Building A

138 East Beaver Ave

State College, PA

Report By: Robert Dawson

Advisor: DR. Linda Hanagan

Structural Notebook C

Letter of Transmittal

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Dr. Hanagan,

The following report is a lateral analysis study for the new Building A. This building is located in downtown State College, PA and is intended to be used for mixed use/student housing. In addition to the analysis of the existing structural design, there is a study on three alternate systems for the building.

This submission is made up of calculations and a model for the lateral system that was studied. The table of contents shows the order of the calculations. At the end of the report is an appendix with more information on how certain values were determined as well as assumptions that were made.

Thank you,

Robert Dawson.

Table of Contents

1.0 Executive Summary

Building A (fake name) is a mixed-use building in downtown State College, PA. The building will serve as an apartment building for students at The Pennsylvania State University and will feature retail spaces along the street level for local people to enjoy. The building is 132,000sf with 5 stories of residential space and 2 stories of commercial retail space. The designing architects are WTW Architects and the builder is the general contractor Leonard S. Fiore. The project's delivery method is Design – Bid – Build and it is on a 2-year project schedule. Construction is to start on September 1st, 2018 and it is to be completed by June 1st, 2020. The total cost for this project is $21,764,00.

The building will be constructed with concrete slabs and CMU blocks. The building features a parking garage on the 1st and 2nd floors and columns hold up the structure here. From floors 3-7 these columns do not continue to maximize apartment living space. The third floor features a very thick (26") transfer slab to allow for this and the CMU block units bear most of the gravity weight in the residential floors of the building.

The design for this building is in accordance with IBC 2009. The concrete design follows ACI and the steel is designed with the AISC reference standard.

2.0 Abstract

2.1 Project Team and Info

Owner: HFL Corporation

Architect: Penn Tera Engineering

Builder: Leonard S. Fiore

No. of Stories:7 above grade, 2 below

Occupancy Type: Mixed Use/Student Housing Cost:

$21,764,000

2.2 Systems

Construction

- Design – Bid – Build
- 2-year timeline
- September 2018 August 2022
- Demo site before construction

Structural

- Hollow-core plank on block
- Transfer slab between parking garage and residential spaces □ Concrete frame

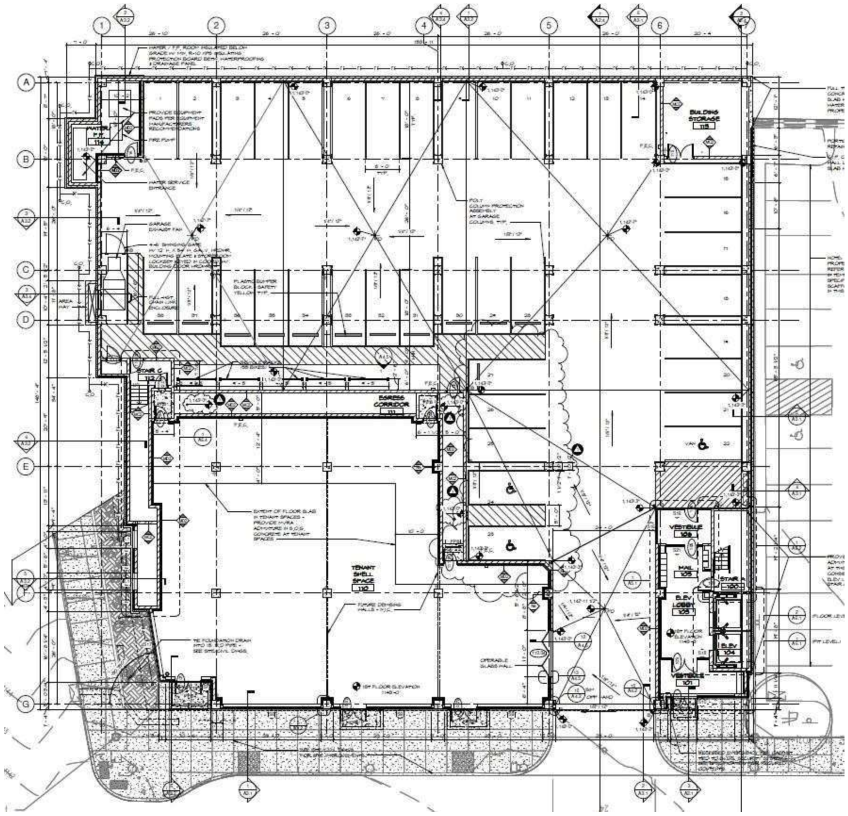
MEP

- PTAC AC units in each apartment
- Efficient Lighting

3.0 Site Plan

Figure 1

SouthPughStreet



East Beaver Avenue

4.0 Applicable Codes and Documents

Building A complies to IBC 2009 and IBC 2015 for Ch. 11 only. Wind and Seismic design is in accordance with 2009 IBC. The reference standard for concrete in this building is ACI. The reference standard for steel construction AISC.

Documents

- Building A Construction Plan
- Specs
- Building A Drawings

5.0 Gravity Loads

5.1 Roof Bay

Loads:

- Live – 30psf
- Snow – 40psf (2009 IBC)
- Dead – 110psf o 8" Hollow Core Roof Plank – 100psf o Rigid Insulation – 1.5psf o Misc. (MEP, Ceiling) – 8psf

ASCE Load Combination 3 controls roof design.

$$1.2D + 1.6S + L = \underline{226psf}$$

5.2 Floor Bay

Residential Loads:

- Live – 40 psf
- Dead – 135psf o 8" Hollow Core Plank – 100psf o CMU Partitions – 25 psf o Misc. (MEP, Ceiling) – 10psf

ASCE Load Combination 2 controls residential floor design.

$$1.2D + 1.6L = \underline{226psf}$$

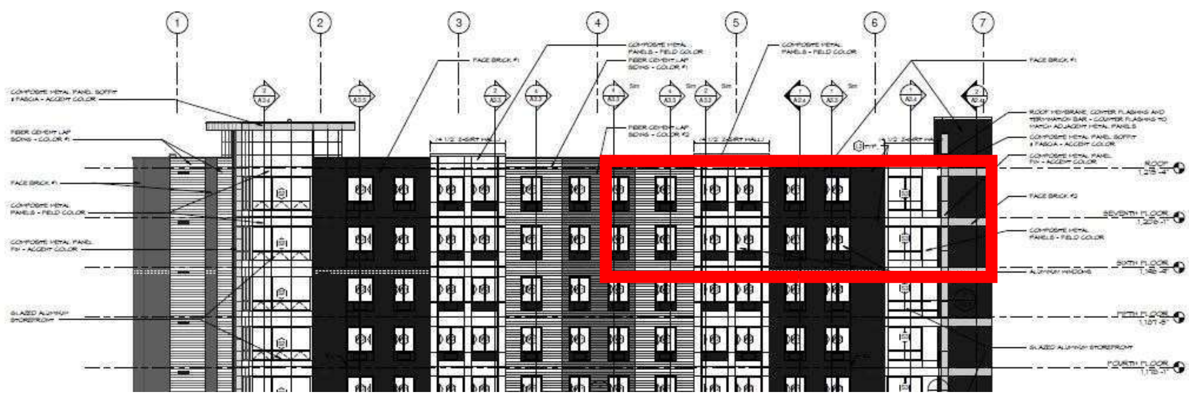


Figure 2

Typical Roof and Floor Bay: 72’ – 4 5/8” x 26’

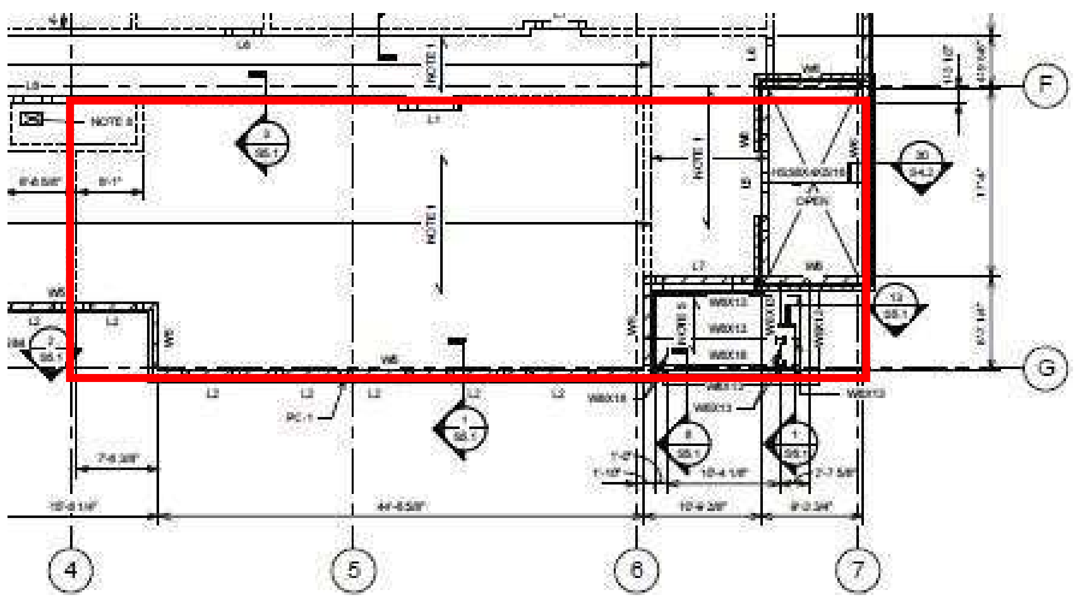
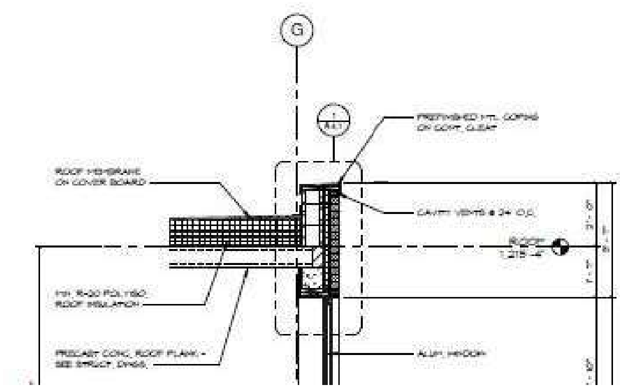


Figure 3

Cross Sections of Typical Floor and Roof Construction.



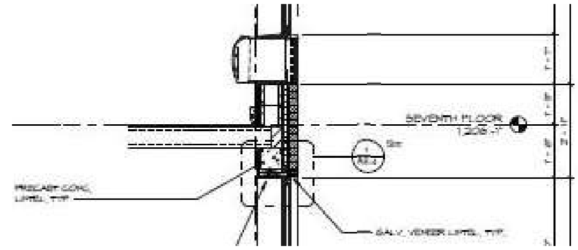


Figure 4

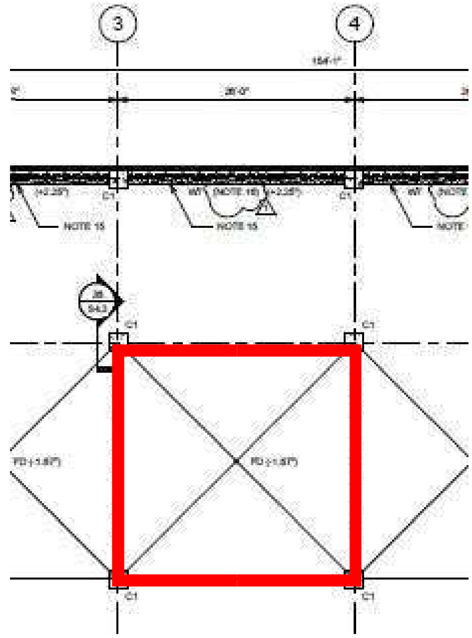
Parking Garage Loads:

- Live – 40psf + 3000lb point load
- Dead – 160psf o 12" Reinforced Slab – 120psf o Misc. (MEP) – 10psf

ASCE Load Combination 2 controls parking garage design.

1.2D + 1.6L = <u>256psf + 4800lb</u>

Figure 5



Typical Garage Bay:

26’ x 26’

5.3 Exterior Wall

Typical Wall Load:

1. Dead – 75 o Exterior Brick – 55psf o Glazing – 15psf o Siding – 5psf

Exterior brick is not supported by the floor slab.

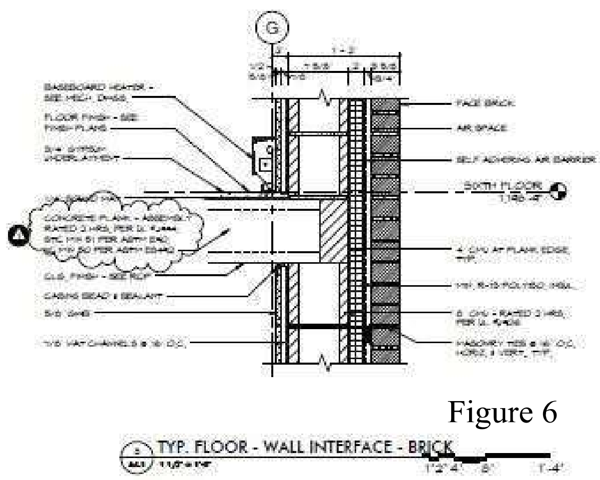


Figure 6

6.0 Lateral Loads

6.1 Wind Loads

Building A meets the conditions for the ASCE-7 “Simplified Directional Procedure for Buildings <160ft” <u>Class 2 Building Requirements</u>

1. Meets Section 26.2 Simple Diaphragm
2. Mean Roof Height = 72’ (60’< 72’ < 160’)

3. L/B = 1.07 OR 0.93 (0.2 < 1.07 < 5.0)
4. N_a = 1.042
5. K_{zt} = 1.0 (No adjustment)

Risk Category: Category II (Apartments/Offices/Retail Space)

Terrain: Sloped Terrain

Basic Wind Speed: V = 115mph (90 in drawings. State College, PA)

Exposure Category: B

Topographic Factor: K_{zt} = 1.0

From Table 27.6-1: Net pressures on walls @ the top and base:

Direction	L/B	P_h	P_o	P_z
N-S	1.07	28.9	22.4	29.6
E-W	0.963	29.1	22.7	29.8

Table 1

Values Linearly Interpolated based on L/B and h = 72’

Total E–W Base Shear:

= ((29.1+22.7)/2)(72’)(143.33’) + (29.8)(4’)(143.33’)(2.25)

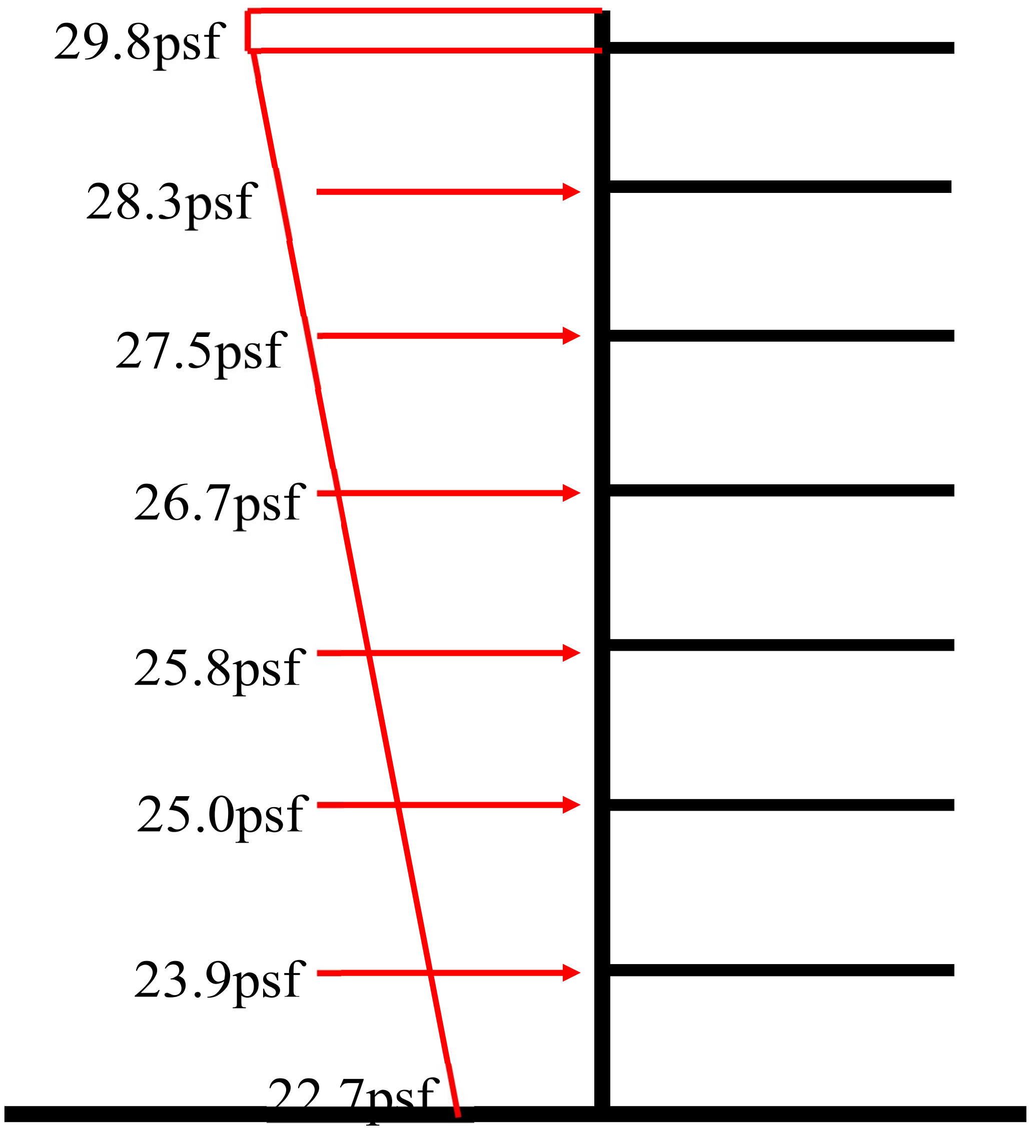
= 306kip

Total N–S Base Shear:

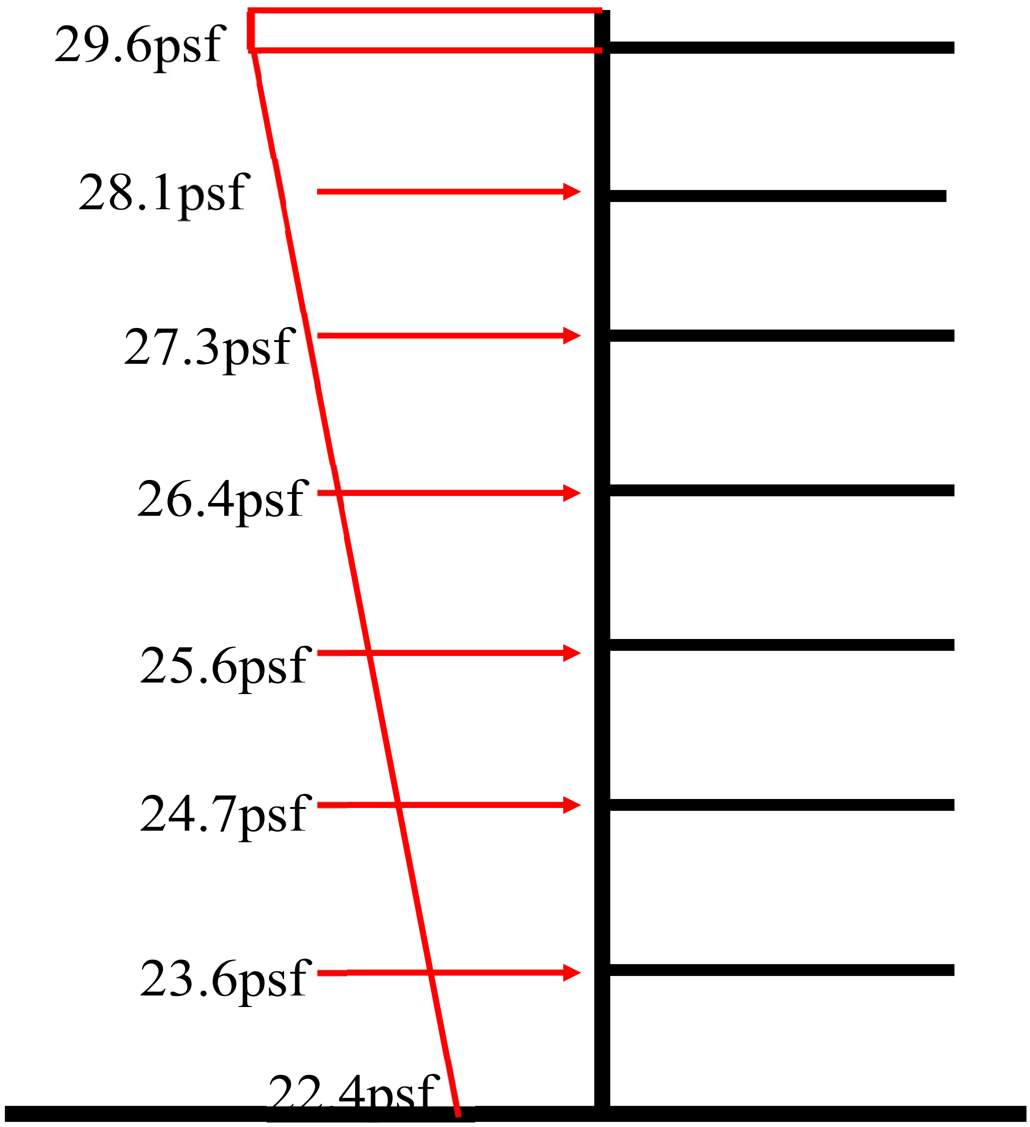
= ((28.9+22.4)/2)(72')(154') + (29.6)(4')(154')(2.25)

= 326kip

E-W Diagram



Base Shear = 306kip Figure 7 N-S Diagram



Base Shear = 326kip

Figure 8

6.2 Seismic Loads

Seismic Loads determined from ASCE 7-10.

Risk Category: Category II

$S_1 = 0.049$

$S_s = 0.147$

$S_{DS} = 0.098$ (Category A)

$S_{D1} = 0.033$ (Category A)

Seismic Response Coefficient:

$C_{s\ max}$:

$T_a = (C_t)(h_i^x) = 0.7511$

$C_t = 0.016$

$h_i = 72'$

$x = 0.9$

$T_a = 0.1N = 0.7$

N = 7 (Stories above grade)

$T_a = \underline{0.7511}$ $T < T_L$; Use Eqn. 12.8-3

$C_{s\ max} = S_{D1}/(T(R/I_e)) = 0.01719$

R = 1.5 (Ordinary Plain Masonry Shear Walls)

$I_e = 1.0$ (Used in Design)

$C_{s\ min}$ Check:

$C_{s\ min} = (0.044)(0.098)(1.0) = 0.004312$ $C_{s\ max}$ OK Seismic Weight:

<u>Floors</u>

Floor	Floor Area (ft^2)	Loading (psf)	Weight (kip)
1	22,126	160	3541
2	22,109	160	3538
3-7	16550	135	2235

Roof	16550	110	1821

Table 2

Total Floor Weight = 11,135kip

Exterior Wall

Group 1 (Walls around floors 1-2)

Surface area of group = 9520ft^2

Material	Material Area (ft^2)	Loading (psf)	Weight (kip)
Masonry	7616	100	762
Glazing	1904	15	26

Table 3

Group 2 (Walls around floors 3-7)

Material	Material Area (ft^2)	Loading (psf)	Weight (kip)
Fiber Siding	24,000	5	120
Metal Panels	4,800	10	48
Brick	4,800	55	264
Glazing	14,400	15	216

Table 4

Total Wall Weight = 1436kip

Total Building Weight = 12,600kip

Base Shear (same in both directions)

$$V = 0.01719(12,600) = 220\text{kip}$$

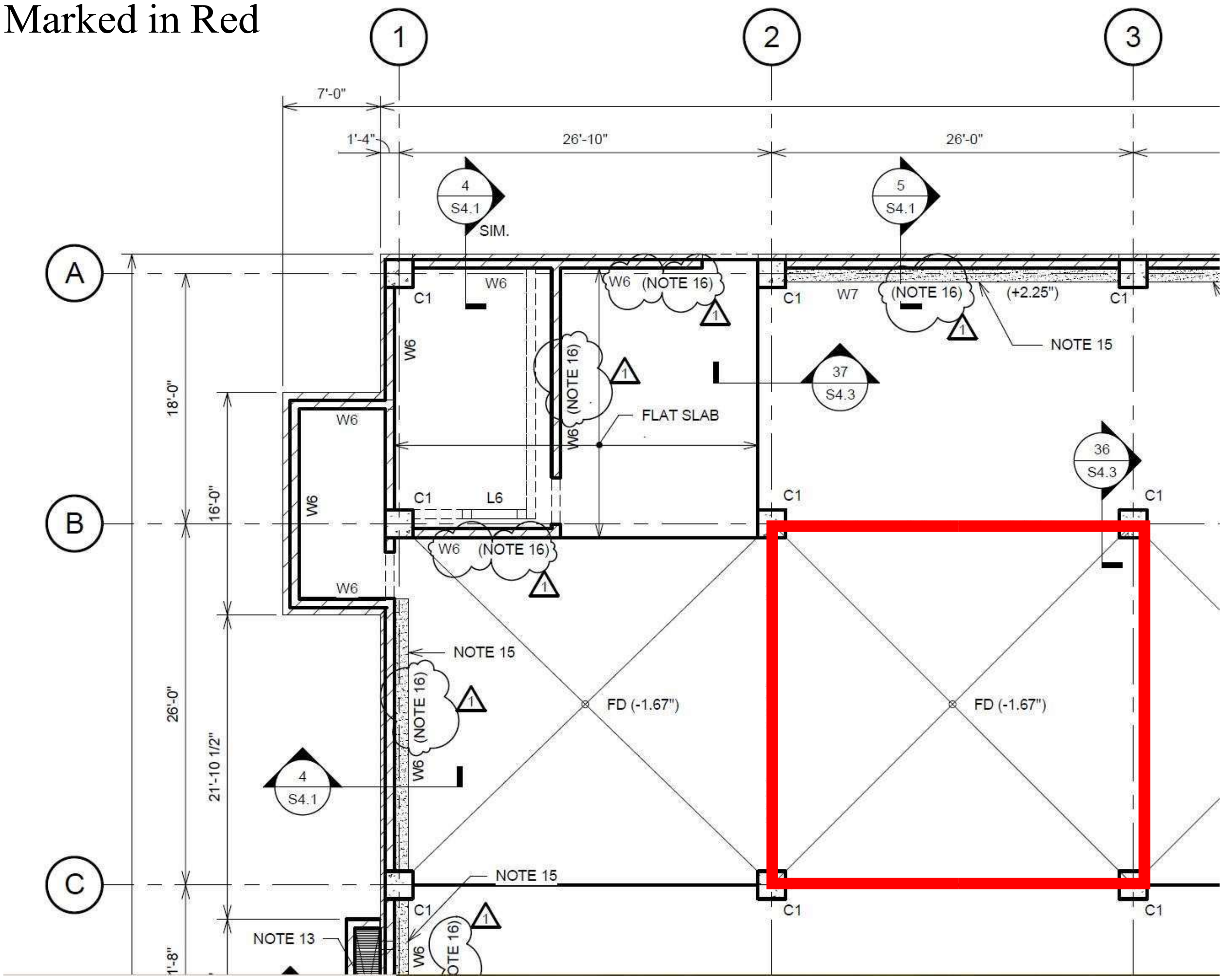
Typical Bay and Member Spot Checks:

Bay:

The typical bay chosen in this building is a 26x26 reinforced flat slab bay with 2x2 column. This bay is located on the first and second floors in the parking section of the building. The other bay that was studied is located on floors 3 – Roof. It is made of hollow core planks and rests on CMU masonry bearing walls.

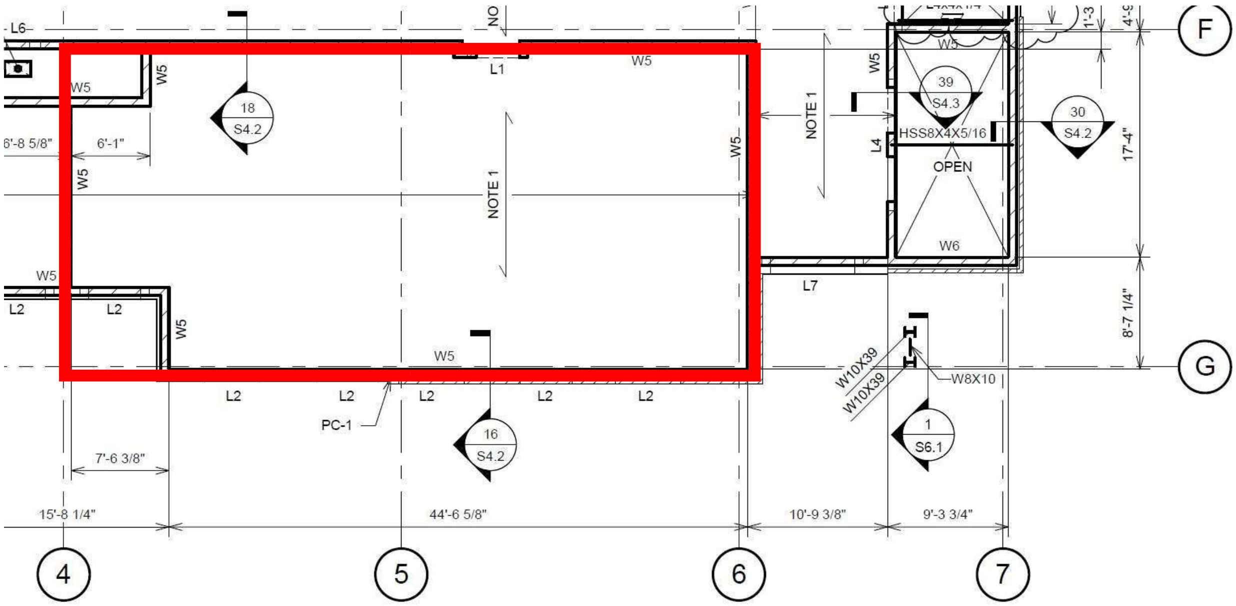
Garage Bay:

Marked in Red



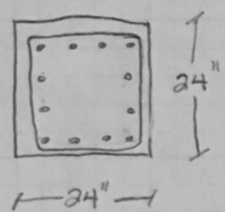
Residential Bay:

Marked in Red


L6
W5
L1
W5
18
S4.2
6'-1"
W5
NOTE 1
W5
NOTE 1
W5
39
S4.3
HSS8X4X5/16
L4
OPEN
30
S4.2
17'-4"
W6
L7
W5
L2
L2
W5
W5
8'-7 1/4"
W10X39
W10X39
W8X10
L2
L2
L2
L2
L2
PC-1
16
S4.2
1
S6.1
7'-6 3/8"
15'-8 1/4"
44'-6 5/8"
10'-9 3/8"
9'-3 3/4"
F
G
4
5
6
7

7.0 Notebook B

Column C1 Max Strength:



Column C1. $f'_c = 5000psi$ $f_y = 60ksi$

Reinforced w/ (12) #7 & # Closed Ties

12" O.C

$P_u = .393^k$ see Column Spot Checks (Exterior)

$P_u = 933^k$ see Column Spot Checks (Interior)

$A_g = B \times D = 24in \times 24in = 576\,in^2$

$A_{st} = N \times A_b = 12 \cdot 0.6 = 7.2\,in^2$

$P_o = 0.85\, f'_c\,(A_g - A_{st}) + f_y A_{st}$

$= 0.85(5)(576 - 7.2) + (7.2)(60)$

$= 2849.4^k$

$\phi P_n = 0.8 \times 0.65 \times 2849.4 = 1482^k$

Exterior: $1482^k > 393^k$ ∴ Why so high?

Interior: $1482^k > 933^k$ OK

<u>Column Spot Checks</u> (Exterior)

Column Type: C1

Along lines A & 3

Exterior Column

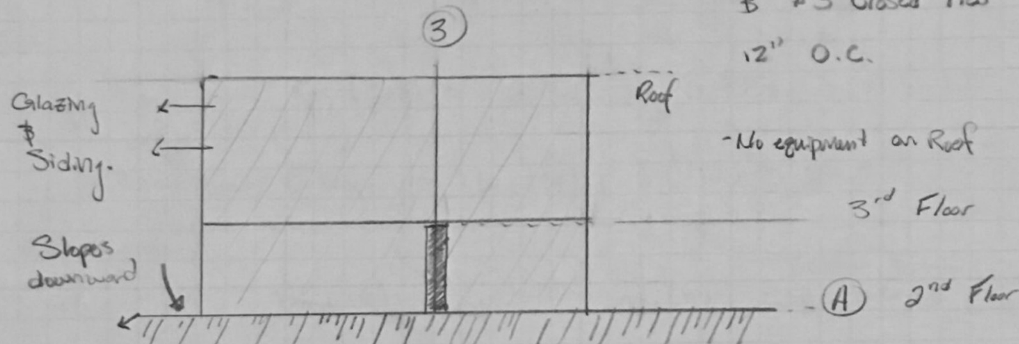
From Drawings: Column C1

24" × 24"

Reinforced (12) #7 & #3 Closed Ties

12" O.C.

- No equipment on Roof



Elevation.

Column C1 continues the 3rd Floor Base

@ A3 C1 Runs grade to 3rd Floor 16' hieght

Column A3 Loads per floor:

Trib Area per floor = 26' × 9' = 234 ft^2

Influence Area/floor = 52' × 18' = 936 ft^2

Exterior Wall Length = 26'

LL (Unreduced) = 40 psf (see gravity section)

LL reduction (3-7) Floor = $\min\left|\begin{array}{l}0.25 + \frac{15}{\sqrt{4(936)}} = 0.50 \leftarrow \\ 0.4\end{array}\right.$

LL / Floor = 0.5(234)(40) = 4.68^k

Roof DL = 120 psf see gravity section

Roof Ext Wall Load = 0.520 klf (See gravity section)

Roof DL = 120(234) + 0.520(4) = 28.1^k

Snow Load = 40psf (see gravity) $40(234) = 9.4^k$

Dead Load (Residential) = 135 psf see gravity section
(5 floors) + 0.933^k see gravity section

Dead Load (Parking) = 160 psf see gravity section
(1 floor) + 0.1^k see gravity Section.

Dead Load/ Res floor (4 floors) = $135(234) + 0.933(26) = 31.7^k$

Dead Load/ Parking = $160(234) + 0.1(26) = 37.5^k$

Load Summary:

A3	Dead (K)	Live (K)	Snow (K)	Transfer Slab added weight
Roof:	28.1	x	9.4	t = 28, Already accounted for 8"
Floors: 3-7	31.7	4.68	x	t = 20
Floor: 2	37.5	11.7	x	$\frac{20}{12} \times 150 = 250$
Transfer Slab:	58.5^k	x	x	250psf × 234 = 58.5
Total	283	30.42	9.4	

Combo: $-1.2D + 1.6L + 0.5S$

$1.2(251) + 1.6(30.42) + 0.5(9.4) = \underline{393^k}$

Column Spot Checks (Interior)

Column Type C1

Along Line B3

Interior Column

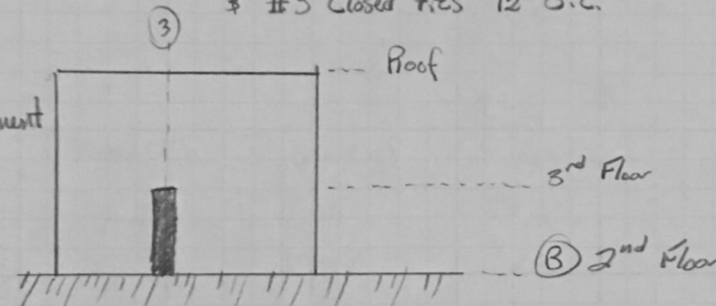
From Drawings:

C1 is 24" x 24"

Reinforced (12) #7
& #3 Closed ties 12" o.c.

- No Roof Equipment

Elevation



Column B3 Loads.

Trib Area / Floor = $(9+13)(26) = 572\ ft^2$

Influence Area / Floor = $(18+26)(52) = 2288\ ft^2$

LL (Unreduced) = 40 psf (see gravity section)

LL Reduction (3-7) floor = $\min \begin{cases} 0.25 + \dfrac{15}{\sqrt{4(2288)}} = 0.41 \leftarrow \\ 0.4 \end{cases}$

LL/ Floor = $0.41(572)(40) = 9.4^k$

Roof DL = 120 psf see gravity section

Roof DL = $120(572) = 68.7^k$

Snow Load = 40 psf see gravity section $40(572) = 22.9^k$

Dead Load (Residential) = 135 psf

Dead Load (Parking) = 160 psf

Dead Load / Residential Floor = 135 (572) = 77.3^k

Dead Load / Parking = 160 (572) = 91.6^k

Transfer Slab extra thickness = $\frac{20}{12} \times 150 \times 572 = 143^k$

(8" already accounted for)

Load Summary:

B3	Dead (K)	Live (K)	Snow (K)
Roof:	68.7	x	22.9
Floors: (3-7)	77.3	9.4	x
Floor 2:	91.6	11.7	x
Transfer: Slab	143.	x	x
Total:	689.8	58.7	22.9

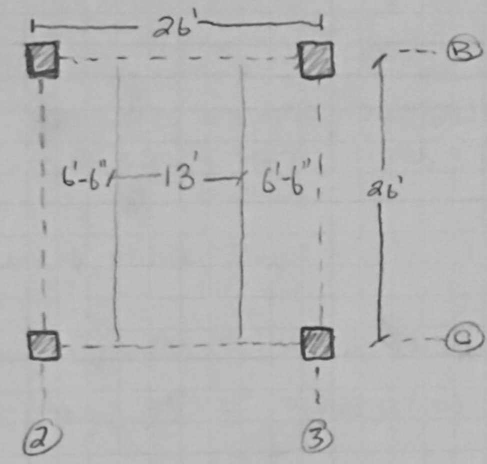
Combo:

$1.2D + 1.6L + 0.5S$

$1.2(689.8) + 1.6(58.7) + 0.5(22.9) = \underline{933^k}$

Garage Slab Spot Check;

Bay:



Loads on Bay:

Dead Load: 160 psf see gravity Loads

Live Load: 40 psf see gravity loads.

Check for DDM

1) 3 continuous span both ways ✓
2) 26/26 = 1 < 2 ✓
3) Spans = length ✓
4) No Column offsets ✓
5) $2 \times D = 320 > 40 = L$ ✓
6) No beams ✓
7) No moment redistribution ✓

ok for DDM according to ACI 318-11

$1.2D + 1.6L = 1.2(160) + 1.6(40) = 256$ psf

$M_o = q_u l_2 l_n^2/8 = \frac{0.256\text{psf}\,(26)^2(13)}{8} = 282^k$ Kip-ft

Both directions Interior.

$-M_o$, $B2-C_2 = M_o$, $B_3-C_3 = M_o$, $B_2-B_3 = M_o$, C_2-C_3

Factored Static Moment

M_{int}^-	M^+	M_{int}^-
0.65(282)	0.35(282)	0.65(282)
= 183.3 ft-k	= 98.7	= 183.3 ft-K

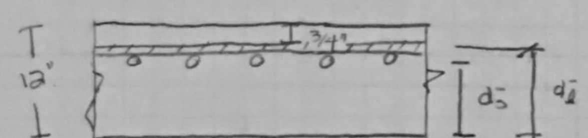
Moments in Column & Middle Strip:

- Column-Strip $M^- = 0.75(183.3) = 137.5$
- Middle-Strip $M^- = 0.25(183.3) = 45.83$

- Column-Strip $M^+ = 0.6(98.7) = 59.22$
- Middle-Strip $M^+ = 0.4(98.7) = 39.48$

Strip	Left M	Middle M	Right M
Middle S	-22.9	39.48	-22.9
Column S	-68.75	59.22	-68.75

Negative Moments

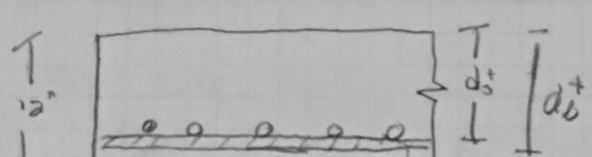


Typical:

Column: #6 @ 10"

Middle: #4 @ 12"

Positive Moments



Typical:

Column: #5 @ 10"

Middle: #4 @ 12"

Depth of Bars: $- \frac{3}{4}''$ Clear Cover

$$d_\ell^- = 12 - \frac{3}{4} - \frac{1}{2}(0.5) = \underline{11 \text{ in}}$$

$$d_\ell^+ = 12 - \frac{3}{4} - \frac{1}{2}(0.5) = \underline{11 \text{ in}}$$

$$d_s^- = 12 - \frac{3}{4} - 0.5 - \frac{1}{2}(0.750) = \underline{10.375 \text{ in}}$$

$$d_{s^+} = 12 - \frac{3}{4} - 0.5 - \frac{1}{2}(0.625) = \underline{10.4375 \text{ in}}$$

Middle B: Design OK

	Left Side M^-	Middle M^+	Right Side M^-	
Mu (ft-k)	-22.9	39.48	-22.9	
MS width (in) b	13(12) = 156"	156"	156"	
d (in)	11	11	11	
As req	0.49	0.84	0.49	
As min	3.37	3.37	3.37	← Controls
Smax	24	24	24	
N	6.2	6.2	6.2	
Nmin	6.5	6.5	6.5	
Spacing	25.16	25.16	25.16	

Eqns.

$A_s\ req = \dfrac{M_u(12)}{0.9 f_y \, 0.95 d}$

$A_s\ min = 0.0018bh$

$f_y = 60$ ksi

$S_{max} = 2h$

$N = A_s/A_b$

$N_{min} = b/S_{max}$

Spacing $= b/N$

Column B:

	Left M^-	Middle M^+	Right M^-
M_u	-68.75	55.22	-68.75
C_s (b)	79.2	79.2	79.2
d	11	11	11
$A_{s\,req}$	1.46	1.17	1.46
$A_{s\,min}$	1.56	1.56	1.56
S_{max}	24	24	24
N	6.74	6.74	6.74
N_{min}	6.5	6.5	6.5
Spacing	23.14	23.14	23.14

← Controls

Design OK in All Directions.

Determine if Deflection Calcs Needed:

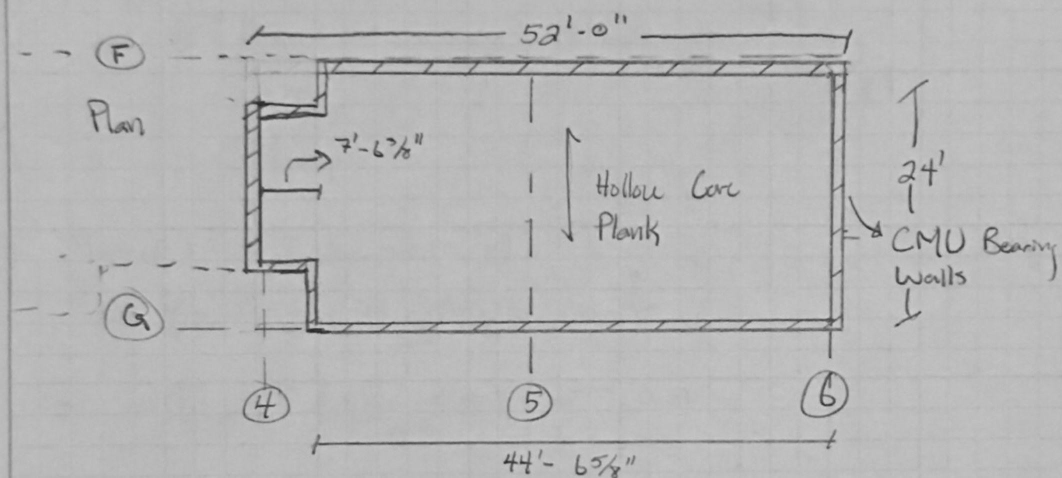
Interior Panel

No edge beams: $\frac{l_n}{28} = \frac{(26-2)\times 12}{28} = 10.29''$

$10.29'' < 12''$ OK ✓

Hollow Core Slab Spot Check:

Typical Bay:



Hollow Core = 8" thick

f'_c = 5000 psi

Loading:

Dead Load: 135 psf see gravity Section

Live Load: 40 psf

Combo: $1.2D + 1.6L$ = 226 psf

Prestressing Steel:

4 1/2 in diameter, 270 ksi, low relaxation strands

$A_{ps} = 4(0.153) = 0.612 \text{ in}^2$

$d_p = 7 \text{ in}$ | $l_{pc} = 24'6''$

$f_{pi} = 0.7 f_{pu}$ | $l = 25$

$f'_c = 5000 \text{ psi}$

Method 1: ACI Equation (18-1)

Use $\gamma_p = 0.28$ for low-relaxation strands

$$\beta_1 = 0.85 \left(\frac{5000-4000}{1000}\right) 0.05 = 0.80$$

$$\rho_p = \frac{A_{ps}}{b d_p} = \frac{0.612}{(48)(8)} = 0.00159$$

$$f_{ps} = f_{pu}\left(1 - \frac{\gamma_p}{\beta_1}\rho_p \frac{f_{pu}}{f'_c}\right) = 270\left[1 - \frac{0.28}{0.8} \cdot 0.00159 \left(\frac{270}{5}\right)\right]$$

$$f_{ps} = 261.89 \text{ ksi}$$

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{0.612 \cdot 261.89}{0.85 \cdot 5 \cdot 48} = 0.786 \text{ in}$$

$$c = \frac{a}{\beta_1} = 0.786/0.8 = 0.9825 \text{ in}$$

$$\varepsilon_t = \frac{d_p - c}{c}(0.003) = \frac{8 - 0.9825}{0.9825}(0.003) = 0.02 > 0.005$$

$\phi = 0.9$

$$\phi M_n = \phi A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right) = 0.9(0.612)(261.89)\left(8 - \frac{0.786}{2}\right)$$

$$= 1097.3 \text{ Kip-in/slab}$$

$$= 91.5 \text{ Kip-ft/slab}$$

→ ACI 318-11

$$\max \begin{cases} 1.4D \\ 1.2D + 1.6L = 226 \text{psf} \end{cases}$$

$$M_u = \frac{(24.5)^2}{8}(0.226)(4) = 67.83 \text{ Kip-ft}$$

$$67.83 \text{ Kip-ft} < 91.5 \text{ Kip-ft} \quad \text{OK}$$

Alternate Designs:

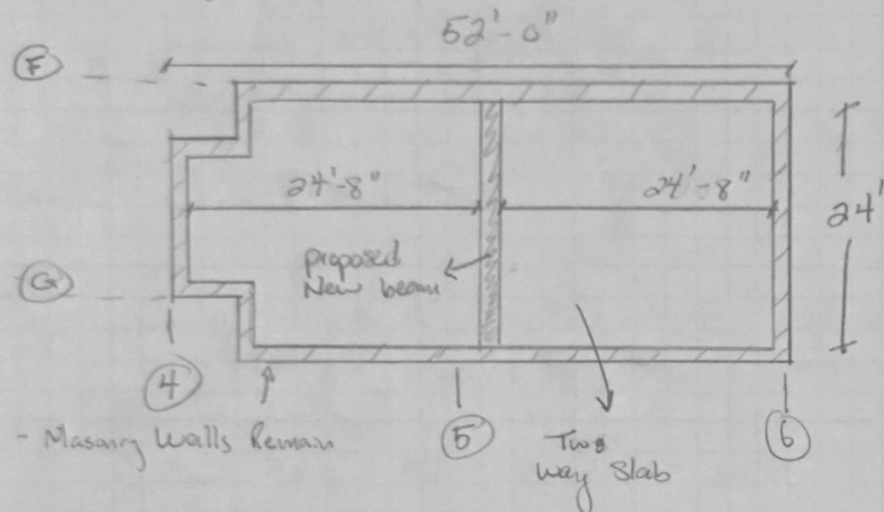
The first alternate design was tested in the residential bay to see how a two-way slab would work in place of the hollow core plank. A beam was place in the middle of the bay spanning across to the CMU bearing walls. The CMU walls surrounding the slab take the vertical weight of the floor system.

The second system is a flat slab with drop panels. It is designed to replace the existing parking garage bay. The main purpose for choosing this system is to eliminate the some of the concrete in the original floor and only keep it at the critical sections of the floor.

The third alternate system is a steel frame and composite deck. This was chosen to see if there was a realistic steel option for the structural system. This new system will also include columns that run continuously through the building with a splice between floors 5 and 6. This is done in order to eliminate the transfer slab and long span sections that the apartment layouts create. These columns and their locations will need further addressing and study to determine if their continuous run will be too much of an obstruction for the apartment units.

Alternate System 1 - Two Way Slab

Typical Bay



- CMU Walls Will remain & take load

Design of Two way Slab:

CMU Walls act as beams in this design.

$l/s < 2.0 = \frac{24}{24.67} = 0.97 < 2.0$ ✓

Direct design Method applies see Garage Bay Spot Check

Assume 8" thick Slab.

see grav.

Dead Load = 100 psf $\quad 1.2(135) + 1.6(40) = 226\text{ psf}$

$$M_{o_1} = \frac{q_u l_2 l_n^2}{8} = \frac{(0.226)(24)(24.67)^2}{8} = 412 \text{ Kip-ft}$$

$$M_{o_2} = \frac{q_u l_2 l_n^2}{8} = \frac{(0.226)(24)^2(24.67)}{8} = 401.4 \text{ Kip-ft}$$

Use 412 kip-ft for Calcs

Factored Static Moment:

M_{ex}^-	M^+	M_{int}^-
0.16(412)	0.57(412)	0.7(412)
-65.92'k	235'k	-288.4'k

Moments in Column & Middle Strip

Interior: Column Strip $M^- = 0.75(-288.4) = -216.3'K$

Middle Strip $M^- = 0.25(-288.4) = -72.1'k$

Column Strip $M^+ = 0.6(235) = 141'K$

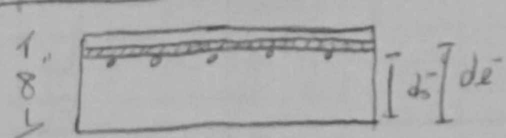
Middle Strip $M^+ = 0.4(235) = 94'k$

Exterior: Column Strip $M^- = 1.0(65.92) = -65.92'K$

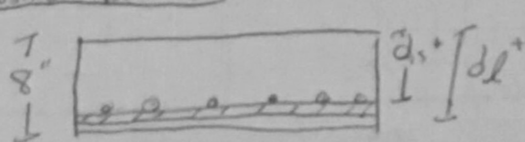
Middle Strip $M^- = 0$

Strip	Left M	Middle M^+	Right M^-
Middle	0	94'	36.05
Column	32.96	141	108.15

Negative Moments



Positive Moments



Assume # 6 bars

3/4" Clear Cover

$d_{\ell}^{-} = d_{\ell}^{+} = 8 - 3/4 - \frac{1}{2}(0.75) = 6.875"$

$d_{s}^{-} = d_{s}^{+} = 8 - 3/4 - 0.75 - \frac{1}{2}(0.75) = 6.125"$

Middle:

	Left M^-	Middle M^+	Right M^-
M_u	0	94'	36.05
$M_{s(w)}$	148"	148"	148"
d	6.875	6.875"	6.875"
A_s req	0	3.20	1.23
A_s min	2.13	2.13	2.13
S_{max}	16	16	16
N	5	8	5
N_{min}	10	10	10
Spacing	30.6"	20.36	30.6

Use # 6 bars

L & R

(10) #6 @ 24"

Mid

(16) #6 @ 20"

← Controls.

Column

	Left M^-	Middle M^+	Right M^-
Mu	32.96	141	108.15
C_s (b)	74"	74"	74"
d	6.875"	6.875"	6.875"
As req	1.12	4.8	3.68
As min	1.07	1.07	1.07
S_{max}	16	16	16
N	3	11	9
N min	5	5	5
Spacing	14.8	6.73	8.22

Use #6 bars

Left (5) #6
14"

Mid (11) #6
6.5"

Right (9) #6
8"

Alternate System 1.

Beam Design - See two way design for graphic.

$L = 24'$ $\quad f_y = 60\ ksi$

$f'_c = 5000\ psi \rightarrow 0.85 - 0.05\left(\frac{1000}{1000}\right) = 0.8$

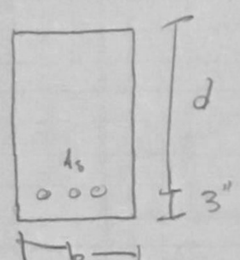
Load: See gravity loads & Two way design

226 psf

$226\ psf \times (24' - 8") = 5.6\ klf$

Cross Section.

$b = 0.5d$



$M_u = \frac{wl^2}{8} = \frac{5.6(24)}{8} = 16.8\ k\text{-}ft$

$\rho = \frac{0.25(5)(0.8)}{60} = 0.0167$

$\omega = \frac{0.0167 \cdot 60}{5} = 0.2004$

$R = 0.2004(5)(1 - 0.59(0.2004))$

$= 0.88$

$\rho_{req} = \frac{0.85(5000)}{60}\left(1 - \sqrt{1 - \frac{2(88)}{0.85\sqrt{5000}}}\right) = \underline{0.001}$

$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} b_w \cdot d = \frac{3\sqrt{5000}}{60000} 0.5d^2$

Assume (3) #9

$a = \frac{2.37(60)}{0.85(5)(12)} = 2.79\ in$

$c = a/\beta_1 = \frac{2.79}{0.8} = 3.49 \text{ in}$

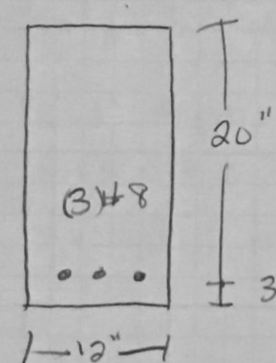
$\varepsilon_s = \frac{(20 - 3.49)}{3.49} 0.003 = 0.014 > 0.005$ ok $\phi = 0.9$

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

$0.9(2.37)(60)\left(20 - \frac{2.79}{2}\right) = 198' k$

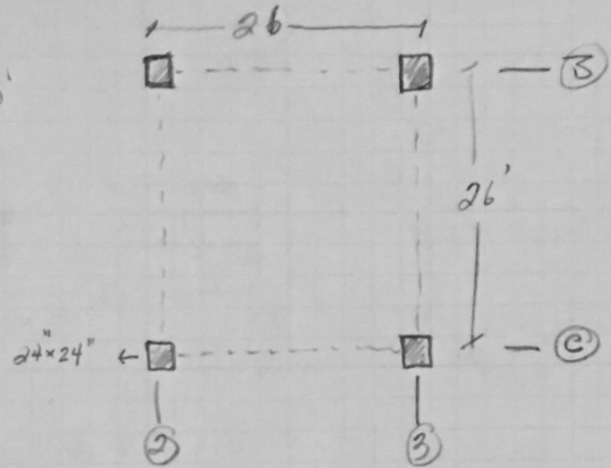
$198'k > 168'k$ ok

Design



Alternate System 2. Flat Slab with Drop panels

Bay:



Loads on Bay: See gravity section

Live: 40 psf

Dead: SW + 10 psf (see gravity Section)

Determine Min Slab depth to eliminate deflection Calcs.

Interior panel:

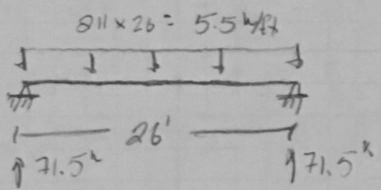
Drop panels edge Beams ← $\frac{l_n}{34}$ $\quad \frac{26-1}{34} \times 12 = 8.82 \rightarrow$ Use 9"

Dead load ⇒ $\frac{9}{12} \times 150 = 112.5$ psf

Combo: $1.2(122.5) + 1.6(40) = \underline{211 \text{ psf}}$

Determine if Slab adequate for Shear.

One way action



$V_u = 71.5 - 5.5\left(\frac{7}{12}\right) = 66$

$V_n = V_c = 2\lambda\sqrt{f'_c}\cdot b_w \cdot d$

$V_n = 2(1.0)\sqrt{5000}\cdot 26\cdot 12\cdot 9$

$= 398^k$

$\phi V_n \geq V_u$ ✓

- Adequate in one way ∴ both directions are the same

Two way action:

$\text{tributary area} = 26 \times 26 - \left[\frac{24+9}{12} * \frac{24+9}{12}\right] = 670.5\ ft^2$

$670.5 \times 211 = 142^k$

$b_o = [(24\times 2) + (9\times 2)]\cdot 2 = 132''$

$\beta_1 = 24/24 = 1.0$

$\alpha_s = 40$

$$\begin{cases} (2 + \frac{4}{1})(1)\sqrt{5000}\cdot 132\cdot 9/1000 = 504^k \\ (\frac{40\cdot 9}{132} + 2)(1)\sqrt{5000}\cdot 132\cdot 9/1000 = 397^k \\ 4(1.0)\sqrt{5000}\cdot 132\cdot 9/1000 = 336^k \rightarrow \text{Controls.} \end{cases}$$

$0.75(336) = 252^k$

$252^k > 142^k$ Good for Shear Strength

Provide drop panels:

Start with 3" drop 12" total

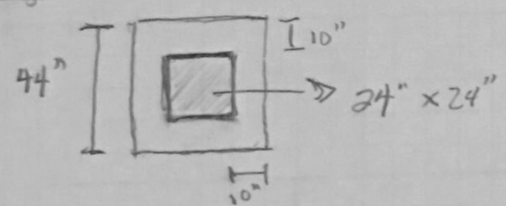
$V_u = 211 \times \left[26 \times 26 - \left(\frac{x+9}{12} + \frac{x+9}{12} \right) \right]$ $\quad b_o = 4(x+9)$

$\phi V_c = 0.75(4)\sqrt{5000}\,(4[x+9])(9)$

→ Set = & Solve for $x = 40$ in

Use 10" either side of column

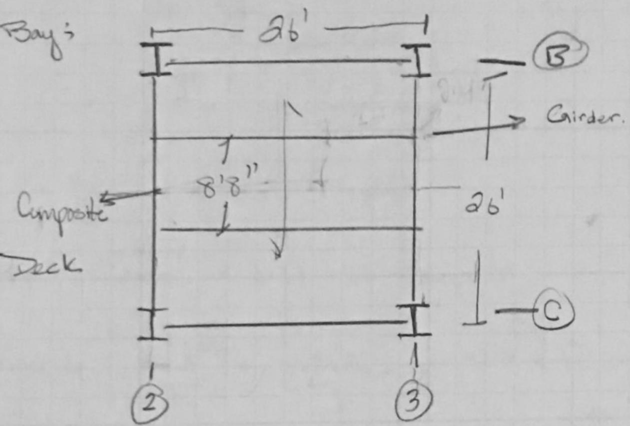
Design



Columns will remain 24" x 24" from original design because this change is lighter than original design.

Alternate System 3 - Composite Deck

Bay:



Deck Choice.

2.5 hr fire rating

3 spans 8'-8"

Load :

Live: 40psf

Dead: 5 for Steel allowance

10 see gravity (Misc)

: 55 + SW

- Deck Type: 3LVF22 Composite deck (45psf)

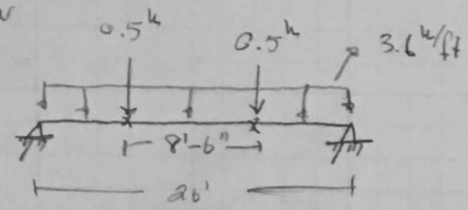
3 span Max 11'-8" > 8'-6"

8'-6" = 161 psf

Check S.W

55 + 45psf = 100psf < 161psf ✓ OK

Design Girder



Braced at 3rd points

Load: 136 psf See Alt design 3

$136 \times 26' = 3.6^{k}/ft$

Max Moment:

$$\frac{wl^2}{8} + Pa = \frac{3.6 \cdot (26)^2}{8} + 0.5(8.67) = 309 \text{ k-ft}$$

$C_b = 1.$ conservative from Table 3-1

$\phi M_n = \frac{309}{1.01} = 306$ k-ft Table 3-10 8'-6" Unbraced Length

↳ W16 x 50

Deflection:

$$\frac{26 \times 12}{360} = 0.867 \text{ in}$$

$$\Delta = \frac{5wl^4}{384 \cdot E \cdot I} + \frac{Pl^3}{48 \cdot E \cdot I} = \frac{5(3.6)(26^4) \cdot 1728}{384 \cdot 29000 \cdot 659} + \frac{15(26^3)144}{48 \cdot 29000 \cdot 659}$$

Negligible

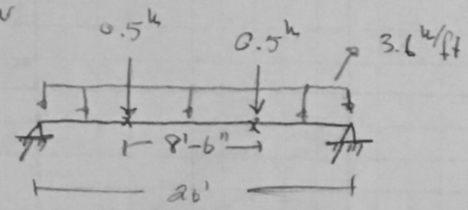
$= 1.98$ × No good

Solve for I → $I = \frac{5(3.6)(26^4) \cdot 1728}{384 \cdot 29000 \cdot .862} = 1506$

Go with W24x62

$\phi M_n = 485$ k-ft ✓ ok

Design Girder



Braced at 3rd points

Load : 136 psf See Alt design 3

$136 \times 26' = 3.6^{k}/ft$

Max Moment :

$$\frac{wl^2}{8} + Pa = \frac{3.6 \cdot (26)^2}{8} + 0.5(8.67) = 309 \text{ k-ft}$$

$C_b = 1.$ conservative from Table 3-1

$\phi M_n = \frac{309}{1.01} = 306$ k-ft Table 3-10 8'-6" Unbraced Length

↳ W16 x 50

Deflection :

$$\frac{26 \times 12}{360} = 0.867 \text{ in}$$

$$\Delta = \frac{5wl^4}{384 \cdot E \cdot I} + \frac{Pl^3}{48 \cdot E \cdot I} = \frac{5(3.6)(26^4) \cdot 1728}{384 \cdot 29000 \cdot 659} + \frac{15(26^3)144}{48 \cdot 29000 \cdot 659}$$

Negligible

$= 1.98$ × No good

Solve for I → $I = \frac{5(3.6)(26^4) \cdot 1728}{384 \cdot 29000 \cdot .862} = 1506$

Go with W24x62

$\phi M_n = 485$ k-ft ✓ ok

Alternate Design 3

Column B3

- Interior Column.
- Steel Design will feature columns Running from floor 1-5. This will effect apt architecture / layouts.
- Splice column @ floor 5.5 to Roof.

Column Loads / Floor

Trib Area / floor = $(9+13)(26) = 572\ ft^2$

Influence Area / floor = $(18+26)(52) = 2288\ ft^2$

LL (Unreduced) = 40 (gravity section)

LL Reduction = $\Big|_{0.4}\ 0.25 + \dfrac{15}{\sqrt{4(2288)}} = 0.41$

LL / floor = $0.41 \cdot 572 \cdot 40 = 9.4^k$

Roof DL = 100 psf see end of section

Roof DL = $100(572) = 57.2^k$

Snow Load = 40 (see gravity section) -> $40(572) = 22.9^k$

Dead Load = 100 psf see Alt design 3

Dead Load / floor = 57.2^k

Load Summary:

B3

	Dead (k)	Live (k)	Snow (k)
Roof :	57.2^k	x	22.9^k
2, 3, 4, 5, 6, 7 Floors :	57.2^k	9.4^k	x
Total	400.4	56.4	22.9^k

Combo:

$$1.2(400.4) + 1.6(56.4) + 0.5(22.9) = \underline{582.17^k}$$

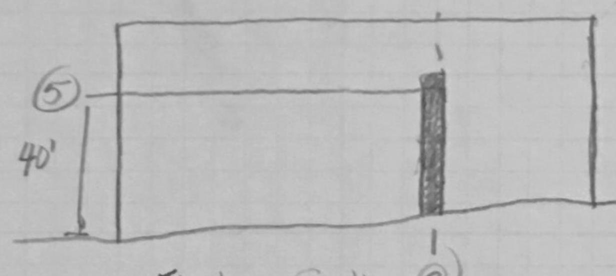
Column 40' high

Column is braced at every floor

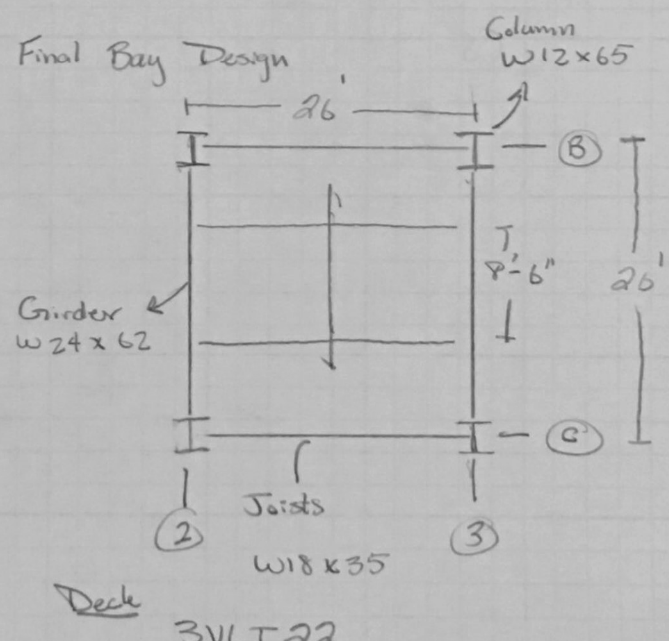
Longest L_e w/out bracing is between grade & floor 3

$16' = L_e$

W12 x 65



Elevation South



System Comparison:

	Existing	Two-Way	Flat Slab w/Drop Panels	Composite Steel
Weight (psf)	100 OR 150	100	112.5	100
Cost	$14 psf	$16 psf	$15 psf	$15 psf
Depth	8”- 12”	8” slab + 23” Beam	9” – 12”	5” deck + 24” Beam
Fire Rating	2 Hour	2.5 Hour	2.5 Hour	2.5 Hour
Reasonable System		No	Yes	Yes

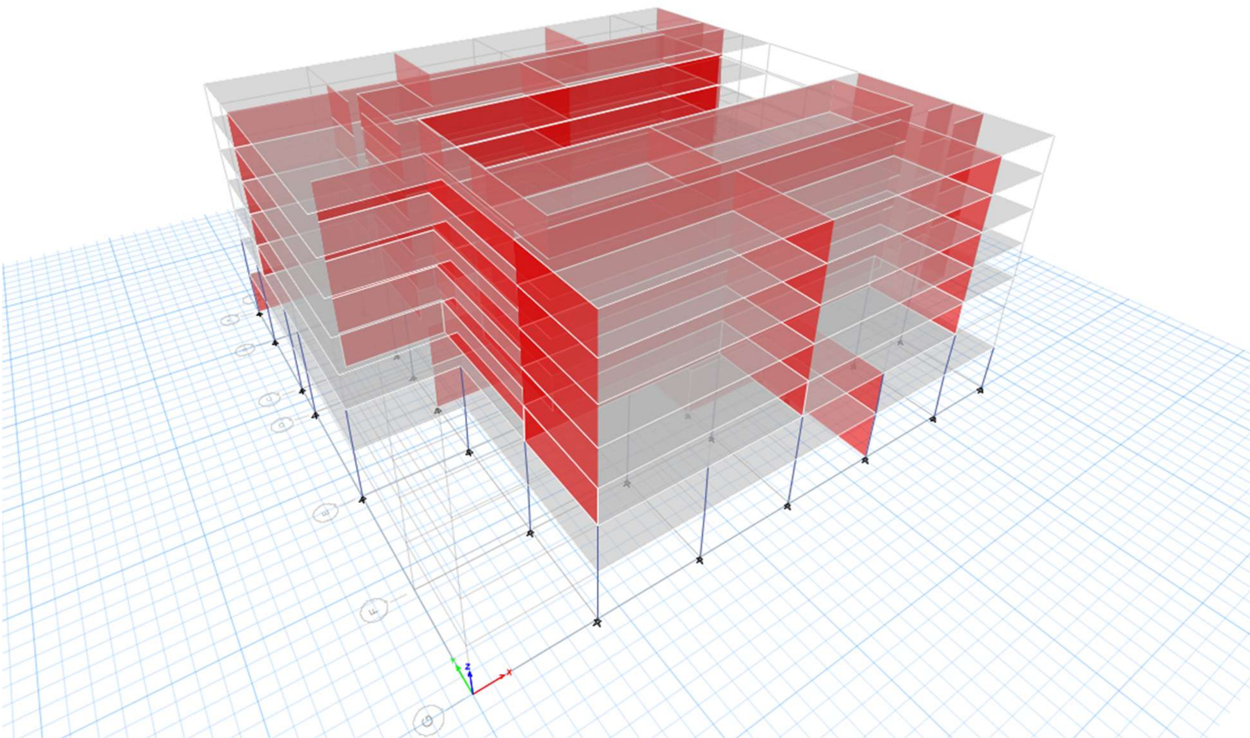
The two-way system has been ruled out because of the large beam and overall high cost. The system does not seem to be practical because the 23” beam is located under the apartments. In areas of the building with more load, it will be more work than its worth to pack in multiple large beams.

The flat slab with drop panels method will be the best option in the parking garage because it cuts out unneeded concrete and calls for more concrete in the critical areas. This also cuts back on cost and the overall weight of the floor system. For this system to be adopted, the next step is to check the large transfer slab to see if some concrete cut out of that slab.

The most promising design alternative seems to be the composite steel deck system. The major issue with this system is the column run. In order to eliminate the transfer slab between parking and residential floors, the columns will continue up through the building. This will be a challenge to change apartment layouts and to convince the architect that the columns in the apartments are not an issue. This system will also allow for much faster construction because CMU blocks to do not have to be laid by hand.

8.0 Lateral Analysis

The lateral analysis for building A consists of an ETABS model and hand calculations.



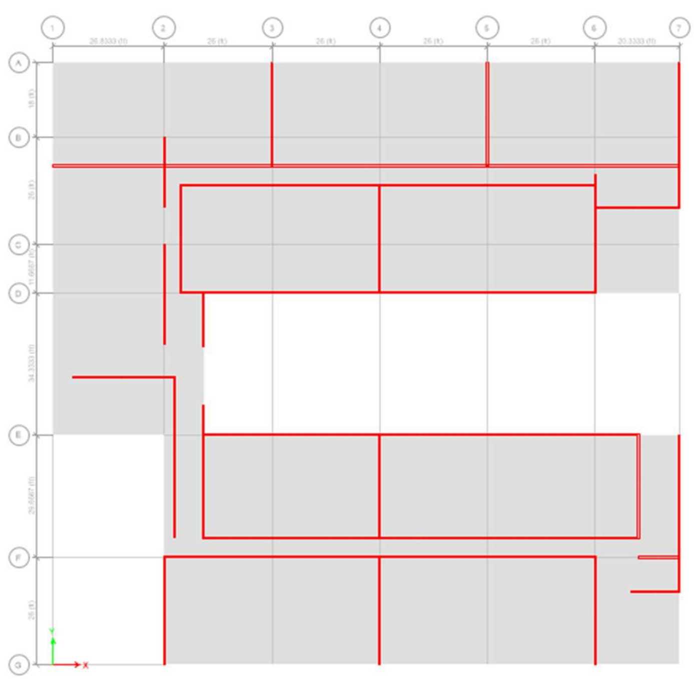
8.1 Model Information

ETABS was used as the modeling software to study the lateral system in Building A. Dead loads were not included in the model, but the self-weight of the structure was included. All wind and seismic loads were taken from section 6&7. These loads were hand calculated as story shear loads and entered as user loads into the software.

Model Assumptions

- Columns on the first floor are all equal length. Longest length chosen to verify strength. (13’ 8”)
- Foundation not modeled. Base reactions are all pins.
- Interior walls that feature doors were modeled as continuous. The longest wall of 154’ only has 3 doorways @4’. These openings were considered to be negligible to the entire wall.
- All columns modeled are Column C1 (see column section)
- All walls modeled as “W5”. Indicated in drawings. 8” CMU #5 bars @48” O.C.
    - For model, the thickness was modified to 4” thick to more accurately depict wall. (see CMU section).

Overall building simplified to this design for easy calcs and modeling. Slight variation, does not impact the overall building. Outside walls are not modeled because there are too many openings.



8.2 Loading

Hand calcs compared to the User input load base shear results.

Base Reactions	Computer Output	Notebook A Calcs	% diff
Wind E-W	283k	306k	7.5%
Wind N-S	295k	326k	9.5%
Seismic E-W	226k	220k	2.7%
Seismic N-S	226k	220k	2.7%

Computer analysis provided results that were acceptable for seismic. Wind results, were slightly lower than the hand calculated values but they should not present any issues with further analysis.

8.3 Drift and Serviceability

Displacements (in)				
Floor	Wind E-W	Wind N-S	Seismic E-W	Seismic N-S
1	0.00533	0.005743	0.003704	0.003716
2	0.01115	0.007797	0.00693	0.007335
3	0.01225	0.00848	0.00805	0.00823
4	0.01303	0.008856	0.00896	0.00847
5	0.01375	0.009201	0.00925	0.00923
6	0.01441	0.00952	0.01125	0.00967
7	0.01501	0.00981	0.0125	0.01007

All displacements vary. The E-W wind case creates the greatest displacement for each floor. This were checked against code below, all were much lower than code requires.

Displacement Checks

The controlling displacemnt was the E-W wind case.

Seismic loading also cleared the code displacemnt.

Wind E-W	h/400				
Floor	Height	Height From B	Code	ETABS	Check
7	9.33	72.32	2.1696	0.01501	Yes
6	9.33	62.99	1.8897	0.01441	Yes
5	9.33	53.66	1.6098	0.01375	Yes
4	9.33	44.33	1.3299	0.01303	Yes
3	9.33	35	1.05	0.01225	Yes
2	12	25.67	0.7701	0.01115	Yes
1	13.67	13.67	0.4101	0.00533	Yes

Seismic E-W	0.02h			
Floor	Height	Code	ETABS	Check
7	9.33	2.2392	0.0125	Yes
6	9.33	2.2392	0.01125	Yes
5	9.33	2.2392	0.00925	Yes
4	9.33	2.2392	0.00896	Yes
3	9.33	2.2392	0.00805	Yes
2	12	2.88	0.00693	Yes
1	13.67	3.2808	0.003704	Yes

Story Drift Checks

Floor	Height	.007h	Drift X	Drift Y	Passed
1	13.67	0.09569	0.00004	0.00002	Yes
2	12	0.084	0.00004	0.00002	Yes
3	9.33	0.06531	0.000006	0.000001	Yes
4	9.33	0.06531	0.000003	0.000001	Yes
5	9.33	0.06531	0.000003	0.000004	Yes
6	9.33	0.06531	0.000003	0.000004	Yes
7	9.33	0.06531	0.000003	0.000004	Yes

Story drift was checked using the maximum load case (Wind N-S). All were well beyond the code for masonry buildings

8.4 Stiffness

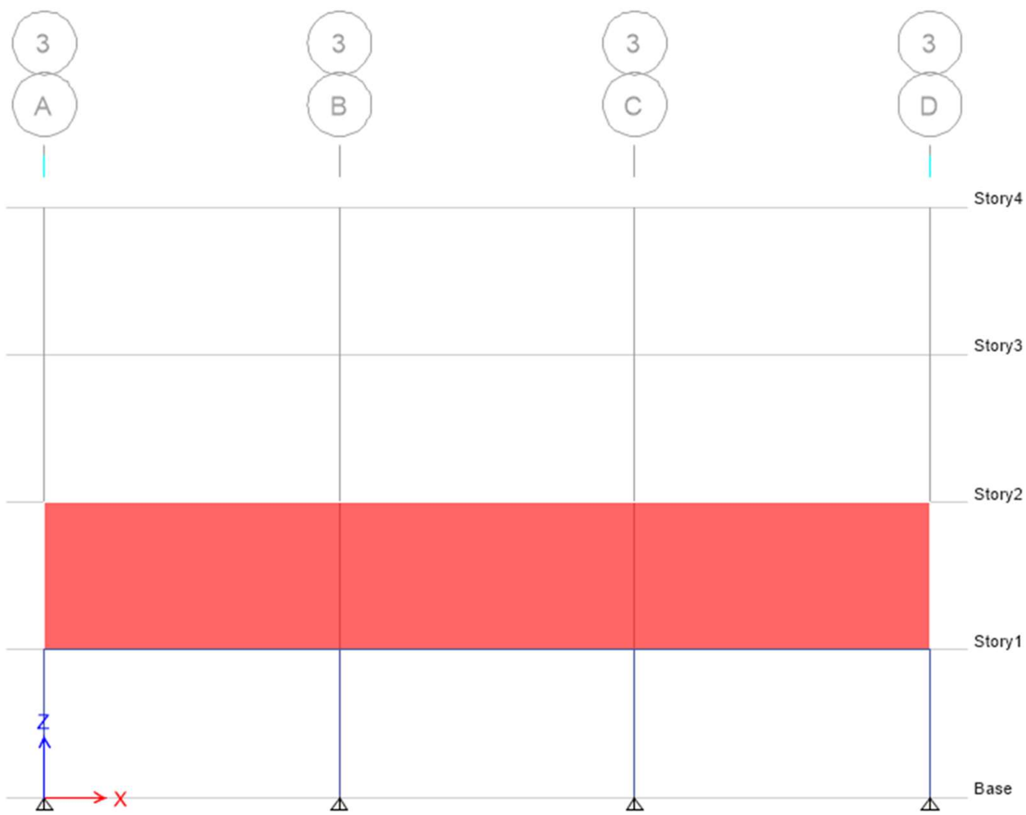
The stiffness of the CMU wall was calculated using ETABS. A separate model was built to check this. This model tests the same wall that was put in to the building model. The model assumes a fixed base due to the rebar tie ins located every 48” at a minimum.

Applying a 1k load to the top of the wall section resulted in displacement of 0.000134in.

To calculate the stiffness of the wall, the inverse of the displacement was taken. This gave a stiffness of 7463 k/in.

Another section was tested for stiffness as well (seen below). The same wall was modeled resting on a 26” deep beam and 13’8” columns. This was to simulate the podium that the building rests on.

The stiffness of this system was 362 k/in. This is much less than the fixed base CMU wall. This indicates that the top floors (all CMU walls) are resisting most of the lateral load.



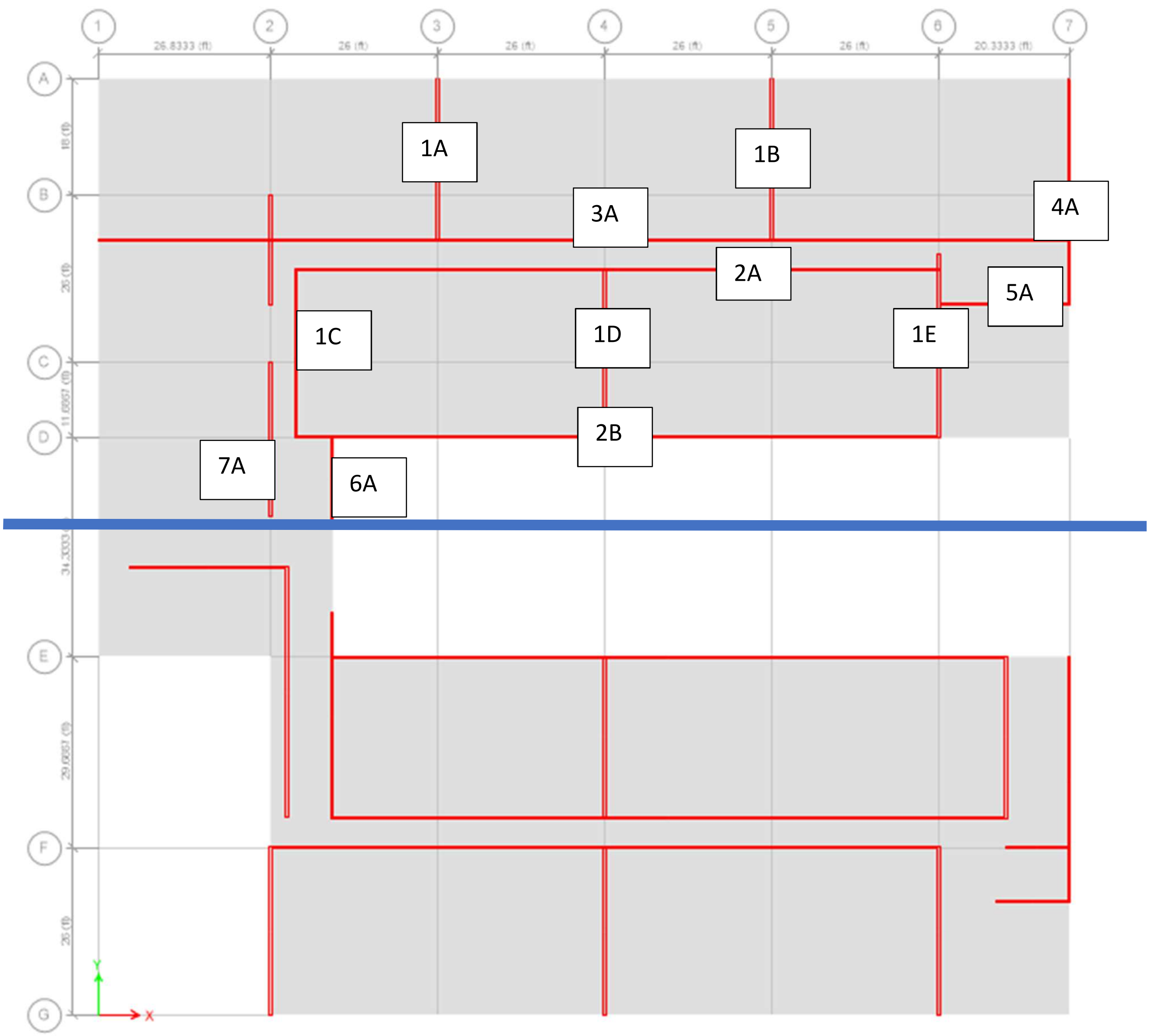
8.5 Center of Mass and Center of Rigidity

The center of mass and center of rigidity were hand calculated at the 3rd floor to verify the computer model.

Center of Mass

The center of mass was calculated by diving the building into a top half and bottom half, then averaging the two. Below are the tables used to calculate the top half of floor 3. (see appendix for floor calcs and COM averaging.)

This image shows where the building was divided and indicates where each wall is.



COM Top Half

All 8" CMU	Density	Length	Height	Mass (lb)
1	125	26	9.33	30322.5
2	125	104	9.33	121290
3	125	159	9.33	185433.75
4	125	57	9.33	66476.25
5	125	16	9.33	18660
6	125	13	9.33	15161.25
7	125	45.5	9.33	53064.375
Sum				

Wall Coordinates	X	Y	Mass	Mi*X	Mi*Y
1A	47	13	30322.5	1425157.5	394192.5
1B	100	13	30322.5	3032250	394192.5
1C	32	45	30322.5	970320	1364512.5
1D	84	45	30322.5	2547090	1364512.5
1E	68.5	45	30322.5	2077091.25	1364512.5
2A	84	32	121290	10188360	3881280
2B	84	58	121290	10188360	7034820
3A	76	26	185433.75	14092965	4821277.5
4A	152	28.5	66476.25	10104390	1894573.125
5A	68.5	45	18660	1278210	839700
6A	36	64.5	15161.25	545805	977900.625
7A	25	46.5	53064.375	1326609.375	2467493.438
Floor	76	28.5	609700	46337200	17376450
			1342688.125	**104113808.1**	**44175417.19**

This shows the center of mass for the top half of the building.

Column1	Column2
MX	104113808
Weight	**1342688**
X	**77.54132606**

Column1	Column2
MY	44175417
Weight	1342688
Y	**32.90073122**

COR Y

Wall Type	Stiffness	Y	KY
1	7463	26	194038
2	7463	32	238816
3	362	57	20634
4	362	90	32580
5	7463	115.5	861976.5
6	7463	122	910486
Sum	30576		2258531

COR X

Wall Type	Stiffness	X	KX
1	362	48	17376
2	362	100	36200
3	7463	152	1134376
4	7463	32	238816
5	7463	84	626892
6	7463	136	1014968
7	7463	25	186575
8	7463	39	291057

3rd Floor COM and COR

	Center of Mass	Center of Rigidity
Computer	(79’, 75.8’)	(88.7’, 75.4)
Calculated	(79.5’, 74.75’)	(84’, 74’)

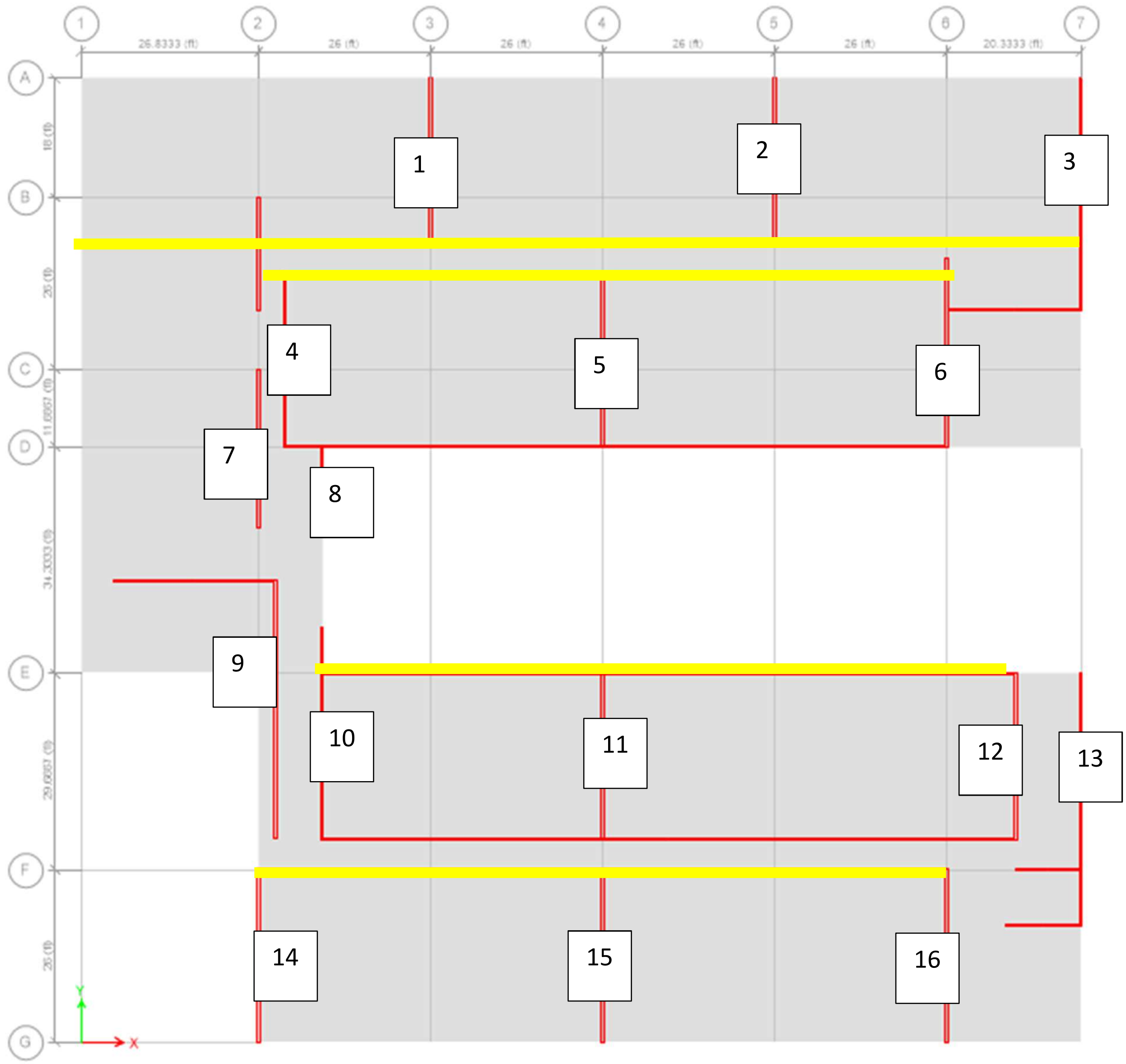
Both the calculated COM and COR are close to their respective computer values. The COR value is slightly off, this is most likely due to my assumption of stiffness in each wall. Below are hand calcs that show how the COR was Calculated.

Direct and Torsional Shear

Wall	Direction	K	X	d (COR @ 84')	Kd	Kd^2	V (N-S Wind Shear)	M	Direct	Torsional	Total
1	X	362	48	-36	-13032	469152	37	166.5	0.16	-4.625	-4.465
2	X	362	100	16	5792	92672	37	166.5	0.16	10.40625	10.56625
3	X	7463	152	68	507484	34508912	37	166.5	3.29	2.448529412	5.738529
4	X	7463	32	-52	-388076	20179952	37	166.5	3.29	-3.201923077	0.088077
5	X	7463	84	0	0	0	37	166.5	3.29	0	3.29
6	X	7463	136	52	388076	20179952	37	166.5	3.29	3.201923077	6.491923
7	X	7463	25	-59	-440317	25978703	37	166.5	3.29	-2.822033898	0.467966
8	X	7463	39	-45	-335835	15112575	37	166.5	3.29	-3.7	-0.41
9	X	7463	32	-52	-388076	20179952	37	166.5	3.29	-3.201923077	0.088077
10	X	7463	39	-45	-335835	15112575	37	166.5	3.29	-3.7	-0.41
11	X	7463	90	6	44778	268668	37	166.5	3.29	27.75	31.04
12	X	7463	143	59	440317	25978703	37	166.5	3.29	2.822033898	6.112034
13	X	7463	152	68	507484	34508912	37	166.5	3.29	2.448529412	5.738529
14	X	362	26	-58	-20996	1217768	37	166.5	0.16	-2.870689655	-2.71069
15	X	362	79	-5	-1810	9050	37	166.5	0.16	-33.3	-33.14
16	X	362	133	49	17738	869162	37	166.5	0.16	3.397959184	3.557959
						214666708					

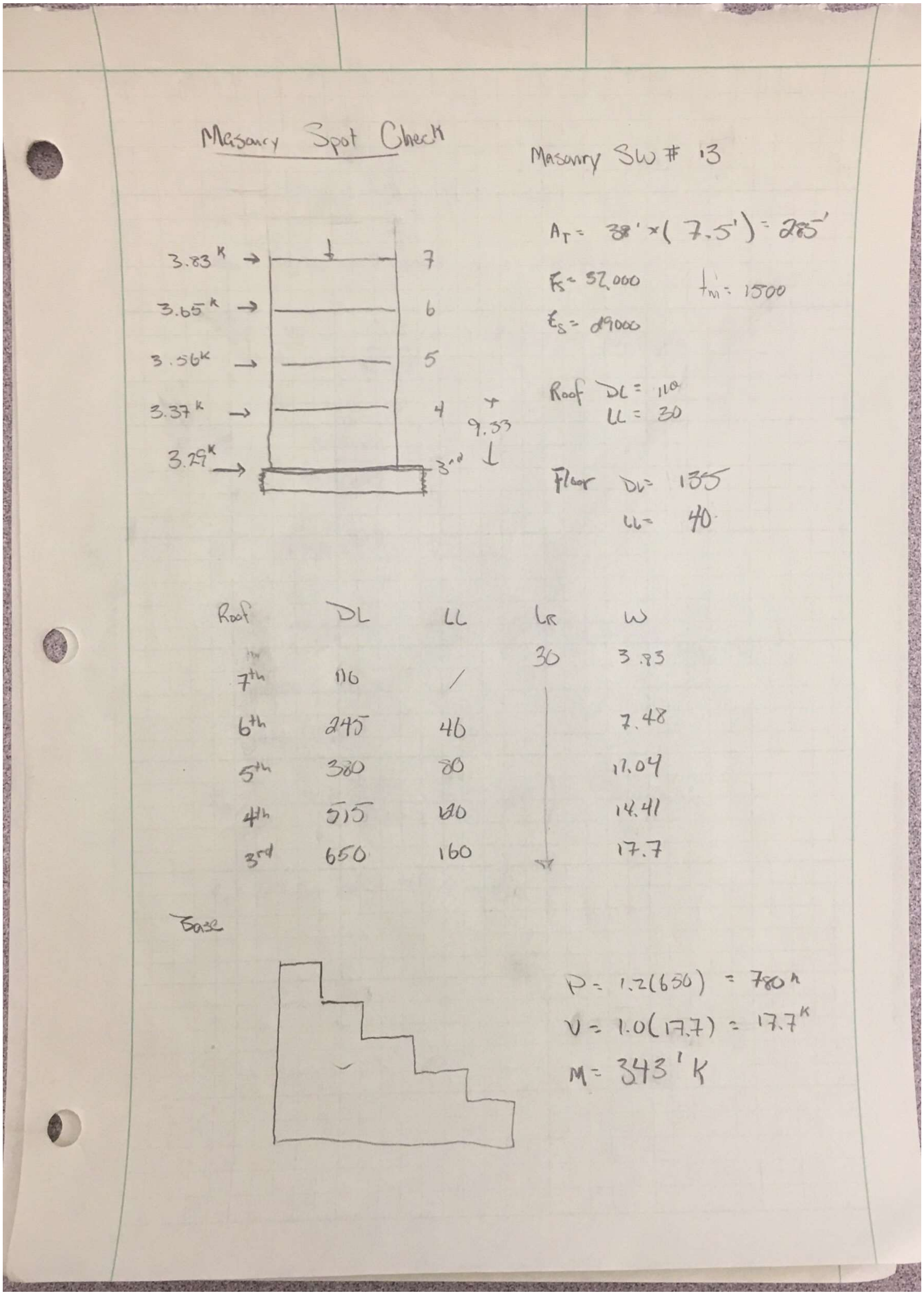
The direct and torsional components of the force are shown here in this chart (see appendix for eccentricity diagram). The stiffness for each wall was determined by looking at where the walls in floor 3 sit above to column. In the case of walls 1, 2, 14, 15,and 16, they all rested in a similar fashion to the podium tested in the stiffness section. They were given a k value of 362k/in to represent this. See the spot check section for verification of the direct force seen in the walls.

In all cases, the y direction is not a critical area for torsion because the COM ad COR were relatively close to each other. The wind N-S direction will be investigated to see how much shear and torsion is seen in the walls because it is the largest shear. The next page shows a diagram for the wall numbering system.



The walls highlighted in yellow were neglected because about half of each wall rested to either side of the line of action, thus canceling each other out for torsion.

8.6 Spot Check



$d = (9'3")(12) - 3" = 108"$

$A_{nv} = t \cdot d = 7.625(108) = 823.5$

$f_v = \frac{V}{A_{nv}} = \frac{17.7}{823.5} = 0.0215 \rightarrow 21.5 psi$

$\frac{M}{Vd} = \frac{343 \times 12}{17.7(108)} = 2.15 \leq 1.0$

$F_v = 2\sqrt{1500} = 77.5 psi$

$F_{vm} = 0.5\left[4 - 1.75(1.0)\sqrt{1500}\right] + 0.25\left(\frac{650(1000)}{7.625(132)}\right)$

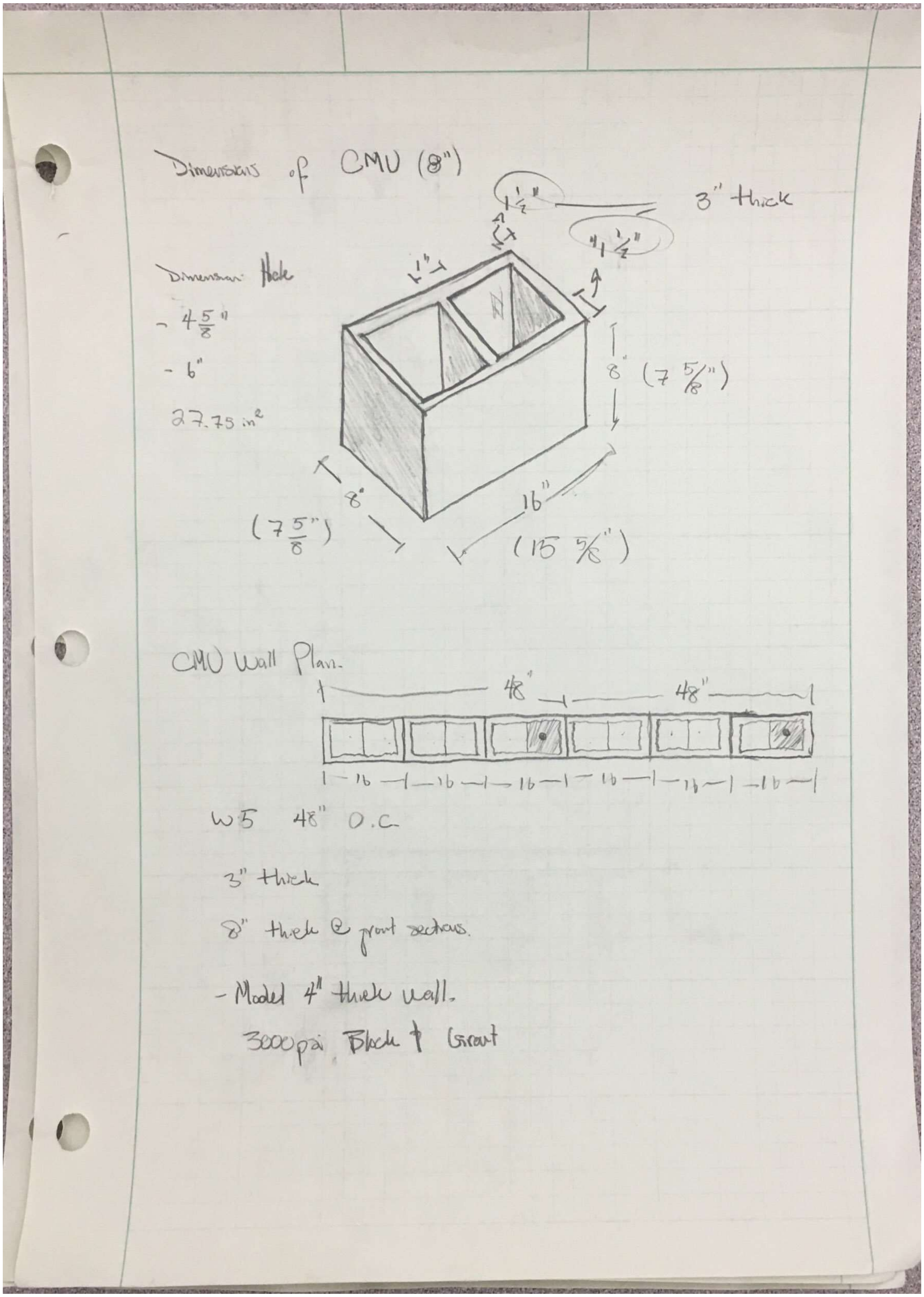
$108 \geq 21.5$ ok

Design OK

$F_{vs} = 0.5\left(\frac{(0.625)(32,000)(108)}{(823.5)(48)}\right) = 27.3 psi$

$F_v = 27.3 + 108 \quad 717.2$ ✓

Appendix


Dimensions of CMU (8")
3" thick
Dimension Hole
- 4 5/8"
- 6"
27.75 in²
8" (7 5/8")
8" (7 5/8")
16" (15 5/8")
CMU Wall Plan
48"
48"
16 16 16 16 16 16
W5 48" O.C.
3" thick
8" thick @ pront sections.
- Model 4" thick wall.
3000psi Block & Grout

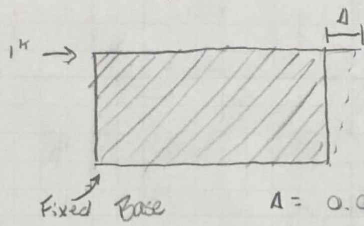
Calculate Stiffness.

$$K_{wall} = \frac{1}{\left[\frac{H^3}{E t L^3} + \frac{1.2H}{G t L}\right]}$$

W5 - CMU Thickness 8"

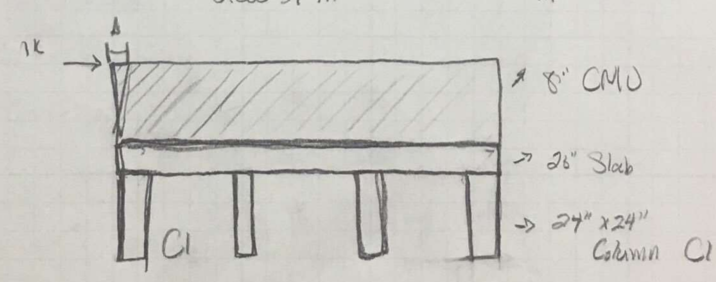
#5 bars @ 48" O.C.

From ETABS →



$\Delta = 0.000134$ in

$$\text{Stiffness} = \frac{1\ k}{0.000134\ in} = 7462.7\ \frac{kip}{in}$$



$\Delta = 0.002763$ in

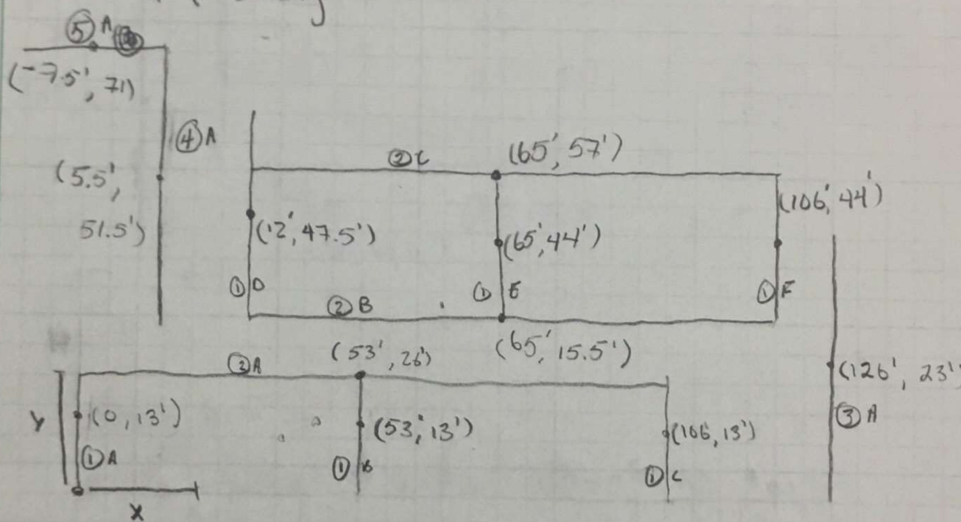
$$\text{Stiffness} = \frac{1^k}{0.002763\ in} = 361.92\ \frac{k}{in}$$

COM

Calculate Center of Mass Floor 3

$\bar{x} = \dfrac{\Sigma M_i \, x_i}{\Sigma M_T}$ | $\bar{y} = \dfrac{\Sigma M_i \cdot y_i}{\Sigma M_T}$

Bottom Half of Building



- Assume wall weight is equal in all dir, will cancel out.
  - only slab & CMU Partitions Considered.

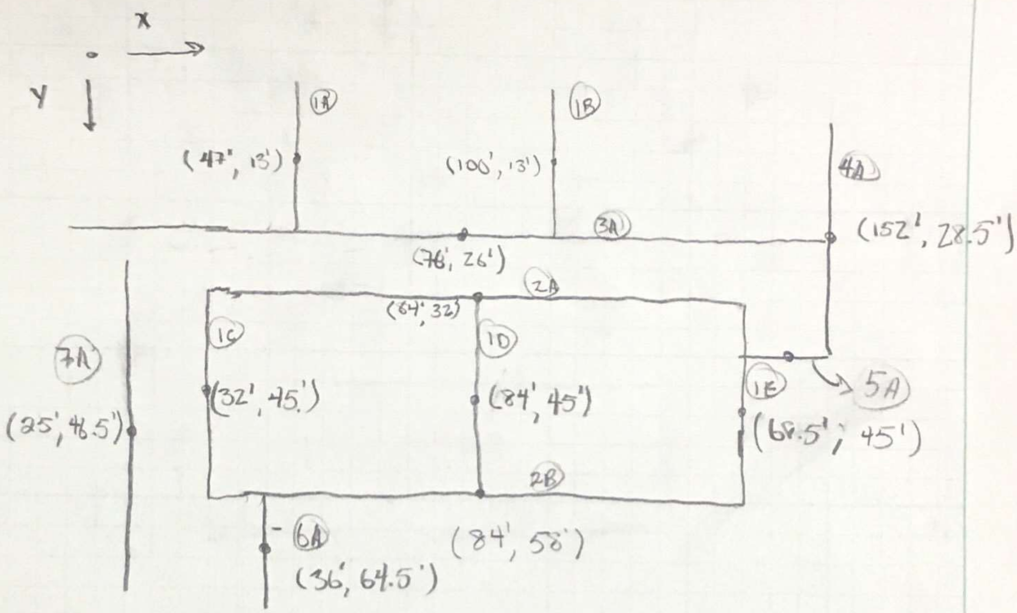
Floor Center | $\frac{126'}{2} = 53'$ | 67 psf x 8441

$\frac{57'}{2} = 28.5'$

Floor Weight: 8" Slab Hollow Core

$\frac{8}{12} \times 150 = 100 \text{ psf} \times 0.67 \text{ VR}$ 67 psf

COM Top Half



– Floor 9100 sf.

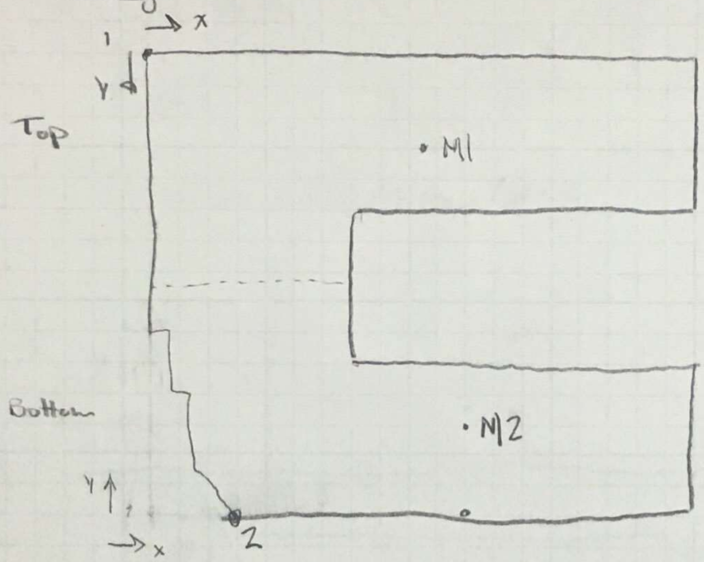
8" thick → 67 psf × 9100 = 609700

Center $\frac{152}{2} = 76'$

$\frac{57}{2} = 28.5$

COM Entire

Building



Top : (77.5', 32.9') point 1

Bottom : (55.5', 31.5') point 2

- Need to move point 2 to point 1.

$1 \rightarrow 2$ (26', 147.5')

$\rightarrow$ x: 26' + 55.5' = 81.5' ✓

y: 147.5' - 31.5' = 116 ✓

Avg. Points: x: $\frac{81.5 + 77.5}{2}$ = 79.5'

y: $\frac{32.9 + 116}{2}$ = 74.75'

Computer

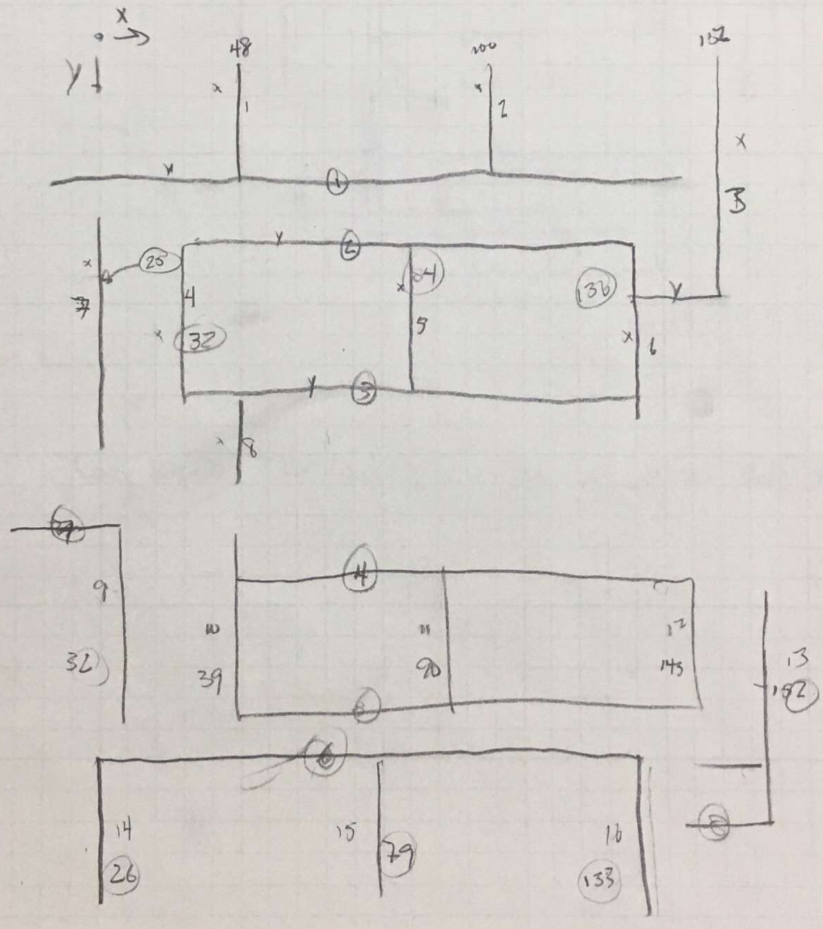
x: 79.0'

y: 75.8'

COR

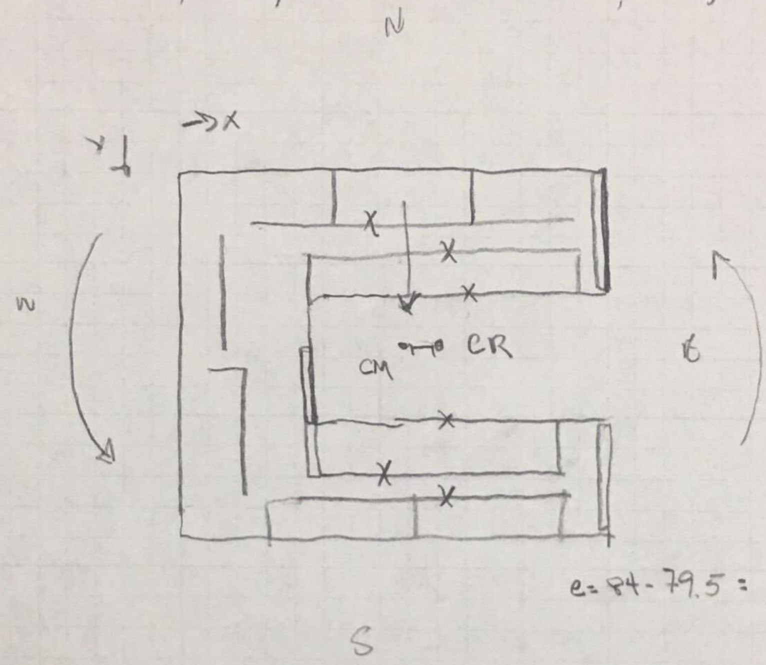
$$\frac{\Sigma K_i \cdot X_i}{\Sigma K_i} = \bar{X} \qquad \frac{\Sigma K_i \cdot Y_i}{\Sigma K_i} = \bar{Y}$$

- Inside walls lie on Columns



COR (84', 74')

CM (79.5, 75')



$e = 84 - 79.5 = \underline{4.5'}$

- Long Walls (Horizontal) will Not be Counted, Half + half ☺