Technical Report I
Structural Concepts / Structural Existing Conditions Report

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

Executive Summary|

This report contains information regarding the existing structural systems and loading 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 6 floors with a mechanical penthouse and has concrete floors with a steel frame using composite floor deck, composite beams and composite girders.

A description of the layout of the building and associated diagrams is the first information given in the report. Next follows a description of the building's structural systems. Typical floor framing systems, lateral load resisting systems, foundation systems, roof systems and columns are all described in detail. The materials used and their strengths precedes a list of the relevant codes and standards used in the design of the building. Following the codes introduction they are applied to find the loading that acts on the building. Loads calculated include live, dead, snow, wind, and seismic. Other loads are recognized to act on the building but are not considered in this assignment. Finally after determining the loads acting on the building by using the building codes spot checks are performed on several systems and members in the building. The wind and seismic loading calculated in the loading portion of the report are used to perform a full seismic and wind analysis of the lateral force resisting system. Spot checks of the lateral system include analyzing the loading of a moment frame and checking the size of its columns and beams. Spot checks of the gravity system include determining the load from the floor system through the beams, into the girders and to the columns and checking each member. An appendix follows the actual report and includes all of the calculations and procedures used to reach my conclusions.

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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.

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 at most 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. The piles used are HP10x57 and HP12x74 sections with allowable working loads of 100 k and 130 k respectively. Piles are driven in groups 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.

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 girders are most typically composite steel W33x141 and W33x201.

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.

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 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 most column loads in the range of 200 k to 800 k.

Material Strength|

Reinforced Concrete|

Compressive Strength
f'_c = 4000 psi (except where noted otherwise)
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 I|

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. The partition load allowance will be added to classroom, lab and office areas but was taken as part of the live load for this analysis. 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 I) 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 will be calculated using the analytical procedure as prescribed in Section 6.1.2 of ASCE 7 – 05. Wind loads in this technical assignment are calculated using the long leg of the "L". The long leg of the "L" was chosen because it was the part of the building that met the thesis height requirements. It was also chosen because its wind loading is the worst case and because the moment frames that were analyzed are located in it.

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) $\alpha = 7.0$ (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)

East - West (Long Leg of "L")

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

length range	h	qz	G	Cp	Roof Pressure (PSF)
0' - 48.5'	-	19.857	0.85	-0.9	-15.19
48.5' - 97'	-	19.857	0.85	-0.9	-15.19
97' - 194'	-	19.857	0.85	-0.5	-8.44
194' - 250'	-	19.857	0.85	-0.3	-5.06

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

North - South (Long Leg of "L")

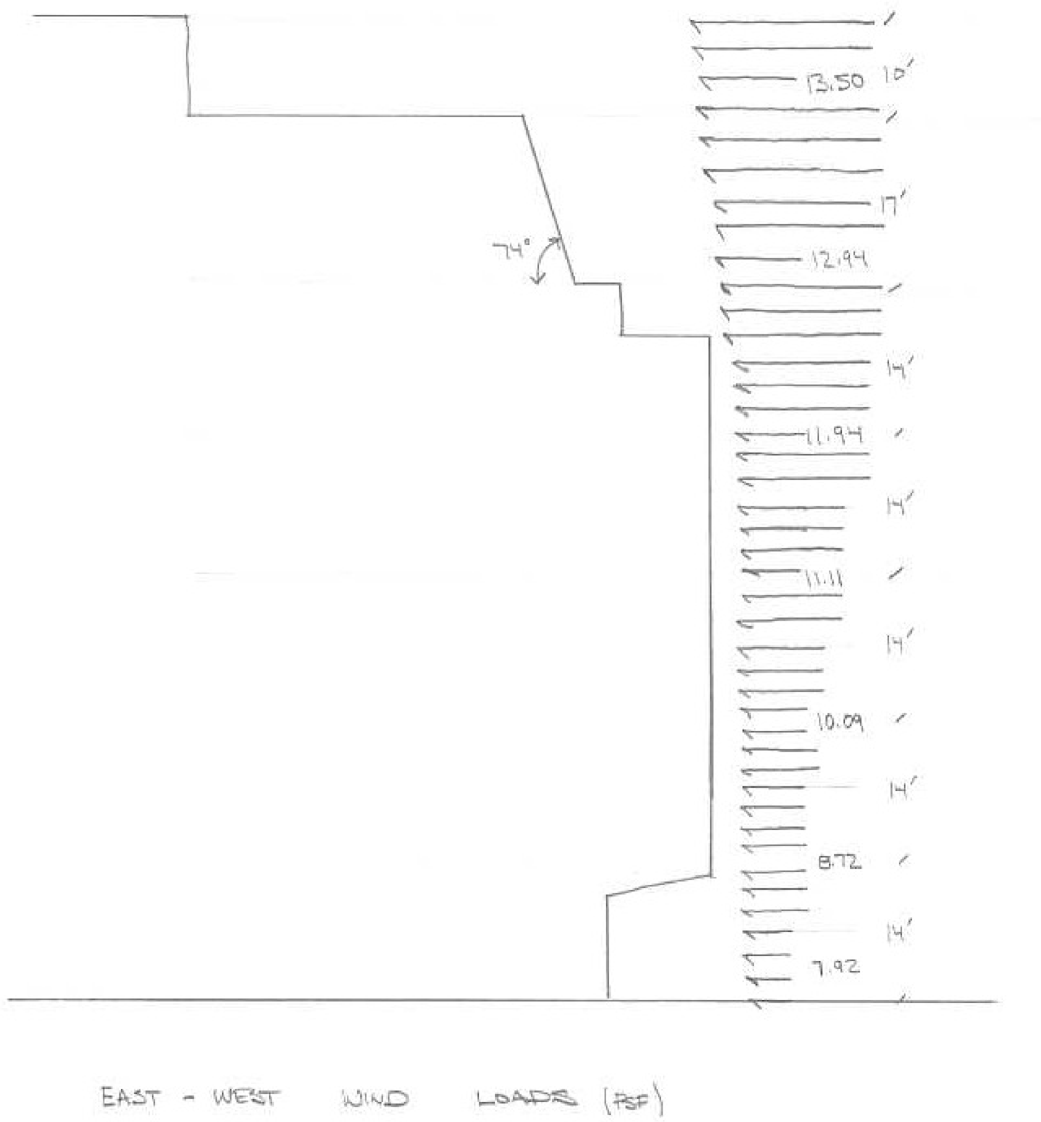
height range	h (max)	qz	G	Cp	Windward Wall Pressure (PSF)
83.5 '- 97'	97	19.857	0.85	0.80	13.50
63' - 83.5'	83.5	19.025	0.85	0.80	12.94
49' - 63'	63	17.554	0.85	0.80	11.94
35' - 49'	49	16.337	0.85	0.80	11.11
21' - 35'	35	14.840	0.85	0.80	10.09
7' - 21'	21	12.825	0.85	0.80	8.72
0' - 7'	7	11.649	0.85	0.80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

length range	h	qz	G	Cp	Roof Pressure (PSF)
0' - 48.5'	-	19.857	0.85	-1.3	-21.94
48.5' - 72'	-	19.857	0.85	-0.7	-11.82

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

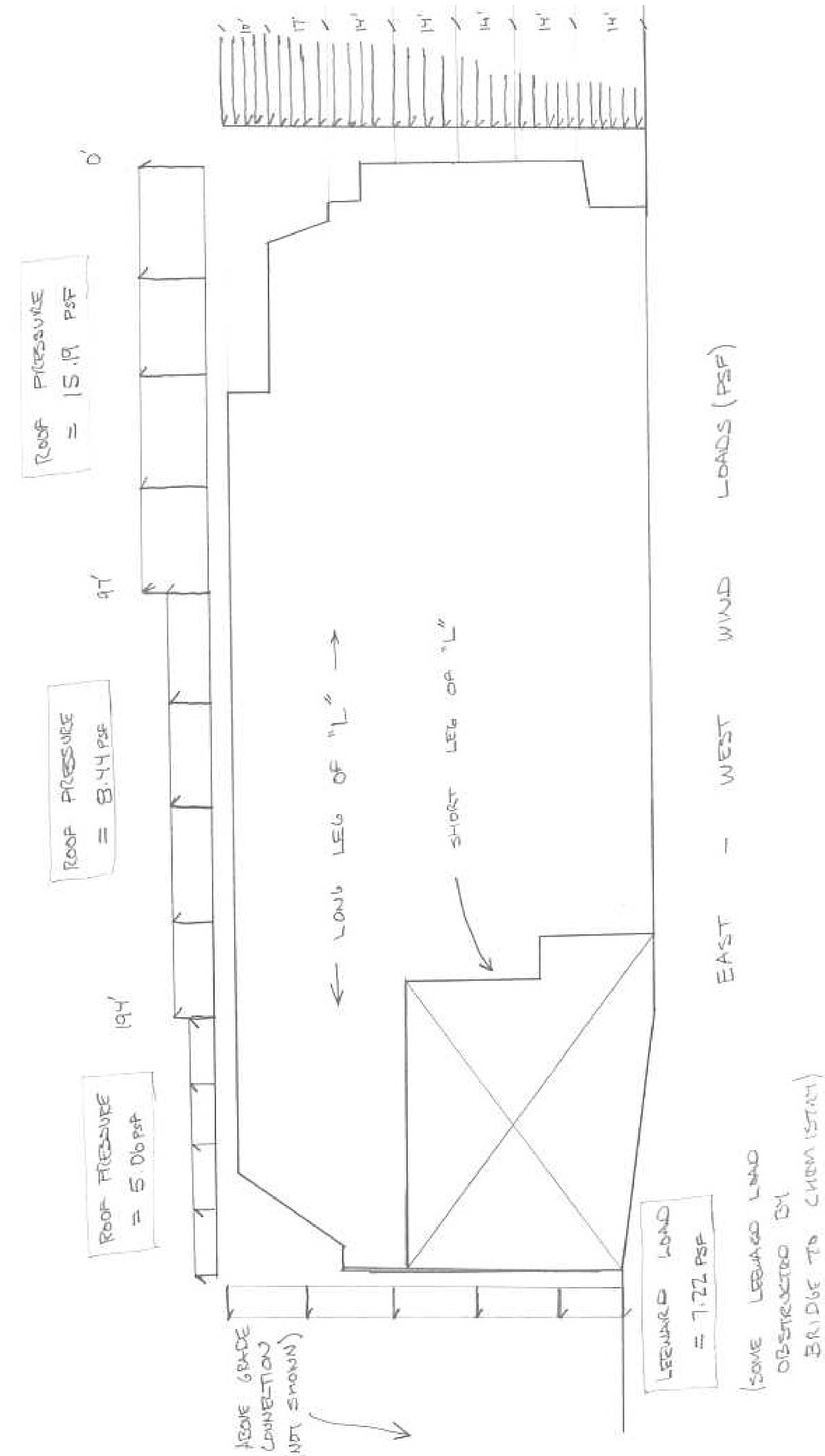
Wind Load (Continued)

East – West Wind Load Diagram - Detail



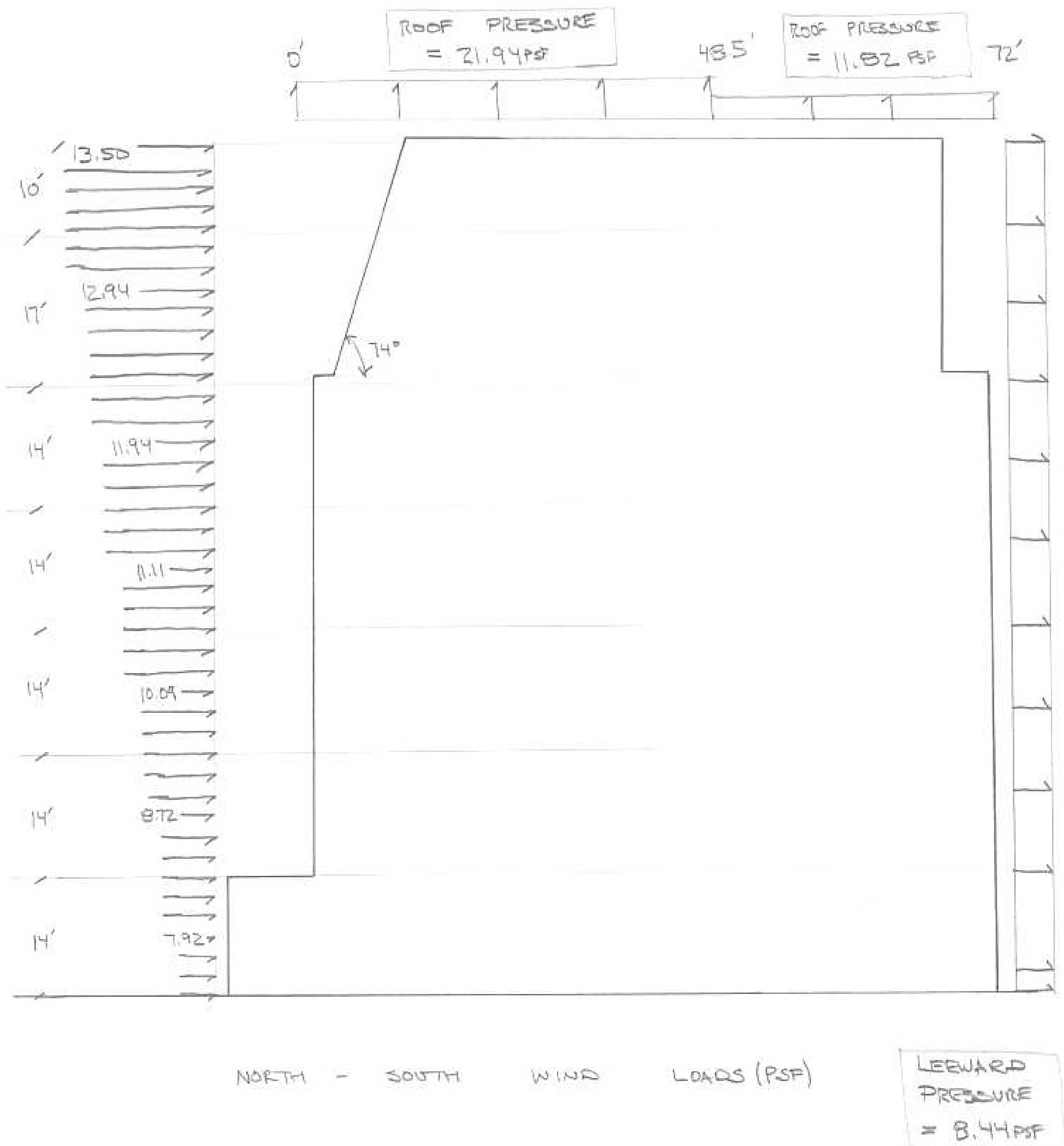
Wind Load (Continued)

East – West Wind Load Diagram - Overall



Wind Load (Continued)

North – South Wind Load Diagram - Overall



Seismic Load|

Seismic load will be calculated 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 (given as a live load in this document) was included as a dead load for the purposes of calculating the seismic lateral forces.

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

Base Shear| V_{base} = 183.04 k

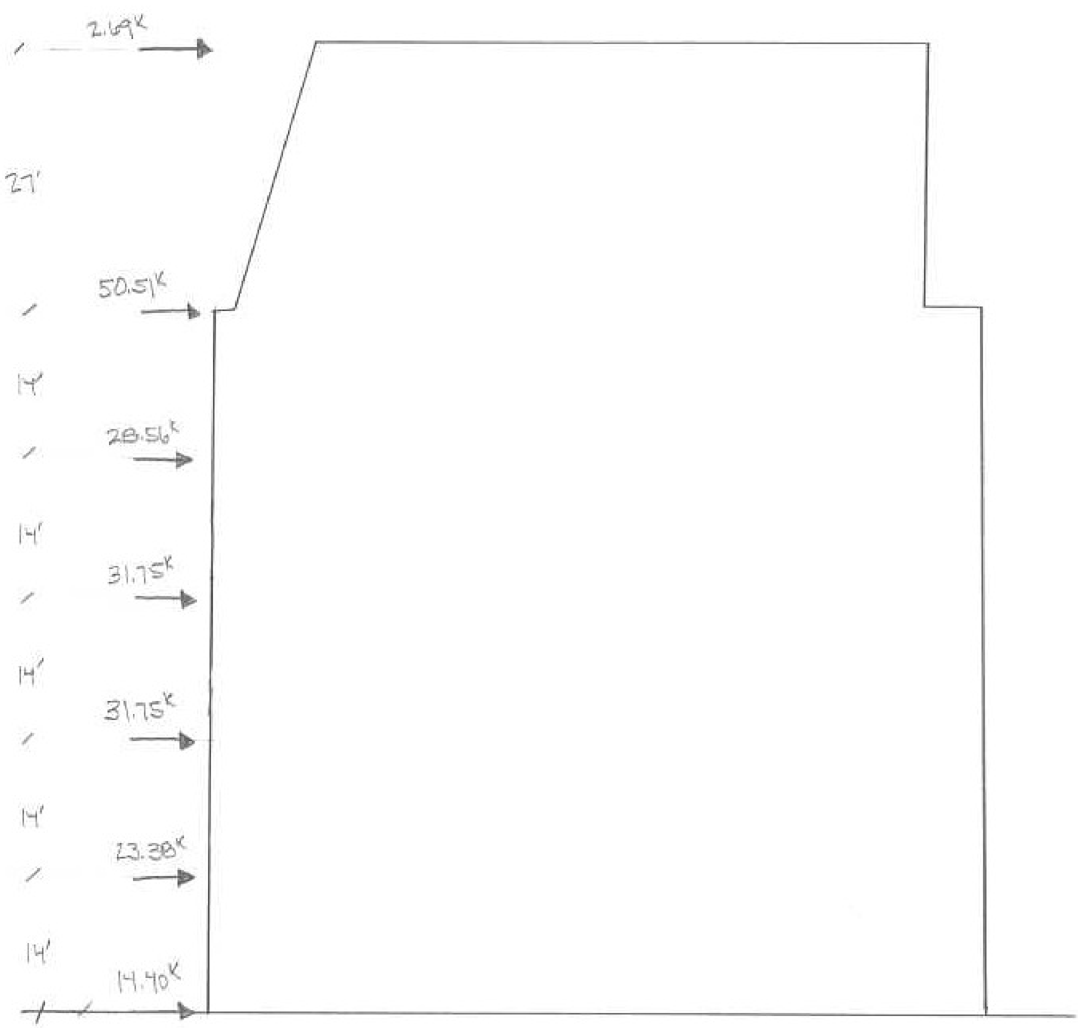
Overturning Moment| M_{ot} = 7945.81 k-ft

Seismic Load (Continued)|

	Floor System					Exterior Wall System					Seismic Load
	Width	Depth	Area	Dead Load	Floor Load (k)	Perimeter	Height	Area	Dead Load	Wall Load	At Story
	(ft)	(ft)	(ft^2)	(lbs/ft^2)	(k)	(ft)	(ft)	(ft^2)	(lbs/ft^2)	(k)	(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)|

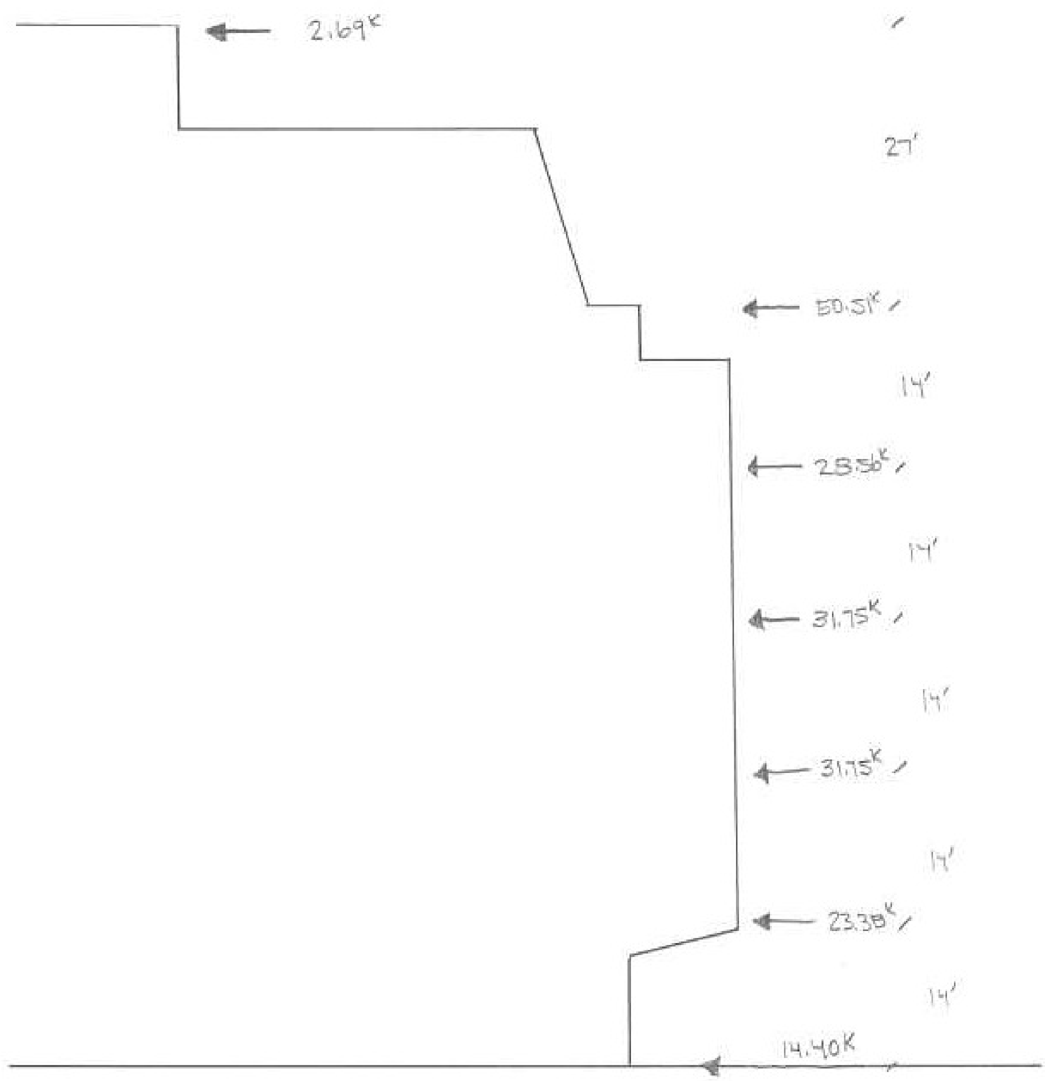
North – South Seismic Story Shear Diagram|



NORTH – SOUTH SEISMIC LOADS

Seismic Load (Continued)

East – West Seismic Story Shear Diagram



Other Loads – Not Considered

In addition to neglecting the snow drifting loads in this initial analysis for the reasons listed in the snow load section, other loads were also neglected to simplify this analysis. Soil lateral loads were neglected on the building's two levels that occur below grade. Soil loads were also neglected on retaining walls and other construction associated with the building. Other loading on the building related to geotechnical issues such as the water table was also neglected. The design of the foundation was studied, but not analyzed in depth and its loading was not calculated.

The wind load and how it affects the main wind force resisting system was studied. This allowed wind loads on side walls and roofs to be neglected. Wind load and how it affects the exterior walls, roof and components and cladding will need to be addressed at some point. Construction and erection loads were unknown and not considered; also the effects of mechanical and electrical equipment cannot be fully understood until more research goes into the dimensions and weights of that equipment.

Because this is a building housing mostly research labs a more in depth look should be taken to study the requirements for sidesway, vibration, and beam deflections. Laboratory spaces typically have much higher requirements for structural stiffness than most spaces. The analysis of structural systems and members undertaken in Technical Assignment I only considered the loads on members from a member strength point of view. Serviceability requirements were not considered.

Other features of the building such as cantilevers of upper stories and the structure of various entrance canopies and cantilevers will be studied in greater detail in following reports.

Structural System Spot Checks|

Spot checks were performed on a number of different structural elements to gather a better idea of the loading assumed and the buildings design. The elements of a lateral moment resisting frame were checked, as well as elements supporting gravity loads (beam, girder, column). LRFD and its associated phi factors and load combinations were used for all spot checks.

Lateral System Check|

I found that the wind load was the controlling lateral load case and I applied the wind loading in the east – west direction to the long leg of the "L" to begin my check of the lateral system. There are several moment and braced frames resisting lateral loads in this direction, but I assumed that the two moment frames that form the outer wall of the long leg of the "L" share the wind load on that section equally. I divided the east wall of the long leg of the "L" in half using tributary area and distributed the wind load to each moment frame as a point load applied horizontally at each floor level. I then added the dead and live gravity loads from the floors inside and put my moment frame and loading into SAP for analysis. Appropriate LRFD load combinations were used throughout. Additional information regarding the lateral frame analyzed can be found in the appendix.

I checked the worst case load on each different steel section used in forming the moment frame. My analysis found that most of the composite beams are oversized substantially – considering only strength, not deflections. Almost all of the sections were acceptable when compared with the given loading without even considering composite action. To save time I did not consider composite action for beams and girders unless the non composite carrying capacity of the member was exceeded. For one steel section in the lateral frame the given loading exceed both the non composite and composite moment capacity. However, for this steel section the applied moment only exceeded the moment capacity of the steel section considering composite action by 4.1%, which is reasonably close. The most probable reason my analysis had those steel members failing is because I incorrectly estimated the load they carry. The members that failed supported the mechanical penthouse level, but on the edge of the building where their tributary area is not the mechanical space but actually a small stepped roof. I assumed the loading for the entire mechanical penthouse level (roof or actual mechanical space) to be uniform over the entire area. It is very possible that these members were designed to only support the roof live loads (30 psf) which are substantially smaller than the mechanical room live loads (200 psf).

Structural System Spot Checks (continued)

I believe that the steel beam sections were oversized when compared to my analysis because I considered only strength, not the deflections of the members or frames as a whole. Steel beam sizes in the moment frames could be increased to reduce the sidesway of the building or to combat the actual deflections of the floors themselves. Because my time, knowledge, and resources were limited in this first technical assignment I will have to put off considering the deflections of the lateral force resisting system until a later assignment.

I also checked the columns of the lateral force resisting system. Initially only the moment applied to the columns was checked to make sure that it didn't exceed each sections capacity. All of the columns were well under the moment capacity they can carry. I then chose to refine my analysis and consider the effects of combined loading (axial + moment) for the columns. This was done in order to have a more accurate idea of how close my calculations were to the original design. Each column of the moment frame carried a fairly typical load so one typical column axial load calculation was done and applied to each column. All of the columns in my moment frame were checked for combined axial and bending on the first story where the effects of axial load and moment are the greatest. All of the columns in the lateral support system were sized appropriately to handle the combined axial and flexural loading.

My conclusions on the lateral support system are that it is more than adequately sized to handle the loads. I feel that some of the members are oversized when compared to my calculations because I only calculated for strength, not deflections. Additional information on the lateral support moment frame that was analyzed can be found in the appendix.

Floor Framing – Beam

Most of the beams frame into girders that frame into moment frames so the only choice was basically column D – 1 and its associated beams and girders. It is an exterior column in the knuckle portion of the building along the north exterior wall. It supports the first through fourth floors, mechanical penthouse and roof.

Beams framing into the girder that is supported by column D – 1 were checked first. The composite steel beams were oversized when checked for strength, the beams were not checked for deflections. The beams were probably oversized with respect to strength because of deflections. The beams also have a camber of 1" over a 31' span. Calculations and other information regarding the beam spot check can be found in the appendix.

Structural System Spot Checks (continued)|

Floor Framing – Girder|

The typical beam analyzed previously then frames into a typical girder on both ends. The girder is a composite steel section which supports three beams at quarter points on either side. The girder was found to be satisfactory – again a bit oversized. The fact that the girder is oversized is probably attributed to the fact that it was designed for deflections and not strength. Girder calculations can be found in the appendix.

Floor Framing – Column|

The column I analyzed was found to be about twice as strong axially as it needed to be. This could be the result of several factors. Live loads may have been reduced in this analysis and they weren't in the original design. ASCE 7-05 states "Live loads that exceed 100 lb/ft^2 shall not be reduced." In this analysis live loads of 100 psf were allowed to be reduced by my interpretation of the code because they do not *exceed* 100 psf. Also the roof framing system above column D – 1 is a complex steel framed hipped roof system with a slope of about 74 degrees making it form more of a wall than a roof. It also bears down on a series of columns that are supported by girders that are then supported by column D – 1. Rather than perform a complex analysis of the roof structure I just did rough calculations of the beam weight and spacing, along with rough calculations of the roof surface and decking. Because my time was limited and I am just getting familiar with the structure of my building these simplifying assumptions may have neglected a significant portion of the weight of the roof structure. Another factor in the columns being oversized could be the fact that there are eccentricities in the column that I was not aware of and the larger size is needed to compensate for the moment that occurs in the column. Column check calculations are available in the appendix.

Structural Spot Checks – Conclusion|

Overall my structure was very capable of handling the loads that will be placed on it from a strength perspective. However, any further analyses of the structure should begin to consider serviceability requirements in the design of the structural system. Refinements also could be made to the methods used to determine the loads on members to achieve more accurate results.

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.

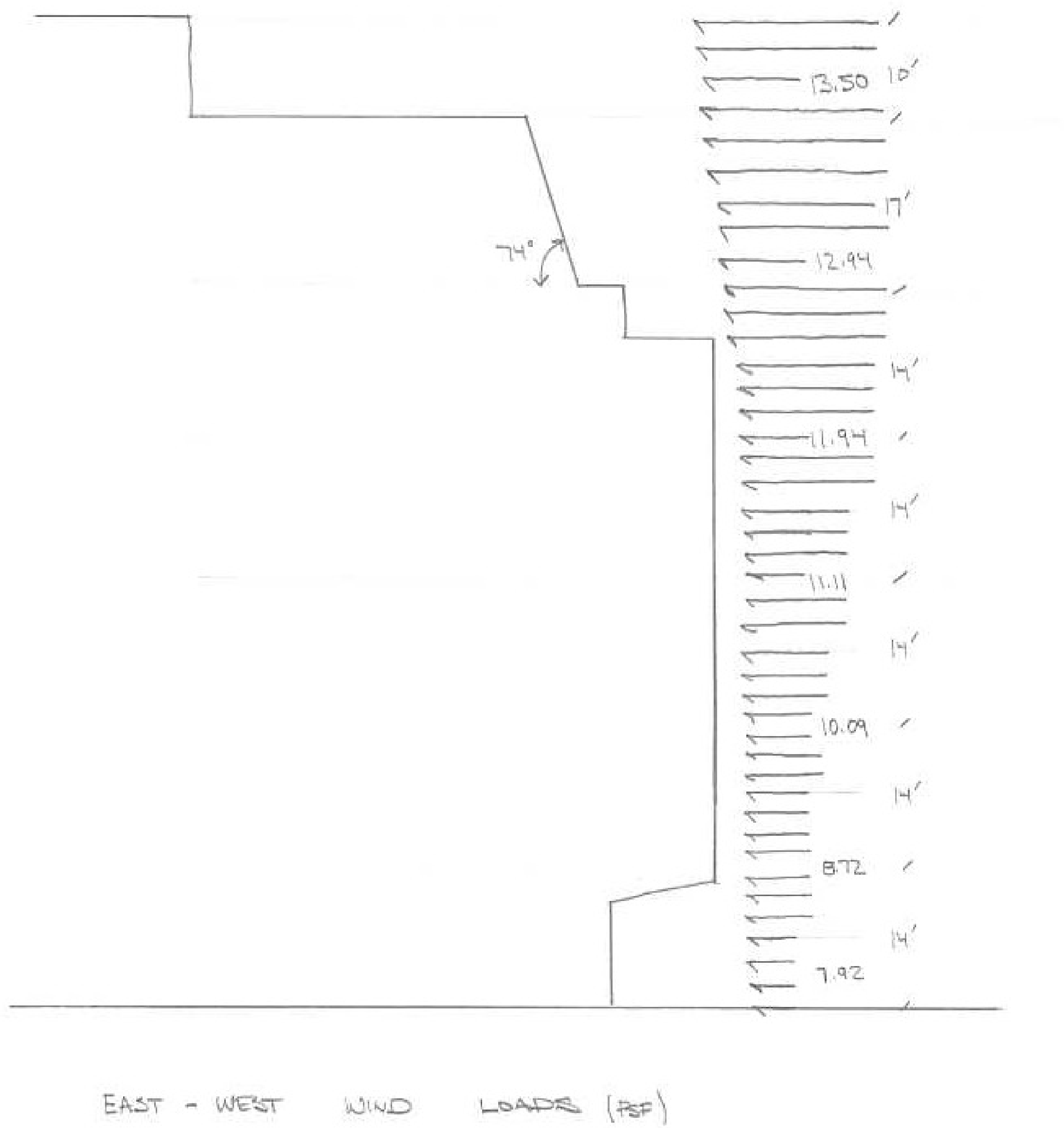
FLAT ROOF (NORMAL & PARALLEL):

$h = 97'$ ↕ $L = 250'$ $h/L = 97/250 = .388$
LONG LEG OF "L"

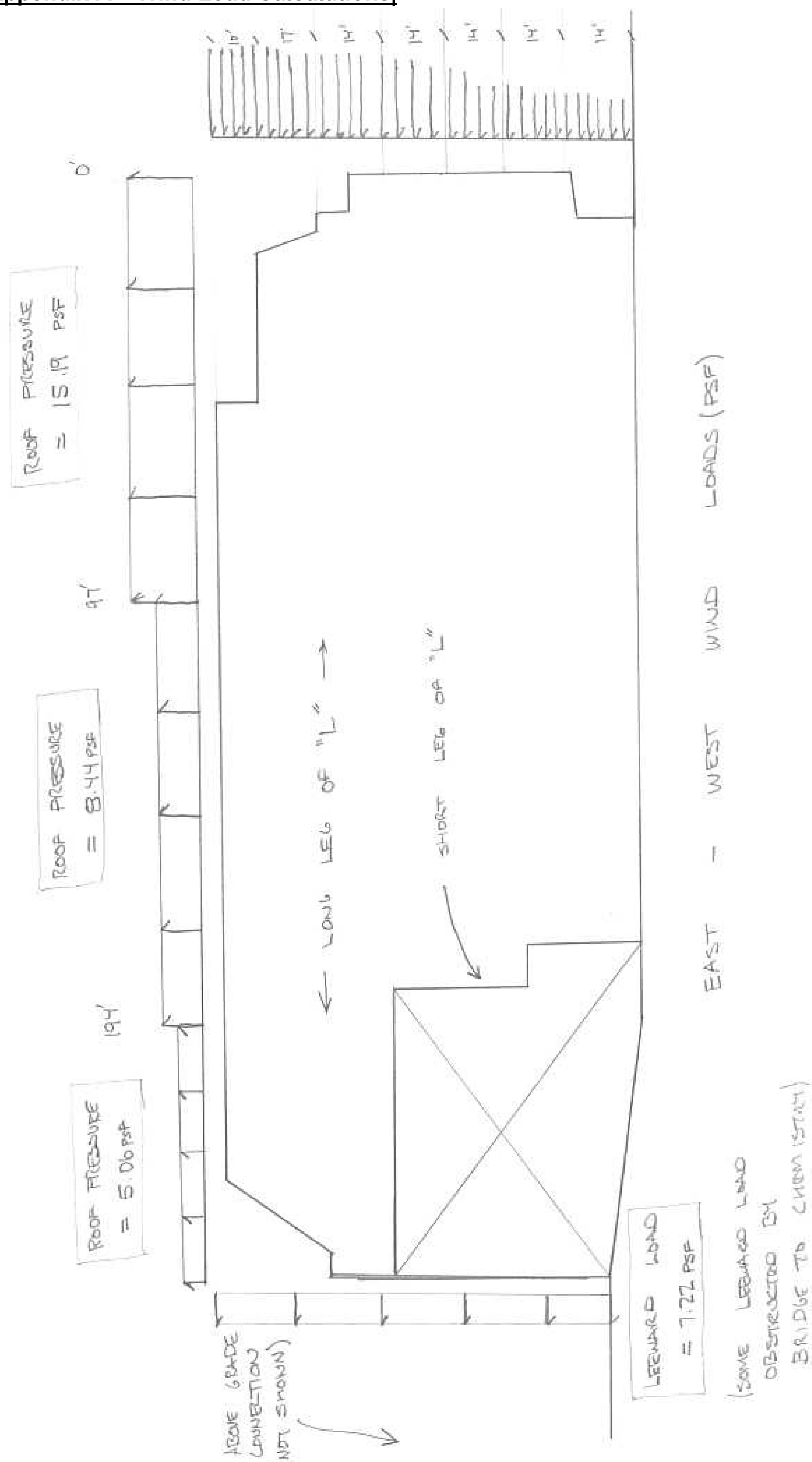
0 → 48.5'	$C_p = -.9$
48.5' → 97'	$C_p = -.9$
97' → 194'	$C_p = -.5$
194' → 250'	$C_p = -.3$

FOR WIND RUNNING ALONG LONG LEG OF "L"

Appendix A – Wind Load Calculations



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:

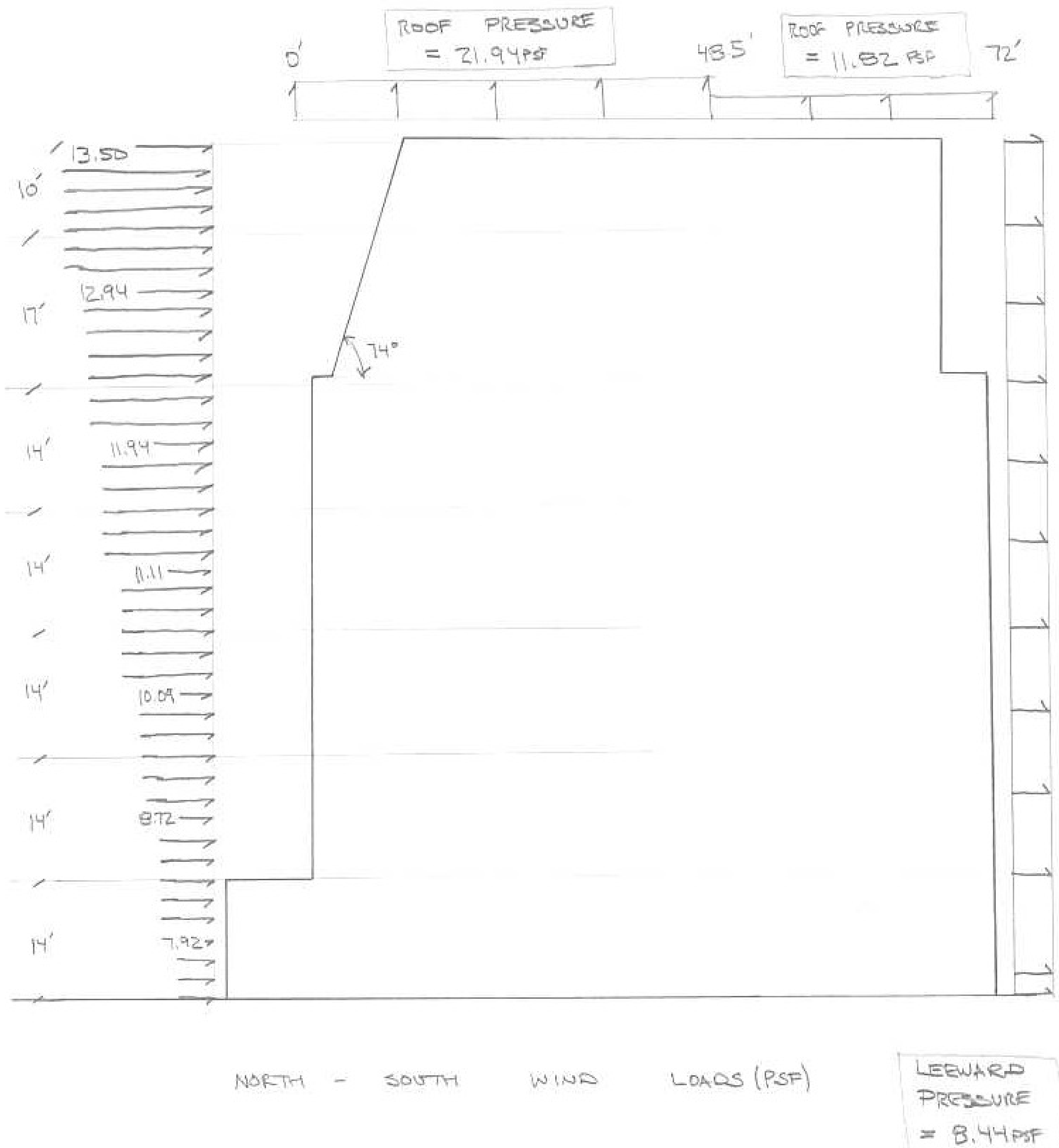
$h = 97'$ $L = 72'$

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

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

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

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 W18x50 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

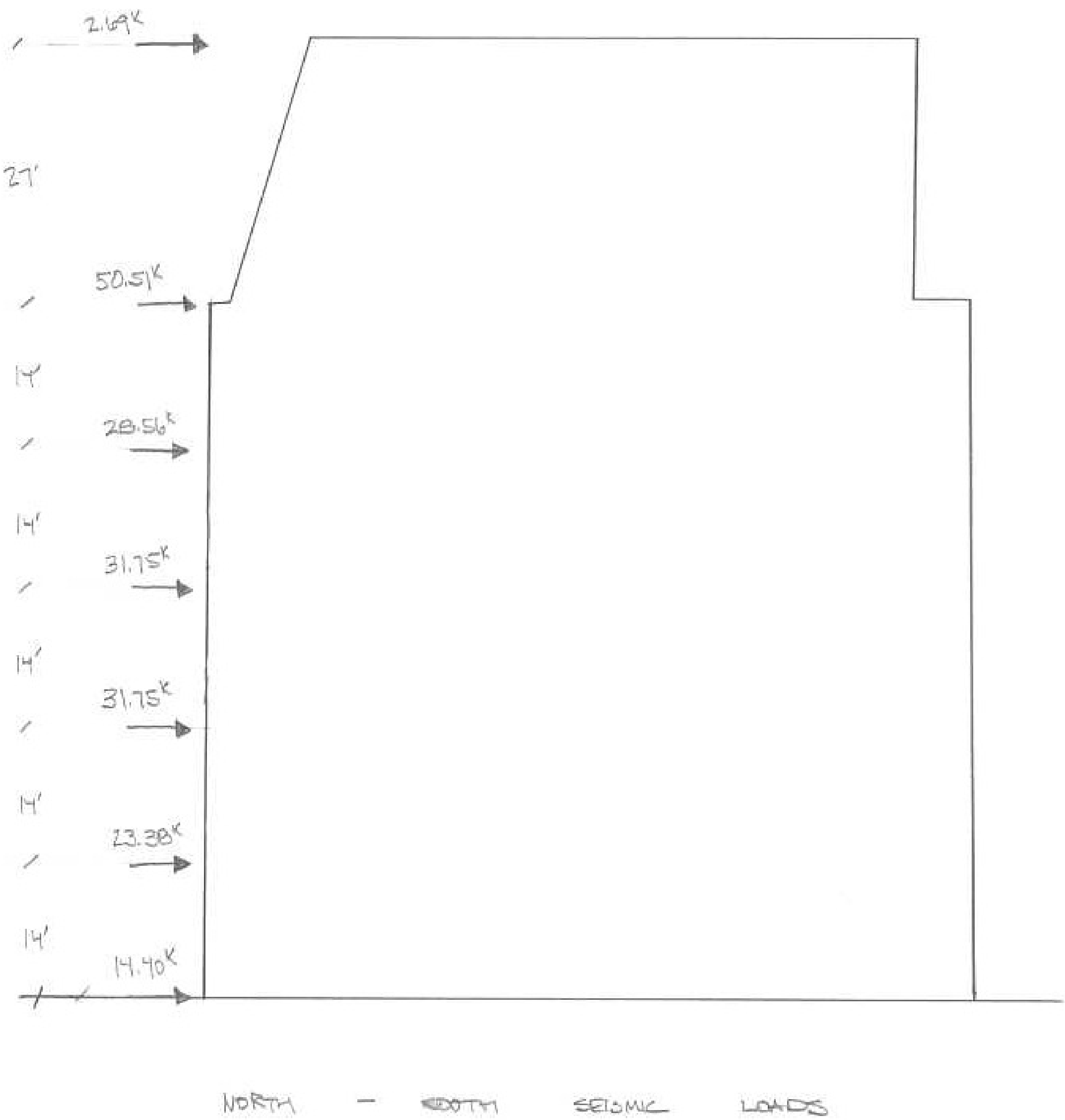
DECK = 2 PSF

FRAMING = W16x31 @ 8' o.c.

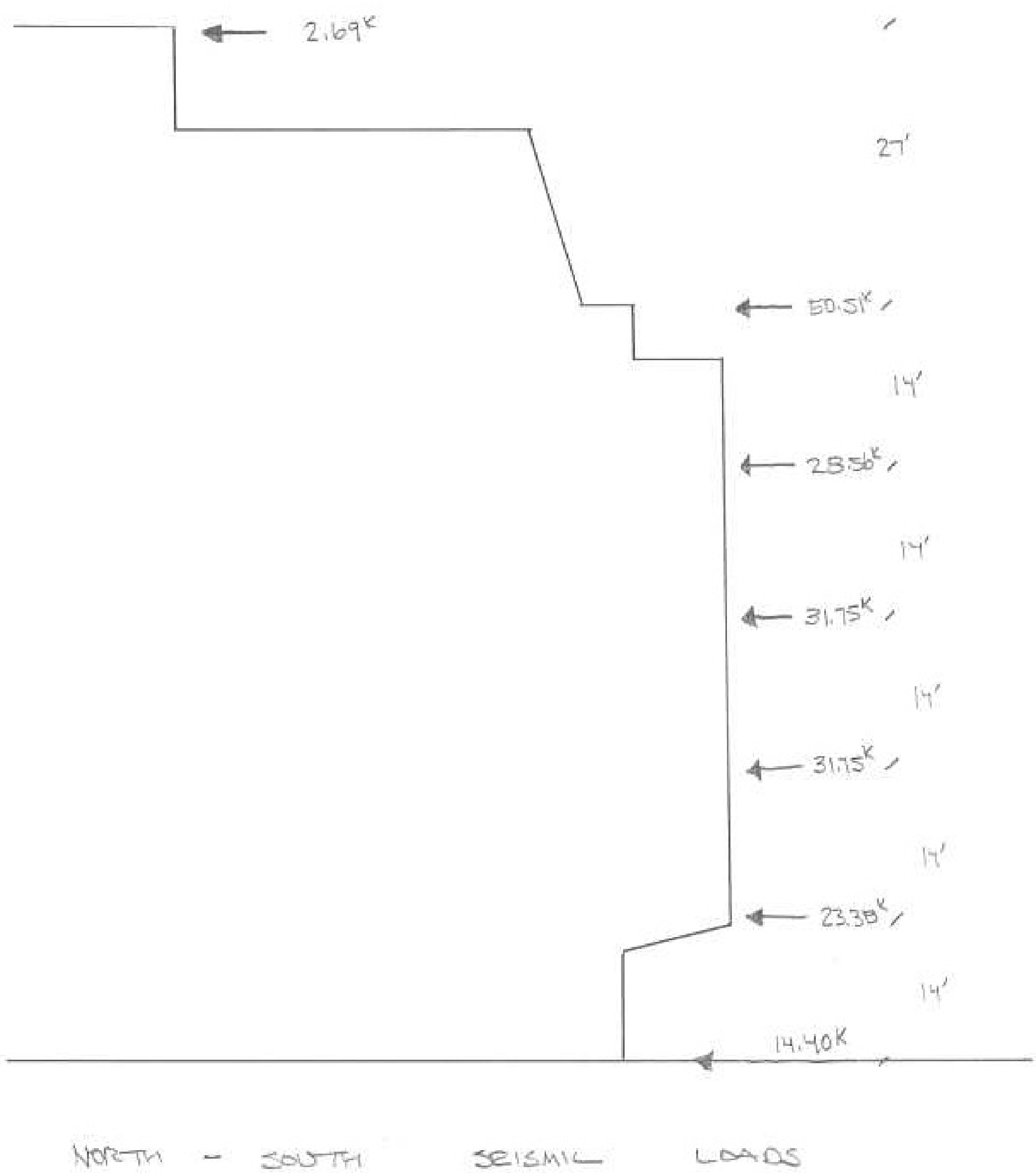
31/8 = 3.875

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

Appendix B – Seismic Load Calculations

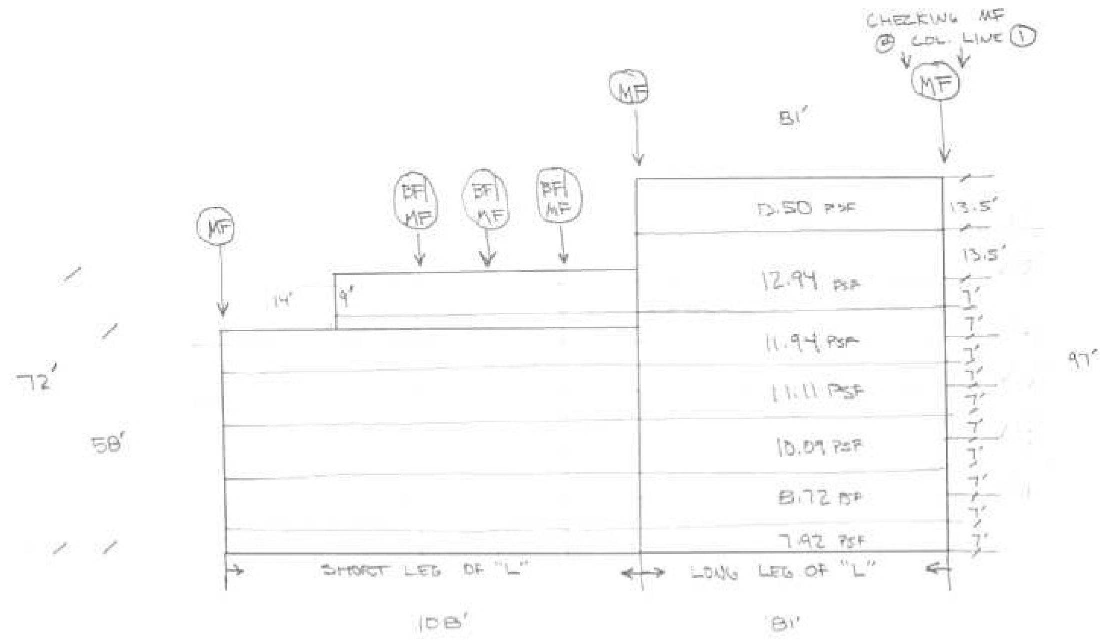


Appendix B – Seismic Load Calculations



Appendix C
Wind v. Seismic – Controlling Load

Appendix C – Wind v. Seismic – Controlling Load



EAST ELEVATION

A	PSF	WINDWARD P_{WIND}	WINDWARD + LEEWARD $P_{WIND\ TOT}$	$P_{SEISMIC}$	FLOOR
7(189)	7.92	10.478^{K}	20.030^{K}	2.69^{K}	GROUND FLOOR
14(189)	8.72	23.073^{K}	42.177^{K}	50.51^{K}	1
14(189)	10.09	26.698^{K}	45.802^{K}	28.56^{K}	2
14(189)	11.11	29.397^{K}	48.501^{K}	31.75^{K}	3
≈14(189)	11.94	31.593^{K}	50.697^{K}	31.75^{K}	4
9(80) + 20.5(81)	12.94	30.804^{K}	47.991^{K}	50.51^{K}	PENTHOUSE
13.5(81)	13.50	14.762^{K}	22.657^{K}	2.69^{K}	ROOF
			↑ $WIND_{TT}$	↑ $SEISMIC_{TT}$	

LOAD COMBINATIONS:

$1.2D + .8W$
$1.2D + 1.6W + .5L$
$1.2D + E + .5L$
$.9D + 1.6W$
$.9D + E$

Appendix C – Wind v. Seismic – Controlling Load

DIST. LOAD DUE TO FLOOR:

$A_T = 4'$ (SPAN) $\longrightarrow$ $4'(31') = 124\,FT^2 (K_{LL}) = 248$

$K_{LL} = 2$

DEAD LOAD:

SLAB/DECK: 71 PSF

PARTITIONS 20 PSF

91(4)

$\boxed{DL = 364 \text{ LBS/FT}}$

LIVE LOAD:

OFFICE/LAB/CLASSROOM: 80 PSF

↑ NON REDUCIBLE.

MECHANICAL: 200 PSF

↑ NON REDUCIBLE

$LL_{TYP} = 320$ LBS/FT

$LL_{MECH} = 800$ LBS/FT

WIND LOAD ON FRAME & SEISMIC LOAD:

- ASSUMING THAT MOMENT FRAME ON COLUMN LINE ① TAKES 1/2 OF WIND LOAD ON PROJECTED AREA.
- BY OBSERVATION WIND LOAD WILL CONTROL.

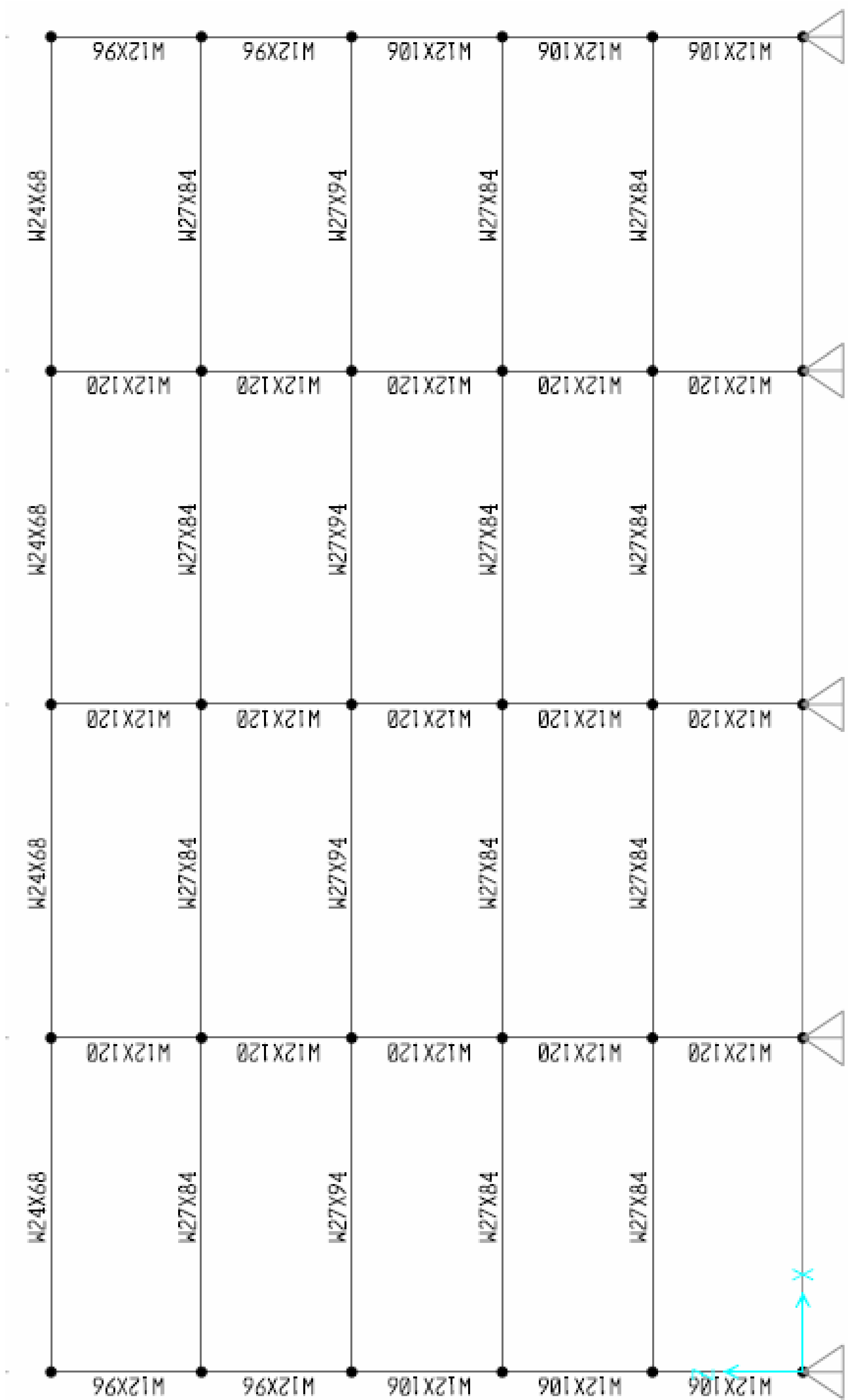
		WINDWARD	LEEWARD	TOTAL	AREA	LOAD	1.6W	1.2D	.5L
P	(40.5)(13.5)	13.50	7.22	20.72	546.75	28.067 K	44.907 K		
	(40.5)(20.5)	12.94	7.22	20.16	830.25			.437 K/FT	.4 K/FT
4	(40.5)(14)	11.94	7.22	19.16	567	10.864 K	17.382 K	.437 K/FT	.16 K/FT
3	(40.5)(14)	11.11	7.22	18.33	567	10.393 K	16.629 K	.437	.16 K/FT
2	(40.5)(14)	10.09	7.22	17.31	567	9.815 K	15.704 K	.437	.16 K/FT
1	(40.5)(14)	8.72	7.22	15.94	567	9.038 K	14.461 K	.437	.16 K/FT

Appendix D
Lateral – Moment Frame Analysis

Appendix D – Lateral – Moment Frame Analysis|

Moment Frame on Column Line 1

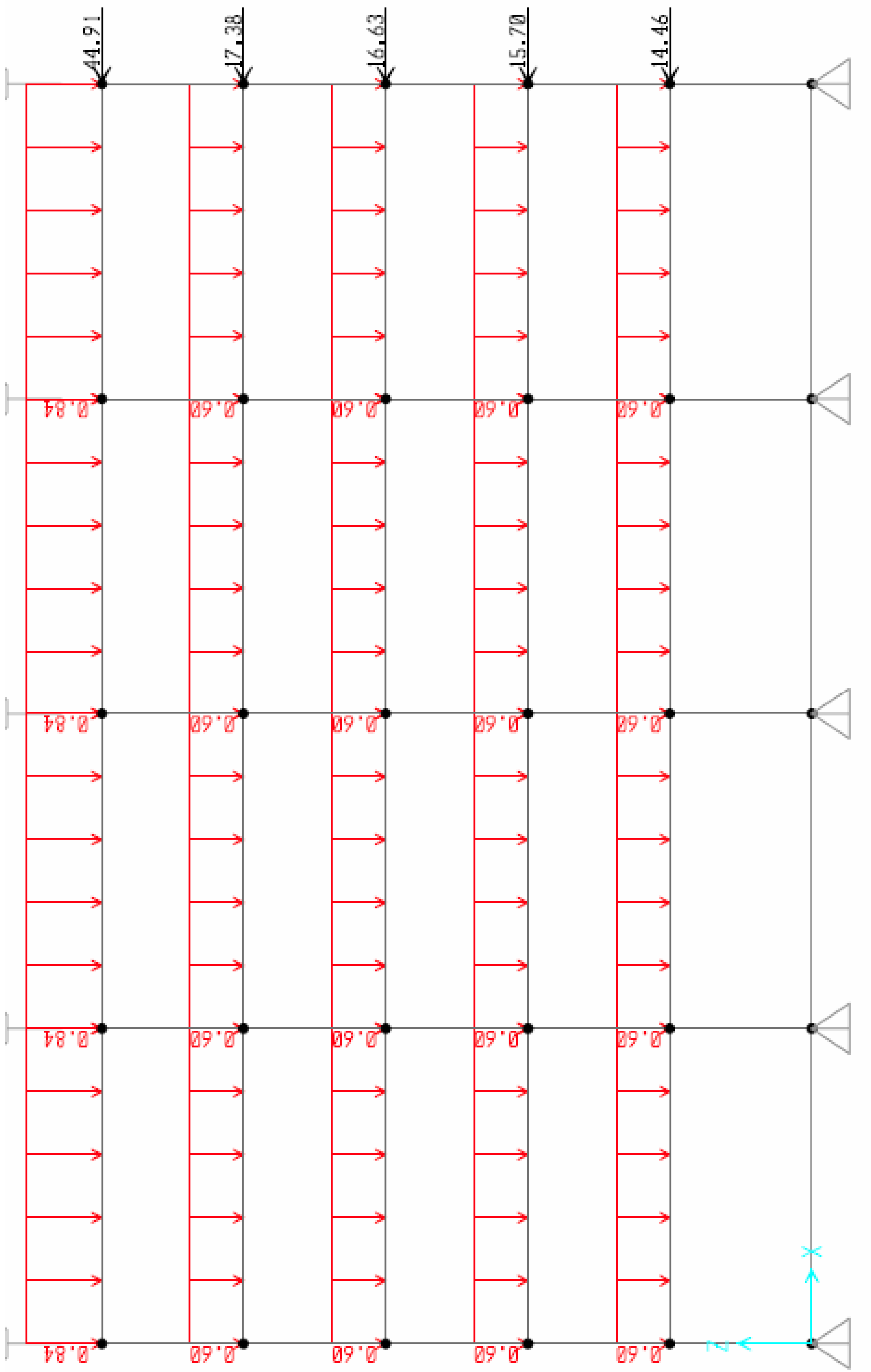
Section Properties



Appendix D – Lateral – Moment Frame Analysis

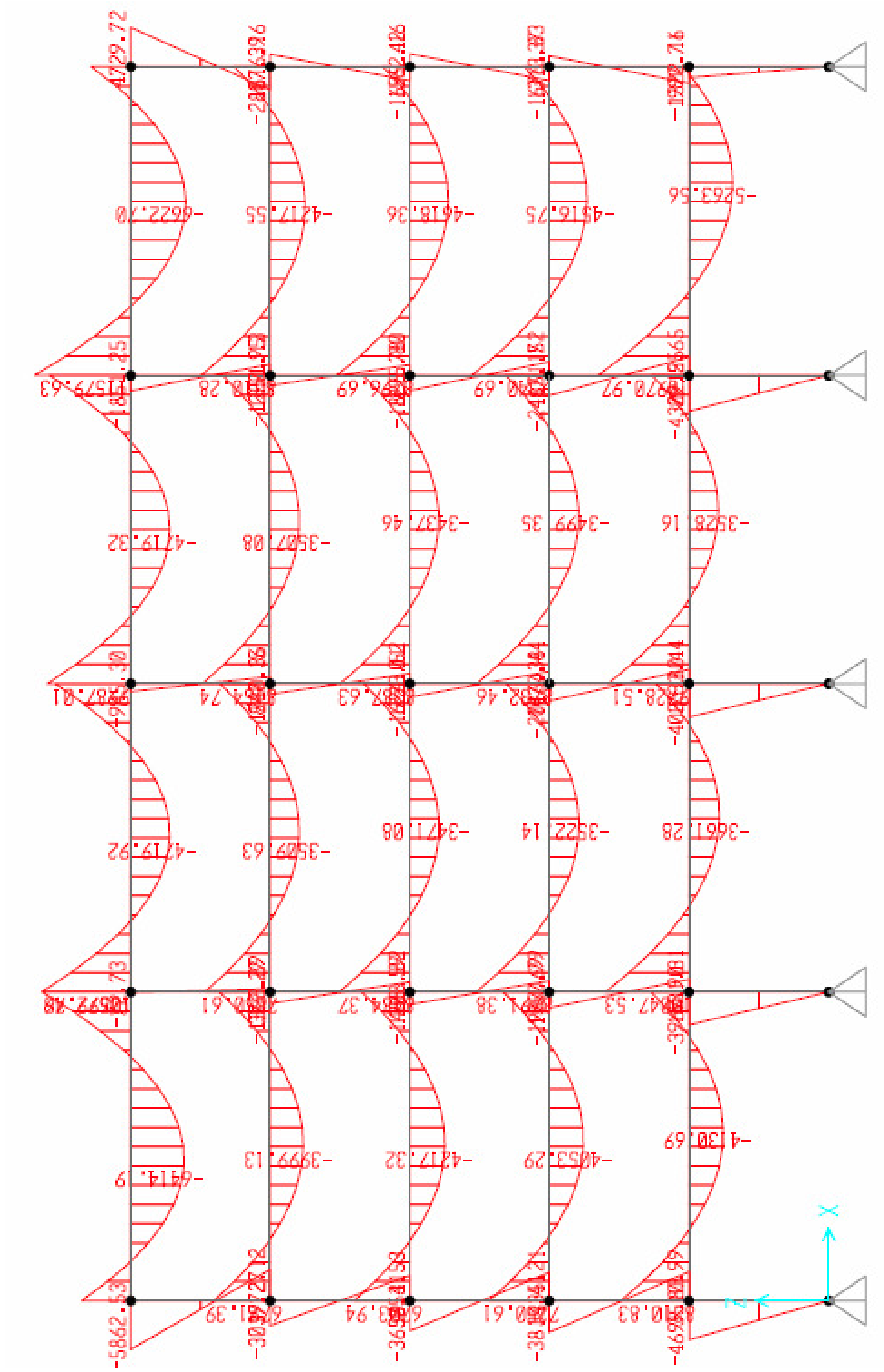
Moment Frame on Column Line 1

Loading (LRFD Factored) (kips)



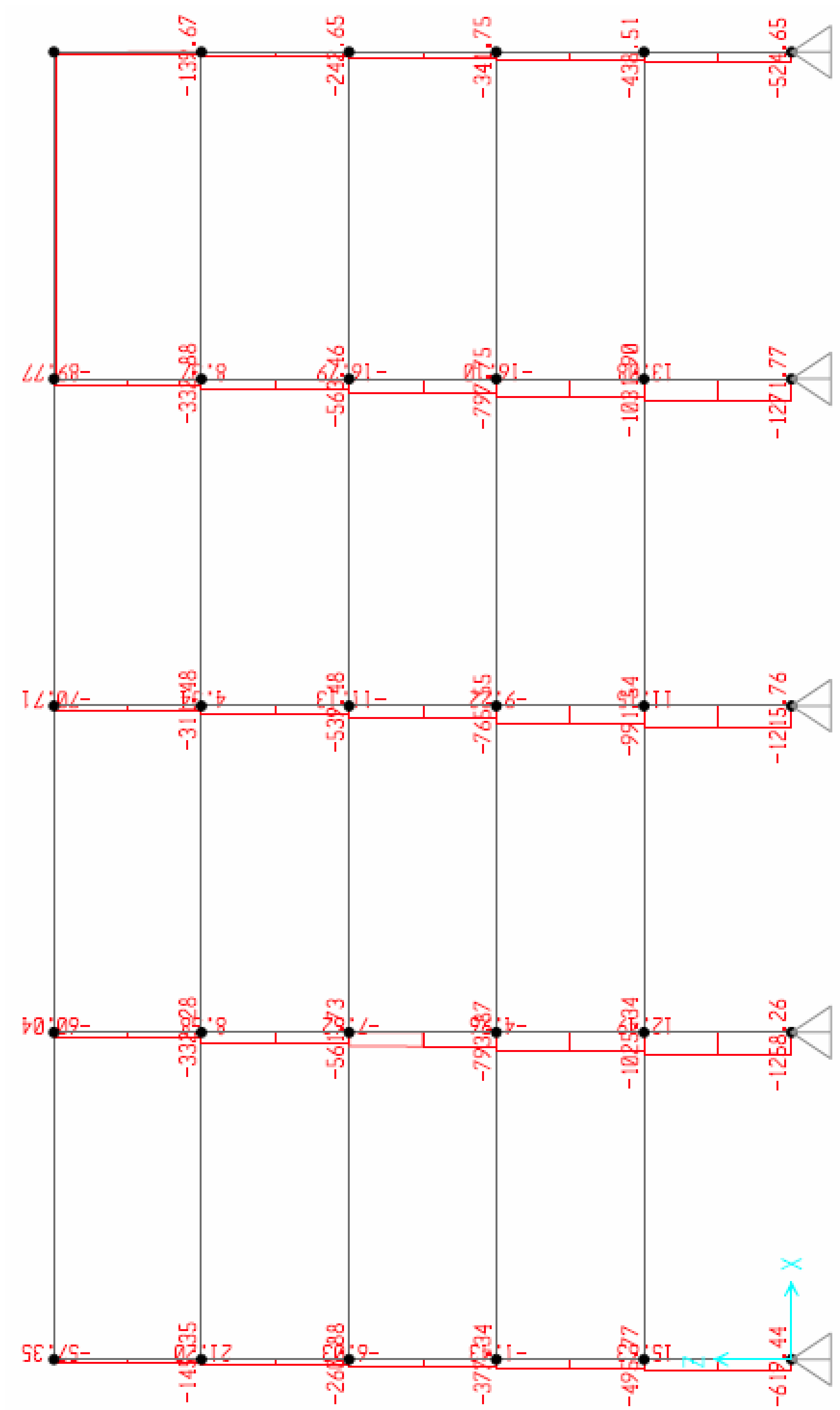
Appendix D – Lateral – Moment Frame Analysis

Moment Frame on Column Line 1
Moments (k-in)



Appendix D – Lateral – Moment Frame Analysis

Moment Frame on Column Line 1
Axial Force (k)



Appendix D – Lateral – Moment Frame Analysis|

Moment Frame on Column Line 1

√ CHECK COMPOSITE BEAMS:

	M_{MAX}	ϕM_P	
W27×94	692.5 FT·K	1040 FT·K	√OK
W27×84	914.2 FT·K	915 FT·K	√OK
W24×68	964.9 FT·K	664 FT·K	FAILS

↑ NON COMPOSITE

USING COMPOSITE PROPERTIES

$F_y = 50$ KSI √OK

– CHECK GIRDER THAT FAILED ASSUMING NON COMPOSITE ACTION.

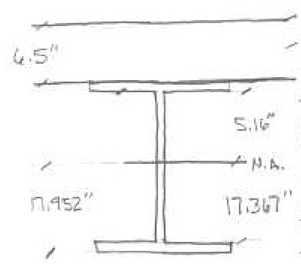
W24×68

$b = 1/8\,(31') = 46.5''$ > $b_{EFF} = 46.5''$

$b = 1/2\,(8') = 48''$

SHEAR STUD $= 13.3^K/\text{STUD}$ → $\Sigma Q_n = (13.3)(20) = 266^K$

$$a = \frac{266^K}{.85(4)(46.5)} = 1.682''\downarrow$$



$1.682''$ $C_c = 266^K$

$t_F = .585''$ $T_S = 20.1(50) = 1005^K$

$b_F = 8.97''$ $1005^K - 266^K = \frac{739^K}{2} = 369.5^K$

$d = 23.7''$

$A_{ST} = 20.1\ \text{in}^2$

$t_w = .415$

$$M = 266\left(\frac{1.682}{2}\right) + 262.3(6.7925) + 107.128(9.665)$$
$$- 360.4(20.929) - 262.373(29.908)$$

$\boxed{\phi M_n = 926.2 \text{ FT·K} \quad \text{FAILS, BUT CLOSE}}$

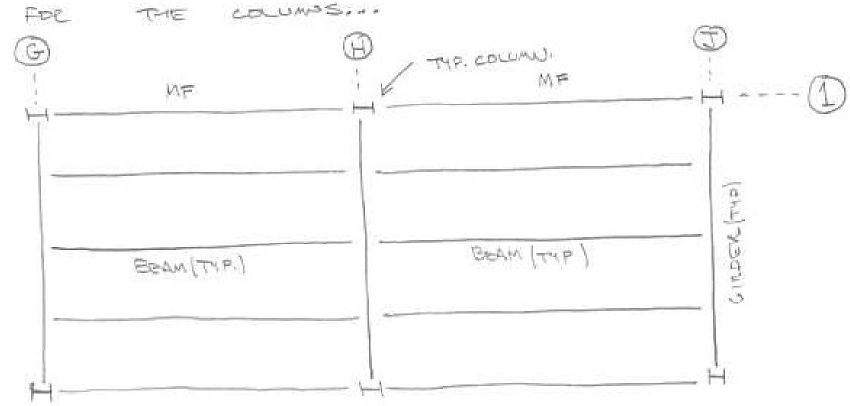
Appendix D – Lateral – Moment Frame Analysis

Moment Frame on Column Line 1

√ CHECK COLUMNS:

	MAX M	ϕM_P	
W12x120	359.1 K·FT	698 K·FT	√OK
W12x106	391.6 K·FT	615 K·FT	√OK
W12x96	488.5 K·FT	551 K·FT	√OK

– COLUMNS ARE UNDER BOTH AXIAL & MOMENT LOADING SO ϕM_P MIGHT BE SO LARGE TO SUPPORT COMBINED LOADING. A CHECK OF GRAVITY LOADS FOLLOWS FOR THE COLUMNS...



TYPICAL BAY ARRANGEMENT 31' x 31'

TYPICAL COLUMN FLOORS 1-4:

$A_{T\ COL\ H} = (31')(15.5') = 480.5\ FT^2\ (4) = 1922\ FT^2$

$LL = 80\ PSF \longrightarrow LL_R = 80\left(.25 + \frac{15}{\sqrt{4(1922)}}\right) = 33.7\ PSF\ (.5) = \boxed{16.87}$

$DL = 20 + 10 + 71 = 101\ PSF\ (1.2) = \boxed{121.2}$

$P = 138.05\,(1922)$

$\boxed{P = 265.3^K}$

TYPICAL COLUMN (PENTHOUSE):

$A_{T\ COL\ H} = 480.5\ FT^2$

$LL = 200\ PSF\ (.5) = \boxed{100}$

$DL = 20 + 10 + 71 = 101\ PSF\ (1.2) = 121.2$

$P = 480.5\,(221.2)$

$\boxed{P = 106.3^K}$

TYPICAL COLUMN (ROOF):

$\frac{3}{4}\ A_{T\ COL\ H} = 296\ FT^2$

$LL = 30\ PSF$ (N.R.)

$DL = 32\ PSF$

$TL = 1.2(32) + 1.6(30) = 86.4\ PSF$

$P = 296\,(86.4)$

$\boxed{P = 25.5^K}$

Appendix D – Lateral – Moment Frame Analysis

Moment Frame on Column Line 1

√ COLUMNS (CONTINUED):

COMBINED LOADING @ COLUMN BASES:

	P_r	M_r	ϕP_c	ϕM_c
W12x106	397.1 K	391.6 FT·K	1130 K	615 FT·K
W12 x 120	397.1 K	359.1 FT·K	1290 K	698 FT·K

W12x106

$$\frac{P_r}{P_c} = \frac{397.1}{1130} = .351$$

$$.351 + \frac{8}{9}\left(\frac{391.6}{615}\right) \le 1.0$$

$$\boxed{.923 \le 1.0} \quad \checkmark OK$$

W12x120

$$\frac{P_r}{P_c} = \frac{397.1}{1290} = .307$$

$$.307 + \frac{8}{9}\left(\frac{359.1}{698}\right) \le 1.0$$

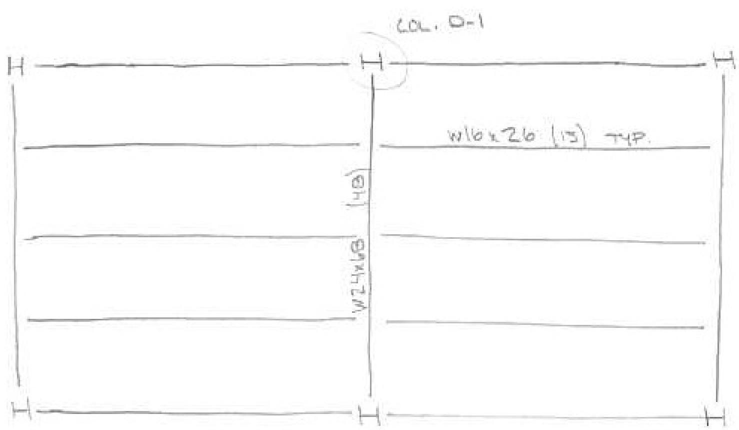
$$\boxed{.764 \le 1.0} \quad \checkmark OK$$

LATERAL FORCE RESISTING MOMENT FRAME IS ADEQUATE.

Appendix E
Gravity – Beam Analysis

Appendix E – Gravity – Beam Analysis|

CHECKING BEAMS | GIRDERS TYING INTO COLUMN (D)–(1)



CHECK TYPICAL W16 x 26 BEAM:

SPAN = 31'

SPACING = 7'-9"

A_T = 240.25 FT²

LIVE LOAD:

80 PSF

LL = 80 PSF

DEAD LOAD:

10 PSF + 20 PSF + 71 PSF

DL = 101 PSF

REDUCE LL:

$$80\left(.25 + \frac{15}{\sqrt{(2)(240.25)}}\right) = 74.7438 \text{ PSF}$$

TL = 1.2 (101) + 1.6(74.7438) = 240.79 LBS (7.75') = 1866.12 LBS/FT

$$M = \frac{1.866\ (31)^2}{8} = \boxed{224.168 \text{ K·FT}} \qquad M_{+\text{SELF WEIGHT}} = \frac{1.8972(31)^2}{8} = \boxed{227.9}$$

SHEAR STUD 3/4" φ

Q_n = 13.3^K $\quad \Sigma Q_n$ = (15)(13.3) = 199.5^K

b_{EFF} = .5(7.75)(12) = 46.5 > 46.5(2) = $\boxed{b_{EFF} = 93''}$

.125(31)(12) = 46.5

Appendix E – Gravity – Beam Analysis|

CHECK TYPICAL W16x26 BEAM:

COMPOSITE ACTION w/ 15 SHEAR STUDS

$\Sigma Q_n = 199.5^K$ $\qquad Y_2 = 6.5'' - \frac{199.5}{1.7(4)(93)} = 6.18''$

{AISC T.3-19} $\qquad \rightarrow a = .64''$

$\boxed{\phi M_n = 311 \text{ K·FT} \text{ AT LEAST}}$

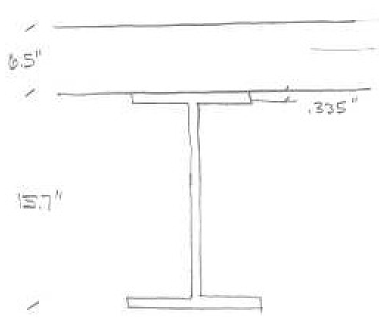
$a = \frac{199.5}{.85(4)(93)} = .63093''$

$C_c = .85(4)(93)(.63093) = 199.5^k$

$T_s = 7.68(50) = 384^K$

$T_s = 384 - 199.5 = \frac{184.5^K}{2} = 92.25^K$

$C_c = 199.5^K$ $\qquad T_s = 199.5^K$
$C_s = 92.25^K$ $\qquad T_s = 92.25^K$



$a = .63093''$

$t_f = .345''$
$b_f = 5.50''$

$d = 15.7''$
$t_w = .250''$

$M_n = 199.5(.315465) + 92.125(6.675) - 2.624(6.83977) - 187.625(14.35) - 94.875(22.0275)$

$M_n = 343.53$ FT·K

$\boxed{\phi M_n = 309.2 \text{ FT·K}}$

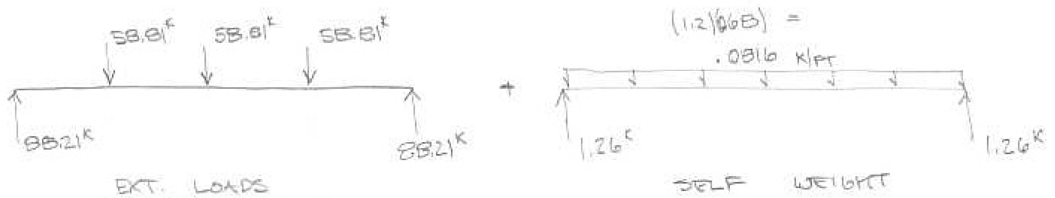
$\geq$

227.9 FT·K

BEAM IS ADEQUATELY SIZED.

Appendix F
Gravity – Girder Analysis

Appendix F – Gravity – Girder Analysis|



CHECK W 24x68 GIRDER:

$M = 911.5$ K·FT $\qquad M = \frac{.0816\,(31)^2}{8} = 9.802$ K·FT

$\boxed{M_u = 921.302 \text{ K·FT}}$

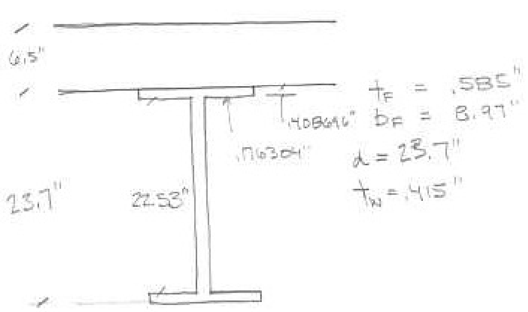
$\Sigma Q_n = (13.3)(48) = \boxed{638.4^K}$

$b_{EFF} = \begin{cases} .5(31)(12) = 186.0'' \\ .125(31)(12) = 46.5'' \end{cases} \qquad \boxed{b_{EFF} = 93''}$

$a = \frac{638.4}{.85(4)(93)} = 2.02''\downarrow$

$C_c = 638.4^K \qquad T_s = 50(20.1) = 1005^K$

$1005 - 638.4 = \frac{366.6}{2} = 183.3^K$



$M_n = 638.4\,(1.01) + 183.3\,(6.70435)$
$- 79.07\,(6.99685) - 467.498\,(18.35)$
$- 262.373\,(29.9075)$

$M_n = 1258.75$

$\boxed{\phi M_n = 1132.8 \text{ K·FT} > 921.3 \text{ K·FT}}$

GIRDER IS ACCEPTABLE.

Appendix G

Gravity – Column Analysis

Appendix G – Gravity – Column Analysis|

COLUMN D-1:

ROOF:

DEAD LOAD: $\underset{\text{SUPERIMPOSED}}{30} + \underset{\text{ASSUMED STRUCTURE WEIGHT}}{30}$ $DL_{ROOF} = 60$ PSF

LIVE LOAD: 30 $LL_{ROOF} = 30$ PSF

$A_T = 480.5$ $TL = 1.2(60) + 1.6(30) = 120$

$P_{ROOF} = 57.66^k$

PENTHOUSE:

DEAD LOAD: $\underset{\text{STRUCTURE}}{11} + \underset{\text{DECK/SLAB}}{71}$ $DL_{PENT} = 82$ PSF

LIVE LOAD:

200 $LL_{PENT} = 200$ PSF

30 $LL_{ROOF} = 30$ PSF

$A_T = (480.5)$ $TL = 1.2(82) + 1.6(200) = 418.4 = 201.04^k$

$A_T = (124)$ $TL = 1.2(82) + 1.6(30) = \quad = 18.15^k$

$P_{PENT} = 219.9^k$

FOURTH – FIRST:

DEAD LOAD: $\underset{\text{SUPERIMPOSED}}{10} + \underset{\text{STRUCTURE}}{6} + \underset{\text{SLAB/DECK}}{71}$ $DL = 87$ PSF

LIVE LOAD:

$100 \rightarrow 100\left(.25 + \frac{15}{\sqrt{4(480.5)(4)}}\right)$ $LL = 42.11$ PSF

$A_T = 480.5$ $TL = 1.2(87) + 1.6(42.11) = 171.776$

	4	3	2	1
LL(PSF)	59.2	49.19	44.75	42.11
LL (k)	28.445^k	23.636^k	21.502^k	20.233^k
DL (k)	50.16^k	50.16^k	50.16^k	50.16^k
TL (k)	356.16^k	425.15^k	492.55^k	559.13^k

CUMULATIVE LOADS

$P_{ROOF} = 57.66^k$

$P_{PENT} = 277.56^k$

$P_4 = 356.16^k$

$P_3 = 425.15^k$

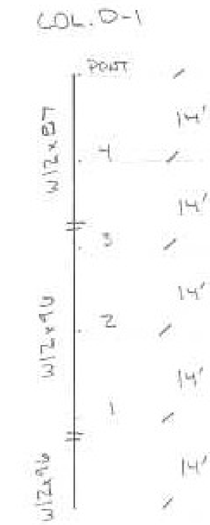
$P_2 = 492.55^k$

$P_1 = 559.13^k$

Appendix G – Gravity – Column Analysis

$P_{PENT + ROOF} = 277.56^k$ ✓ok

W12x87
$\ell = 14'$ $\phi P_n = 924^k$



$P_{FOURTH} = 356.16^k$ ✓ok

W12 x 87
$\ell = 14'$ $\phi P_n = 924^k$

$P_{THIRD} = 425.15^k$ ✓ok

W12x96
$\ell = 14'$ $\phi P_n = 1020^k$

$P_{SECOND} = 492.55^k$ ✓ok

W12x96
$\ell = 14'$ $\phi P_n = 1020^k$

$P_{FIRST} = 554.13^k$

W12x96
$\ell = 14'$ $\phi P_n = 1020^k$ ✓ok