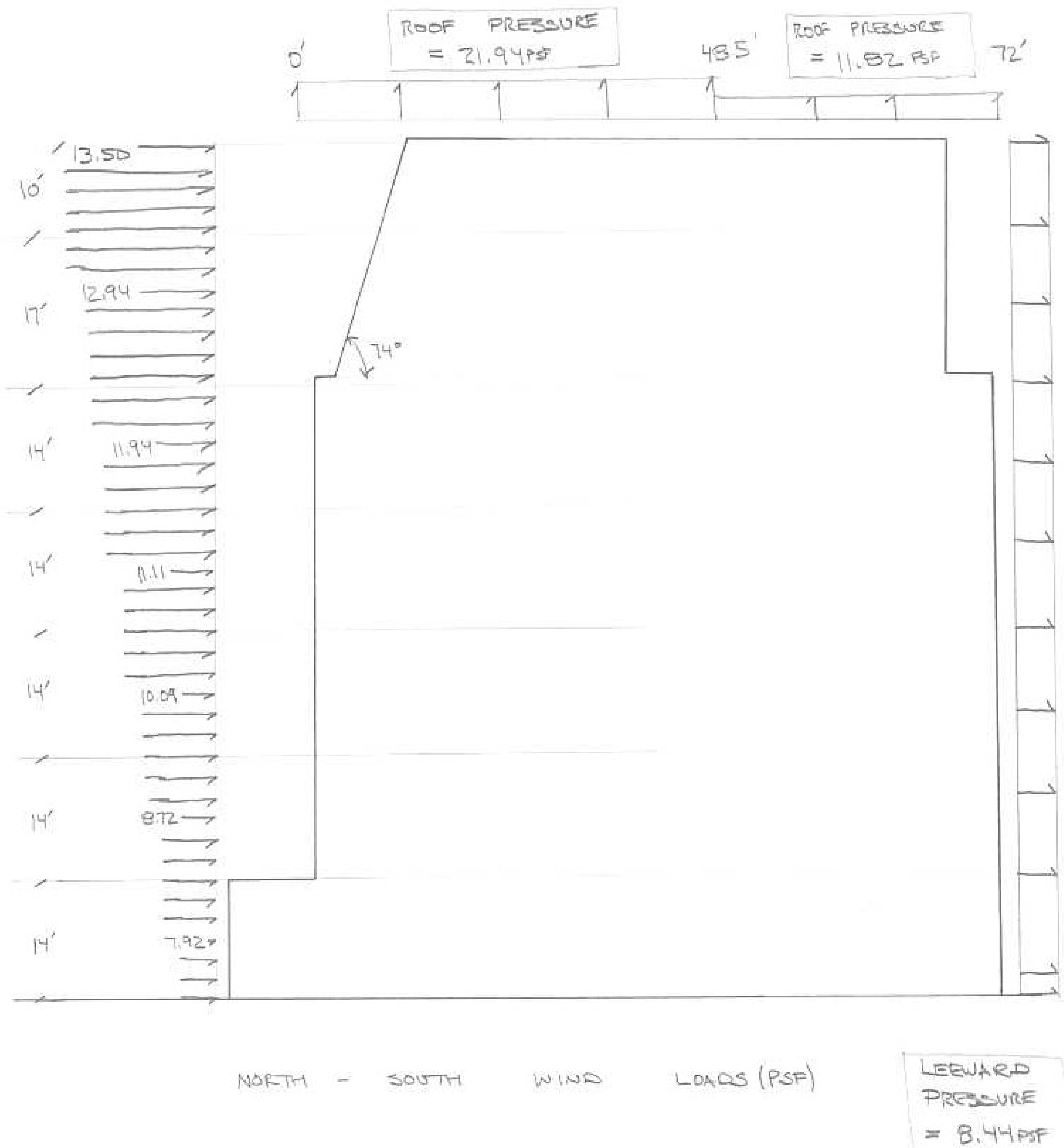
Wind Load (Continued)|

East – West Wind Load Diagram - Overall|



Wind Load (Continued)|

North – South Wind Load Diagram - Overall|



Seismic Load|

Seismic load was calculated in Technical Assignment I and verified in Technical Assignment III using the requirements for *seismic design category "A"* as prescribed in Section 11.7 of ASCE 7 – 05. Geotechnical information came from the geotechnical report provided by Gannett Fleming. A copy of this report is available for review. The weight of mechanical equipment and the partition allowance (given as live loads in this document) were included as dead loads for the purposes of calculating the seismic lateral forces. The seismic force was calculated for all floors, however, the seismic load of the ground and first floor doesn't contribute to the lateral force resisting system because the floors are at or below grade.

Acceleration Parameters|

S_s = .15 (S.11.4.1 & Fig. 22-1)
S_1 = .05 (S.11.4.1 & Fig. 22-2)

Site Class| Hard rock no more than 8' below spread footings
Site Class "B" (S.20.3)

MCE Acceleration Parameters|

S_{MS} = .15 (S.11.4.3 & Table 11.4-1)
S_{M1} = .05 (S.11.4.3 & Table 11.4-2)

Design Acceleration Parameters|

S_{DS} = .10 (S.11.4.4)
S_{D1} = .0333 (S.11.4.4)

Importance Factor| More than 300 people congregate in one area
College building with capacity greater than 500 people
Occupancy Category III (Table 1-1)
I = 1.25 (Table 11.5-1)

Seismic Design Category| Approximate Fundamental Period (S.12.8.2.1)
T_a = .615
Seismic Design Category "A"

Lateral Forces| Distributed to each level using: $F_x = .01(w_x)$

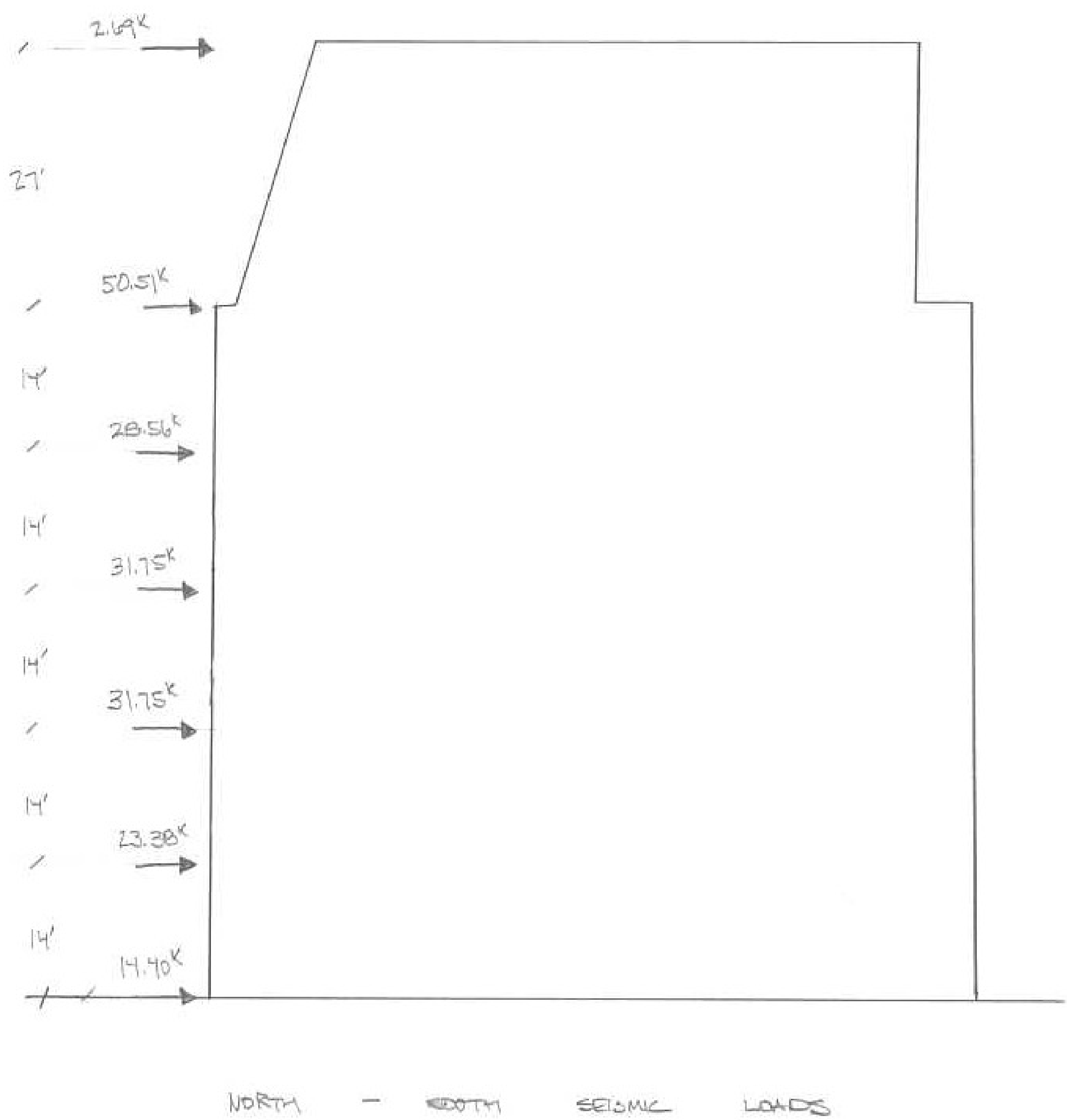
2.69 k	Roof Level
50.51 k	Penthouse Level
28.56 k	Fourth Floor
31.75 k	Third Floor
31.75 k	Second Floor
23.38 k	*First Floor*
14.40 k	*Ground Floor*

Seismic Load (Continued)|

	Floor System					Exterior Wall System					Seismic Load
	Width (ft)	Depth (ft)	Area (ft^2)	Dead Load (lbs/ft^2)	Floor Load (k) (k)	Perimeter (ft)	Height (ft)	Area (ft^2)	Dead Load (lbs/ft^2)	Wall Load (k)	At Story (k)
Roof											**Roof**
	175	48	8400	32	269						**2.69**
Penthouse											**Penthouse**
Long Leg of "L"	232	75	17400	282	4907	446	27	12042	12	145	**50.51**
Fourth Floor											**Fourth Floor**
Total					2535					321	**28.56**
Long Leg of "L"	165	75	12375	109	1349	382	14	5348	60	321	
Short Leg of "L"	64	170	10880	109	1186						
Third Floor											**Third Floor**
Total					2535					640	**31.75**
Long Leg of "L"	165	75	12375	109	1349	382	14	5348	60	321	
Short Leg of "L"	64	170	10880	109	1186	380	14	5320	60	319	
Second Floor											**Second Floor**
Total					2535					640	**31.75**
Long Leg of "L"	165	75	12375	109	1349	382	14	5348	60	321	
Short Leg of "L"	64	170	10880	109	1186	380	14	5320	60	319	
First Floor											**First Floor**
Total					1915					423	**23.38**
Long Leg of "L"	125	85	10625	109	1158	335	14	4690	60	281	
Short Leg of "L"	62	112	6944	109	757	168	14	2352	60	141	
Ground Floor											**Ground Floor**
Long Leg of "L"	125	85	10625	109	1158	335	14	4690	60	281	**14.40**

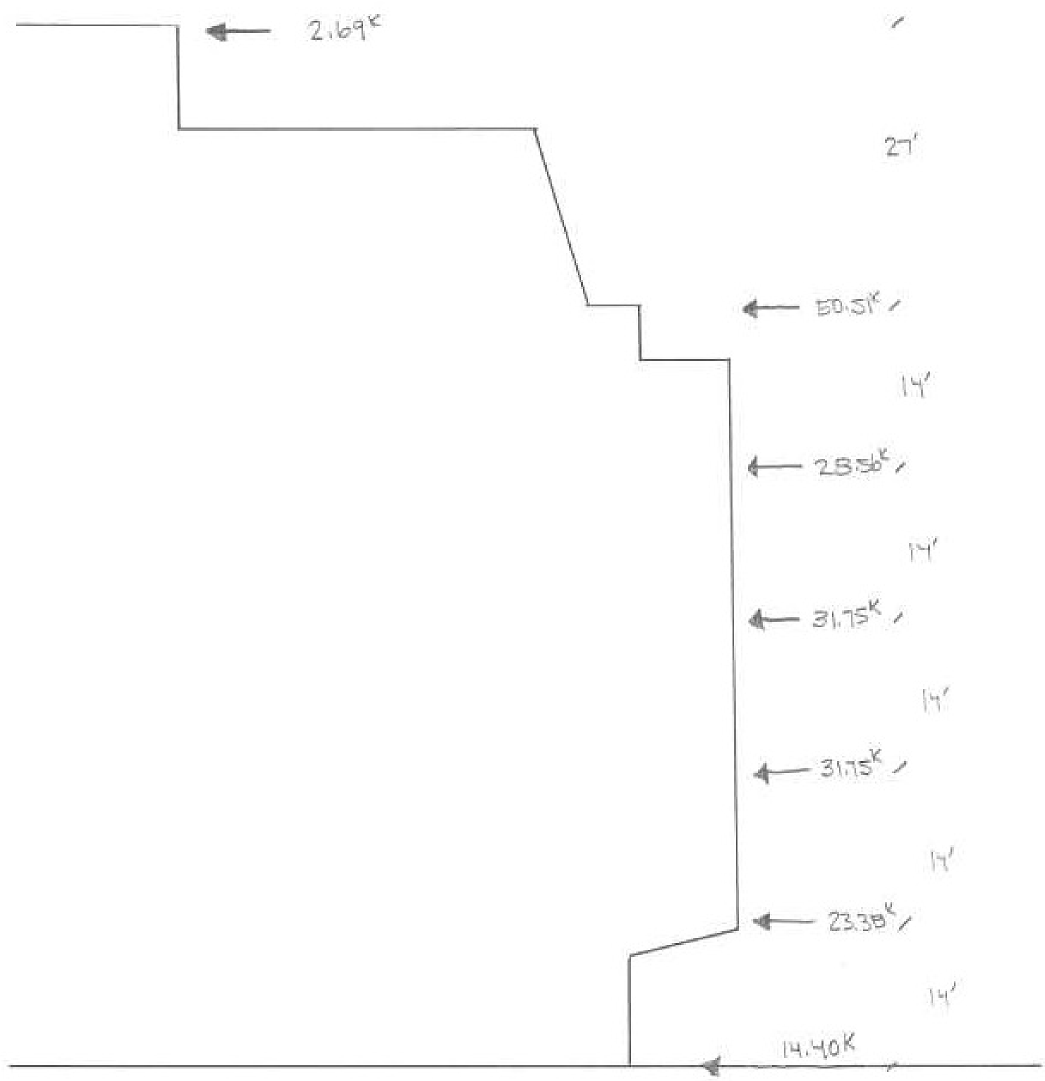
Seismic Load (Continued)|

North – South Seismic Story Shear Diagram|



Seismic Load (Continued)

East – West Seismic Story Shear Diagram



Controlling Load Combination|

The following load combinations were considered in the preparation of this report and used to determine the loads applied to the members for spot checks.

1.4D

1.2D + 1.6L + .5Lr

1.2D + 1.6Lr + (L or .8W)

1.2D + 1.6W + L + .5Lr

1.2D + 1.0E + L

.9D + 1.6W

.9D + 1.0E

Controlling Lateral Load – Wind v. Seismic

Both the wind and seismic loads were hand calculated in Technical Report I and confirmed for this report for the North – South and East – West directions. Then using the loads in each direction the base shear and overturning moment was calculated, for wind and seismic, and compared to see what case would control the design of the building lateral system. As expected for a building in central Pennsylvania with a seismic classification of "A" wind was the controlling load in both directions. This was especially reinforced when the load factor of 1.6 was applied to wind loads – as opposed to a load factor of 1.0 for seismic loads. Detailed calculations for both loads in both directions can be seen below, and story force diagrams are on the following pages.

North - South Wind Story Forces

Floor	Force (k)	Height (ft)
Roof	54.50	76
Penthouse	105.19	62.5
4th	68.48	42
3rd	65.69	28
2nd	62.26	14

V_{base} = 356.11 k
M_{over} = 16303.21 k-ft

North - South Seismic Story Forces

Floor	Force (k)	Height (ft)
Roof	2.69	76
Penthouse	50.51	62.5
4th	28.56	42
3rd	31.75	28
2nd	31.75	14

V_{base} = 145.26 k
M_{over} = 5894.34 k-ft

East - West Wind Story Forces

Floor	Force (k)	Height (ft)
Roof	22.94	76
Penthouse	48.40	62.5
4th	50.97	42
3rd	48.76	28
2nd	46.04	14

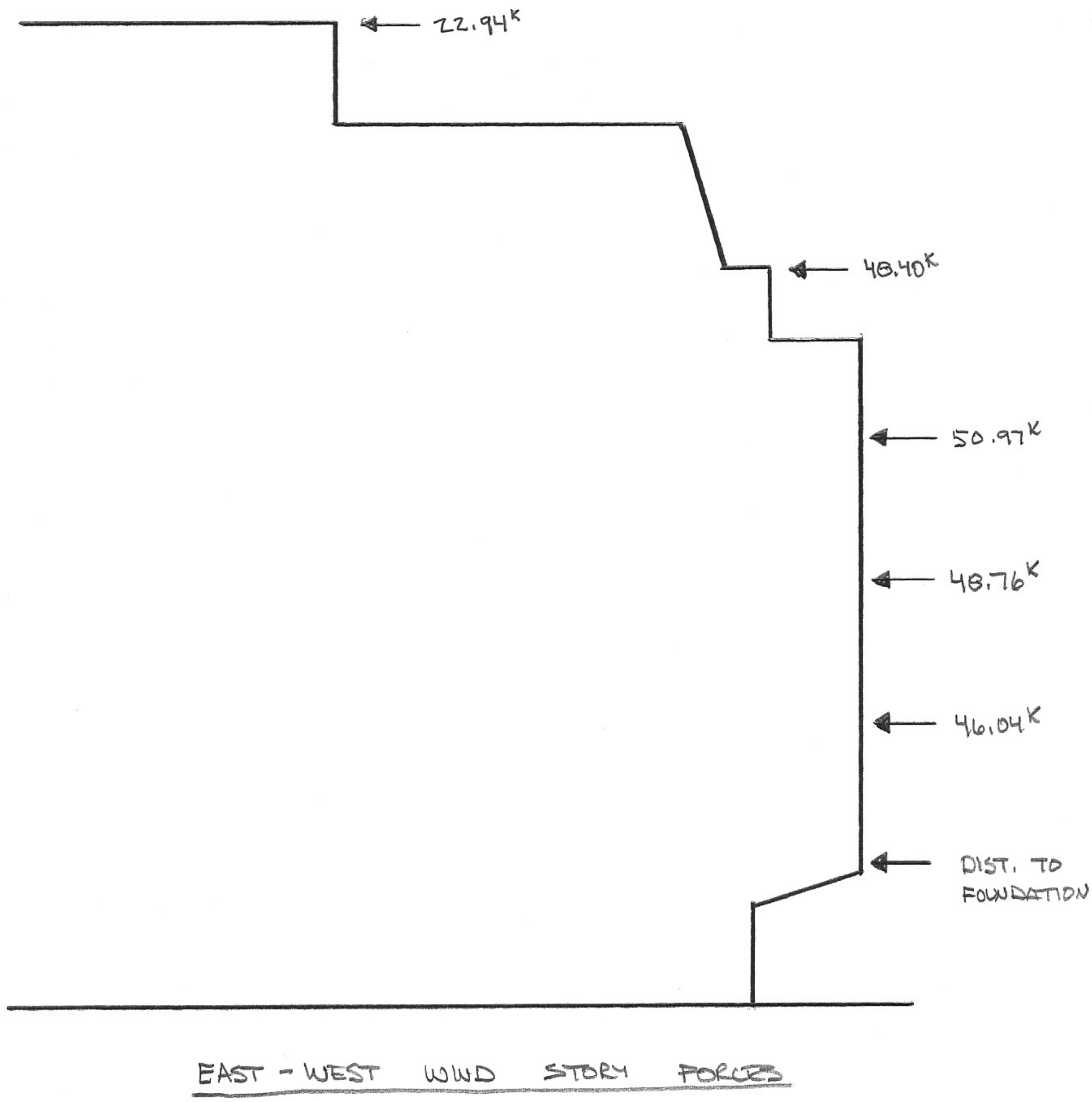
V_{base} = 217.11 k
M_{over} = 8918.87 k-ft

East - West Seismic Story Forces

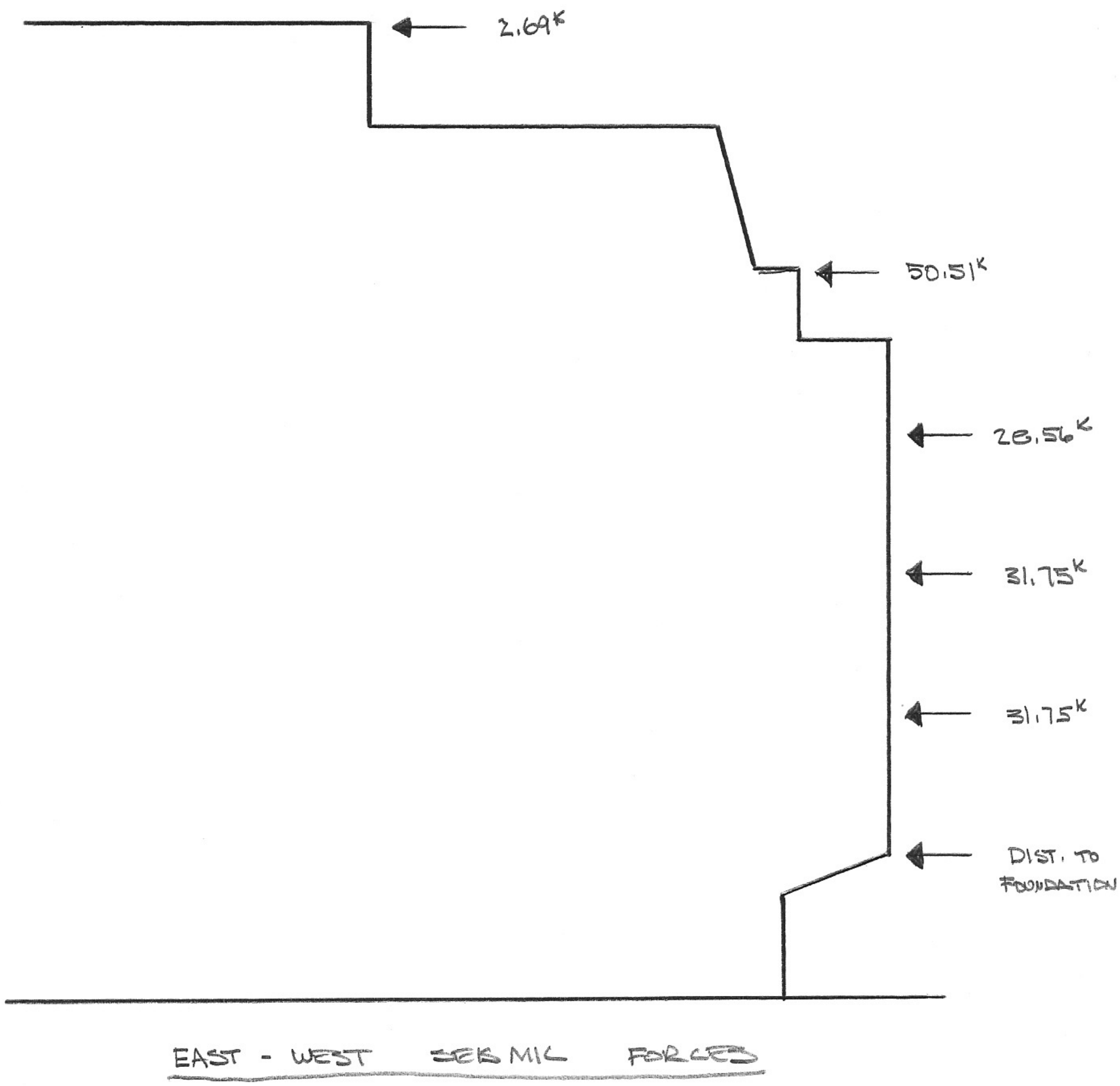
Floor	Force (k)	Height (ft)
Roof	2.69	76
Penthouse	50.51	62.5
4th	28.56	42
3rd	31.75	28
2nd	31.75	14

V_{base} = 145.26 k
M_{over} = 5894.34 k-ft

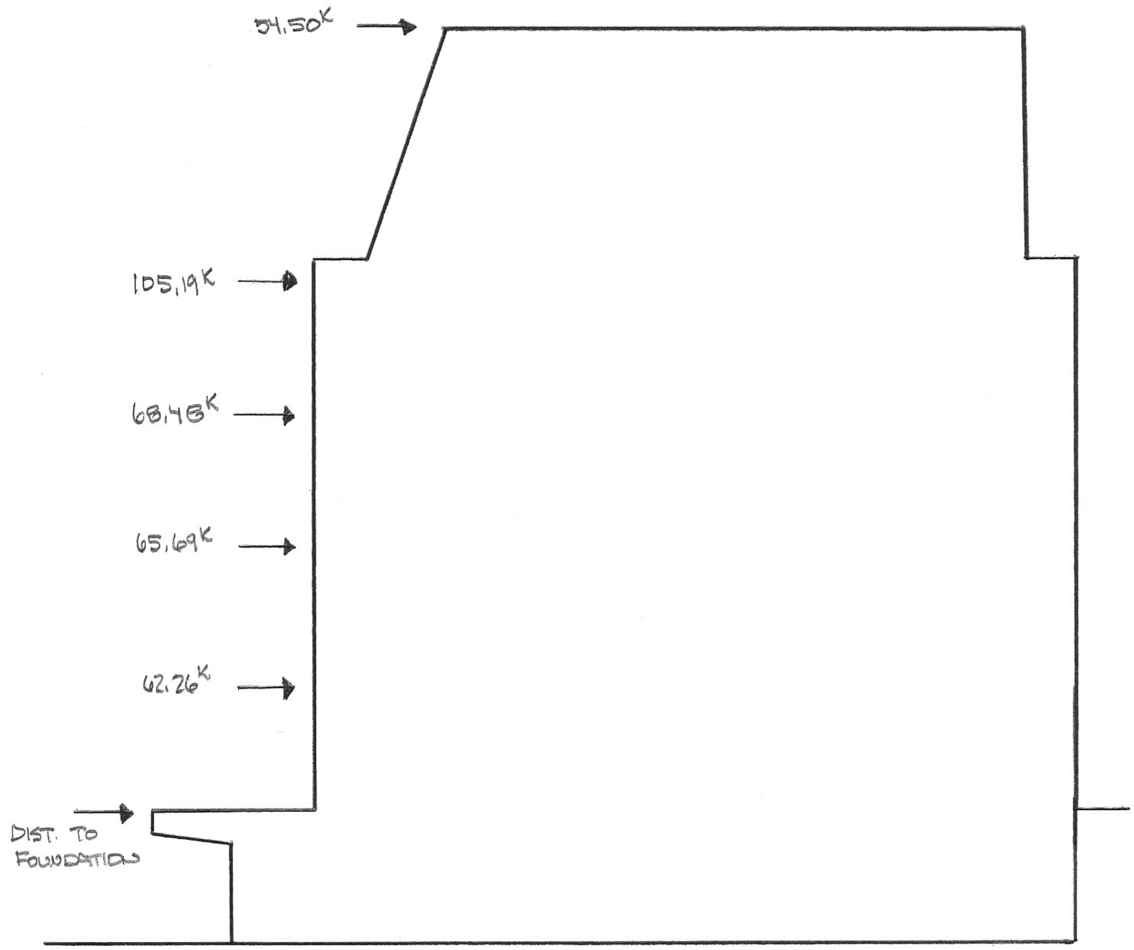
Controlling Lateral Load – Wind v. Seismic (continued)



Controlling Lateral Load – Wind v. Seismic (continued)|

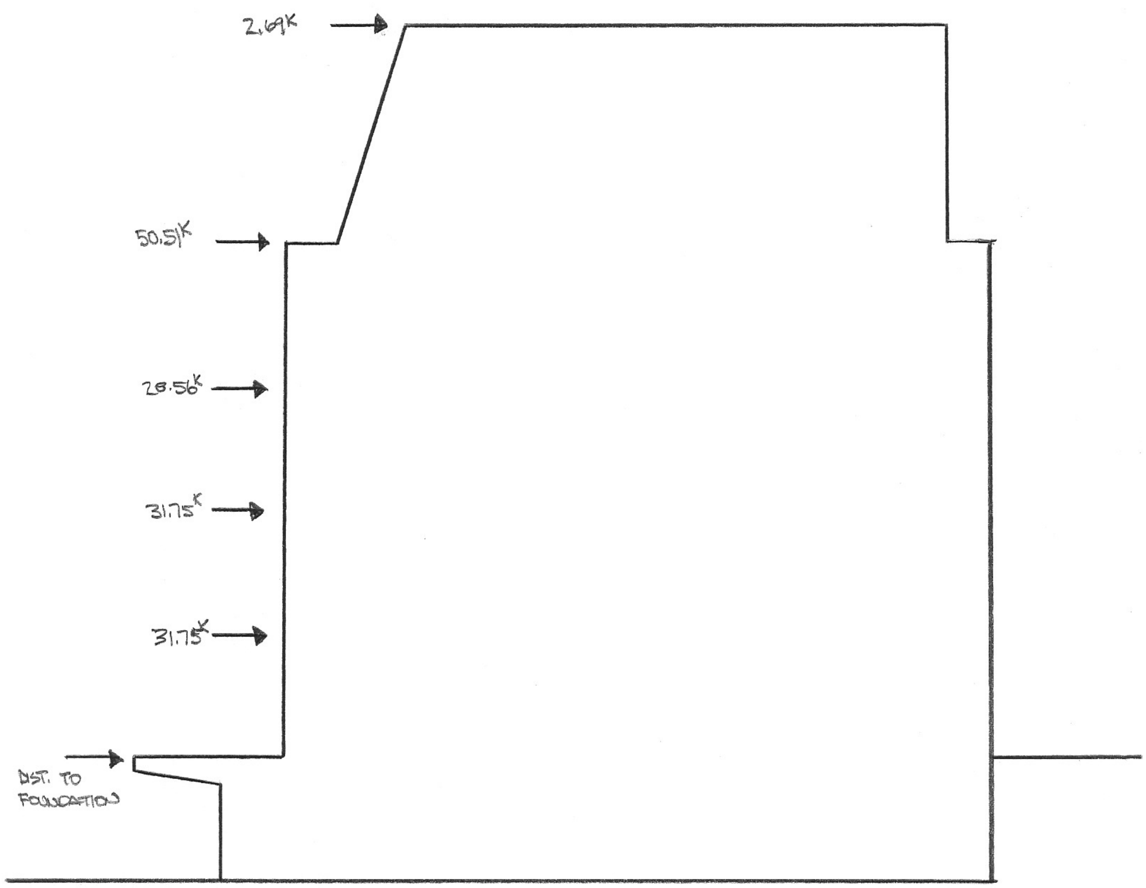


Controlling Lateral Load – Wind v. Seismic (continued)



NORTH - SOUTH WIND STORY FORCES

Controlling Lateral Load – Wind v. Seismic (continued)|



NORTH - SOUTH SEISMIC FORCES

ETABS Analysis Results|

East – West Lateral Force Resisting System|
Because the lateral force resisting system of the Life Sciences Building was very complicated and composed of many different frames; ETABS was used to determine the portion of the total lateral force that was distributed to each frame when the building was subjected to an east – west wind loading condition. A break down of the load and percentage of the total east – west base shear taken by each frame is as follows.

From Hand Analysis:	**$V_{tot,e\text{-}w}$ = 217.11**	
	Force (k)	**Percentage**
Moment Frame 1	56.21	25.9
Moment Frame 4	58.68	27.0
Hybrid Frame 5.3	9.24	4.3
Hybrid Frame 6	8.97	4.1
Hybrid Frame 7	8.84	4.1
Moment Frame 9	12.71	5.9
TOTAL E-W FRAMES	**154.65**	**71.2**
N-S Frames	**62.46**	**28.8**
Interior Columns 2.8	33.66	

As expected, Moment Frame 1 and Moment Frame 4 each take a relatively equal portion of the loading. This is consistent with the results of my earlier hand analysis of Moment Frame 4 in Technical Assignment I that showed that Moment Frame 1 and Moment Frame 4 should equally share the lateral load due to wind in the long leg of the "L".

It is also interesting to note from the ETABS analysis that the interior columns between Moment Frame 1 and Moment Frame 4 take a substantial portion of the east – west lateral load. This is probably due to the fact that they are part of the north – south lateral force resisting system frames and the frames to which the belong are tied at either end to Moment Frame 1 and Moment Frame 4. This creates a large box like structure out of the lateral framing which may need to be investigated further.

The three hybrid moment frames in the east – west lateral system also are interesting when analyzed further in ETABS. The lower moment frame portion only takes about 9k of shear at the base level for each frame. However, when the braced frame that is on top of the moment frame is analyzed all three frames have shears at the level on top of the moment frame ranging from 41k to 45k. This is probably due to the reduced stiffness of the long span moment frames as compared to the braced frames above. The rigid diaphragms at the second floor level transfer the shear from the braced frames into other lateral force resisting frames elsewhere in the building.

ETABS Analysis Results (continued)|

East – West Lateral Force Resisting System (continued)|

The deflections of each lateral frame were checked against the allowable deflection of L/400 for each floor and the overall frame. A summary of the maximum deflections of each frame is given below.

	Displacement (in)	Location (floor)	Height
Moment Frame 1	0.691250	Penthouse	62.5
Moment Frame 4	0.667200	Penthouse	62.5
Hybrid Frame 5.3	0.504056	4th	42
Hybrid Frame 6	0.498032	4th	42
Hybrid Frame 7	0.492009	4th	42
Moment Frame 9	0.482311	4th	42

The deflection of every frame in the east – west direction is well within the limits of:

Penthouse: (62.5*12) / 400 = **1.875"**

4^{th} Floor: (42*12) / 400 = **1.26"**

Watching the animation of the wind load deflections on ETABS and looking more closely at the deflections of each frame it seems that torsion will be a factor in the design of the lateral system. What makes torsion stand out as a factor in design is that the deflections at one end of the building are greater than they are on the other end – resulting in a net twisting of the structure. It is also important to note that due to the irregular shape and height of the building the center of mass, center of rigidity, and where the wind load resultant force is applied are all at different locations. This will amplify the effects of torsion. Due to the complicated nature of the lateral force resisting system torsion issued will be studied more in depth in future reports.

A quick check verified that the dead weight of the building, having two levels below grade, and its inherently stable "L" shape with a low height to width ratio all combine to negate any overturning moments being transferred to the foundation. The foundations will have to be designed to handle the combination of gravity loads acting downward in conjunction with wind loads also acting downward.

ETABS Analysis Results (continued)|

North – South Lateral Force Resisting System|

Once again the complicated nature of the lateral force resisting system and the sheer number of frames involved necessitated the use of ETABS to determine the portion of the total lateral force that was distributed to each frame. This time the building was subjected to a north – south wind loading condition. A break down of the load and percentage of the total north – south base shear taken by each frame is as follows.

From Hand Analysis:	**$V_{tot,n\text{-}s}$ = 356.11**	
	Force (k)	**Percentage**
Braced Frame C	81.76	23.0
Hybrid Frame C.2	0	0.0
Hybrid Frame D.8	0	0.0
Braced Frame E	17.91	5.0
Braced Frame G	113.14	31.8
Braced Frame J	88.85	25.0
Braced Frame K	50.48	14.2
TOTAL E-W FRAMES	**352.14**	**98.9**
E-W Frames	**3.97**	**1.1**

A limitation of my model becomes evident when looking at the results for the two hybrid frames. I was unable to model the north – south bracing in this direction which resulted in each frame taking no north – south wind base shear forces. I will work to correct this in later analyses.

Another result that stands out is the comparatively low percentage of the base shear taken by the opposite direction's lateral force resisting system for north – south wind loads. In the previous analysis for east – west wind loads the north – south system resisted almost one third of the east – west lateral forces. However, when the north – south wind loads are applied, the east – west lateral system only resists about 1% of the forces that were applied in the orthogonal direction.

The analysis also shows that generally, concentrically braced frames are more effective than eccentrically braced frames at resisting lateral loading. Additionally, the deflections in the north – south direction were considerably less than the deflections in the east – west direction – even without modeling two braces in the north – south hybrid frames. This shows that moment resisting frames tend to deflect the most, followed by eccentrically braced frames. Concentrically braced frames seem to be the optimal choice when stiffness and deflections are an issue and will be the preferred alternative in redesign.

ETABS Analysis Results (continued)|

North – South Lateral Force Resisting System (continued)|

The deflections of each lateral frame were checked against the allowable deflection of L/400 for each floor and the overall frame. A summary of the maximum deflections of each frame is given below.

	Displacement (in)	Location (floor)	Height
Braced Frame C	0.506266	Penthouse	62.5
Hybrid Frame C.2	0.401493	4th	42
Hybrid Frame D.8	0.384844	4th	42
Braced Frame E	0.494063	Penthouse	62.5
Braced Frame G	0.481860	Penthouse	62.5
Braced Frame J	0.469657	Penthouse	62.5
Braced Frame K	0.463556	Penthouse	62.5

The deflection of every frame in the north - south direction is well within the limits of:

Penthouse: (62.5*12) / 400 = **1.875”**

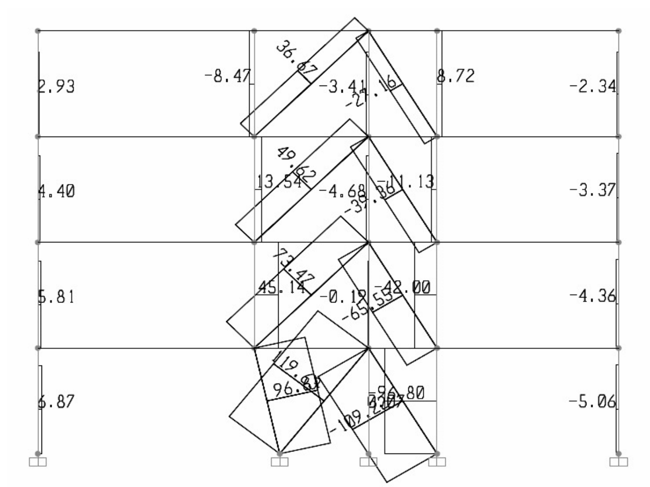
4^{th} Floor: (42*12) / 400 = **1.26”**

Watching the animation of the wind load deflections on ETABS and looking more closely at the deflections of each frame it seems that torsion will be a factor in the design of the lateral system in both the north – south and east – west directions. What makes torsion stand out as a factor in design is that the deflections at one end of the building are greater than they are on the other end – resulting in a net twisting of the structure for both wind force directions. It is also important to note that due to the irregular shape and height of the building the center of mass, center of rigidity, and where the wind load resultant force is applied are all at different locations. This will amplify the effects of torsion. Due to the complicated nature of the lateral force resisting system torsion issued will be studied more in depth in future reports.

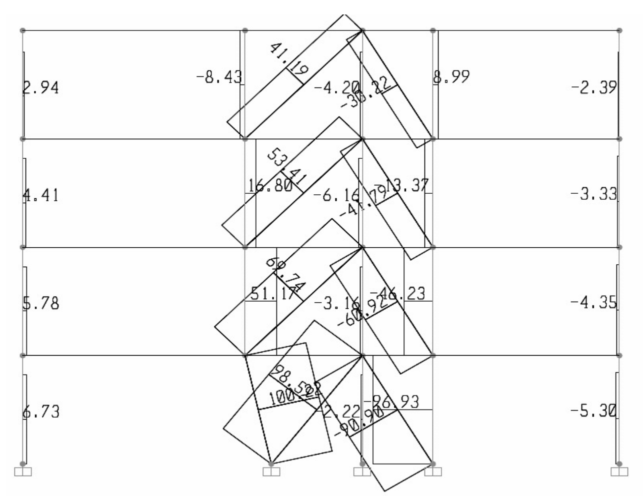
A quick check verified that the dead weight of the building, having two levels below grade, and its inherently stable “L” shape with a low height to width ratio all combine to negate any overturning moments being transferred to the foundation. The foundations will have to be designed to handle the combination of gravity loads acting downward in conjunction with wind loads also acting downward. However, the effects of lateral loads on overturning and the design of foundations add no special considerations to the design of the structure.

Member Spot Checks|

Member spot checks were performed on the lateral bracing in Braced Frame G and Braced Frame J. Checks were only performed on the pinned, diagonal bracing members because the ETABS model was limited to only lateral force systems. Therefore, dead loads and live loads due to the gravity framing system were not applied to the lateral force resisting frames in this model. By choosing to analyze the braces only, the fact that the gravity framing system is not in the model can be ignored. This is true because the braces will only ever resist the lateral forces and will show the same forces that they would if the gravity framing system and gravity loads were modeled. However, a load factor of 1.6 was applied to the loads on the diagonal braces to reflect the critical load combination that would be used in the final design. It is also important to note that the square and rectangular HSS were assumed to be ASTM A500, Gr. B steel with a F_y = 46ksi and a F_u = 58ksi. Elevations of the two frames whose diagonal braces are being analyzed are shown below with the axial forces illustrated.



Braced Frame G



Braced Frame J

Member Spot Checks (continued)|

All of the diagonal bracing members were found to be sufficiently sized. They were quite oversized when strength was the evaluation criteria. This is probably due to several factors. The first being that my model in ETABS didn't perfectly model the actual structure. The second is that the Life Sciences Building is a laboratory building and vibration and sway are key issues. The members were probably sized so that the building met strict sway and vibration limits.

Conclusion|

This report was a combination of hand calculations and computer modeling that were used to determine the lateral loads and lateral load distribution through the lateral system of the Life Sciences Building. The wind and seismic loads applied to the building were calculated by hand and applied to an ETABS model. The building was analyzed for strength and serviceability and compared to code limits for both. The result was that the lateral system was sufficiently designed to meet both the strength and serviceability requirements. Specifically, the drift of every point of the building meets the specified requirements and a spot check of the most critical members for strength revealed that they were sufficiently sized to handle the applied lateral loading. Simple hand calculations were utilized to ensure that the forces shown to be resisted by each lateral frame in ETABS was a realistic representation of the real building.

I have concluded that the original design of the lateral system is more than sufficient to support the building. Completion of Technical Report III has enabled me to understand the lateral system well enough to begin to make informed decisions with regard to the lateral system in future reports. However, my work studying the lateral system is not totally complete. Additional research and analysis of torsion needs to be undertaken and my ETABS model should be refined further as my work with the Life Sciences Building progresses throughout the year.

Appendix A
ASCE 7-05 Wind Load Calculations

Appendix A – Wind Load Calculations|

EAST - WEST WIND LOADS:

WINDWARD WALL :

$\boxed{C_p = .8}$

LEEWARD WALL :

$L/B = 250'/184' = 1.3587$

↑ LEEWARD

1	1.36	2
-.5	-.428	-.3

$\boxed{C_p = -.428}$

ROOF PRESSURE :

→ $\theta = 74°$ WHICH IS CLOSE TO $\theta = 80°$ FOR VERTICAL WALLS OR ROOFS $\theta > 80°$

→ SO I AM CONSERVATIVELY ASSUMING THAT C_p ON SLOPED ROOF / WALLS IS EQUAL TO C_p ON VERTICAL WALLS THUS $\boxed{C_p = .8}$
ALSO FOR ROOFS $\theta > 80° \rightarrow C_p = .8$

→ ASSUMPTIONS RESULT IN INCREASING THE HEIGHT OF BUILDING & PUTTING FLAT ROOF ON IT.

FLAT ROOF (NORMAL & PARALLEL):

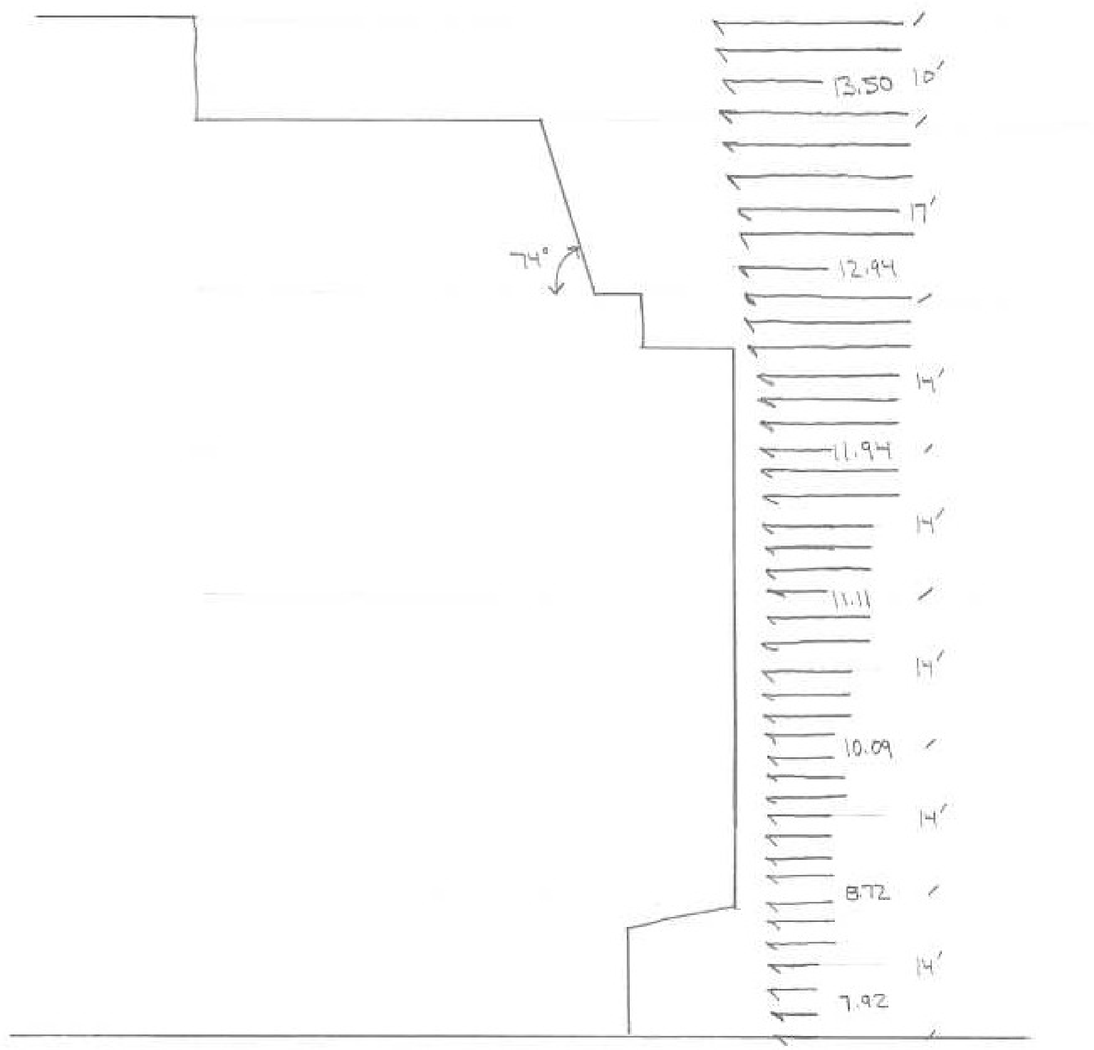
$h = 97'$ ↕ $L = 250'$ $h/L = 97/250 = .388$

LONG LEG OF "L"

0 → 48.5'	$C_p = -.9$
48.5' → 97'	$C_p = -.9$
97' → 194'	$C_p = -.5$
194' → 250'	$C_p = -.3$

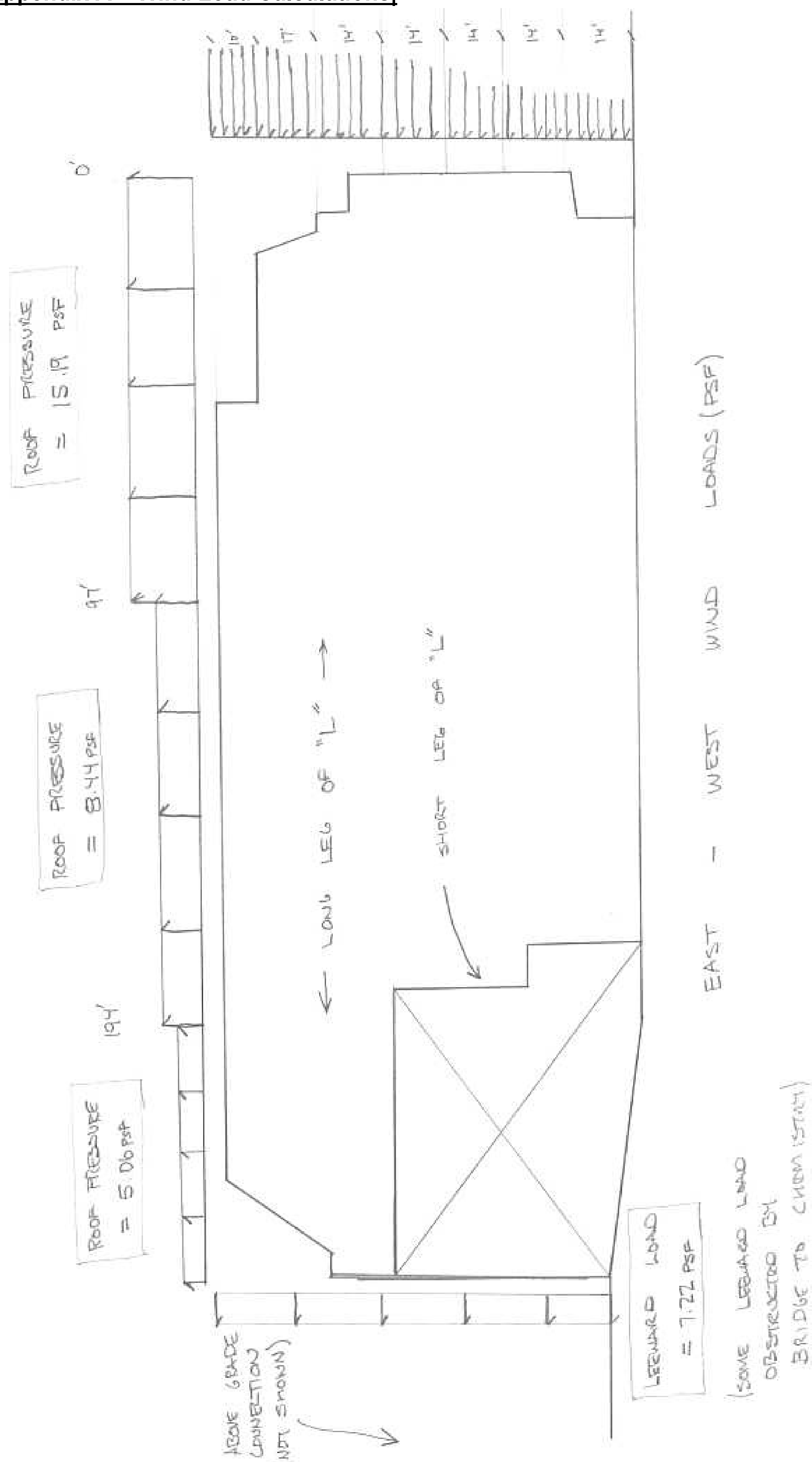
FOR WIND RUNNING ALONG LONG LEG OF "L"

Appendix A – Wind Load Calculations|



EAST - WEST WIND LOADS (PSF)

Appendix A – Wind Load Calculations|



Appendix A – Wind Load Calculations|

NORTH - SOUTH WIND LOADS:

WINDWARD WALL:

$$\boxed{C_p = .8}$$

LEEWARD WALL:

$$L/B = 72/184 = .391304$$

$$\boxed{C_p = -.5}$$

ROOF PRESSURE:

AGAIN ROUNDING $\theta = 74°$ TO $80°$
AND CONSIDERING SLOPED ROOF/
WALL TO BE PART OF VERTICAL
EXTERIOR WALLS W/ SLOPE: $\boxed{C_p = .8}$

FLAT ROOF:

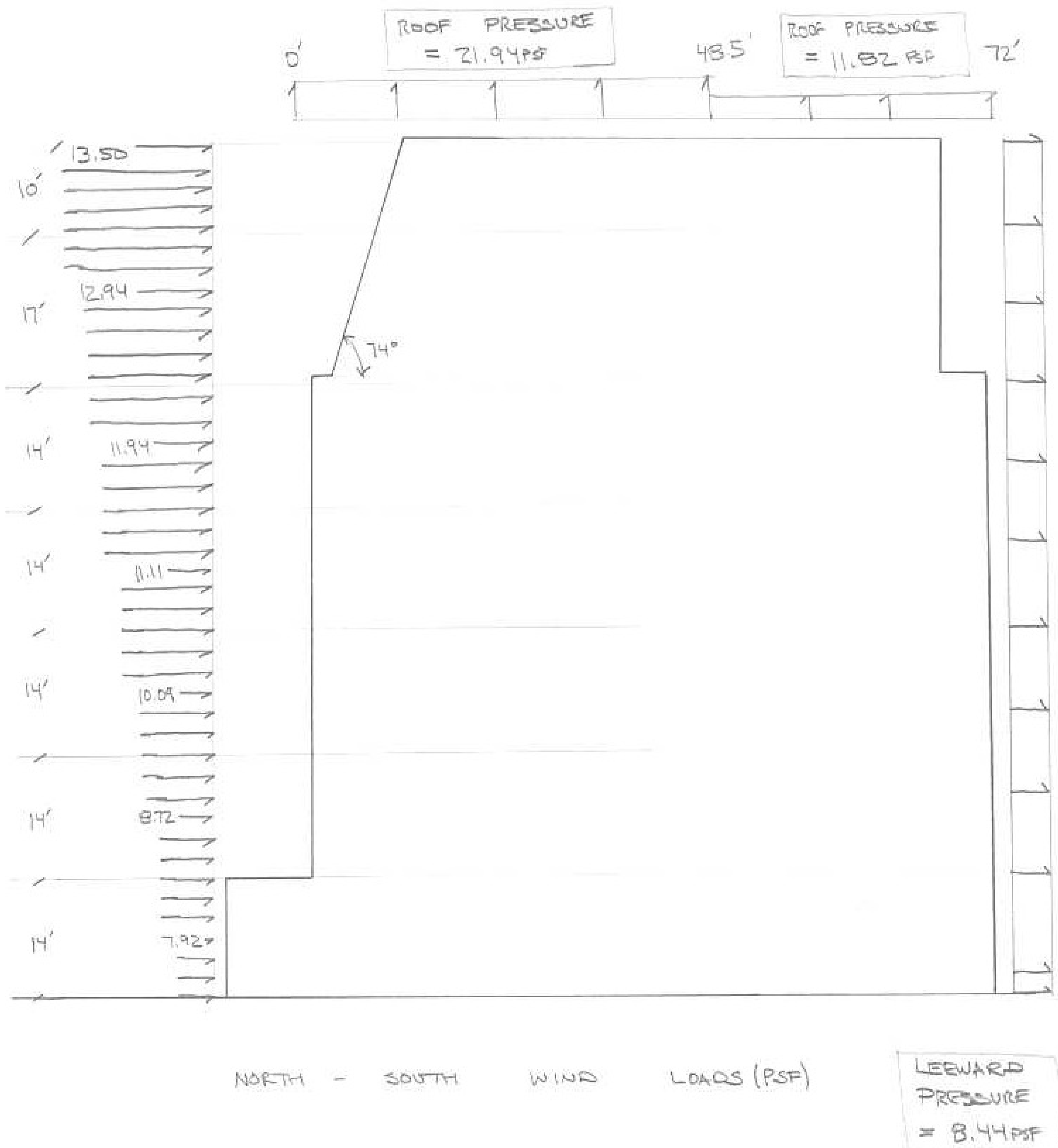
$h = 97'$ $L = 72'$

$$h/L = 97/72 = 1.35 > 1.0$$

$0' \rightarrow 48.5' =$ $C_p = -1.3$

$48.5' \rightarrow 72' =$ $C_p = -.7$

Appendix A – Wind Load Calculations|



Appendix B
ASCE 7-05 Seismic Load Calculations

Appendix B – Seismic Load Calculations|

TYPICAL FLOOR:

FLOOR SLAB:

2" DECK: $\frac{2}{12}\left(\frac{1}{2}\right)(150) = 12.5$ LBS/FT²

4.5" SLAB: $\frac{4.5}{12}(150) = 56.25$ LBS/FT²

$\boxed{\text{SLAB} = 69 \text{ PSF}}$

SUPERIMPOSED (STEEL FRAMING):

- ASSUMING W16X31 SPACED EVERY 8'
- ASSUMING 130 LB/FT GIRDER ON 30' CENTERS

BEAM: $31/8 = 3.875$ LBS/FT²

GIRDER: $130/30 = 4.333$ LBS/FT²

DECKING: $\frac{.0474''(490)}{12} = 1.9355$ LBS/FT²

$\boxed{\text{FRAMING} = 10 \text{ PSF}}$

PARTITIONS:

$\boxed{\text{PARTITIONS} = 20 \text{ PSF}}$

EXTERIOR WALL:

- ASSUMING 60 LBS/FT² ALLOWANCE

$\boxed{\text{EXT WALL} = 60 \text{ PSF}}$

COLLATERAL D.L.:

$\boxed{\text{COLLATERAL} = 10 \text{ PSF}}$

Appendix B – Seismic Load Calculations

PENTHOUSE LEVEL:

FLOOR SLAB: (TYPICAL)

$\boxed{\text{SLAB} = 69 \text{ PSF}}$

STEEL FRAMING:

- ASSUMING W18 x 50 SPACED @ 8'
- ASSUMING 150 LB/FT GIRDER ON 30' CENTERS

BEAMS: $50/8 = 6.25$ PSF

GIRDERS: $150/30 = 5$ PSF

DECK: $= 2$ PSF

$\boxed{\text{FRAMING} = 13 \text{ PSF}}$

NO PARTITIONS:

MECHANICAL EQUIPMENT:

$LL_{MECH} = 200$ PSF

$\boxed{\text{EQUIP.} = 200 \text{ PSF}}$

EXTERIOR WALL / ROOF

ROOF ASSEMBLY (TYP). + 30 LBS (MECH SPACE CEILING)

ROOFING = 6 PSF

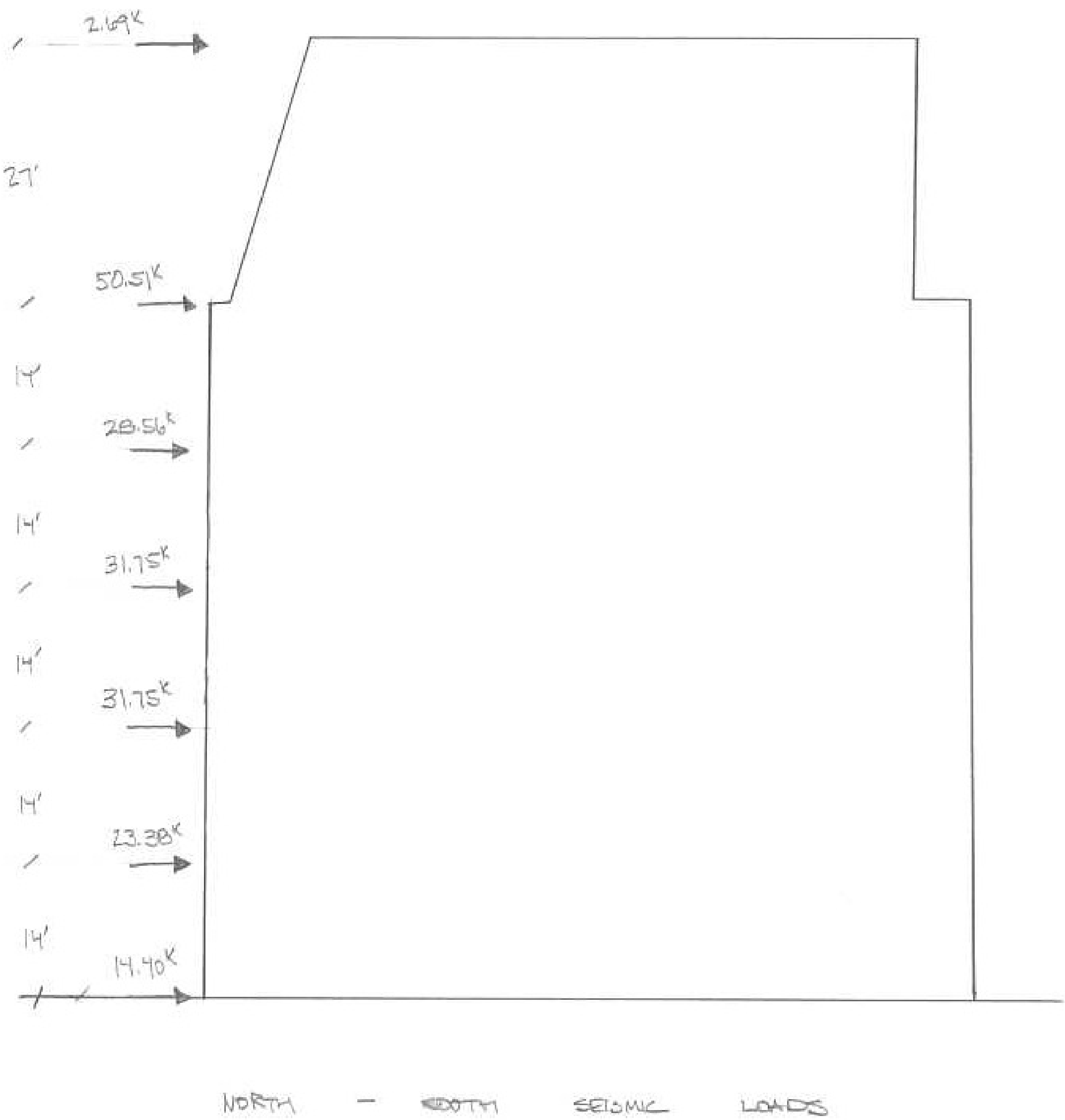
DECK = 2 PSF

FRAMING = W16x31 @ 8' o.c.

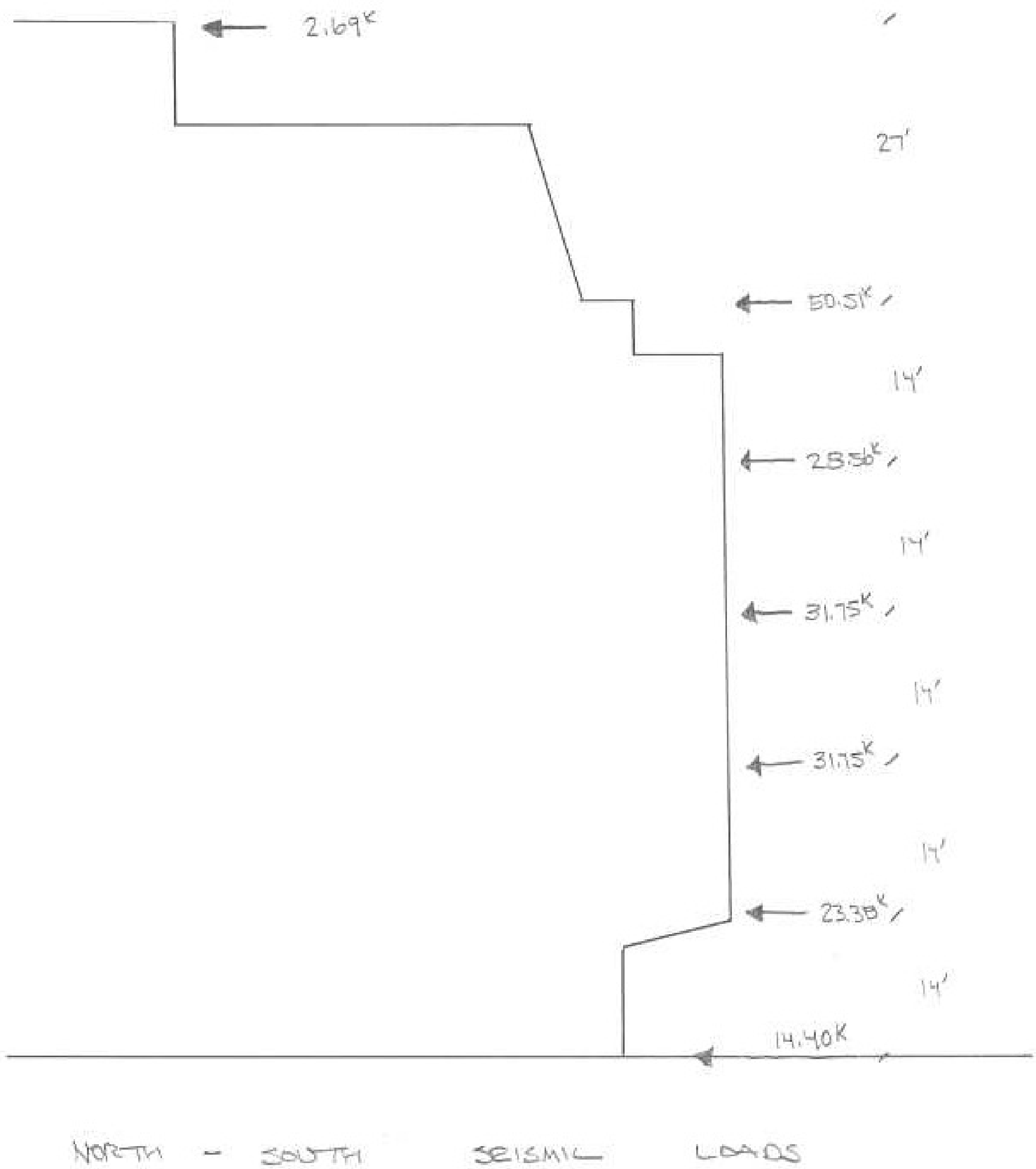
$31/8 = 3.875$

$\boxed{\text{ROOF} = 12 \text{ PSF}}$

Appendix B – Seismic Load Calculations



Appendix B – Seismic Load Calculations



Appendix C
Wind v. Seismic – Controlling Load

Appendix C – Wind v. Seismic – Controlling Load

North - South Wind Story Forces

Floor	Force (k)	Height (ft)
Roof	54.50	76
Penthouse	105.19	62.5
4th	68.48	42
3rd	65.69	28
2nd	62.26	14

V_{base} = 356.11 k
M_{over} = 16303.21 k-ft

North - South Seismic Story Forces

Floor	Force (k)	Height (ft)
Roof	2.69	76
Penthouse	50.51	62.5
4th	28.56	42
3rd	31.75	28
2nd	31.75	14

V_{base} = 145.26 k
M_{over} = 5894.34 k-ft

East - West Wind Story Forces

Floor	Force (k)	Height (ft)
Roof	22.94	76
Penthouse	48.40	62.5
4th	50.97	42
3rd	48.76	28
2nd	46.04	14

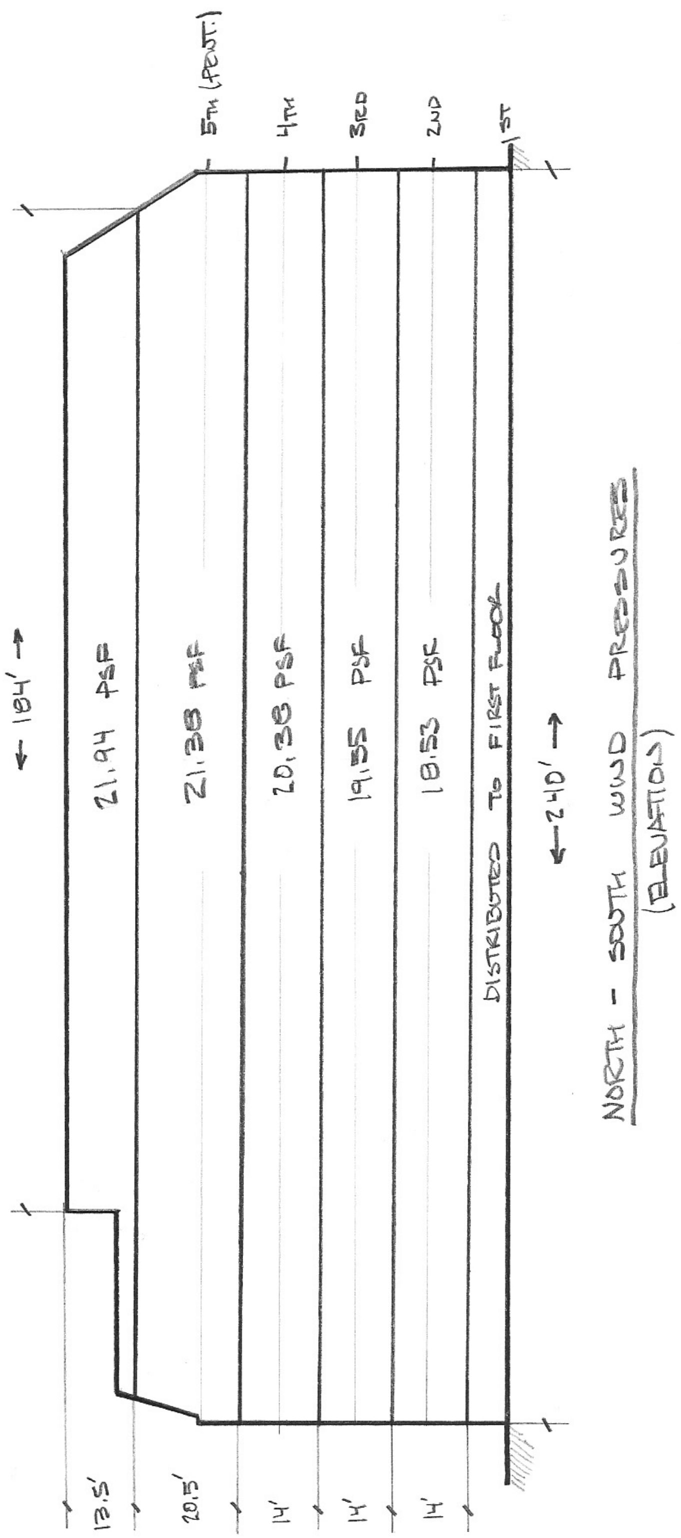
V_{base} = 217.11 k
M_{over} = 8918.87 k-ft

East - West Seismic Story Forces

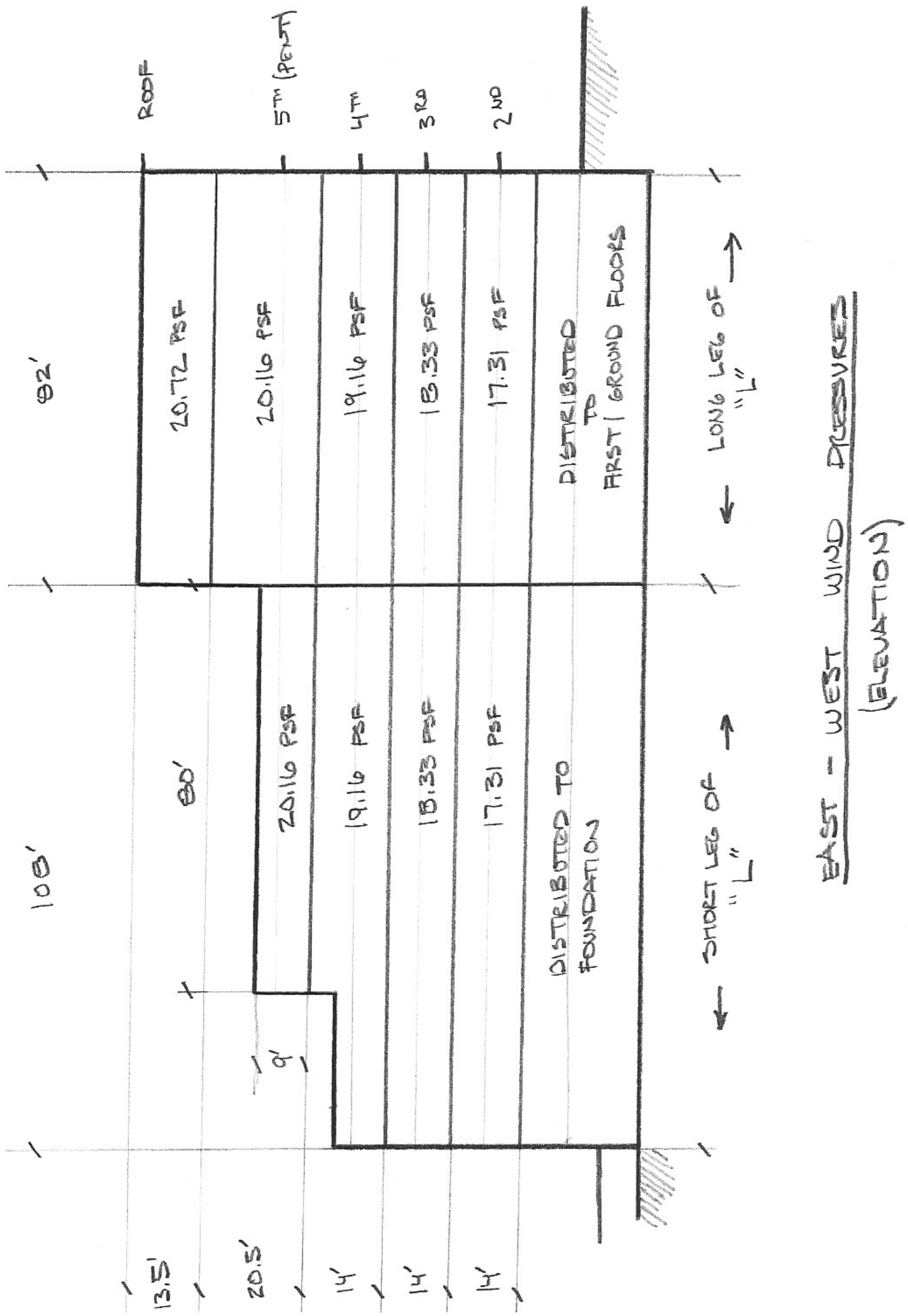
Floor	Force (k)	Height (ft)
Roof	2.69	76
Penthouse	50.51	62.5
4th	28.56	42
3rd	31.75	28
2nd	31.75	14

V_{base} = 145.26 k
M_{over} = 5894.34 k-ft

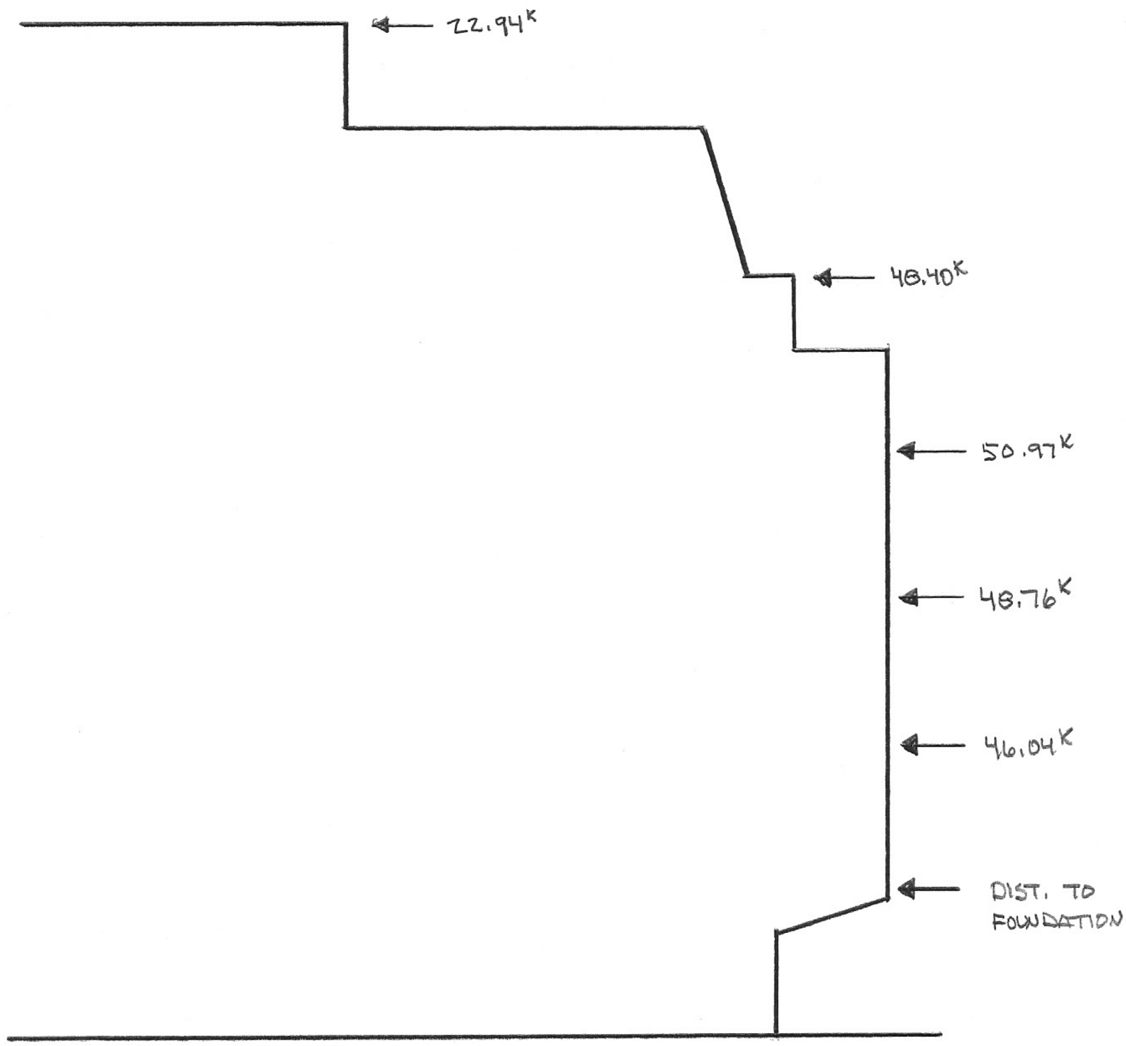
Appendix C – Wind v. Seismic – Controlling Load



Appendix C – Wind v. Seismic – Controlling Load

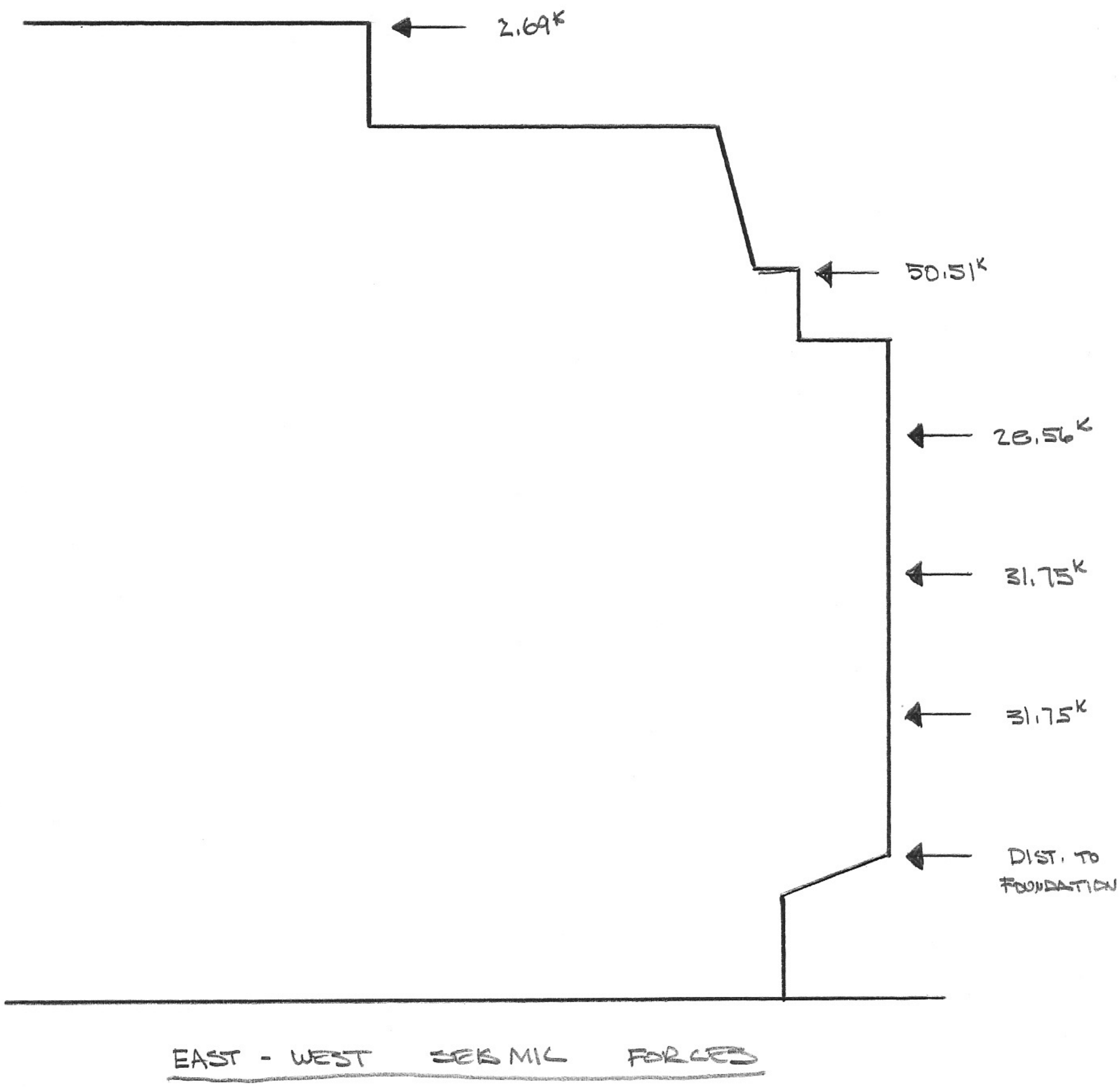


Appendix C – Wind v. Seismic – Controlling Load

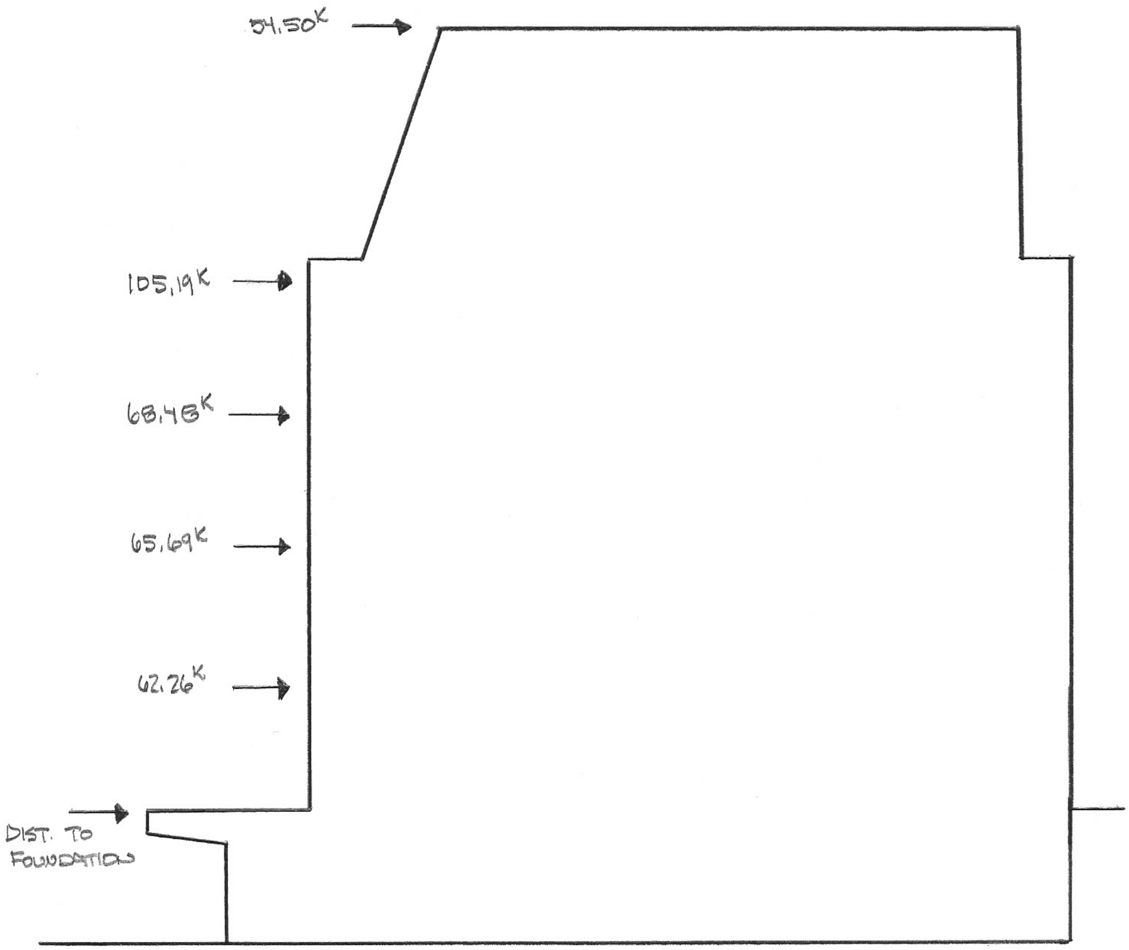


EAST - WEST WIND STORY FORCES

Appendix C – Wind v. Seismic – Controlling Load

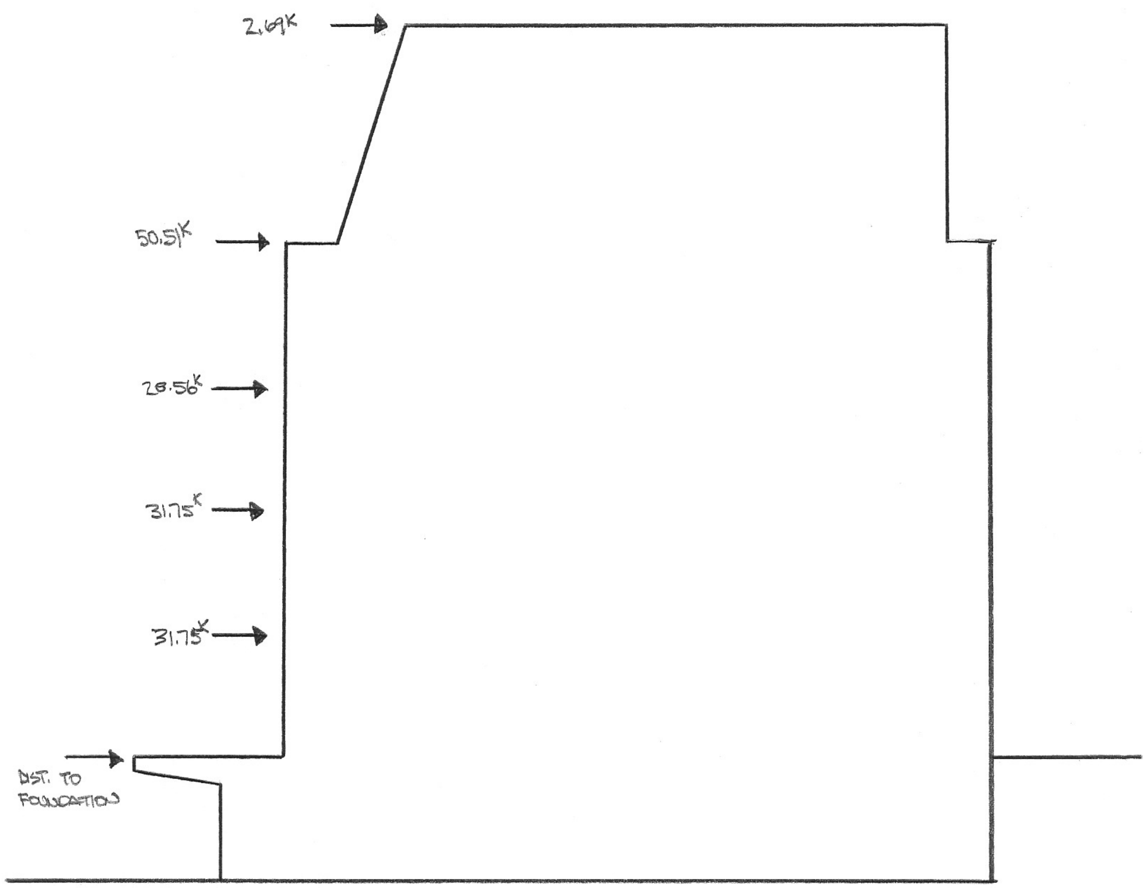


Appendix C – Wind v. Seismic – Controlling Load



NORTH - SOUTH WIND STORY FORCES

Appendix C – Wind v. Seismic – Controlling Load



NORTH - SOUTH SEISMIC FORCES

Appendix D
Member Spot Checks

Appendix D – Member Spot Checks

FRAME G – BRACING MEMBER CHECK

(1.6 LOAD FACTOR APPLIED TO WIND LOADS)

TOP LEVEL – 4TH FLOOR

HSS 10 x 6 x 1/4
$\ell = 19.92'$
$P_{ALL} = 192^K$
$P_{ACT} = 58.67^K$ TENS

HSS 10 x 6 x 1/4
$\ell = 16.37'$
$P_{ALL} = 187^K$
$P_{ACT} = 43.46^K$ COMP

3RD FLOOR

HSS 10 x 6 x 1/4
$\ell = 19.92'$
$P_{ALL} = 192^K$
$P_{ACT} = 79.39^K$ TENS

HSS 10 x 6 x 1/4
$\ell = 16.37'$
$P_{ALL} = 187^K$
$P_{ACT} = 62.98^K$ COMP

2ND FLOOR

HSS 10 x 6 x 3/8
$\ell = 19.92'$
$P_{ALL} = 224^K$
$P_{ACT} = 117.55^K$ TENS

HSS 10 x 6 x 3/8
$\ell = 16.37'$
$P_{ALL} = 268^K$
$P_{ACT} = 104.88^K$ COMP

1ST FLOOR

HSS 10 x 6 x 1/2
$\ell = 17.80'$
$P_{ALL} = 322^K$
$P_{ACT} = 154.8^K$

HSS 10 x 6 x 1/2
$\ell = 16.37'$
$P_{ALL} = 341^K$
$P_{ACT} = 174.75^K$ COMP.

Appendix D – Member Spot Checks

FRAME J – BRACING MEMBER CHECK
(1.6 LOAD FACTOR FOR WIND)

4TH FLOOR

HSS 10 x 6 x 1/4
$\ell = 19.92'$
$P_{ALL} = 192^k$
$P_{ACT} = 65.904^k$ T

HSS 10 x 6 x 1/4
$\ell = 16.37'$
$P_{ALL} = 187^k$
$P_{ACT} = 48.352^k$ C

3RD FLOOR

HSS 10 x 6 x 1/4
$\ell = 19.92'$
$P_{ALL} = 192^k$
$P_{ACT} = 85.456^k$ T

HSS 10 x 6 x 1/4
$\ell = 16.37'$
$P_{ALL} = 187^k$
$P_{ACT} = 66.864^k$ C

2ND FLOOR

HSS 10 x 6 x 3/8
$\ell = 19.92'$
$P_{ALL} = 224^k$
$P_{ACT} = 111.584^k$ T

HSS 10 x 6 x 3/8
$\ell = 16.37'$
$P_{ALL} = 268^k$
$P_{ACT} = 97.42^k$ C

1ST FLOOR

HSS 10 x 6 x 1/2
$\ell = 17.80'$
$P_{ALL} = 322^k$
$P_{ACT} = 157.747^k$ T

HSS 10 x 6 x 1/2
$\ell = 16.37'$
$P_{ALL} = 341^k$
$P_{ACT} = 145.44^k$ C