Letter of Transmittal

October 14, 2016

Dr. Aly Said
The Pennsylvania State University
209 Engineering Unit A
University Park, PA 16802
aly.said@engr.psu.edu

Dear Dr. Said,

The attached document contains a detailed analysis of the gravity system for the Brendan Iribe Center for Computer Science and Innovation in College Park, MD.

This report includes spot checks for the gravity loads determined from Notebook Submission A. Three alternative systems were designed to determine which systems are viable options to use moving forward.

Thank you for taking time to review this technical report. I look forward to your feedback and discussing where to go from here.

Best Regards,

Brendan Barrett

NOTEBOOK SUBMISSION B

THE BRENDAN IRIBE CENTER FOR COMPUTER SCIENCE AND INNOVATION

COLLEGE PARK, MD

Brendan Barrett

Structural Option

Advisor: Dr. Said

Executive Summary

As one of the world's top computer science institutions, the University of Maryland continues to grow. There is no longer enough room in the existing facilities to keep up with the latest advancements in virtual reality. The Brendan Iribe Center for Computer Science and Innovation will help separate the University of Maryland from its competitors.

Six stories of collaborative classrooms, research labs, seminar rooms, offices, and many common areas will welcome students and faculty alike. A 300-seat auditorium will provide the University of Maryland an opportunity to showcase its latest research such as cybersecurity, computational biology, and quantum computing. The open floor plans will help promote collaborating amongst peers, and ultimately set these students up for successful careers.

Structurally, the Brendan Iribe Center for Computer Science and Innovation utilizes steel wide flange girders and columns to support gravity loads. The curvilinear shape of the building results in unequal bays as infill beams change as the shape of the building changes. Due to the irregular shape, there are several unique components of this system such as curved HSS beams along the southern wall. The 300- seat Antonov Auditorium utilizes wide flange girders and columns, as well as a 90' truss to support the different levels and roof.

From a lateral standpoint, the Brendan Iribe Center for Computer Science and Innovation uses ordinary moment frames and vertical trusses throughout each wing of the building and the auditorium. All loads are in accordance with the 2015 International Building Code and ASCE 7-10.

This report will provide gravity and lateral calculations which will be used for further analysis of the building.

Contents

1. General Information

1.1 Site Plan

The Brendan Iribe Center for Computer Science and Innovation is located at the eastern part of campus at the intersection of Baltimore Pike and Campus Drive.

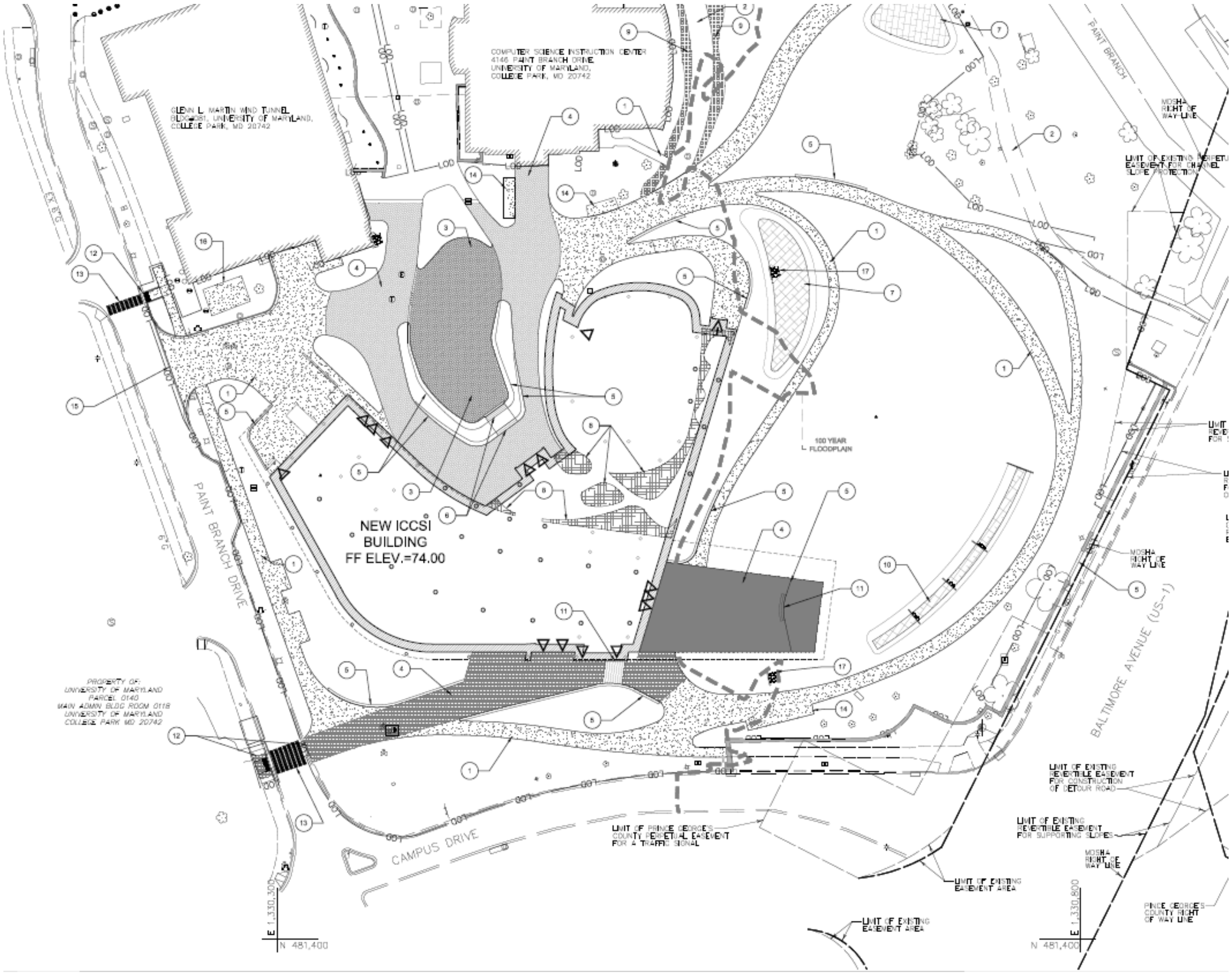


Figure 1: Site Plan

1.2 Documents used in Preparation of Report

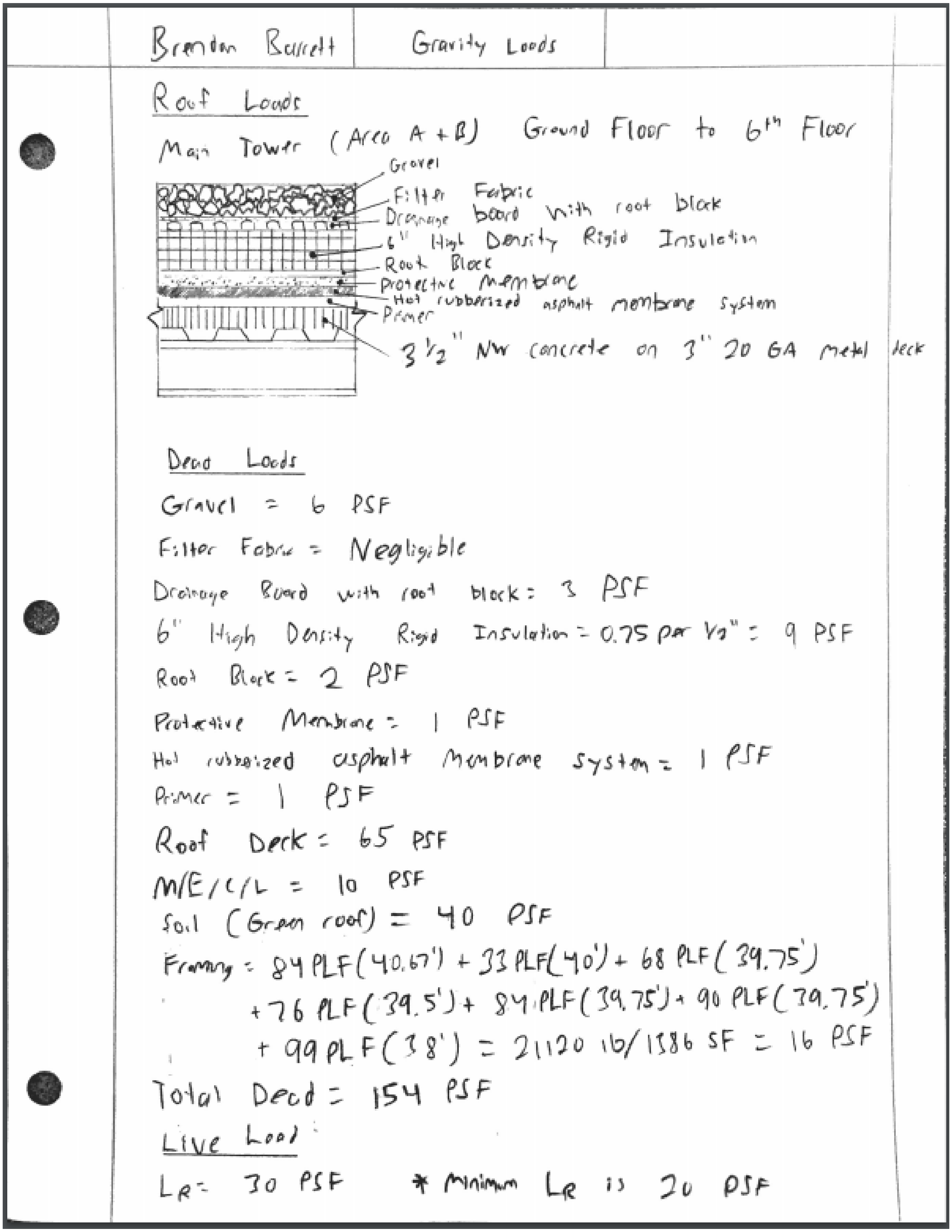
The following is a list of codes, standards, and other references that were used for calculations throughout this report.

- Brendan Iribe Center for Computer Science and Innovation
    - Structural Drawings
- International Code Council
    - 2015 International Building Code
- American Society of Civil Engineers
    - ASCE 7-10: Minimum Design Loads for Buildings and Other Structures

2. Gravity Loads

2.1 Roof Loads

See Appendix A to view bay used in determination of gravity loads



Brendan Barrett | Gravity Loads

Roof Loads

Main Tower (Area A + B) Ground Floor to 6th Floor

Gravel
Filter Fabric
Drainage board with root block
6" High Density Rigid Insulation
Root Block
Protective Membrane
Hot rubberized asphalt membrane system
Primer
3 1/2" NW concrete on 3" 20 GA metal deck

Dead Loads

Gravel = 6 PSF

Filter Fabric = Negligible

Drainage Board with root block = 3 PSF

6" High Density Rigid Insulation = 0.75 per 1/2" = 9 PSF

Root Block = 2 PSF

Protective Membrane = 1 PSF

Hot rubberized asphalt Membrane system = 1 PSF

Primer = 1 PSF

Roof Deck = 65 PSF

M/E/C/L = 10 PSF

Soil (Green roof) = 40 PSF

Framing = 84 PLF(40.67') + 33 PLF(40') + 68 PLF(39.75')
+ 76 PLF(39.5') + 84 PLF(39.75') + 90 PLF(39.75')
+ 99 PLF(38') = 21120 lb/1386 SF = 16 PSF

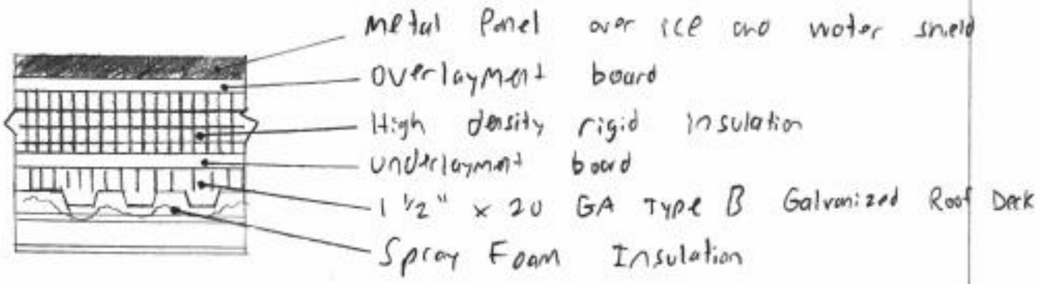
Total Dead = 154 PSF

Live Load

L_R = 30 PSF * Minimum L_R is 20 PSF

Brendan Barrett | Gravity Loads

Auditorium (Area C)



Dead Loads

Metal Panel over ice and water shield = 1 PSF

Overlayment board = 0.75 PSF

High Density Rigid Insulation = 9 PSF

Underlayment board = 0.75 PSF

Roof Deck = 2 PSF

Spray Foam Insulation = 1 PSF

M/E/C/L = 10 PSF

Framing = 22 PLF (32')(3) + 30 PLF (32') + 26 PLF (32')
+ 19 PLF (16.5') + 120 PLF (16.5')
= 6200 lb / 530 SF = 12 PSF

Total Dead = 36.5 PSF

Live Load

L_R = 30 PSF * Minimum L_R is 20 PSF

2.2 Snow Loads

Brendan Barrett Gravity Loads

Snow Loads

Ground Snow load $p_g = 35$ PSF (Figure 7-1)

$p_f = 0.7 C_e C_t I_s p_g$

$C_e = 0.9$ (Terrain Cat B, Fully exposed)

$C_t = 1.0$ (All Structures)

$I_s = 1.1$ (Risk Category III)

$p_f = 0.7(0.9)(1.0)(1.1)(35)$

$= 24.26$ PSF + Unbalanced, drifting, and sliding

Drift at rooftop garden :

- Leeward drift → $\ell_u = 265'$

$h_d = 0.43 \sqrt[3]{\ell_u} \sqrt[4]{p_g + 10} - 1.5$

$= 0.43 \sqrt[3]{265} \sqrt[4]{35 + 10} - 1.5$

$= 5.66$ ft

$\gamma = 0.13 p_g + 14$

$= 0.13(35) + 14$

$= 18.6$ pcf

$h_b = 24.26 \text{ psf} / 18.6 \text{ pcf} = 1.3'$ ⇒ flat roof height

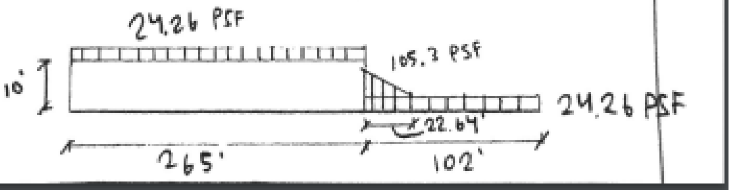
$h_c = 10' - 1.3' = 8.7'$ $\frac{h_c}{h_b} = \frac{8.7}{1.3} = 6.7 > 0.2 \therefore$ drift

$h_d < h_c \rightarrow w = 4 h_d = 4(5.66) = 22.64'$

$p_d = h_d \gamma$

$= 5.66(18.6)$

$= 105.3$ PSF



Brendan Barrett | Gravity Loads

Drift from Tower onto auditorium:

Leeward drift -> $\ell_u = 58'$

$h_d = 0.43\sqrt[3]{58}\sqrt[4]{35+10} - 1.5$

$= 2.81$ ft.

$\gamma = 18.6$ pcf

$h_b = 1.3'$

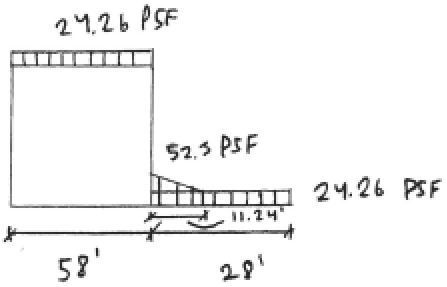
$h_c = 68' - 1.3' = 66.7'$

$h_d < h_c \rightarrow w = 4h_d = 4(2.81) = 11.24'$

$p_d = h_d \gamma$

$= (2.81)(18.6)$

$= 52.3$ PSF

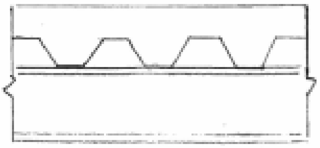


2.3 Floor Loads

Brendan Barrett Gravity Loads

Floor Loads

Typical Floor (Ground Floor to 6th Floor)



3 1/4" LW Concrete
3" 20 GA Galvanized deck

Dead Load:

Beams = 84 PLF(43.167') + 76 PLF(42.75') + 68 PLF(42.67') + 76 PLF(42.58') + 76 PLF(42.67') = 16256

Girders = 116 PLF(38.33') + 84 PLF(31.58')
= 7100 lbs

Framing = $\frac{16256 + 7100 \text{ lbs}}{1600 \text{ SF}}$ = 14.6 PSF = 15 PSF

Slab = 46 PSF

Metal Deck = 2 PSF

M/E/C/L = 10 PSF

Total Dead = 73 PSF

Live Load: (Table 4-1)

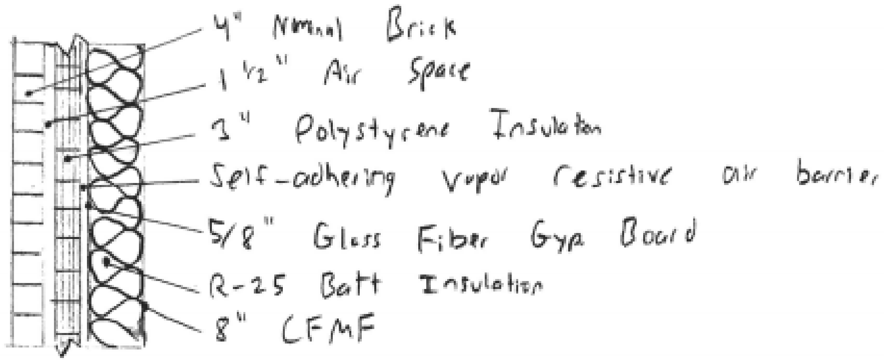
L_o = 100 PSF (Corridors)

* Minimum L_R is 100 PSF

2.4 Perimeter Loads

Brendan Barrett Gravity Loads

Exterior Wall at Auditorium



Dead Load:

Steel Manual 4" Brick = 40 PSF

ASCE 7-10 3" Polystyrene Insulation = 0.2 PSF/1" = 0.6 PSF

ASCE 7-10 5/8" Glass Fiber Gypsum Board = 0.55 PSF/1/8" = 0.55(5) = 2.75 PSF

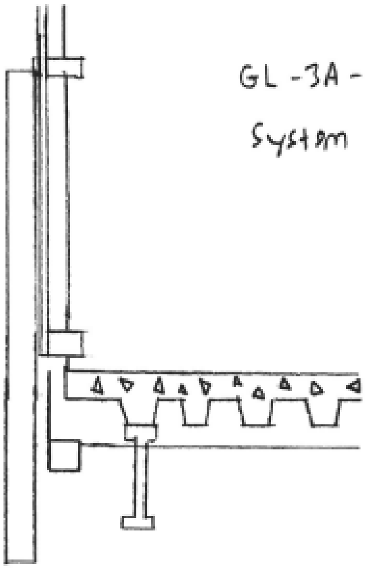
R-25 Batt Insulation = 0.04 PSF/1" = 0.04(8) = 0.32 PSF

Steel Manual 8" CFMF = 1 PSF

Total = 45 PSF × 29'-10 3/4" = 1345 PLF

Brendan Barrett | Gravity Loads

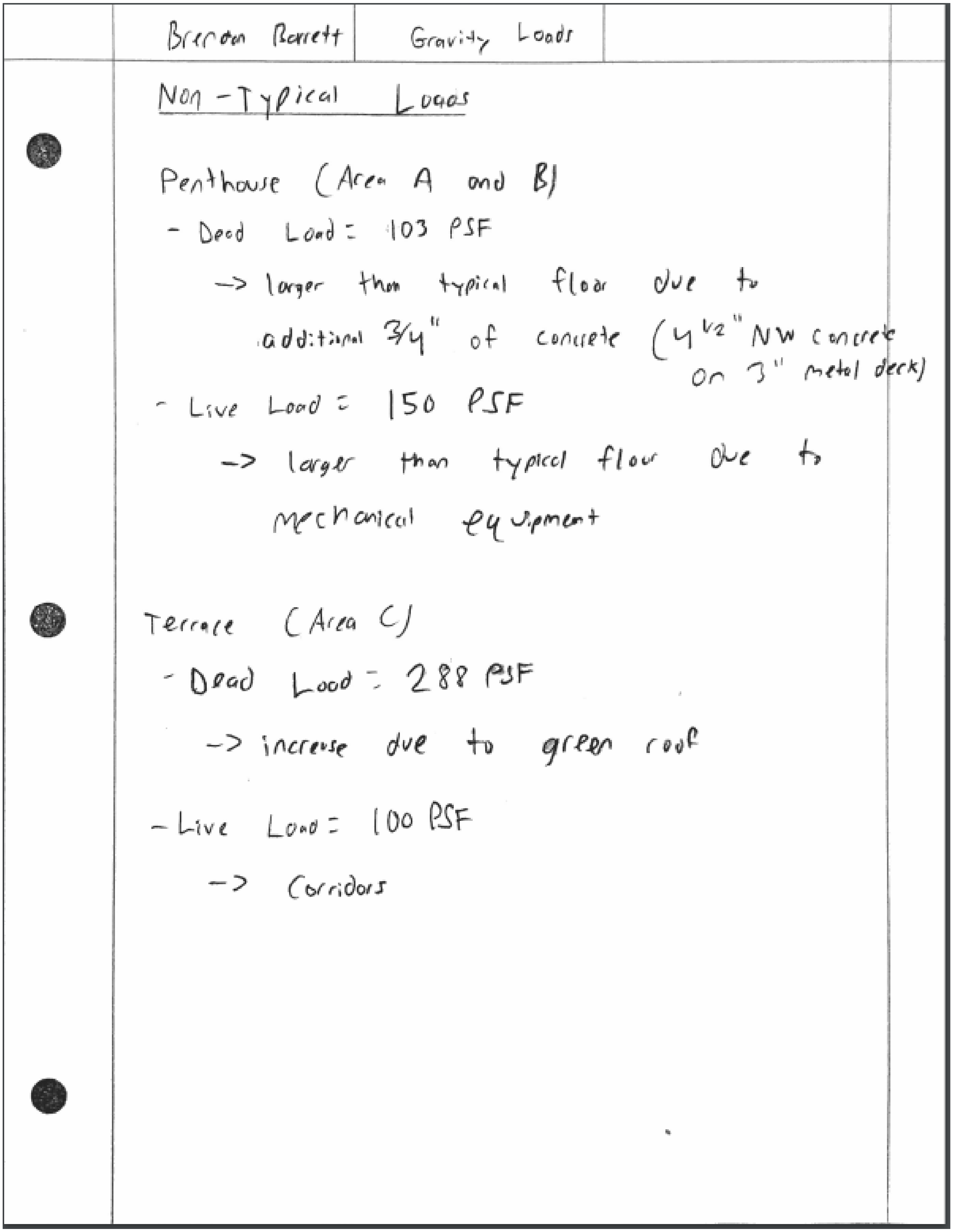
Exterior Wall at North Facade



GL-3A - Monolithic Glass Fins in Curtain Wall System w/ Frit Pattern

Dead Load = 15 PSF x 98' = 1470 PLF

2.5 Non-Typical Loads



Brendan Barrett | Gravity Loads

Non-Typical Loads

Penthouse (Area A and B)

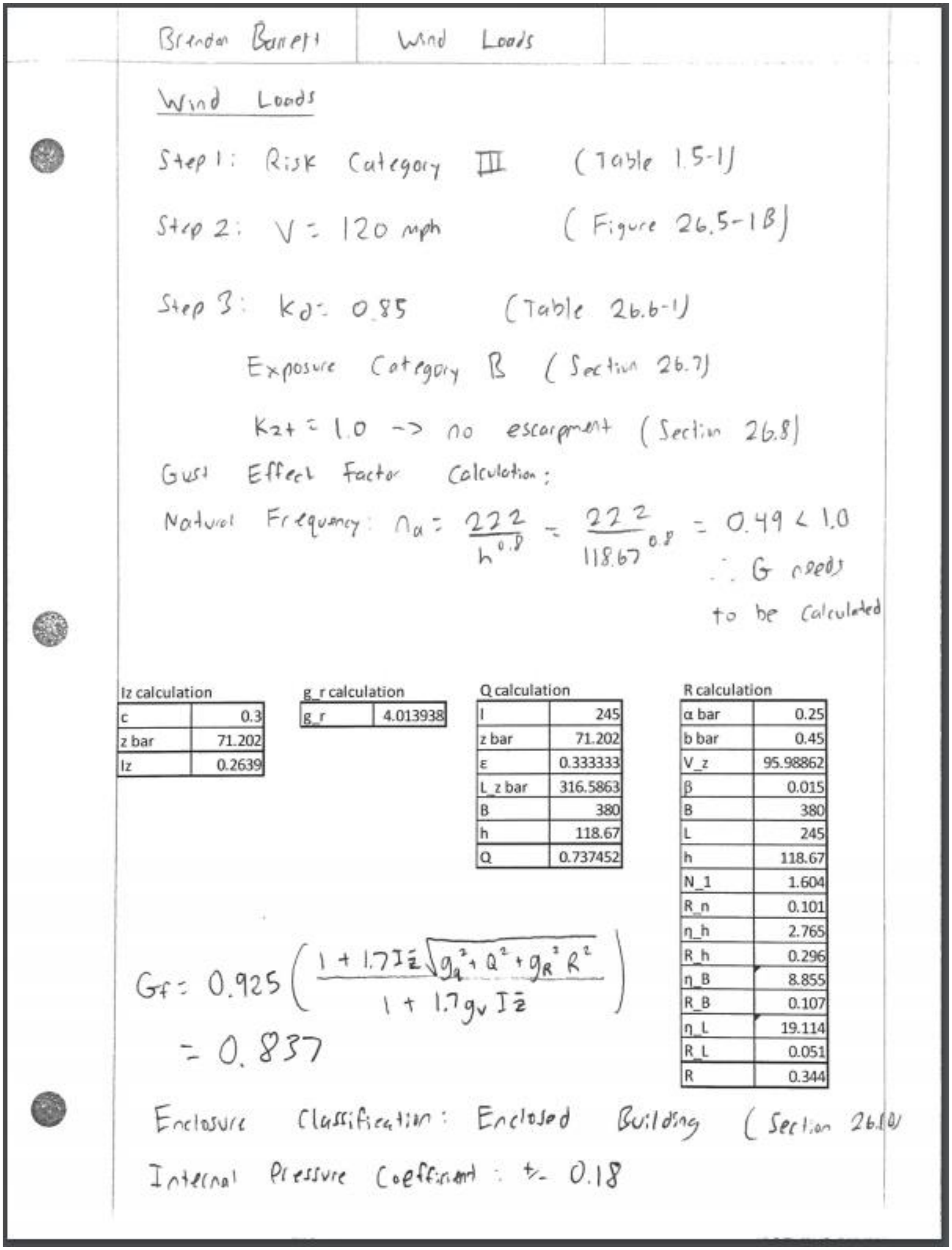
- Dead Load = 103 PSF
  -> larger than typical floor due to additional 3/4" of concrete (4 1/2" NW concrete on 3" metal deck)
- Live Load = 150 PSF
  -> larger than typical floor due to mechanical equipment

Terrace (Area C)

- Dead Load = 288 PSF
  -> increase due to green roof
- Live Load = 100 PSF
  -> Corridors

3. Wind Loads

See Appendix B for determination of wind load direction



Brendan Barrett | Wind Loads

Wind Loads

Step 1: Risk Category III (Table 1.5-1)

Step 2: V = 120 mph (Figure 26.5-1B)

Step 3: $K_d = 0.85$ (Table 26.6-1)

Exposure Category B (Section 26.7)

$K_{zt} = 1.0$ -> no escarpment (Section 26.8)

Gust Effect Factor Calculation:

Natural Frequency: $n_a = \frac{222}{h^{0.8}} = \frac{222}{118.67^{0.8}} = 0.49 < 1.0$

$\therefore$ G needs to be calculated

Iz calculation

c	0.3
z bar	71.202
Iz	0.2639

g_r calculation

g_r	4.013938

Q calculation

l	245
z bar	71.202
ε	0.333333
L_z bar	316.5863
B	380
h	118.67
Q	0.737452

R calculation

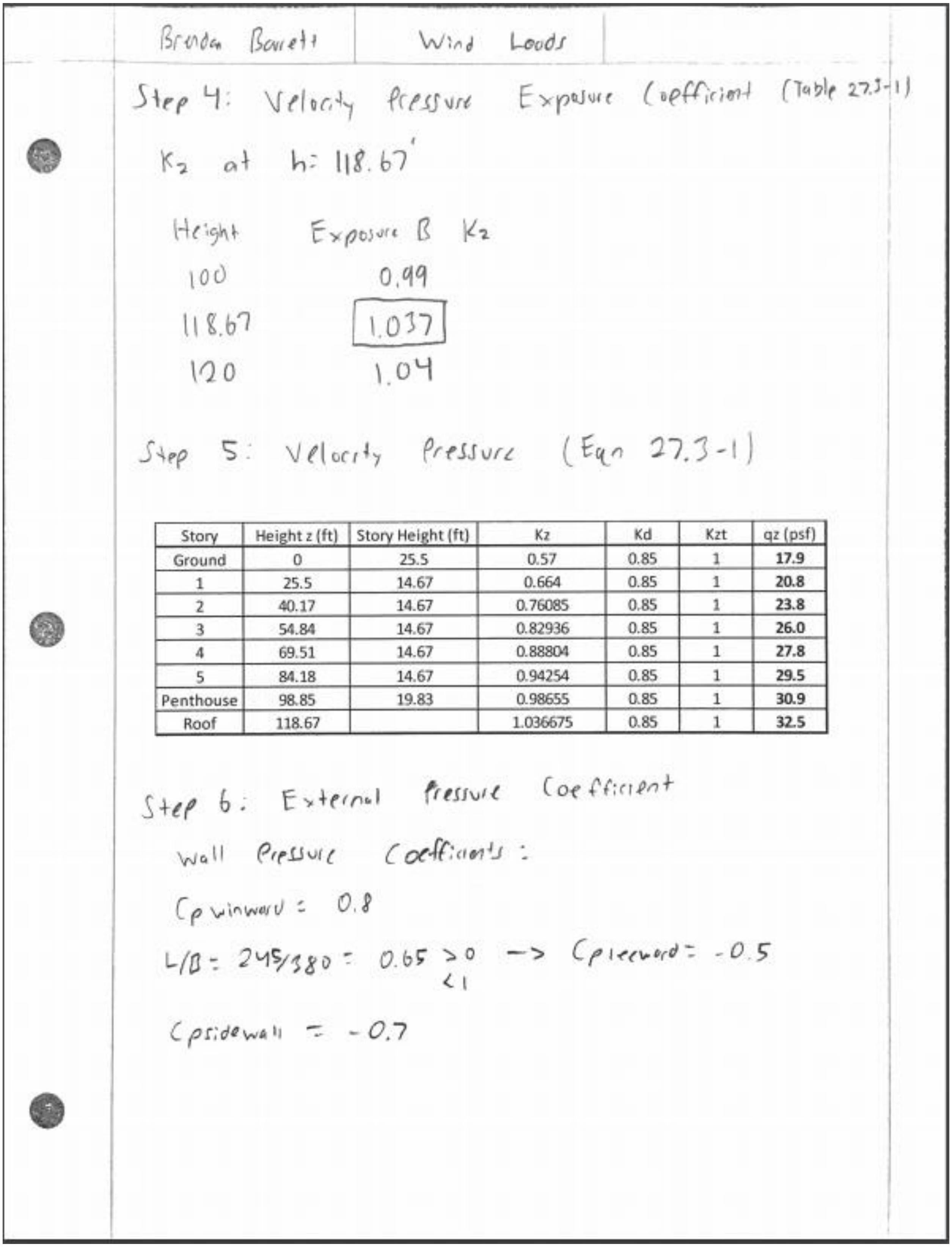
α bar	0.25
b bar	0.45
V_z	95.98862
β	0.015
B	380
L	245
h	118.67
N_1	1.604
R_n	0.101
n_h	2.765
R_h	0.296
n_B	8.855
R_B	0.107
n_L	19.114
R_L	0.051
R	0.344

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 + Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$= 0.837$$

Enclosure Classification: Enclosed Building (Section 26.10)

Internal Pressure Coefficient: ± 0.18



Brendan Barrett | Wind Loads

Step 4: Velocity Pressure Exposure Coefficient (Table 27.3-1)

K_z at h: 118.67'

Height	Exposure B K_z
100	0.99
118.67	$\boxed{1.037}$
120	1.04

Step 5: Velocity Pressure (Eqn 27.3-1)

Story	Height z (ft)	Story Height (ft)	Kz	Kd	Kzt	qz (psf)
Ground	0	25.5	0.57	0.85	1	**17.9**
1	25.5	14.67	0.664	0.85	1	**20.8**
2	40.17	14.67	0.76085	0.85	1	**23.8**
3	54.84	14.67	0.82936	0.85	1	**26.0**
4	69.51	14.67	0.88804	0.85	1	**27.8**
5	84.18	14.67	0.94254	0.85	1	**29.5**
Penthouse	98.85	19.83	0.98655	0.85	1	**30.9**
Roof	118.67		1.036675	0.85	1	**32.5**

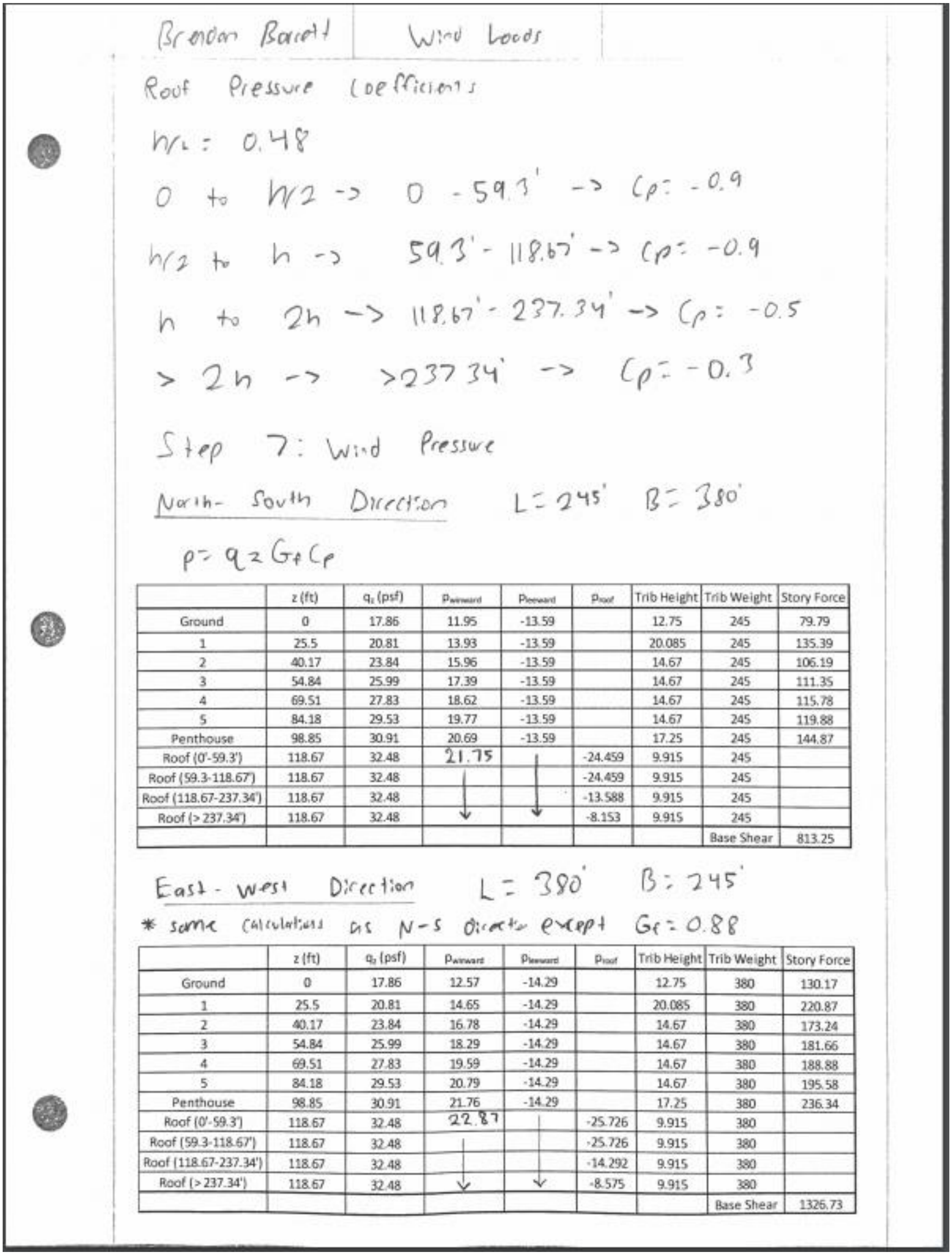
Step 6: External Pressure Coefficient

Wall Pressure Coefficients:

$C_{p\,\text{winward}}$ = 0.8

$L/B = 245/380 = 0.65 \underset{<1}{>0} \rightarrow C_{p\,\text{leeward}} = -0.5$

$C_{p\,\text{sidewall}} = -0.7$



Brendan Barrett | Wind Loads

Roof Pressure Coefficients

$h/L = 0.48$

0 to $h/2$ -> $0 - 59.3'$ -> $C_p = -0.9$

$h/2$ to h -> $59.3' - 118.67'$ -> $C_p = -0.9$

h to $2h$ -> $118.67' - 237.34'$ -> $C_p = -0.5$

$> 2h$ -> $> 237.34'$ -> $C_p = -0.3$

Step 7: Wind Pressure

North-South Direction $L = 245'$ $B = 380'$

$p = q_z G_f C_p$

	z (ft)	q_z (psf)	$p_{winward}$	$p_{leeward}$	p_{roof}	Trib Height	Trib Weight	Story Force
Ground	0	17.86	11.95	-13.59		12.75	245	79.79
1	25.5	20.81	13.93	-13.59		20.085	245	135.39
2	40.17	23.84	15.96	-13.59		14.67	245	106.19
3	54.84	25.99	17.39	-13.59		14.67	245	111.35
4	69.51	27.83	18.62	-13.59		14.67	245	115.78
5	84.18	29.53	19.77	-13.59		14.67	245	119.88
Penthouse	98.85	30.91	20.69	-13.59		17.25	245	144.87
Roof (0'-59.3')	118.67	32.48	21.75	↓	-24.459	9.915	245	
Roof (59.3-118.67')	118.67	32.48	↓	↓	-24.459	9.915	245	
Roof (118.67-237.34')	118.67	32.48	↓	↓	-13.588	9.915	245	
Roof (> 237.34')	118.67	32.48	↓	↓	-8.153	9.915	245	
							Base Shear	813.25

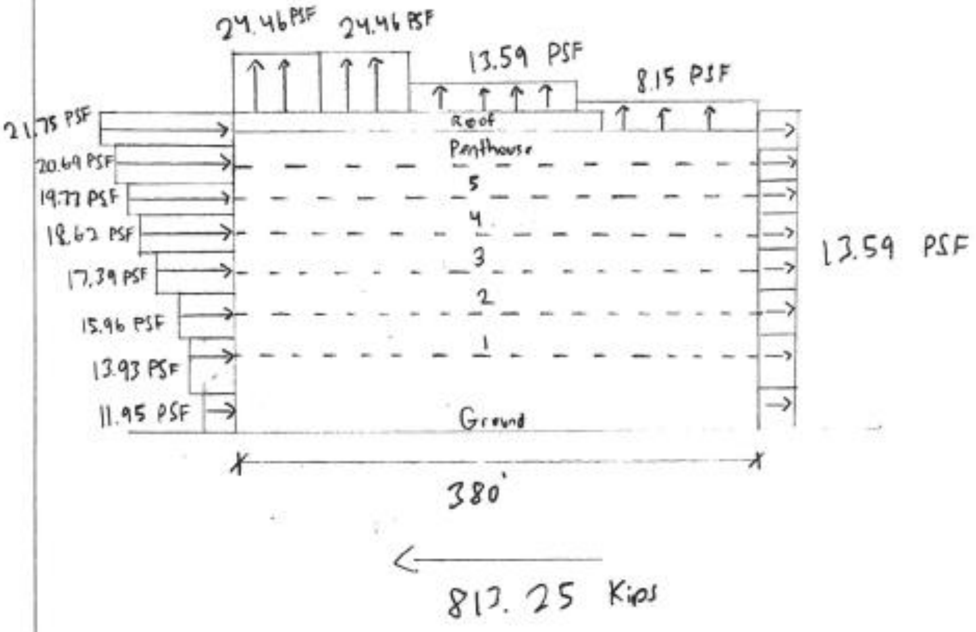
East-West Direction $L = 380'$ $B = 245'$

* same calculations as N-S direction except $G_f = 0.88$

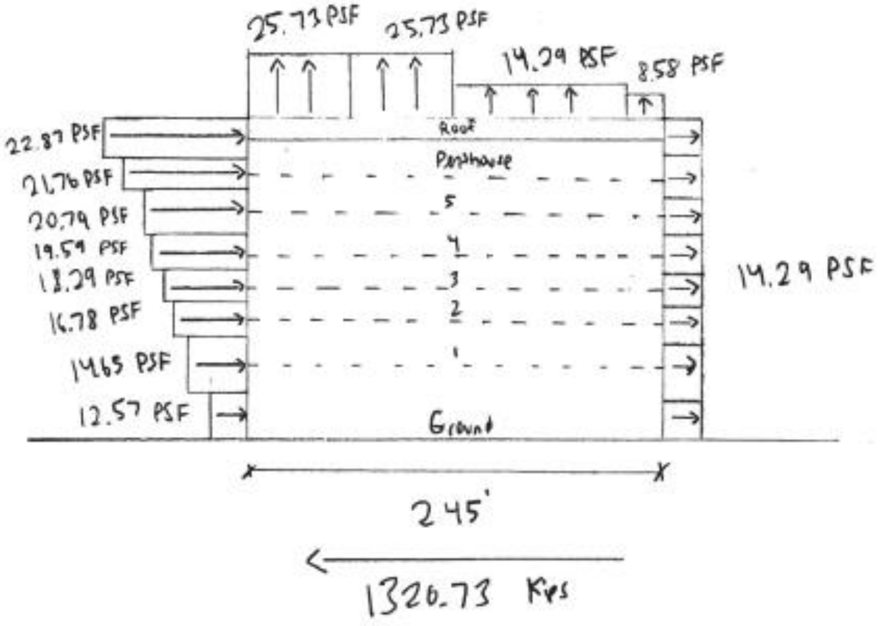
	z (ft)	q_z (psf)	$p_{winward}$	$p_{leeward}$	p_{roof}	Trib Height	Trib Weight	Story Force
Ground	0	17.86	12.57	-14.29		12.75	380	130.17
1	25.5	20.81	14.65	-14.29		20.085	380	220.87
2	40.17	23.84	16.78	-14.29		14.67	380	173.24
3	54.84	25.99	18.29	-14.29		14.67	380	181.66
4	69.51	27.83	19.59	-14.29		14.67	380	188.88
5	84.18	29.53	20.79	-14.29		14.67	380	195.58
Penthouse	98.85	30.91	21.76	-14.29		17.25	380	236.34
Roof (0'-59.3')	118.67	32.48	22.87	↓	-25.726	9.915	380	
Roof (59.3-118.67')	118.67	32.48	↓	↓	-25.726	9.915	380	
Roof (118.67-237.34')	118.67	32.48	↓	↓	-14.292	9.915	380	
Roof (> 237.34')	118.67	32.48	↓	↓	-8.575	9.915	380	
							Base Shear	1326.73

Brendan Barrett | Wind Loads

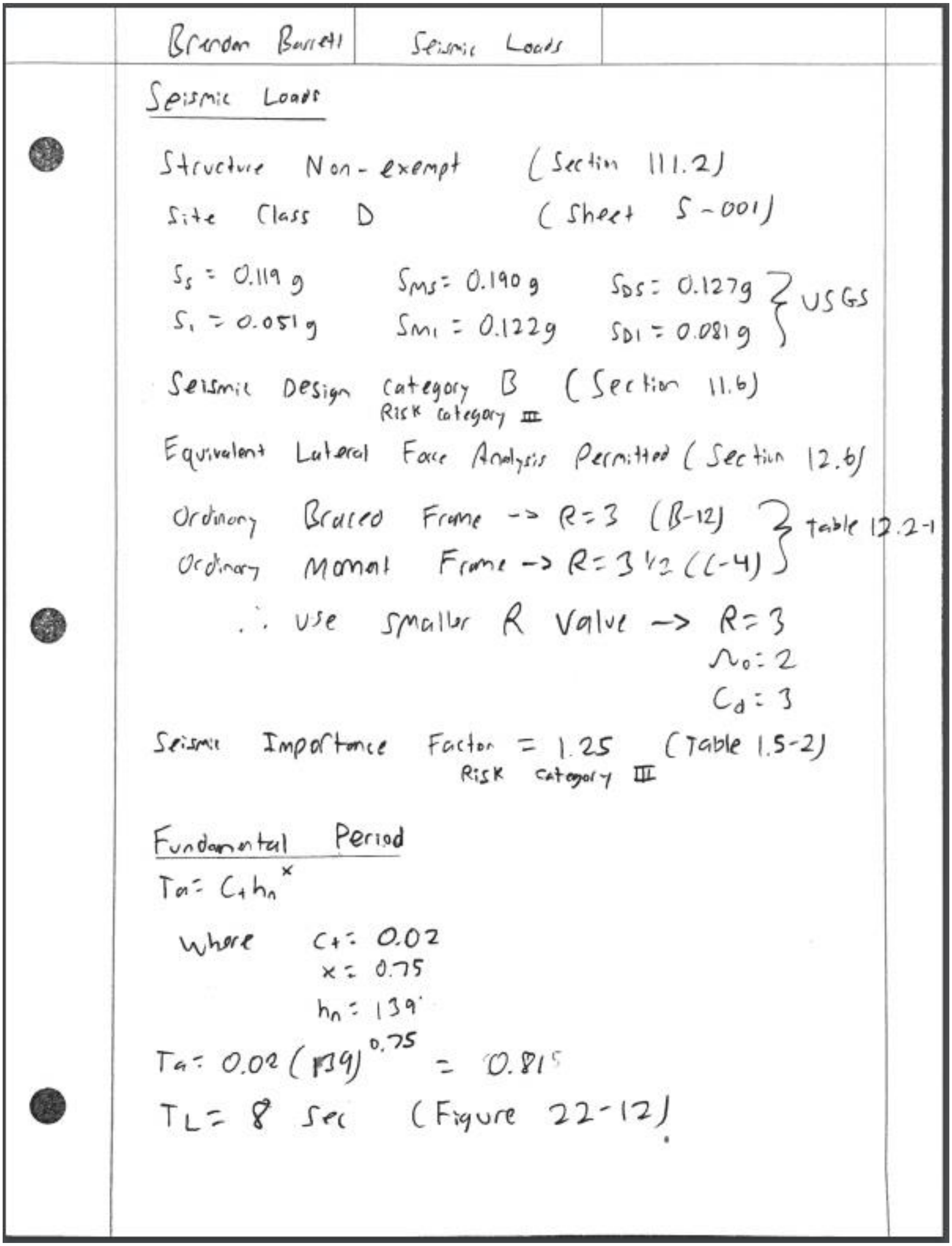
North-South Direction



East - West Direction



4. Seismic Loads



Brendan Barrett | Seismic Loads

Seismic Loads

Structure Non-exempt (Section 11.2)

Site Class D (Sheet S-001)

$$\left.\begin{array}{lll} S_s = 0.119\,g & S_{MS} = 0.190\,g & S_{DS} = 0.127\,g \\ S_1 = 0.051\,g & S_{M1} = 0.122\,g & S_{D1} = 0.081\,g \end{array}\right\} \text{USGS}$$

Seismic Design category B (Section 11.6)
Risk category III

Equivalent Lateral Face Analysis Permitted (Section 12.6)

$$\left.\begin{array}{l} \text{Ordinary Braced Frame} \rightarrow R = 3 \ (B\text{-}12) \\ \text{Ordinary Moment Frame} \rightarrow R = 3\tfrac{1}{2} \ (C\text{-}4) \end{array}\right\} \text{table 12.2-1}$$

∴ use smaller R value -> $R = 3$

$\Omega_0 = 2$

$C_d = 3$

Seismic Importance Factor = 1.25 (Table 1.5-2)
Risk category III

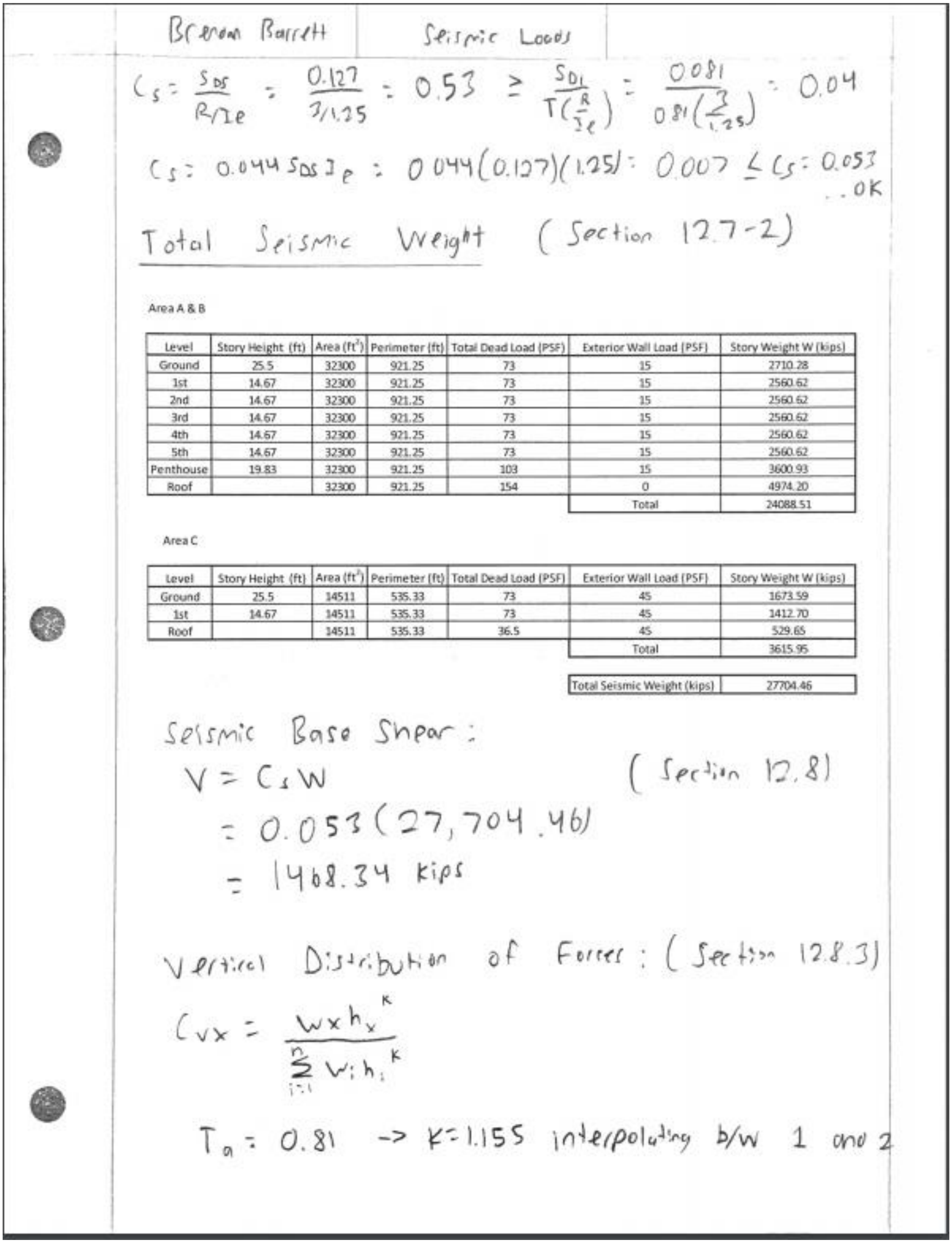
Fundamental Period

$T_a = C_t h_n^x$

where $C_t = 0.02$
$x = 0.75$
$h_n = 139'$

$T_a = 0.02(139)^{0.75} = 0.815$

$T_L = 8$ sec (Figure 22-12)



Brenan Barrett Seismic Loads

$$C_s = \frac{S_{DS}}{R/I_e} = \frac{0.127}{3/1.25} = 0.53 \geq \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.081}{0.81\left(\frac{3}{1.25}\right)} = 0.04$$

$$C_s = 0.044 S_{DS} I_e = 0.044(0.127)(1.25) = 0.007 \leq C_s = 0.053$$

..OK

Total Seismic Weight (Section 12.7-2)

Area A & B

Level	Story Height (ft)	Area (ft^2)	Perimeter (ft)	Total Dead Load (PSF)	Exterior Wall Load (PSF)	Story Weight W (kips)
Ground	25.5	32300	921.25	73	15	2710.28
1st	14.67	32300	921.25	73	15	2560.62
2nd	14.67	32300	921.25	73	15	2560.62
3rd	14.67	32300	921.25	73	15	2560.62
4th	14.67	32300	921.25	73	15	2560.62
5th	14.67	32300	921.25	73	15	2560.62
Penthouse	19.83	32300	921.25	103	15	3600.93
Roof		32300	921.25	154	0	4974.20
					Total	24088.51

Area C

Level	Story Height (ft)	Area (ft^2)	Perimeter (ft)	Total Dead Load (PSF)	Exterior Wall Load (PSF)	Story Weight W (kips)
Ground	25.5	14511	535.33	73	45	1673.59
1st	14.67	14511	535.33	73	45	1412.70
Roof		14511	535.33	36.5	45	529.65
					Total	3615.95

Total Seismic Weight (kips)	27704.46

Seismic Base Shear:

$V = C_s W$ (Section 12.8)

$= 0.053(27,704.46)$

$= 1468.34$ kips

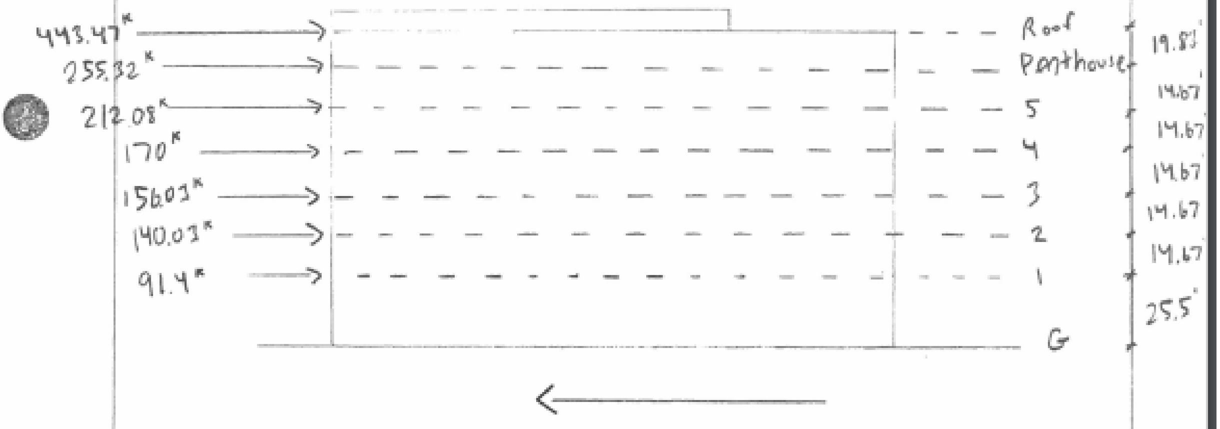
Vertical Distribution of Forces: (Section 12.8.3)

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$T_a = 0.81$ -> $k = 1.155$ interpolating b/w 1 and 2

Brendan Barrett | Seismic Loads

Level	h_x	W_x	h_x^k	$W_x h_x^k$	C_{vx}	F_x	V_x
Ground	25.5	4383.87	42.13	184676.57	0.06	91.40	1468.34
1st	40.17	3973.32	71.20	282917.88	0.10	140.03	1376.93
2nd	54.84	3090.27	102.01	315249.45	0.11	156.03	1236.91
3rd	69.51	2560.62	134.14	343486.70	0.12	170.00	1080.88
4th	84.18	2560.62	167.35	428510.65	0.14	212.08	910.88
5th	98.85	2560.62	201.46	515873.68	0.17	255.32	698.79
Penthouse	118.68	3600.93	248.83	896025.21	0.30	443.47	443.47
Roof		4974.20	0.00	0.00	0.00	0.00	0.00
Total		27704.46		2966740.15	1.00	1468.34	



V = 1468.34 kips

* Seismic Base Shear is the same in both directions

5. Typical Member Spot Checks for Gravity Loads

The following section analyzes the existing gravity system of the Brendan Iribe Center for Computer Science and Innovation. The existing system is composite steel framing with 3 ¼" lightweight concrete on 3" 20 gage metal deck. The bay that was chosen to be analyzed is highlighted in Figure 2 below and was selected as it represents a fairly standard size bay throughout the building. The columns circled below represent the interior and exterior columns that are analyzed. Note that the Dead Load for a typical floor from Notebook Submission A has been reduced from 73 PSF to 68 PSF as the framing allowance was reduced from 15 PSF to 10 PSF.

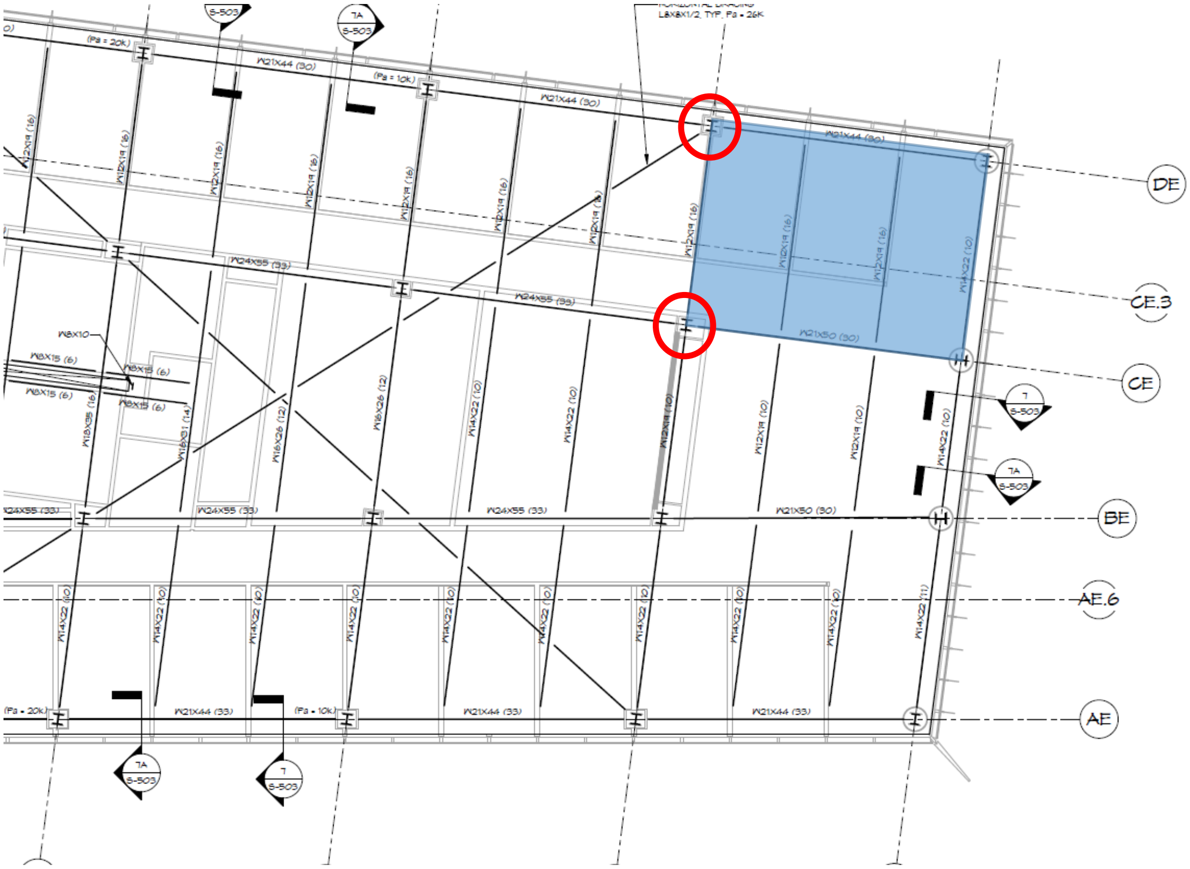
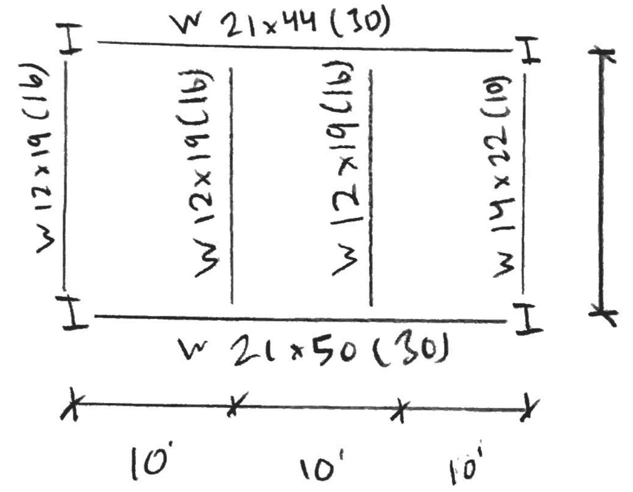


Figure 2: Bay used in analysis

Composite Steel Framing



Floor System:
3 1/4" LW concrete
on 3" 20 GA
composite metal
deck (6 1/4" total)

Loading = Dead = 68 PSF
Live = 100 PS

Metal Deck Check

3VLI 20 with 3 1/4" LWC

3 span Unshored clear span = 13'-3" > 10' ∴ ok

Superimposed live load = 100 PSF

Superimposed LL @ 10' clear span = 149 PSF > 100 PSF
∴ OK

W 12×19 (16) Infill Beam Check

Live Load Reduction:

$K_{LL}A_T = (10'+10')(21') = 420\ ft^2 > 400\ ft^2$

$L_0 = 100 \times \max\left| \begin{matrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{420}} = 0.982 \end{matrix} \right. = 98.2\ PSF$

$W_u = \left| \begin{matrix} 1.4(68) = 95.2\ PSF \\ 1.2(68) + 1.6(98.2) = 238.7\ PSF \end{matrix} \right.$

$W_u = 238.7\ PSF(10') = 2387\ PLF$

$M_u = 2387(21)^2/8 = 131.6^{'K}$

Check composite strength

$b_{eff} = \min\left| \begin{matrix} 21(12)/4 = 63 \Leftarrow \text{controls} \\ 10(12) = 120 \end{matrix} \right.$

Check Shear Stud capacity

$n = 16 \Rightarrow \Sigma Q_n = \frac{16}{2} \times 17.2 = 137.6^K$

From Table 3-21
Deck ⊥
weak studs position

$A_sF_y = (5.57\ in^2)(50\ ksi) = 278.5\ K$

$0.85 f'c\, b_{eff}\, t = 0.85(3.5\ ksi)(63\ in)(6.25\ in) = 1171.4^K$

Since $\begin{matrix} A_sF_y \\ 0.85 f'c\, b_{eff}\, t \end{matrix} > \Sigma Q_n$ ∴ Partially composite

$$x = \frac{A_s F_y - \Sigma Q_n}{2 b_f F_y} = \frac{278.5 - 137.6}{2(4)(50)} = 0.352'' > 0.35''$$

∴ NA is in web

$$a = \frac{137.6}{0.85(3.5)(63)} = 0.734'' \Rightarrow y_2 = 6.25 - \frac{0.734}{2} = 5.88''$$

$$\phi M_n = 0.9\left[137.6(5.88'') + 278.5\left(\frac{12.2}{2}\right) - 2(50)(4)(0.352)\left(\frac{0.352}{2}\right)\right]$$

$$\phi M_n = 186.2^{\prime k} > M_u = 131.6^{\prime k}$$ ∴ OK

Check unshored Strength

$$W_u = 1.4(48)(10') + 1.4(19) = 0.6986 \text{ klf}$$

$$W_u = 1.2(48(10) + 19) + 1.6(\underset{\uparrow}{20})(10') = 0.9188 \text{ klf}$$

Construction LL

$$M_u = \frac{0.9188(21)^2}{8} = 50.6^{\prime k}$$

ϕM_n (Table 3-2) $= 92.6^{\prime k} > 50.6^{\prime k}$ ∴ OK for Unshored Strength

Wet concrete deflection

$$W_{wc} = 48(10) + 19 = 0.499 \text{ klf}$$

$$I_x = 130 \text{ in}^4$$

$$\Delta_{wc} = \frac{5(0.499)(21)^4(1728)}{384(29000)(130)} = 0.58'' < \frac{l}{360} = \frac{21(12)}{360} = 0.7''$$

∴ OK

Live Load Deflection

$W_{LL} = 98.2(10) = 0.982$ klf

			Y_2	
I_{LB}		5.5	5.88	6
Y_1	0.35	378	381.52	400

$I_{LB} \approx 381.5$ @ $Y_2 = 5.88$ & $\Sigma Q_n = 137.6^{k}$

$$\Delta_{LL} = \frac{5(0.982)(21)^4(1728)}{384(29000)(381.5)} = 0.39'' < \frac{\ell}{360} = 0.7'' \quad \therefore \text{OK}$$

=> W 12×19 (16) Infill Beam is OK

W 21 x 50 (30) Girder Check

Live Load Reduction:

$K_{LL}A_T = 1148\ ft^2$

$$L_o = 100 \times \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{1148}} \end{array} \right. = 0.693 = 69.3\ PSF$$

$$W_u = \begin{array}{l} 1.4(68) \\ 1.2(68) + 1.6(69.3) = 192.5 \end{array}$$

Point Loads from Infill Beams

$$P = 192.5(10')(\tfrac{21'}{2}) + 192.5(10')(\tfrac{19'}{2}) = 38.5^K$$

$$M_u = Pa = 38.5^K(10') = 385'^K$$

Check Composite Strength

$$b_{eff} = \left| \begin{array}{l} \frac{19(12)}{2} = 114 \\ \quad + \\ \frac{30(12)}{8} = 45 \end{array} \right| \begin{array}{l} \frac{30(12)}{8} = 45 \\ \\ \frac{21(12)}{2} = 126 \end{array} = 90\ in.$$

Check Shear Stud Capacity

$$n = 30 \Rightarrow \Sigma Q_n = \frac{30}{2} \times 17.2 = 258^K$$

$$A_sF_y = (14.7\ in^2)(50\ ksi) = 735^K$$

$$0.85f'_c b_{eff} t = 0.85(3.5\ ksi)(90'')(6.25'') = 1673^K > \Sigma Q_n = 258^K$$

$\therefore$ Partially composite

$$x = \frac{A_sF_y - \Sigma Q_n}{2 b_f F_y} = \frac{735 - 258}{2(6.53)(50)} = 0.73 > t_f = 0.535''$$

$\therefore$ NA is in web

$$a = \frac{258}{0.85(3.5)(90)} = 0.96'' \Rightarrow Y_2 = 6.25 - \frac{0.96}{2} = 5.77''$$

$$\phi M_n = 0.9\left[258(5.77'') + 735\left(\frac{20.8}{2}\right) - 2(50)(6.53)(0.73)\left(\frac{0.73}{2}\right)\right]$$

$$\phi M_n = 671.9^{\prime k} \geq M_u = 385^{\prime k} \;\therefore \text{OK}$$

Check unshored strength

$$W_u = 1.4(48)\left(\frac{21}{2} + \frac{19}{2}\right) + 1.4(50) = 1.414 \text{ klf}$$

$$W_u = 1.2\left(48\left(\frac{21}{2} + \frac{19}{2}\right) + 50\right) + 1.6(20)\left(\frac{21}{2} + \frac{19}{2}\right) = 1.852 \text{ klf}$$

$$M_u = \frac{1.852(30)^2}{8} = 208.3^{\prime k}$$

$$\phi M_n \text{ (Table 3-2)} = 413^{\prime k} > 208.3^{\prime k} \;\therefore \text{OK for unshored strength}$$

Wet concrete deflection

$$W_{wc} = 48\left(\frac{21+19}{2}\right) + 50 = 1.010 \text{ klf}$$

$$I_x = 890 \text{ in}^4 \text{ (Table 3-2)}$$

$$\Delta_{wc} = \frac{5(1.010)(30)^4(1728)}{384(29000)(890)} = 0.71'' < \frac{l}{360} = \frac{30(12)}{360} = 1'' \;\therefore \text{OK}$$

Live Load Deflection

$W_{LL} = 69.3\left(\frac{21+19}{2}\right) = 1.386$ klf

I_{LB}			Y_2	
		5.5	5.77	6
	0.535	2260	2308	2350
Y_1	0.73		2287	
	2.91	2020	2058	2090

$I_{LB} \approx 2287 \text{ in}^4$ @ $Y_2 = 5.77$ & $\Sigma Q_n = 258^k$

$$\Delta_{LL} = \frac{5(1.386)(30)^4(1728)}{384(29000)(2287)} = 0.38'' < \frac{l}{360} = 1''$$

$\therefore$ OK

$\Rightarrow$ W 21 x 50 (30) Girder is OK

W 21x44 (30) Girder Check

Live Load Reduction

$K_{LL} A_T = 30(21') = 630 \text{ ft}^2 > 400 \text{ ft}^2$

$$L_o = 100 \times \max\left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{630}} = 0.848 \end{array} \right. = 84.8 \text{ PSF}$$

$W_u = 1.4(68) = 95.2 \text{ PSF}$

$1.2(68) + 1.6(84.8) = 217.28 \text{ PSF} \Leftarrow$ controls

Point Loads from Infill Beams:

$P = 217.28(10)\left(\frac{21'}{2}\right) = 22.8^k$

$M_u = 22.8^k (10') = 228'^k$

Check composite strength

$$b_{eff} = \min\left| \begin{array}{l} 1'\text{-}8'' \\ \frac{30(12)}{8} = 45 \end{array} \right. + \min\left| \begin{array}{l} \frac{30(12)}{8} = 45 \\ \frac{21(12)}{2} = 126 \end{array} \right. = 46.67 \text{ in}$$

Check shear stud capacity:

$n = 30 \Rightarrow \Sigma Q_n = \frac{30}{2} \times 17.2 = 258^k$

$$A_sF_y = (13.0 \text{ in}^2)(50 \text{ ksi}) = 650^{K}$$

$$0.85 f'_c b_{eff} t = 0.85(3.5 \text{ ksi})(46.67 \text{ in})(6.25 \text{ in}) = 867.8^{K}$$

Since $\begin{matrix} A_sF_y \\ 0.85 f'_c b_{eff} t \end{matrix} > \Sigma Q_n$ ∴ Partially composite

$$x = \frac{A_sF_y - \Sigma Q_n}{2 b_f F_y} = \frac{650 - 258}{2(6.50)(50)} = 0.6'' > t_f = 0.45''$$

∴ NA is in web

$$a = \frac{258}{0.85(3.5)(46.67)} = 1.86'' \Rightarrow y_2 = 6.25 - \frac{1.86}{2} = 5.32''$$

$$\phi M_n = 0.9\left[258(5.32) + 650\left(\frac{20.7}{2}\right) - 2(50)(6.5)(0.6)\left(\frac{0.6}{2}\right)\right]$$

$$\phi M_n = 598.7^{'k} > M_u = 228^{'k} \therefore \text{OK}$$

Check unshored strength

$$W_u = 1.4(48)\left(\frac{21}{2}\right) + 1.4(44) = 0.7672 \text{ klf}$$

$$W_u = 1.2\left(48\left(\frac{21}{2}\right) + 44\right) + 1.6(20)\left(\frac{21}{2}\right) = 0.9936 \text{ klf}$$

$$M_u = \frac{0.9936(30)^2}{8} = 111.8^{'k}$$

ϕM_n (Table 3-2) $= 358^{'k} > 111.8^{'k}$ ∴ OK for unshored strength

Wet Concrete Deflection

$$W_{wc} = 48\left(\frac{21}{2}\right) + 44 = 0.548 \text{ klf}$$

$I_x = 843 \text{ in}^4$ (Table 3-2)

$$\Delta_{wc} = \frac{5(0.548)(30)^4(1728)}{384(29000)(843)} = 0.41'' < \frac{l}{360} = \frac{30(12)}{360} = 1''$$

∴ OK

Live Load Deflection

$W_{LL} = 84.8\left(\frac{21}{2}\right) = 0.8904$ klf

			Y_2	
I_{LB}		5	5.32	5.5
	0.450	1930	1974.8	2000
Y_1	0.6		1961.7	
	2.92	1720	1758.4	1780

$I_{LB} \approx 1961.7 \text{ in}^4$ @ $Y_2 = 5.32''$ & $\Sigma Q_n = 258^k$

$$\Delta_{LL} = \frac{5(0.8904)(30)^4(1728)}{384(29000)(1961.7)} = 0.29'' < \frac{l}{360} = 1''$$

$\therefore$ OK

$\Rightarrow$ W 21×44 (30) Girder is OK

Exterior Column Check (W 12×65)

Typical Loading

Dead = 68 PSF

Live = 100 PSF

Curtain Wall Load = 15 PSF

Roof Live = 30 PSF

Roof Dead = 148 PSF

Live Load Reduction:

$$A_T = \left(\frac{21'}{2} + 1'\text{-}8''\right)(30') = 365\ ft^2$$

$$K_{LL} = \underset{\text{For exterior column}}{\underset{\uparrow}{3}} \Rightarrow K_{LL} A_T = 1095\ ft^2 > 400\ ft^2$$

$$L_o = 100 \times \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{1095}} = 0.70 \end{array} \right. = 70\ PSF$$

Loading:

Typical Floor = $1.2(68) + 1.6(70) = 193.6$ PSF

Roof $= 1.2D + 1.6(L_R \text{ or } S \text{ or } R)$

$= 1.2(148) + 1.6(30)$

$= 225.6$ PSF

P_u = 6 typical Floors + Roof + curtain wall

$= 6(193.6\ PSF)(365\ ft^2) + (225.6\ PSF)(365\ ft^2) + 15PSF(90')(30')$

$= 547^K$

W 12×65 unbraced length ≈ 15'

$\phi P_n = 663^K > 547^K \therefore$ OK

Interior Column Check (W 12 x 106)

Typical Loading:

Dead = 68 PSF

Live = 100 PSF

Roof Live = 30 PSF

Roof Dead: 148 PSF

Live Load Reduction:

$$A_T = \left(\frac{21}{2} + \frac{20}{2}\right)(30) = 615 \text{ ft}^2$$

$$K_{LL} = \underset{\uparrow}{4} \Rightarrow K_{LL} A_T = 2460 \text{ ft}^2$$

for interior column

$$L_o = 100 \times \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{2460}} = 0.552 \end{array} \right. = 55.2 \text{ PSF}$$

Loading:

Typical Floor = 1.2(68) + 1.6(55.2) = 170 PSF

Roof = 1.2(148) + 1.6(30) = 225.6 PSF

P_u = 6 typical Floors + roof

$$= 6(170 \text{ PSF})(615 \text{ ft}^2) + 225.6 \text{ PSF}(615 \text{ ft}^2)$$

$$= 766^K$$

W 12 x 106 Unbraced length ≈ 15'

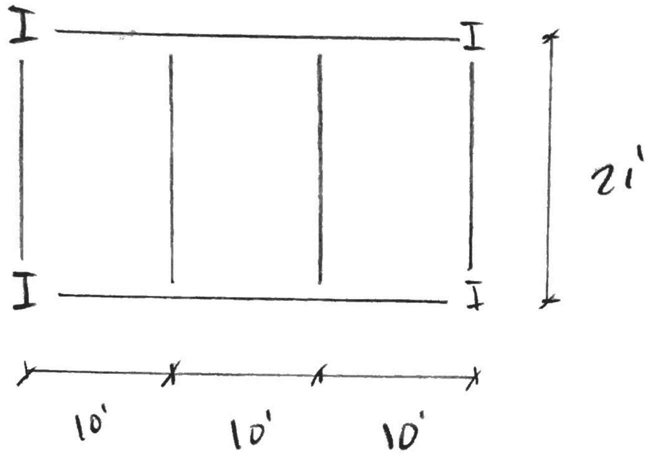
$$\phi P_n = 1100^K > 766^K \therefore \text{OK}$$

6. Alternative Framing Systems for Gravity Loads

6.1 Alternate Design #1: Non-Composite Steel Framing

The same bay that was analyzed above will now be redesigned using non-composite steel framing. The deck is designed using the Vulcraft Catalog.

Design #1 : Non-Composite Steel Framing



Misc DL = 20
Slab = 57
Live = 100

Decking

3 Span Max construction clear span ≥ 10'

Try 3C18 W/ 3" NW concrete

Max construction span = 14'-2" ≥ 10' ∴ OK

$$\text{Total Load} = \underset{LL}{100} + \underset{Slab}{57} + \underset{M/E/C/L}{10} + \underset{Framing}{10} = 177 \text{ PSF}$$

Allowable load @ 10' = 193 PSF > 177 PSF ∴ OK

W_{LL} = 100 PSF < $l/240$ = 155 PSF ∴ OK

=> use 3C18 W/ 3" NW concrete

Infill Beam Design

Live Load Deflection:

$$\Delta_{LL} = \frac{l}{360} = \frac{21(12)}{360} = 0.7''$$

$$W_{LL} = 98.2 \text{ PSF } (10') = 0.982 \text{ klf}$$

$$I_{req} = \frac{5(0.982)(21)^4(1728)}{384(29000)I} \leq 0.7''$$

$$I_{req} \geq 211 \text{ in}^4$$

Total Load Deflection:

$$\Delta_{TL} = \frac{l}{240} = \frac{21(12)}{240} = 1.05''$$

$$W_{TL} = (57+10+10+98.2)(10') = 1.752 \text{ klf}$$

$$I_{req} = \frac{5(1.752)(21)^4(1728)}{384(29000)I} \leq 1.05''$$

$$I_{req} \geq 251 \text{ in}^4$$

Try W 14x30 $I = 291 \text{ in}^4$

Check Flexure:

$W_u = 1.4(77) = 107.8$

$1.2(77) + 1.6(98.2) = 249.5$ PSF ⇐ controls

$W_u = 249.5(10') = 2495$ PLF

$$M_u = \frac{2495(21)^2}{8} = 137.5^{'k}$$

ϕM_n (Table 3-2) $= 177^{'k} > M_u = 137.5^{'k}$ ∴ OK

=> Use W14x30 Infill Beams

Spandrel Girder Design

Live Load Deflection:

$$\Delta_{LL} = \frac{l}{360} = \frac{30(12)}{360} = 1''$$

$P_{LL} = 84.8(10)(\frac{21}{2}) = 8.9^k$

$$\Delta_{LL} = \frac{8.9(10)}{24(29000)I}\left[3(30)^2 - 4(10)^2\right](1728) \le 1''$$

$I_{req} \geq 508\ in^4$

Total Load Deflection:

$$\Delta_{TL} = \frac{l}{240} = \frac{30(12)}{240} = 1.5''$$

$$P_{TL} = (77 + 84.8)(10)\left(\frac{21'}{2}\right) = 17.0^{K}$$

$$\Delta_{TL} = \frac{34.0(10)}{24(29000)I}\left[3(30)^2 - 4(10)^2\right](1728) \le 1.5''$$

$$I_{req} \ge 647 \text{ in}^4$$

Try W 21x44. $I = 843 \text{ in}^4$

Check Flexure

$$W_u = 1.4(77) = 107.8$$

$$1.2(77) + 1.6(84.8) = 228.1 \text{ PSF} \Leftarrow \text{controls}$$

$$P_u = 228.1(10')\left(\frac{21'}{2}\right) = 23.95^{K}$$

$$M_u = Pa = 23.95(10') = 239.5^{'K}$$

$$\phi M_n = 358^{'K} > M_u = 239.5^{'K} \therefore \text{OK}$$

⇒ use W 21x44 Spandrel girder

Girder Design

Live Load Deflection $\Rightarrow \frac{l}{360} = 1''$

$$P_{LL} = 69.3(10)\left(\frac{21}{2}\right) + 69.3(10)\left(\frac{19}{2}\right) = 13.9^{k}$$

$$\Delta_{LL} = \frac{13.9(10)}{24(29000)I}\left[3(30)^2 - 4(10)^2\right](1728) \le 1''$$

$$I_{req} \ge 804 \text{ in}^4$$

Total Load Deflection $\Rightarrow \frac{l}{240} = 1.5''$

$$P_{TL} = (77+69.3)(10)\left(\frac{21}{2}\right) + (77+69.3)(10)\left(\frac{19}{2}\right) = 29.3^{k}$$

$$\Delta_{TL} = \frac{29.3(10)}{24(29000)I}\left[3(30)^2 - 4(10)^2\right](1728) \le 1.5''$$

$$I_{req} \ge 1130 \text{ in}^4$$

Try W 24x55 $I = 1350 \text{ in}^4$

Flexure check

$$W_u = \left| \begin{array}{l} 1.4(77) = 107.8 \text{ PSF} \\ 1.2(77) + 1.6(69.3) = 203.28 \text{ PSF} \end{array} \right.$$

$$P_u = 203.28(10)\left(\frac{21}{2}\right) + 203.28(10)\left(\frac{19}{2}\right) = 40.6^{k}$$

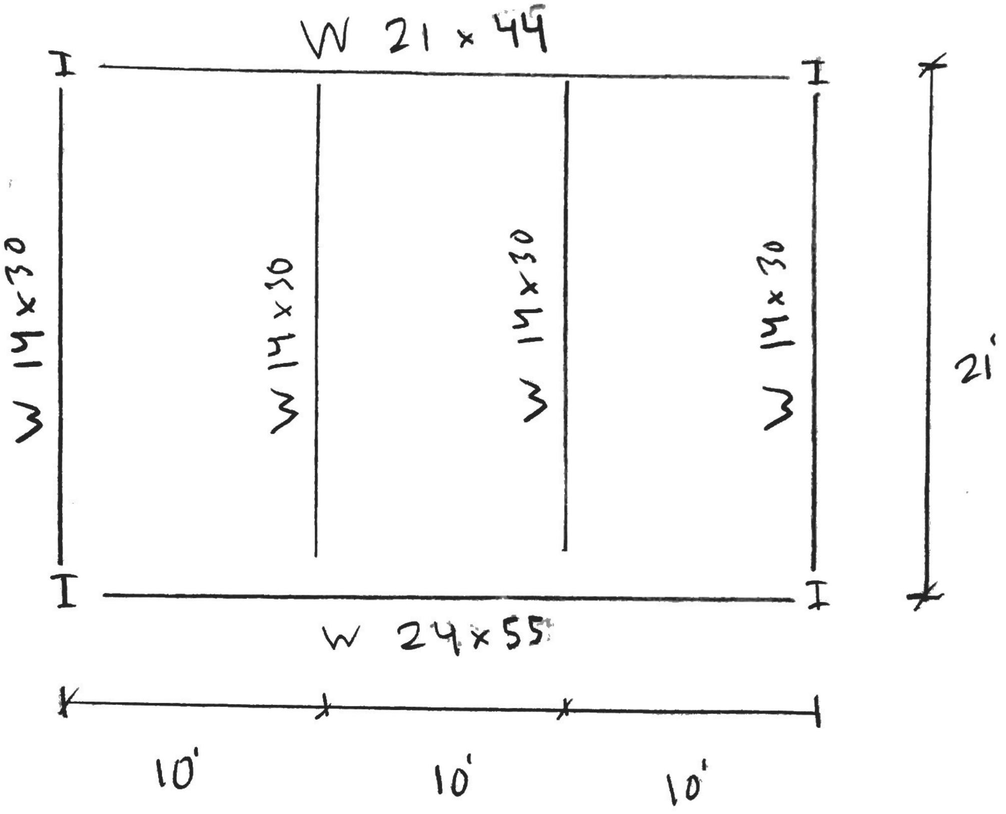
$$M_u = 40.6^{k}(10') = 406^{'k}$$

$$\phi M_n = 503^{'k} > M_u = 406^{'k} \therefore OK$$

$\Rightarrow$ use W 24x55 Girder

Final Design Layout

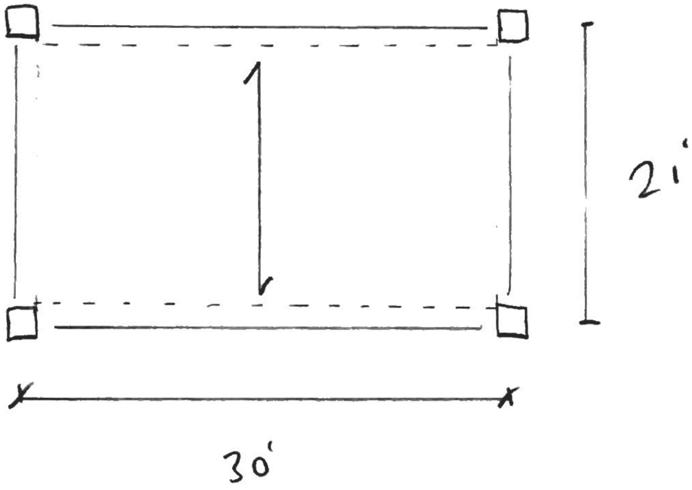
Deck = 3C18 w/ 3" NW concrete



6.2 Alternate Design #2: One-Way Slab with Edge Beam

This 21’ x 30’ bay will now be designed using a one-way slab with edge beams. The slab will span parallel to the 21’ direction.

Design # 2 : One - Way Slab



SDL = 20 PSF

LL = 100 PSF

Slab Design

$f'c = 3500$ psi

LW concrete => 115 PCF

Minimum slab thickness :

From ACI 318-14 Table 7.3.1.1

- one end continuous (end bay) => $t = \frac{\ell}{24} = \frac{21(12)}{24} = 10.5''$

∴ use 11" slab

$d = 11 - \underset{\uparrow \text{ Clear Cover}}{0.75} - \underset{\uparrow \text{ \#4 bars}}{0.5/2} = 10''$

Loads:

Dead Load $= \left(\frac{11''}{12}\right) \times 115$ PCF $+ 20$ PSF $= 126$ PSF

Live Load $= 100 \times 0.25 + \frac{15}{\sqrt{30 \times 21}} = 84.8$ PSF

$$W_u = \left| \begin{array}{l} 1.4(126) = 174.4 \text{ PSF} \\ 1.2(126) + 1.6(84.8) = 286.9 \text{ PSF} \end{array} \right.$$

$$M_u = \frac{286.9(\overset{\text{unit strip method}}{1'})(21)^2}{8} = 15.8^{1K}$$

$$A_s \geq \frac{M_u}{\phi F_y(d - \frac{a}{2})} = \frac{15.8 \times 12}{0.9(60)\underbrace{(0.95)(10)}_{\text{Assume } jd}} = 0.37 \text{ in}^2/\text{ft}$$

$$a = \frac{(0.37 \text{ in}^2)(60 \text{ ksi})}{0.85(3.5 \text{ ksi})(12 \text{ in})} = 0.62 \Rightarrow c = \frac{0.62}{0.85} = 0.73$$

$$\varepsilon_s = \frac{0.003(10 - 0.73)}{0.73} = 0.038 > 0.005$$

∴ Steel yielded

tension controlled => $\phi = 0.9$

=> use #6 @ 12" o.c. $A_s = 0.44 \text{ in}^2/\text{ft}$

Minimum Reinforcement:

$$A_{s\,min} = 0.0018bh = 0.0018(12)(11) = 0.237 \text{ in}^2 < 0.44 \text{ in}^2$$

∴ OK

Max Spacing:

$$S_{max} = \min \left| \begin{array}{l} 3h = 3(11) = 33'' \\ 18 = 18'' \Leftarrow \text{controls} \end{array} \right.$$

∴ OK

Max Spacing for Crack Control:

$$S = \left| \begin{array}{l} 15\left(\frac{40000}{f_s}\right) - 2.5C_c = 15\left(\frac{40000}{\frac{2}{3}(60000)}\right) - 2.5(0.75) = 13.125 \\ 12\left(\frac{40000}{f_s}\right) = 12\left(\frac{40000}{\frac{2}{3}(60000)}\right) = 12'' \end{array} \right.$$

$S_{max} = 12'' \geq 12'' \therefore$ OK

Check One way Shear:

$$V_u = \frac{1.15\,w_u l}{2} = \frac{1.15(286.9)(21)}{2} = 3.5^k$$

$$\phi V_c = \phi 2 \lambda \sqrt{f'c}\, b_w d$$

$$= 0.75(2)(0.75)\sqrt{3500}\,(12)(10)$$

$$= 8.0^k > 3.5^k \therefore \text{OK}$$

Check Flexure:

$$A_s F_y = 0.85 f'c\, b a$$

$$a = \frac{0.44(60)}{0.85(3.5)(12)} = 0.74 \Rightarrow c = \frac{0.74}{0.85} = 0.87$$

$$d = 11 - 0.75 - 0.75/2 = 9.88 \text{ in}$$

$$\varepsilon_s = \frac{0.003(9.88 - 0.87)}{0.87} = 0.03 > 0.005$$

$\therefore$ Steel yields $\phi = 0.9$

$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2} \right)$$

$$= 0.9(0.44)(60)\left(9.88 - \frac{0.74}{2}\right)$$

$$= 18.8^{1k} > M_u = 15.8^{1k} \therefore \text{OK}$$

Shrinkage and Temperature Reinforcement:

$$A(S+T) = 0.0018bh = 0.0018(12)(11) = 0.237 \text{ in}^2$$

$$S_{max} = \min \left| \begin{array}{ll} 5h & = 5(11) = 55'' \\ 18 & = 18'' \Leftarrow \text{controls} \end{array} \right.$$

$\Rightarrow$ Use #5 @ 12'' $\quad A_s = 0.31 \text{ in}^2$

Beam Design

$$w_u = 286.9 \text{ PSF} \left(\frac{21'}{2}\right) = 3012.5 \text{ PLF}$$

$$M_u = \frac{3012.5(30)^2}{8} \times 1.1 \text{ Self weight of beam} = 372.8^{1k}$$

Calculate tentative ρ

$$\rho = \frac{0.25 f'_c \beta_1}{f_y} = \frac{0.25(3.5)(0.85)}{60} = 0.0124$$

$$M_n = \frac{M_u}{\phi} = \frac{372.8}{0.9} = 414.2^{1k}$$

$$w = \frac{\rho f_y}{f'_c} = \frac{0.0124(60)}{3.5} = 0.213$$

$R = w f'c (1 - 0.59 w)$

$= 0.213(3.5)(1 - 0.59(0.213))$

$= 0.65 \text{ Ksi}$

$M_n = Rbd^2$

$bd^2 = \frac{M_n}{R} = \frac{414.2 \times 12}{0.65 \text{Ksi}} = 7646 \text{ in}^3$

Try. $b = 18''$ $d = 24''$ $h = 27''$

$A_{s\,req} = \frac{M_u}{\phi F_y jd} = \frac{372.8 \times 12}{0.9(60)(0.95)(24)} = 3.63 \text{ in}^2$

Use 4 #9 $A_s = 4.0 \text{ in}^2$

Check Flexure:

$A_s F_y = 0.85 f'c\, ba$

$a = \frac{4(60)}{0.85(3.5)(18)} = 4.48 \Rightarrow c = \frac{4.48}{0.85} = 5.27$

$\varepsilon_s = \frac{0.003(24 - 5.27)}{5.27} = 0.01 > 0.005$

$\therefore$ Tension Controlled $\phi = 0.9$

$\phi M_n = 0.9(4)(60)\left(24 - \frac{4.48}{2}\right)$

$= 391.7^{'k} > 372.8^{'k}$ $\therefore$ OK

Check Shear:

$$W_u = 3012.5 \text{ PLF} + \frac{22 \times 18}{144} \times 115 = 3.4 \text{ klf}$$

$$V_u = \frac{wl}{2} = \frac{3.4(30)}{2} = 51.0^k$$

$$\phi V_c = \phi 2 \lambda \sqrt{f'c}\, b_w d$$

$$= 0.75(2)(0.75)\sqrt{3500}(18)(24)$$

$$= 28.8^k < V_u \quad \therefore \text{ need shear Reinforcement}$$

$$V_s = \frac{V_u}{\phi} - V_c$$

$$= \frac{51.0}{0.75} - 38.3$$

$$= 29.7^k$$

$$\text{check } 8\sqrt{f'c}\, b_w d = 204^k > V_s \quad \therefore \text{ OK}$$

Solve for Stirrup Spacing using #3 2 branch (0.22 in²) ✓

$$S \le \frac{A_v f_{yt} d}{V_s} = \frac{0.22(60)(24)}{29.7} = 10.67''$$

∴ use 10'' spacing

$$S_{max} = \min \begin{vmatrix} d/2 = 12'' \Leftarrow \text{govern} \\ 24 \end{vmatrix}$$

$$A_{v_{min}} = \max \begin{vmatrix} 0.75\sqrt{f'c} \\ 50 \end{vmatrix} \left(\frac{b_w s}{f_{yt}}\right) \begin{matrix} = 0.13 \text{ in}^2 \\ = 0.15 \text{ in}^2 < 0.22 \text{ in}^2 \therefore \text{ ok} \end{matrix}$$

=> use #3 2 branch @ 10" o.c.

Total Load Deflection:

$$I = \frac{bh^3}{12} = \frac{(18)(27)^3}{12} = 29524 \text{ in}^4$$

$$W_{TL} = \underbrace{226 \text{ PSF} \left(\frac{21}{2}\right)}_{\text{DL + LL from Slab}} + \underbrace{\left(\frac{27 \times 18}{144}\right) 115 \text{ PCF}}_{\text{Self weight of beam}} = 2761 \text{ PLF}$$

$$\Delta_{TL} = \frac{5(2.761)(30)^4(1728)}{384(4415)(29524)} = 0.39'' \leq \frac{l}{240} = 1.5''$$

$\therefore$ OK

Live Load Deflection:

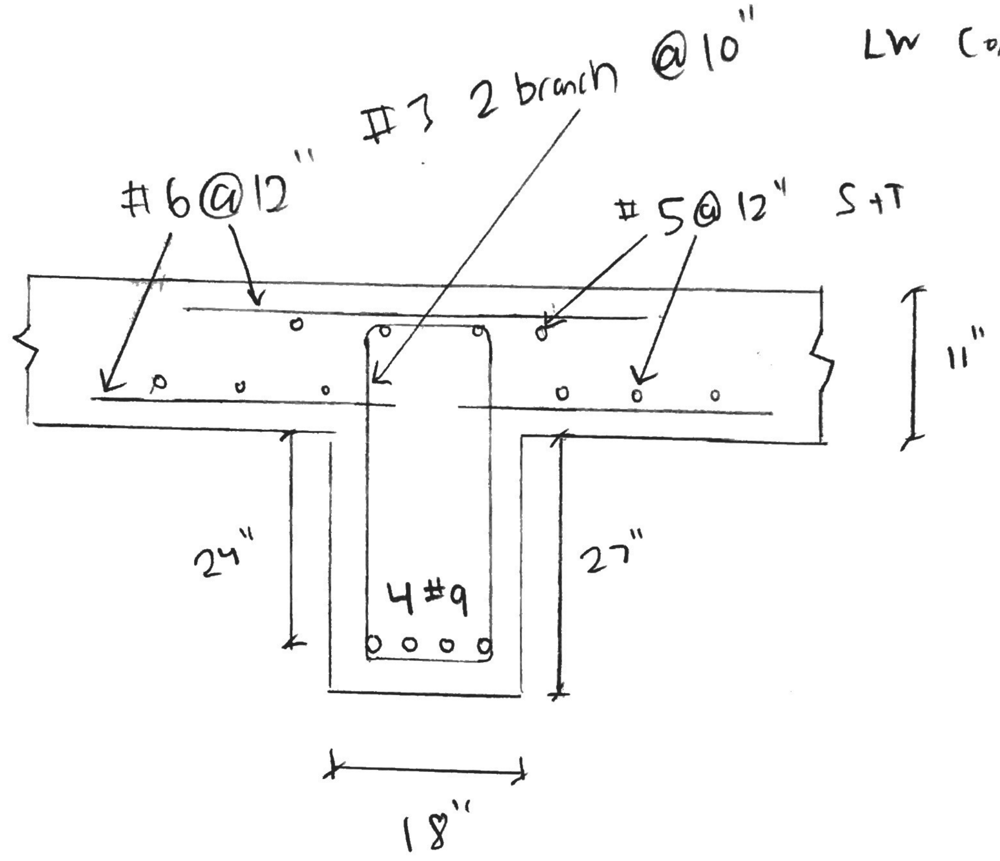
$$W_{LL} = 100 \text{ PSF} \left(\frac{21}{2}\right) = 1050 \text{ PLF}$$

$$\Delta_{LL} = \frac{5(1.050)(30)^4(1728)}{384(4415)(29524)} = 0.15'' \leq \frac{l}{360} = 1''$$

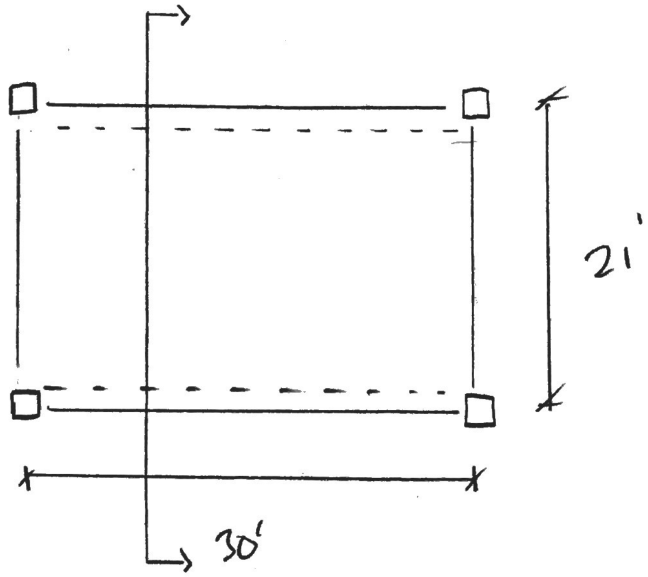
$\therefore$ OK

Final Design Layout

f'c = 3500 psi
fy = 60000 psi
LW Concrete



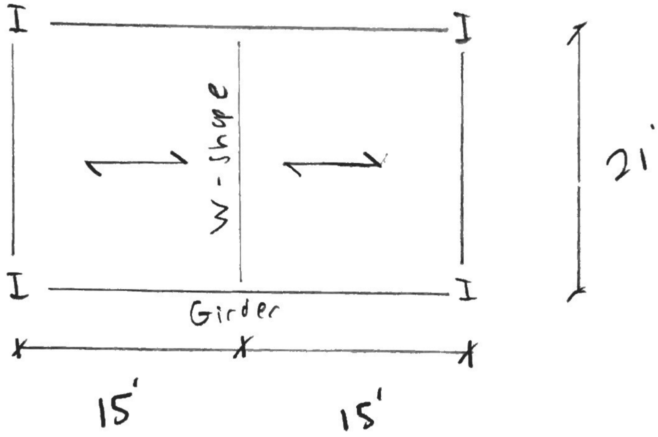
Section View Along edge beam



6.3 Alternate Design #3: Hollow Core Plank on Wide Flanges

The final design will be a hollow core plank slab on wide flanges. The hollow core plank was designed using Nitterhouse Prestressed Nicore Planks. The specification for the design used is included at the end of the section.

Design #3 : Hollow core Plank on wide Flanges



Misc DL = 20 PSF
Live = 100 PSF

From Nitterhouse => 6" x 4'-0 Prestressed Concrete Plank
1 Hour Fire Resistance Rating (Untopped)

Self Weight = 48.75 PSF
Superimposed DL = 20 PSF
Live Load = 100 PSF

$W_u = 1.2(20) + 1.6(100) = 184$ PSF

6 1/2" Ø Strands w/ max span = 15'

Safe Superimposed Service Loads = 273 PSF > 184 PSF ∴ OK

Live Load Deflection:

$W_{LL} = 100$ PSF $(4') = 400$ PLF

$E = 57000\sqrt{f'c} = 57000\sqrt{6000} = 4415$ Ksi

$I = 757$ in^4

$$\Delta_{LL} = \frac{5(0.400)(15)^4(1728)}{384(4415)(757)} = 0.14'' < \frac{l}{360} = \frac{15(12)}{360} = 0.5''$$

∴ OK

Total Load Deflection:

$W_{TL} = (20+100)(4') = 480$ PLF

$$\Delta_{TL} = \frac{5(0.480)(15)^4(1728)}{384(4415)(757)} = 0.16'' < \frac{\ell}{240} = 0.75''$$

$\therefore$ OK

Flexure check:

$W_u = 184$ PSF$(4') = 736$ PLF

$$M_u = \frac{736(15)^2}{8} = 20.7^{'K}$$

$M_{ult} = 67.2^{'K} \geq M_u = 20.7^{'K} \therefore$ OK

W-Shape Design

Live Load Reduction:

$K_{LL}A_T = (15+15)(21) = 630 \text{ ft}^2 > 400 \text{ ft}^2$

$$L_o = 100 \times \left| \begin{matrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{630}} = 0.848 \end{matrix} \right. = 84.8 \text{ PSF}$$

Live Load Deflection: $\Rightarrow \ell/360$

$W_{LL} = 84.8$ PSF$(15') = 1.272$ Klf

$$I_{req} = \frac{5(1.272)(21)^4(1728)}{384(29000)I} \leq \frac{21(12)}{360} = 0.7''$$

$I_{req} \geq 274 \text{ in}^4$

Total load Deflection => $\ell/240$

$w_{TL} = (20 + 48.75 + 84.8)(15') = 2.303$ Klf

$$I_{req} = \frac{5(2.303)(21)^4(1728)}{384(29000)\,I} \le \frac{21(12)}{240} = 1.05''$$

$I_{req} \geq 331 \text{ in}^4$

Try W 16x31 $I = 375 \text{ in}^4$

Check Flexure:

$$W_u = \begin{cases} 1.4(20 + 48.75) = 96.25 \text{ PSF} \\ 1.2(20 + 48.75) + 1.6(84.8) = 218.18 \text{ PSF} \end{cases}$$

$w_u = 218.18(15) = 3273$ PLF

$$M_u = \frac{3273(21)^2}{8} = 180.4^{'k}$$

ϕM_n (Table 3-2) $= 203^{'k} > M_u = 180.4^{'k}$ ∴ OK

=> Use W 16x31 W-Shape

Girder Design

Live Load: 69.3 PSF (Same as W 21 x 50 Girder check)

Live Load Deflection:

$$P_{LL} = 69.3(15)\left(\frac{21'}{2}\right) + 69.3(15)\left(\frac{19}{2}\right) = 20.8^{K}$$

$$\Delta_{LL} = \frac{20.8(30)^3(1728)}{48(29000)I} \le 1''$$

$$I_{req} \ge 697 \text{ in}^4$$

Total Load Deflection:

$$P_{TL} = (20 + 48.75 + 69.3)(15)\left[\frac{21}{2} + \frac{19}{2}\right] = 41.4^{K}$$

$$\Delta_{TL} = \frac{41.4(30)^3(1728)}{48(29000)I} \le 1.5''$$

$$I_{req} \ge 925 \text{ in}^4$$

Try W 21x55 $I = 1140 \text{ in}^4$

Flexure check:

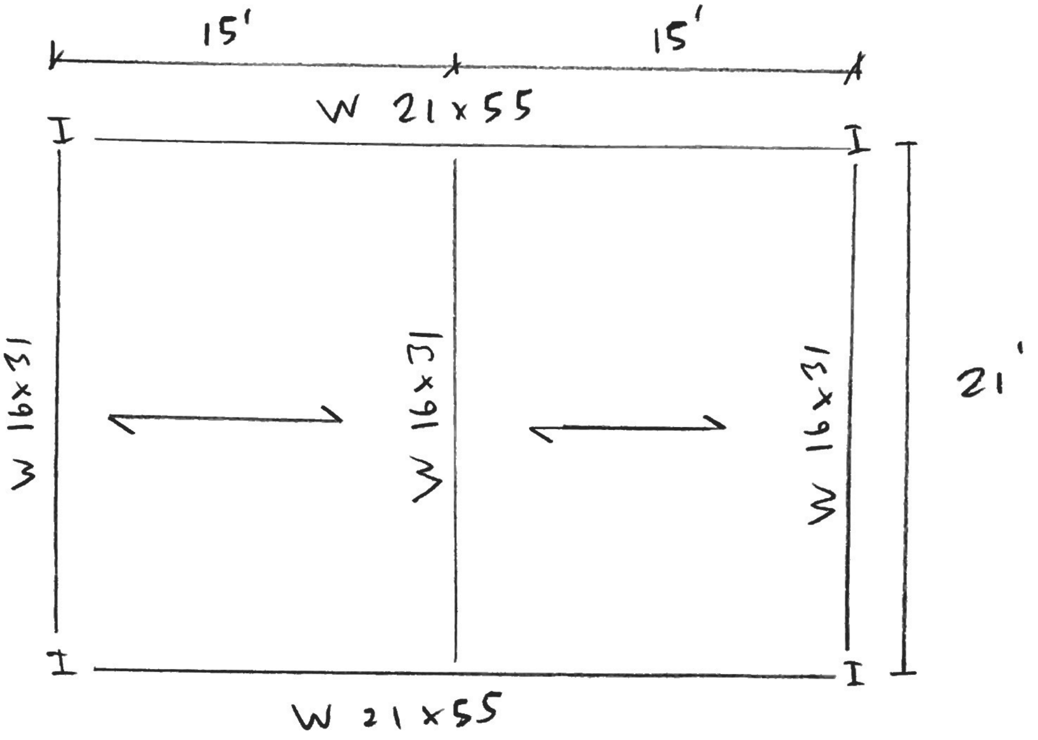
$$W_u = \left| \begin{array}{l} 1.4(20+48.75) = 96.25 \text{ PSF} \\ 1.2(20+48.75) + 1.6(69.3) = 193.4 \text{ PSF} \end{array} \right.$$

$$P_u = 193.4(15)\left(\frac{21}{2}\right) + 193.4(15)\left(\frac{19}{2}\right) = 58.0^{K}$$

$$M_u = \frac{58.0(30)}{4} = 435'^{K} < \phi M_n = 473'^{K} \therefore OK$$

$\Rightarrow$ Use W 21x55 Girder

Final Design Layout



Slab = 6" x 4'-0 Prestressed Concrete NiCore Plank

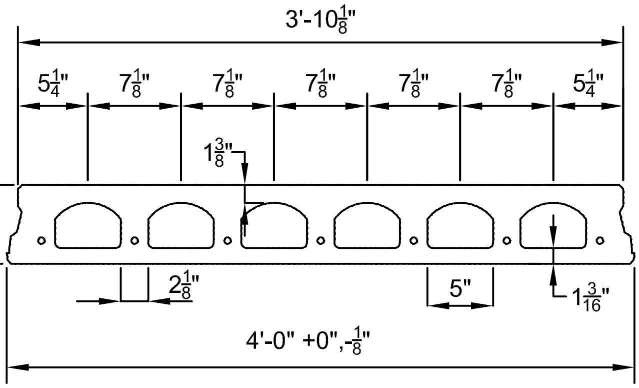
1 Hour Fire Resistance Rating (Untopped)

Prestressed Concrete
6"x4'-0" NiCore Plank

1 Hour Fire Resistance Rating (Untopped)

PHYSICAL PROPERTIES
Precast

A = 187 in.^{2}	b_w = 16.13 in.
I = 757 in.^{4}	S_b = 245 in.^{3}
Y_b = 3.09 in.	S_t = 260 in.^{3}
Y_t = 2.91 in.	Wt.= 195 PLF
e = 1.34 in.	Wt.= 48.75 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3800 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   7-3/8"Ø, 270K = 46.4 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 67.2 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 75.5 k-ft at 60% jacking force
7. Maximum bottom tensile stress is 10 $\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Load values to the left of the solid line are controlled by ultimate shear strength.
12. Load values to the right are controlled by ultimate flexural strength or allowable service stresses.
13. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2012 & ACI 318-11 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
6 - 1/2"ø	LOAD (PSF)	353	322	295	273	244	215	197	175	155	149	132	118	104	92	81	73	64	57	50
7 - 1/2"ø	LOAD (PSF)	407	372	341	303	269	244	226	202	183	166	149	133	118	105	94	83	74	66	59

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box 2013
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 1 Hour & 0 Minute fire resistance rating.

03/25/14

6F1.0

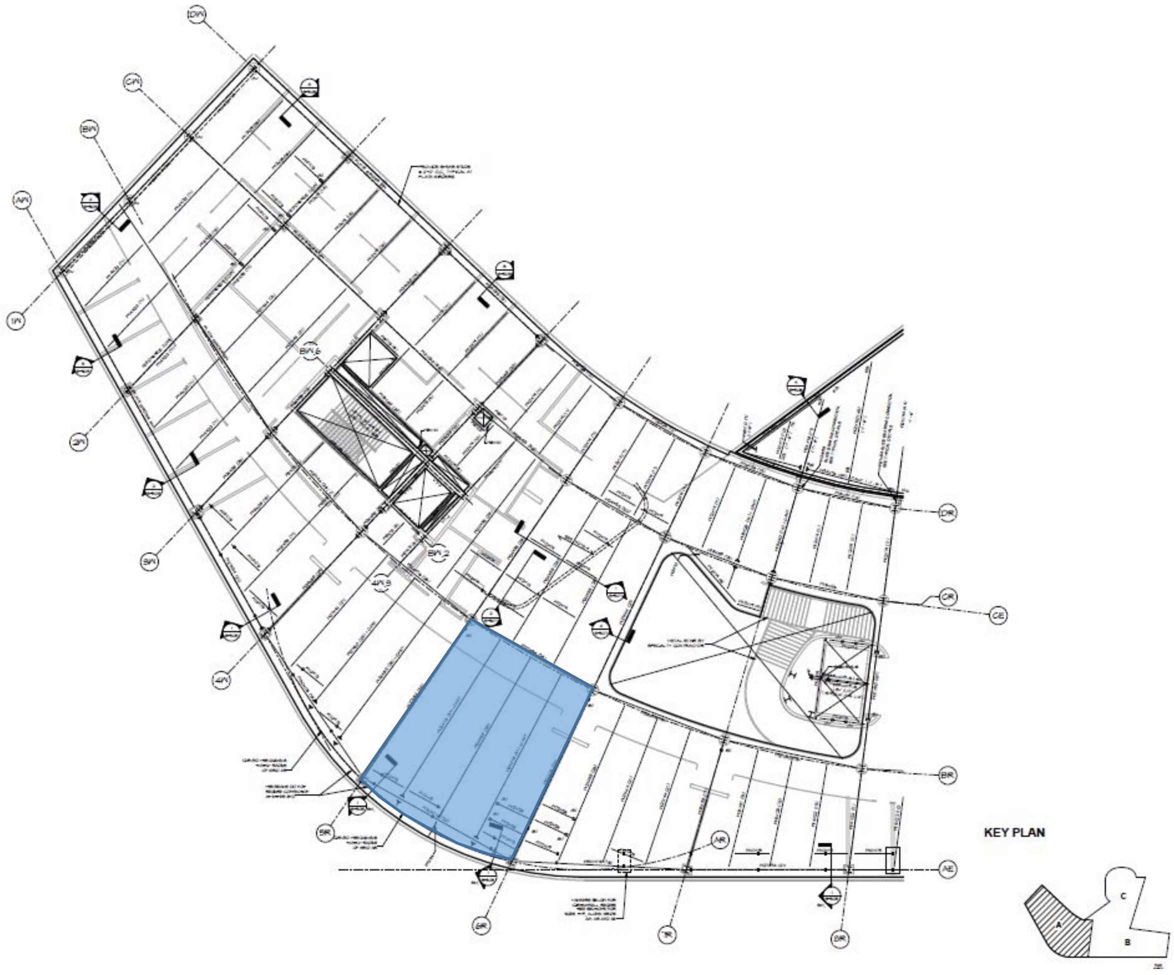
7. Systems Comparison

Considerations	Composite Steel Framing	Non-Composite Steel Framing	One- Way Slab	Hollow Core Plank on Wide Flanges
Architectural				
Depth	27"	30"	11"	27"
Fire Rating	2 Hour	2 Hour	2 Hour	1 Hour
Construction Information				
Cost/SF	$7.53	$7.60	$5.96	$7.17
Weight	57.0 PSF	65.7 PSF	142.4 PSF	57.1 PSF
Future Design Considerations				
Advantages	Lightweight, fairly cheap, minimal formwork	Lightweight, fairly cheap, minimal formwork	Smallest depth, cheapest option, minimal vibrations	Lightweight, fairly cheap, faster construction
Disadvantages	Large Depth, vibration	Largest depth, vibration	Largest weight, requires most formwork	Large depth, difficult to fit rectangular panels in irregular shaped bays
Further Research	N/A	Yes	Yes	No

Analyzing the four different systems shows that composite framing is the best option for this project as it is one of the cheaper, lightweight options that allows for an irregular layout. Moving forward, non-composite framing and one-way slab could be viable options as non-composite framing could reduce vibrations due to the larger depth while one way slab is the cheapest and smallest depth. The hollow core plank on wide flanges does not appear to be a viable option due to difficult constructability because of the building layout.

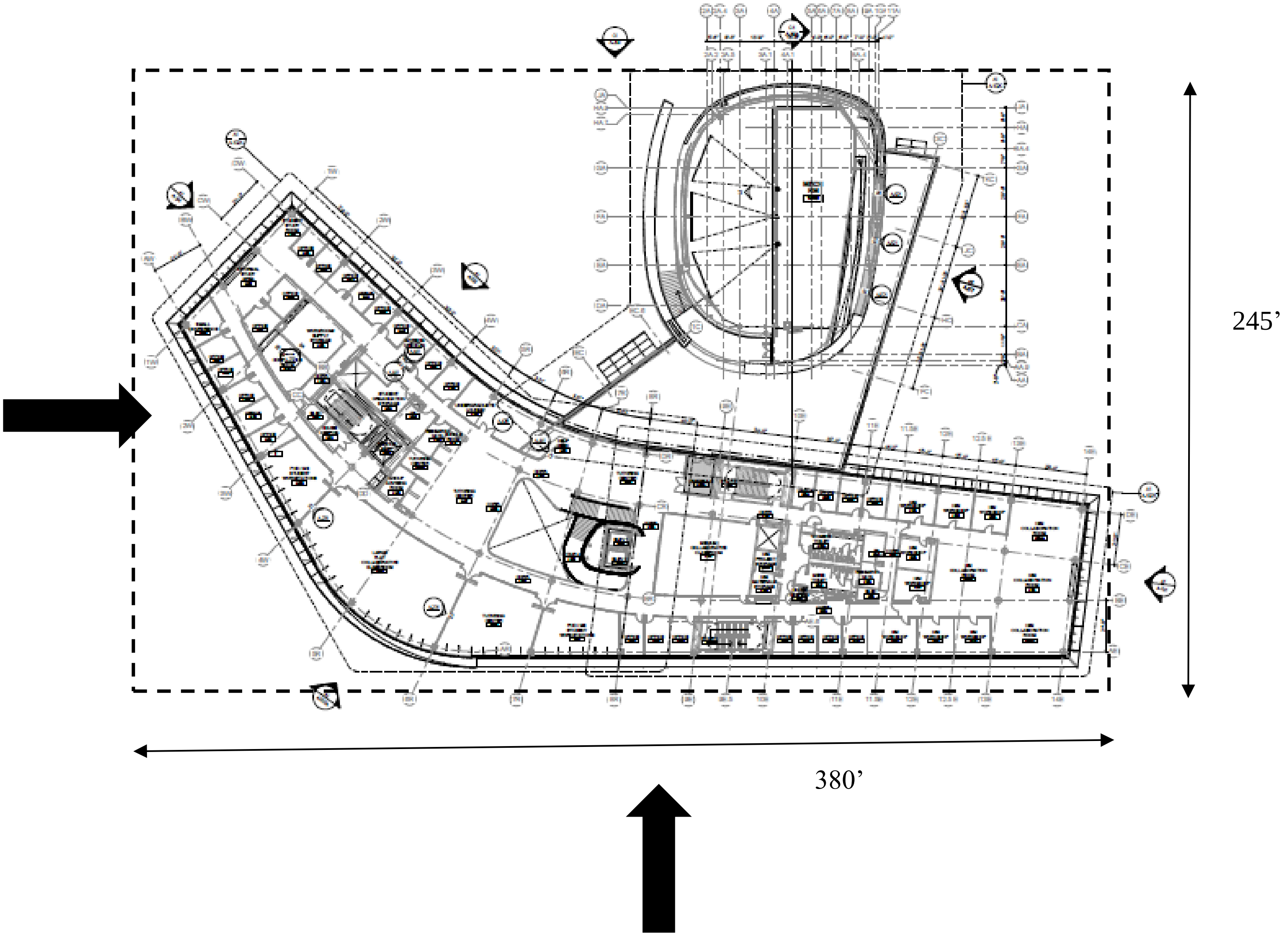
Appendix A

The highlighted bay was used for determination of gravity loads at a typical floor and the roof. This bay was used because it has the largest spans throughout the building, which results in a higher dead load and is thus more conservative.



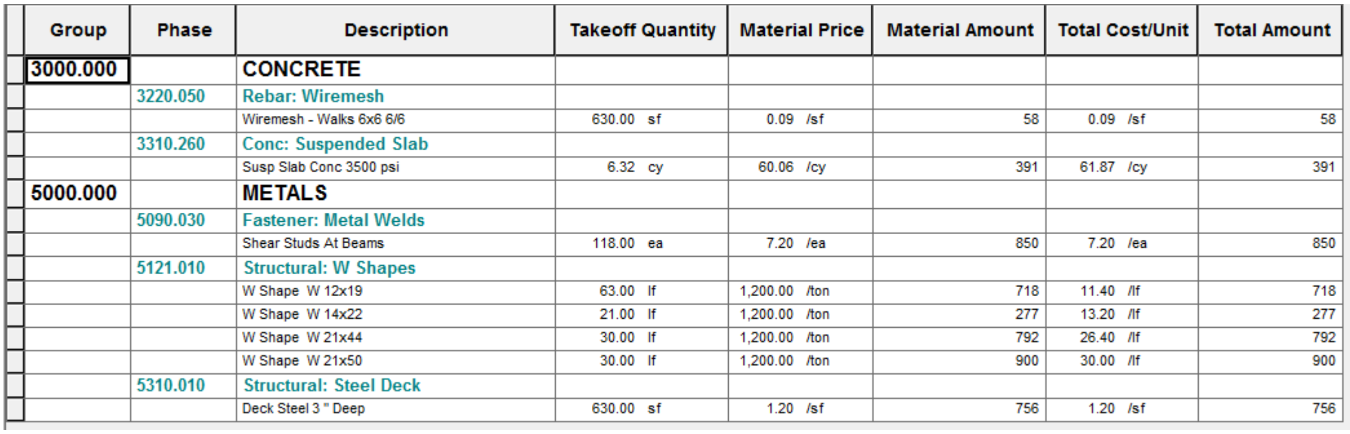
Appendix B

This diagram shows the orientation of the direction that the wind load was applied. Due to the irregular shape of the building, the buildings largest dimensions were used to yield a more conservative analysis.



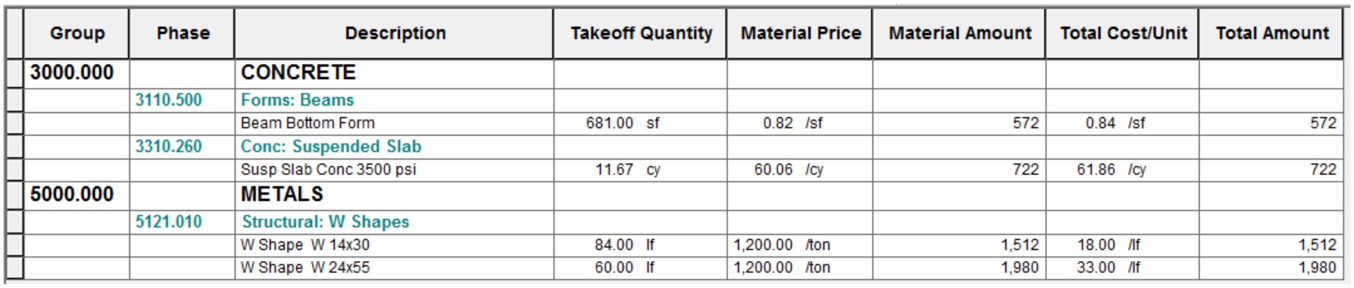
Appendix C- Cost Estimate

Composite Framing



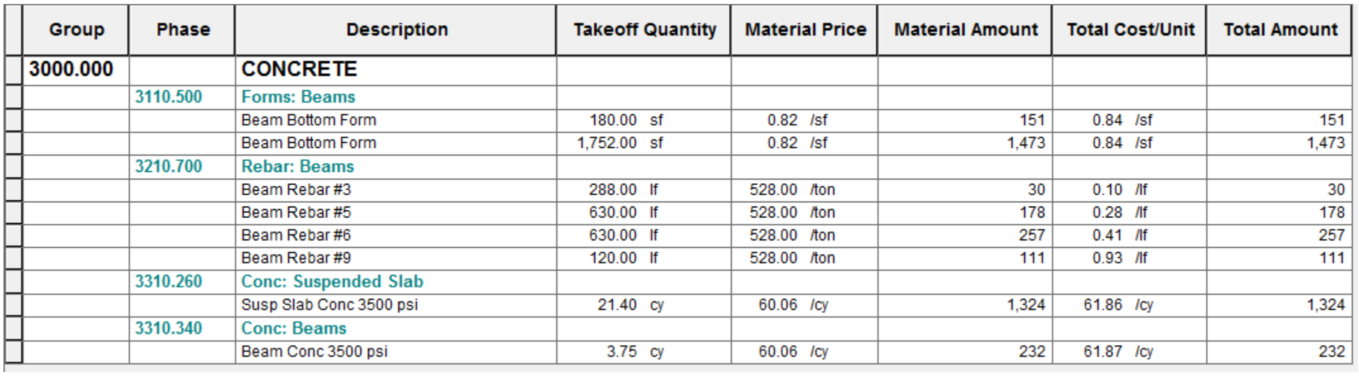
Group	Phase	Description	Takeoff Quantity	Material Price	Material Amount	Total Cost/Unit	Total Amount
3000.000		**CONCRETE**					
	3220.050	Rebar: Wiremesh					
		Wiremesh - Walks 6x6 6/6	630.00 sf	0.09 /sf	58	0.09 /sf	58
	3310.260	Conc: Suspended Slab					
		Susp Slab Conc 3500 psi	6.32 cy	60.06 /cy	391	61.87 /cy	391
5000.000		**METALS**					
	5090.030	Fastener: Metal Welds					
		Shear Studs At Beams	118.00 ea	7.20 /ea	850	7.20 /ea	850
	5121.010	Structural: W Shapes					
		W Shape W 12x19	63.00 lf	1,200.00 /ton	718	11.40 /lf	718
		W Shape W 14x22	21.00 lf	1,200.00 /ton	277	13.20 /lf	277
		W Shape W 21x44	30.00 lf	1,200.00 /ton	792	26.40 /lf	792
		W Shape W 21x50	30.00 lf	1,200.00 /ton	900	30.00 /lf	900
	5310.010	Structural: Steel Deck					
		Deck Steel 3 " Deep	630.00 sf	1.20 /sf	756	1.20 /sf	756

Non-Composite Framing



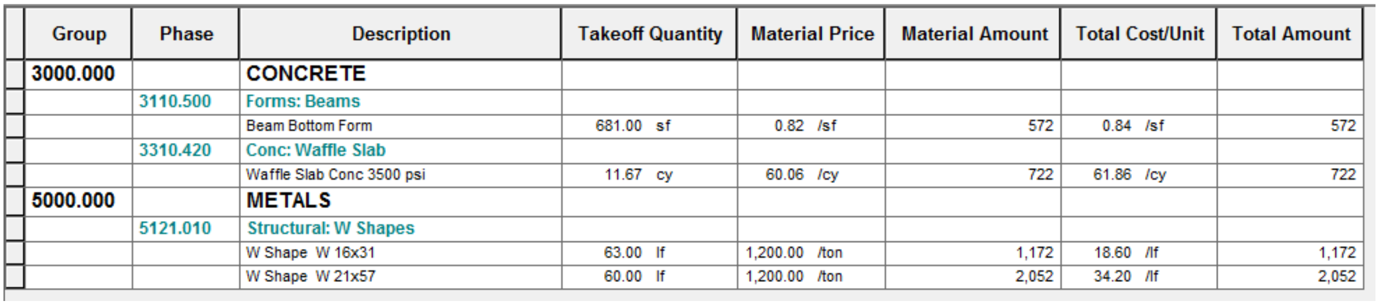
Group	Phase	Description	Takeoff Quantity	Material Price	Material Amount	Total Cost/Unit	Total Amount
3000.000		**CONCRETE**					
	3110.500	Forms: Beams					
		Beam Bottom Form	681.00 sf	0.82 /sf	572	0.84 /sf	572
	3310.260	Conc: Suspended Slab					
		Susp Slab Conc 3500 psi	11.67 cy	60.06 /cy	722	61.86 /cy	722
5000.000		**METALS**					
	5121.010	Structural: W Shapes					
		W Shape W 14x30	84.00 lf	1,200.00 /ton	1,512	18.00 /lf	1,512
		W Shape W 24x55	60.00 lf	1,200.00 /ton	1,980	33.00 /lf	1,980

One- Way Slab



Group	Phase	Description	Takeoff Quantity	Material Price	Material Amount	Total Cost/Unit	Total Amount
3000.000		CONCRETE					
	3110.500	Forms: Beams					
		Beam Bottom Form	180.00 sf	0.82 /sf	151	0.84 /sf	151
		Beam Bottom Form	1,752.00 sf	0.82 /sf	1,473	0.84 /sf	1,473
	3210.700	Rebar: Beams					
		Beam Rebar #3	288.00 lf	528.00 /ton	30	0.10 /lf	30
		Beam Rebar #5	630.00 lf	528.00 /ton	178	0.28 /lf	178
		Beam Rebar #6	630.00 lf	528.00 /ton	257	0.41 /lf	257
		Beam Rebar #9	120.00 lf	528.00 /ton	111	0.93 /lf	111
	3310.260	Conc: Suspended Slab					
		Susp Slab Conc 3500 psi	21.40 cy	60.06 /cy	1,324	61.86 /cy	1,324
	3310.340	Conc: Beams					
		Beam Conc 3500 psi	3.75 cy	60.06 /cy	232	61.87 /cy	232

Hollow Core Plank on Wide Flanges



Group	Phase	Description	Takeoff Quantity	Material Price	Material Amount	Total Cost/Unit	Total Amount
3000.000		CONCRETE					
	3110.500	Forms: Beams					
		Beam Bottom Form	681.00 sf	0.82 /sf	572	0.84 /sf	572
	3310.420	Conc: Waffle Slab					
		Waffle Slab Conc 3500 psi	11.67 cy	60.06 /cy	722	61.86 /cy	722
5000.000		METALS					
	5121.010	Structural: W Shapes					
		W Shape W 16x31	63.00 lf	1,200.00 /ton	1,172	18.60 /lf	1,172
		W Shape W 21x57	60.00 lf	1,200.00 /ton	2,052	34.20 /lf	2,052