Letter of Transmittal

November 14, 2016

Dr. Aly Said

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University Park, PA 16802

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Dear Dr. Said,

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

This report includes a comparison of hand calculations and RAM output for center of rigidity/mass calculations, total shear into each lateral frame, and wind/seismic loads. After validating the computer model, spot checks are performed to determine serviceability.

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 C

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.

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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.

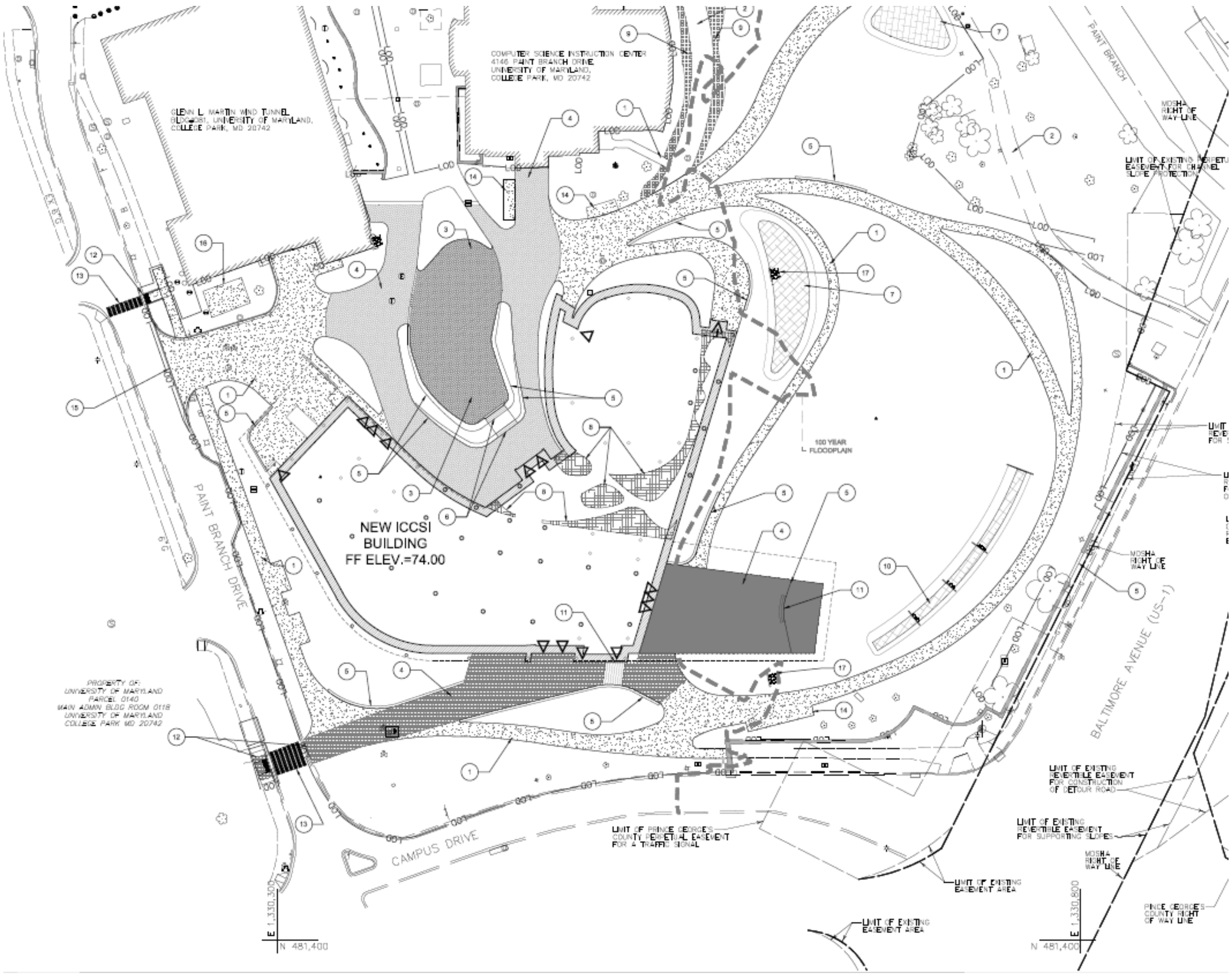


Figure 1: Site Plan

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

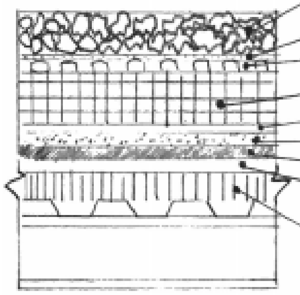
2.1 Roof Loads

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 6^{th} Floor



Dead Loads

Gravel = 6 PSF

Filter Fabric = Negligible

Drainage Board with root block = 3 PSF

6" High Density Rigid Insulation = 0.75 per ½" = 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

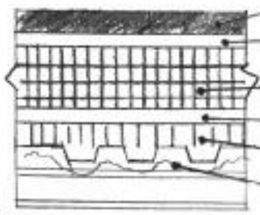
Total Dead = 154 PSF

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

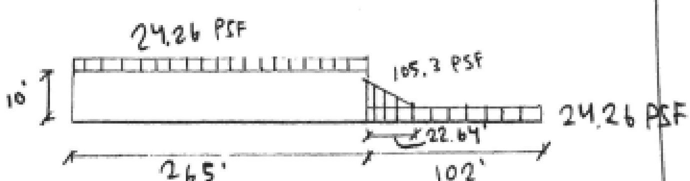
$h_c = 10' - 1.3' = 8.7'$ $\quad \frac{h_c}{h_b} = \frac{8.7}{1.3} = 6.7 > 0.2$ ∴ drift

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

$p_d = h_d \gamma$

$= 5.66(18.6)$

$= 105.3$ 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

$h_b = 1.3'$

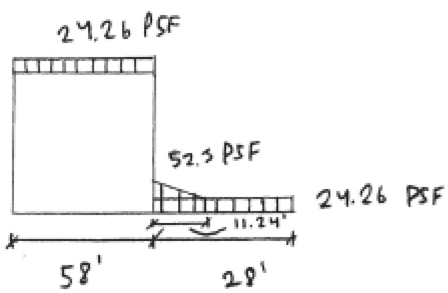
$h_c = 68' - 1.3' = 66.7'$

$h_d < h_c$ -> $w = 4h_d = 4(2.81) = 11.24'$

$p_d = h_d\gamma$

$= (2.81)(18.6)$

$= 52.3$ PSF



2.3 Floor Loads

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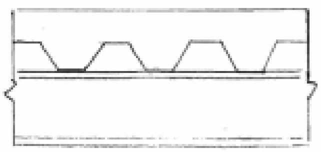
Floor Loads

Typical Floor (Ground Floor to 6^{th} 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

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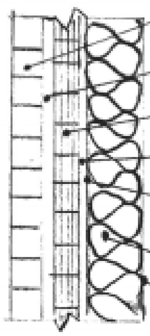
Exterior Wall at Auditorium



4" Nominal Brick
1 1/2" Air Space
3" Polystyrene Insulation
Self-adhering vapor resistive air barrier
5/8" Glass Fiber Gyp Board
R-25 Batt Insulation
8" CFMF

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

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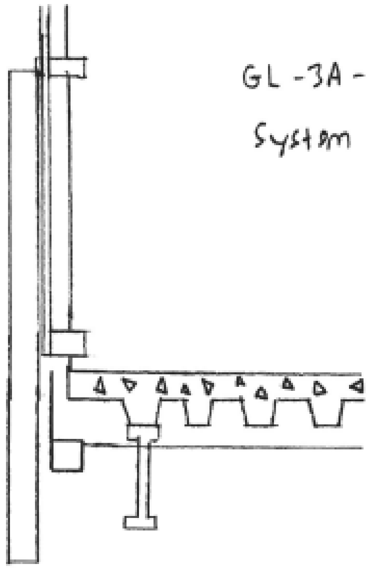
Exterior Wall at North Facade



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

Dead Load = 15 PSF × 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
η_h	2.765
R_h	0.296
η_B	8.855
R_B	0.107
η_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

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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:

$C_{p\,winward} = 0.8$

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

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

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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)	$P_{windward}$	$P_{leeward}$	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

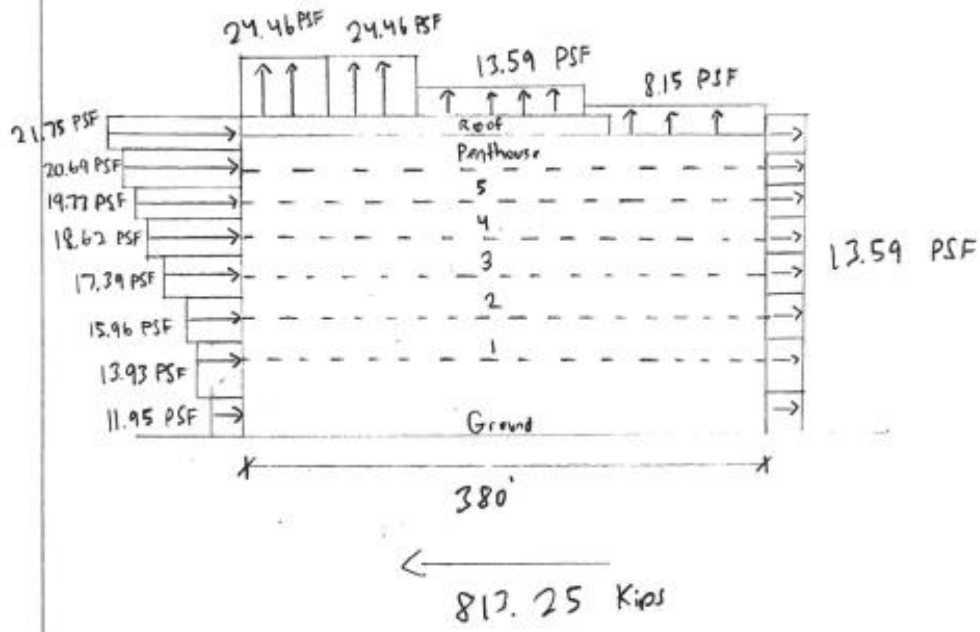
East-West Direction L = 380' B = 245'

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

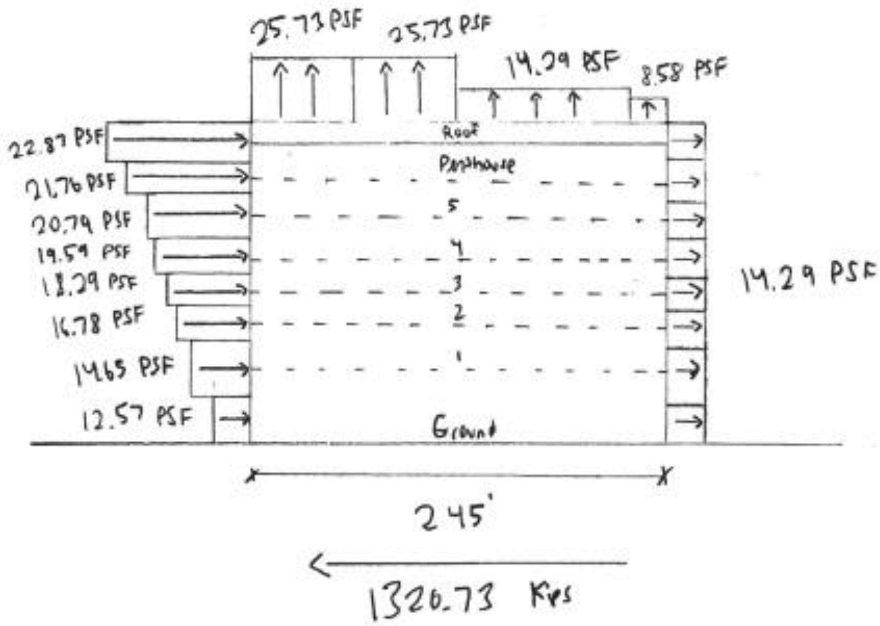
	z (ft)	q_z (psf)	$P_{windward}$	$P_{leeward}$	P_{roof}	Trib Height	Trib Weight	Story Force
Ground	0	17.86	12.57	-14.29		12.75	380	130.17
1	25.5	20.81	14.65	-14.29		20.085	380	220.87
2	40.17	23.84	16.78	-14.29		14.67	380	173.24
3	54.84	25.99	18.29	-14.29		14.67	380	181.66
4	69.51	27.83	19.59	-14.29		14.67	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 11.1.2)

Site Class D (Sheet S-001)

$S_s = 0.119\,g$ $\quad S_{MS} = 0.190\,g$ $\quad S_{DS} = 0.127\,g$

$S_1 = 0.051\,g$ $\quad S_{M1} = 0.122\,g$ $\quad S_{D1} = 0.081\,g$ $\Big\}$ USGS

Seismic Design Category B (Section 11.6)
Risk Category III

Equivalent Lateral Force Analysis Permitted (Section 12.6)

Ordinary Braced Frame -> $R = 3$ (B-12)
Ordinary Moment Frame -> $R = 3\frac{1}{2}$ (C-4) $\Big\}$ Table 12.2-1

$\therefore$ 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)

Brendan 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 (ft²)	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 (ft²)	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

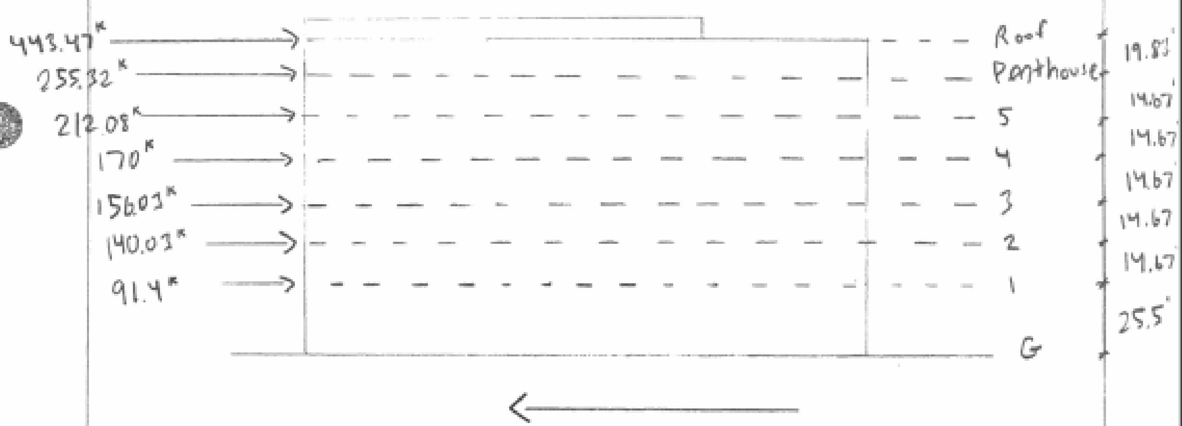
Vertical Distribution of Forces: (Section 12.8.3)

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

$T_a = 0.81 \rightarrow k = 1.155$ 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

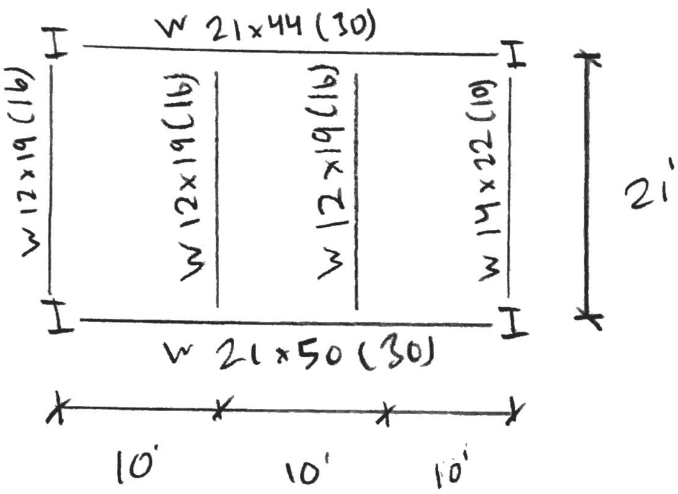
* Seismic Base Shear is the same in both directions

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 12x19 (16) Infill Beam Check

Live Load Reduction:

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

$L_o = 100 \times \max \begin{vmatrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{420}} = 0.982 \end{vmatrix} = 98.2\ PSF$

$W_u = \begin{vmatrix} 1.4(68) = 95.2\ PSF \\ 1.2(68) + 1.6(98.2) = 238.7\ PSF \end{vmatrix}$

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

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

Check Composite Strength

$b_{eff} = \min \begin{vmatrix} 21(12)/4 = 63 \Leftarrow \text{controls} \\ 10(12) = 120 \end{vmatrix}$

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

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

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

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

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

$\therefore$ NA is in web

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

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

$\phi M_n = 186.2^{'k} > M_u = 131.6^{'k} \quad \therefore$ OK

Check Unshored Strength

$W_u = 1.4(48)(10') + 1.4(19) = 0.6986$ klf

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

Construction LL

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

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

Wet concrete deflection

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

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

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

$\therefore$ OK

Live Load Deflection

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

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

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

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

$\therefore$ OK

=> W 12x19 (16) Infill Beam is OK

W 21 x 50 (30) Girder Check

Live Load Reduction:

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

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

$W_u = 1.4(68)$

$1.2(68) + 1.6(69.3) = 192.5$

Point Loads from Infill Beams

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

$M_u = Pa = 38.5^k(10') = 385^{1k}$

Check Composite Strength

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

Check Shear Stud Capacity

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

$A_s F_y = (14.7\ in^2)(50\ ksi) = 735^k$

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

$\therefore$ Partially composite

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

$\therefore$ NA is in web

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

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

$$\phi M_n = 671.9^{1k} \geq M_u = 385^{1k} \therefore OK$$

Check unshored strength

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

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

$$M_u = \frac{1.852(30)^2}{8} = 208.3^{1k}$$

$$\phi M_n \text{ (Table 3-2)} = 413^{1k} > 208.3^{1k}$$ $\therefore$ OK for unshored strength

Wet concrete deflection

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

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

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

Live Load Deflection

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

I_{LB}

		Y_2	
Y_1	5.5	5.77	6
0.535	2260	2308	2350
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 21 x 44 (30) Girder Check

Live Load Reduction

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

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

$W_u = 1.4(68) = 95.2\ PSF$

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

Point Loads from Infill Beams:

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

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

Check composite strength

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

Check shear stud capacity:

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

$A_s F_y = (13.0\ in^2)(50\ ksi) = 650^K$

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

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

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

$\therefore$ NA is in web

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

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

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

Check unshored strength

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

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

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

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

Wet Concrete Deflection

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

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

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

$\therefore$ OK

Live Load Deflection

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

I_{LB}

		5	5.32 (Y_2)	5.5
	0.450	1990	1974.8	2000
Y_1	0.6		1961.7	
	2.92	1720	1758.4	1780

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

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

$\therefore$ 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:

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

$$K_{LL} = 3 \Rightarrow K_{LL} A_T = 1095\ ft^2 > 400\ ft^2$$

For exterior column

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

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:

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

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

for interior column

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

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

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

$= 766^K$

W 12 x 106 unbraced length ≈ 15'

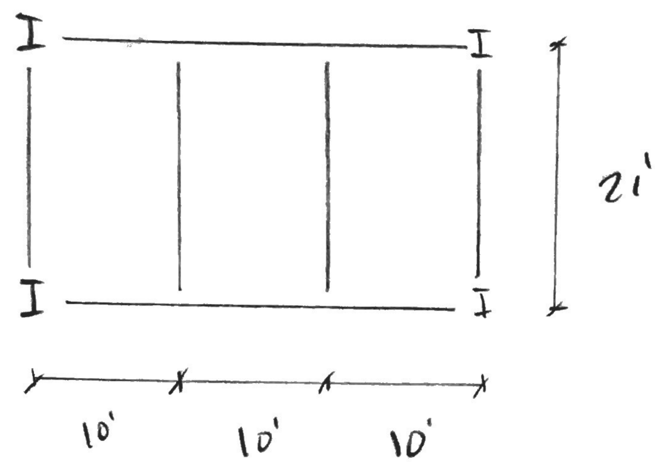
$\phi P_n = 1100^K > 766^K$ ∴ OK

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 $\geq$ 10'

Try 3C18 w/ 3" NW concrete

Max construction span = 14'-2" $\geq$ 10' $\therefore$ OK

Total Load = 100 + 57 + 10 + 10 = 177 PSF

LL Slab M/E/C/L Framing

Allowable load @ 10' = 193 PSF > 177 PSF $\therefore$ OK

W_{LL} = 100 PSF < $l/240$ = 155 PSF $\therefore$ 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 14 x 30 $\quad I = 291 \text{ in}^4$

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 W14×30 Infill Beams

<u>Spandrel Girder Design</u>

Live Load Deflection:

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

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

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

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

Total Load Deflection:

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

$$P_{TL} = (77 + 84.8)(10')(\tfrac{21}{2}') = 17.0^{k}$$

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

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

Try W 21 x 44. $I = 843 \text{ in}^4$

Check Flexure

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

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

$$P_u = 228.1(10')(\tfrac{21}{2}') = 23.95^{k}$$

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

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

$\Rightarrow$ use W 21 x 44 Spandrel girder

Girder Design

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

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

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

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

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

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

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

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

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

Flexure check

$$W_u = \begin{cases} 1.4(77) = 107.8 \text{ PSF} \\ 1.2(77) + 1.6(69.3) = 203.28 \text{ PSF} \end{cases}$$

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

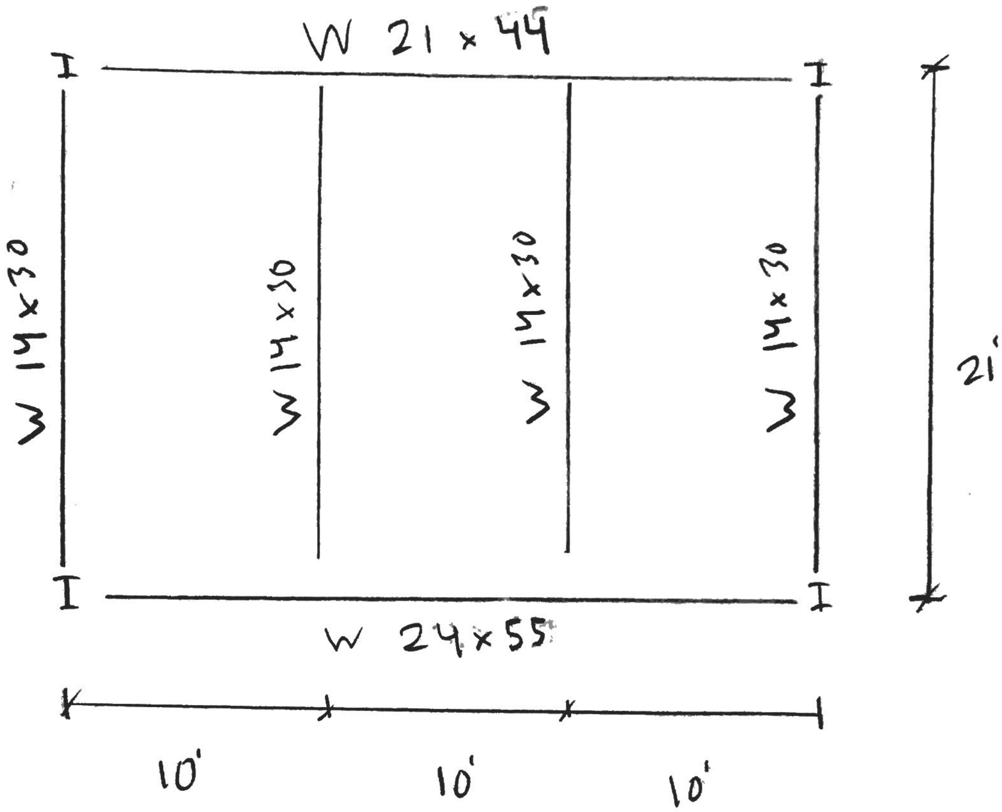
$$M_u = 40.6^k(10') = 406^{'k}$$

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

$\Rightarrow$ use W 24x55 Girder

Final Design Layout

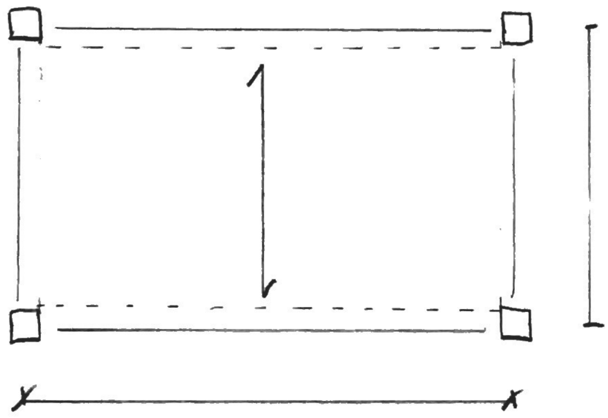
Deck = 3C18 w/ 3" NW concrete



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 => $t = \frac{l}{24} = \frac{21(12)}{24} = 10.5''$
(end bay)

$\therefore$ use 11" slab

$d = 11 - 0.75 - 0.5/2 = 10''$

0.75 = Clear cover; 0.5 = #4 bars

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

$$W_u = \begin{cases} 1.4(126) = 174.4 \text{ PSF} \\ 1.2(126) + 1.6(84.8) = 286.9 \text{ PSF} \end{cases}$$

unit strip method

$$M_u = \frac{286.9(1')(21)^2}{8} = 15.8^{1K}$$

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

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

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

$\therefore$ Steel yielded

tension controlled $\Rightarrow \phi = 0.9$

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

Minimum Reinforcement:

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

$\therefore$ OK

Max Spacing:

$$S_{max} = \min \begin{cases} 3h = 3(11) = 33'' \\ 18 = 18'' \Leftarrow \text{controls} \end{cases}$$

$\therefore$ OK

Max Spacing for Crack Control:

$$S = \begin{cases} 15\left(\frac{40000}{f_s}\right) - 2.5c_c = 15\left(\frac{40000}{\frac{2}{3}(60000)}\right) - 2.5(0.75) = 13.125 \\ 12\left(\frac{40000}{f_s}\right) = 12\left(\frac{40000}{\frac{2}{3}(60000)}\right) = 12'' \end{cases}$$

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

Check One way Shear:

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

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

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

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

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 $\phi = 0.9$

$$\phi M_n = \phi A_s F_y (d - \tfrac{a}{2})$$

$$= 0.9(0.44)(60)(9.88 - \tfrac{0.74}{2})$$

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

Shrinkage and Temperature Reinforcement:

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

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

$\Rightarrow$ Use #5 @ 12'' $A_s = 0.31\ in^2$

Beam Design

$$w_u = 286.9\ PSF\,(\tfrac{21'}{2}) = 3012.5\ PLF$$

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

Calculate tentative ρ

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

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

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

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

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

$= 0.65$ Ksi

$M_n = R b d^2$

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

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

$A_s req = \frac{M_u}{\emptyset F_y j d} = \frac{372.8 \times 12}{0.9(60)(0.95)(24)} = 3.63 \; in^2$

Use 4 #9 $A_s = 4.0 \; in^2$

Check Flexure:

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

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

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

$\therefore$ Tension Controlled $\emptyset = 0.9$

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

$= 391.7'^k > 372.8'^k \quad \therefore OK$

Check Shear:

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

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

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

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

$= 28.8^k < V_u$ ∴ need shear Reinforcement

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

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

$= 29.7^k$

check $8\sqrt{f'c}\, b_w d = 204^k > V_s$ ∴ ok

Solve for Stirrup Spacing using #3 2 branch ($0.22 in^2$)

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

∴ use 10'' spacing

$S_{max} = \min \left| \begin{array}{l} d/2 = 12'' \Leftarrow \text{govern} \\ 24 \end{array} \right.$

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

$\Rightarrow$ 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:

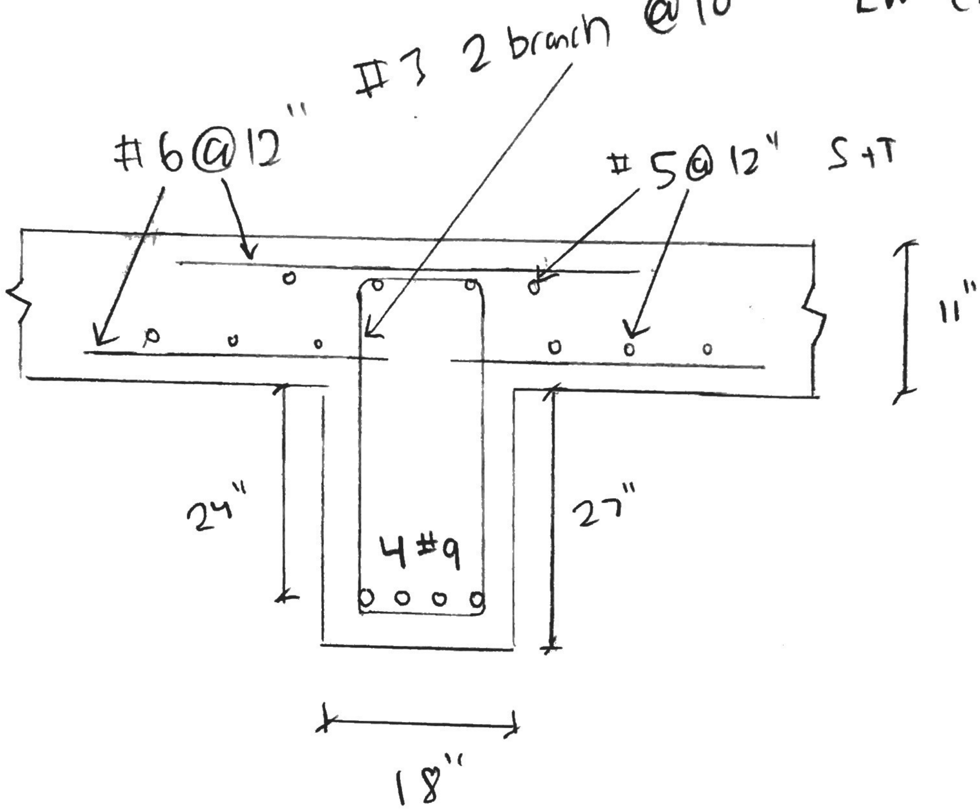
$$W_{LL} = 100 \text{ PSF} \left(\frac{21}{2}\right) = 1050 \text{ PLF}$$

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

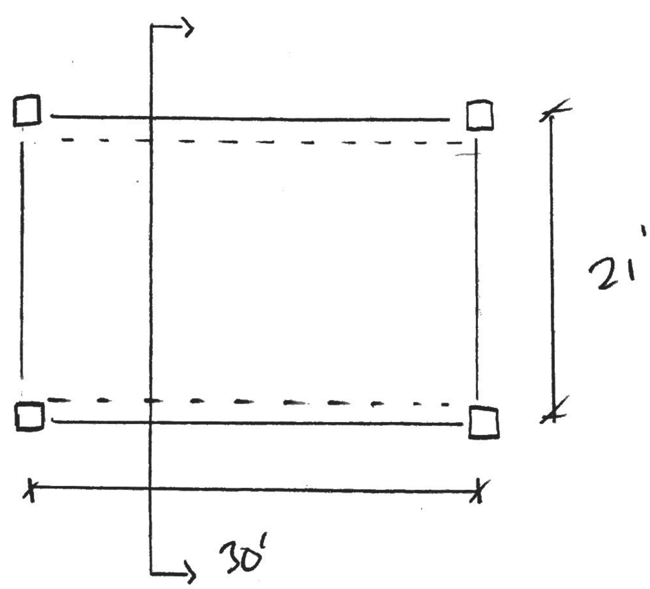
$\therefore$ OK

Final Design Layout

$f'_c = 3500$ psi
$f_y = 60000$ psi
LW Concrete



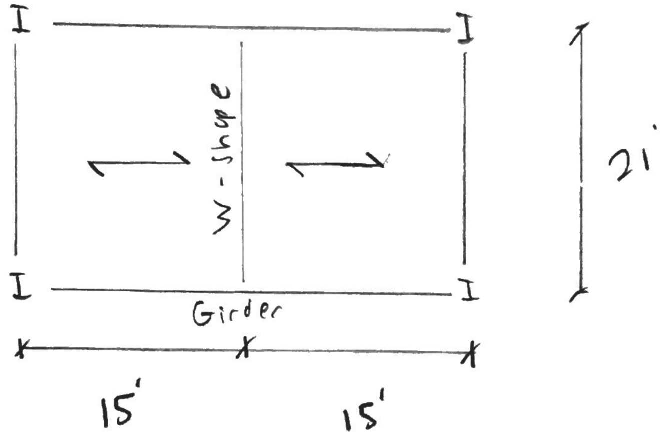
Section View Along edge beam



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:

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

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

$I = 757\text{ in}^4$

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

∴ 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

$$M_u = \frac{736(15)^2}{8} = 20.7^{1K}$$

$M_{ult} = 67.2^{1K} \geq M_u = 20.7^{1K}$ $\therefore$ OK

W-Shape Design

Live Load Reduction:

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

$$L_0 = 100 \times \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{630}} = 0.848 \end{array} \right. = 84.8 \text{ PSF}$$

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

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

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

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

Total load Deflection => $l/240$

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

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

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

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

$$M_u = \frac{3273(21)^2}{8} = 180.4^{1K}$$

ϕM_n (Table 3-2) $= 203^{1K} > M_u = 180.4^{1K}$ $\therefore$ OK

=> Use W 16x31 W-Shape

Girder Design

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

Live Load Deflection:

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

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

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

Total Load Deflection:

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

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

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

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

Flexure check:

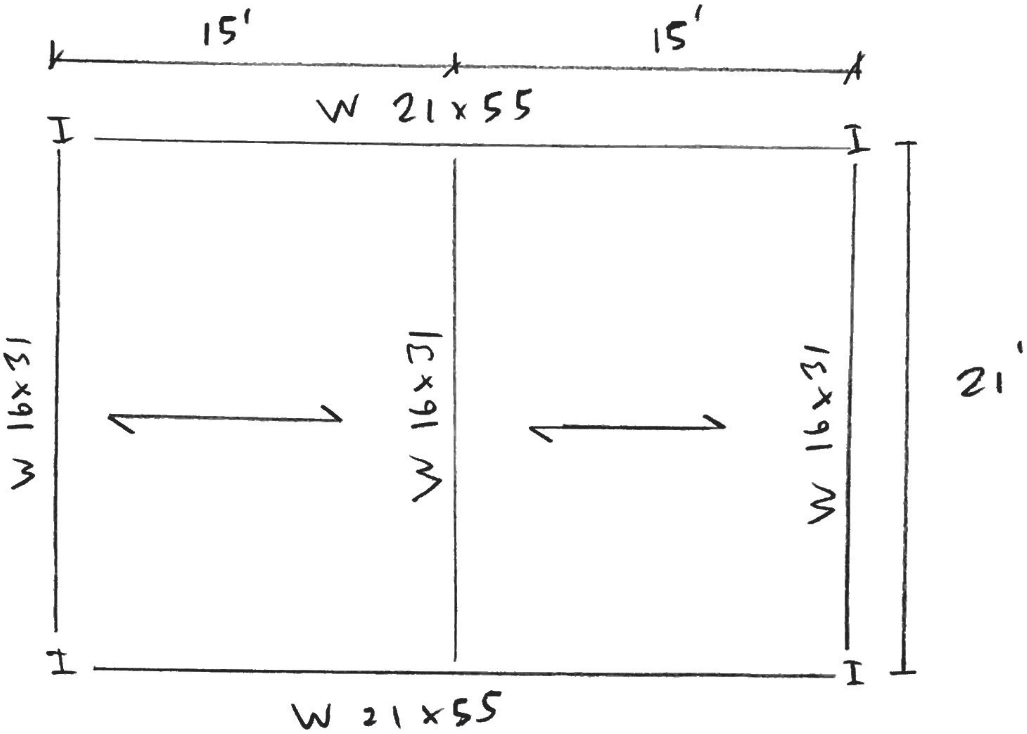
$W_u = \begin{cases} 1.4(20 + 48.75) = 96.25 \text{ PSF} \\ 1.2(20 + 48.75) + 1.6(69.3) = 193.4 \text{ PSF} \end{cases}$

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

$M_u = \frac{58.0(30)}{4} = 435^{1k} < \phi M_n = 473^{1k}$ ∴ OK

⇒ Use W 21x55 Girder

Final Design Layout



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

1 Hour Fire Resistance Rating (Untopped)

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.

8. Lateral Analysis

This section analyzes the existing lateral system in the Brendan Iribe Center for Computer Science and Innovation. RAM Structural System was used to create a computer model as shown in Figure 3 below. In addition to the RAM model, hand calculations have been performed to validate the accuracy of this computer model.

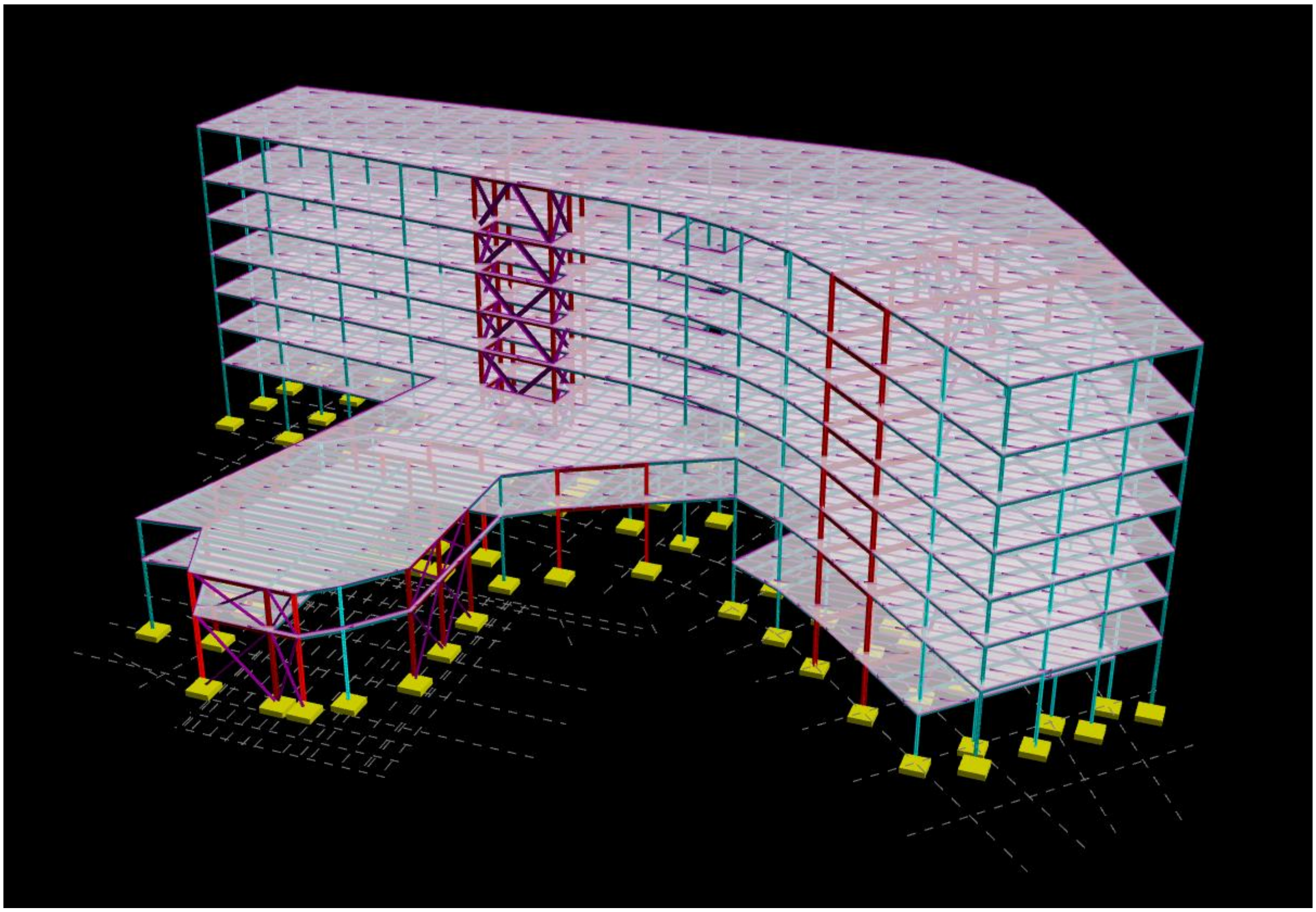


Figure 3: RAM model

8.1 Modeling Information

Several factors were considered in the creation of this model

Diaphragm:

8. Assigned to be a rigid diaphragm at each level
9. Uniform dead and live load assigned from Notebook Submission A
10. Self-weight of diaphragm included in RAM

Moment Frames:

- Beams and columns are fixed-fixed
- Column bases are fixed-fixed with spread footings
- Self-weight of framing included in RAM

Braced Frames:

- Braces are pinned-pinned to beams/columns
- Column bases are fixed-fixed with spread footings
- Self-weight of framing included in RAM

Figure 4 shows a plan of the lateral members throughout the building. There are 17 separate lateral members, including moment frames, braced frames, combination of the two. Frames 1-10 continue from the base to the roof, while frames 11-17 continue from the ground to the 2nd floor, which is the roof for the auditorium

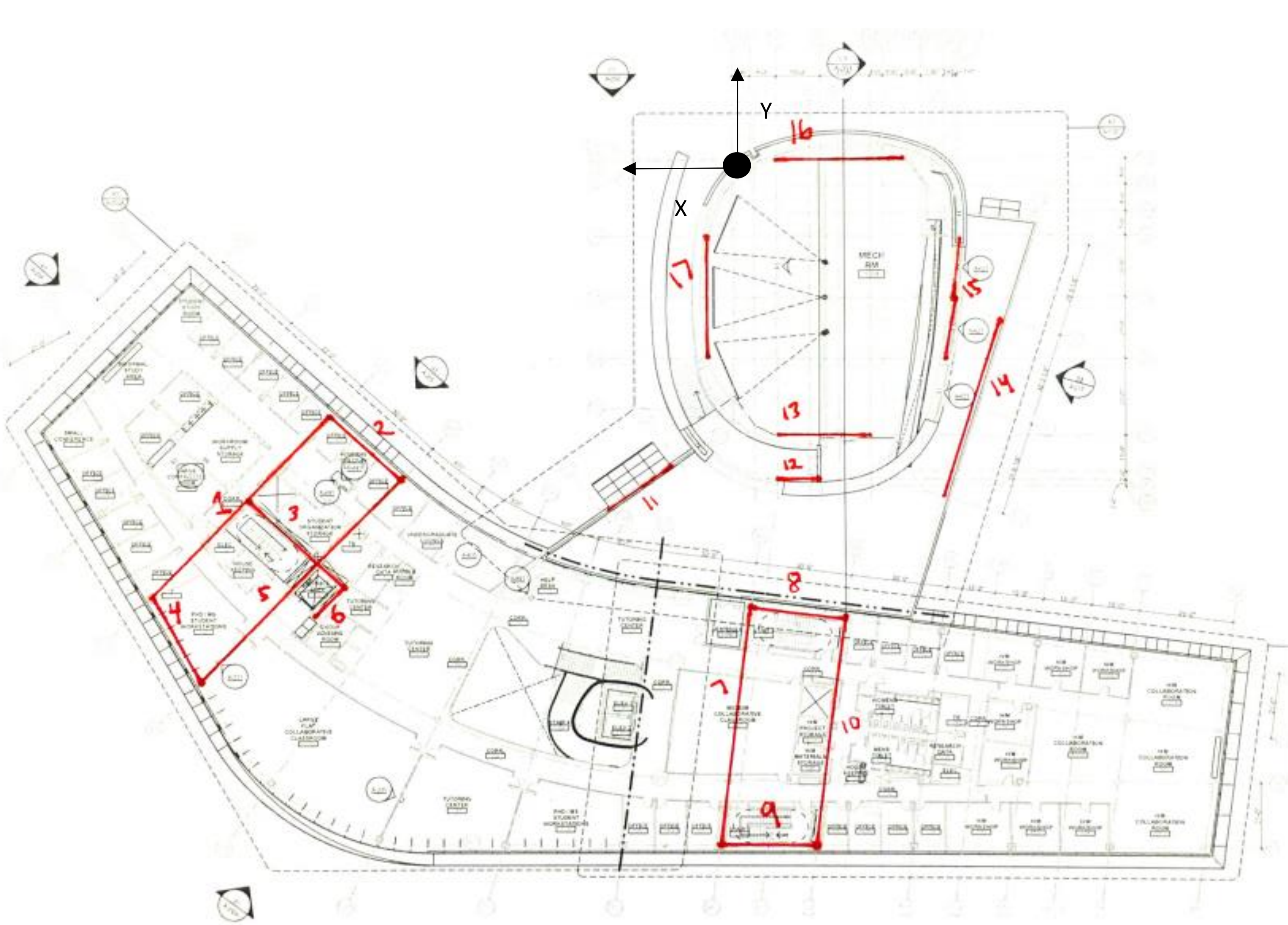
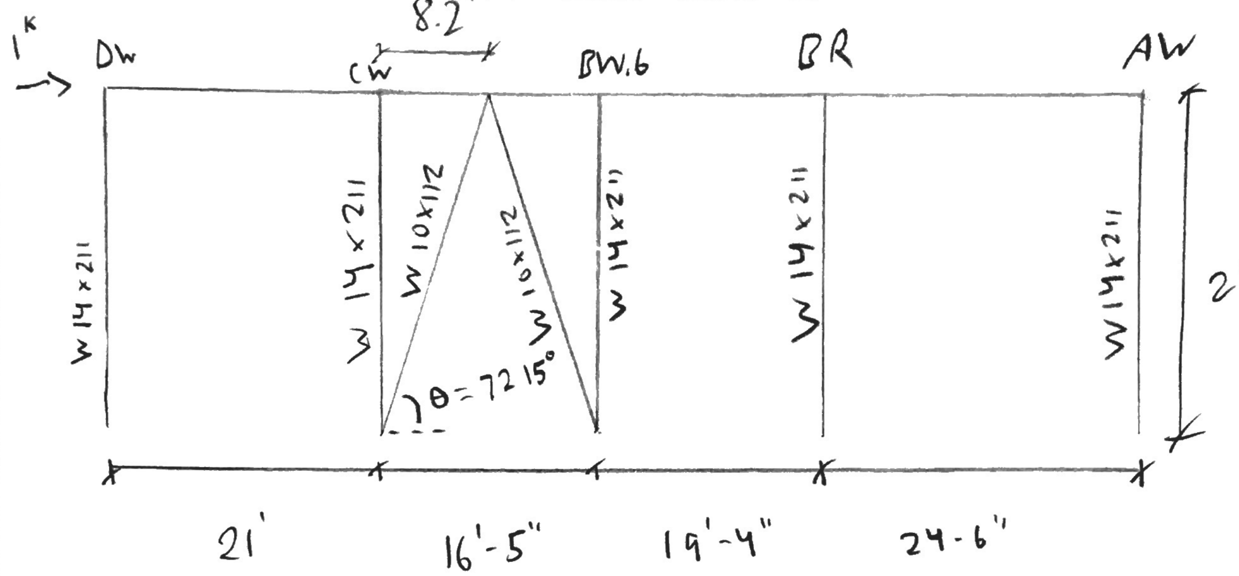


Figure 4: LFRS plan

8.2 Model Validation

8.2.1 Stiffness Calculation

Frame 1 (Column Line 3W)

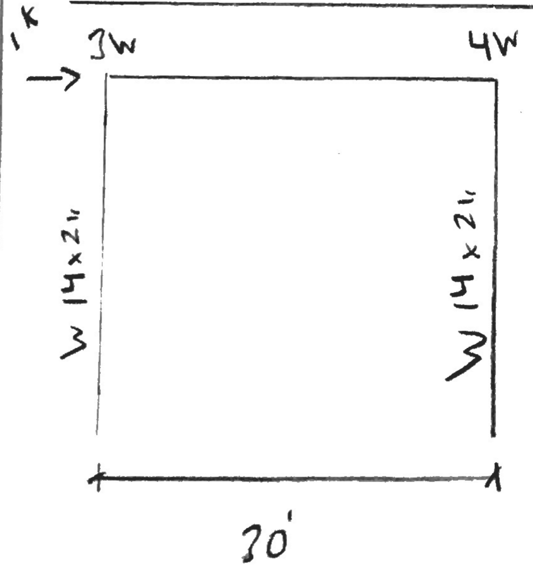


$$K_{col} = \frac{12(29000)(5 \times 2660)}{(25.5 \times 12)^3} = 161.5 \text{ K/in}$$

$$K_{brace} = \frac{32.9(29000)}{(26.79 \times 12)} \times \cos^2(72.15^\circ) \times 2 = 557.7 \text{ K/in}$$

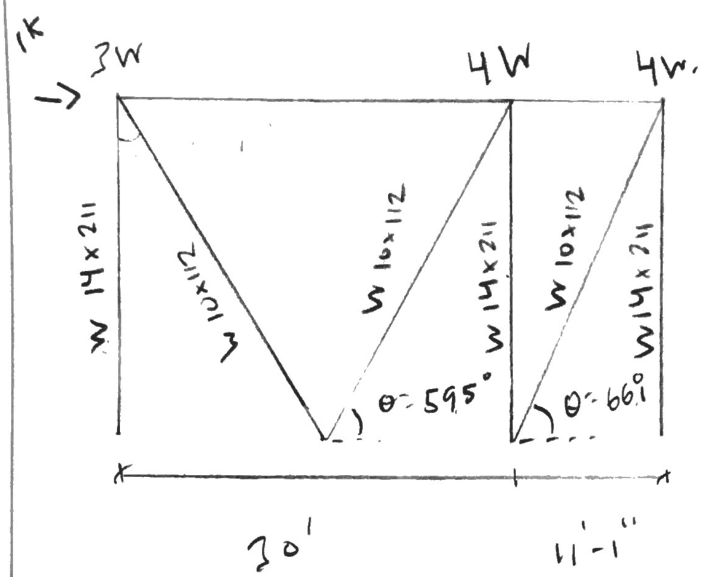
$$K = 161.5 + 557.7 = 719.2 \text{ K/in}$$

Frame 2 (Column Line DW)



$$K_{col} = \frac{12(29000)(2 \times 2660)}{(25.5 \times 12)^3} = 64.6 \text{ K/in}$$

Frame 3 (Column Line BW.6)



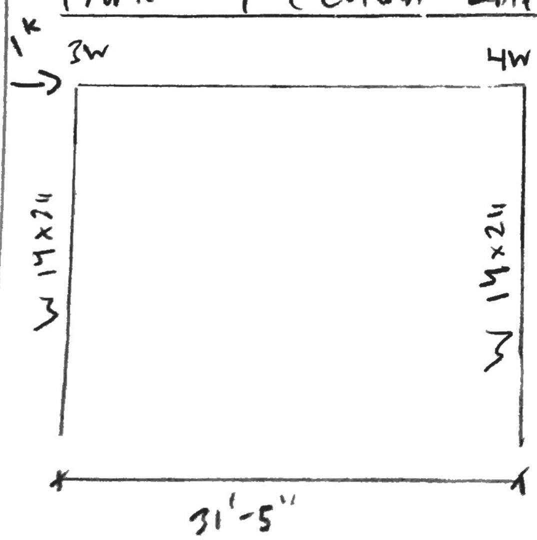
$$K_{col} = \frac{12(29000)(3 \times 2660)}{(25.5 \times 12)^3} = 96.9 \text{ k/in}$$

$$K_{brace\ 1} = \frac{32.9(29000)}{(29.6 \times 12)} \times \cos^2(59.5) \times 2 = 1383.9 \text{ k/in}$$

$$K_{brace\ 2} = \frac{32.9(29000)}{(27.8 \times 12)} \times \cos^2(66.1°) = 469.5 \text{ k/in}$$

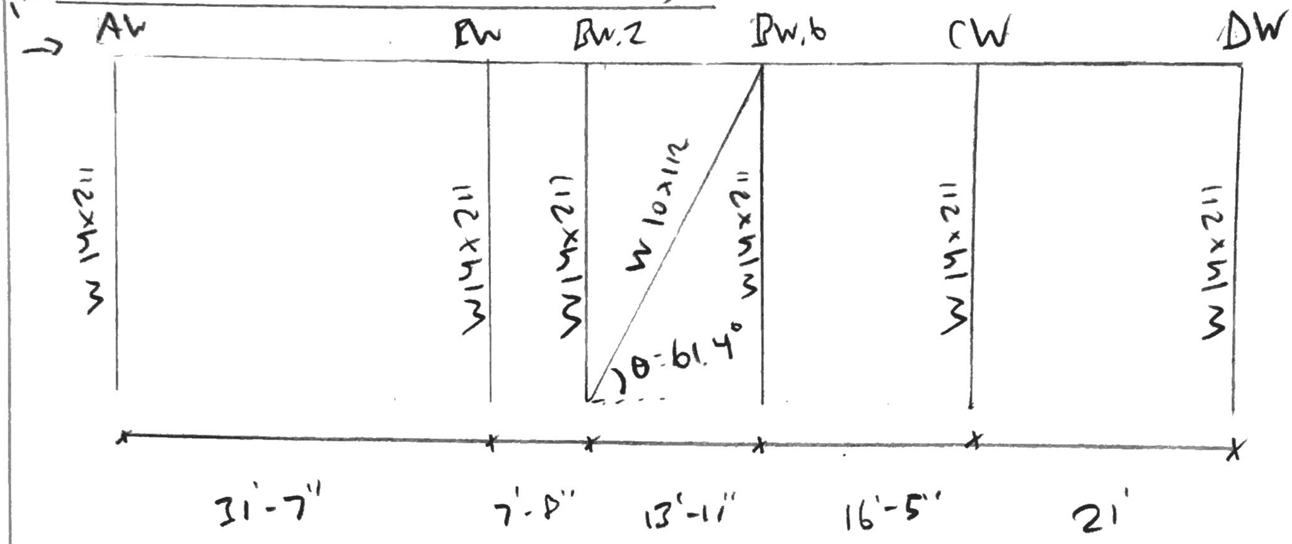
$$K_{total} = 96.9 + 1383.9 + 469.5 = 1950.3 \text{ k/in}$$

Frame 4 (Column Line AW)



$$K_{col} = \frac{12(29000)(2 \times 2660)}{(25.5 \times 12)^3} = 64.6 \text{ k/in}$$

Frame 5 (Column Line 4W)

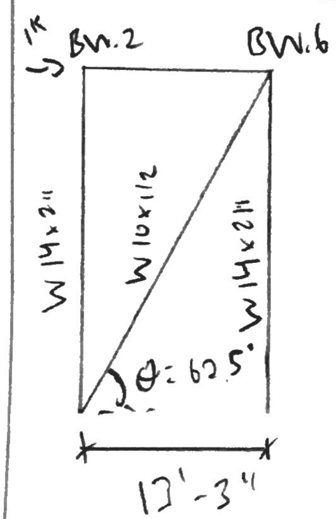


$$K_{col} = \frac{12(29000)(6 \times 2660)}{(25.5 \times 12)^3} = 193.8 \text{ k/in}$$

$$K_{brace} = \frac{32.9(29000)}{(29.05 \times 12)} \times \cos^2(61.4°) = 627.2 \text{ k/in}$$

$$K_{total} = 193.8 + 627.2 = 821 \text{ k/in}$$

Frame 6 (Column Line 4W.3)

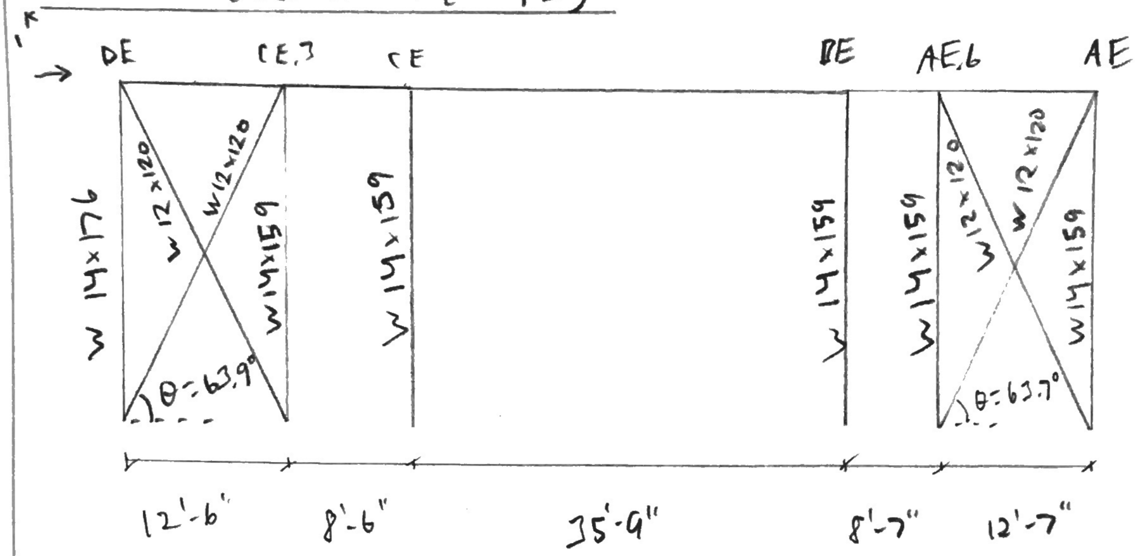


$$K_{col} = \frac{12(29000)(2 \times 2660)}{(25.5 \times 12)^3} = 64.6 \text{ k/in}$$

$$K_{brace} = \frac{32.9(29000)}{(28.7 \times 12)} \cos^2(62.5) = 590.7 \text{ k/in}$$

$$K_{total} = 64.6 + 590.7 = 655.3 \text{ k/in}$$

Frame 7 (Column Line 9E)



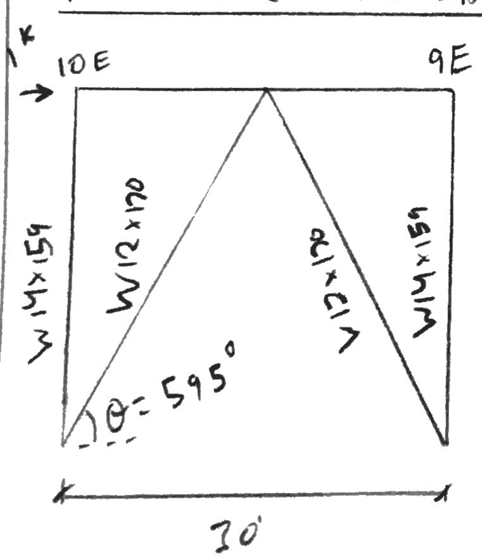
$$K_{col} = \frac{12(29000)(2140 + 5(1900))}{(25.5 \times 12)^3} = 141.4 \ k/in$$

$$K_{brace1} = \frac{35.2(29000)}{(28.4 \times 12)} \times \cos^2(63.9) \times 2 = 1159.5 \ k/in$$

$$K_{brace2} = \frac{35.2(29000)}{(28.4 \times 12)} \times \cos^2(63.7) \times 2 = 1176 \ k/in$$

$$K_{total} = 141.4 + 1159.5 + 1176 = 2476.9 \ k/in$$

Frame 8 (Column Line DE)



$$K_{col} = \frac{12(29000)(2 \times 1900)}{(25.5 \times 12)^3} = 46.2 \ k/in$$

$$K_{brace} = \frac{50(29000)}{(29.6 \times 12)} \times \cos^2(59.5) \times 2 = 2103.1 \ k/in$$

$$K_{total} = 46.2 + 2103.1 = 2149.3 \ k/in$$

Frame 9 (Column Line AE)

Same as Frame 8 except $L = 30'-3"$

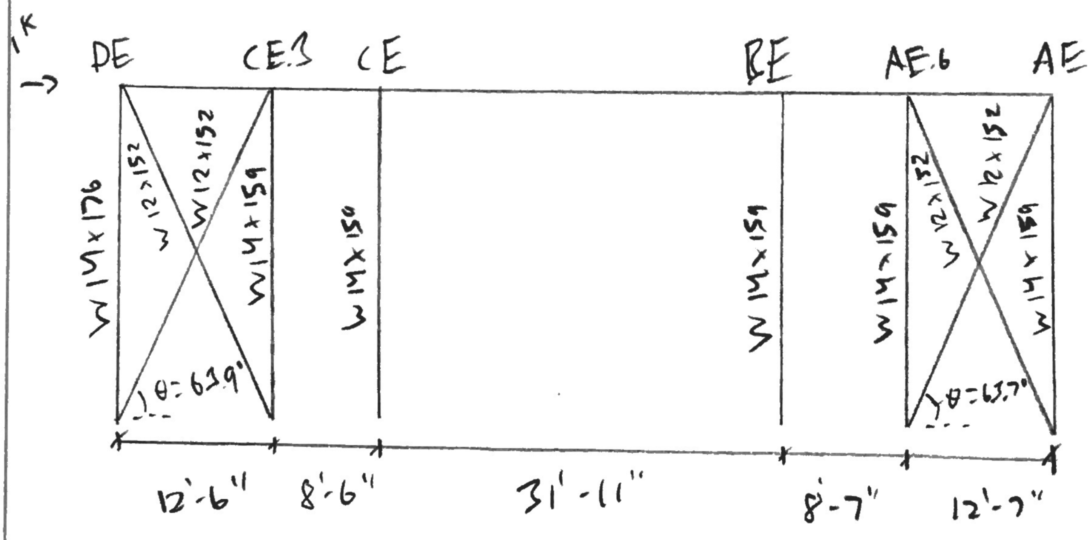
$$\theta = \tan^{-1}\left(\frac{25.5}{15.125}\right) = 59.3°$$

$K_{col} = 46.2$ k/in

$$K_{brace} = \frac{50(29000)}{(29.6 \times 12)} \times \cos^2(59.3°) \times 2 = 2128 \text{ k/in}$$

$K_{total} = 46.2 + 2128 = 2174.2$ k/in

Frame 10 (Column Line 10E)



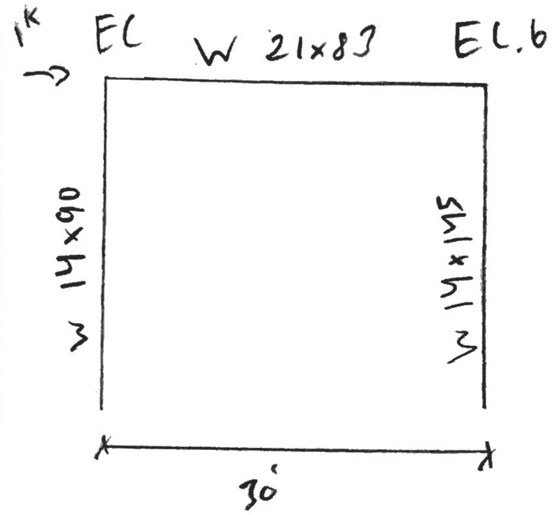
$K_{col} = 141.4$ k/in

$$K_{brace,1} = \frac{44.7(29000)}{(28.4 \times 12)} \times \cos^2(63.9°) \times 2 = 1472.4 \text{ k/in}$$

$$K_{brace\,2} = \frac{44.7(29000)}{(28.4 \times 12)} \times \cos^2(63.7°) \times 2 = 1493.4 \text{ k/in}$$

$K_{total} = 141.4 + 1472.4 + 1493.4 = 3107.2$ k/in

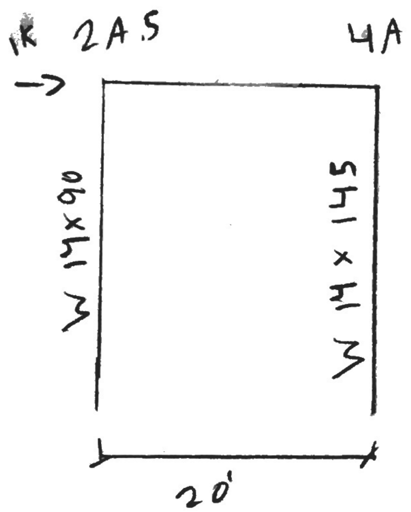
Frame 11 (Column Line 1C)



$$K_{col} = \frac{12(29000)(999 + 1710)}{(25.5 \times 12)^3}$$

$$= 32.9 \text{ k/in}$$

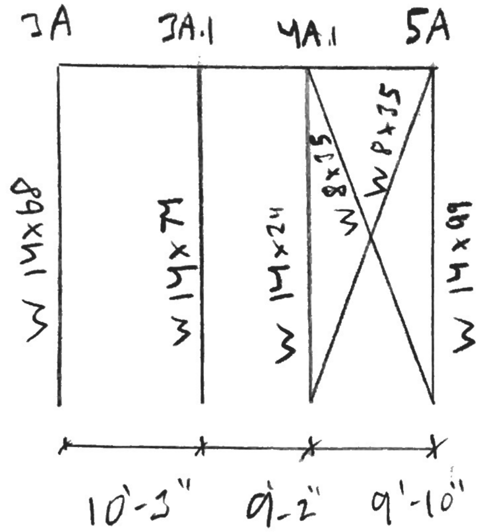
Frame 12 (Column Line AA)



$$K_{col} = \frac{12(29000)(2 \times 1710)}{(25.5 \times 12)^3}$$

$$= 41.5 \text{ k/in}$$

Frame 13 (Column Line CA)



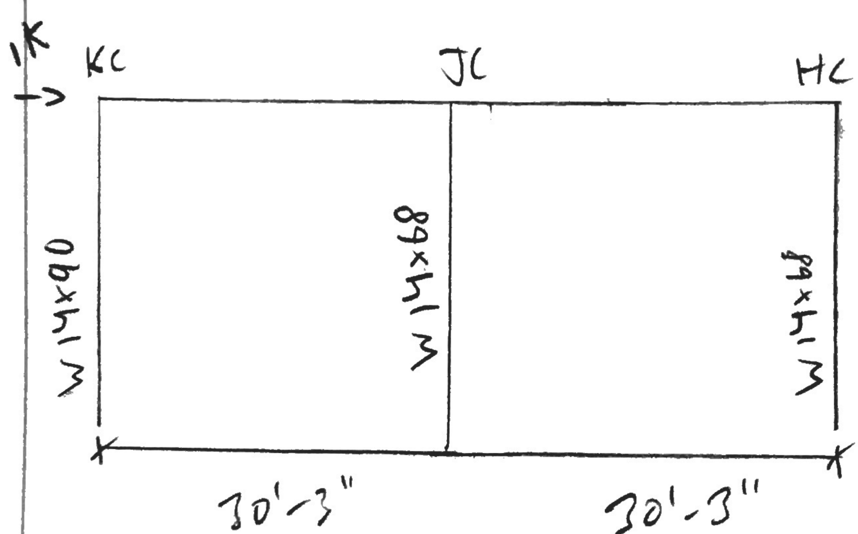
$$K_{col} = \frac{12(29000)(722 + 795 + 2660 + 1110)}{(25.5 \times 12)^3}$$

$$= 64.6 \text{ K/in}$$

$$K_{brace} = \frac{10.3(29000)}{27.33 \times 12} \times \cos^2(68.9) \times 2 = 236.1 \text{ K/in}$$

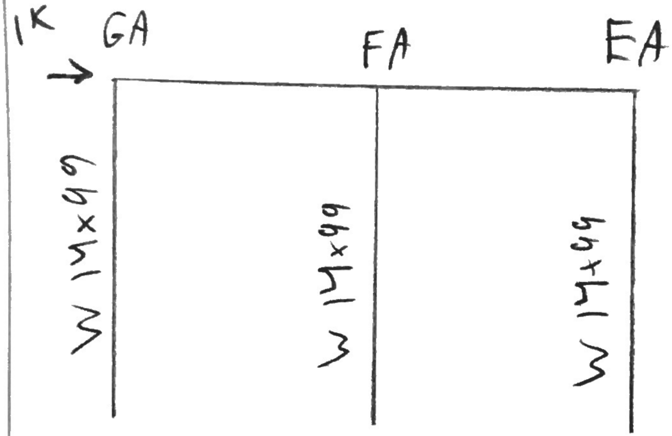
$$K_{Total} = 64.6 + 236.1 = 300.7 \text{ K/in}$$

Frame 14 (Column Line 3C)



$$K_{col} = \frac{12(29000)(2(722) + 999)}{(25.5 \times 12)^3} = 29.7 \text{ K/in}$$

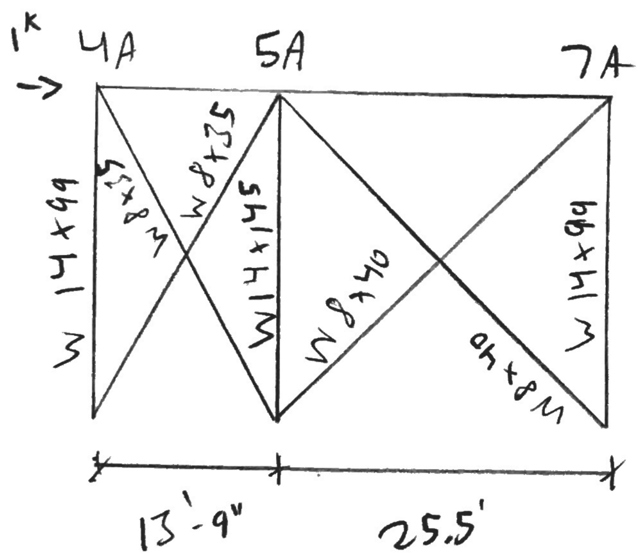
Frame 15 (Column Line 10A)



$$K_{col} = \frac{12(29000)(3(1110))}{(25.5 \times 12)^3}$$

$$= 40.4 \ \text{k/in}$$

Frame 16 (Column Line JA)



$$K_{col} = \frac{12(29000)(2(1110 + 1710))}{(25.5 \times 12)^3}$$

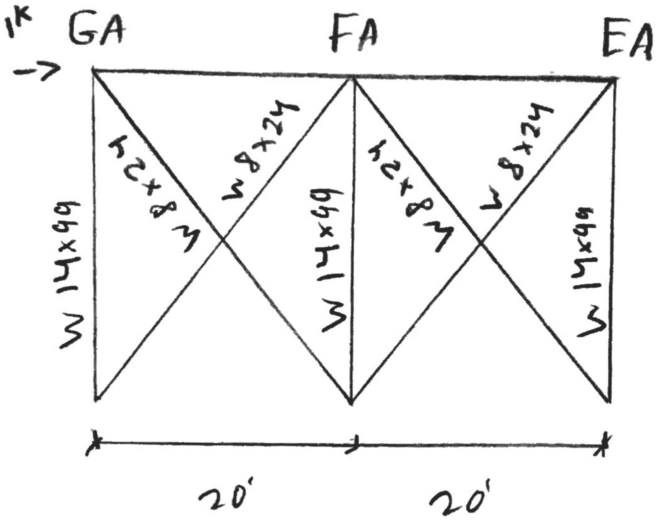
$$= 47.7 \ \text{k/in}$$

$$K_{brace\,1} = \frac{10.3(29000)}{(28.97 \times 12)} \times \cos^2(61.66) \times 2 = 387.2 \ \text{k/in}$$

$$K_{brace\,2} = \frac{11.7(29000)}{(36.06 \times 12)} \times \cos^2(45) \times 2 = 784.1 \ \text{k/in}$$

$$K_{Total} = 47.7 + 387.2 + 784.1 = 1218.7 \ \text{k/in}$$

Frame 17 (Column Line 1A)



$$K_{col} = \frac{12(29000)(3 \times 1110)}{(25.5 \times 12)^3} = 40.4 \text{ K/in}$$

$$K_{brace} = \frac{7.08(29000)}{(32.46 \times 12)} \times \cos^2(51.89) \times 4 = 803.1 \text{ K/in}$$

$$K_{Total} = 40.4 + 803.1 = 843.5 \text{ K/in}$$

8.2.2 Center of Rigidity Calculation

Frame Number	Element Direction	x	y	Rx	Ry	Rx*Y	Ry*X
1	x		-110.5	490.49		-54199.5	
	y	156.4			525.99		82264.72
2	x		-96.9	47.25		-4578.09	
	y	115.75			44.06		5099.605
3	x		-126.3	1426.36		-180149	
	y	139.7			1330.10		185815
4	x		-155	31.32		-4854.41	
	y	174.5			56.50		9859.321
5	x		-137	559.92		-76709.2	
	y	137			600.44		82260.42
6	x		-145	446.91		-64802.5	
	y	129.2			479.26		61919.85
7	x		-186.5	301.86		-56297.1	
	y	1			2458.44		2458.437
8	x		-149.5	2133.28		-318925	
	y	-17.22			261.93		-4510.5
9	x		-225.8	2174.20		-490934	
10	x		-189.4	378.68		-71721.3	
	y	-28.7			3084.04		-88511.9
11	x		-107.67	26.62		-2865.82	
	y	29.25			19.34		565.64
12	x		-105.67	41.27		-4361.28	
	y	-19.5			4.34		-84.5896
13	x		-90.5	300.70		-27213.4	
14	x		-82.67	8.68		-717.862	
	y	-75.75			28.40		-2151.47
15	x		-45.75	3.52		-161.092	
	y	-69.2			40.25		-2785.04
16	x		0	1218.70			
17	y	-7.9			843.50		-6663.65
			Σ	9589.763503	9776.578	-1358490	325535.9

Table 1: COR calculation

$$\overline{X_R} = \frac{\sum R_y X}{\sum R_y} = \frac{325535.9}{9776.578} = 33.29'$$

$$\overline{Y_R} = \frac{\sum R_x Y}{\sum R_x} = \frac{-1358490}{9589.763} = -141.66'$$

8.2.3 Center of Mass Calculations

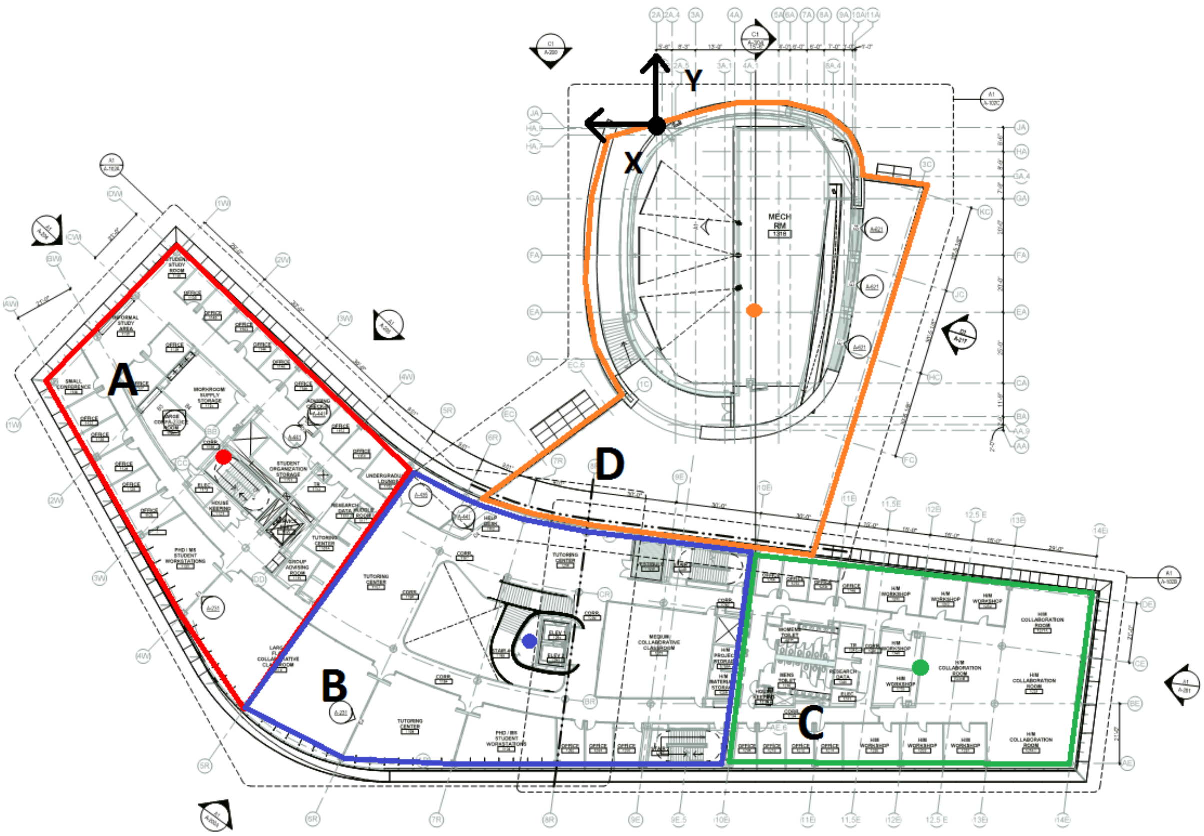


Figure 5: COM determination

Element	Area (ft^2)	Weight(psf)	W (k)	x	y	W*x	W*Y
Floor (A)	9779	48	469.39	151.4	-116.25	71065.95	-54566.82
Floor (B)	13093	48	628.46	43.75	-181.5	27495.30	-114066.22
Floor (C.)	7914	48	379.87	-93.25	-192.25	-35423.06	-73030.39
Floor (D)	14911	48	715.73	-35.88	-65.15	-25680.32	-46629.68
Σ	45697		2193.456			37457.86416	-288293.1072

Table 2: COM calculation

$$\overline{X_{COM}} = \frac{\sum W_x}{\sum W} = \frac{37457.864}{92193.456} = 17.08'$$

$$\overline{Y_{COM}} = \frac{\sum W_y}{\sum W} = \frac{-288293.107}{92193.456} = -141.43'$$

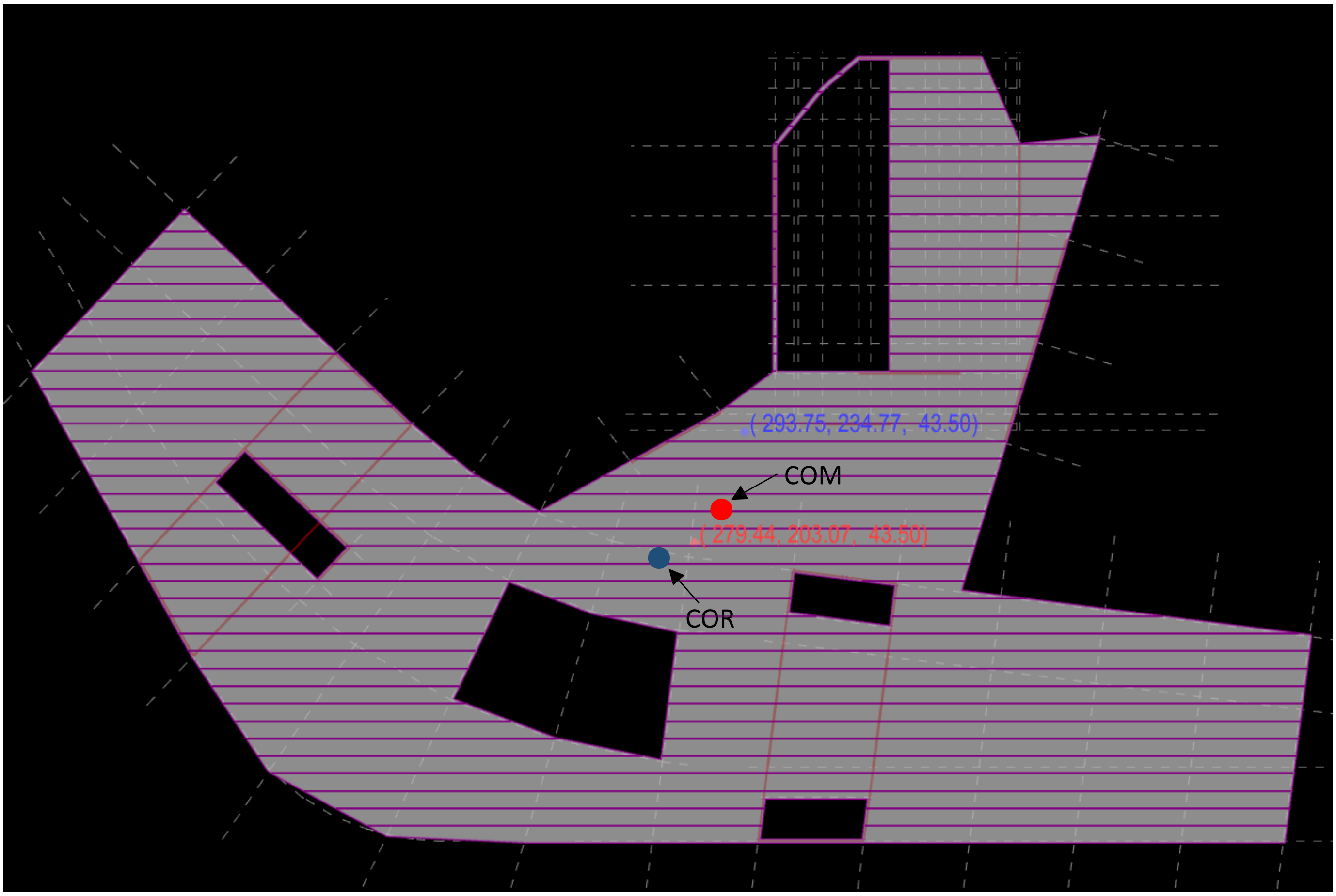


Figure 6: COR and COM comparisons

The center of rigidity calculation is off in the x direction by about 19 feet in the x direction and about 36 feet in the y direction. The hand calculated center of rigidity could be off for a number of reasons. The most probable reason is the result of breaking the angled frames into its respective x and y components. In reality, the frame does not act in the x and y direction, it only acts in line with the frame. However, breaking the frame into components is necessary for performing this calculation by hand. The angled members are most certainly the cause of discrepancy between RAM and the hand calculations. The center of mass calculation is off by about 9 feet in the x and y direction. Due to the irregular geometry of the building floor plan, the building had to be broken into separate shapes as shown in Figure 5 on the previous page. This irregular shape will not result in an exact center of mass. In addition, RAM takes into account slap openings which the hand calculations do not, which also increases the discrepancy.

8.2.4 Total Shear into Frames

After comparing the total building shear of each case in RAM, it has been determined that Wind Case 1 (North-South direction) is the controlling lateral case. A building shear of 1254.26 kips is applied at the center of geometry, results in an eccentricity of 3.8' from the center of rigidity. This yields a torsional moment of 4766.188 ft-k.

							Direct Shear		Torsional Shear	Total Shear	RAM			
Frame	Rx	Ry	R	di	Ridi	$Ridi^2$	Vdx	Vdy	Vt	V	Vx	Vy	V	% error
1	490.49	525.99	719.20	107.63	77407.50	8331368.79	64.15	67.48	4.41	88.70	57.26	64.14	85.98	3.16
2	47.25	44.06	64.60	23.67	1529.08	36193.37	6.18	5.65	0.09	8.29	3.25	3.97	5.13	61.52
3	1426.36	1330.10	1950.30	60.50	117993.15	7138585.58	186.56	170.64	6.73	246.10	174.79	171.79	245.08	0.42
4	31.32	56.50	64.60	132.33	8548.52	1131225.39	4.10	7.25	0.49	7.84	0.00	4.33	4.33	81.03
5	559.92	600.44	821.00	78.00	64038.00	4994964.00	73.23	77.03	3.65	102.64	48.71	55.89	74.14	38.44
6	446.91	479.26	655.30	67.00	43905.10	2941641.70	58.45	61.48	2.50	82.33	48.77	52.82	71.89	14.52
7	301.86	2458.44	2476.90	37.30	92388.37	3446086.20	39.48	315.40	5.27	312.59	21.12	183.60	184.81	69.14
8	2133.28	261.93	2149.30	1.90	4083.67	7758.97	279.01	33.60	0.23	280.80	126.76	15.96	127.76	119.78
9	2174.20	0.00	2174.20	82.90	180241.18	14941993.82	284.37	0.00	10.28	274.09	4.63	0.95	4.73	98.28
10	378.68	3084.04	3107.20	67.90	210978.88	14325465.95	49.53	395.66	12.03	386.72	46.70	379.30	382.16	1.19
11	26.62	19.34	32.90	25.50	838.95	21393.23	3.48	2.48	0.05	4.23	1.78	2.44	3.02	39.95
12	41.27	4.34	41.50	36.60	1518.90	55591.74	5.40	0.56	0.09	5.51	1.18	1.46	1.88	193.69
13	300.70	0.00	300.70	51.67	15537.17	802805.52	39.33	0.00	0.89	38.44	10.31	0.47	10.32	272.49
14	8.68	28.40	29.70	86.80	2577.96	223766.93	1.14	3.64	0.15	3.67	0.11	2.82	2.82	30.03
15	3.52	40.25	40.40	90.30	3648.12	329425.24	0.46	5.16	0.21	5.39	0.52	5.45	5.47	1.52
16	1218.70	0.00	1218.70	141.20	172080.44	24297758.13	159.40	0.00	9.81	149.59	129.60	2.47	129.62	15.40
17	0.00	843.50	843.50	26.10	22015.35	574600.64	0.00	108.21	1.26	109.47	0.46	223.61	223.61	51.04
Σ	9589.76	9776.58			J [(k/in)*ft2]	83600625.19								

Table 3: Total shear into each frame

To check the torsional shear, an equilibrium check has been performed. $\Sigma(V_t * d_i)$ = (107.63*4.41)+(23.67*0.09)+(60.50*6.73) +…(141.20*9.81)*(26.10*1.26) = 4766.188 ft-k. As previously stated when comparing center of rigidity and center of mass, the angled frame members throw off the total shear into each frame. Several frames, including frame 3,10, and 15 have percent error of less than 2%, whereas frames 12 and 13 have percent error of more than 100%. For lateral spot checks in this report, Frame 10 will be analyzed due to the similarity between the hand calculations and RAM.

8.2.5 Wind Load Comparisons

The wind load calculations from notebook submission A have been revised and are shown below.

	z (ft)	q_z (psf)	$p_{winward}$	$p_{leeward}$	p_{roof}	Trib Height	Trib Weight	Story Force
Ground	18	17.86	11.93	-14.15		12.75	380	126.37
1	43.5	24.36	16.28	-14.15		20.085	380	232.23
2	58.17	26.40	17.64	-14.15		14.67	380	177.23
3	72.84	28.24	18.87	-14.15		14.67	380	184.07
4	87.51	29.85	19.94	-14.15		14.67	380	190.04
5	102.18	31.19	20.84	-14.15		14.67	380	195.05
Penthouse	116.85	32.34	21.61	-14.15		17.25	380	234.39
Roof (0'-68.33)	136.67	33.89	22.64	-14.15	-25.931	9.915	380	138.63
Roof (68.33-136.67')	136.67	33.89			-24.818	9.915	380	
Roof (136.67-273.33')	136.67	33.89			-14.479	9.915	380	
Roof (> 273.33')	136.67	33.89			-9.146	9.915	380	
							Base Shear	1478.02

Table 4: Wind loads from Notebook Submission A

As stated previously, Case 1 (Wind north-south and east-west) will be used to validate the wind loads. The tables below compare wind in the north-south direction and wind in the east-west direction.

		Hand Calculations	RAM	
Level	Height	Fx	Fy	% error
Roof	136.67	138.63	128.75	7.13
Penthouse	116.85	234.39	220.39	5.97
5th Floor	102.18	195.05	182.71	6.33
4th Floor	87.51	190.04	177.97	6.35
3rd Floor	72.84	184.07	172.61	6.23
2nd Floor	57.17	177.23	166.38	6.12
1st Floor	43.5	232.23	273.85	17.92
Ground	18	126.37	77.2	38.91
	Base Shear	1478.01	1399.86	5.29

Table 5: Wind loads in the north-south direction

Level	Height	Fx	Fx	% error
Roof	136.67	93.88	56.19	40.147
Penthouse	116.85	158.73	95.9	39.583
5th Floor	102.18	132.09	79.11	40.109
4th Floor	87.51	128.7	76.66	40.435
3rd Floor	72.84	124.65	74.68	40.088
2nd Floor	57.17	120.025	83.62	30.331
1st Floor	43.5	157.23	138.94	11.633
Ground	18	85.58	94.9	10.89
	Base Shear	1000.885	700	30.062

Table 6: Wind loads in the east-west direction

The hand calculations for the wind in the north-south direction are fairly accurate compared to the RAM model. In the east-west direction however, it is off by about 30-40% at each level. To determine the wind loads for this building, a rectangle was drawn around the buildings largest dimensions, as shown in Appendix A. It makes sense that wind in the north-south direction will be more accurate as the building's actual east-west dimension spans the 380', resulting in more accurate surface area for the north-south wind to apply to the building. However, the dimensions in the north-south direction do not span the whole 245'. This also explains why the hand calculations are larger than the RAM loads, as the wind loads were being applied to a larger surface area than the building's actual dimensions.

8.2.6 Seismic Load Comparisons

		Hand Calculations	RAM	
Level	Height	Fx	Fx	% error
Roof	136.67	480.92	435	9.6503
Penthouse	116.85	290.52	213	26.742
5th Floor	102.18	176.93	182	2.8825
4th Floor	87.51	147.93	152	3.0149
3rd Floor	72.84	119.68	123	3.1668
2nd Floor	57.17	109.19	130	18.711
1st Floor	43.5	102.39	94.9	7.3445
Ground	18	40.77	8.56	79.004
	Base Shear	1468.33	1338	8.857

Table 7: Seismic Loads

The hand calculation and RAM seismic loads are fairly accurate, thus validating the seismic loads.

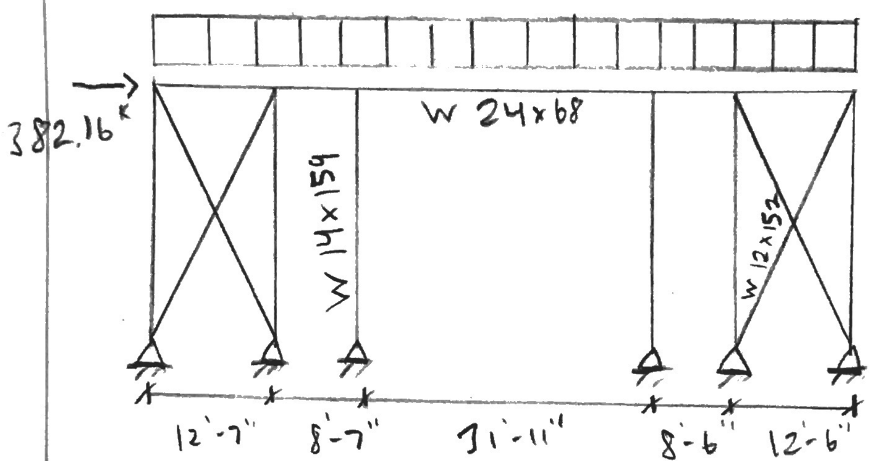
8.3 Lateral System Checks

8.3.1 Lateral Spot Checks

As stated previously, Frame 10 will be used for lateral spot checks due to the accuracy between hand calculations and RAM. The controlling load case is 1.2D + 0.5 L + 1.0 W.

Lateral Member Spot Checks

D = 2190 PLF , L = 3000 PLF



Trib width = 30'

Controlling Load case : $1.2D + 0.5L + 1.0W$

Dead = 73 PSF

Live = 100 PSF

Wind = 382.16^K

W 24x68 Beam Check

From SAP 2000 Model :

$M_u = 389.4^{'K}$

$V_u = 24.2^K$

From Table 3-2 : $\phi M_n = 664^{'K} > 389.4^{'K}$ $\therefore$ OK

$\phi V_n = 295^K > 24.2^K$ $\therefore$ OK

W 12×152 Brace Check

From SAP 2000 Model:

$P_u = 262^{k}$

$M_u = 78.2^{'k}$

From Table 4-1: $\overbrace{\phi P_n}^{\text{unbraced length} = 30'} = 793^{k} > P_u = 262^{k} \therefore \text{ok}$

$\phi M_n = 911^{'k} > M_u = 78.2^{'k} \therefore \text{ok}$

W 14×159 Column Check

From SAP 2000 Model:

$P_u = 1060^{k}$

From Table 4-1: $\underbrace{\phi P_n = 1350^{k}}_{\text{Unbraced length} = 26'} > P_u = 1060^{k} \therefore \text{ok}$

8.3.2 Story Drift

The allowable story drift at each level is h/400. In RAM, the drift ratio provides a ration of the allowable drift per foot. Therefore, the allowable drift ratio is h/400 = 1/400 = 0.0025. Figure 7 below shows the story drift of each load combination at the roof at the corner of the building, where drift is expected to control. Three of the load combinations (W14, W17, and W23) do not pass as the ratios are greater than 0.0025. The largest ratio is 0.0030, which is 20% greater than the allowable. One explanation for this could be the frames were not modeled correctly in RAM (i.e. wrong size assigned, did not assign all bracing). Further investigation will be done to determine why drift is not passing.

Story	**LdC**	**Displacement**		**Story Drift**		**Drift Ratio**	
		X	**Y**	**X**	**Y**	**X**	**Y**
		in	**in**	**in**	**in**		
Roof	D	-0.0865	0.0305	-0.0232	0.0052	0.0001	0.0000
	Lp	-0.0982	0.0589	-0.0251	0.0118	0.0001	0.0000
	W13	0.7209	-0.3029	0.0778	-0.0744	0.0003	0.0003
	W14	0.4542	3.2742	0.1334	0.6387	0.0006	0.0027
	W15	0.4055	-0.5286	0.0438	-0.1064	0.0002	0.0004
	W16	0.6759	0.0743	0.0729	-0.0052	0.0003	0.0000
	W17	0.9244	3.8109	0.1665	0.7137	0.0007	0.0030
	W18	-0.2431	1.1003	0.0335	0.2443	0.0001	0.0010
	W19	0.8813	2.2285	0.1584	0.4232	0.0007	0.0018
	W20	0.2001	-2.6828	-0.0417	-0.5348	0.0002	0.0022
	W21	0.1218	0.4288	0.0580	0.1034	0.0002	0.0004
	W22	1.2002	2.9139	0.1796	0.5314	0.0008	0.0022
	W23	-0.3892	-3.2546	-0.0920	-0.6151	0.0004	0.0026
	W24	0.6892	-0.7695	0.0295	-0.1872	0.0001	0.0008
	E5	1.9964	-0.1487	0.3100	-0.0906	0.0013	0.0004
	E6	2.2724	0.5582	0.3530	0.0601	0.0015	0.0003
	E7	0.6456	3.9289	0.1800	0.8842	0.0008	0.0037
	E8	0.0958	2.5109	0.0934	0.5803	0.0004	0.0024

Figure 7: Story Drift

8.3.3 RAM Member Code Check

Figure 8 shows the member code check that RAM offers. The darker the color, the lower the interaction; the lighter the color, the higher the interaction (i.e. <0.4 is blue, <0.6 green, <0.8 yellow). Any member above an interaction of 1.0 is red. Several members in the model do not pass, all of which are braced members in the auditorium. Similar to story drift, further investigation will need to be done to determine why these members are not passing for strength.

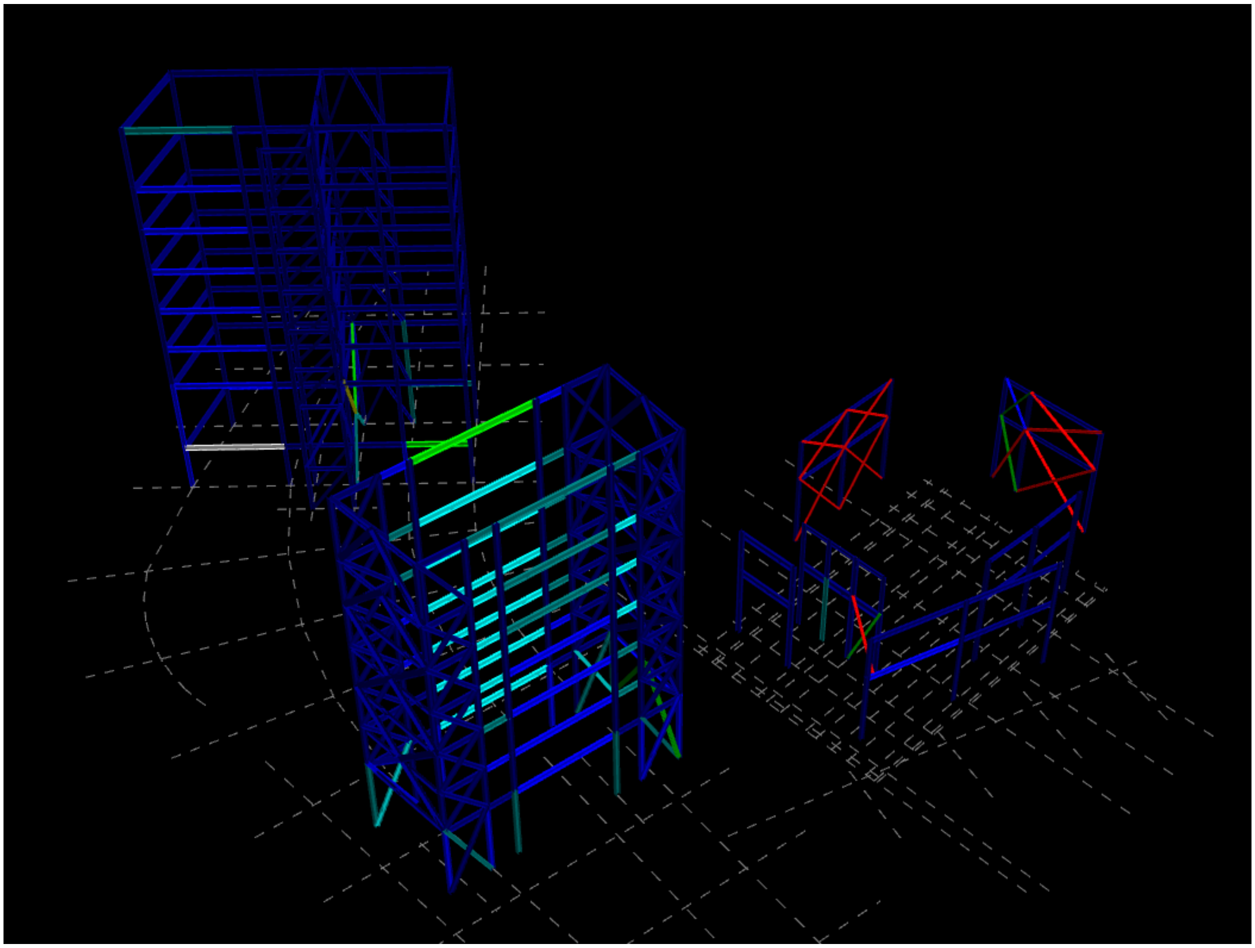
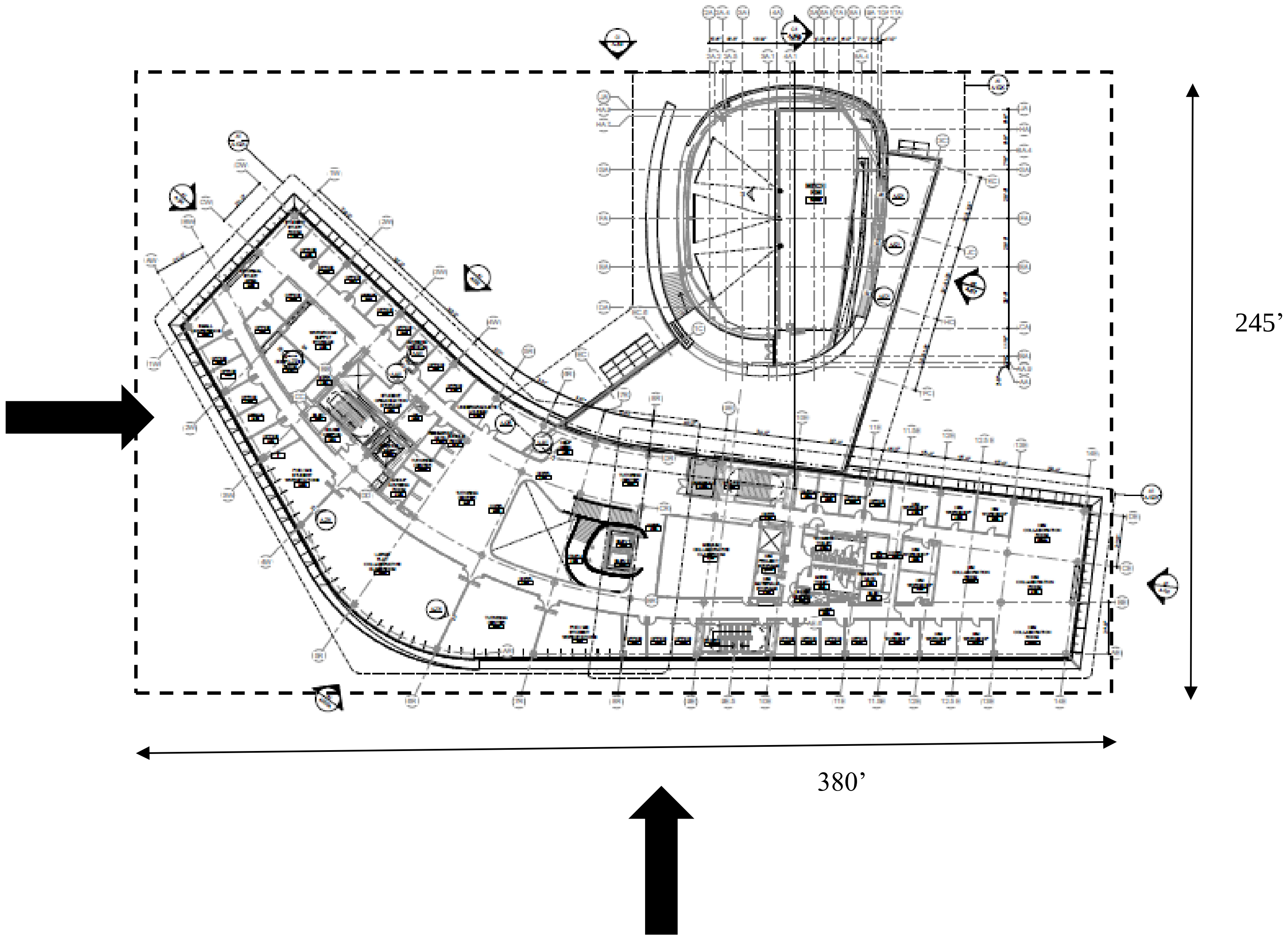


Figure 8: RAM member code check

Appendix A- Wind Load Calculation

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 B- 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