Letter of Transmittal

Date: November 16, 2015

To: Dr. Aly Said
The Pennsylvania State University
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University Park, PA 16802
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From: Mark Bland
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Dear Dr. Said,

The enclosed documents include my Structural Notebook Submission C for AE481W – Senior Thesis. This report is a detailed structural analysis of West Village Housing's new North Building located on Towson University's campus.

This report includes a detailed lateral analysis of the building. The analysis includes results obtained from 2D/3D modeling software's. In addition, various spot checks were performed to check computer software outputs. Information regarding strength, drift, overturning and foundations were all investigated under the previously determined loading conditions.

Thank you for taking the time to read and review my report.

Sincerely,

Mark Bland

Lateral System Analysis Study

Structural Notebook Submission C

West Village Housing Phases III & IV

Towson, Maryland

Mark Bland [Structural Option]

Advisor: Dr. Aly Said

November 16, 2015

Executive Summary

West Village Housing Phases III & IV is located on Emerson Drive in Towson Maryland on Towson University's campus. The project consists of two residential halls which will contain approximately 325,000 gross square feet of apartment-style accommodations for upper level students. The 9 and 11 story residence halls will contain a mix of two and four bedroom apartments. Each with single occupancy rooms and shared bathrooms, kitchens and living areas. Green roofs, penthouses and a basement are also planned to be included. The two buildings have not been named yet.

Structurally, the buildings are composed of 8" thick two-way post-tensioned concrete flat plates supported by concrete columns. Bays are roughly 27' by 20' with slight variances as the buildings shape changes. They are reinforced with ½" diameter un-bonded tendons in each direction and mild reinforcing, as required. In addition to the floor composition, perimeter steel angles will be provided at each floor level to support the exterior brick veneer with metal frame back up. 12" thick concrete shear walls will effectively resist the forces imposed on the building from all lateral loads. It shall be assumed that all stair and elevator walls be concrete shear walls.

The buildings began construction simultaneously in September of 2014 to address the continued demand for on-campus housing and are planned to be finished in the summer of 2016. They were designed considering live loads, gravity loads, snow loads, wind loads, seismic loads, and lateral loads. The lateral force resisting system in the building is primarily made up of shear walls that are located around the two stair towers of the structure. The project uses the 2012 Edition of the International Building Code and ASCE 7-10. Design loads were determined based on these codes, additional Baltimore Maryland County Codes and Ordinances, as well as practical engineering judgments.

For purposes of clarity and organization, this report and those following will be based off of the design and construction of the North building shown in Figure 1. Financial figures are being withheld upon request of the owner.

The continuation of this report will cover all of the above elements of this project and more in greater detail.

Table of Contents

1 General Information

1.1 Purpose and Scope

The objective of this report is to perform a detailed analysis of Townson University's, West Village Housing Phases III & IV North building by performing typical member spot checks for gravity loads that were previously determined in Notebook Submission A.

This report will include an overview of the site location and plan. It will discuss and present calculations for the current structural system as well as three alternative framing systems. A list of relevant resource documents used in design are also presented.

The knowledge documented in this report will be used as reference in future technical reports.

1.2 Site Location and Plan

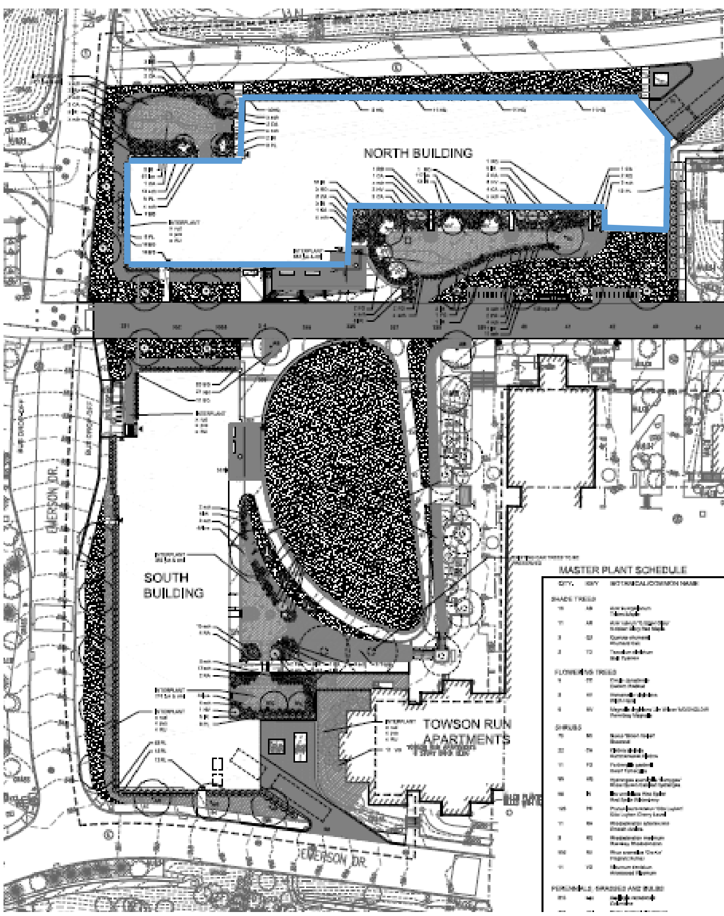


Figure 1: Site Landscaping

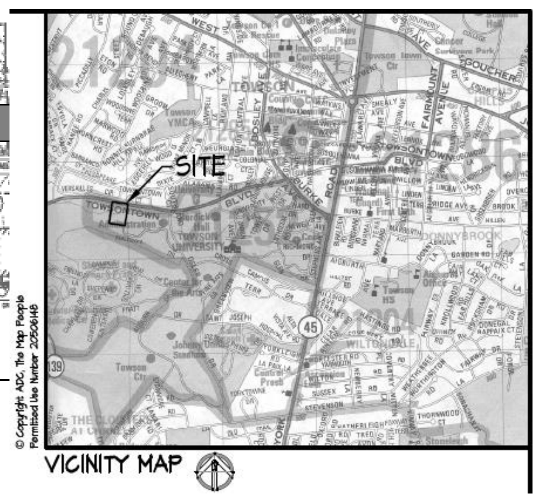


Figure 2: Vicinity Map

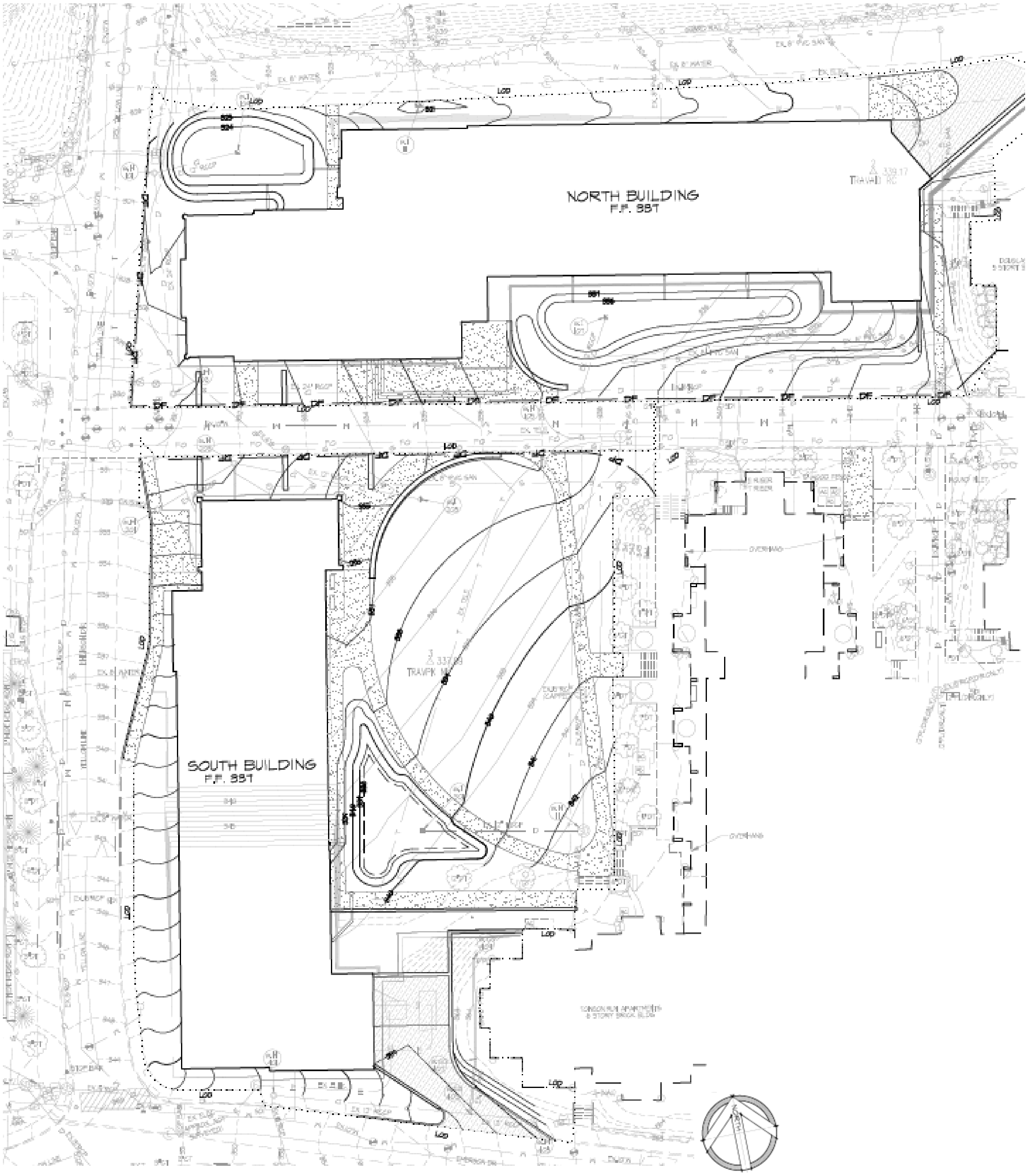


Figure 1: General Grading Plan

1.3 Documents Used During Preparation of Report

The following is a list of the design codes, standards or other references used on the project. These codes will be used to structurally analyze the loads on the structure of the West Village Housing Phases III & IV North building.

<u>International Code Council</u>

- International Building Code, 2012 Edition
- International Building Code, 2006 Edition
  - Used for drift and sliding snow loads only

<u>American Society of Civil Engineers</u>

- ASCE 7-10: Minimum Design Loads for Building and Other Structures

<u>West Village Housing Phases III & IV</u>

- Construction Drawings
- Specifications and details
- Correspondence with Project Engineers

<u>Previous Course and Internship Notes/Resources</u>

2 Gravity Loads

The gravity load calculations include dead, live and snow loads. The calculated loads will be compared to the actual loads used in the design of the building.

2.1 Dead and Live Loads

Figures 2 and 3 are sections taken from the architectural drawings on this project. Both are used to determine the composition of the component in order to calculate the dead and live loads for the typical roof construction, floor construction and exterior wall construction. A typical floor cross section is provided in the calculations due to its absence in the drawings.

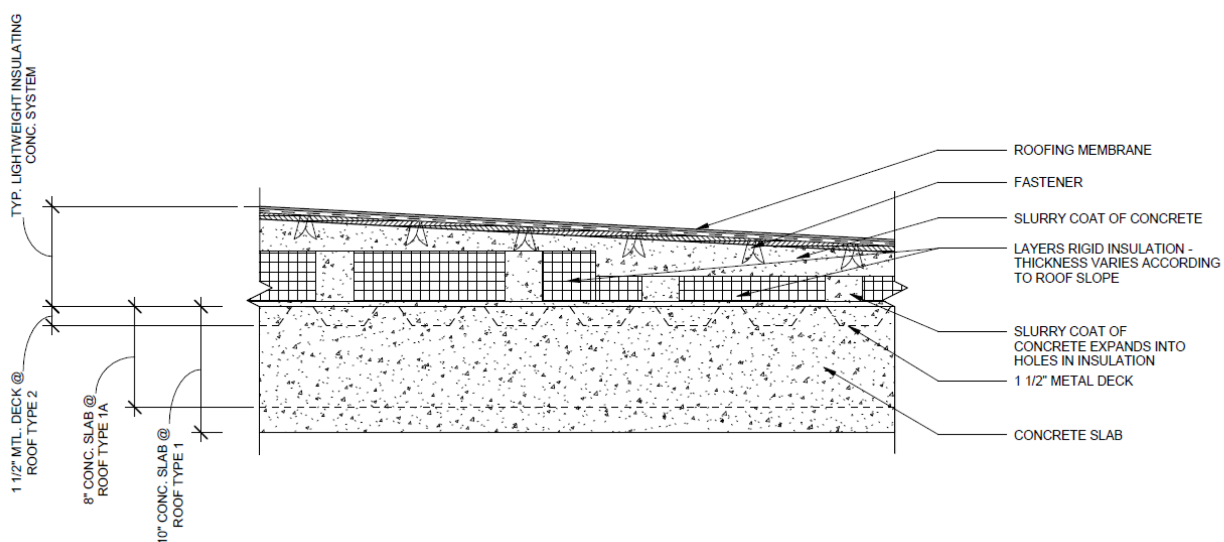


Figure 2: Typical roof bay cross section

Typical Roof Bay

Dead Load:

- Roofing membrane	=	3 PSF	
- Rigid Insulation	=	1.5 psf (4") = 6 psf	→ thickness
- 8" Concrete slab	=	12.5 PSF (8") = 100 psf	varies w/ slope
- 1½" metal deck	=	2 PSF	
- MEP	=	3 PSF	
- Ceilings (lighting/electrical)	=	5 PSF	
- Collatoral	=	6 PSF	

Total = 125 PSF

This is equivalent to loads specified on S1.00

Roof Live:

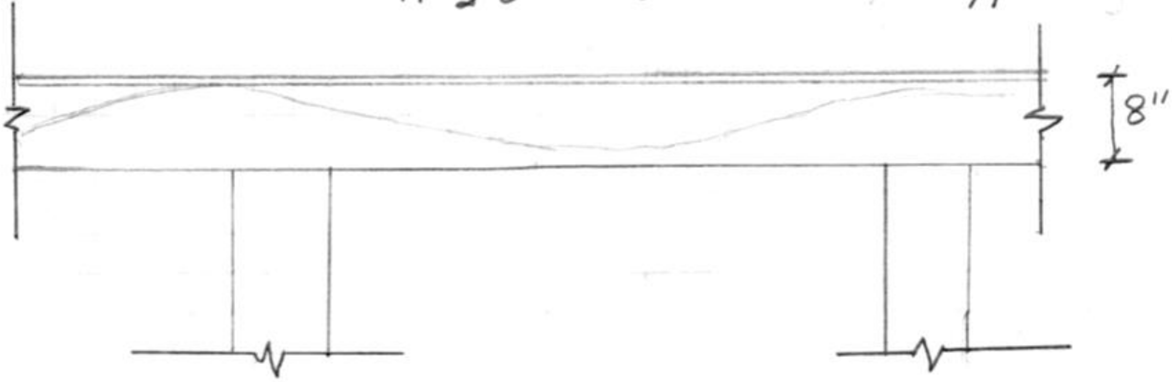
- 20 PSF per ASCE 7-10 table 4-1

* The Engineer chose to increase the minimum live load, provided in ASCE 7-10, to 30 psf. This may have been to a number of possible factors such as: roof maintenance, foot traffic

* Another portion of the roof does not have metal decking but has 10" of concrete slab. This dead load would be 148 PSF

Typical Floor Bay

- Floor consists of 8" thick post-tensioned concrete slab reinforced with ½" diameter tendons & typical reinf.



Dead load:

- 8" concrete = 125 psf (8") = 100 psf
- Floor finish = 2 psf
- MEP = 6 psf

Total = 108 psf

Live load:

- Private rooms and corridors serving them = 40 psf per ASCE 7-10 table 4-1
- Partitions = 15 psf

Total = 55 psf

* Note that live load for corridors and public spaces is equal to 100 psf

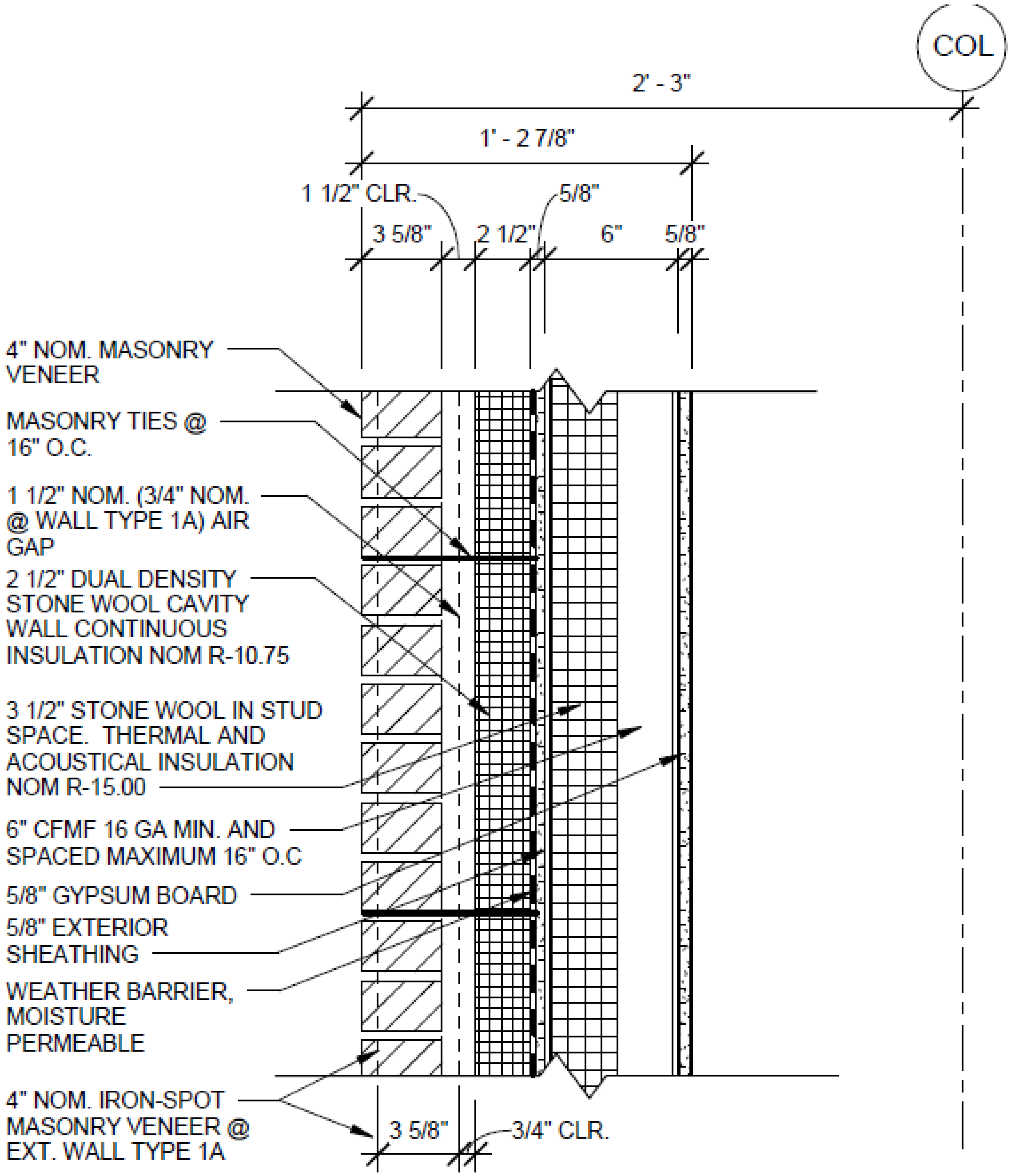


Figure 3: Typical Exterior Wall Detail

Typical Exterior Wall

Dead load:

- 4" veneer brick = 38 psf
- 2½" insulation = 3 psf
- 5/8" sheating = 3 psf
- 6" metal studs = 3 psf
- 5/8" gypsum board = 3 psf
- collatoral = 2 psf

Total = 52 psf

Steel relieving angles are provided at each floor level to support exterior brick veneer. The weight of the typical exterior brick wall is taken by the angle, and transferred into the concrete floor system. This load is tranferred to the columns and then down to the foundation

Non Typical Dead Loads

- Store front curtain wall system (level 1) = 10 psf
- Exterior wall with metal paneling (not as large as brick) = 25 psf
- Penthouse floor = 150 psf
  Design values based on equipment weight ranging from 11,000 lbs – 19,000 lbs

Non Typical Live Loads

- Penthouse Floor = 100 psf
  Due to heavier mechanical roof traffic

Snow load Calculations

Flat roof snow load: using ASCE 7-10

$P_f = 0.7 C_e C_t I P_g$ (eq 7.3-1)

$P_g = 25\,psf$ (Figure 7-1)
$C_e = 0.9$ (Table 7-2)
$C_t = 1.0$ (Table 7-3)
$I = 1.0$ (Table 1.5-2) category II

$P_f = 0.7(0.9)(25) = \underline{15.75\,psf}$

Snow Drift

-calculated for drift from mechanical penthouse roof

$h_b = \frac{P_s}{\gamma}$ $P_s = P_f = 15.75\,psf$

$h_b = \frac{P_s}{0.13P_g + 14} = \frac{15.75}{0.13(25)+14} = 0.91$

$h_c = 11' - 1' = 10'$ $h_c/h_b > 0.2 \rightarrow$ drift load must be calculated

$l_u = 28.3'$

Leeward:

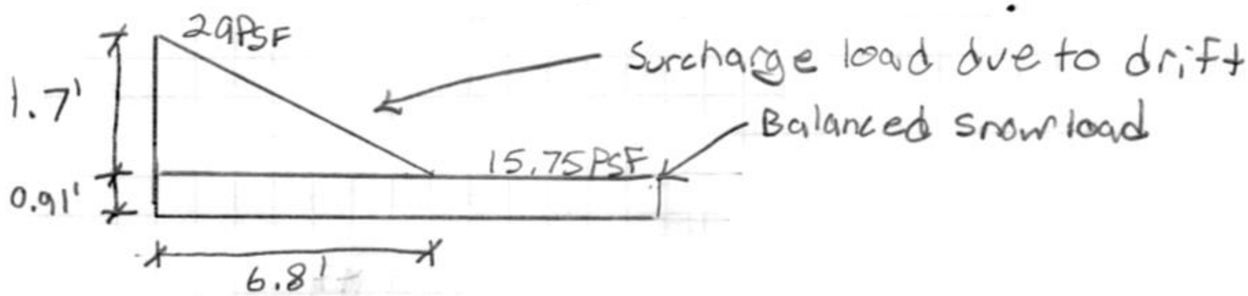
$h_d = 1.7$ (Figure 7-9)

$h_d < h_c, \therefore w = 4h_d = 6.8\,ft$

Windward:

$h_d = \frac{3}{4}(1.7) = 1.3\,ft < 1.7\,ft \therefore$ use $h_d = 1.7\,ft$

$P_d = h_d \gamma$
$= 1.7(0.13(25)+14) = 29\,psf$



3 Wind Loads

This section provides an overview of the wind loads considered for design. Tables 1 and 2 show the total windward and leeward pressures experienced on the building in both the E-W and N-S directions. Following these tables are hand calculations showing the wind load procedure found in ASCE 7-10 section 26. The calculations evaluate the North building residence hall. This includes 8 residence levels with a smaller penthouse at the lower roof level.

3.1 Calculations

Table 1: Wind Force Determination E-W

Building Level	Height above ground level z (ft)	K_z	q_z	$p_{z(W)}$	$p_{h(L)}$	Total (psf)
Level 1	0.0	0.575	16.54	10.59	-11.11	21.70
Level 2	12.0	0.575	16.54	10.59	-11.11	21.70
Level 3	21.3	0.635	18.28	11.70	-11.11	22.81
Level 4	30.7	0.705	20.29	12.99	-11.11	24.10
Level 5	40.0	0.761	21.89	14.01	-11.11	25.12
Level 6	49.3	0.807	23.24	14.87	-11.11	25.98
Level 7	58.7	0.849	24.42	15.63	-11.11	26.74
Level 8	68.0	0.885	25.47	16.31	-11.11	27.42
Lower Roof	78.8	0.923	26.57	17.01	-11.11	28.12
PH Roof	86.7	0.949	27.30	17.48	-11.11	28.59

Table 2: Wind Force Determination N-S

Building Level	Height above ground level z (ft)	K_z	q_z	$p_{z(W)}$	$p_{h(L)}$	Total (psf)
Level 1	0.0	0.575	16.54	11.25	-4.72	15.97
Level 2	12.0	0.575	16.54	11.25	-4.72	15.97
Level 3	21.3	0.635	18.28	12.43	-4.72	17.15
Level 4	30.7	0.705	20.29	13.80	-4.72	18.52
Level 5	40.0	0.761	21.89	14.88	-4.72	19.60
Level 6	49.3	0.807	23.24	15.80	-4.72	20.52
Level 7	58.7	0.849	24.42	16.61	-4.72	21.33
Level 8	68.0	0.885	25.47	17.32	-4.72	22.04
Lower Roof	78.8	0.923	26.57	18.07	-4.72	22.79
PH Roof	86.7	0.949	27.30	18.56	-4.72	23.28

Wind Load Calculations

1) Risk Category: (table 1.5-1)
Category II

2) Basic wind speed, V: (Figure 26.5-1A)
V = 115mph

3) Wind load parameters:
- Wind directionality Factor, K_d (table 26.6-1)
  $K_d = 0.85$
- Exposure Category (section 26.7)
  Exposure Category B
  * used by engineer
- topographic Factor (table 26.8-1)
  $K_{zt} = 1.0$
  * building is not located on a ridge, escarpment or hill
- Gust effect Factor (section 26.9)
  $T_a = \frac{0.0019}{\sqrt{C_w}} h_n$ (12.8-9) > 1
  ∴ use G = 0.85 enclosed building

Internal Pressure Coefficients
$GC_{pi} = +/- 0.18$

4) Velocity pressure exposure coefficient, K_z:
- K_z varies w/ height (table 27.3-1)
  * see spreadsheet

5) Velocity pressure, q_z:
$q_z = 0.00256 K_z K_{zt} K_d V^2$

K_z = varies w/ height

$K_{zt} = 1.0$

$K_d = 0.85$

V = 115mph

$q_z = 0.00256 K_z (1.0)(0.85)(115)^2 = 28.78 K_z$

* see spreadsheet for values of q_z

6) External pressure Coefficient, C_p:

North-South: $\frac{L}{B} = \frac{300'}{61'} = 4.9$

East-West: $\frac{L}{B} = \frac{61'}{300'} = 0.69$

Walls: - windward $C_p = 0.8$
- Leeward $C_p = -0.2$ in N/S, $C_p = -0.5$ in E/W
- side wall $C_p = -0.7$

Roofs: - Flat roof, $\theta = 0$

$h/L < 0.5$ — horizontal distance from windward edge $= 300' > 2h$

$C_p = -0.3, -0.18$

7) Wind pressure

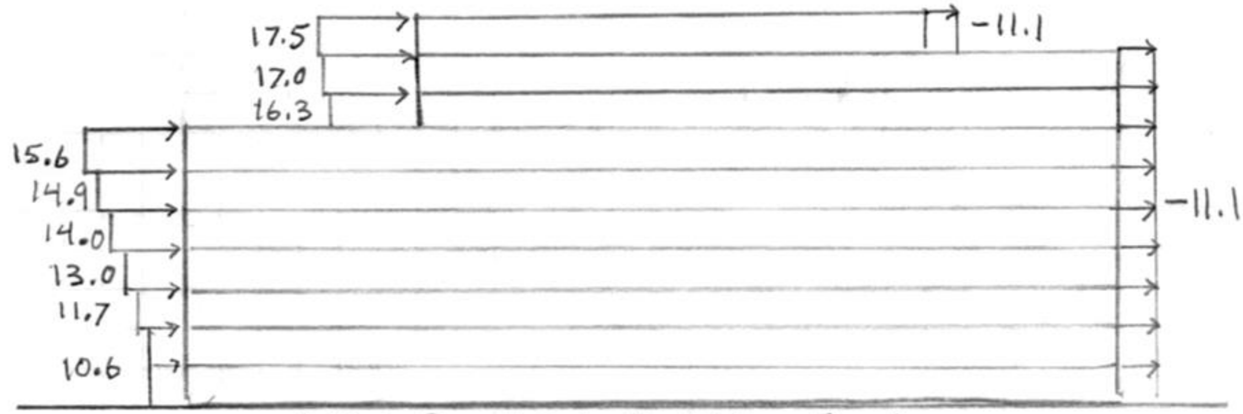
$p = q G C_p - q_i (G C_{pi})$ (27.4-1)

* see spreadsheet for p values

Table 3: Base Shear Determination E-W				
Building Level	**Height above ground level z (ft)**	**Tributary Height (ft)**	**Total Pressure (psf)**	**Total Lateral Story Force (kip)**
Level 1	0.0	6.00	21.70	**39.06**
Level 2	12.0	10.65	21.70	**69.32**
Level 3	21.3	9.34	22.81	**63.89**
Level 4	30.7	9.35	24.10	**67.59**
Level 5	40.0	9.32	25.12	**70.20**
Level 6	49.3	9.34	25.98	**72.77**
Level 7	58.7	9.35	26.74	**75.01**
Level 8	68.0	10.08	27.42	**82.90**
Lower Roof	78.8	9.34	28.12	**78.75**
PH Roof	86.7	3.92	28.59	**33.62**
Total Base Shear (kips) =				**653.11**

Table 4: Base Shear Determination N-S				
Building Level	**Height above ground level z (ft)**	**Tributary Height (ft)**	**Total Pressure (psf)**	**Total Lateral Story Force (kip)**
Level 1	0.0	6.00	15.97	**5.84**
Level 2	12.0	10.65	15.97	**10.37**
Level 3	21.3	9.34	17.15	**9.77**
Level 4	30.7	9.35	18.52	**10.56**
Level 5	40.0	9.32	19.60	**11.14**
Level 6	49.3	9.34	20.52	**11.69**
Level 7	58.7	9.35	21.33	**12.16**
Level 8	68.0	10.08	22.04	**13.55**
Lower Roof	78.8	9.34	22.79	**12.98**
PH Roof	86.7	3.92	23.28	**5.57**
Total Base Shear (kips) =				**103.63**

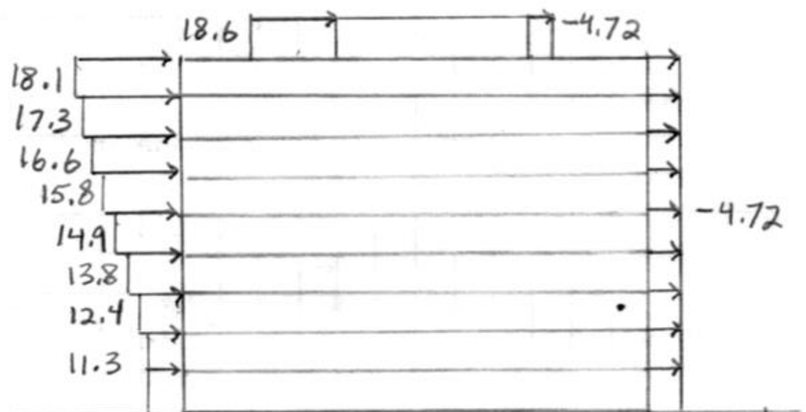
Wind Diagrams (values in psf)



Wind Parallel to Building E-W

←

Base shear = 653.11 Kips



Wind Perpendicular to Building N-S

←

Base shear = 103.63 Kips

4 Seismic Loads

This section provides an overview of the seismic loads considered for design. Below is the standard procedure to calculate seismic loads which is in accordance with ASCE 7-10 section 11. The main lateral force resisting elements in the structure are ordinary reinforced concrete shear walls

4.1 Calculations

Seismic Load Calculations

1) Find mapped Acceleration Parameters:

$S_s = 0.175 \quad S_1 = 0.051$

2) Site Classification:

Site Classification C (verifed by geotechnical report)

3) Max Considered spectral Response Acceleration Parameters:

$Fa = 1.2 \qquad Fv = 1.7$

$S_{ms} = Fa S_s = 1.2(0.175) = 0.21$

$S_{m1} = Fv S_1 = 1.7(0.051) = 0.087$

4) Design spectral Parameters (11.4.4)

$S_{DS} = \frac{2}{3} S_{ms} = 0.14$

$S_{D1} = \frac{2}{3} S_{m1} = 0.058$

5) Importance Factor: $I_e = 1.0$ (1.5-2)

6) Risk Category: II (section 11.6)

7) Seismic Design Category: A (table 11.6-1)

Basic Seismic Force Resisting System
- ordinary Reinforced concrete shear walls

$R = 5$

$\Omega = 2\frac{1}{2}$ (table 12.2-1)

$C_d = 4\frac{1}{2}$

8) Analysis Procedure selection:

(Section 11.7) - Buildings with Seismic Design Category A are exempt from seismic Design Criteria and must only comply with section 1.4.

9) Lateral Forces:

$F_x = 0.01\, W_x$ (section 1.4.3)

W_x = total dead load per story

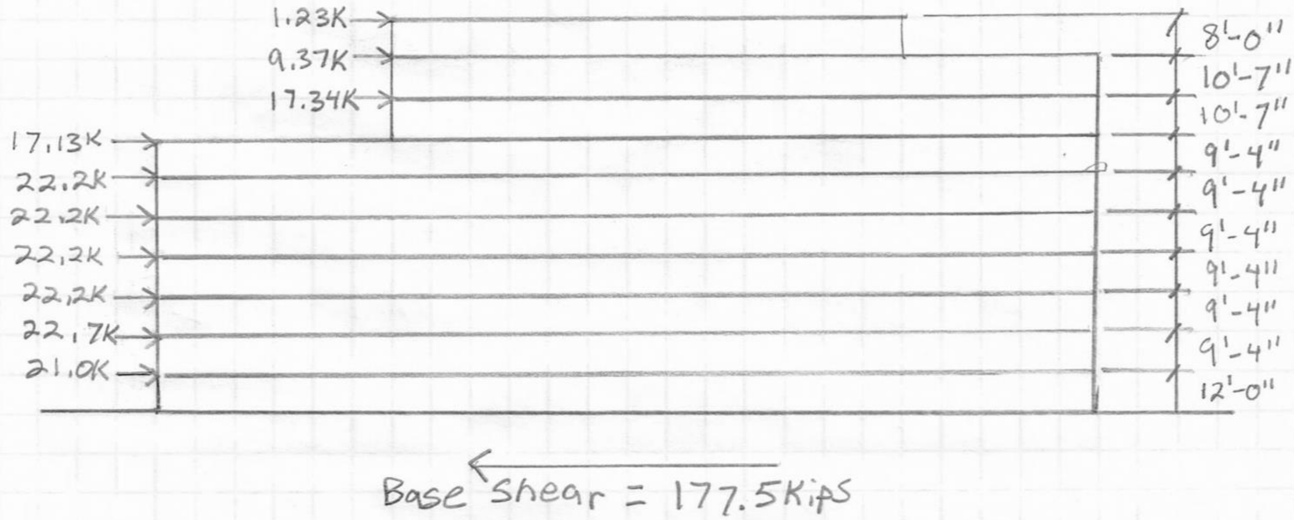
* see spreadsheet for floor weights and story forces.

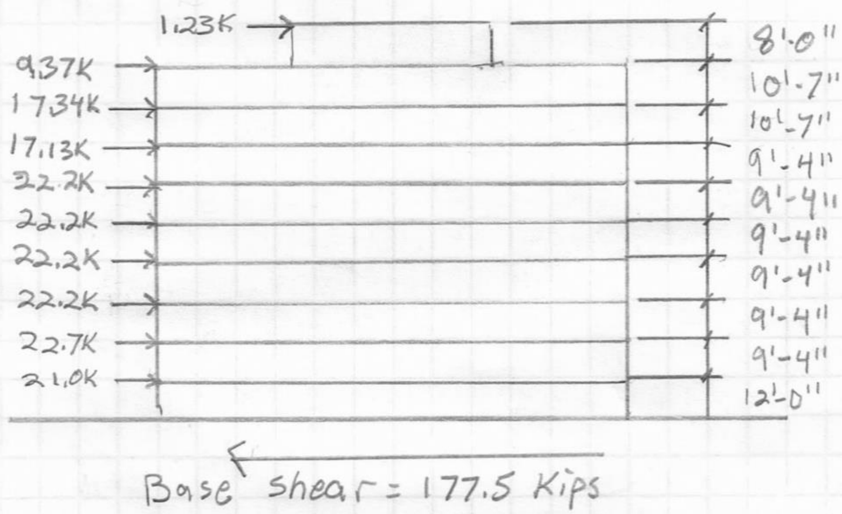
Table 5: Seismic Load Determination									
Building Level	Height above ground level z (ft)	Tributary Height (ft)	Total Floor Dead Load (psf)	Total Floor Area (s.f.)	Total Exterior Wall Load (psf)	Permiter (ft)	Total Story Weight W (kip)		
Level 1	0.0	6.00	108	17400	52	700	2097.6	0.01	**20.98**
Level 2	12.0	10.65	108	17400	52	700	2266.9	0.01	**22.67**
Level 3	21.3	9.34	108	17400	52	700	2219.0	0.01	**22.19**
Level 4	30.7	9.35	108	17400	52	700	2219.5	0.01	**22.20**
Level 5	40.0	9.32	108	17400	52	700	2218.3	0.01	**22.18**
Level 6	49.3	9.34	108	17400	52	700	2219.0	0.01	**22.19**
Level 7	58.7	9.35	108	13320	52	564	1712.8	0.01	**17.13**
Level 8	68.0	10.08	108	13320	52	564	1734.2	0.01	**17.34**
Lower Roof	78.8	9.34	150	4928	52	408	937.3	0.01	**9.37**
PH Roof	86.7	3.92	25	4928	0	408	123.2	0.01	**1.23**
Total Base Shear (kips) =									**177.48**

*Exterior wall types vary throughout the building. In order to be conservative, the heaviest wall type (brick) will be used on all floors

Mark Bland | Seismic loads | Notebook Submission A

Seismic Load vs Story Height

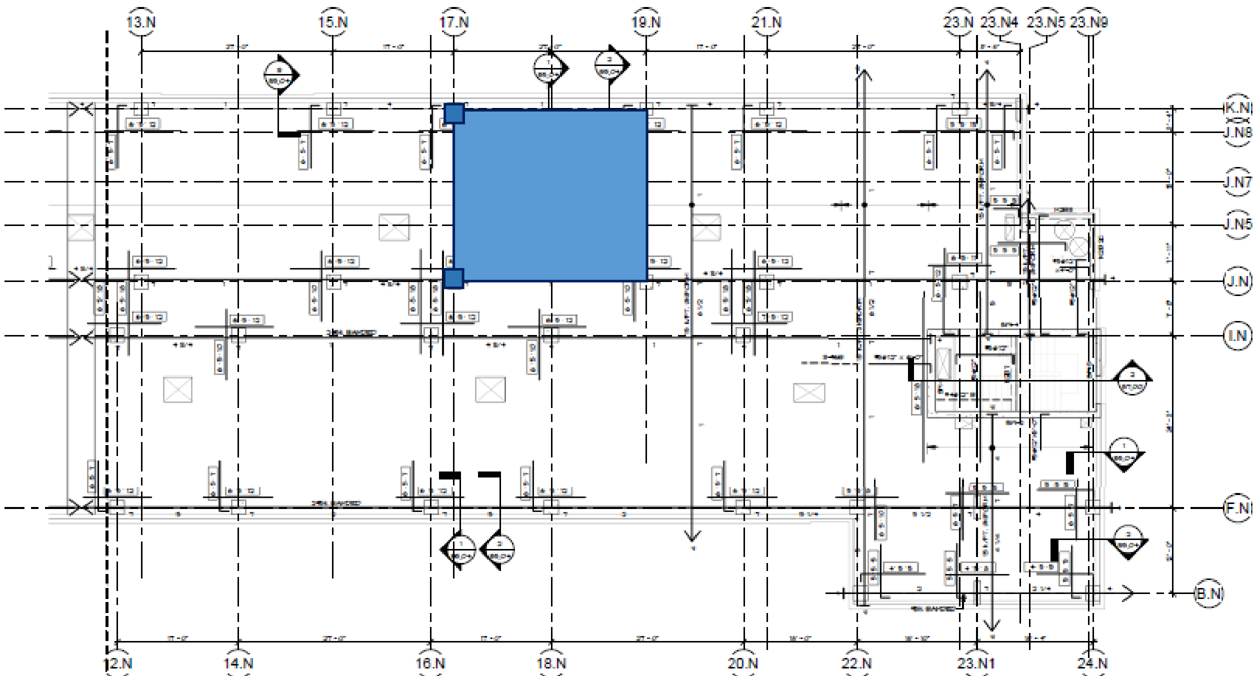




* With simplified Method for buildings in a seismic design category A, the seismic story forces are the same in both directions.

5 Existing System Analysis

Typical Bay and Columns Analyzed



The prestressing (post-tensioning) in this slab introduces compression into what would be the tension zone of the slab if it was not prestressed. To analyze this system the load balancing concept was used. This procedure involves removing the tendons and replacing them with equivalent loads composed of horizontal and vertical forces, moments at the external supports and transverse forces along the tendon profile. Counter-active forces in the tendons balance out a portion of the dead load. The following analysis uses this method to determine the upward force exerted on the slab.

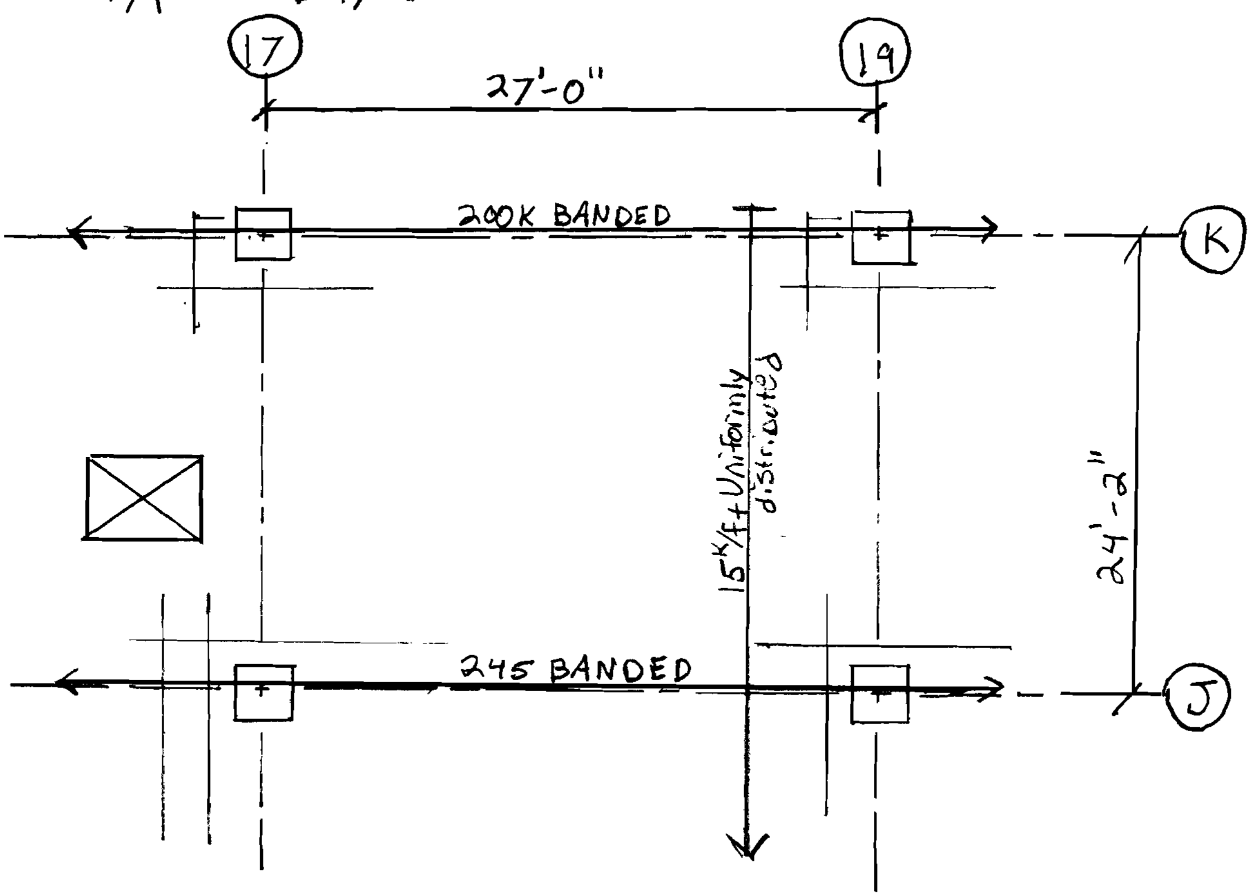
Bay Under Analysis

- The typical bay is bounded by grid lines 17, 19, K and J
- Bay size: 27'-0" x 24'-2"
- * Analysis is for typical floor, which occurs on floors 3-6
- Post-tensioned two-way flat slab construction

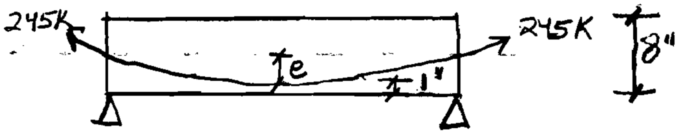
Assumptions

- Assume columns are centered on column grids
- Assume the presence of openings in the slab is negligible
- when checking shear just account for floor loading because the drape of the post-tensioning isn't balancing load as much near the columns.

Typical Bay Detail



Post Tensioning Balancing load



Upward Force/Moment from tendons:

$$w_p = \frac{8Fe}{L^2}$$

$e = 4'' - 1'' = 3''$

* accounts for 1" tendon cover noted on plans

$$w_p = \frac{8(245k)(3'')\left(\frac{1}{12''}\right)}{(27')^2} = 0.675\ K/ft$$

$$w_p = \frac{8(15\ K/ft)(3'')\left(\frac{1}{12}\right)}{24'} = 1.251$$

Banded tendons → 0.675 K/ft ↑

Uniformly Distributed tendons → 1.25 K/ft ↑

Load on slab

$LL = 55\,psf$

$DL_w = 150\,pcf\left(\frac{10''}{12''}\right) = 125\,psf$

Misc DI = 10 PSF

$1.2(125+10) + 1.6(55)$

$= 250\,psf \times 27' = 6.75\,Klf$ ← floor loading

$-0.68\,Klf$ ← Banded tendons

$-1.25\,Klf$ ← Uniform. dist.

$4.82\,KlF$

or

$= 250\,psf$

$0.68/27 = 25\,psf$

$1.25/24 = 52.\,psf$

$173\,psf$

* upon structural engineers notes, the % of DL balance should be $40\% \leq X \leq 150\%$. With a goal of 75%.

$\frac{173}{250} = .69$ ∴ only balancing 31% of load.

Use 63psf and continue with two-way slab analysis

Span/Depth Ratio

two-way slab 40-45 recommended by PTI

$$\frac{27 \times 12}{8} = 40.5 \checkmark$$

Direct Design Method check

$q_u = 63$ psf $\quad M_o = \frac{q_u \ell_2 \ell_n^2}{8}$

slab Moment Determination

Long Direction: $M_o = [63(24)(27 - \frac{24}{2})^2]/8 = 42.5$ ftk

Short Direction: $M_o = [63(27)(24 - \frac{24}{2})^2]/8 = 30.6$ ftk

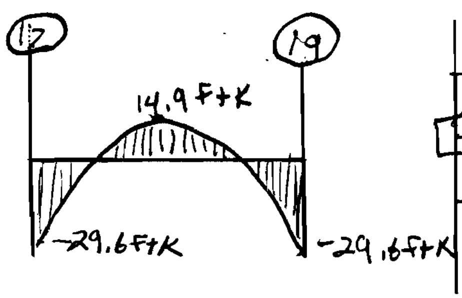
Coefficients for Factored moments

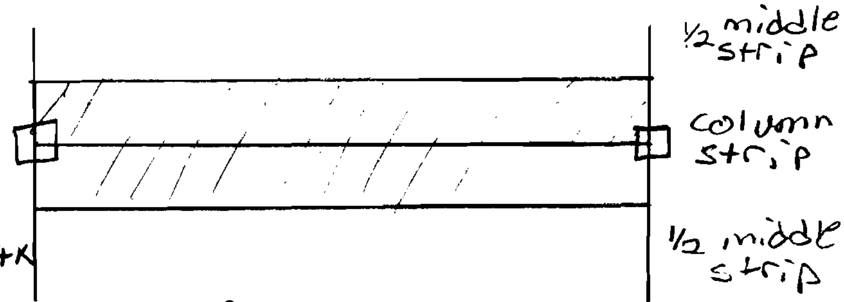
interior span:
- at Midspan → $0.35 M_o$
- at Column → $0.65 M_o$

Exterior span:
→ $0.52 M_o$
→ $0.70 M_o$

No interior beams:
$\alpha = 0$
$\beta = 0$

	M-	M+
Long Direction	$0.65 M_o$ = 27.6 ftK	$0.35 M_o$ = 14.9 ftK
short Direction	$0.7 M_o$ = 21.42 ftK	$0.52 M_o$ = 15.9 ftK





% negative moment (13.6.4.1)

$\alpha_F \times \frac{\ell_2}{\ell_1} = 0$

75% to CS = 0.75(-27.6) = -20.7 FtK

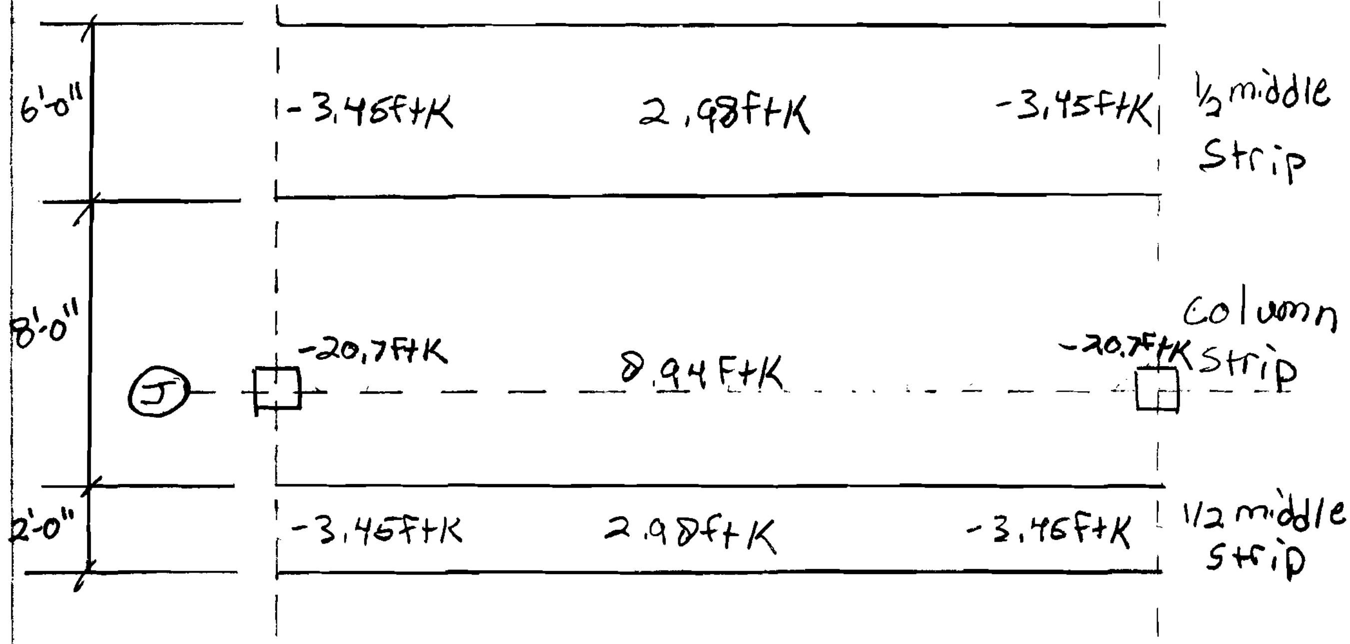
25% to MS = 0.25(-27.6) = -6.9 FtK

% positive moment (13.6.4.1)

$\alpha_F \times \frac{\ell_2}{\ell_1} = 0$

60% to CS = 0.60(14.9) = 8.94 FtK

40% to MS = 0.40(14.9) = 5.96 FtK



Check Reinforcement for Bending

Column Strip $M+ = 8.94\,ftK$

$A_s = (6)\#5 @ 12"$ Top bars
$\#4 @ 24"$ bottom bars

$A_s = 6(0.31) + 4(.2) = 2.66\,in^2$

$d = 8" - 0.75" - .5 - \frac{.625}{2} = 6.44"$

$$a = \frac{(2.66in^2)(60000psi)}{0.85(4000)(27'\times 12")} = 0.093$$

$$\phi M_n = 0.9(2.66)\left(6.44 - \frac{0.093}{2}\right)(60000)\frac{1}{12}\frac{1}{1000}$$
$$= 76.5\,ftK > 8.94\,ftK \checkmark$$

* reinforcing stays the same throughout the bay

Middle strip $M+ = 5.96\,ftK$

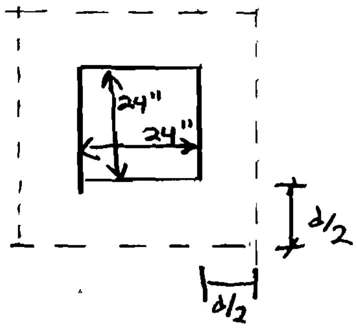
$\phi M_n = 76.5\,ftK > M+ = 5.96\,ftK \checkmark$

Column Strip $M- = -20.7\,ftK$

$\phi M_n = 76.5\,ftK > M- = -20.7\,ftK \checkmark$

Check Shear Capacity (two-way)

Column J17 $q_u = 63\,psf$



$d = 6.44''$

$d/2 = 3.22''$

$b_o = 4(24 + 3.22) = 108.88\,in^2$

$\beta = \frac{24''}{24''} = 1.0 \quad \alpha_s = 40$

$V_{cmin} = $
- $2 + 4/\beta = 6''$
- $2 \times \frac{\alpha_s d}{b_o} = 4.73''$
- $4''$ ← controls

$V_c = V_{cmin}\sqrt{f'c}\,(b_o)(d)$

$= 4\sqrt{4000}\,(108.88)(6.44) = 177.39K$

$\phi V_c = 0.75(177.39K) = 133K$

$q_u = 250\,psf$ ← see note under assumptions

$V_u = 0.25\left[22 \times 16 - \frac{(30.44 \times 30.44)}{144}\right] = 86.4\,K$

$\phi V_c = 133K > 86.4K$ ✓

(one - way shear)

$V_u = (0.25 KSF)(22 - \frac{27}{12})(16 - \frac{24}{12})$
$= 70K$

$V_c = 2\lambda\sqrt{f'_c}\, b_w\, d$
$= 2(1.0)\sqrt{4000}\,(24 \times 12)(6.44)/1000$
$= 234.6K$

$\phi V_c = 0.75(234.6)$
$\phi V_c = 175.9K > V_u = 70K$ ✓

Column Gravity check

- Check Exterior Column at Base K17
- $f'_c = 4000$ psi
- ground floor to 9th floor (roof)

size: 24"x24"
reinforcing: 8 #9, #3 ties @ 18"

- Check Interior column at Base J17
- $f'_c = 4000$ psi
- ground floor to 9th floor (roof)

size: 24"x24"
reinforcing: 8#9, #3 ties @ 18"

AMPAD

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Exterior Column K17

$$\text{Tributary Area} = \left(\frac{17'+27'}{2}\right)\left(\frac{24'2''}{2}\right)$$
$$= 265.8\ ft^2$$

Typical Exterior Wall = $52 psf \times 9.3' = 484 plf$

~~DL + LL + Lr~~

$$\text{Dead} = 108 psf(265.8\ ft^2) + 484 plf\left(\frac{17'+27'}{2}\right)$$
$$= 39.4K$$

$$\text{Roof} = 125 psf(265.8\ ft^2)$$
$$= 33.2K$$

$$\text{Live} = 55 psf(265.8\ ft^2)$$
$$= 14.6K$$

$$DL = 39.4K(8) + \frac{150 psf}{1000}\left[\left(\frac{24}{12}\times\frac{24}{12}\times 300'\right) + \left(\frac{24}{12}\times\frac{24}{12}\times 61\right)\right]$$
$$= 531.8K$$

$$LL = 14.6K(9) = 131.4K$$

$$P_u = 1.2\,DL + 1.6\,LL + 0.5\,L_r$$
$$= 1.2(531.8) + 1.6(131.4) + 0.5(33.2)$$
$$= 865K$$

$$\phi P_n = 0.8\phi\left[0.85 f'_c(A_g - A_s) + f_y A_s\right] \longrightarrow \text{ACI section } 10.3.6.2$$

$$\phi = 0.65,\quad A_g = 24'' \times 24'' = 576 in^2 \quad A_s = 8\overset{\#9\ bars}{(1.00)} = 8 in^2$$

$$\phi P_{n,max} = 0.8(0.65)\left[0.85(4000)(576-8) + 60000(8)\right]$$

$$\phi P_{n\,max} = 1311.7K$$

$$P_u = 865K \leq 1311.7K\ \checkmark$$

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Interior Column J17

Tributary Area $= \left(\frac{17'+27'}{2}\right)\left(\frac{24'+8'}{2}\right)$

$= 352 ft^2$

DL + LL + Lr

Dead $= 108 Psf (352 ft^2)$

$= 38.0 K$

Roof $= 125 Psf (352 ft^2)$

$= 44.0 K$

Live $= 55 Psf (352 ft^2)$

$= 19.4 K$

$DL = 38.0(8) + \frac{150 PCF}{1000}\left[\left(\frac{24}{12} \times \frac{24}{12} \times 300'\right) + \left(\frac{24}{12} \times \frac{24}{12} \times 6'\right)\right]$

$= 520.6 K$

$LL = 19.4(9) = 174.6 K$

$P_u = 1.2(520.6) + 1.6(174.6) + 0.5(44)$

$= 926.08 K$

* column dimensions and properties are the same as exterior column. Therefore, $\phi P_{n,max}$ is the same $= 1311.7 K$

$P_u = 926.1 K \leq \phi P_n = 1311.7 K$ ✓

6 Design Alternate 1: Two Way Flat Slab

The typical 24’x27’ bay will now be redesigned as a two way concrete slab without the use of post-tensioning. This will allow comparison to be drawn between post tensioned and non-post tensioned systems.

Design Alternate #1

- Two way flat plate slab
- Analyze Typical Bay bound by grid 17, 19, K, J
- exclude post tensioning
- Determine slab thickness, reinforcement size & layout
- Check shear, flexure and deflection

  * Design follows ACI 318-11 code, referenced sections are noted in parenthesis

Slab thickness (9.5.3, Table 9.5C)

↳ No drop panels → interior panel, $F_y = 60000psi$

$$l_n/33 = 27'/33 = 0.82' = 9.8''$$ use 10" slab

Loading on slab

$LL = 55psf$

$DL_w = 150pcf\left(\frac{10''}{12''}\right) = 125psf$

Misc, $DL = 10psf$

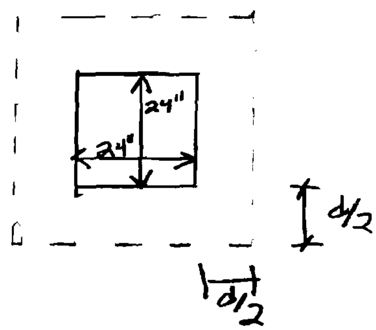
$$q_u = 1.2(125+10) + 1.6(55)$$
$$= 250 psf$$

* since the slab was sized using ACI 318-11 Table 9.5(c) deflection may be neglected

Check shear capacity (Two-Way)

Column J17 $\quad q_u = 250$ psf

$\frac{3}{4}$" cc assumed



$d = 10 - .75 - \frac{3(.625)}{2}$

$d = 8.3125"$

$d/2 = 4.156"$

$b_0 = 4(24 + 4.156) = 112.6 in^2$

$\beta = \frac{24"}{24"} = 1.0 \quad \alpha_s = 40$

$$V_{cmin} = \begin{cases} 2 + 4/\beta = 6" \\ 2 + \frac{\alpha_s d}{b_0} = 5.91" \\ 4" \leftarrow \text{controls} \end{cases}$$

$V_c = V_{cmin} \sqrt{f'_c}\,(b_0)(d)$

$= 4\sqrt{4000}(112.6)(8.3125) = 237K$

$\phi V_c = 0.75(237K) = 177.78K$

$V_U = (.250)\left[22 \times 16 - \left(\frac{35.3 \times 35.3}{144}\right)\right] = 85.8K$

$\phi V_c = 177.78 > V_u = 85.8K$ ✓

- slab passes for punching shear

(one-way shear)

$$V_U = (0.25\ \text{ksf})\left(22 - \frac{24}{12}\right)\left(16 - \frac{24}{12}\right)$$
$$= 70\ K$$

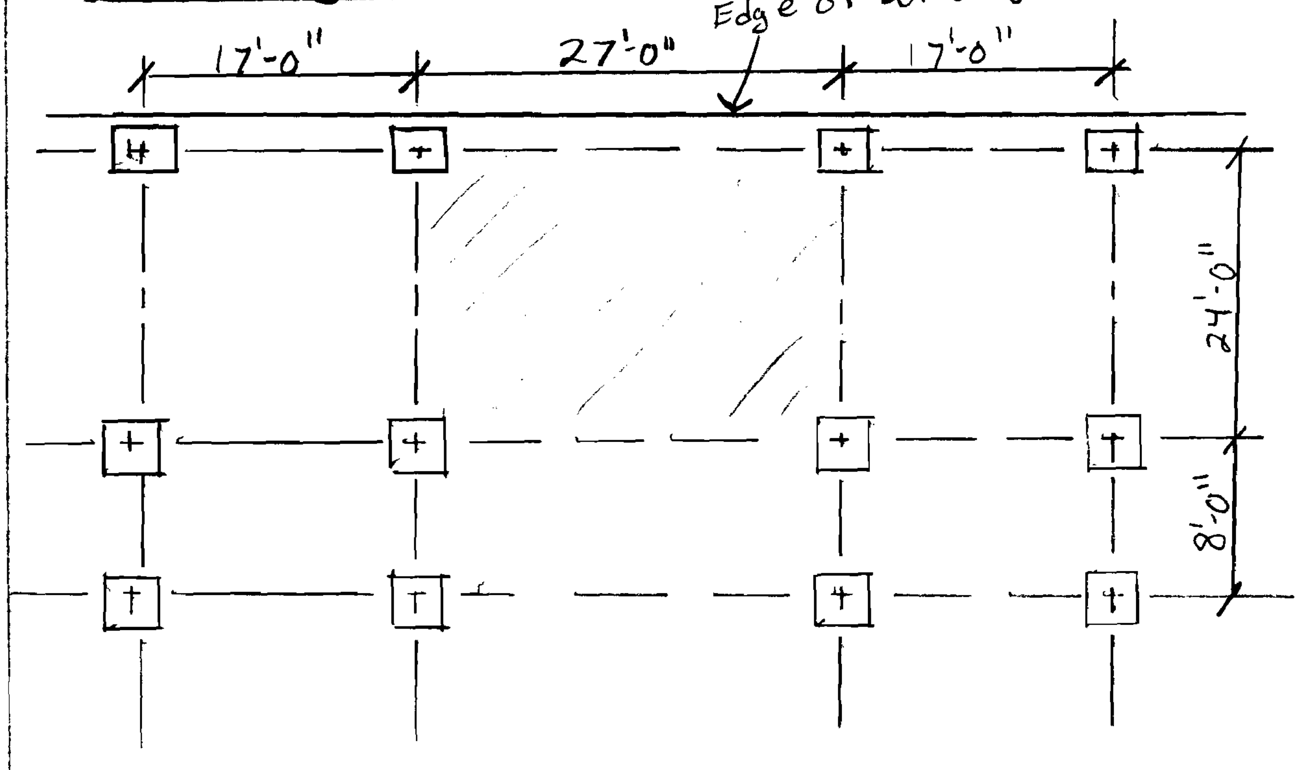
$$V_C = 2(1.0)\sqrt{4000}\ (24 \times 12)(8.3125)/1000$$
$$= 302.8\ K$$

$$\phi V_C = 0.75\ (302.8)$$
$$= 227$$

$$\phi V_c = 227K > V_U = 70K\ \checkmark$$

- slab passes one way shear

Direct Design Method



- All columns are 24"x24"

$q_U = 250\text{psf}$ $\quad M_0 = \dfrac{q_U\, l_2\, l_n^2}{8}$

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Slab Moment Determination

Long Direction: $M_o = \left[250(24)\left(27-\frac{24}{2}\right)^2\right]/8 = 469\text{ ftK}$

Short Direction: $M_o = \left[250(27)\left(24-\frac{24}{2}\right)^2\right]/8 = 408\text{ ftK}$

Coefficients for Factored moments:

interior spans:

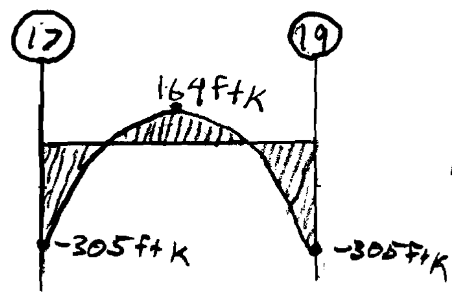
- at Midspan → $0.35M_o$
- at column → $0.65M_o$

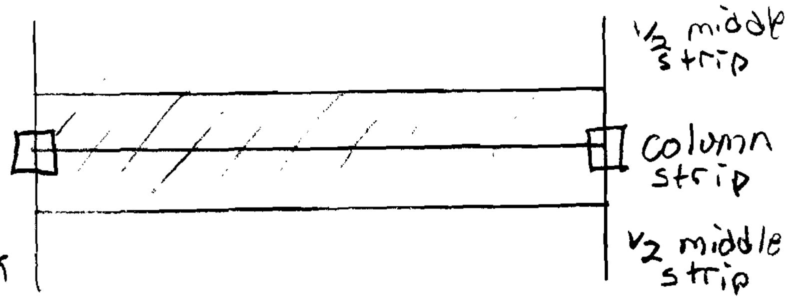
Exterior Edge:

- at midspan → $0.52M_o$
- at column → $0.70M_o$

	M −	M +
long direction	$0.65M_o =$ 305 ftK	$0.35M_o =$ 164 ftK
short direction	$0.7M_o =$ 286 ftK	$0.52M_o =$ 212 ftK

No interior beams ∴ $\alpha = 0$, $\beta = 0$





% negative moment (13.6.4.1)

$\alpha_f \times \frac{l_2}{l_1} = 0$

75% to CS $= 0.75(-305) = -228.8$ ftK

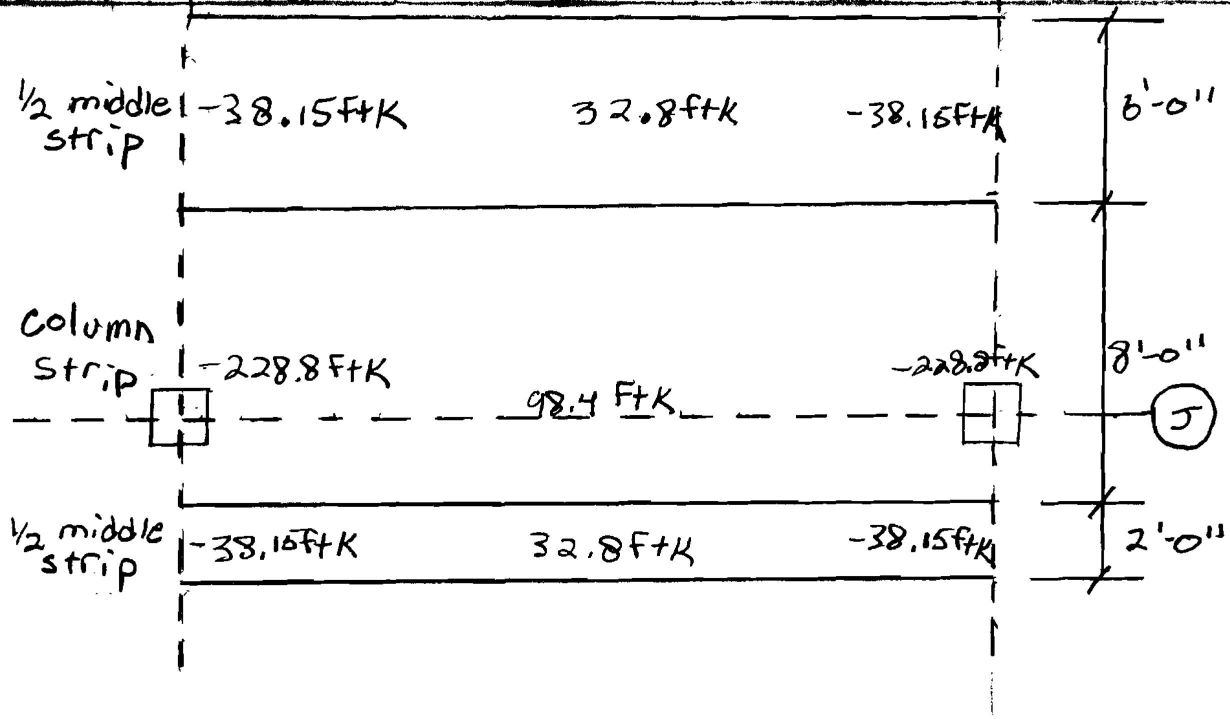
25% to MS $= 0.25(-305) = -76.3$ ftK

% positive moment (13.6.4.1)

$\alpha_f \times \frac{l_2}{l_1} = 0$

60% to CS $= 0.60(164) = 98.4$ ftK

40% to MS $= 0.40(164) = 65.6$ ftK



Design Reinforcement for Bending

$$As_{req} = \frac{M_u}{\phi f_y j d} = \frac{-228.8(12")}{0.9(29000)(.95)(8.325)}$$

$$= 13.3 \text{ in}^2$$

Spacing (13.3.2)

$$S_{max} \leq 2h = 2(10) = 20"$$

Minimum Reinforcement

$$As_{min} \geq 0.0018bh \qquad h = 10" \quad b = \text{column strip width} = 8' \times 12 = 96"$$

$$\geq 0.0018(96)(10)$$

$$As_{req} \geq 1.73 \text{ in}^2 \checkmark$$

Try 18 #6's @ 12" $18(.44) = 7.92 > 1.73$

$$a = \frac{(7.92 \text{ in}^2)(60000)}{0.85(4000)(10 \times 12)} = 1.16$$

$$\phi M_n = 0.9(7.92)\left(8.325 - \frac{1.16}{2}\right)(60000)\frac{1}{1000}\frac{1}{12}$$

$$\phi M_n = 276 \text{ ftK} > 228.8 \text{ FtK} \checkmark$$

Middle Strip M− $M = 76.3$ ft K

$$A_{s,reqd} = \frac{76.3 \times 12}{0.9(60)(.95)(8.325)} = 2.14 \text{ in}^2$$

Try (8) #6 @ 12" $8(.44) = 3.52 \text{ in}^2$

$$a = \frac{(3.52)(60000)}{0.85(4000)(10 \times 12)} = 0.52$$

$$\phi M_n = 0.9(3.52)\left(8.325 - \frac{0.52}{2}\right)(60)\frac{1}{12}$$

$\phi M_n = 127$ ft K ≥ 76.3 ft K ✓

Column strip M+ $M = 98.4$ ftK

$$A_{s,reqd} = \frac{98.4 \times 12}{0.9(60)(.95)(8.325)} = 2.76 \text{ in}^2$$

use same configuration as above for ease of constructibility

(8) #6 @ 12"

$\phi M_n = 127$ ftK ≥ 98.4 ftK

Middle strip, M+ $M = 32.8$ ftK

$$A_{s,reqd} = \frac{32.8 \times 12}{0.9(60)(.95)(8.325)} = 0.92$$

Use same configuration as above for ease of constructibility

(8) #6 @ 12"

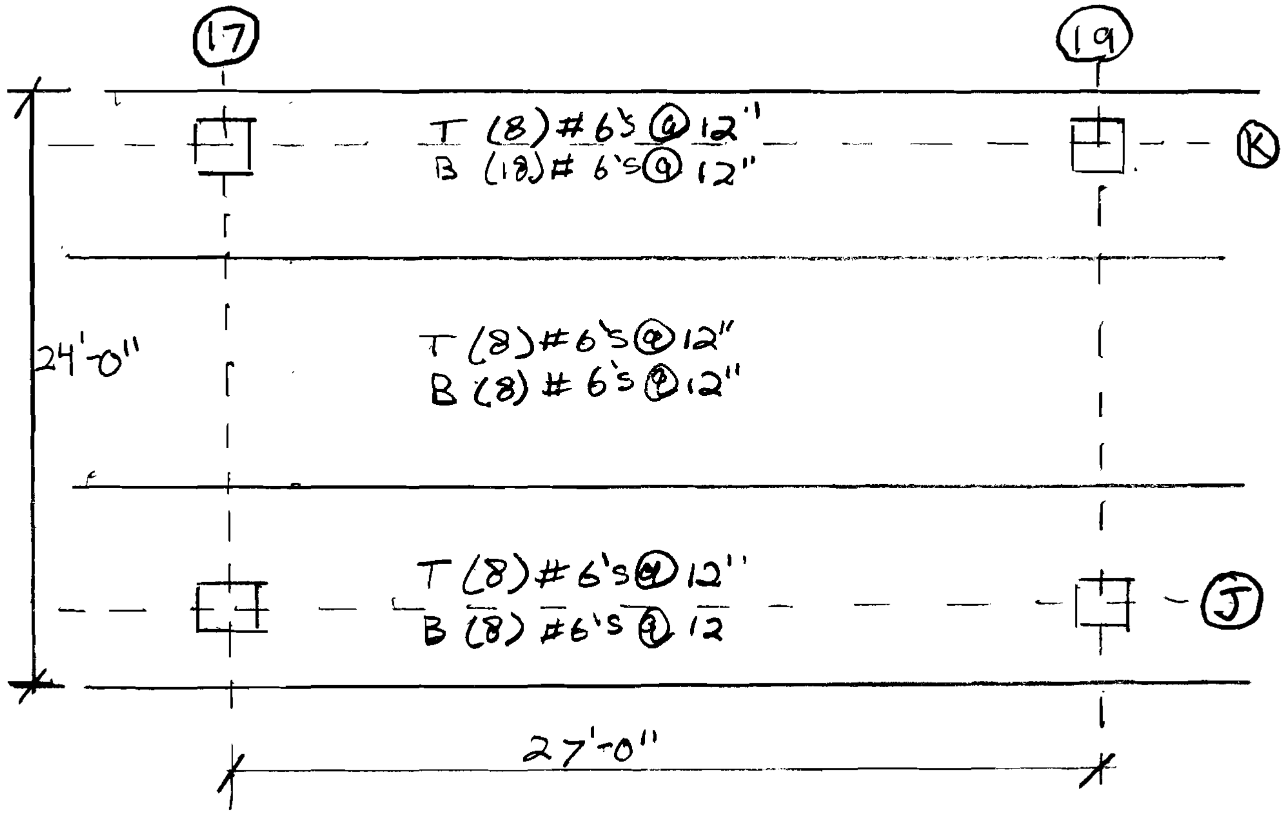
$\phi M_n = 127 \geq 32.8$

Slab Reinforcing Summary

Column strip: Top = (8) #6's @ 12"
Bottom = (18) #6's @ 12"

Middle strip: Top = (8) #6's @ 12"
Bottom = (8) #6's @ 12"

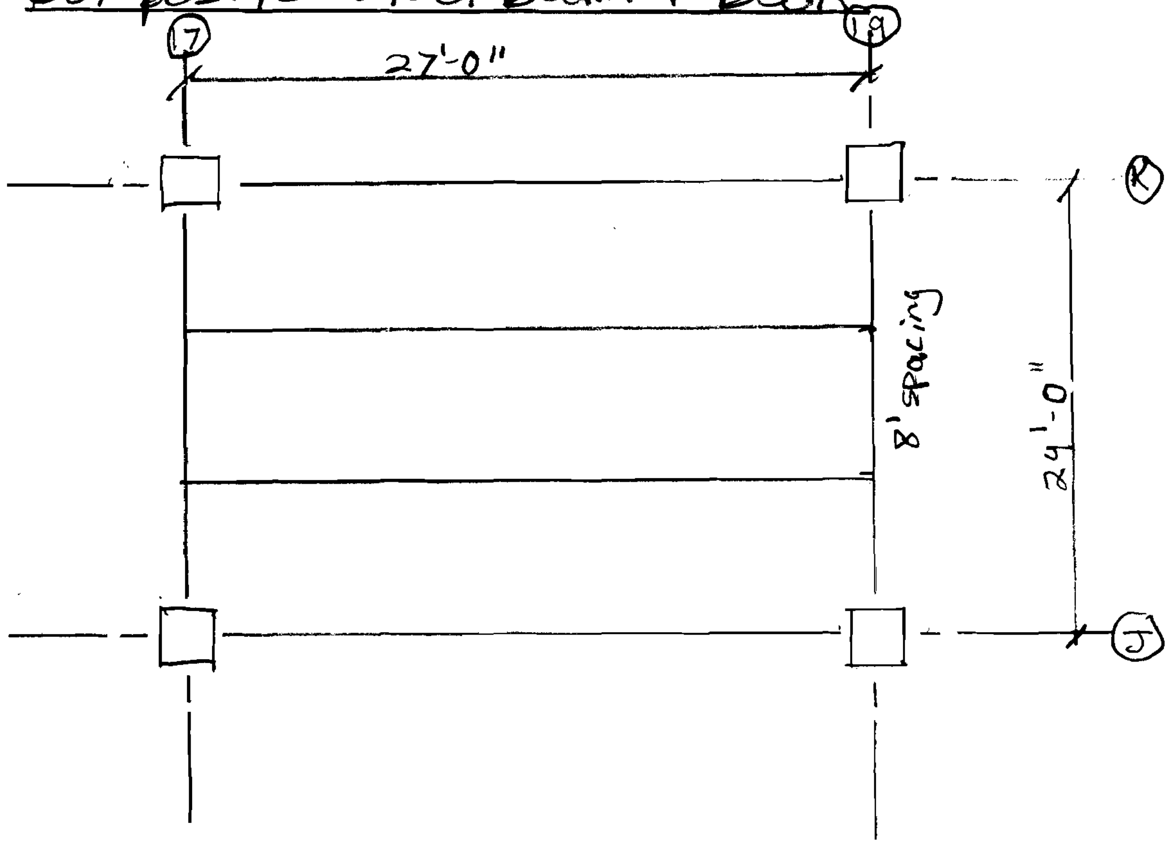
Design #1 layout



7 Design Alternate 2: Composite Steel Beam

The typical 24'x27' bay will now be redesigned using composite steel deck and beams. Decks are designed using Vulcraft catalogs and the beams/girders will be selected based on economy sizes within the AISC Steel Manual.

Design Alternate # 2:
Composite Steel Beam + Deck



Decking (using vulcraft catalog)

1.5VLR18 3.5" NW concrete
- 5 total thickness
- 56PSF
- 3 Span clear span = 9'-3"
- 293psf

Design loads

Account for beam allowance, miscelaneous, finishes

$DL = 56 + 15 + 5 + 3 = 79 psf$

$LL = 55 psf$

$W_u = 1.2D + 1.6L$

$= 1.2(79) + 1.6(55) = 183 psf < 293 psf$ ✓

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Live load reduction:

$$LL = 55\left(0.25 + \frac{15}{\sqrt{648 \times 1}}\right) = 46.2\,psf$$

$$W_U = 1.2(79) + 1.6(46.2) = 169\,psf$$

$$W_U = 169(8' \text{spacing}) = 1.35\,KLF$$

Design Moment:

$$M_U = \frac{1.35(27)^2}{8} = 123\,K\text{-}ft$$

1 stud/rib = 17.2^K/stud

assume $a = 1''$ $\rightarrow$ $Y_2 = 5'' - \frac{1}{2} = 4.5''$

<u>Beam sizes</u> (AISC table 3-19)

W12×16 $\rightarrow$ $\Sigma Q_n = 94.3$ $\quad \phi M_n = 134\,K\text{-}ft > 123\,K\text{-}ft$ ✓

$\#studs = \frac{94.3}{17.2} = 5.5 \rightarrow 6 \times 2 = 12$ studs/beam

W12×14 $\rightarrow$ $\Sigma Q_n = 119$ $\quad \phi M_n = 132\,K\text{-}ft > 123\,K\text{-}ft$ ✓

$\#studs = \frac{119}{17.2} = 6.9 \rightarrow 7 \times 2 = 14$ studs/beam

W10×17 $\rightarrow$ $\Sigma Q_n = 117$ $\quad \phi M_n = 132\,K\text{-}ft > 123\,K\text{-}ft$ ✓

$\#studs = \frac{117}{17.2} = 6.8 \rightarrow 7 \times 2 = 14$ studs/beam

W10×~~45~~ $\rightarrow$ $\Sigma Q_n = 140$ $\quad \phi M_n = 129\,Kft > 123\,K\text{-}ft$ ✓

$\#studs = \frac{140}{17.2} = 8.12 \rightarrow 9 \times 2 = 18$ studs/beam

$$b_{eff} = \min\begin{cases} \frac{span}{8} \times 2 = \frac{27(12)}{8} \times 2 = 81'' \leftarrow \text{controls} \\ \frac{spacing}{2} \times 2 = \frac{8(12)}{2} \times 2 = 96'' \end{cases}$$

Check economy

1stud = 10 lb steel

W12x16[12]

$16^{lb}/_{ft} \times 27ft + 10(12) = 552\ lbs$

W12x14[14]

$14^{lb}/_{ft} \times 27ft + 10(14) = 518\ lbs$

W10x15[18]

$15^{lb}/_{ft} \times 27ft + 10(18) = 585\ lbs$

$a = \frac{119}{0.85(4)(81)} = 0.43'' < 1''$ ✓

Check unshored strength

W12x14 $\phi M_p = 65.3$ $I_x = 88.6\ in^4$

$W_u = 1.2(79\times8 + 14) + 1.6(46.2) = 0.85\ klf$

$M_u = \frac{0.85(27)^2}{8} = 77.38\ ftK$

W12x14 $\phi M_p = 65.3\ ftK < 77.38$ ✗ No good

W10x26 $\phi M_p = 117\ ftK > 77.38\ ftK$ ✓

$I_x = 96.3\ in^4$

Check size, studs

W10x26 → $\Sigma Q_n = 95.1$ $\phi M_n = 173\ kft > 123\ K\text{-}ft$ ✓

$\#\ studs = \frac{95.1}{17.2} = 5.5 \rightarrow 6\times2 = 12$ studs/beam

Check wet concrete Deflection

$W_{wet} = 79(8) + 26 = 0.658\ klf$

$\Delta_{wc} = \frac{5(0.658)(27)^4(1728)}{384(29000)(144)} = 1.62''$

$\Delta_{max} = \frac{L}{240} = \frac{27\times12}{240} = 1.35'' < 1.62''$ ✗ No good

-could camber or use shoring

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Camber

W10×26 $= 0.8(1.62) = 1.29 \rightarrow 1''$

- 1" camber needed
- could try larger depth to avoid camber

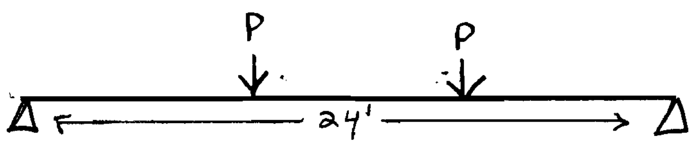
Check live load Deflection

$W_{LL} = 55\text{psf}\,(8\text{ft})/1000 = 0.44\text{ klf}$

$$\Delta_{LL} = \frac{5(0.44)(27)^4\,1728}{384(29000)(271)} = 0.67''$$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{(27)(12)}{360} = 0.9'' > 0.67'' \checkmark$$

Composite Girder Design



$P_u = 1.2(79) + 1.6(46.2) = 169\text{ psf}$

$P_u = 169\text{psf}\left(24' \times \left(\frac{27'}{2} + \frac{12'}{2}\right)\right) = 89.2\text{ K}$

Design moment:

$M_u = Pa = 89.2\text{K} \times 8' = 713.9\text{ ftK}$

assume $a = 1''$ $y_2 = 5'' - \frac{1}{2} = 4.5''$ $b_{eff} = 81''$

Try W21×62

$\Sigma Q_n = 229$ $\phi M_n = 752\text{ ftK} > 713.9\text{ ftK} \checkmark$

$\#\text{studs} = \frac{229}{17.2} = 13.3 \times 2 = 27\text{ studs}$

$$a = \frac{229}{0.85(4)(81)} = 0.83'' < 1'' \checkmark$$

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Check unshored strength

W 21x62 $\phi M_n = 540$ ftk $I = 1550$ in^4

$W_U = 1.2(79 \times 8 + 14) + 1.6(46.2) = 0.85$ KLF

$M_U = \frac{0.85(24)^2}{8} = 61.2$ ftK < 540 ftK ✓

Check Wet concrete Deflection

$W_{wc} = 79(8) + 62 = 0.694$ KLF

$\Delta_{wc} = \frac{Pa}{24EI}(3l^2 - 4a^2)$

$= \frac{89.2K(8)}{24(29000)(1550)}(3(24)^2 - 4(8)^2)$

$= 0.0097'' < \frac{L}{240}$ ✓

Check live load Deflection

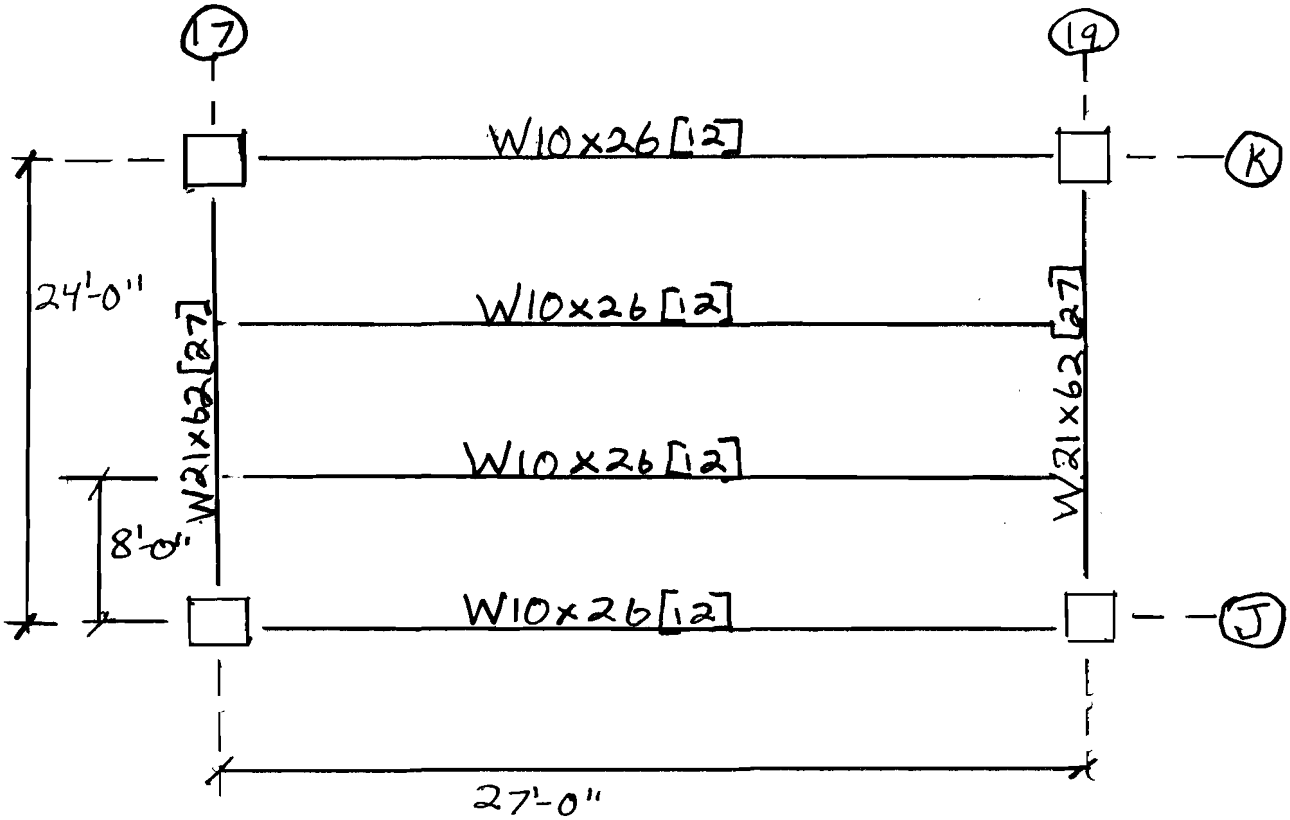
$W_L = 55$ psf $(8$ft$)/1000 = 0.44$ KLF $\times 22 = 9.68$K

$\Delta_{LL} = \frac{9.68(8)}{24(29000)(1550)}(3(24)^2 - 4(8)^2)$

$= 0.0001'' < L/360$ ✓

Design #2 layout

Deck → 1.5VL18, 5" NW Concrete

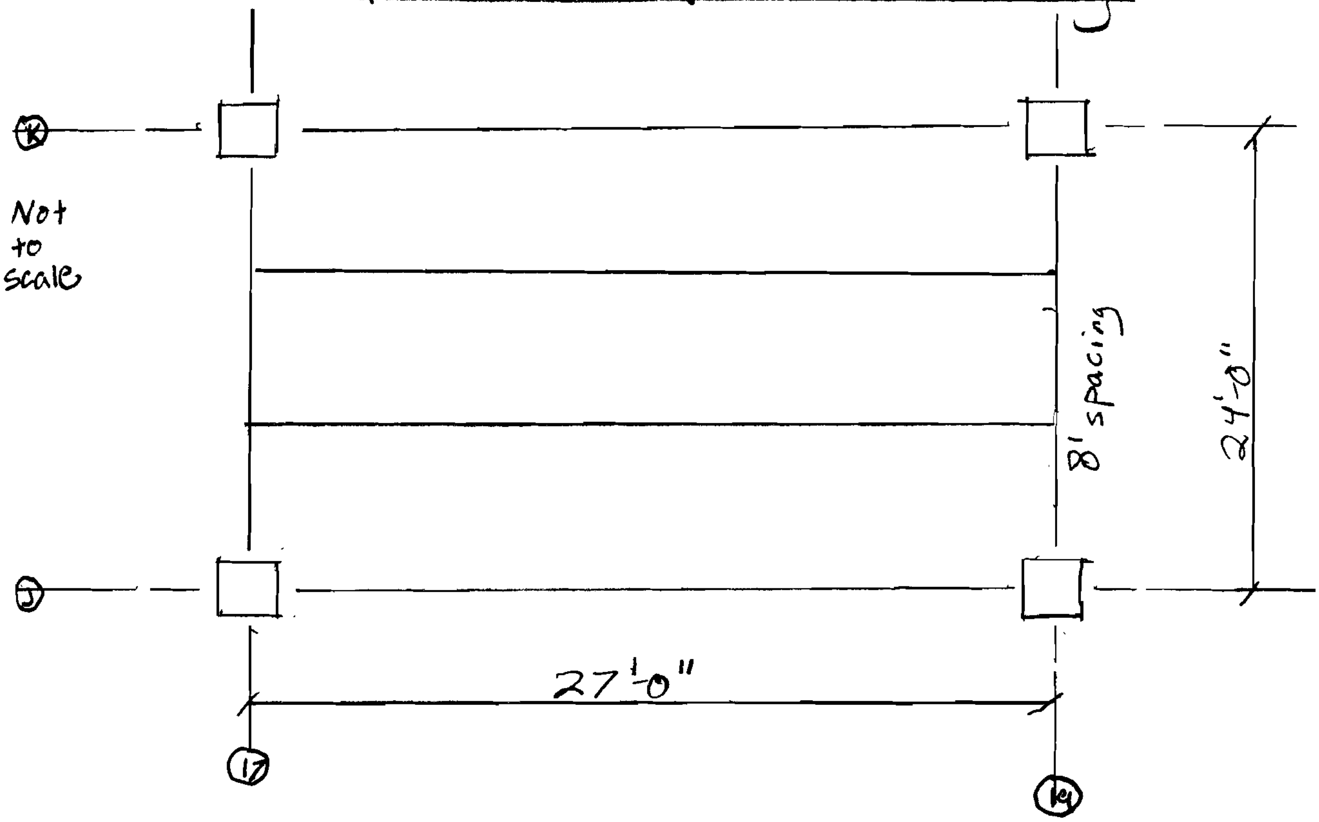


8 Design Alternate 3: Non-Composite Steel Beam

The typical 24’x27’ bay will now be redesigned using non-composite steel deck and beams. Decks are designed using Vulcraft catalogs and the beams/girders will be selected based on economy sizes within the AISC Steel Manual. This will allow comparison to be drawn between composite and non-composite systems.

Design Alternate #3

Non Composite Steel Beam + Decking



Decking (using vulcraft catalog)

Try 1.5C20, 3.5" NW concrete

- t = 2.00"
- 38 psf
- 3 spans clear span - 9'-1"

DL = 38 + 15 + 5 + 3 = 61 psf
superimposed, collateral

$W_u = 1.2(61) + 1.6(55) = 161.2 \times 8'$ spacing

$w_u = 1.29$ KLF

$M_u = \frac{(1.29)(27)^2}{8} = 117.6$ ftK

Strength

Try W 12x26 $\phi M_n = 140$ ftK > 117.6 ftK

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Check Deflection

Find required I_x to limit deflections to $L/240$

$$I_{req} = \frac{5(240)(61+55)(8)(27\times12)^3}{384(29{,}000)\ 12\ ^{in}/_{ft}} = 283.4\ in^4$$

Find required I_x to limit live loads Δ to $L/360$

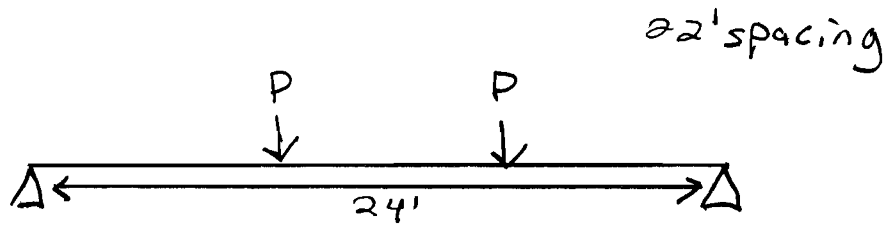
$$I_{req} = \frac{5(360)(55)(8)(27\times12)^3}{384(29{,}000)\ 12\ ^{in}/_{ft}} = 201.6\ in^4$$

Use W14x30

$\phi M_n = 177\ ftK > 117.6\ ftK$

$I_x = 291\ in^4 > 283.4\ in^4$

Girder Design



$P = (\overset{DL}{61} + \overset{LL}{55})(8')(24') = 22.3$ Kips

$M_{max} = Pa = 22.3K(8')$

$= 178.4\ ftK$

Check Deflection

Find required I_x to limit deflections to $L/240$
↳ reference steel manual table 3-23 #9

$$\frac{L}{240} = \frac{Pa}{24EI}\left(3l^2 - 4a^2\right)$$

$$I_{req} = \frac{240\,Pa}{24EL}\left(3l^2 - 4a^2\right)$$

$$= \frac{240(22.3)(8\times12)}{24(29000)(24\times12)}\left(3(24\times12)^2 - 4(8\times12)^2\right)$$

$$= 543.3 \text{ in}^4$$

Find required I_x to limit live loads to $L/360$

$$I_{req} = \frac{360(10.6)(8\times12)}{24(29000)(24\times12)}\left(3(24\times12)^2 - 4(8\times12)^2\right)$$

$$= 387.4 \text{ in}^4$$

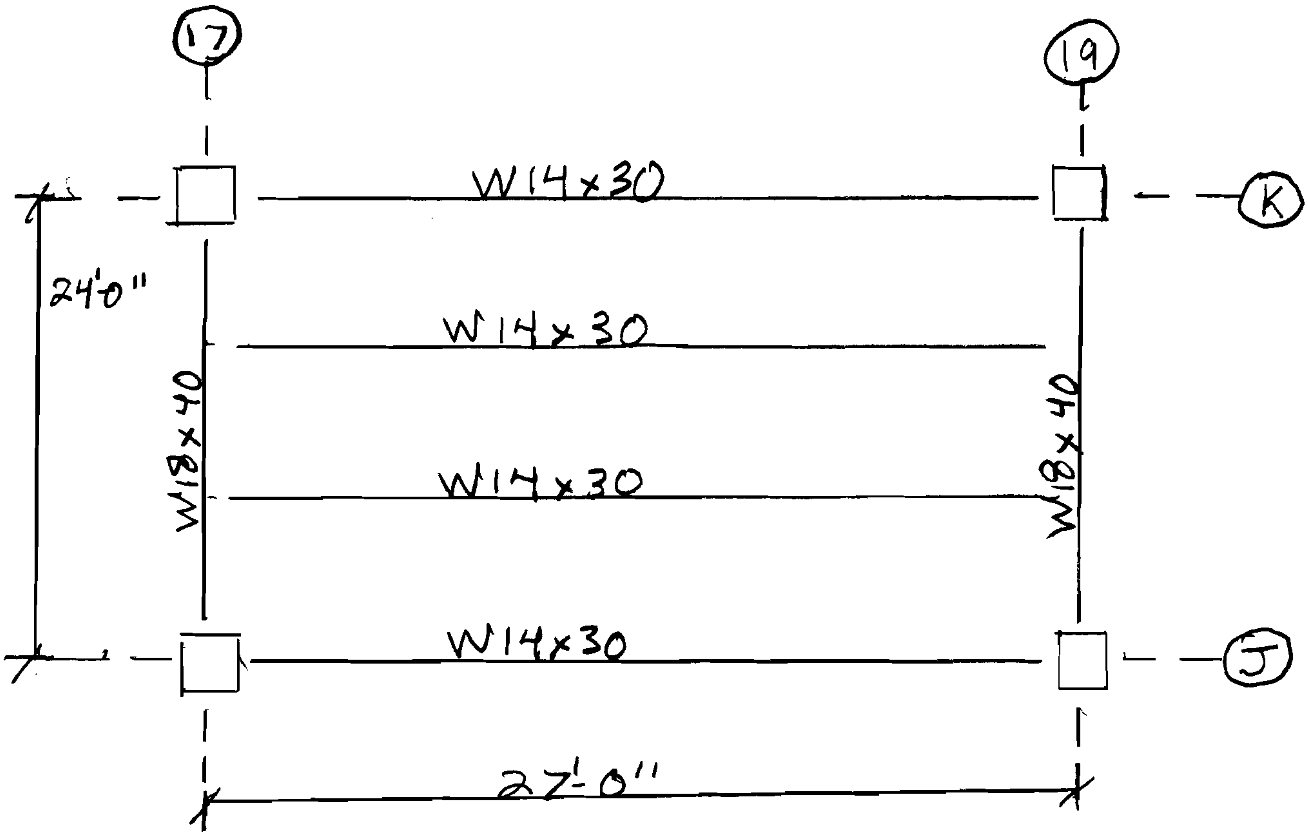
Use W18x40

$$\phi M_n = 294 \text{ ftK} > 178.4 \text{ ftK}$$

$$I_x = 612 \text{ in}^4 > 543.3 \text{ in}^4$$

Design #3 layout

Deck-1.5C20, 3.5" N.W concrete



System Comparison				
Considerations	**Post Tensioned Two Way Slab**	**Typical Reinforced Two Way Slab**	**Composite Steel**	**Non-Composite Steel**
Architectural				
Total System Depth	8"	10"	26"	21.5"
Fire Rating	2Hr.	2Hr.	2Hr.	2Hr.
System Information				
Weight	125psf	135psf	79psf	61psf
Cost per square foot	$21.36/SF	$20.25/SF	$19.22/SF	$24.31/SF
Servicability				
Vibrations	Minimal	Minimal	Expected	Expected
Future Design Potential				
Advantages	Smallest depth, limits vibrations, PT decrease load by ~70%	Small depth, limits vibrations	Cheapest option, light weight, minimal formwork	Lightest system, minimal formwork
Disadvantages	Large weight, requires formwork, additional labor hours adds to cost	Largest weight, requires formwork	largest depth, chance of vibrations	large depth, chance of vibrations, most expensive system

In conclusion, when comparing the post-tensioned slab and the typical reinforced two way slab, one can tell that post tensioning does affect the slab depth. The typical reinforced slab requires more reinforcement creating a greater cost per square foot. I would recommend the post tensioned slab for a typical bay when comparing the two.

When looking at a composite steel system vs a non-composite system, two factors affect whether to use the system or not. They are cost and depth. The non-composite steel system costs a large amount more per square foot than the composite system does. However, in this analysis, the depth of the composite system is greater. Upon further analysis and engineering strategies, I believe the depth of the composite system could be reduced making it a more viable option.

9 Lateral Analysis Description

Notebook Submission C involves performing a detailed lateral analysis on West Village Housing north building. I utilized both 3D and 2D modeling to help perform calculations and checks of the buildings lateral force resisting system, concrete shear walls. Since wind was the controlling case over seismic, as determined in notebook submission A, only wind was considered in the hand calculations of this report. Seismic loads will be studied in addition to wind in the computer models. The three dimensional model was constructed using RAM Structures System, while the two dimensional rigidity and drift checks were conducted in SAP 2000.

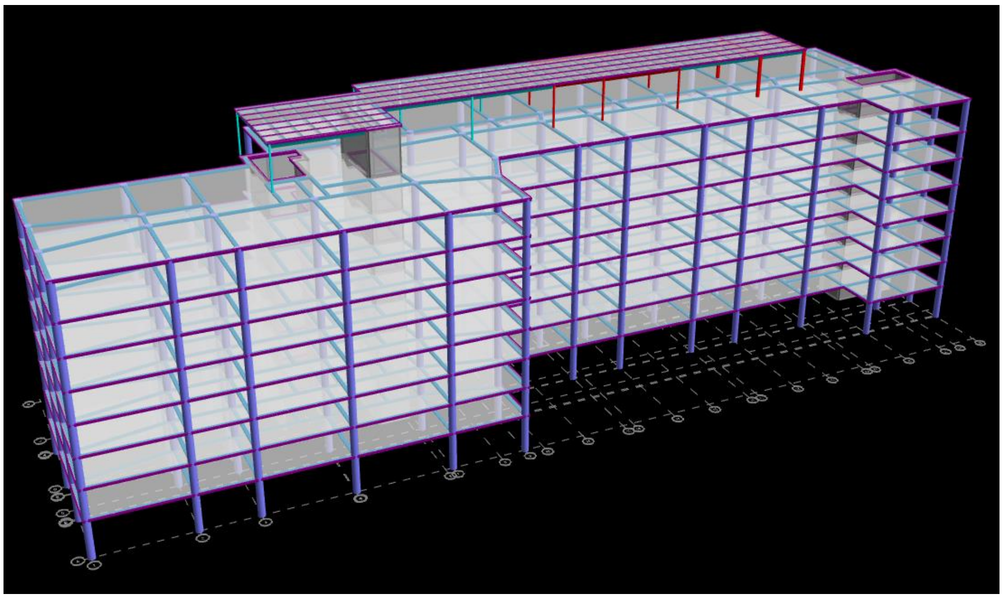


Figure 4: RAM model of north building

The buildings lateral system includes concrete shear walls which were modeled in the figure above. 10 shear walls total the buildings lateral force resisting system. All the walls are 12” thick and extend up to the roof and penthouse roof.

The next section contains the two halves of the typical floor plan. The shear walls are located in red and labeled in blue.

9.1 Lateral Floor Plan

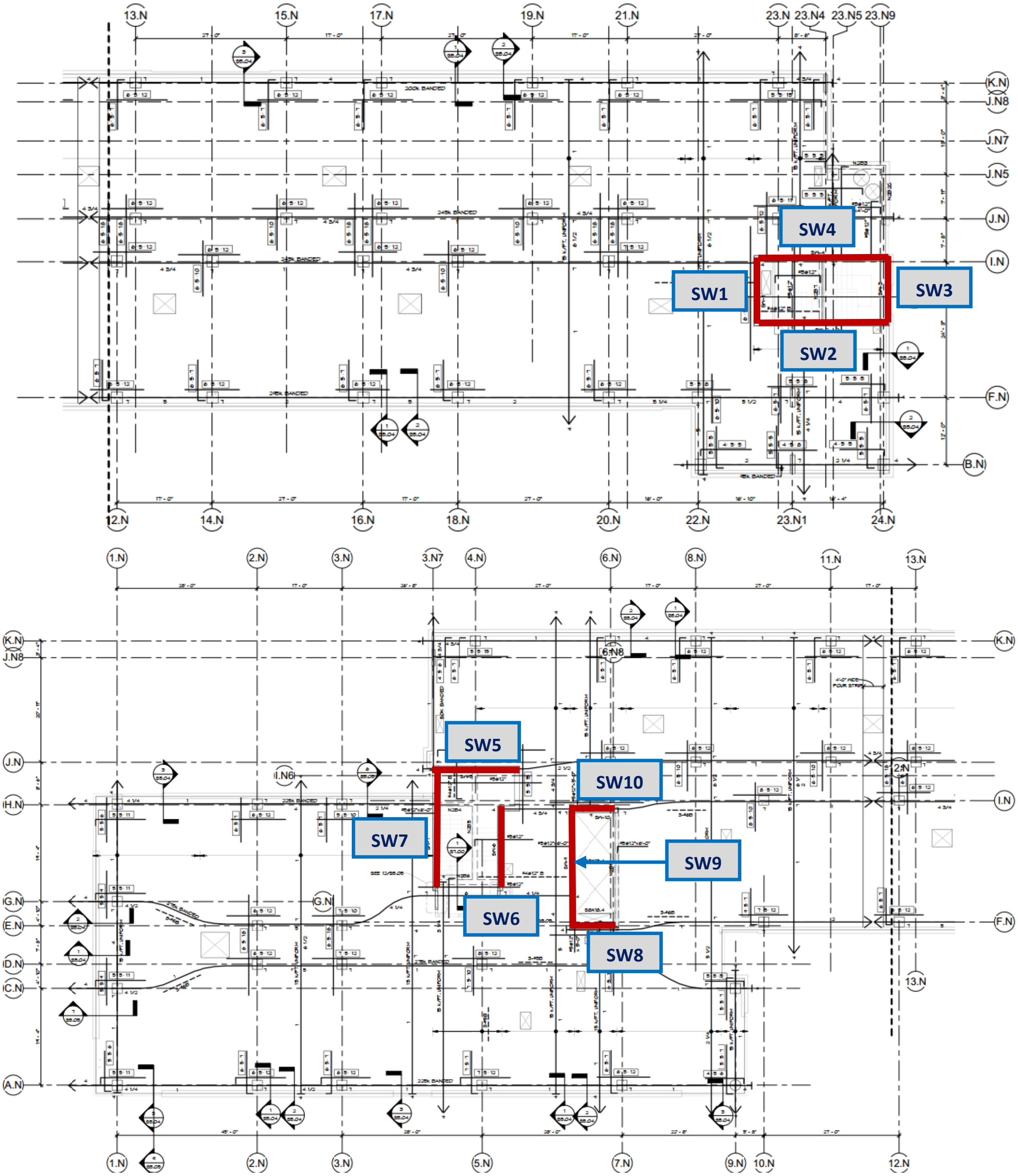


Figure 5: Shear wall locations and labels on typical floor plan

9.2 Modeling Decisions and Assumptions

Two different models were used to help clarify and check the others outputs which were all used in comparison with hand calculations of loads and member checks. Through prior internship experience and help from the structural engineers on the project, a 3D RAM model of West Village Housing was created. With this model, loads were applied to the model by automatic generation. With the 2D SAP 2000 model, the loads that were applied were the previously determined manually calculated loads.

Figure 6: 2D SAP model of shear walls

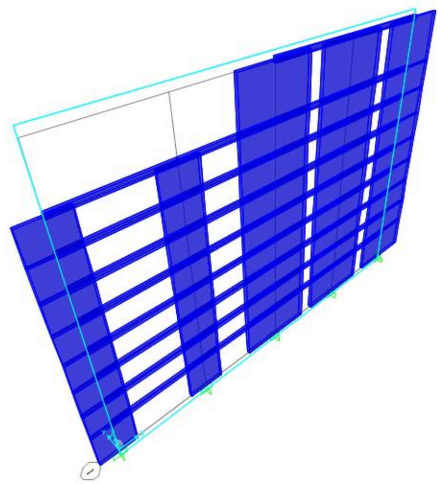


Figure 7: Extruded view of shear wall in 2D SAP model

SAP 2000 was used mainly to model the lateral shear walls on an individual level. Using the concept of coupled shear wall, one can determine the drift and strength of each individual wall using SAP. The idea behind using coupled shear walls, is that the connecting beams allow only axial forces to be distributed to every floor of each shear wall. Hence, a rigid diaphragm is represented by these connecting beams. This model is a simplified version that was used to check outputs of the RAM model. A difference between the SAP and RAM models is that SAP allows you to accurately design the reinforcing in the wall which is just automatically generated in RAM. The following is a compilation of the calculations and checks associated with the analysis of West Village Housing's lateral system.

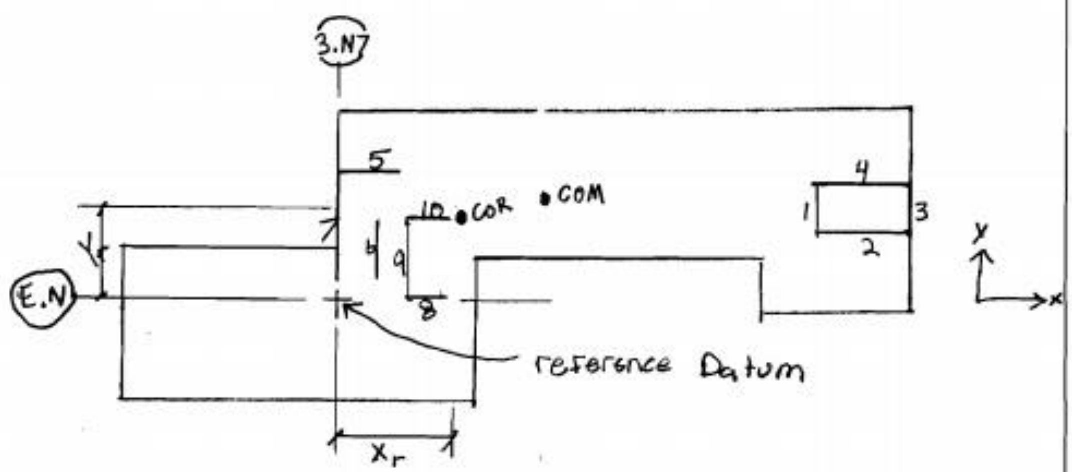
10 Building Properties

The nature of a lateral load depicts where the lateral loads will be applied on a building. For example, wind is a pressure force whereas seismic force is a function of mass. Therefore, these forces will act at different locations on the building. The tables below located those different points at which the forces act on.

10.1 Center of Rigidity Calculations

The center of rigidity of a building is the centroid of the stiffness for that building. The stiffness elements considered for this report include all 10 shear walls as they are the only lateral force resisting elements. Any forces that are applied on a point of the building other than the COR will cause torsion on the building. This is due to the eccentricity of the load applied to the centroid of stiffness. Shear walls are applied at either end of the building to help reduce the difference between the center of rigidity and the center of mass. This leads to a minimal torsion on the building.

Mark Bland	CoR	Notebook Submission C



Eccentricity

$e = C.O.R. - C.O.M.$

$e_y = 103.6' - 145.2' = -41.6'$

$e_y = 54.1 - 51.9' = 2.2'$

vertical shearwalls will resist

horizontal shear walls will resist

Mark Bland	COR	Notebook Submission C

Determine Center of Rigidity

$F'_c = 4000\,psi$

- All shear walls will be treated as cantilevers

$G_{concrete} = 0.4E \qquad E = 57000\sqrt{F'_c} = 3605\,Ksi$

- reference shear wall numbers labeled on lateral floor plan

shear wall stiffness

$$K = \frac{E}{4\left(\frac{h}{b}\right)^3 + 3\left(\frac{h}{b}\right)}$$

* All thickness are the same so assume $t=1$

$$K_1, K_3 = \frac{3605}{4\left(\frac{77.3}{11.5}\right)^3 + 3\left(\frac{77.3}{11.5}\right)} = 2.92\,K/in$$

$$K_2, K_4 = \frac{3605}{4\left(\frac{77.3}{22.58}\right)^3 + 3\left(\frac{77.3}{22.58}\right)} = 21.2\,K/in$$

$$K_5 = \frac{3605}{4\left(\frac{78.8}{17.5}\right)^3 + 3\left(\frac{78.8}{17.5}\right)} = 9.5\,K/in$$

$$K_6 = \frac{3605}{4\left(\frac{96}{24.9}\right)^3 + 3\left(\frac{96}{24.9}\right)} = 15.0\,K/in$$

$$K_7 = \frac{3605}{4\left(\frac{96}{24.9}\right)^3 + 3\left(\frac{96}{24.9}\right)} = 15.0\,K/in$$

$$K_8, K_{10} = \frac{3605}{4\left(\frac{96}{9.5}\right)^3 + 3\left(\frac{96}{9.5}\right)} = 0.9\,K/in$$

$$K_9 = \frac{3605}{4\left(\frac{96}{23.5}\right)^3 + 3\left(\frac{96}{23.5}\right)} = 12.7\,K/in$$

TABLE 6: Center of Rigidity							
ELEMENT LABEL	ELEMENT DIRECTION	DIST. FROM REF. DATUM		Rx	Ry	Rx*Y	Ry*X
		X (FT)	Y (FT)				
SW1	Y	205.75	0	0	2.92	0	600.79
SW2	X	0	15.25	21.2	0	323.3	0
SW3	Y	228.33	0	0	2.92	0	666.7236
SW4	X	0	25.75	21.2	0	545.9	0
SW5	X	0	31.67	9.5	0	300.865	0
SW6	Y	13.5	0	0	15	0	202.5
SW7	Y	0.5	0	0	15	0	7.5
SW8	X	0	-0.5	0.9	0	-0.45	0
SW9	Y	27.5	0	0	12.7	0	349.25
SW10	X	0	22	0.9	0	19.8	0
			Sum	53.7	48.54	1189.415	1826.764

Xr = Sum(Ry*X)/ Sum(Ry)	**37.6341904 + 66 = 103.6 ft**
Yr = Sum(Rx*Y)/ Sum(Rx)	**22.1492551 + 32 = 54.1 ft**

Checking these values with the blue numbers in Figure 8, one will notice that the percent difference in the X direction is about 14% and in the Y direction about 11%. This difference is probably due to the fact that the RAM model is considering the concrete wall near gridline, H-4, as part of the lateral system when it is not. Eliminating that from the RAM Model will move the COR closer to the hand calculations performed in table 6.

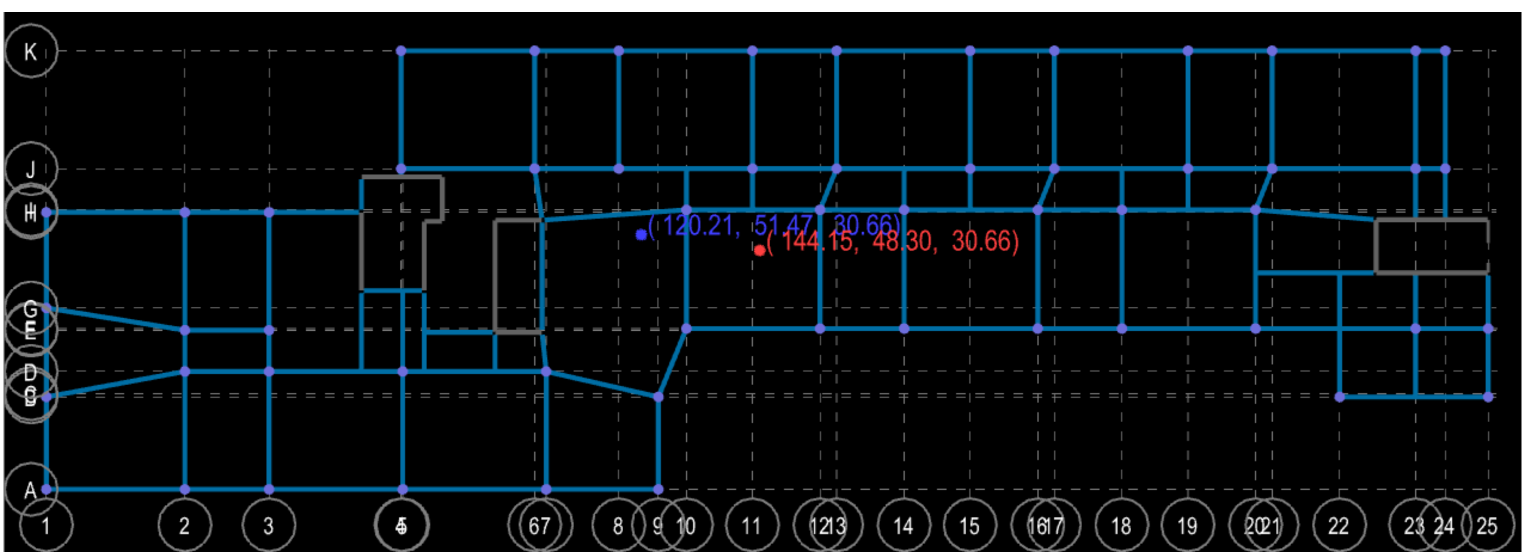


Figure 8: RAM model output of the buildings COR (Blue) and COM (Red)

10.2 Center of Mass Calculations

The center of mass for the building is the average of each floor masses centroid. External loads as moments on the building act through the COM of the building. These loads include the seismic forces calculated in a previous report.

TABLE 7: Center of Mass								
ELEMENT LABEL	AREA (ft2)	HEIGHT (ft)	UNIT W (kcf)	W (k)	DIST. FROM REF. DATUM		W*X	W*Y
					X (FT)	Y (FT)		
Concrete FLOOR Level 1-6	12954	0.67	0.15	1301.88	24.1	7.2	31375.2	9373.51
Concrete FLOOR Level 7-8	5885	0.67	0.15	1182.89	114	30	134849	35486.6
Lower Roof - PH Roof	5885	0.67	0.15	591.443	114	30	67424.4	17743.3
SW1	11.5	9.3	0.15	16.0425	205.75	20	3300.74	320.85
SW2	22.58	9.3	0.15	31.4991	217	15.25	6835.3	480.361
SW3	11.5	9.3	0.15	16.0425	228.33	20	3662.98	320.85
SW4	22.58	9.3	0.15	31.4991	217	25.75	6835.3	811.102
SW5	17.5	9.3	0.15	24.4125	8.75	31.67	213.609	773.144
SW6	24.9	9.3	0.15	34.7355	13.5	13.75	468.929	477.613
SW7	24.9	9.3	0.15	34.7355	0.5	13.75	17.3678	477.613
SW8	9.5	9.3	0.15	13.2525	29.25	-0.5	387.636	-6.6263
SW9	23.5	9.3	0.15	32.7825	27.5	11.75	901.519	385.194
SW10	9.5	9.3	0.15	13.2525	29.25	22	387.636	291.555
			SUM (W)	686.526	SUM		54386.3	13705.2

Xcom = Sum(W*X)/ Sum(W)	79.2196 + 66 = 145.2 ft
Ycom = Sum(W*Y)/ Sum(W)	19.9631 + 32 = 51.9 ft

These values vary slightly from the RAM model's coordinates by about 5-7%. Fortunately the center of mass and center of rigidity are not too far apart, reducing a large amount of incidental torsion created by the loads.

The following tables, 6 and 7, are preliminary steps to determine the torsional shear and total shear that exists in each of the ten shear walls.

TABLE 6: Torsional Rigidity								
ELEMENT LABEL	Rx	Ry	dx	dy	Rx*dy	Ry*dx	Rx*dy^2	Ry*dx^2
SW1	0	2.92	168.12	0	0	490.9104	0	82531.86
SW2	21.2	0	0	-6.9	-146.28	0	1009.332	0
SW3	0	2.92	190.7	0	0	556.844	0	106190.2
SW4	21.2	0	0	3.6	76.32	0	274.752	0
SW5	9.5	0	0	9.52	90.44	0	860.9888	0
SW6	0	15	-24.1	0	0	-361.5	0	8712.15
SW7	0	15	-36.8	0	0	-552	0	20313.6
SW8	0.9	0	0	-22.65	-20.385	0	461.7203	0
SW9	0	12.7	-9.8	0	0	-124.46	0	1219.708
SW10	0.9	0	0	-0.15	-0.135	0	0.02025	0
						SUM	2606.813	218967.5
							221574.2785	

J = SUM (Ridi^2)	221574	(k/in)ft^2

TABLE 7: Direct Shear into Walls					
V = applied shear (kips)	653.1	103.63	Vd= (Ri/SUM(Ri))*V		Applied Direction
ELEMENT LABEL	Rx	Ry	DIRECT SHEAR (Vd)		
SW1	0	2.92	0.0	6.2	Y
SW2	21.2	0	257.8	0.0	X
SW3	0	2.92	0.0	6.2	Y
SW4	21.2	0	257.8	0.0	X
SW5	9.5	0	115.5	0.0	X
SW6	0	15	0.0	32.0	Y
SW7	0	15	0.0	32.0	Y
SW8	0.9	0	10.9	0.0	X
SW9	0	12.7	0.0	27.1	Y
SW10	0.9	0	10.9	0.0	X
Sum	53.7	48.54			

TABLE 8: Torsional Shear into Walls			
ELEMENT LABEL	Direction	Ridi	Vt
SW1	Y	490.9104	0.5051159
SW2	X	-146.28	17.936538
SW3	Y	556.844	0.5729575
SW4	X	76.32	9.3581937
SW5	X	90.44	11.089558
SW6	Y	-361.5	0.3719608
SW7	Y	-552	0.5679733
SW8	X	-20.385	2.4995647
SW9	Y	-124.46	0.1280615
SW10	X	-0.135	0.0001389

Table 8 provides the total shear due to torsion that is present in each of the listed shear walls. Load follows stiffness, so one will notice the stiffer elements receive more torsional shear

TABLE 9: Total Shear				
ELEMENT LABEL	Vd	Vt	Vi	Direction
SW1	6.234	0.505116	6.739141	Y
SW2	257.83	17.93654	275.7712	X
SW3	6.234	0.572957	6.806983	Y
SW4	257.83	9.358194	267.1928	X
SW5	115.54	11.08956	126.6287	X
SW6	32.024	0.371961	32.39606	Y
SW7	32.024	0.567973	32.59208	Y
SW8	10.946	2.499565	13.44537	X
SW9	27.114	0.128062	27.2418	Y
SW10	10.946	0.000139	10.94595	X

Table 9 provides the total shear in each shear wall. This is a combination of the direct shear and torsional shear calculated in the previous tables. In the following calculations, the total wind load (calculated in notebook submission A) is applied to each wall. This is to be conservative when designing and analyzing a lateral force resisting system.

11 Shear Wall Strength Checks

Mark Bland	Shear Wall Checks	Notebook Submission C

Shear Wall Strength Checks

* Wind controls over seismic loads
  ↳ Controlling load Case: (ASCE 7-10, 2.3.2)
  (4) $1.2\underline{D} + 1.0\underline{W} + \underline{L} + 0.5\underline{S}$

* Evaluate a horizontal-shear wall 5 and vertical-shear wall 1

* Analyze at the base

SW1		SW5	
location →	1.N-23.N	location →	J.N-4.N
Height →	12'-0"	Height →	12'-0"
length →	11'-6"	length →	17'-6"
thickness →	12"	thickness →	12"

Shear Wall #1

Dead load calculation:

$$\text{Total } P = \overset{\text{floors}}{8}\left[\overset{\text{Floor DL}}{(108\,\text{psf})}\overset{\text{Trib area}}{(204\,\text{ft}^2)} + 12'(11.5')\left(\tfrac{12}{12}\right)(150\,\text{lb/cf})\right]/1000$$

$$P = 346.18\text{K}$$

Wind load calculation:

$$W = 653.1\text{ K}$$

↳ refer to Notebook Submission A for Base shear due to wind

Live load calculations:

$$\text{Total } L = 8\left[55psf\,(209ft^2)/1000\right]$$

$$L = 92.0K$$

Snow load calculations:

$$\text{Total } S = (15.75psf)(209ft^2)/1000$$

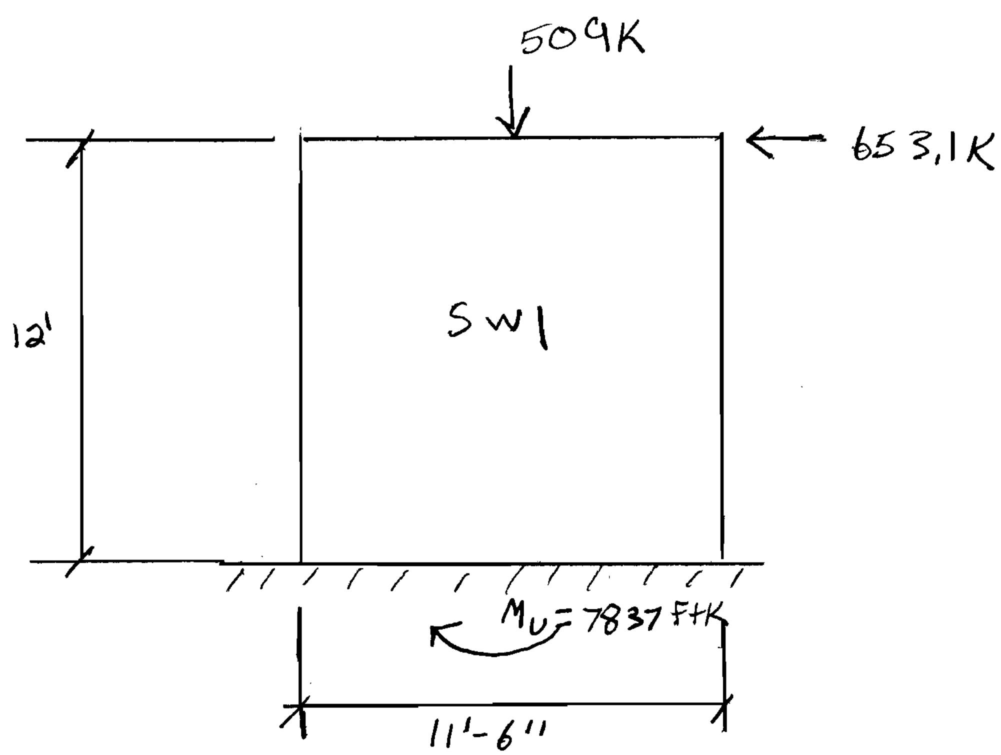
↳ refer to Notebook Submission A for snow load

$$S = 3.29K$$

Final loading:

$$P_U = 1.2(346.18K) + 92.0K + 0.5(3.29K) = 509K$$

$$V_U = 1.0(653.1K) = 653.1K$$



$\phi V_n \geq V_u = 653.1K \qquad V_n = V_c + V_s$

Determine V_c (ACI 11.9)

Properties:
$\lambda = 1.0$
$f'_c = 4000\,psi$
$h = 12''$
$d = 0.8 l_w = 0.8(11.5)\times 12 = 110.4''$
$N_u = 7837\,ftK$

$$V_c = 2\lambda\sqrt{f'_c}\,hd = 2(1.0)\sqrt{4000}(12)(110.4)\frac{1}{1000} = 167.6K$$

$$V_c = 3.3\lambda\sqrt{f'_c}\,hd + \frac{N_u d}{4 l_w}$$
$$= \left[3.3(1.0)\sqrt{4000}(12)(110.4) + \frac{7837(110.4)}{4\cdot 138}\right]\left(\frac{1}{1000}\right)$$
$$= 278.1K$$

$$V_c = \left[0.6\lambda\sqrt{f'_c} + \frac{l_w\left(1.25\lambda\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right]hd$$

$$= \left[0.6\sqrt{4000} + \frac{138\left(1.25\sqrt{4000} + 0.2\frac{7837}{(138\times 12)}\right)}{\frac{12\times 12 V_u}{V_u} - \frac{138}{2}}\right]\left(\frac{12\times 110.4}{1000}\right)$$

$$= \left[37.9 + \frac{11040.5}{75}\right]\times 1.325$$

$$= 245.3K$$

$$V_c = \min\begin{cases} 278.1K \\ 245.3K \end{cases} \rightarrow V_c = 245.3K > 167.6K$$

$\therefore$ use $V_c = 245.3K$

$\phi V_c = 0.75(245.3K) < 653.1K$

∴ Wall w/o reinforcement is no good... Find Vs

Determine Vs

$V_s = \frac{A_v f_y d}{s}$

2 rows, each face
reinforcement: #5@12" each way
$F_y = 60$ Ksi
$d = 110.4"$
$s = 12"$

$A_v = 2(2)(.31) = 1.24 in^2/12 in$

$V_s = \frac{1.24 in^2 (60 Ksi)(110.4)}{12"}$

$= 684.5K$

$V_n = V_c + V_s$

$= 245.3 + 684.5 = 929.8K$

$\phi V_n = 0.75(929.8) = 697.35K > V_u = 653.1K$ ✓

∴ SW1 strength is good

Shear Wall #5

Dead load Calculations:

Total $P = 8\left[(108 psf)(164 ft^2) + 12'(17.5')\left(\frac{12}{12}\right)(150)\right]/1000$

$P = 393.7K$

Wind load Calculations:

$W = 653.1K$

Live load Calculations:

Total $L = 8\left[55 psf(164 ft^2)/1000\right]$

$L = 72.2K$

Snow load calculations:

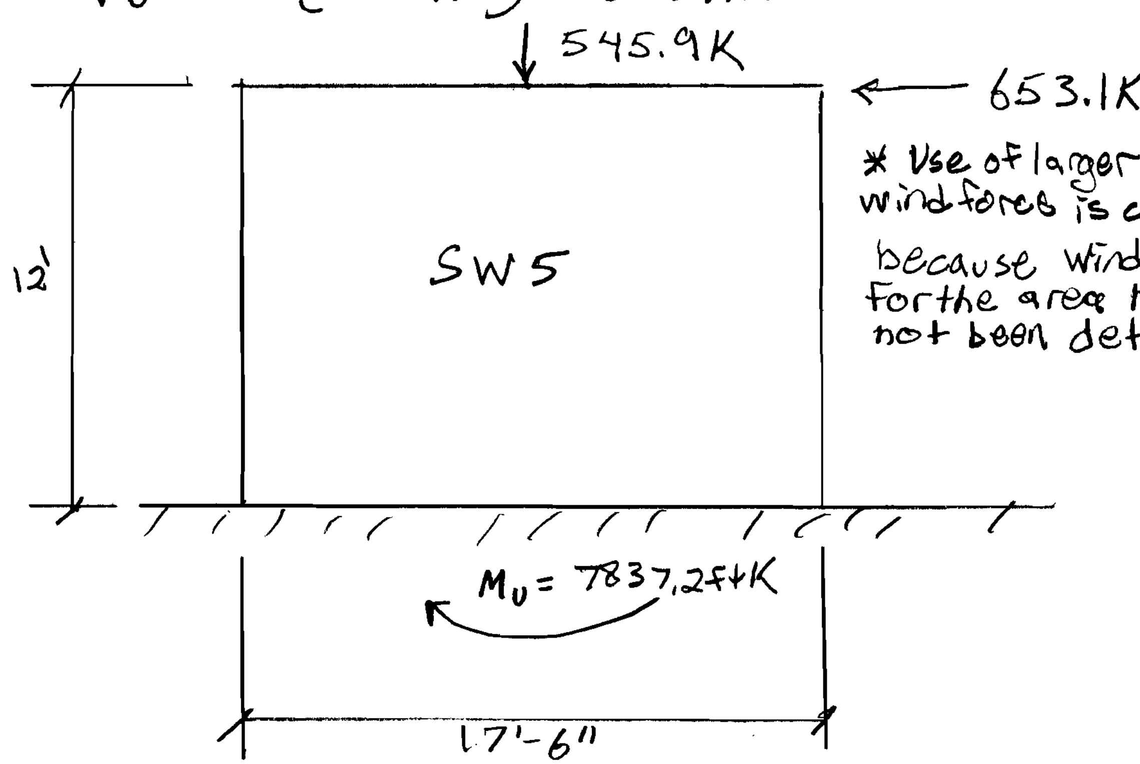
Total $S = (15.75 psf)(164 ft^2)/1000$

$S = 2.6K$

Final loading:

$P_u = 1.2(393.7K) + 72.2K + 0.5(2.6) = 545.9K$

$V_u = 1.0(653.1K) = 653.1K$



* Use of larger lateral wind forces is conservative because wind directions for the area have not been determined

Mark Bland	shear wall checks	Notebook Submission C

$\phi V_n \geq V_u = 653.1\,K \qquad V_n = V_c + V_s$

Determine V_c

Properties: $\lambda = 1.0$
$f'c = 4000$ psi
$h = 12''$
$d = 0.8\,l_w = 0.8(17.5)\times 12 = 168''$
$N_u = 7837.2$ ft K

$V_c = 2\sqrt{4000}\,(12)(168)\frac{1}{1000}$
$= 255.0\,K$

$V_c = \left[3.3\sqrt{4000}\,(12)(168) + \frac{7837.2(168)}{4\,(210)}\right] / 1000$
$= 422.3K$

$V_c = \left[0.6\sqrt{4000} + \frac{210\left(1.25\sqrt{4000} + 0.2\frac{7837}{(210\times 12)}\right)}{39}\right] \frac{(12\times 168)}{1000}$

$= \left[37.9 + 429\right] \times 2.016 = 941$

$\phi V_c < V_u = 653 \quad \therefore$ Find V_s

* reinforcement has the same properties as SW1

$V_s = \frac{1.24in^2\,(60Ksi)(168)}{12''}$

$= 1041.6K$

$V_n = 422.3 + 1041.6 = 1464$

$\phi V_n = 0.75(1464) = 1098K > V_u = 653.1\,K$ ✓

$\therefore$ SW5 strength is good

12 Wind and Seismic Forces

When designing for wind, all 4 load cases from ASCE should be considered. RAM accounts for these cases with automatically generated loads. As previously mentioned, the wind forces are applied at the center of pressure and the seismic forces are applied at the center of mass. As expected, the shear values increase as the building height decreases.

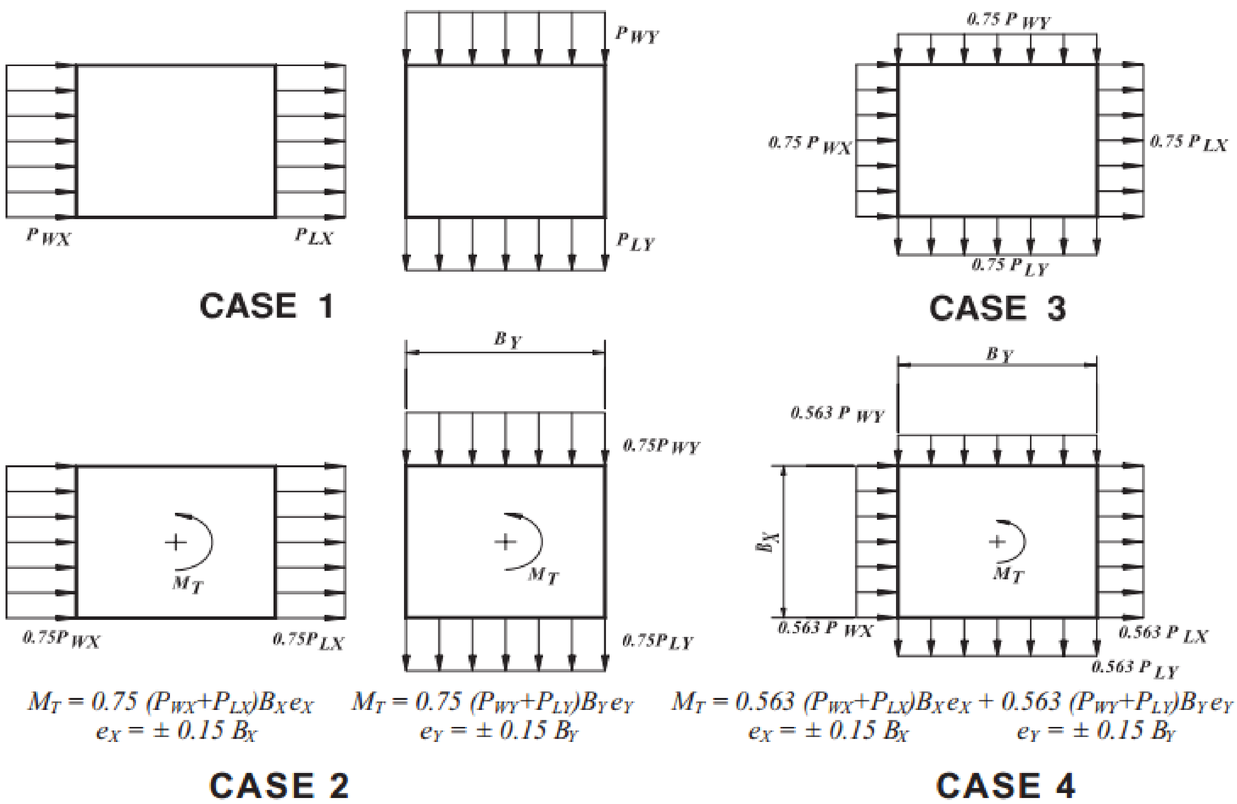


Figure 9: Design wind load cases from ASCE7-10 27.4-8

In the following section, member forces and drifts are provided from both the RAM and SAP models.

SAP 2000

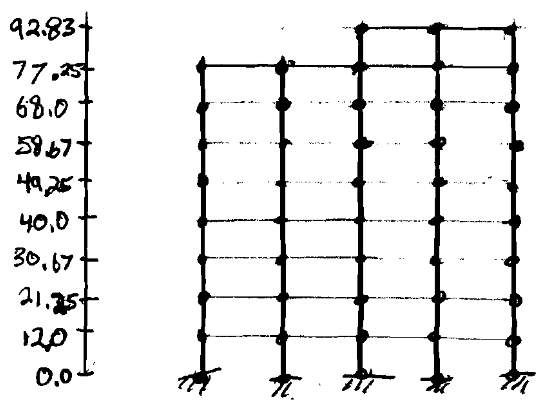
N-S' Shear Walls

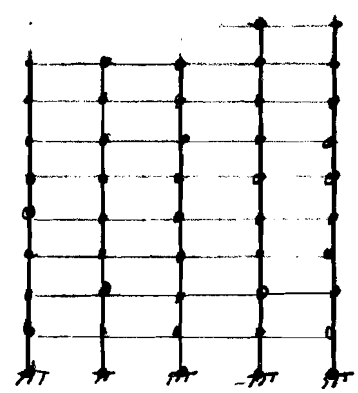
t=12"

	SW1	SW3	SW6	SW7	SW9
L=	17.5'	11.5'	24.9'	24.9'	23.5'
H=	77.3'	77.3'	96'	96'	96'

E-W Shear Walls

	SW2	SW4	SW5	SW8	SW10
L=	22.5'	22.5'	17.5'	9.5'	23.5'
H=	77.3'	77.3'	73.8'	96'	96'





Using calculated loads from Submission A

LEVEL	WIND (Distributed load)		SEISMIC (joint load)
2	21.8psf x ~300ft	= 6.552 lb/ft	20.98K
3	23.0psf	= 6.889 lb/ft	22.67K
4	24.3psf	= 7.277 lb/ft	22.19K
5	25.3psf	= 7.586 lb/ft	22.20K
6	26.2psf	= 7.847 lb/ft	22.18K
7	26.9 psf	= 8.076 lb/ft	22.19K
8	27.6 psf	= 8.279 lb/ft	17.13K
roof	28.3psf	= 8.491 lb/ft	17.34K
PH roof	28.7psf	= 8.632 lb/ft	9.37K

13 Member Checks and Model Output

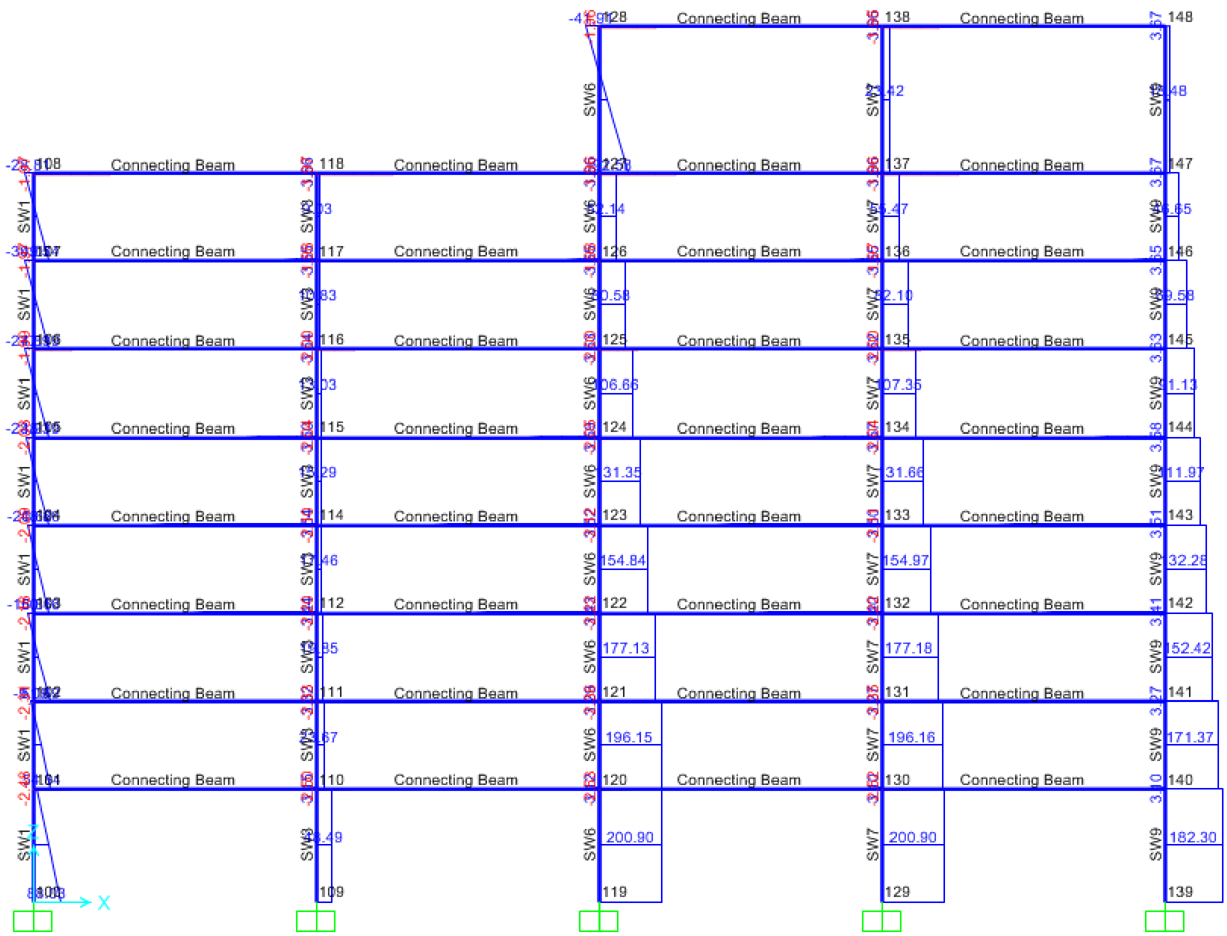


Figure 10: Shear values for the N-S shear walls due to wind

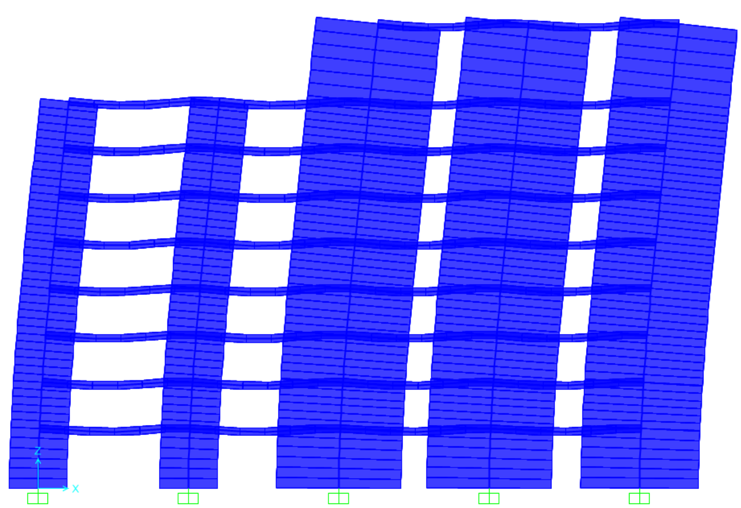


Figure 11: Deflected shape due to wind

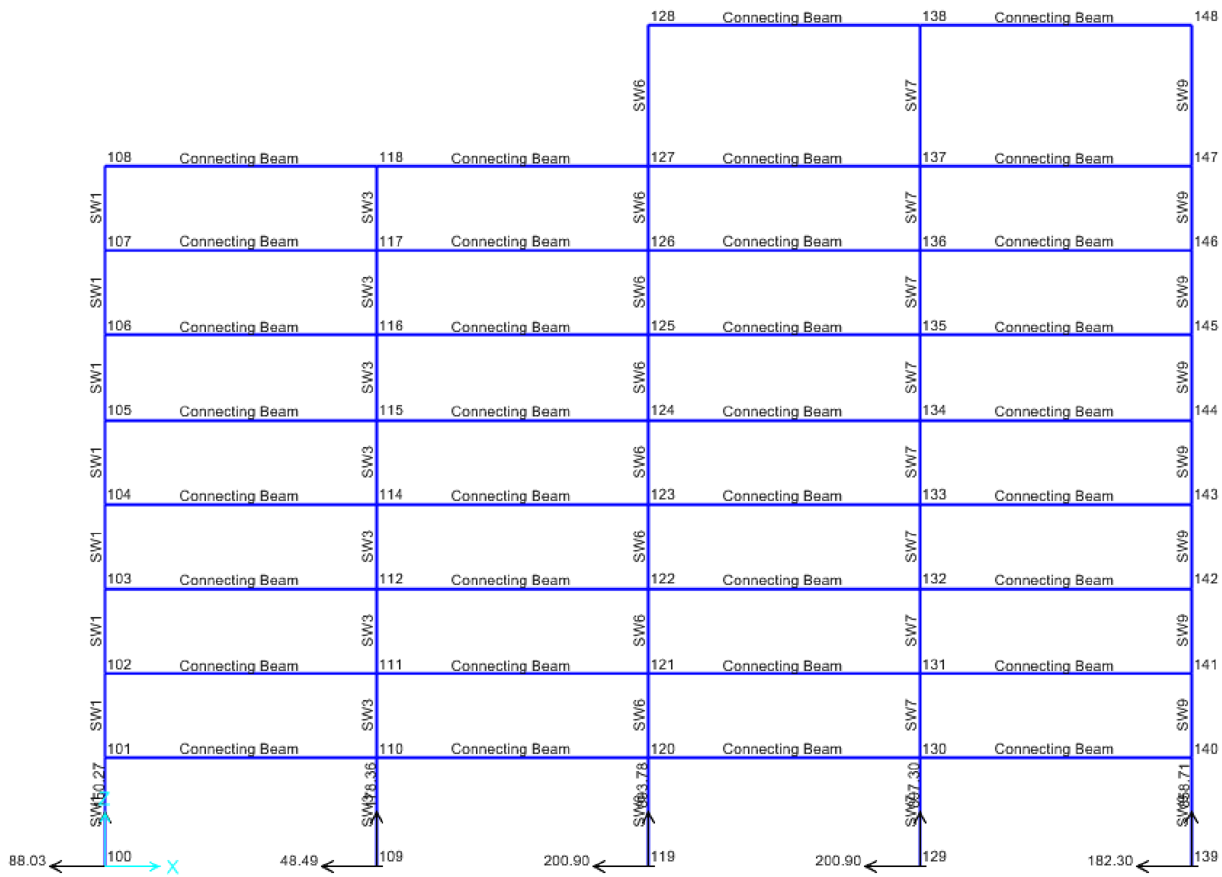


Figure 12: Base shear reactions for N-S walls due to wind case.

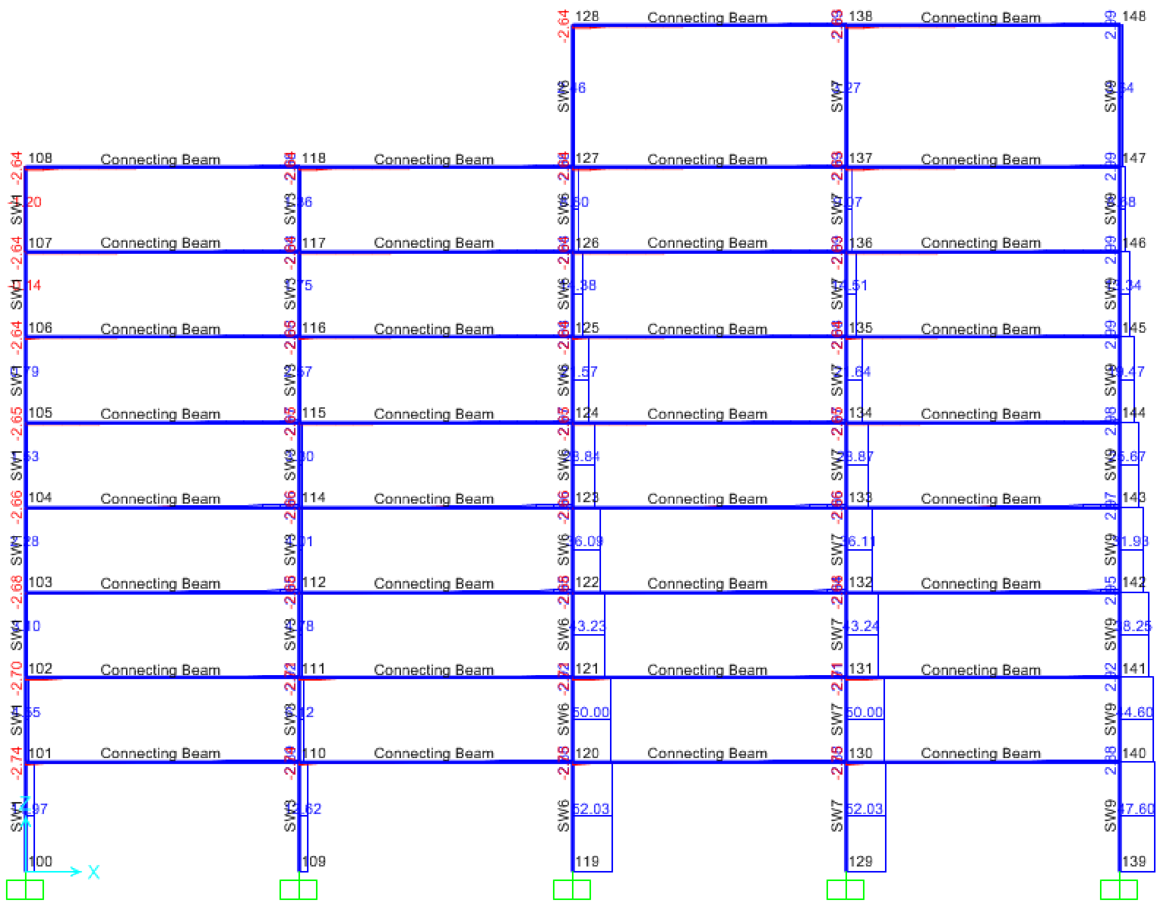


Figure 13: Shear values for the N-S walls due to the non-controlling seismic forces.

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W11	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Roof	D	-0.0073	-0.0106	-0.0014	-0.0024	0.0000	0.0000
	Lp	0.0009	-0.0048	-0.0000	-0.0009	0.0000	0.0000
	W1	0.2298	0.0066	0.0403	-0.0012	0.0003	0.0000
	W2	-0.0956	$\boxed{1.1510}$	-0.0170	0.1895	0.0001	0.0015
	W3	0.1825	-0.0338	0.0319	-0.0070	0.0002	0.0001
	W4	0.1621	0.0436	0.0285	0.0052	0.0002	0.0000
	W5	-0.2135	1.4013	-0.0371	0.2284	0.0003	0.0018
	W6	0.0700	0.3252	0.0116	0.0558	0.0001	0.0004
	W7	0.1006	0.8682	0.0175	0.1412	0.0001	0.0011
	W8	0.2441	-0.8583	0.0429	-0.1430	0.0003	0.0011
	W9	0.1894	0.2185	0.0327	0.0366	0.0003	0.0003
	W10	-0.0385	1.0837	-0.0065	0.1752	0.0000	0.0013
	W11	0.2970	-1.0763	0.0518	-0.1766	0.0004	0.0014
	W12	0.0691	-0.2112	0.0126	-0.0380	0.0001	0.0003
	E1	0.2735	-0.0272	0.0466	-0.0067	0.0004	0.0001
	E2	0.2612	0.0194	0.0445	0.0005	0.0003	0.0000
	E3	-0.0417	0.3899	-0.0070	0.0600	0.0001	0.0005
	E4	-0.0019	0.2390	-0.0003	0.0368	0.0000	0.0003

Figure 14: RAM output of drift at the far corner of the building are provided above. This is where the most drift is expected to take place. The highlighted number is less than the maximum allowable drift that has been calculated by hand.

Level: Roof, Diaph: 1

Center of Mass (ft): (142.50, 48.20)

LdC	Disp X	Disp Y	Theta Z
	in	in	rad
D	0.00510	-0.05390	0.00003
Lp	0.00636	-0.02395	0.00001
W1	0.22982	0.00640	0.00000
W2	0.01774	$\boxed{0.75377}$	0.00024
W3	0.17294	-0.00020	-0.00002
W4	0.17178	0.00980	0.00002
W5	0.00516	0.63519	0.00046
W6	0.02145	0.49546	-0.00010
W7	0.18567	0.57013	0.00018
W8	0.15906	-0.56053	-0.00018
W9	0.14579	0.37145	-0.00009
W10	0.13271	0.48374	0.00036
W11	0.12584	-0.47654	-0.00036
W12	0.11275	-0.36425	0.00009
E1	0.26482	0.00310	-0.00002
E2	0.26413	0.00905	0.00001
E3	0.00466	0.22748	0.00010
E4	0.00687	0.20816	0.00002

Figure 15: RAM output of story displacement is provided in this figure. The maximum story displacement is in the Y direction which makes sense because it is the weak axis of the rectangular building.

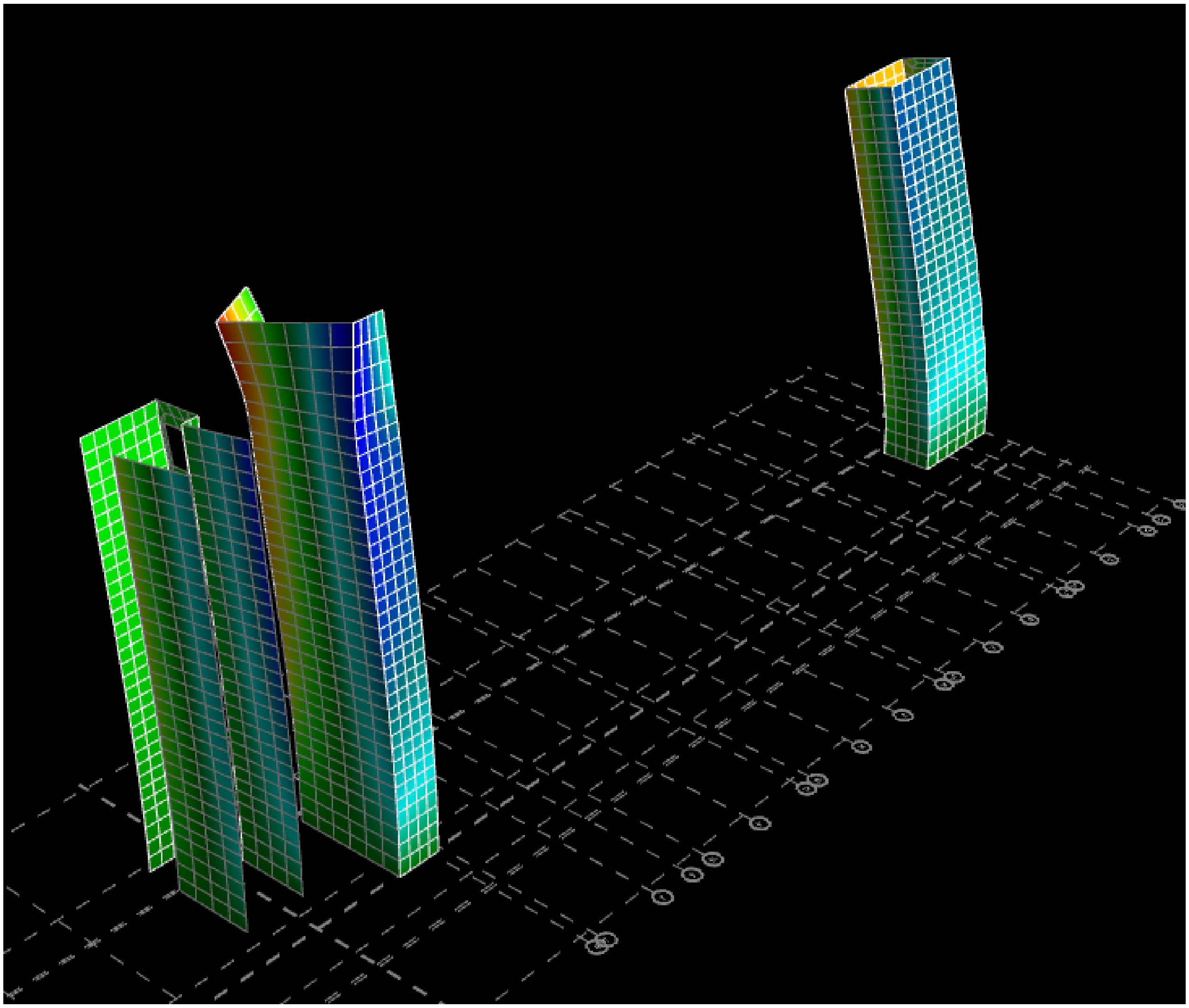


Figure 16: Deflected shape of shear walls due to wind from the RAM model.

Limit checks

h (height to Penthouse Roof) = 96'

Wind (controlling)

$$\frac{h}{400} > \delta_{max}$$

$$\frac{96(12)}{400}\ 2.88'' > 1.15'' \rightarrow \text{Max Roof displacement} \checkmark$$

14 Conclusion

This report as well as the complete lateral system analysis of West Village Housing, has determined that the lateral system is sufficiently designed to resist the lateral loading conditions from wind and seismic loads. In addition to the results from notebook submission B, it can be said that West Village Housing's north building is adequately designed for both gravity and lateral loads. 2D and 3D structural models were utilized to determine if the system was sufficiently designed. Comparing computer models with hand calculations and engineering judgement allows one to fully complete a structural lateral system study.