Building A

138 East Beaver Ave

State College, PA

Report By: Robert Dawson

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Structural Notebook B

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October 19, 2018

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Dr. Hanagan,

The following report is an additional study of a typical bay 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 for the two bays that were studied. The table of contents shows the order of the calculations. At the end of the report is a summary comparison chart that will help determine if an alternate system warrants further study.

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

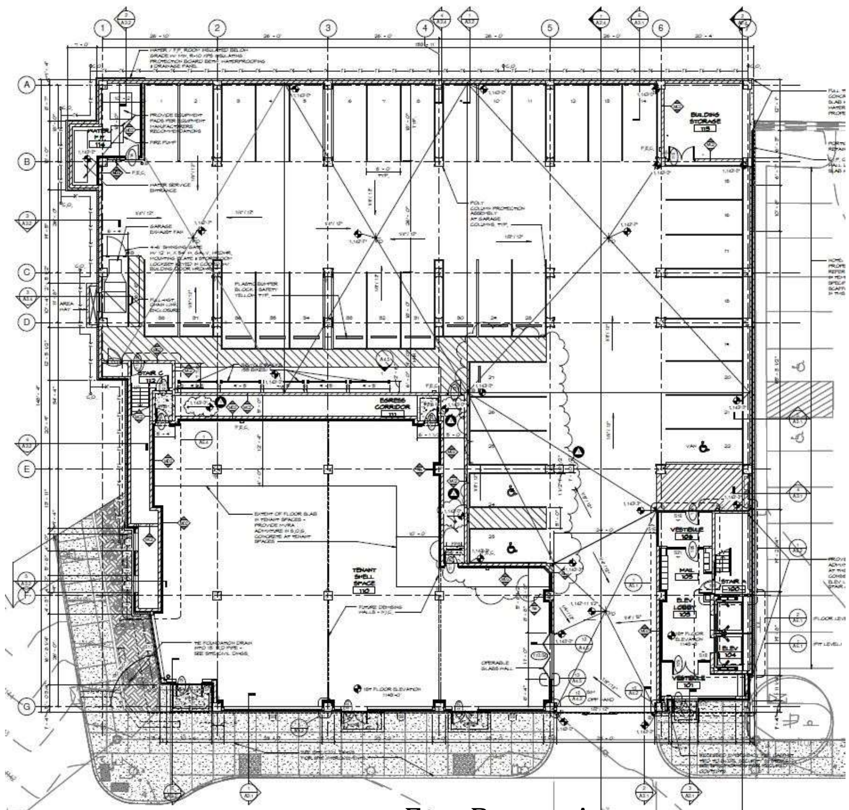
MEP

- PTAC AC units in each apartment
- Efficient Lighting

3.0 Site Plan

Figure 1

South Pugh Street



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
  - 8" Hollow Core Roof Plank – 100psf
  - Rigid Insulation – 1.5psf
  - Misc. (MEP, Ceiling) – 8psf

ASCE Load Combination **3** controls roof design.

1.2D + 1.6S + L = **226psf**

5.2 Floor Bay

Residential Loads:

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

ASCE Load Combination **2** controls residential floor design.

1.2D + 1.6L = **226psf**

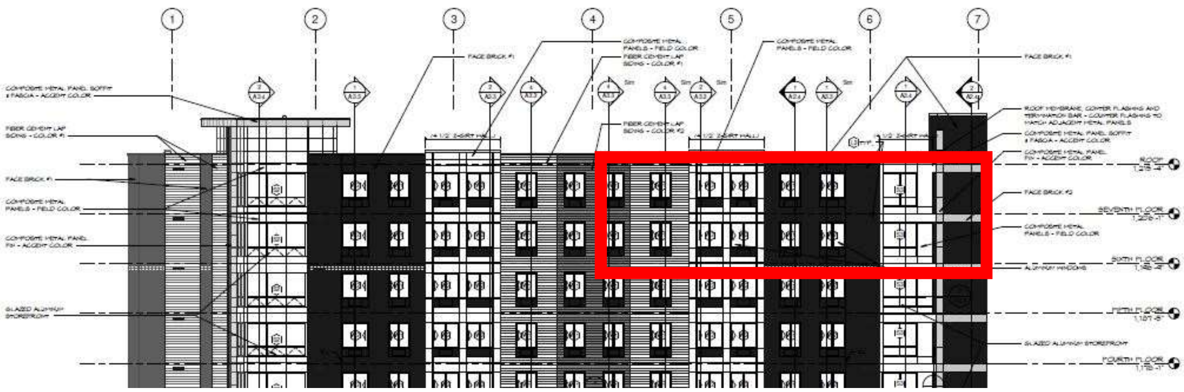


Figure 2

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

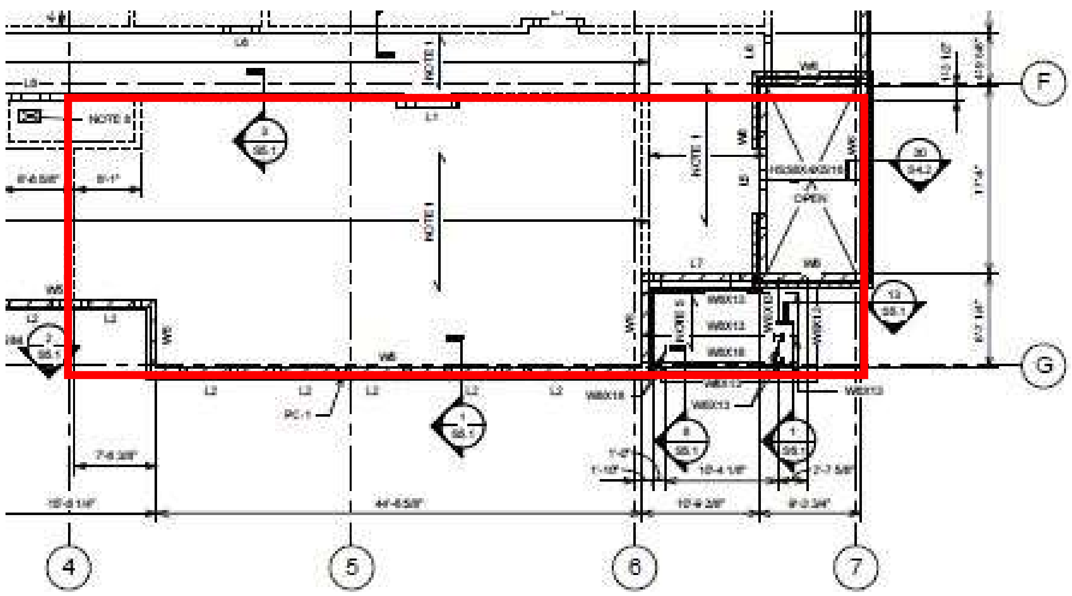


Figure 3

Cross Sections of Typical Floor and Roof Construction.

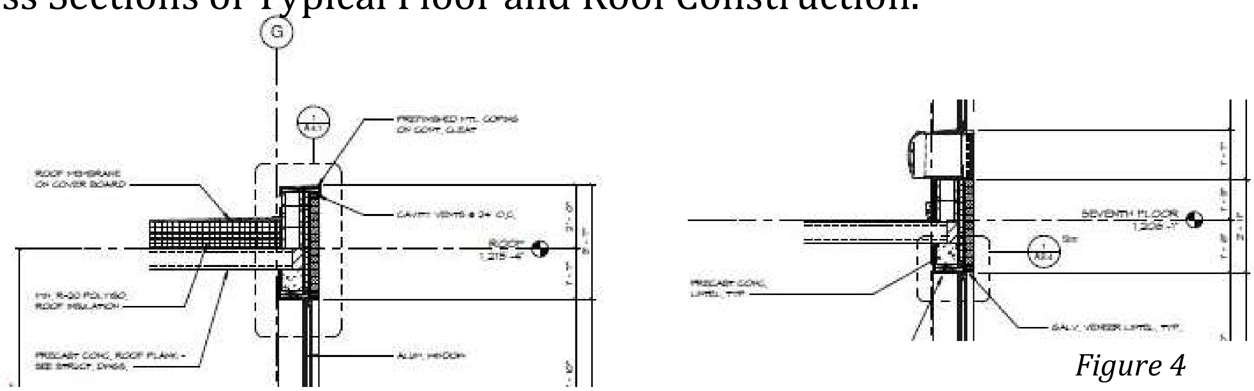


Figure 4

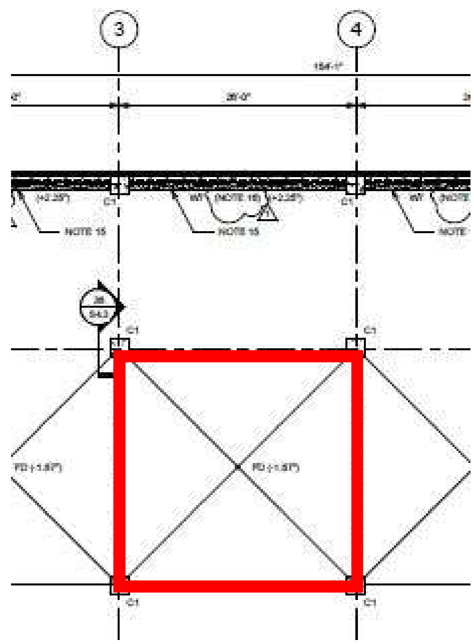
Parking Garage Loads:

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

ASCE Load Combination **2** controls parking garage design.

1.2D + 1.6L = **256psf + 4800lb**

Figure 5



Typical Garage Bay:

26' x 26'

5.3 Exterior Wall

Typical Wall Load:

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

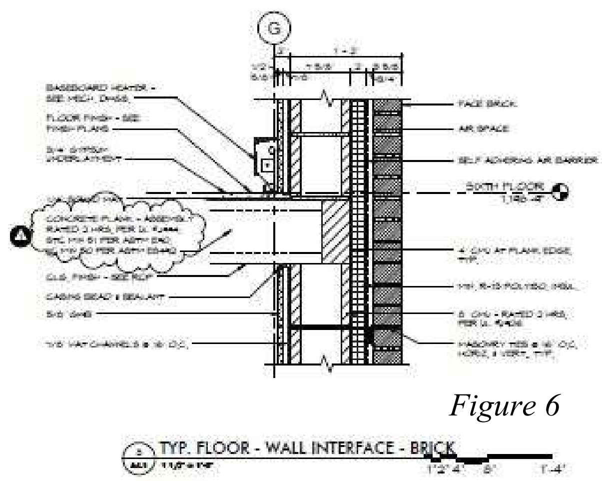


Figure 6

Exterior brick is not supported by the floor slab.

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 <u>Table 27.6-1</u>: 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 <u>E–W</u> Base Shear:

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

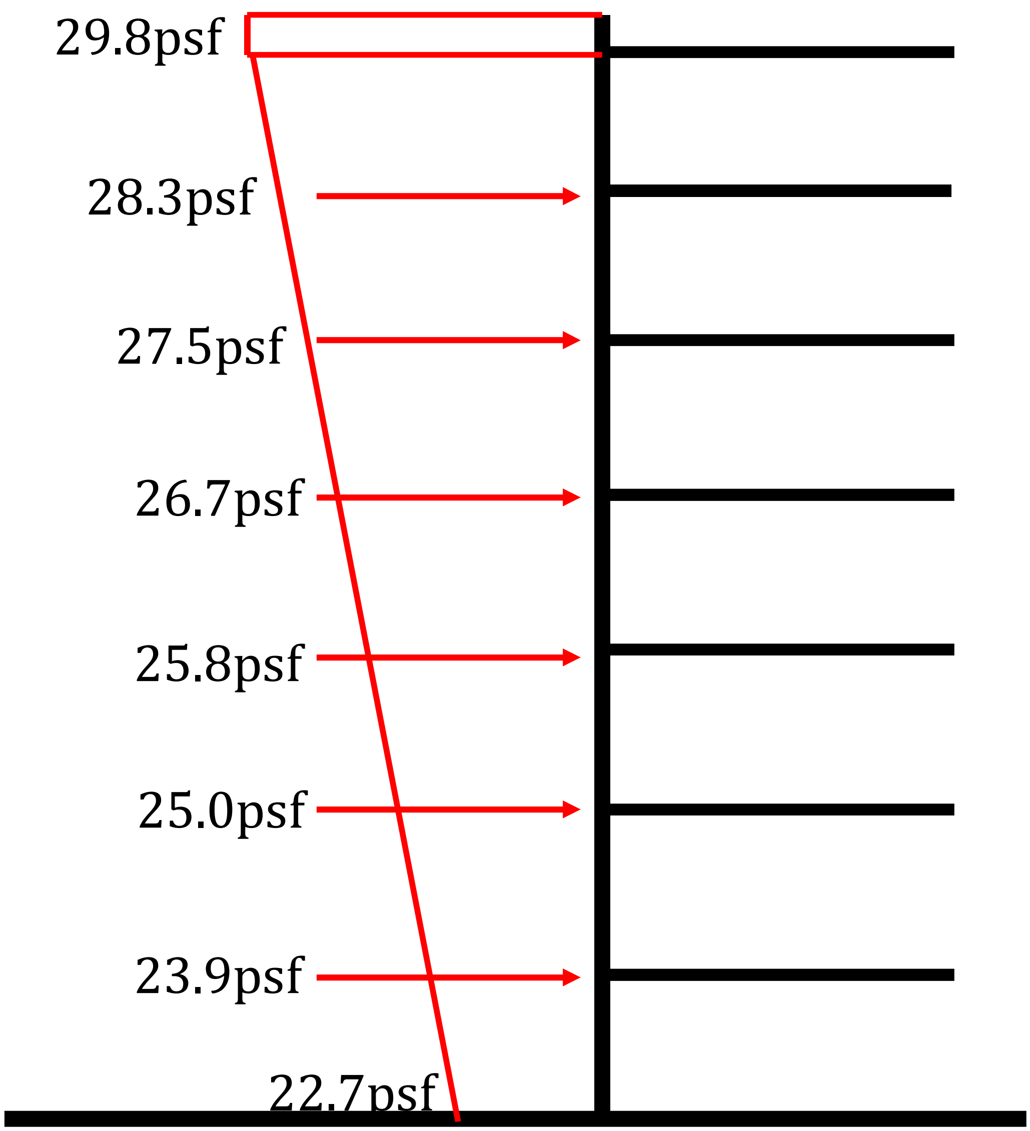
= <u>**306kip**</u>

Total N–S Base Shear:

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

= **326kip**

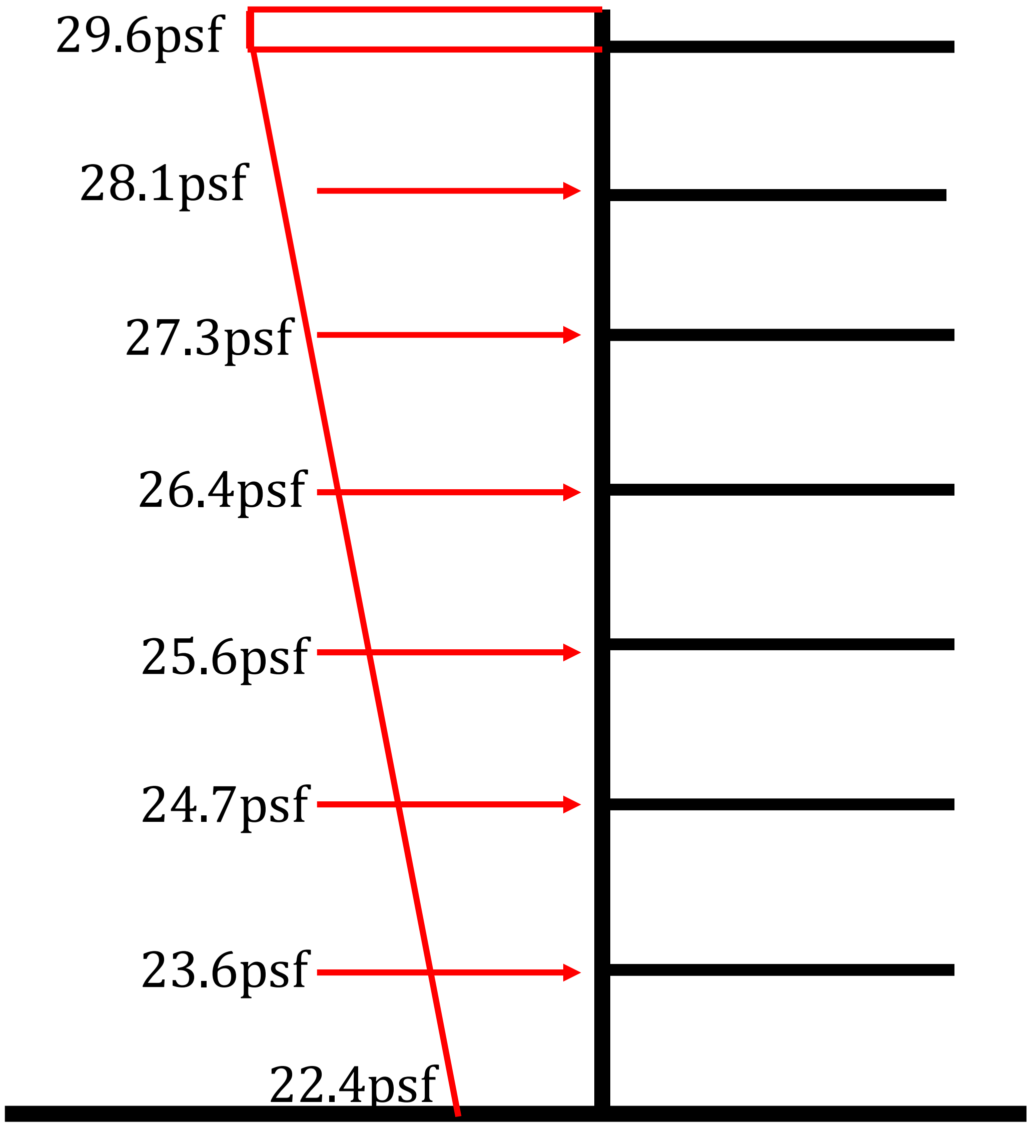
E-W Diagram



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 = **<u>0.7511</u>** $T < T_L$; 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:

Floors

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) = **220kip**

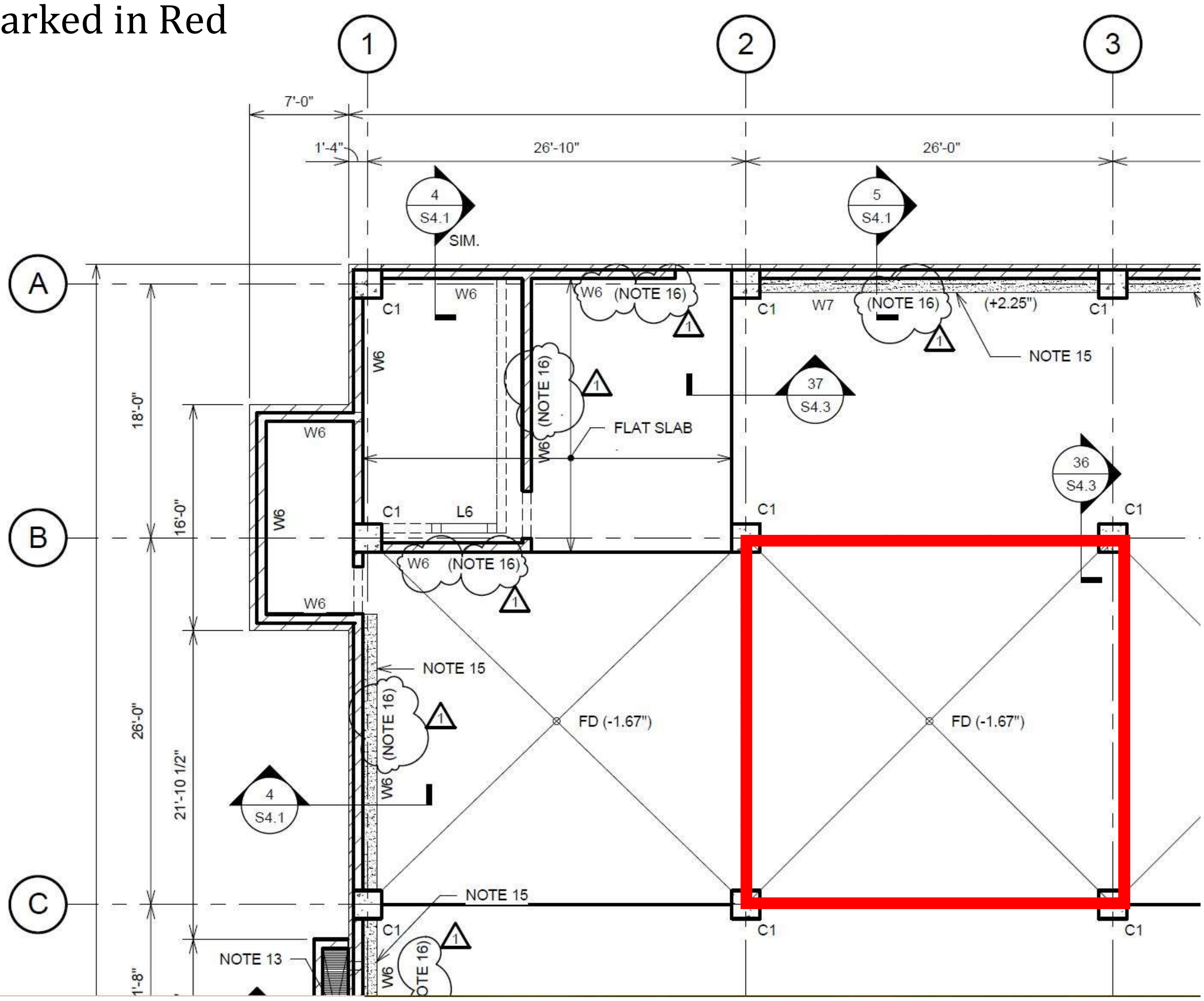
Typical Bay and Member Spot Checks:

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.

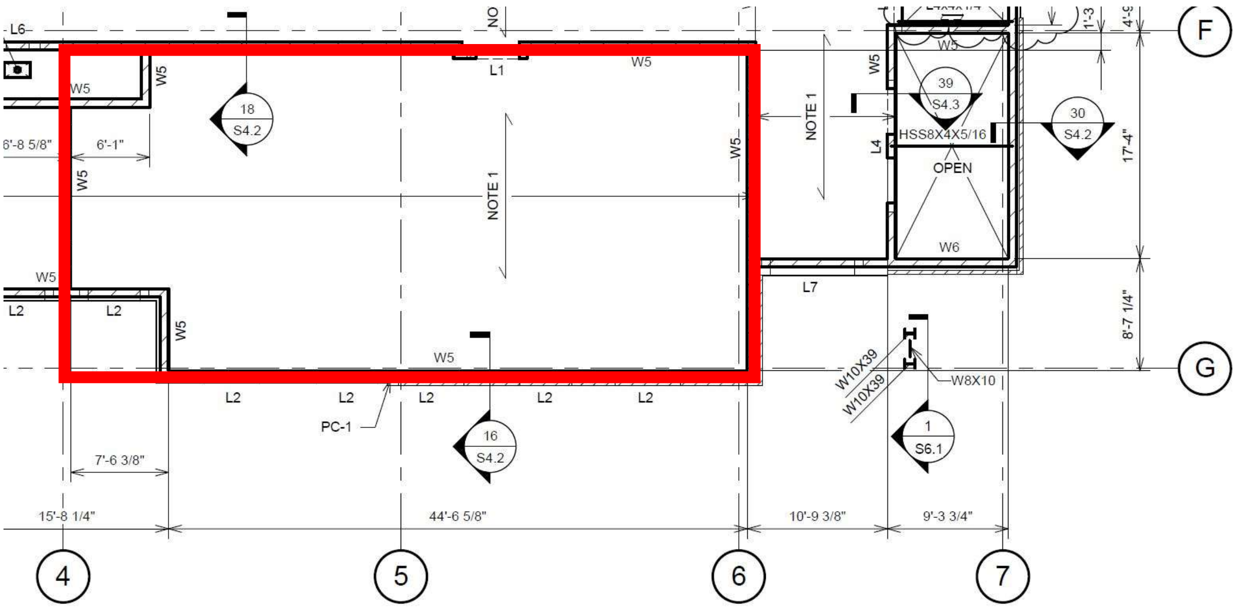
Garage Bay:

Marked in Red

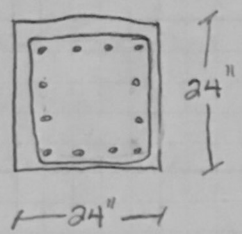


Residential Bay:

Marked in Red



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

Column Spot Checks (Exterior)

Column Type: C1

Along lines A & 3

Exterior Column

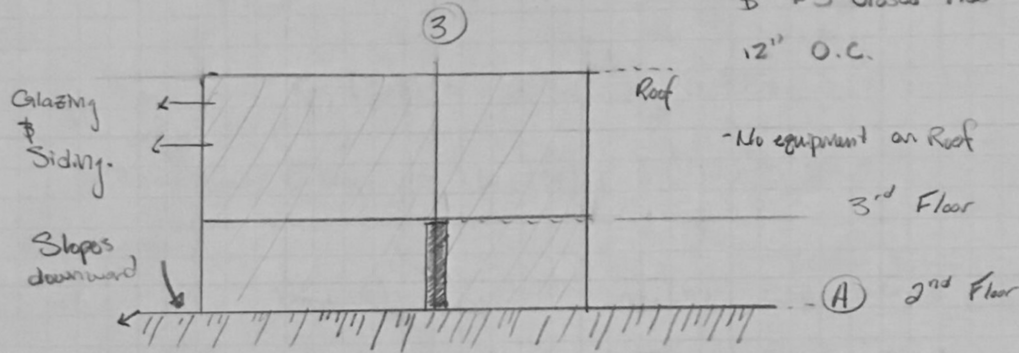
From Drawings: Column C1

24" × 24"

Reinforced (12) #7

& #3 Closed Ties

12" O.C.



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 = $\min \left| 0.4 \right. \quad 0.25 + \frac{15}{\sqrt{4(936)}} = 0.50 \leftarrow$

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^k	t = 28, Already accounted for 8"
Floors 3-7:	31.7^k	4.68	X	t = 20
Floor 2:	37.5	11.7	X	$\frac{20}{12}$ x 150 = 250
Transfer Slab:	58.5^k	X	X	250psf x 234 = 58.5
Total	283	30.42	9.4	

Combo: - 1.2D + 1.6L + 0.5S

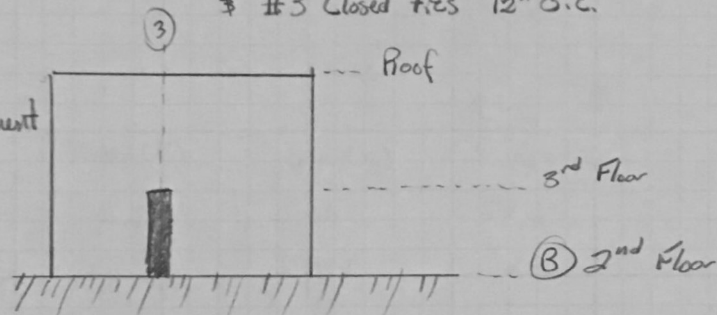
1.2(251) + 1.6(30.42) + 0.5(9.4) = <u>393^k</u>

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 = $\frac{20}{12}$ x 150 x 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

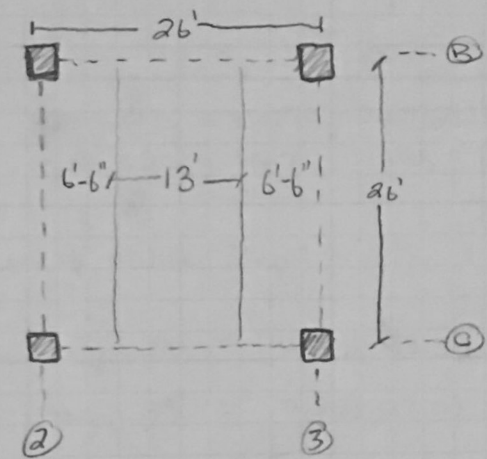
Combo:

1.2D + 1.6L + 0.5S

1.2(689.8) + 1.6(58.7) + 0.5(22.9) = 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 × 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

$M_o = q_u l_2 l_n^2 / 8 = \frac{0.256 psf (26)^2 (13)}{8} = 282^k$ 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) = 183.3 ft-k	0.35(282) = 98.7	0.65(282) = 183.3 ft-k

Moments in Column & Middle Strip:

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

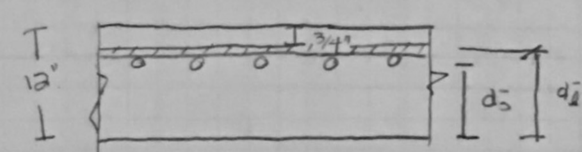
  Middle-Strip $M^- = 0.25(183.3) = 45.83$

- 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 ß	-22.9	39.48	-22.9
Column ß	-68.75	59.22	-68.75

Negative Moments

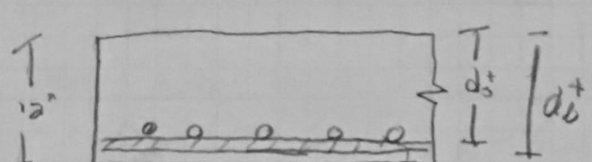


Typical:

Column: #6 @ 10"

Middle: #4 @ 12"

Positive Moments



Typical:

Column: #5 @ 10"

Middle: #4 @ 12"

Depth of Bars: - 3/4" Clear Cover

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

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

$d_{s}^{-} = 12 - \frac{3}{4} - 0.5 - \frac{1}{2}(0.750) = 10.375 in$

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

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
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

← Controls (As min)

Eqns.

$A_{s\,req} = \frac{M_u(12)}{0.9 f_y\, 0.95d}$

$A_{s\,min} = 0.0018bh$

$f_y = 60\ ksi$

$S_{max} = 2h$

$N = A_s / A_b$

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

$Spacing = b/N$

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:

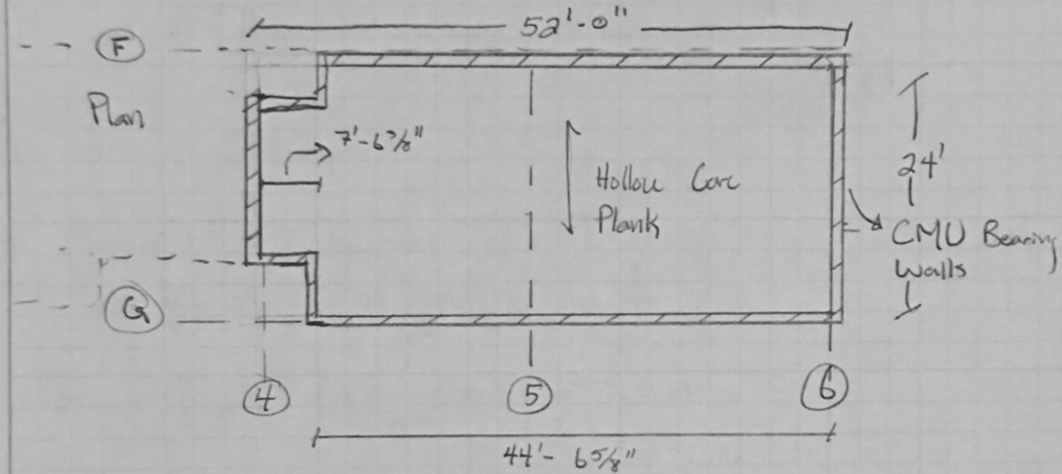
Interior Panel

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 ½ in diameter, 270^{ksi}, low relaxation strands

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

$d_p = 7$ in	$l_{pc} = 24'6''$
$f_{pi} = 0.7 f_{pu}$	$l = 25$
$f'_c = 5000$ psi	

Method 1: ACI Equation (18-1)

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

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

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

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

$f_{ps} = 261.89^{ksi}$

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

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

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

$\phi = 0.9$

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

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

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

→ 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 Kip-ft < 91.5 Kip-ft OK

Alternate Designs:

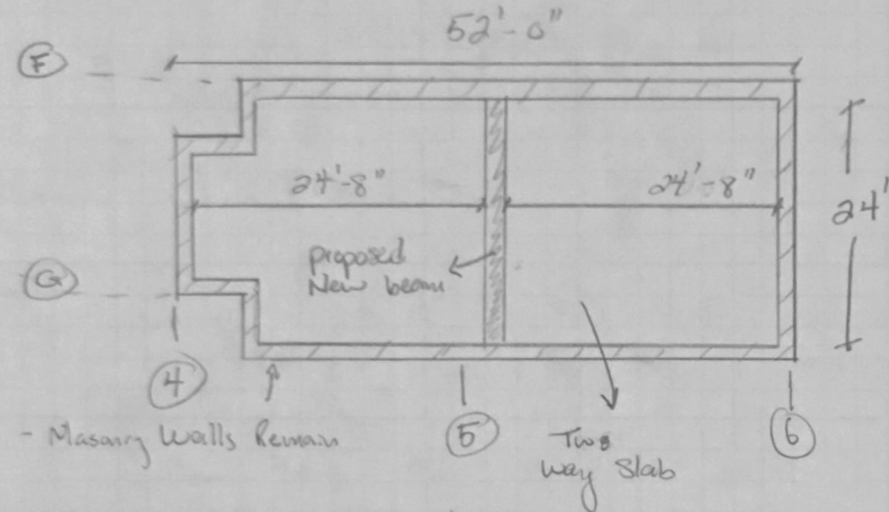
The first alternate design was tested in the residential bay to see how a two-way slab would work in place of the hollow core plank. A beam was place in the middle of the bay spanning across to the CMU bearing walls. The CMU walls surrounding the slab take the vertical weight of the floor system.

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.

$L/S < 2.0 = \frac{24}{24.8} = 0.97 < 2.0$ ✓

Direct design Method applies see Garage Bay Spot Check

Assume 8" thick Slab.

Dead Load = 100 psf $1.2(135) + 1.6(40) = 226$ psf (see garage)

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

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

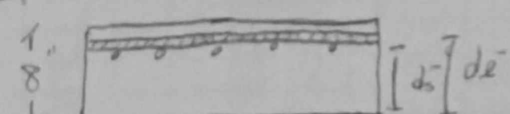
Middle Strip $M^+ = 0.4(235) = 94$'k

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

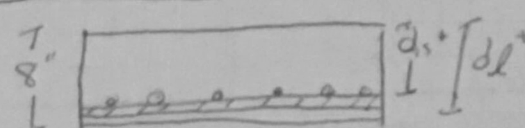
Middle Strip $M^- = 0$

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

← Controls. (for N_{min})

Use #6 bars

L & R
(10) #6 @ 24"

Mid
(16) #6 @ 20"

Column

	Left M^-	Middle M^+	Right M^-
M_u	32.96	141	168.15
C_s (b)	74"	74"	74"
d	6.875"	6.875"	6.875"
A_s req	1.12	4.8	3.68
A_s 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.

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

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

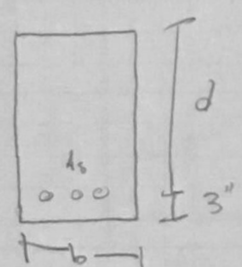
Load: See gravity loads & Two way design

826 psf

826 psf × (24' - 8") = 5.6 klf

Cross Section,

$b = 0.5d$



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

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

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

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

$= 0.88$

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

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

Assume (3) #9

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

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

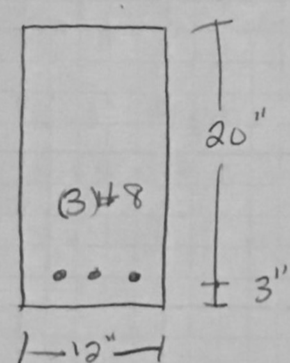
$$\varepsilon_s = \frac{(20 - 3.49)}{3.49}\,0.003 = 0.014 > 0.005 \quad \text{ok} \quad \phi = 0.9$$

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

$$0.9(2.37)(60)\left(20 - \frac{2.74}{2}\right) = 198\,'\text{k}$$

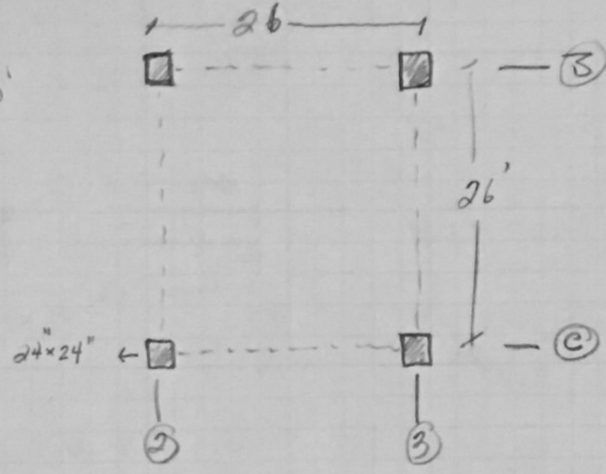
$$198\,'\text{k} > 168\,'\text{k} \quad \text{ok}$$

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 $\geq \frac{l_n}{34}$ $\quad \frac{26-1}{34} \times 12 = 8.82 \rightarrow$ Use 9"

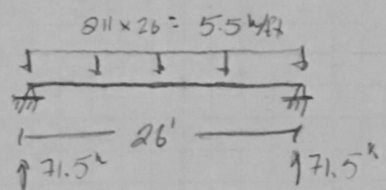
Dead load $\Rightarrow \frac{9}{12} \times 150 = 112.5$ psf

Combo:

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

Determine if Slab adequate for Shear.

One way action



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

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

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

$= 398^k$

$\phi V_n \geq V_u$ ✓

- Adequate in one way ∴ both directions are the same

Two way action:

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

$670.5 \times .211 = 142^k$

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

$\beta_1 = 24/24 = 1.0$

$\alpha_s = 40$

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

$0.75(336) = 252^k$

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

Provide drop panels:

Start with 3" drop 12" total

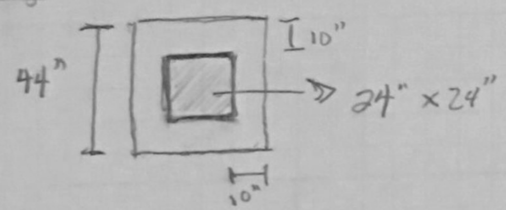
$$V_u = 211 \times \left[26 \times 26 - \left(\frac{x+9}{12} + \frac{x+9}{12} \right) \right] \qquad b_o = 4(x+9)$$

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

→ Set = & Solve for x = 40 in

Use 10" either side of column

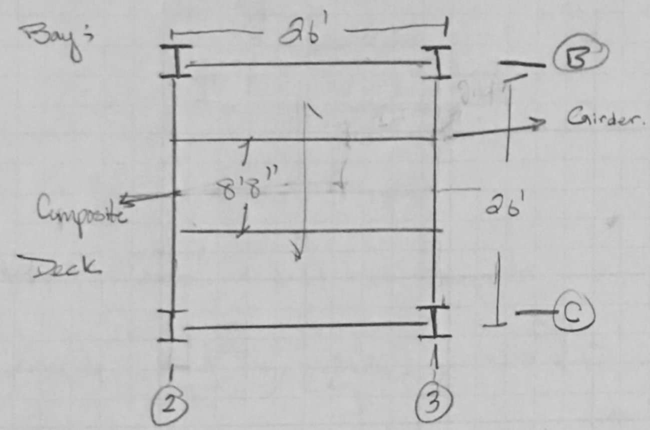
Design



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"

<u>Load</u> :

Live: 40psf

Dead: 5 for Steel allowance

10 see gravity (Misc.)

= 55 + SW

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

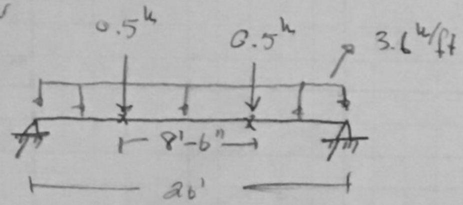
3 span Max 11'-8" > 8'-6"

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

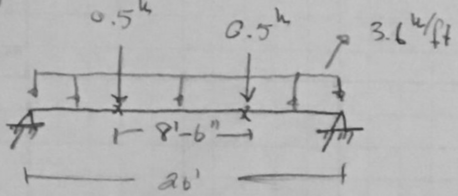
= 1.98 × No good

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

$$\phi M_n = \frac{309}{1.01} = 306 \text{ k-ft}$$ 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 = $\Big|_{0.4}\ 0.25 + \frac{15}{\sqrt{4(2288)}} = 0.41$

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:

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

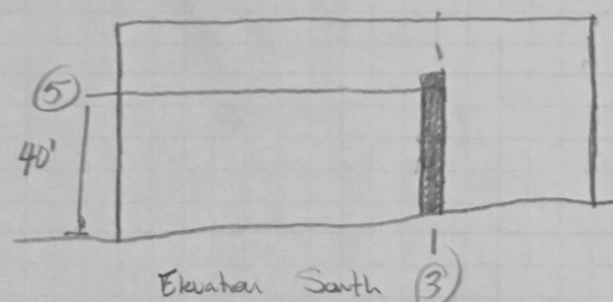
Column 40' high

Column is braced at every floor

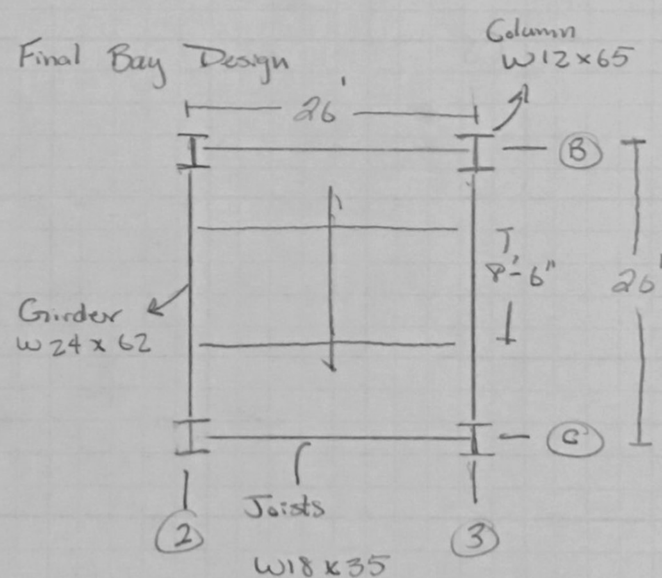
Longest l_e w/out bracing is between grade & floor 3

$16' = L_e$

W12 x 65



Final Bay Design



Deck

3VLI22

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.