UNIVERSITY HEALTH BUILDING

LOCATED IN THE MID-ATLANTIC REGION

TECHNICAL REPORT I

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ADVISOR - HEATHER SUSTERSIC

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Executive Summary

The objective of the following Technical Report was to obtain an overview of the University Health Building's structural system as well as determine some design forces with preliminary calculations and spot checks. To start, the main elements of the structural system were analyzed to determine how the load gets transferred throughout the building. This was completed by looking at the foundation, slabs, lateral, and roofing systems used in the project. The report includes details about the materials used as well as reference to codes, standards, and loads that were used for the design

Furthermore, the report takes an in-depth look at the design calculations for both wind and seismic later loads to better understand what the load case was controlling when the lateral system was designed. Methods from ASCE 7-05 were used to determine building categories and requirements for these design procedures. The wind analysis produced a base shear of 302k and an overturning moment of 18071ft-k in three different test directions. The seismic analysis produced a base shear of 645k and an overturning moment of 47702ft-k. It was concluded that seismic was the controlling factor for the lateral system design.

Also included in the report are 3 spot checks of an interior column, a continuous drop panel, and steel beam all located throughout the building. These 3 elements were analyzed to see if they meet design load. The column and steel beam were satisfactory, but the continuous drop panel was calculated to be lacking in strength significantly. See the Spot Check section for reasons why this may have happened.

Finally, at the end of the report the reader can find appendices which contain back up calculations for items found in the report.

Building Introduction

This new 9 story 161000 square foot building will be a great addition to the university's campus. It is being built to house leaders in the public and private health policy sectors. The building is a mesh between office space and student classrooms nestled around a central sky lit atrium. The architect hopes that this mesh will help to bridge the gap between faculty and students. The classroom area appears as if the classrooms are floating on clouds in a glass enclosure. The concrete structure is enclosed by a curtain wall which is the building's main feature. The curved saw blade-like curtain wall system encompasses one quarter of the building's façade and gives the building an edgy appearance.

The building façade is constructed of many different types of materials, ranging from stone to metal.

The building's first floor is covered be a stone veneer giving the building a very stereotomic base. The rest of the building is clad in a mixture of glazing, metal panels, and terracotta. The West and Southeast facades are relatively similar to one another. They both have a pattern of terracotta, metal paneling, and glazing above the first floor with the majority material being covered with the terracotta. The south and north facades are also very similar except the south facade has an aluminum sunscreen system in place. Otherwise, these ends of the building are almost fully glazed. Lastly, the curved curtain wall with reveals located on the northeast side of the building is com-

Figure 1: Photo of Northwest corner of building showing façade materials. Rendering by Payette Architecture.

posed of mainly glazing with the reveals clad in terracotta. Some of these features can be seen in Figure 1.

The majority of the roof is a garden roofing system. The system used on this project is the Sika Sarnafil Extensive Greenroof system. It uses 3in. of growing medium as well as pavers for maintenance. The rooftop penthouse will be coved with a fully adhered white, 60mm thick PVC membrane with a layer of 8in. thick tapered polyisocyanurate insulation boards underneath.

Lastly, the University Health Building is registered as a LEED – NC 2.2 Silver building. This rating includes many different LEED credits involving the façade, roof, and internal systems. The main points came from the heat island effect roof system, the building's close location in proximity to transit, and use of efficient plumbing and lighting fixtures.

Structural Overview

Foundation

The foundation of University Health Building (UHB) consists of spread footings at the base of each column. On the western block of the building, the engineers utilized a grade beam and spread footing combination to help with the bracing of the basement wall shown in the Figure 2 below. This was not used on the east side of the building due to the absence of any underground levels. The spread footings are to be set on soils suitable to hold about 5000psf according to the Geo Technical report.
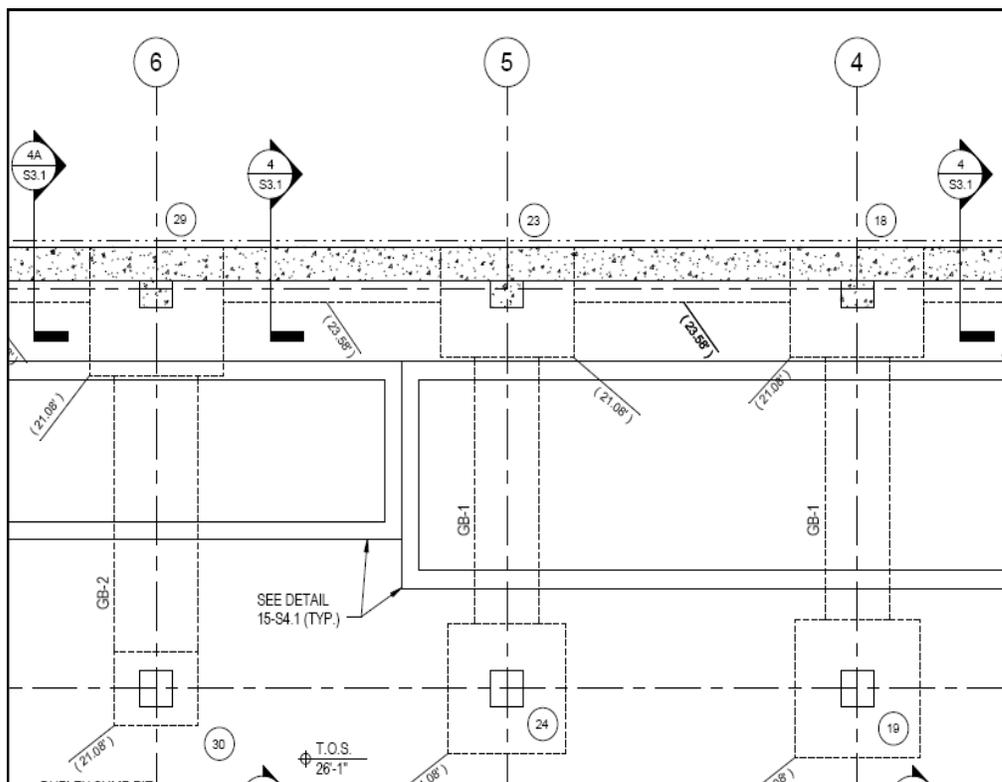

Figure 2: Grade beam and spread footing combination, taken from drawing S1.1

Floor Slabs

The basement level and ground level floor slabs are similar in the fact that they both have a relatively thick floor slab and drop panels comprised of high strength concrete in order to minimize the amount of beams necessary to handle the 21 ft. spans. Once you leave the ground floor, you will find that the slabs change from what was mentioned above to a post tensioning slab system. Also, above the ground floor on the east half of the building, the slabs have large continuous drop panels running between select columns. This type of system extends all the way to the penthouse slab with variations in slab and drop panel thicknesses.

Lateral System

Since the walls of the UHB building are non-load bearing, the lateral loads, due to wind and seismic, must be absorbed by the columns and slabs of the building. The dominant lateral system of the UHB is concrete moment frames consisting of the post-tensioned slab and interior/exterior column system. In the case of wind, the load is transferred from the cladding to the exterior columns and slab edge. Then, its distributed to the interior columns through the slab, and finally, its transferred to the foundation through the columns.

Roof System

The roof system is comprised of two different levels. The first being the lower roof where the green roof is located, and the second is the upper roof that covers the penthouse. The lower roof is a 12-14in. thick post tensioned slab and topped with a green roof system where exposed to the outside. The upper roof is supported by an 8in. post tensioned slab. Also, a portion of the penthouse roof is spanned with steel beams with a glazing system overtop to serve are the skylight for the central stair tower. Figure 3 below shows a partial roof plan showing the integration of the post tensioned con-
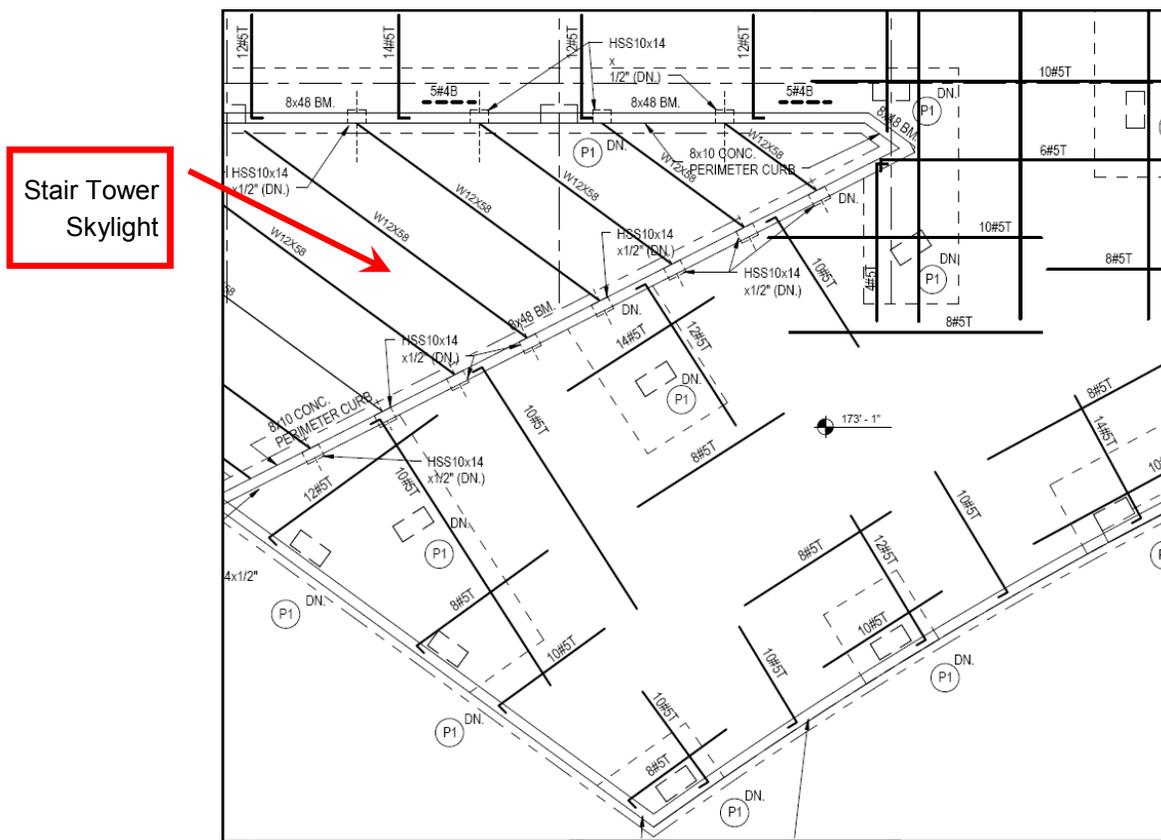

Figure 3: Integrations of both steel and concrete systems on roof, taken from drawing S1.11

Codes & References

Design Codes

    Building Code

        International Building Code - IBC 2006 system

    Reference Codes

        American Society of Civil Engineers - ASCE 7-05

        American Concrete Institute Building Code - ACI 318-05, ACI 530-05, ACI 530.1-05

        American Institute of Steel Construction - AISC 360-05

Thesis Codes

    Building Code

        International Building Code - IBC 2009

    Reference Codes

        American Society of Civil Engineers - ASCE 7-05

        American Concrete Institute Building Code - ACI 318-08

        American Institute of Steel Construction - AISC 14th Edition

Material Strengths

General material strengths were found on S4.9 and are displayed in Figure 5. The general types and strengths can be overwritten per special callouts on the floor plans. On many floors, slab strengths are a combination of 6000psi and 8000psi. See Figure 6 and 7 for good examples of the drawings overwriting the general strengths. The figures show variations in concrete strength as the building elevation increases and slab thickness increases.
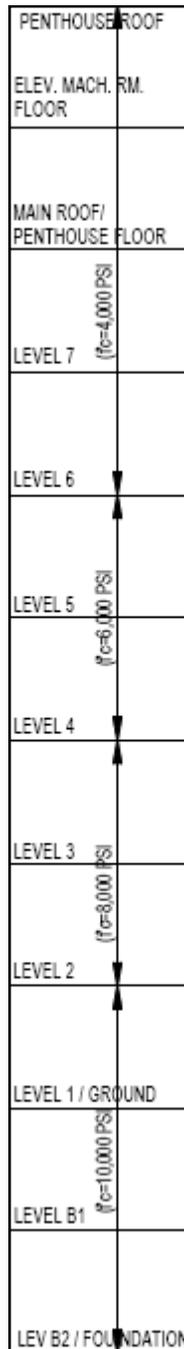
Figure 6: Variations in column concrete strengths per level

| Item	Type	Strength
Steel Beams	ASTM-A992	Fy= 50
Post tensioning Tendons	ASTM A-416	Fu= 270
Reinforcement	ASTM-A615	Fy= 60
Masonry	ASTM C-90	f'c=1.5
Grade Beams	NW Conc.	f'c= 4
Column Footings	NW Conc.	f'c= 5
Slab on grade	NW Conc.	f'c= 5
Floor slabs	NW Conc.	f'c= 6
Columns	NW Conc.	See Fig.

Figure 5: Material strength table
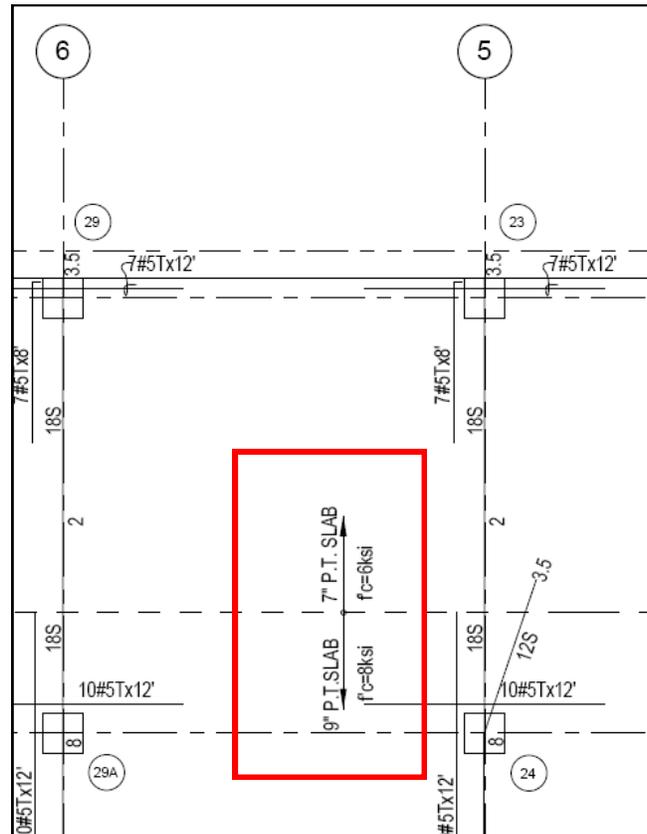
Figure 7: Variations in slab concrete strength

Design Loads

This thesis project will be conducted using the Load and Resistance Factor Design (LRFD) method as it is quickly becoming the industry standard. Thesis loads were determined using ASCE 7-05 unless a category were not listed specifically. Then, design loads were used in its place. At the time this report was written, it was undetermined what the design engineer used for dead loads. See Figure 4 below to see the comparison between design and thesis loads.

Live Loads	(psf)	
	Design	Thesis
Roof	30	20
Mechanical Penthouse	150	150
Green Roof	35	35
Stairways	100	100
Corridors	100	100
Loading Dock	450	450
Light Storage	125	125
Retail	100	100
Office	80	80
Partitions	20	20

Snow	(psf)	
	Design	Thesis
Ground Snow	30	30
Flat Roof	21	21
Snow Exposure Factor	0.7	0.7
Snow Importance Factor	1	1

Dead Load	(psf)	
	Design	Thesis
MEP Allowance	-	5
Roof material	-	5
Green Roof	-	50
	(pcf)	
NW Concrete	150	150

Figure 4: Summary of Live Snow and Dead loads

Lateral Loads

Wind Loading

Design wind loads were determined using the Analytical Procedure from Chapter 6 of ASCE 7-05. It was determined that the building should be designed as a Partially Enclosed building with Exposure Category B. The base shear and overturning moment were calculated to be 302k and 18071ft-k respectively. The base shear was broken down further into a force per story. The per story loading diagram can be seen in Figure 10. To be conservative, when calculating the External Pressure Coefficient (C_p) the Horizontal Distance of the Building parallel to the wind direction (L) was taken from the windward wall to the point on the building furthest from the windward wall.   Also, non-linear walls were estimated as the elevation distance of that portion of the building, known as the Horizontal Distance of the Building, perpendicular to the wind direction (B). These assumptions are demonstrated in Figure 8 below. The results for 3 different winds are shown in the tables in Figure 9 below as well as additional calculations in Appendix A.

Wind Loading Summary

All analyses resulted in the same conclusion due to the assumptions made for distances (L) and (B). These calculations are conservative, so designing for these pressures will be sufficient. Further testing in a wind tunnel could provide more information to help reduce these pressures for a less conservative design.


(L)

(B)
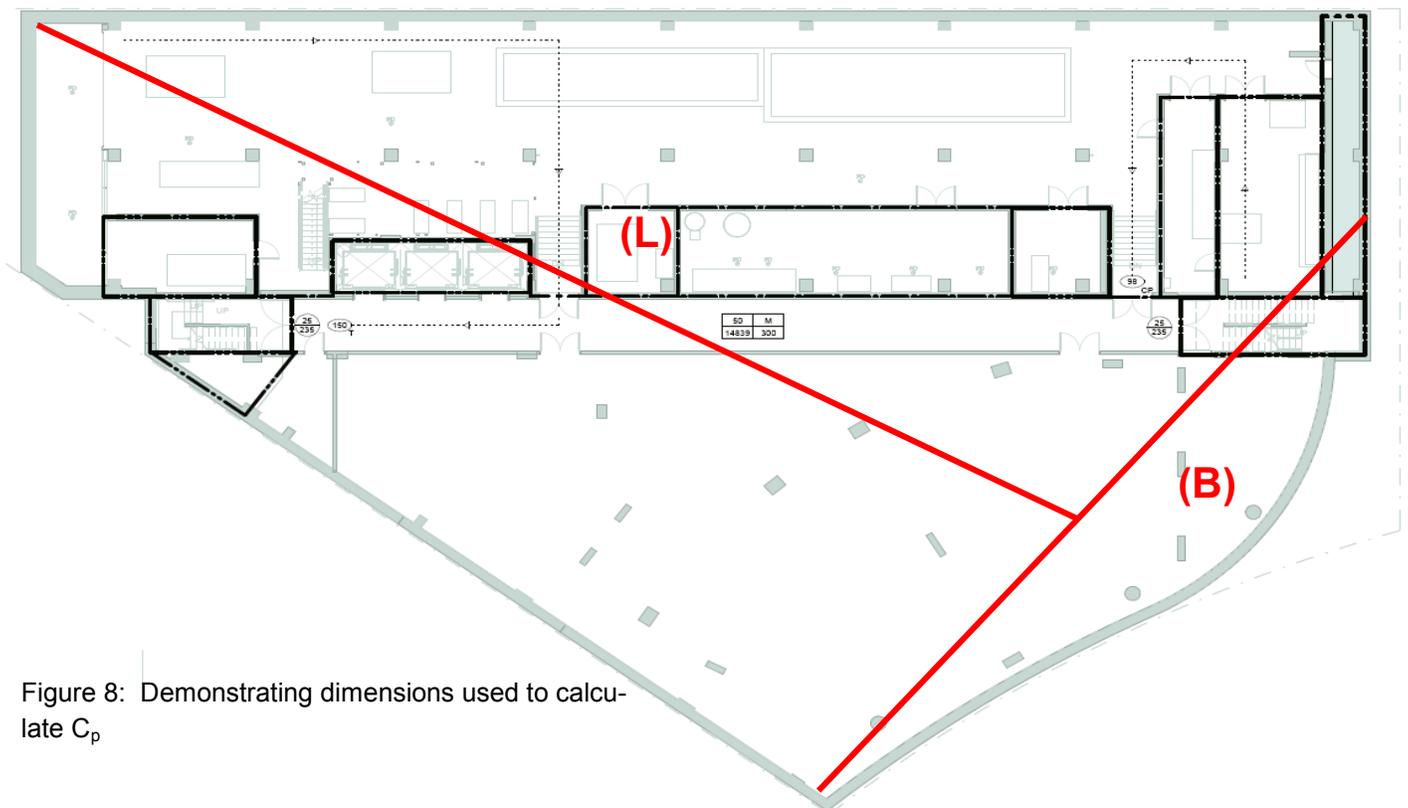
Figure 8: Demonstrating dimensions used to calculate C_p

West Wall							
Story	Height	K_z	q_z	P_w	P_l	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0	0	0	-17.55	17.55	0.00
2	18	0.6	10.58	13.01	-17.55	30.56	550.04
3	30	0.7	12.34	15.18	-17.55	32.73	981.77
4	42	0.77	13.57	16.69	-17.55	34.24	1438.21
5	54	0.83	14.63	17.99	-17.55	35.54	1919.37
6	66	0.87	15.33	18.86	-17.55	36.41	2403.14
7	78	0.92	16.22	19.95	-17.55	37.50	2924.62
Penthouse	91	0.96	16.92	20.81	-17.55	38.36	3490.97
T.O.C. Roof	110	1.02	17.98	22.11	-17.55	39.66	4362.94
					Σ	302.552185	18071.0619

Parameters	
I=	1.00
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.50
K_{zt}=	1.00
K_d=	0.85
Velocity=	90.00
GC_{pi}=	0.55

Southeast Wall							
Story	Height	K_z	q_z	P_w	P_l	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0	0	0	-17.55	17.55	0.00
2	18	0.6	10.58	13.01	-17.55	30.56	550.04
3	30	0.7	12.34	15.18	-17.55	32.73	981.77
4	42	0.77	13.57	16.69	-17.55	34.24	1438.21
5	54	0.83	14.63	17.99	-17.55	35.54	1919.37
6	66	0.87	15.33	18.86	-17.55	36.41	2403.14
7	78	0.92	16.22	19.95	-17.55	37.50	2924.62
Penthouse	91	0.96	16.92	20.81	-17.55	38.36	3490.97
T.O.C. Roof	110	1.02	17.98	22.11	-17.55	39.66	4362.94
					Σ	302.552185	18071.0619

Parameters	
I=	1.00
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.50
K_{zt}=	1.00
K_d=	0.85
Velocity=	90.00
GC_{pi}=	0.55

Northeast Wall							
Story	Height	K_z	q_z	P_w	P_l	Total Story Force (kip)	Overturning Moment (ft-k)
1	0	0	0	0	-17.55	17.55	0.00
2	18	0.6	10.58	13.01	-17.55	30.56	550.04
3	30	0.7	12.34	15.18	-17.55	32.73	981.77
4	42	0.77	13.57	16.69	-17.55	34.24	1438.21
5	54	0.83	14.63	17.99	-17.55	35.54	1919.37
6	66	0.87	15.33	18.86	-17.55	36.41	2403.14
7	78	0.92	16.22	19.95	-17.55	37.50	2924.62
Penthouse	91	0.96	16.92	20.81	-17.55	38.36	3490.97
T.O.C. Roof	110	1.02	17.98	22.11	-17.55	39.66	4362.94
					Σ	302.552185	18071.0619

Parameters	
I=	1.00
G=	0.85
C_p Windward=	0.80
C_p Leeward=	-0.50
K_{zt}=	1.00
K_d=	0.85
Velocity=	90.00
GC_{pi}=	0.55

Figure 9: Windward and Leeward wind force calculation table

22.11k

20.81k

19.95k

18.86k

17.99k   17.55k

16.69k

15.18k

13.01k

18071ft-k

302k
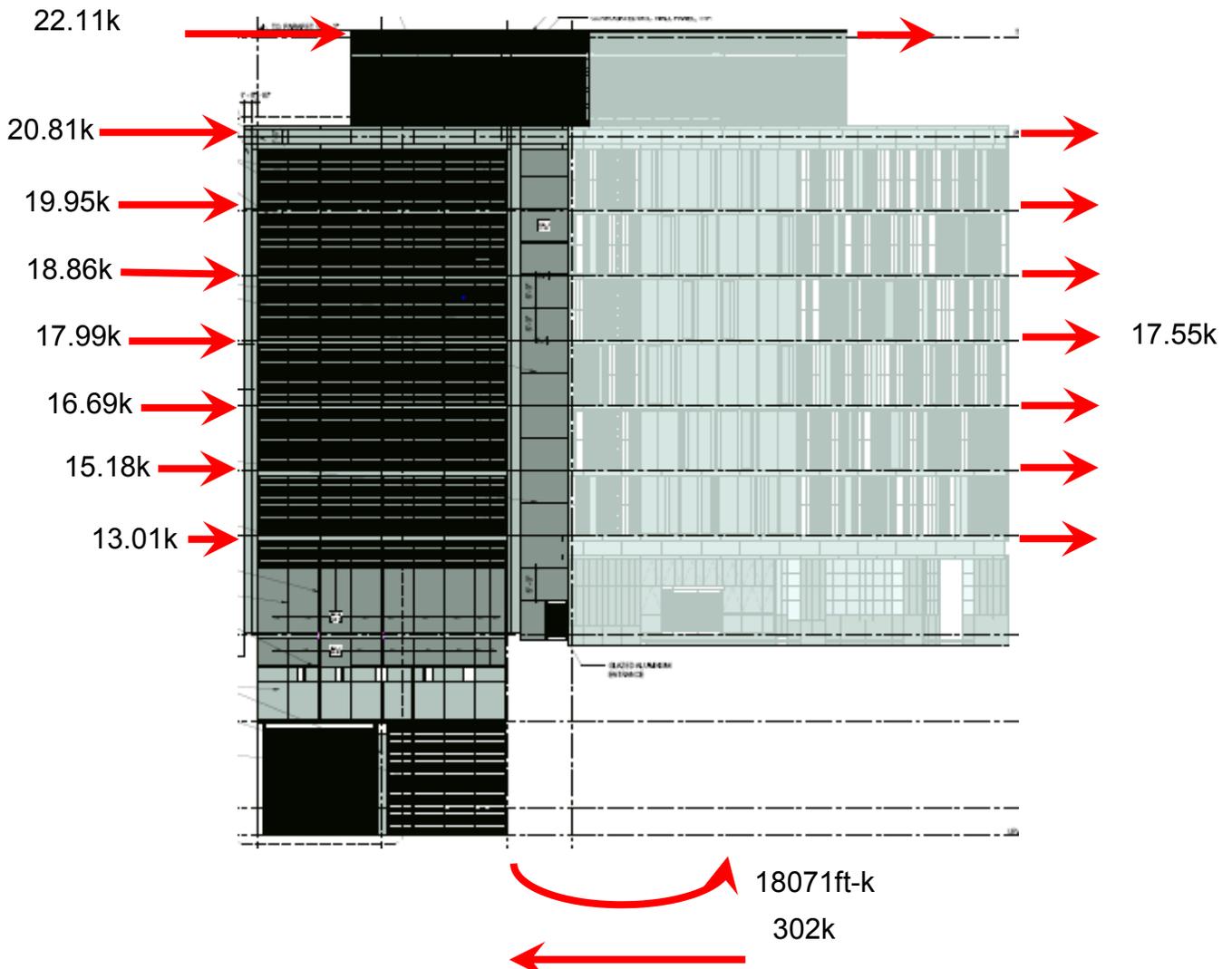
Figure 10: Wind loading diagram

Note: Southeast and Northeast loading diagrams not shown due to similarity to Figure 10 above.

Seismic Loading

Design seismic loads were determined using the Equivalent Lateral Force Procedure from Chapter 12 of ASCE 7-05. Seismic base shear was calculated by first determining building period of vibration and building weight. The base shear was then broken down and shown in Figure 11 on a per floor basis. Figure 12 shows the same per story force in diagram form. The seismic base shear was determined to be 645k and the over turning moment to be 47702ft-k. The large over turning moment is largely due to the thick, heavy green roof and penthouse slab making the building somewhat top heavy. Additional calculations can be found in Appendix B.

Story Forces Calculation								
Story	Height (ft)	W (kips)	k	$\sum w_i h_i$	C_{vx}	V (k)	Total Story Force (k)	Overturning Moment (ft-k)
1	0	0	0	1264056.00	0	645	0.00	0.00
2	18	2430	1.00	1264056.00	0.03	645	22.32	401.74
3	30	2475	1.00	1264056.00	0.06	645	37.89	1136.61
4	42	2483	1.00	1264056.00	0.08	645	53.21	2234.95
5	54	2474	1.00	1264056.00	0.11	645	68.17	3681.13
6	66	2474	1.00	1264056.00	0.13	645	83.32	5498.97
7	78	2638	1.00	1264056.00	0.16	645	104.99	8189.50
Penthouse	91	4196	1.00	1264056.00	0.30	645	194.84	17730.12
T.O.C. Roof	110	1430	1.00	1264056.00	0.12	645	80.26	8829.07
				Σ	1.00		645.00	47702.08

Figure 11: Story Forces Calculation

80.26k

194.84k

105.00k

83.32k

68.17k

53.21k

37.89k

22.32k

47700ft-k

645k
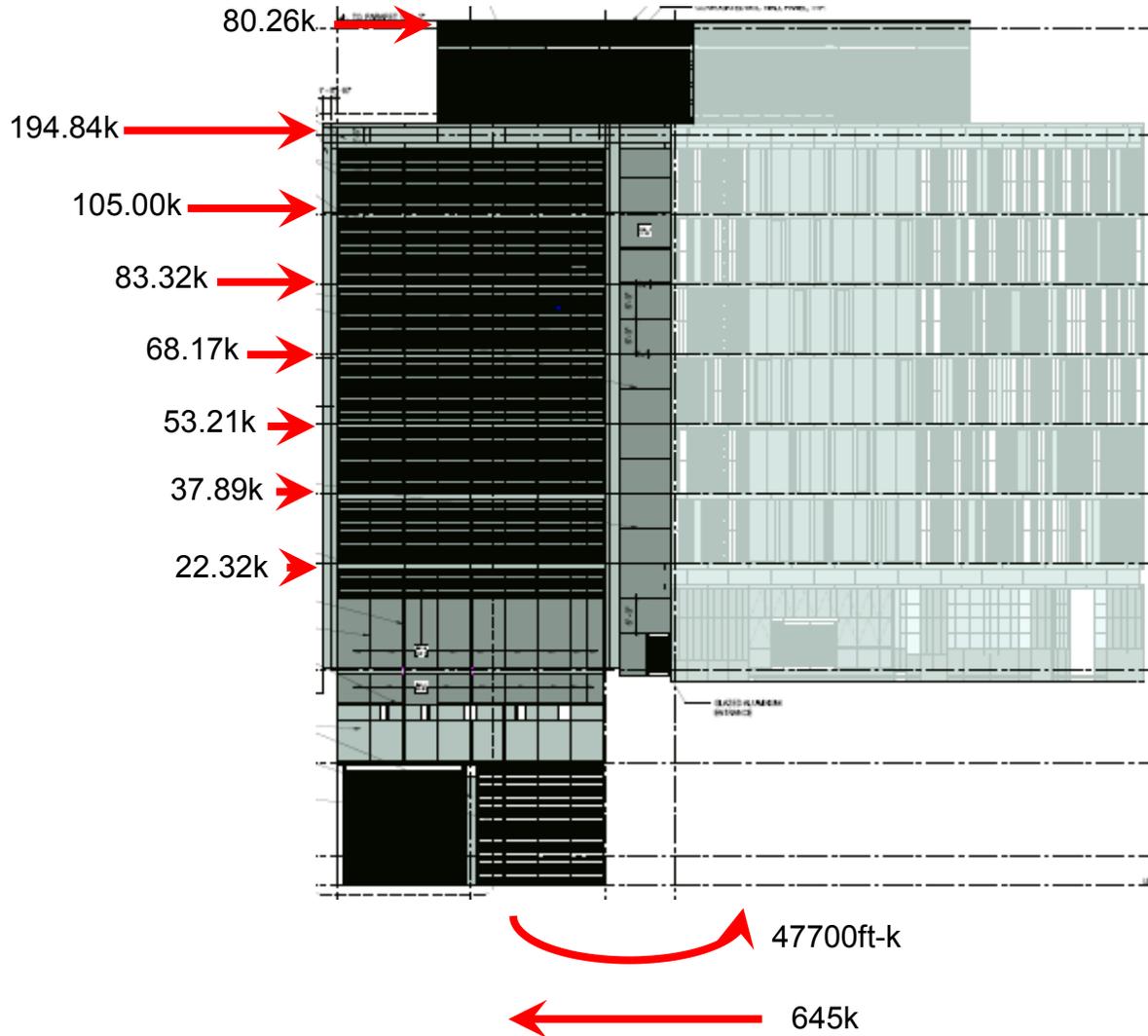
Figure 12: Seismic Loading Diagram

Spot Checks

Interior Column

The interior column #24 is located at the intersection of column lines "B" and "6." The dimensions of this column (24x24) are similar to the other columns found in the UHB building. The reinforcing elements of this column are six #11 rebar spaced evenly in a manner resembling what is shown in Figure 13. Figure 14 shows the column location in the building. This column holds up a tributary area of 456ft^2 and extends from the basement to the penthouse floor. The column was checked at the basement level (B2) where its maximum loading would occur. The loading included the use of live load reduction. It was found that this column has a design strength of 3185k and an actual load of 957k for pure axial loading. The column is over designed for pure axial loading. This conclusion helps to enhance the fact that the building is made up of concrete moment frames. The design of this column is governed by bending induced by lateral forces. See Appendix C for detailed calculations of the column.
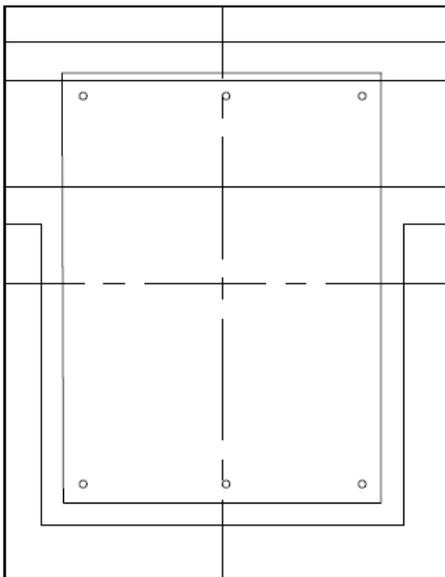

Figure 13: Column detail, taken from S2.2
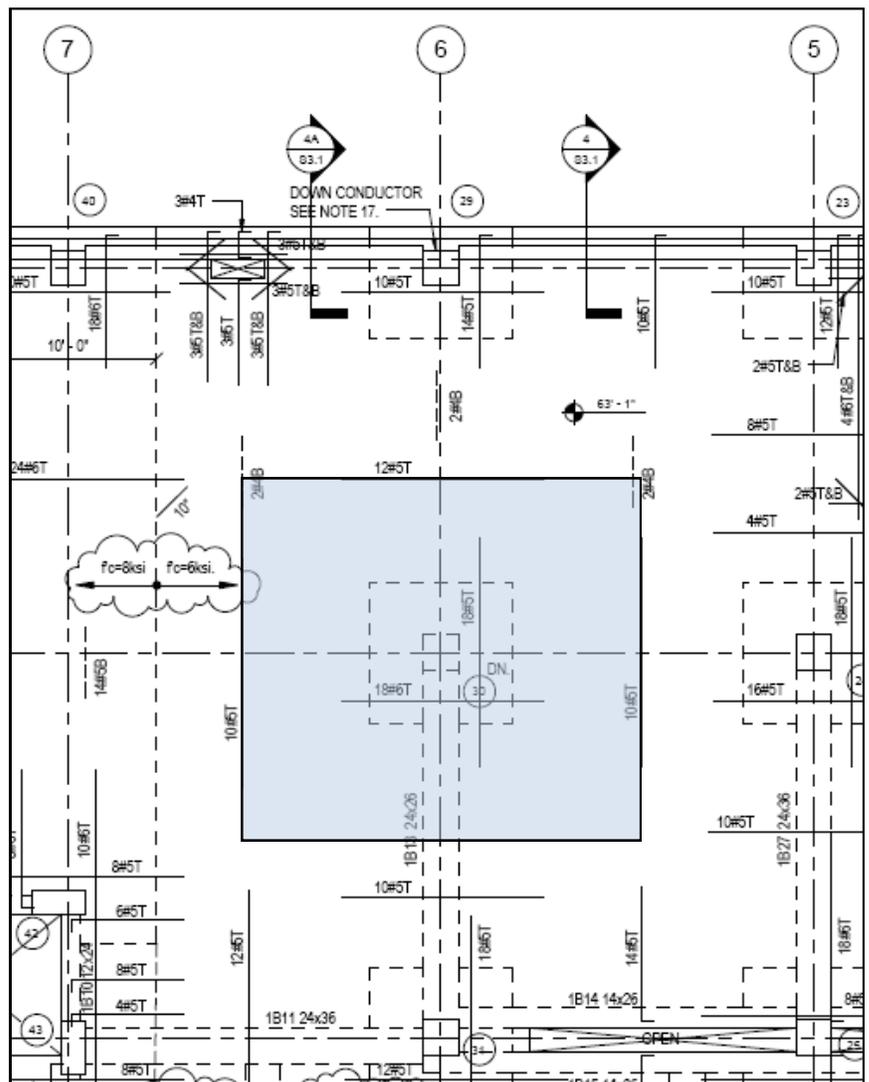
Figure 14: Tributary area, taken from S21.3

Continuous Drop Panel

The analysis approach for this element treated the drop panel as if it were a wide fixed-fixed beam due to its location between two columns with no continuous spans. The beam is located on level 1 in the western half of the building and is shown in Figure 15 carrying a perpendicular tributary length of 21ft. The analysis did not include live load reductions. It was found that the beam was capable of withstanding 549ft-k of negative moment at the east support, 657ft-k of negative moment at the west support, and 368ft-k of positive moment at mid-span. It was found that this beam would fail under the design loading at areas checked. The design load moments were 893ft-k for both ends and 446ft-k for mid-span. This could be due to the assumption that the beam has fixed-fixed connections or that the contribution of the post tensioned slab was not considered. See Appendix D for detailed calculations.
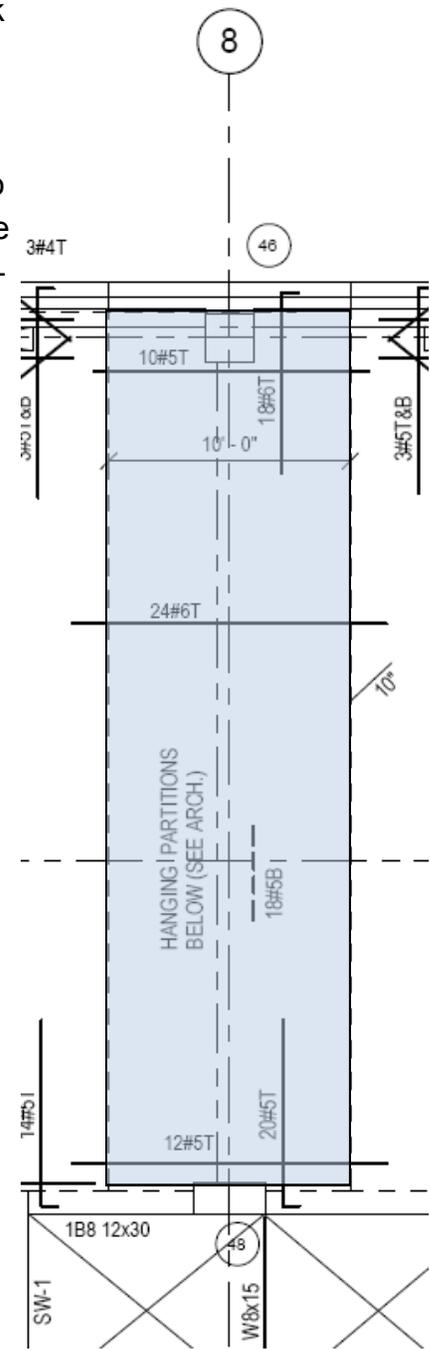
Figure 15: Continuous drop panel, taken from S1.3

Steel Room Beam

A beam from the central stairwell skylight was chosen for a spot check. The member was a W12x58 spanning 18.5ft with a tributary distance of 10ft. A design moment was determined to be 35.6ft-k using dead, live, and snow loads from the roof. The member can be seen in Figure 16 below. With bracing at every 18.5ft, a W12x58 is able to carry a 272ft-k moment, making it largely over designed for this application. Deflections were not checked, but the member choice was concluded to have been determined for ease of construction. By using a W12X58 here there would only need to be one type of W-flange beam used for the skylight. This beam is used for the longer spans in the sky-light as well. See Appendix E for detailed calculations.
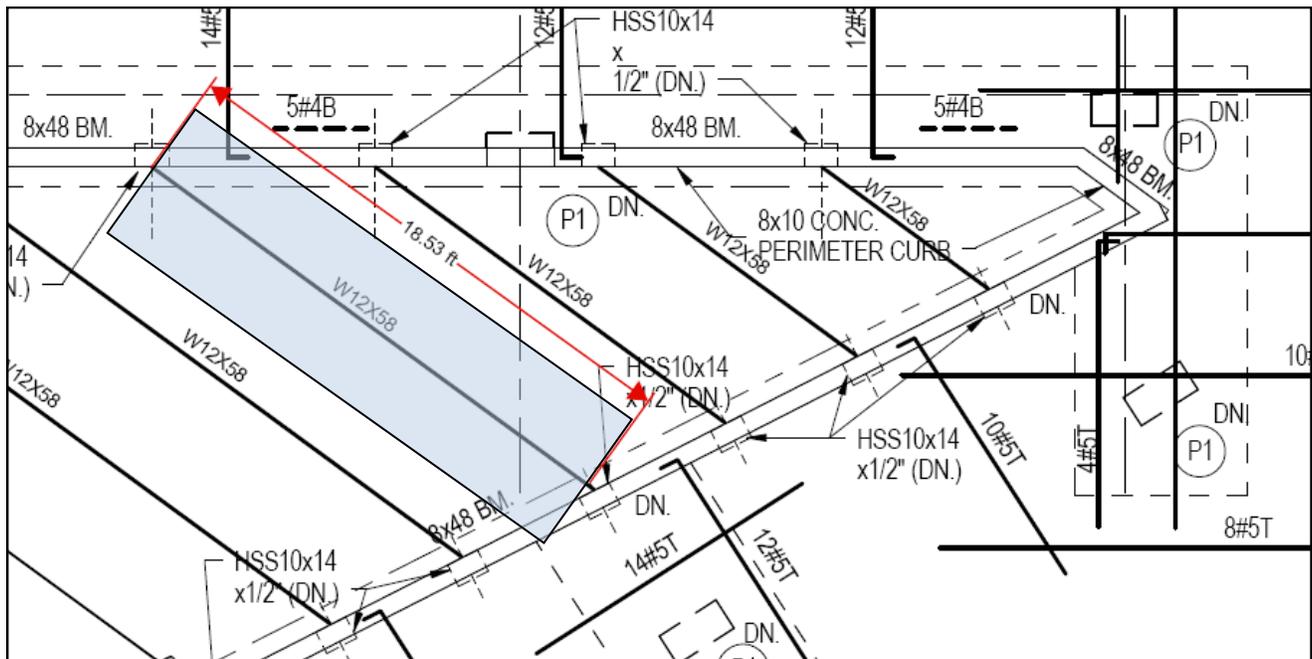
Figure 16: Showing skylight W-flange, taken from S1.11

Conclusion

Technical Report I discussed and analyzed many aspects of the UHB's structural system. A greater understanding of the structural system as a whole has now been obtained. The spot checks determined the current structure to be sufficient, and there is good reason to believe that with more details taken into consideration, the members that failed will be satisfactory.

This report has also brought a better understanding of the materials and strengths of materials used in the structure. Also, the discussion of the design loading for the building will help in future technical reports.

It has also been determined that seismic loads are the dominant lateral load. With almost double the base shear and triple the overturning moment, seismic loads should be the loads in which the lateral system is designed. Upon completing the design, the structure should be checked for both wind and seismic to ensure that nothing was missed. In later technical reports more emphasis will be placed on these lateral loads.

Appendix A

| E. LANDIS | TECH 1 | WIND LOAD |

Wind West Wall

BUILDING OCCUPANCY = III $V = 90$ mph

IMPORTANCE FACTOR = 1.0 $GC_{PI} = +0.55, -0.55$

EXPOSURE CATEGORY = B

$K_z = K_h =$ FOR 110ft $= 1.02$

GUST FACTOR $(G) = 0.85$

ENCLOSURE CLASS = PARTIALLY ENCLOSED

C_p WINDWARD WALL = 0.8   LEEWARD WALL = $L/B = \dfrac{130}{204} = .64$

$K_{zt} = 1.0$ $C_p = -0.5$

$K_d = 0.85$

$q_z = 0.00256 \, K_z \, K_{zt} \, K_d \, V^2 I$

$= 0.00256 (1.02)(1.0)(0.85)(90)^2 (1.0) = 18 \, lb/ft^2$

Design Wind Wind Pressure (MWFRS)

$p = q \, G \, C_p - q_z (GC_{PL})$

$P_w = 18(0.85)(0.8) - 18 (\pm .55) = 22.14 \, psf$

$P_L = 18(0.85)(-0.5) - 18(\pm .55) = -17.55 \, psf$

Southeast Wall

C_p WINDWARD = 0.8   LEEWARD = $L/B = \dfrac{149}{130} = 1.14$

USE $C_p = -0.5$

Northeast Wall

C_p WINDWARD = 0.8   LEEWARD = $L/B = \dfrac{160}{140} = 1.14$

USE $C_p = -0.5$

Appendix B

EVAN LANDIS   EARTHQUAKE   TECH 1

BUILDING INFORMATION

OCCUPANCY = III

S_{ms} 0.243
S_{m_1} = 0.121

SEISMIC IMPORTANCE FACTOR = I = 1.25   (TABLE 1.5-2)

DESIGN SPECTRAL RESPONSE SHORT PERIOD = S_s = 0.15 S_{ds} = 0.16

ls PERIOD = S_1 = 0.050 S_{D1} = 0.08

SEISMIC DESIGN CATEGORY = B

SITE CLASS = D

RESPONSE FACTOR = 3   ORDINARY REIN. CONCRETE MOMENT FRAMES

S_s, S_{ds}, S_1, S_{D1} WERE OBTAINED USING: LAT. 38.8° LONG. -77°

geohazards. usgs.gov/design maps

pg-89

* SEISMIC DESIGN INFORMATION WILL BE CALCULATED USING ASCE 7-05
EQUIVALENT LATERAL FORCE PROCEDURE

* SEISMIC DESIGN CATEGORY → "B"   (TABLE 11.6-1 & 11.6-2)

FUNDAMENTAL PERIOD

$$T = C_T h_N{}^x$$

C_T = 0.016 x = 0.9   ] CONCRETE MOMENT FRAMES
TAKING 100% OF LATERAL LOADS

BUILDING HEIGHT = h = 110 ft

$$T = 0.016(110)^{0.9} = 1.1s$$

$$T \leq T_L$$

$$T_L = 8s   (FIGURE 22-15)$$

E. LANDIS | EARTHQUAKE | TECH 1

SEISMIC RESPONSE COEFFICIENT

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.16}{\left(\frac{3}{1.25}\right)} = .067$$

FOR $T \leq T_L$ $\quad C_S < \frac{S_{D1}}{\left(\frac{R}{I}\right)T}$ $\quad$ & $> .01$

$$= \frac{.08}{\left(\frac{3}{1.25}\right)1.1} = .03 \quad \checkmark \quad \underline{USE \; .03}$$

BUILDING WEIGHT

2ND FLOOR:

7IN SLAB = 12500 SF   8IN SLAB: 2260 SF

SLAB WEIGHT = $150\left(12500\left(\frac{7}{12}\right) + 2260\left(\frac{8}{12}\right)\right) = 1319750 \; lbs$

10" DROP PANEL BEAMS = 2218

$\qquad\qquad 150\left(2218\left(\frac{10}{12}\right)\right) = 277250 \; lbs$

MEP ALLOWANCE: $\left(5 \frac{lb}{SF}\right)\left(12500 + 2260\right) = 73800 \; lbs$

PARTITIONS: $\left(20 \frac{lb}{SF}\right)\left(12500 + 2260\right) = 295200 \; lbs$

$\qquad$ 2ND FLOOR TOTAL = 1,966,000 lbs

3RD FLOOR:

7IN SLAB: 13600 SF   9IN SLAB: 1600 SF.

10" DROP PANEL BEAMS: 1170 SF

8" DROP PANEL BEAMS: 1060 SF

SLAB/BEAM WEIGHT = $150\left(13600\left(\frac{7}{12}\right) + 1600\left(\frac{9}{12}\right) + 1170\left(\frac{10}{12}\right) + 1060\left(\frac{8}{12}\right)\right)$

$\qquad\qquad = 1631450 \; lbs$

E. LANDIS	EARTHQUAKE	TECH 1

MEP ALLOWANCE: $(5\ ^{lb}/_{SF})(13600 + 1600) = 76000\ lbs$

PARTITIONS: $(20\ ^{lb}/_{SF})(13600 + 1600) = 304000\ lbs$

TOTAL 3RD FLOOR WEIGHT = $2011450\ lbs$

4TH FLOOR:

     7 IN SLAB = 13000 SF   9 IN SLAB: 2200 SF

     10 IN DROP = 1100 SF   8 IN DROP: 1040 SF

     MEP ALLOWANCE: $(5\ ^{lb}/_{SF})(13000 + 2200) = 76000\ lbs$

     SLAB WEIGHT = $150(3000(^{7}/_{12}) + 2200(^{9}/_{12}) + 1100(^{10}/_{12}) + 1040(^{8}/_{12}))$

               = $1639150\ lbs$

     PARTITIONS = $(20\ ^{lb}/_{SF})(13000 + 2200) = 304000\ lbs$

         TOTAL 4TH FLOOR WEIGHT = $2019150\ lbs$

5TH FLOOR: COMPARABLE TO FLOORS 3 & 4

         TOTAL = $2010000\ lbs$

6TH FLOOR: COMPARABLE TO FLOOR 5

         TOTAL = $2010000\ lbs$

7TH FLOOR:

     7" SLAB = 8800 SF   8.5" SLAB = 3600 SF

     10" SLAB = 2700 SF   8" DROP = 1500 SF

     9.5" DROP = 1300 SF

         SLAB WEIGHT = $1794000\ lb$ TOTAL = $2174375\ lb$

         MEP ALLOWANCE = $76000\ lb$ (TYP)

         PARTITIONS = $304000\ lb$ (TYP)

E. LANDIS | EARTHQUAKE | TECH 1

8TH FLOOR / LOWER ROOF:

14in SLAB = 2000 SF   12" SLAB = 13200 SF

10in DROP = 2450 SF

SLAB WEIGHT = 2636000 lb

MEP = $50 \, lb/SF \, (13200 + 2000) = 750000 \, lb$

GREEN ROOF = $50 \, lb/SF \, (6000) = 300,000 \, lb$

SNOW = $30 \, lb/SF \, (6000) = 180,000 \, lb$

TOTAL = 3866000 lb

PENTHOUSE ROOF:

'8 SLAB = 9500 SF   8" DROP = 1000 SF

SLAB = 1050000 lb

MEP = $10 \, lb/SF \, (9500) = 95,000 \, lb$

SNOW = $30 \, lb/SF \, (9500) = 285000 \, lb$

TOTAL = 1430000 lb

BUILDING ENVELOP:

$(15 \, lb/SF)(570ft \times 110 \, ft) = 940500 \, lbs$

→ PERIMETER

→ BUILDING HEIGHT

COLUMNS:

40 COLUMNS w/ $\approx 576 \, IN^2$

h = 110 ft

WEIGHT = $110 \, ft \times \frac{576}{144} \times 150 \times 40 = 2640000 \, lb$

$W \approx 21,500 \, kips$

E. LANDIS	EARTHQUAR	TECH 1

BASE SHEAR

$$V = C_S W$$

$$V = .03 (21,500) = \quad 645^k$$

BASE SHEAD DISTRIBUTION

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^{\,k}}{\Sigma w_i h_i^{\,k}}$$

$$T = .03 \quad < .5 \quad \therefore \quad K = 1$$

Appendix C

E. Landis   Tech 1   Column Spot Check

Column at intersecting lines Ⓑ & ⑤

Basement Level B1   Col #25

21'

21'8.5"

21'8.5"

□

21'
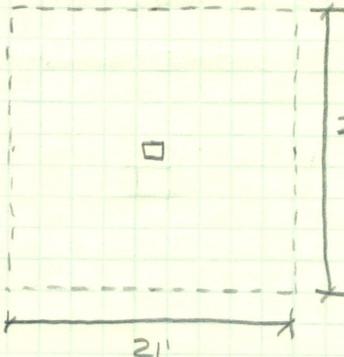
Trib area constant for all floors

= 456 SF

__Loading__

Floor B1: D_L = 8" Sub = $150 \times (456 \times {}^{8}\!/_{12})$ = 45.6^k

$3\frac{1}{2}$" Drop = $150 \times (8 \times 8 \times {}^{3\frac{1}{2}}\!/_{12})$ = 2.8^k

24×26 Beams = $150 \times \left(\frac{24}{12} \times \frac{18}{12} \times 21\right)$ = 9.6^k $\boxed{D_L = 65.8^k}$

14×26 Beams = $150 \times \left(\frac{14}{12} \times \frac{18}{12} \times 21\right)$ = 5.5^k

SDL = 5 psf (456) = 2.3^k

LL = 80 psf (456) = $\boxed{36.5^k}$

Floor 1: D_L = 8' Slab = $150 (456 \times {}^{8}\!/_{12})$ = 45.6^k

$3\frac{1}{2}$" Drop = 2.8^k

24×36 Beam = $150 \left({}^{24}\!/_{12} \times {}^{28}\!/_{12} \times 10.5\right)$ = 7.4^k $\boxed{D_L = 58.1^k}$

SDL = 2.3^k

$\boxed{LL = 36.5^k}$

E. LANDIS	TECH 1	COLUMN SPOT CHECK

FLOOR 2:   DL = 7" SLAB = $150 \times (456 \times 7/12)$ = 39.9^k

SDL = 2.3^k $\boxed{DL = 42.2}$

$\boxed{LL = 36.5^k}$

FLOORS 3, 4, 5, 6   SAME AS 2

FLOOR 7:   SAME + 8" DROP DL = 42.2 + $150(15.5 \times 7 \times 8/12)$ = $\boxed{53^k = DL}$

LL = 36.5^k

PENTHOUS FLOOR: DL: 12" SLAB = $150(456 \times 12/12)$ = 68.4^k

10" DROP = $150(15.5 \times 9 \times 10/12)$ = 17.4^k

GREEN ROOF = $50psf(6 \times 21)$ = 6.3^k $\boxed{DL = 95.9^k}$

SNOW = $30psf(6 \times 21)$ = 3.8^k

LL: MECH ROOM = $150psf(15 \times 21)$ = 47.2^k

GREEN ROOF LL = $50(6 \times 21)$ = 6.3^k

$\boxed{LL = 53.5^k}$

PENTHOUSE ROOF: DL = 8" SLAB = $150(10.5 \times 21 \times 8/12)$ = 22.1^k

ROOFING MAT = $5psf(10.5 \times 21)$ = 1.1^k

SNOW = $30psf(10.5 \times 21)$ = 6.6^k

$\boxed{DL = 29.8^k}$

10.5 ft

21 ft
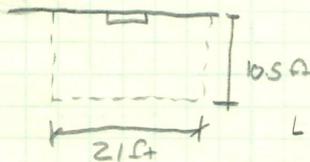
LL = $20psf(10.5 \times 21)$ = $\boxed{4.4^k}$

LIVE LOAD REDUCTION

$K_{LL}A_T = (4)(456) = 1824 > 400$ ✓

$L = L_0\left(0.25 + \dfrac{15}{\sqrt{K_{LL}A_T}}\right) = 349 \cdot k\left(0.25 + \dfrac{15}{\sqrt{1824}}\right) = \boxed{209^k} \geq .5(349)$

↳ EXCLUDES ROOF LL   = 174^k

E. Landis   Tech 1   Column Spot check

Load on column = $1.6(209.8^k) + 1.2(513.6) + .5(10.4)$

2.57" 9.4" 9.4" 2.57"   = 957^k
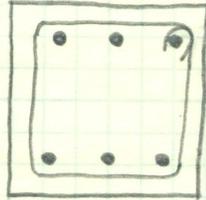
$f'_c = 10,000 psi$

24" x 24"

(6) #11's   #3 ties

1.5"

Pure Axial

$\phi P_o = \phi\, 0.85 f'c (A_c) + A_s f_y$

$= (0.65)0.85(10)(576) + (1.56/144)(6)(60) = 3185^k$

Appendix D

SPAN = 35ft

18'   d'   d

10ft

LOADING: LL = 100psf (21ft) = 2.1^k

DL = 8" SLAB = 150 (21ft × $^8/_{12}$) = 2.1^k

10" DROP = 150 (10 × $^{10}/_{12}$) = 1.3^k

SPL = 5 (21) = 1.1^k

ω = 1.6(2.1) + 1.2(4.5) = 8.76 plf

$M_{ENDS} = \dfrac{8.75(35)^2}{12} = 893 \, k \cdot ft$

$M_{MD} = \dfrac{8.75(35)^2}{24} = 446 \, k \cdot ft$

TOP REIN EAST END = 20 #5's + 11 #4's AT d'

BOTTOM REIN MID = 18 #5's + 11 #4's TOP REIN

TOP REIN WEST END = 18 #6's + 11 #4's AT d'

EAST END

d = 16.5   FOR #5's   d = 11.5" FOR #4's

A_s = 6.2 in² A_s = 2.2 in²
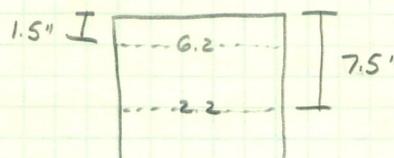
1.5" ┆--- 6.2 ---┆ 7.5" $\dfrac{1.5(6.2) + 7.5(2.2)}{6.2 + 2.2}$ = 3.07"
     ┆--- 2.2 ---┆

d = 15"

E. LANDIS | TECH 1 | DROP PANEL CHECK

$$a = \frac{A_s f_y}{0.85(f'_c)(b)} = \frac{8.4(60,000)}{0.85(8000)(10 \times 12)} = .617$$

$$c = \frac{a}{\beta_1} = \frac{.617}{.65} = ..95$$

$$\varepsilon_s > \varepsilon_y \qquad \varepsilon_s = \frac{0.003}{.95}(15 - .95) = .044 > \varepsilon_y \checkmark$$

$$\emptyset = 0.9$$

$$\emptyset M_n = \emptyset A_s f_y (d - \frac{a}{2}) = 0.9(8.4)(60,000)(15 - \frac{.95}{2}) = 6588 \text{ k} \cdot \text{in}$$

$$= 549 \text{ k} \cdot \text{ft}$$

MIDSPAN

$$A_s = .31 \times 18 = 5.58 \text{ in}^2$$

$$\text{Assume } F_s > f_y \qquad a = \frac{5.58(60,000)}{0.85(8000)(10 \times 12)} = .41$$

$$c = \frac{.41}{.65} = .631"$$

$$\varepsilon_t = .003 \frac{16.5 - .631}{.631} = .075 \longrightarrow \varepsilon_y$$

COMP STEEL NOT NEEDED IN ANALYSIS

$$\emptyset M_n = 0.9(5.58)(60)(15 - \frac{.631}{2}) = 4424 \text{ k} \cdot \text{in}$$

$$= 368 \text{ k} \cdot \text{ft}$$

WEST END

$$A_s = 18(.44) + 11(.20) = 10.12$$

$$a = \frac{10.12(60,000)}{0.85(8000)(10 \times 12)} = .744 \qquad c = \frac{.744}{.65} = 1.14$$

$$\varepsilon_s = \frac{.003}{1.14}(15 - 1.14) = .036 > \varepsilon_y \checkmark$$

$$\emptyset M_n = 0.9(10.12)(60)(15 - \frac{1.14}{2}) = 7885 \text{ k} \cdot \text{in} = 657 \text{ k} \cdot \text{ft}$$

Appendix E

E. LANDIS   TECH 1   STEEL BM SPOT CHECK

SKY LIGHT FRAMING CHECK

W12×58 $l = 18.5ft$ $T_{RIB} = 10ft$

LL: 20psf   DL: 8psf →

SNOW: 30psf

$\omega = 1.2(10(8) + 58) + 1.6(10)(20) + .5(10)(30) = 833\ plf$

$M_u = \dfrac{833(18.5)^2}{8} = 35.6\ k\cdot ft$

ϕM_n @ 18.5 ft bracing = 272 k·ft ✓ OK