Letter of Transmittal

Date: October 16, 2015

To: Dr. Aly Said
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From: Mark Bland
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Dear Dr. Said,

The enclosed documents include my Structural Notebook Submission B 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 structural analysis including typical member spot checks for the gravity loads previously determined in notebook submission A. In addition, 3 new design alternatives for the gravity system are analyzed.

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

Sincerely,

Mark Bland

Typical Member Spot Checks & Alternate Systems Design Study

Structural Notebook Submission B

West Village Housing Phases III & IV

Towson, Maryland

Mark Bland [Structural Option]

Advisor: Dr. Aly Said

October 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

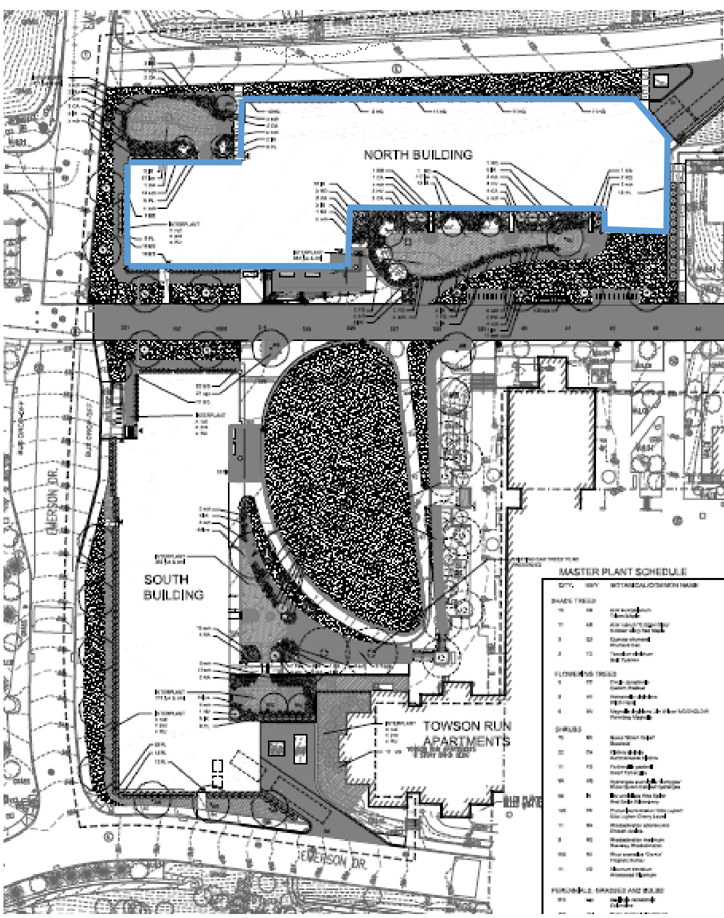


Figure 1: Site Landscaping

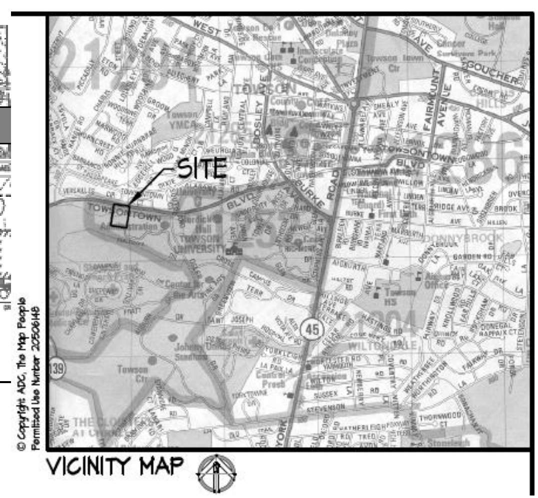


Figure 2: Vicinity Map

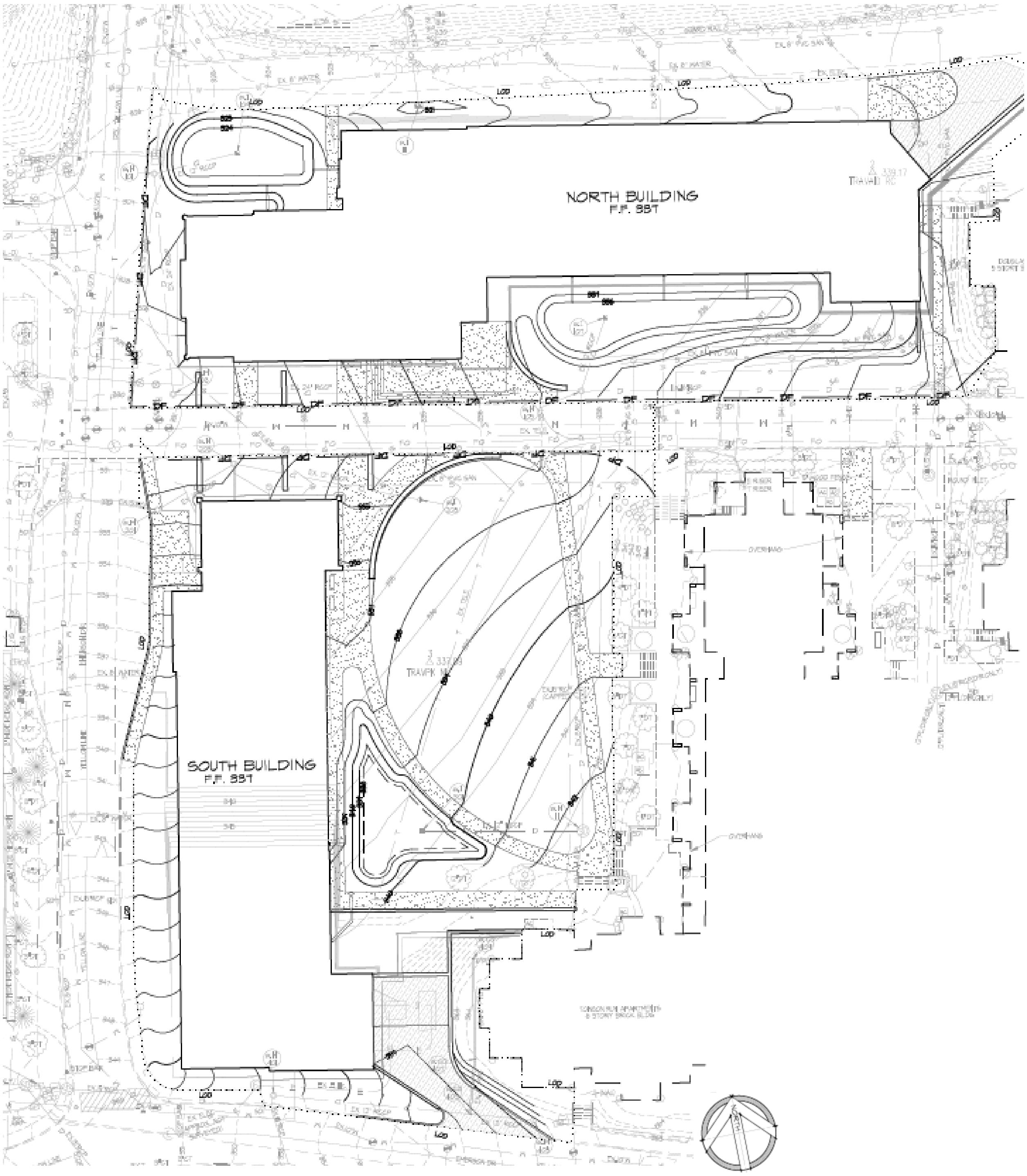


Figure 1: General Grading Plan

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.

International Code Council

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

American Society of Civil Engineers

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

West Village Housing Phases III & IV

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

Previous Course and Internship Notes/Resources

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 inorder 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.

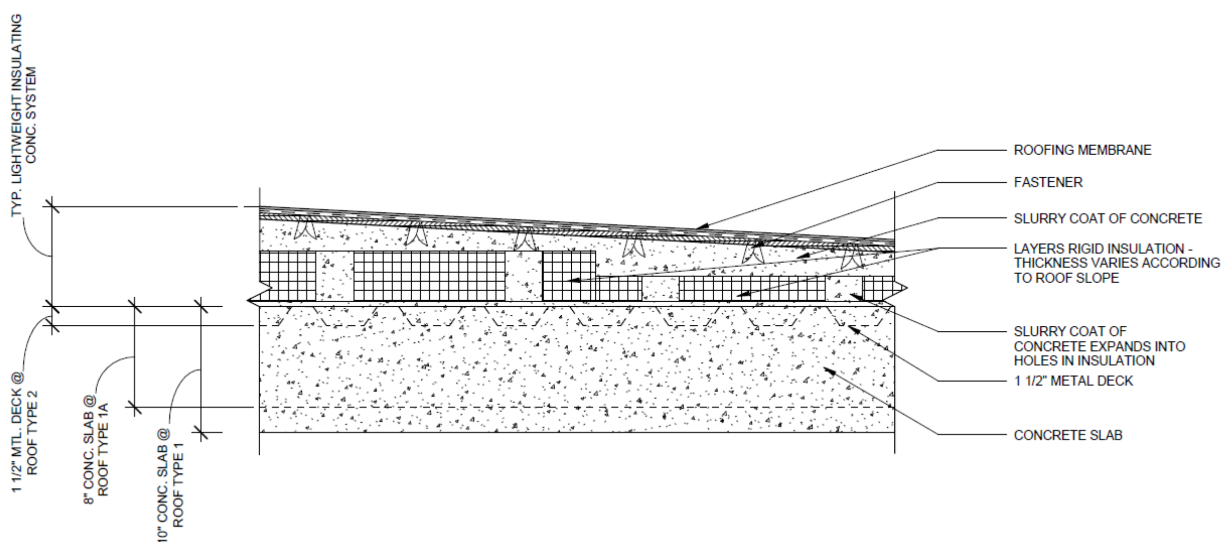


Figure 2: Typical roof bay cross section

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
- $1\frac{1}{2}$" 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:

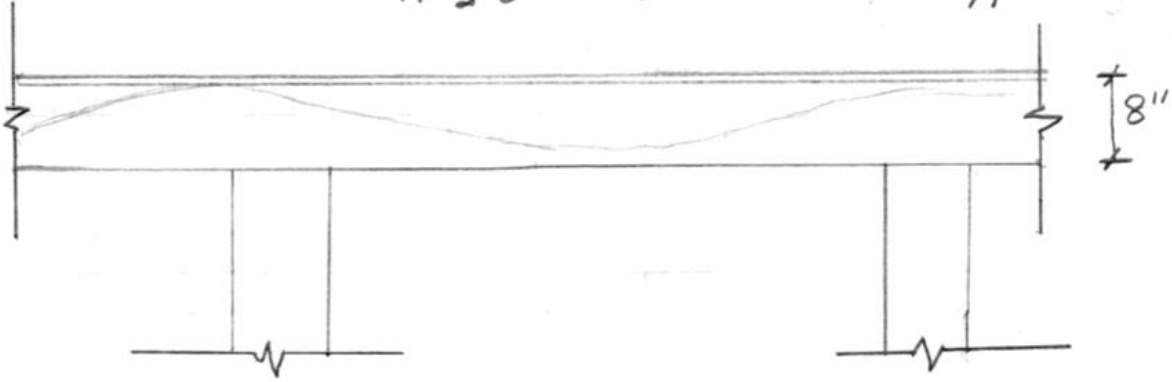
- 20 PSF per ASCE 7-10 table 4-1

* 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

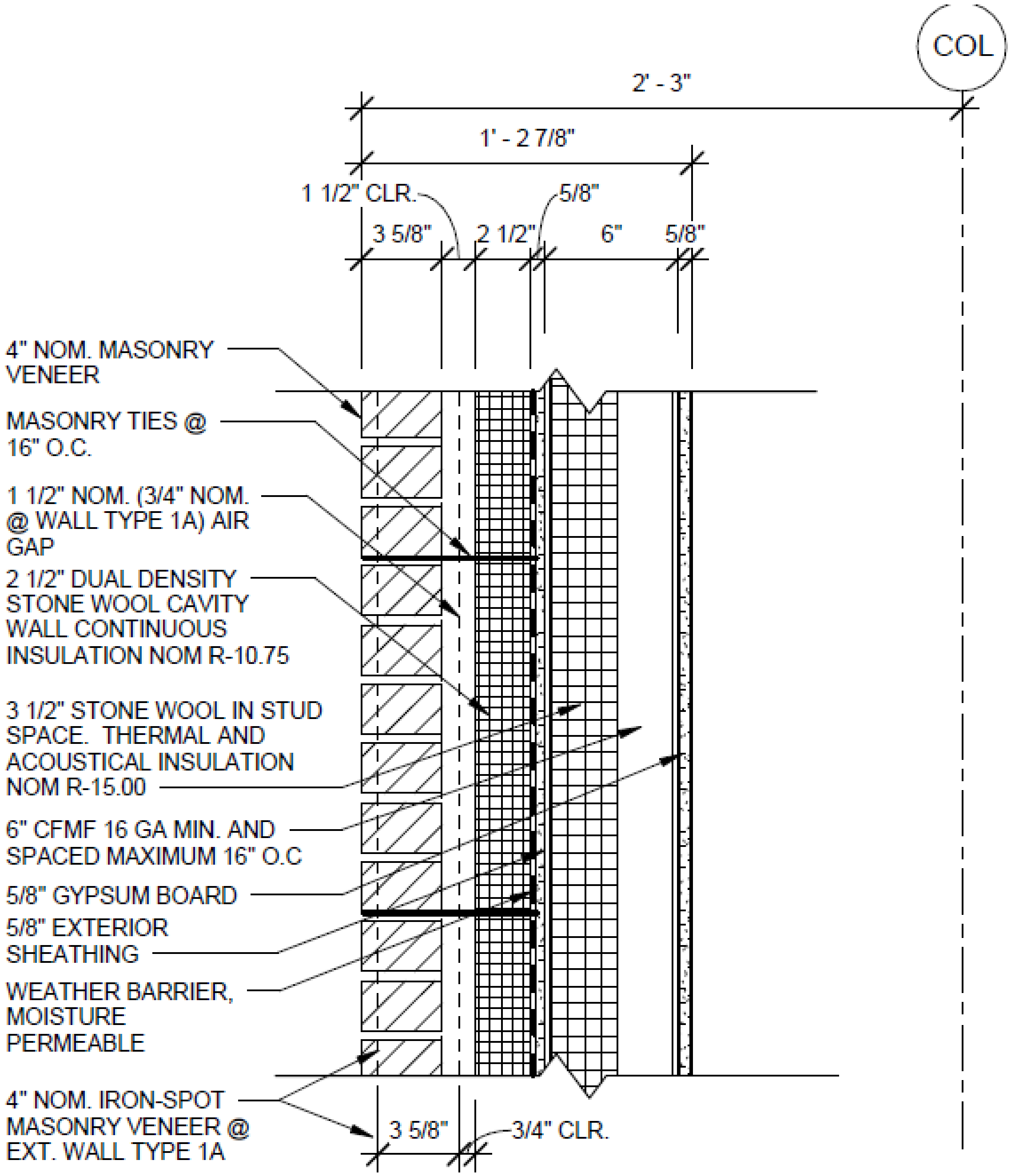


Figure 3: Typical Exterior Wall Detail

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

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

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

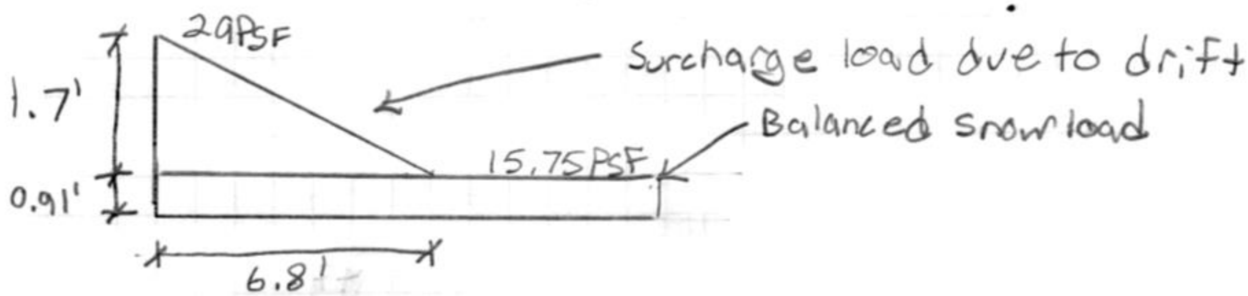
$h_c = 11' - 1' = 10'$ $h_c/h_b > 0.2 \rightarrow$ drift load must be calculated

$l_u = 28.3'$

Leeward:
$h_d = 1.7$ (Figure 7-9)
$h_d < h_c$, $\therefore w = 4h_d = 6.8 ft$

Windward:
$h_d = 3/4(1.7) = 1.3 ft < 1.7 ft$ $\therefore$ 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

   $K_{zt} = 1.0$

   $K_d = 0.85$

   $V = 115$mph

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

   * 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: $\frac{L}{B} = \frac{61'}{300'} = 0.09$

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

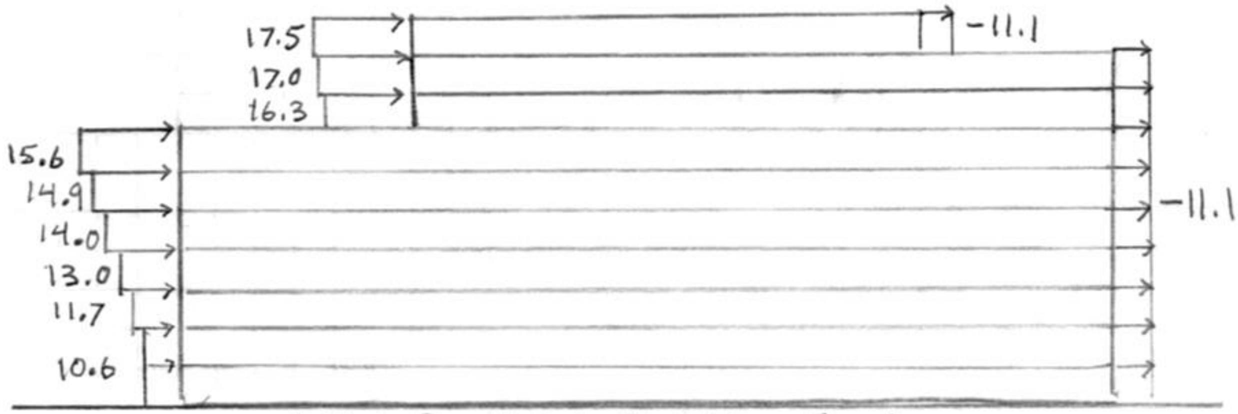
$p = q G C_p - q_i (G C_{pi})$ (27.4-1)

* 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**

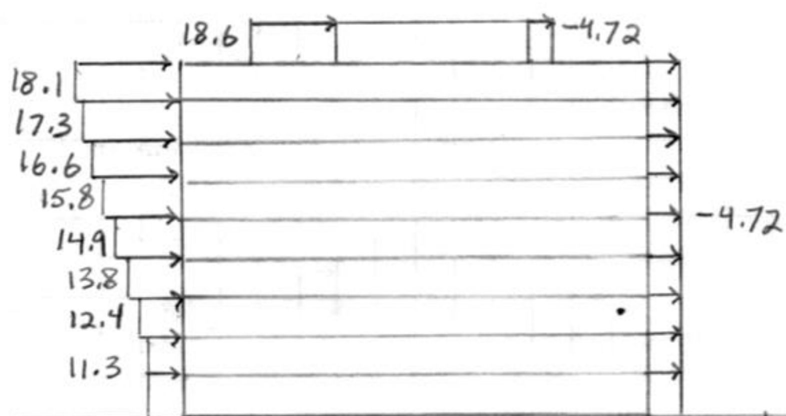
Table 4: Base Shear Determination N-S				
Building Level	**Height above ground level z (ft)**	**Tributary Height (ft)**	**Total Pressure (psf)**	**Total Lateral Story Force (kip)**
Level 1	0.0	6.00	15.97	**5.84**
Level 2	12.0	10.65	15.97	**10.37**
Level 3	21.3	9.34	17.15	**9.77**
Level 4	30.7	9.35	18.52	**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:

$F_a = 1.2 \qquad F_v = 1.7$

$S_{MS} = F_a S_s = 1.2(0.175) = 0.21$

$S_{M1} = F_v S_1 = 1.7(0.051) = 0.087$

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)

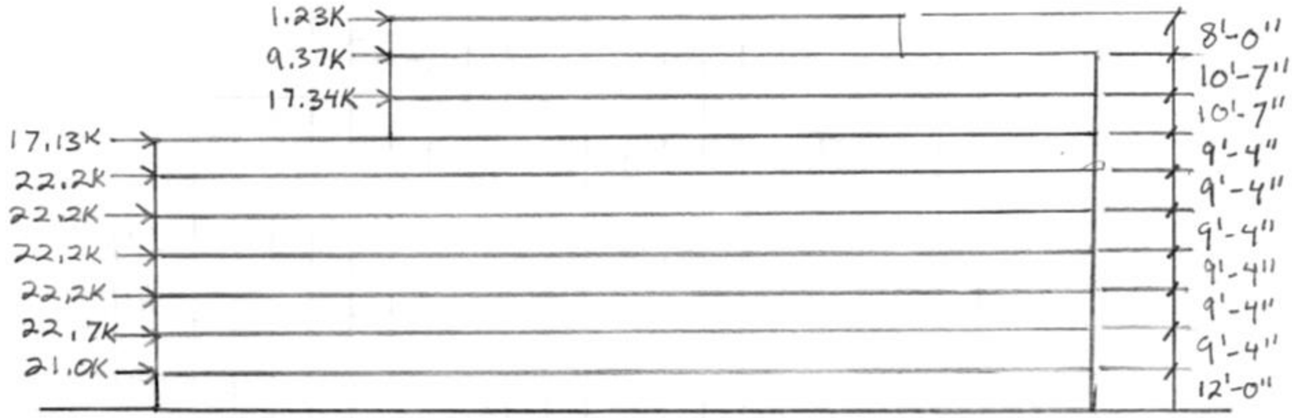
W_x = total dead load per story

* see spreadsheet for floor weights and story forces.

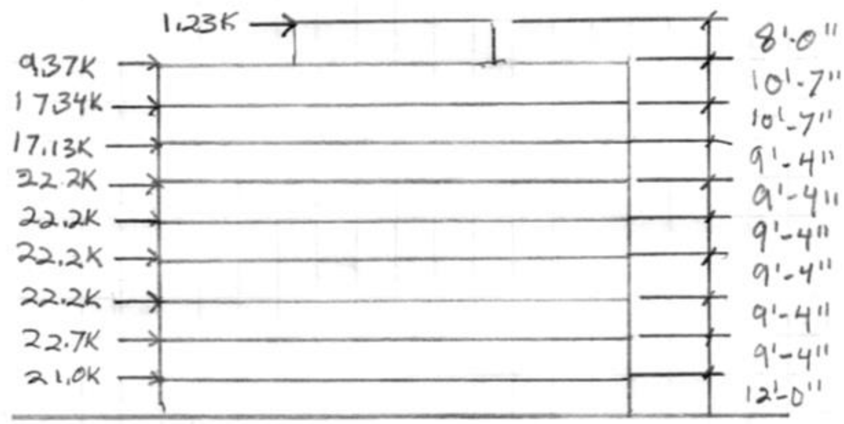
Table 5: Seismic Load Determination									
Building Level	**Height above ground level z (ft)**	**Tributary Height (ft)**	**Total Floor Dead Load (psf)**	**Total Floor Area (s.f.)**	**Total Exterior Wall Load (psf)**	**Permiter (ft)**	**Total Story Weight W (kip)**		
Level 1	0.0	6.00	108	17400	52	700	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

Seismic Load vs Story Height



Base shear = 177.5 Kips

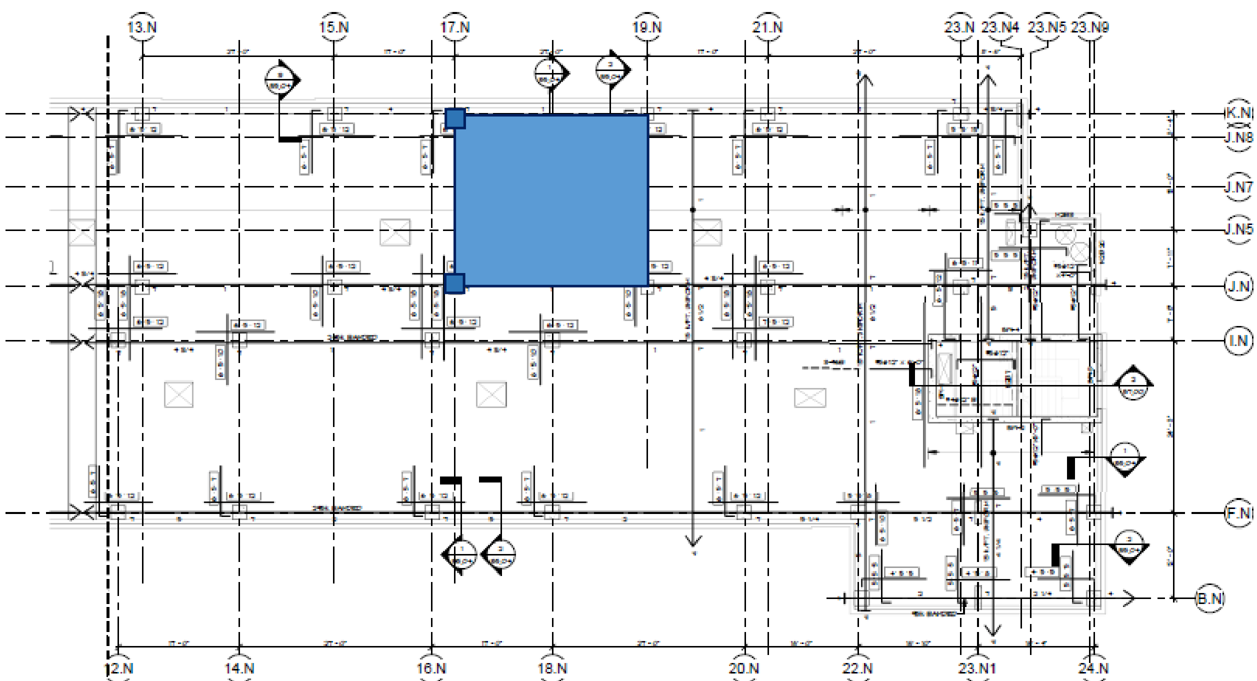


Base shear = 177.5 Kips

* 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.

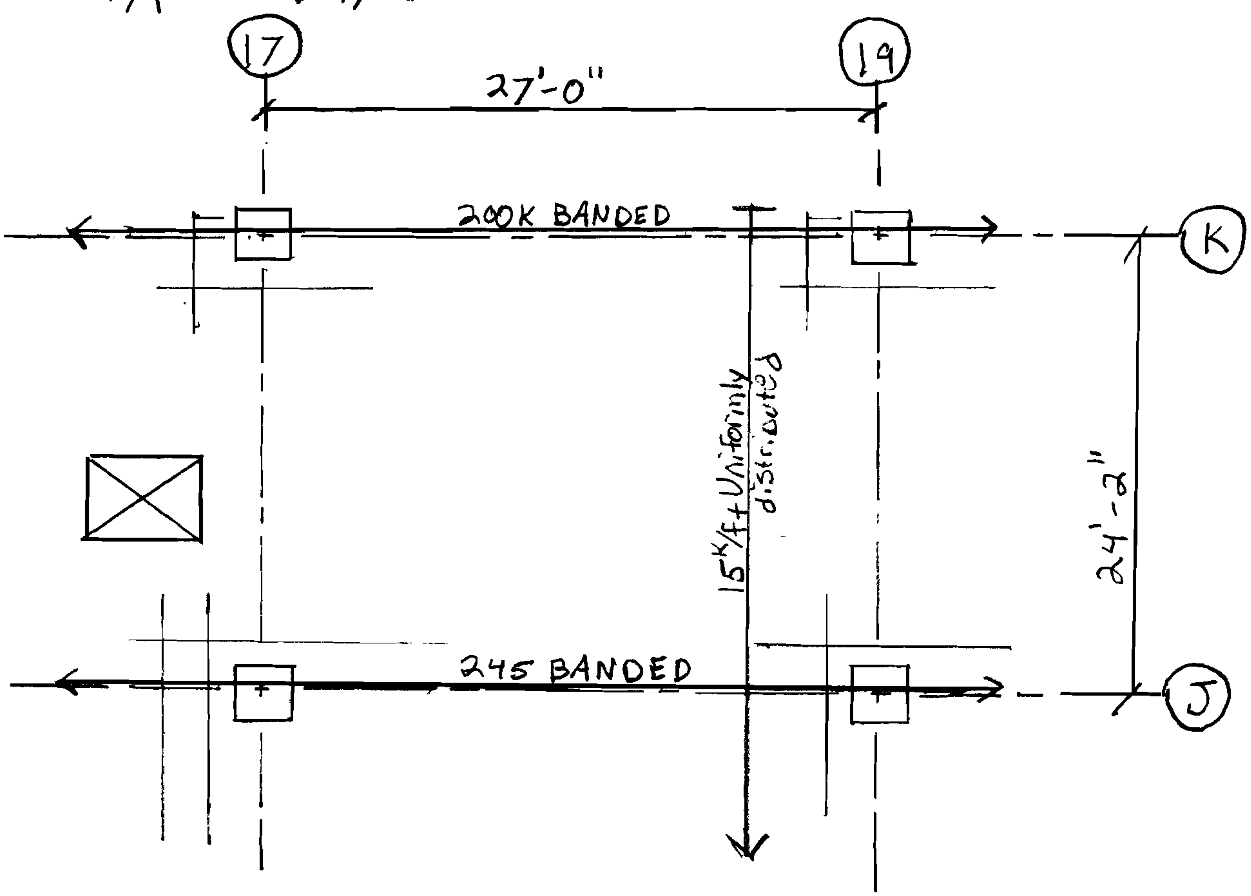
Bay Under Analysis

- 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



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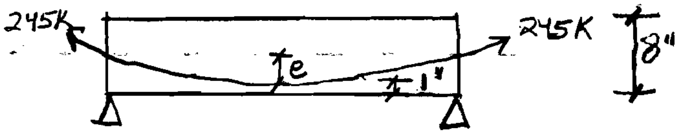
Post Tensioning Balancing load



Upward Force/Moment from tendons:

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

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

* accounts for 1" tendon cover noted on plans

$$w_p = \frac{8(245k)(3'')(\frac{1}{12''})}{(27')^2}$$

$= 0.675\ K/ft$

Banded tendons → 0.675 K/ft ↑

Uniformly Distributed tendons → 1.25 K/ft ↑

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

$= 1.251$

Load on slab

LL = 55 psf

$DL_W = 150\ pcf\ (\frac{10''}{12''}) = 125\ psf$

Misc DL = 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 63 psf 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 $\quad M_o = \frac{q_u \, l_2 \, l_n^2}{8}$

slab Moment Determination

Long Direction: $M_o = [63(24)(27 - \frac{29}{2})^2]/8 = 42.5$ ftk

Short Direction: $M_o = [63(27)(24 - \frac{29}{2})^2]/8 = 30.6$ ftk

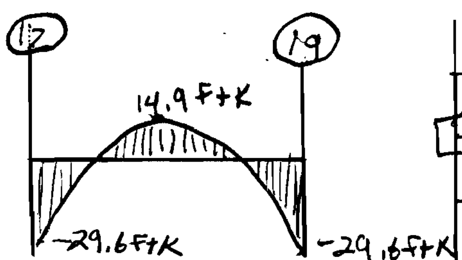
Coefficients for Factored moments

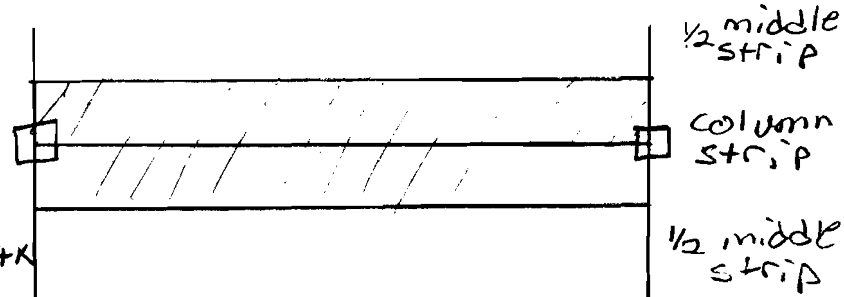
interior span:
- at Midspan → $0.35 M_o$
- at Column → $0.65 M_o$

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)

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

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)

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

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

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

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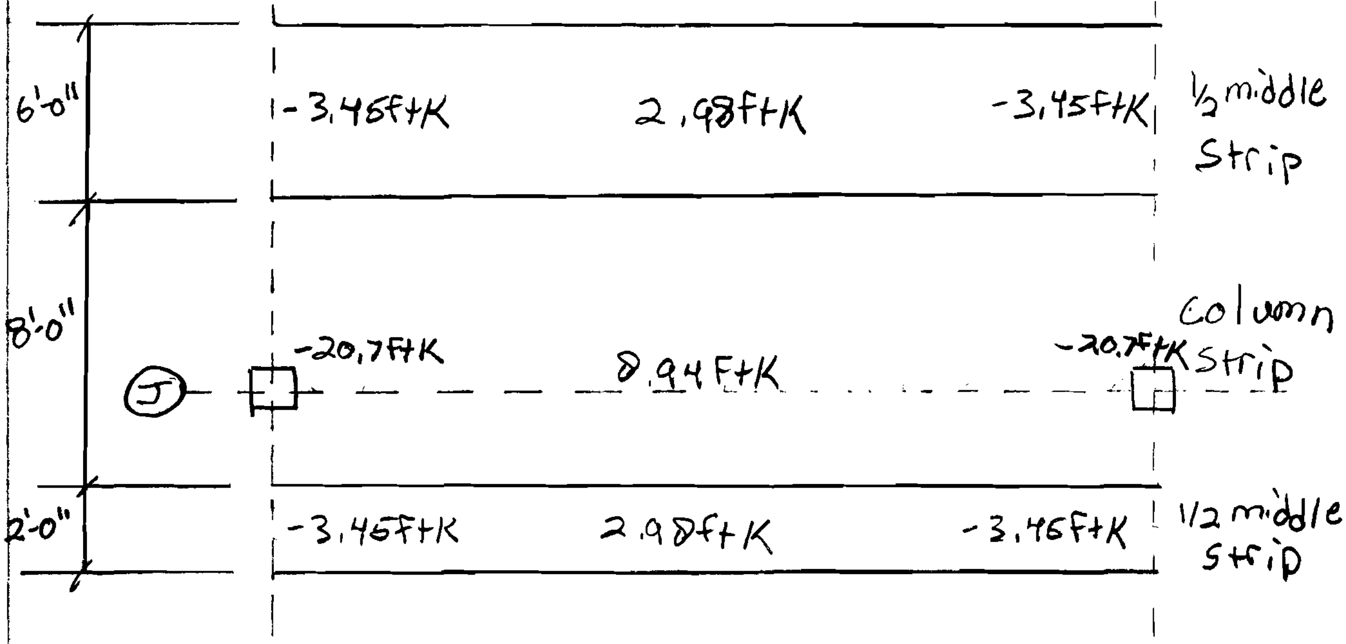


Check Reinforcement for Bending

Column Strip $M+ = 8.94\,ftK$

$As = (6)\#5 @ 12"$ Top bars

$\#4 @ 24"$ bottom bars

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

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

$= 6.44"$

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

$$\phi Mn = 0.9(2.66)\left(6.44 - \frac{0.093}{2}\right)(60000)\frac{1}{12}\frac{1}{1000}$$

$= 76.5\,ftK > 8.94\,ftK$ ✓

* reinforcing stays the same throughout the bay

Middle strip $M+ = 5.96\,ftK$

$\phi Mn = 76.5\,ftK > M+ = 5.96\,ftK$ ✓

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

$\phi Mn = 76.5\,ftK > M- = -20.7\,ftK$ ✓

<u>Check Shear Capacity</u> (two-way)

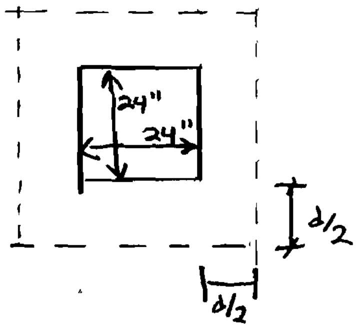
Column J17 $q_u = 63\,psf$

$d = 6.44''$

$d/2 = 3.22''$

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

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



$V_{c\,min} = \left|\begin{array}{l} 2 + 4/\beta = 6'' \\ 2 \times \frac{\alpha_s d}{b_o} = 4.73'' \\ 4'' \leftarrow \text{controls} \end{array}\right.$

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

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

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

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

$V_u = 0.25\left[22\times16 - \frac{(30.44\times30.44)}{144}\right] = 86.4\,K$

$\phi V_c = 133K > 86.4K$ ✓

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(one-way shear)

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

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

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

<u>Column Gravity check</u>

- 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"

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Exterior Column K17

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

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

~~DL + LL + Lr~~

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

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

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

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

$= 531.8K$

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

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

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

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

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

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

$P_u = 865K \leq 1311.7K$ ✓

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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, $F_y = 60000\,psi$

$\ell n/33 = 27'/33 = 0.82' = 9.8''$ use 10" slab

Loading on slab

$LL = 55\,psf$

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

Misc, $DL = 10\,psf$

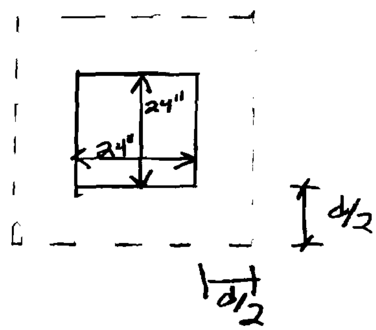
$q_u = 1.2(125+10) + 1.6(55)$
$= 250\,psf$

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

Check shear capacity (Two-Way)

Column J17 $\quad q_u = 250\text{ psf}$

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



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

$d = 8.3125"$

$d/2 = 4.156"$

$b_o = 4(24 + 4.156) = 112.6\text{ in}^2$

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

$$V_{c\,min} = \begin{cases} 2 + 4/\beta = 6" \\ 2 + \frac{\alpha_s d}{b_o} = 5.91" \\ 4" \leftarrow \text{controls} \end{cases}$$

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

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

$\phi V_c = 0.75(237K) = 177.78\text{ K}$

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

$\phi V_c = 177.78 > V_u = 85.8K$ ✓

- slab passes for punching shear

(one-way shear)

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

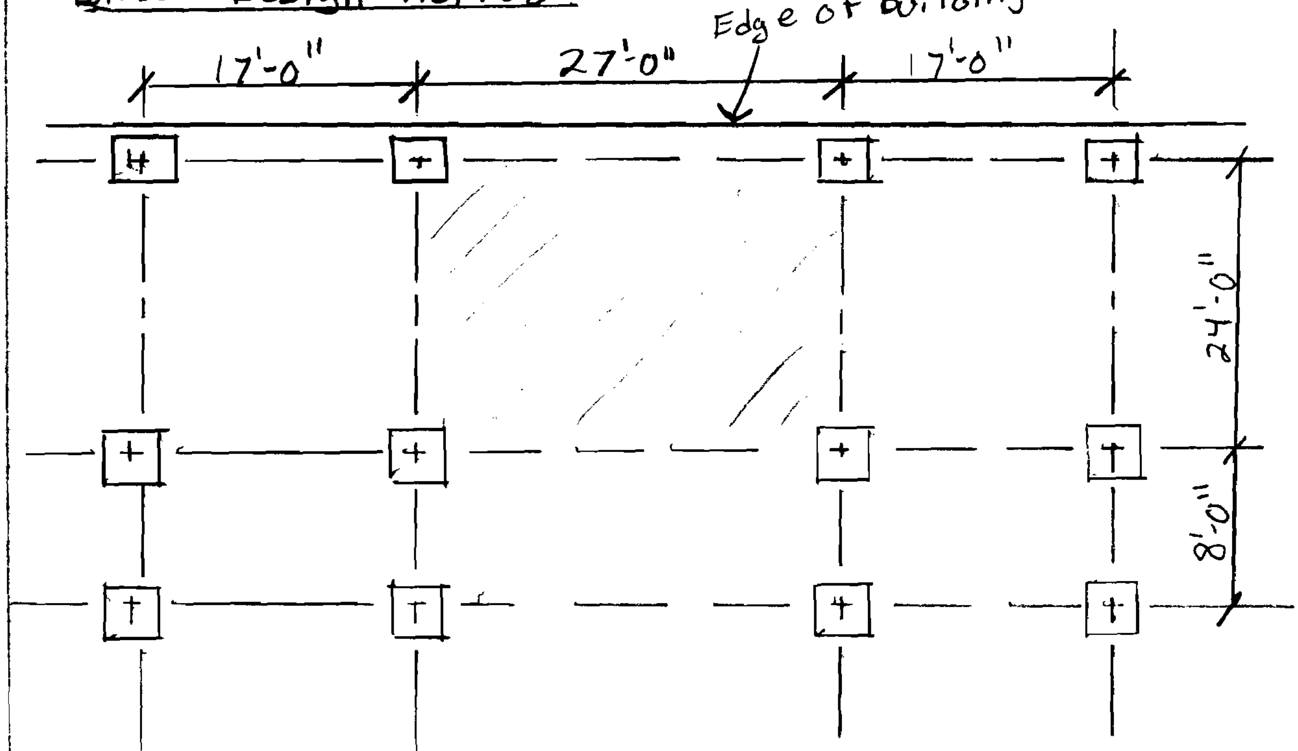
$$V_C = 2(1.0)\sqrt{4000}\,(24\times12)(8.3125)/1000$$
$$= 302.8\ \text{K}$$

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

$$\phi V_c = 227\text{K} > V_U = 70\text{K}\ \checkmark$$

- slab passes one way shear

Direct Design Method



- All columns are 24"x24"

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

Slab Moment Determination

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

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

Coefficients for Factored moments:

interior spans:

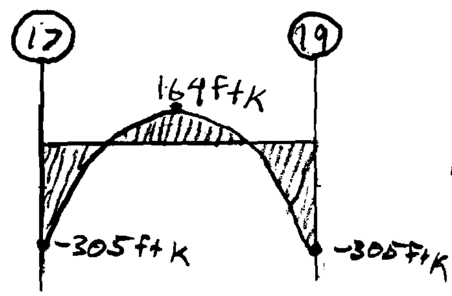
- at Midspan → $0.35M_o$
- at column → $0.65M_o$

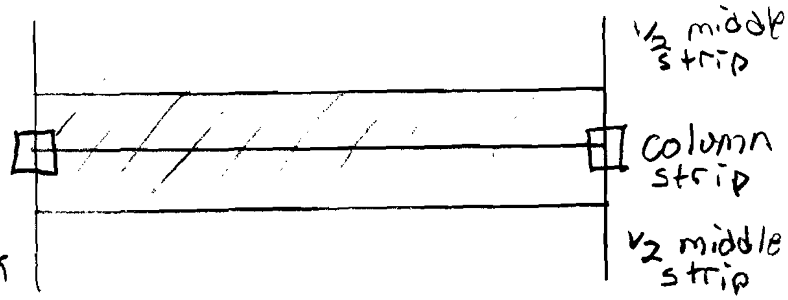
Exterior Edge:

- 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

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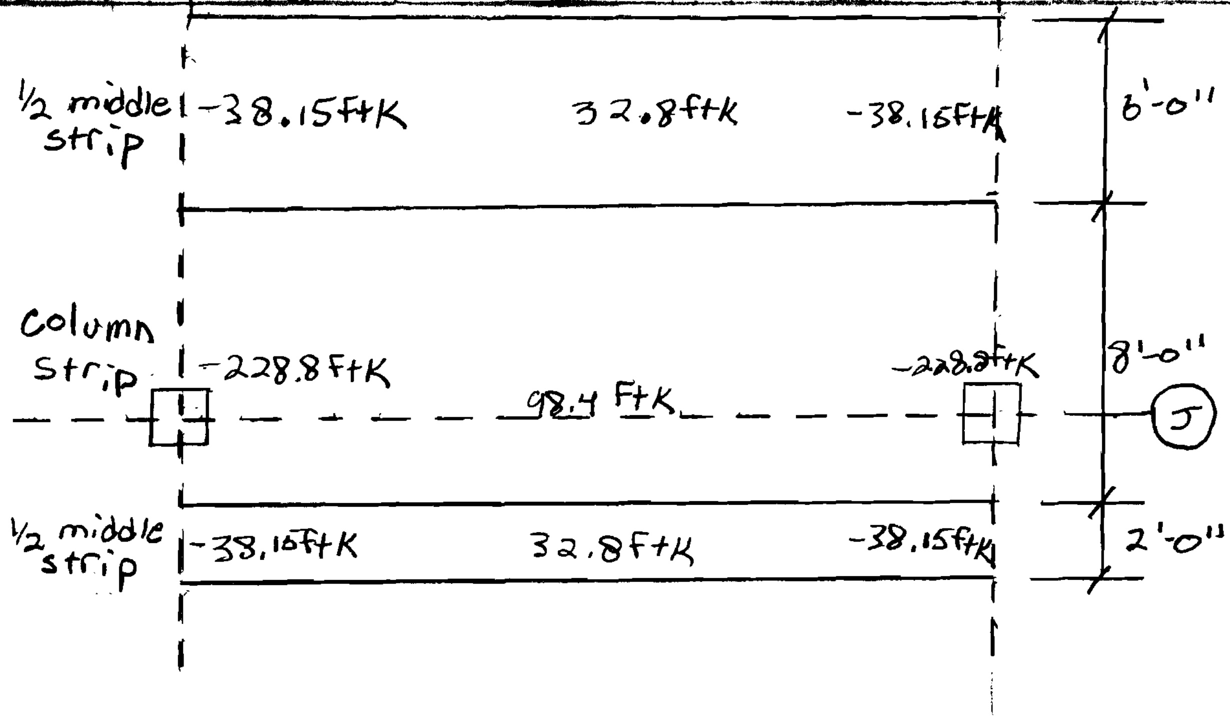


Design Reinforcement for Bending

$$A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{-228.8(12'')}{0.9(60000)(.95)(8.325)}$$

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

Spacing (13.3.2)

$$S_{max} \le 2h = 2(10) = 20''$$

Minimum Reinforcement

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

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

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

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

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

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

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

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" $8(.44) = 3.52 \text{ in}^2$

$$a = \frac{(3.52)(60000)}{0.85(4000)(10\times12)} = 0.52$$

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

$\phi M_n = 127$ 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"

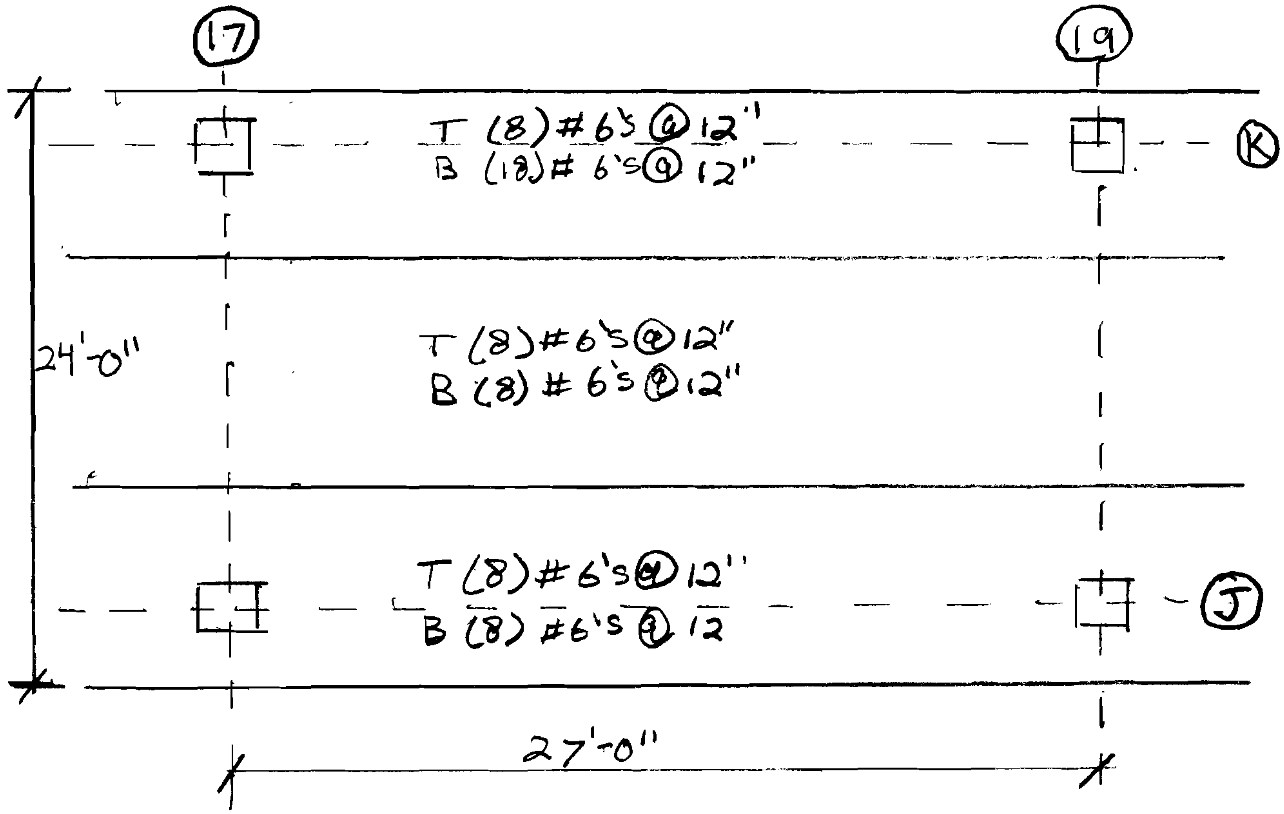
$\phi M_n = 127 \geq 32.8$

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

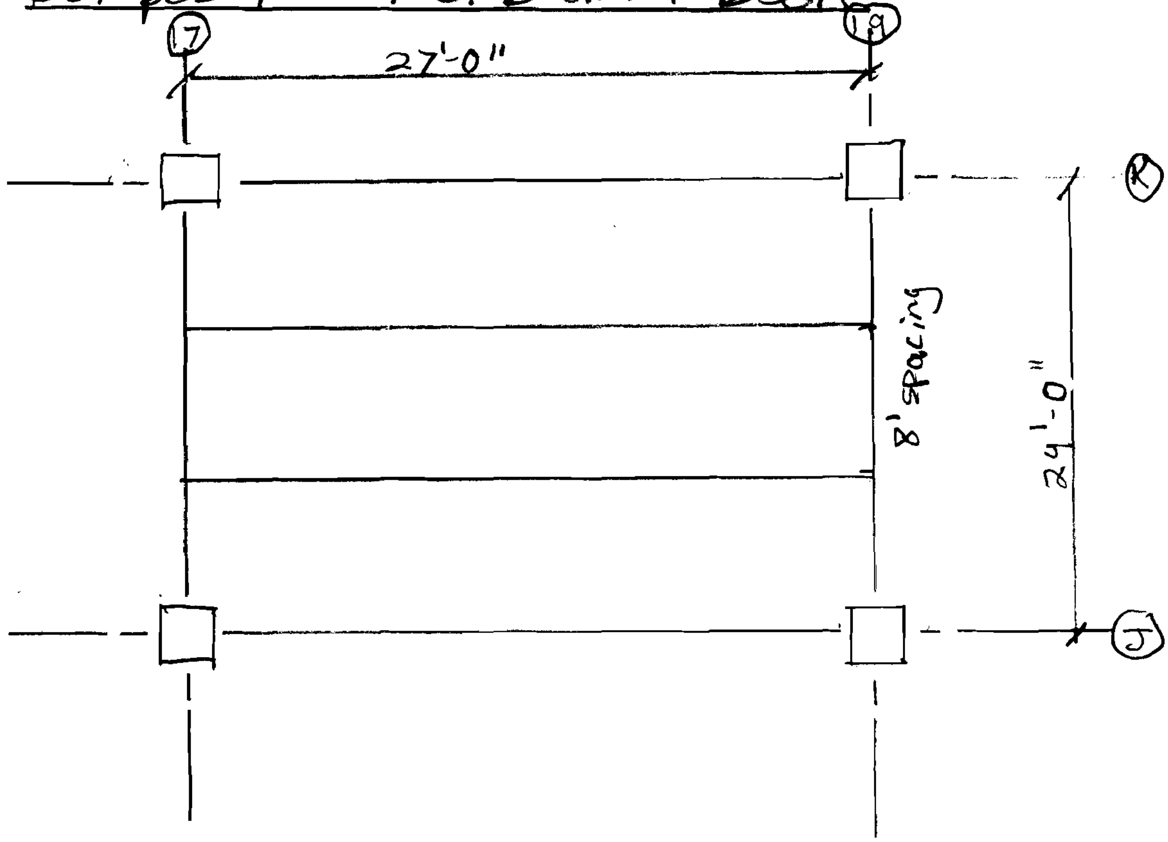


7 Design Alternate 2: Composite Steel Beam

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.

Mark Bland | Alternate #2 | Notebook Submission B

Design Alternate # 2:
Composite Steel Beam + Deck



Decking (using vulcraft catalog)

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

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:

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

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

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

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)

$W12\times16 \rightarrow \Sigma Q_n = 94.3 \quad \phi M_n = 134\,K\text{-}ft > 123\,K\text{-}ft$ ✓

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

$W12\times14 \rightarrow \Sigma Q_n = 119 \quad \phi M_n = 132\,K\text{-}ft > 123\,K\text{-}ft$ ✓

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

$W10\times17 \rightarrow \Sigma Q_n = 117 \quad \phi M_n = 132\,K\text{-}ft > 123\,K\text{-}ft$ ✓

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

$W10\times\cancel{45}15 \rightarrow \Sigma Q_n = 140 \quad \phi M_n = 129\,K\text{-}ft > 123\,K\text{-}ft$ ✓

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

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

Check economy

1 stud = 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]

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

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

Check unshored strength

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

$W_u = 1.2(79 \times 8 + 14) + 1.6(46.2) = 0.85\,klf$

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

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

W10x26 $\phi M_p = 117\,ftK > 77.38\,ftK \checkmark$

$I_x = 96.3\,in^4$

Check size, studs

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

$$\#studs = \frac{95.1}{17.2} = 5.5 \rightarrow 6 \times 2 = 12\,studs/beam$$

Check wet concrete Deflection

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

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

$$\Delta_{max} = \frac{L}{240} = \frac{27 \times 12}{240} = 1.35'' < 1.62'' \text{ X No good}$$

- could camber or use shoring

Camber

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

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

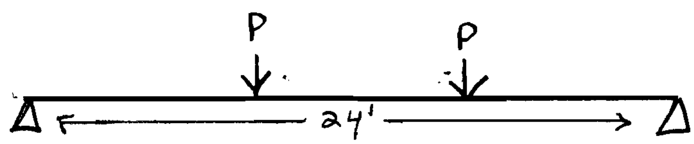
Check live load Deflection

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

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

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

Composite Girder Design



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

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

Design moment:

$M_u = Pa = 89.2K \times 8' = 713.9\,ftK$

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

Try W21×62

$\Sigma Q_n = 229$ $\phi M_n = 752\,ftK > 713.9\,ftK \checkmark$

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

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

Check unshored strength

W21x62 $\phi M_n = 540$ 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 < 540FtK ✓

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

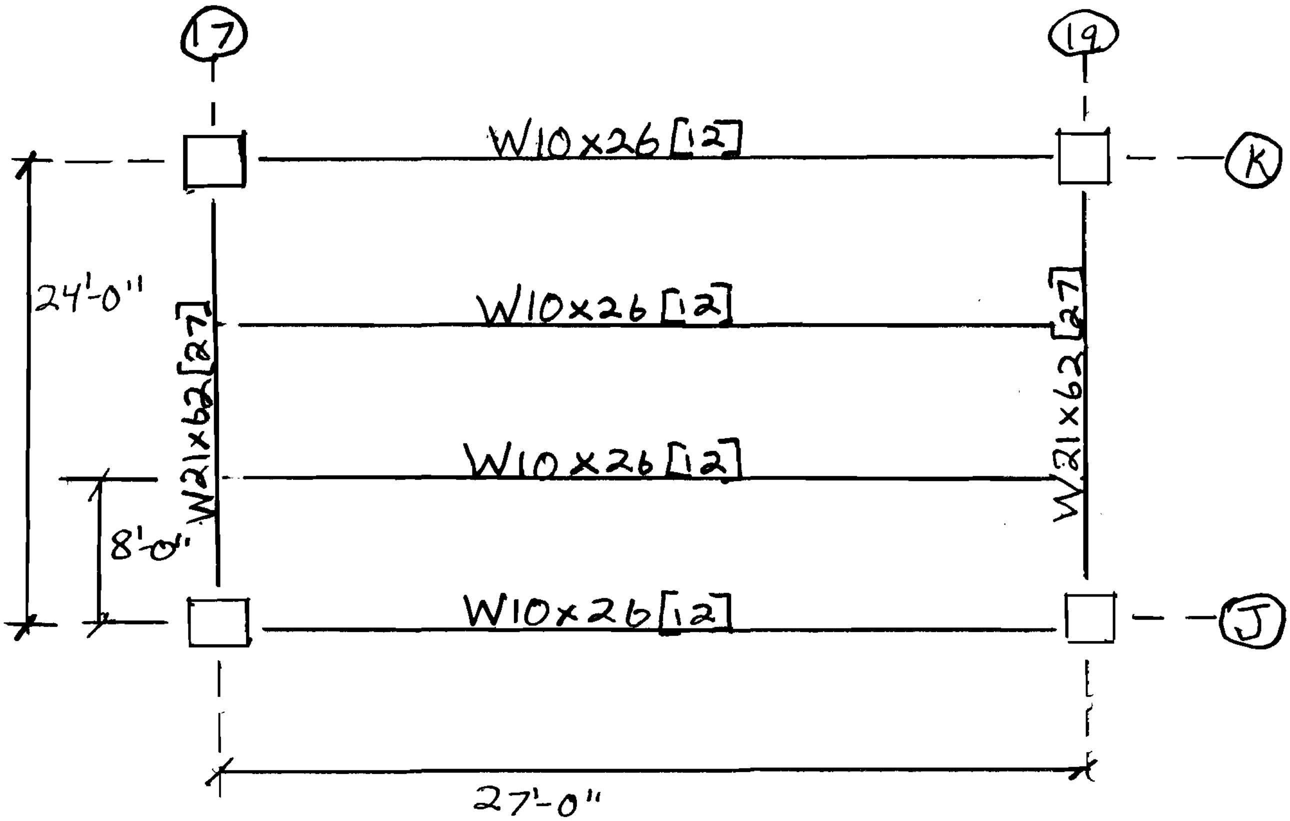
$W_L = 55psf(8ft)/1000 = 0.44$ KlF $\times 22 = 9.68K$

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

$= 0.0001'' < L/360$ ✓

Design #2 layout

Deck → 1.5VL18, 5" NW Concrete

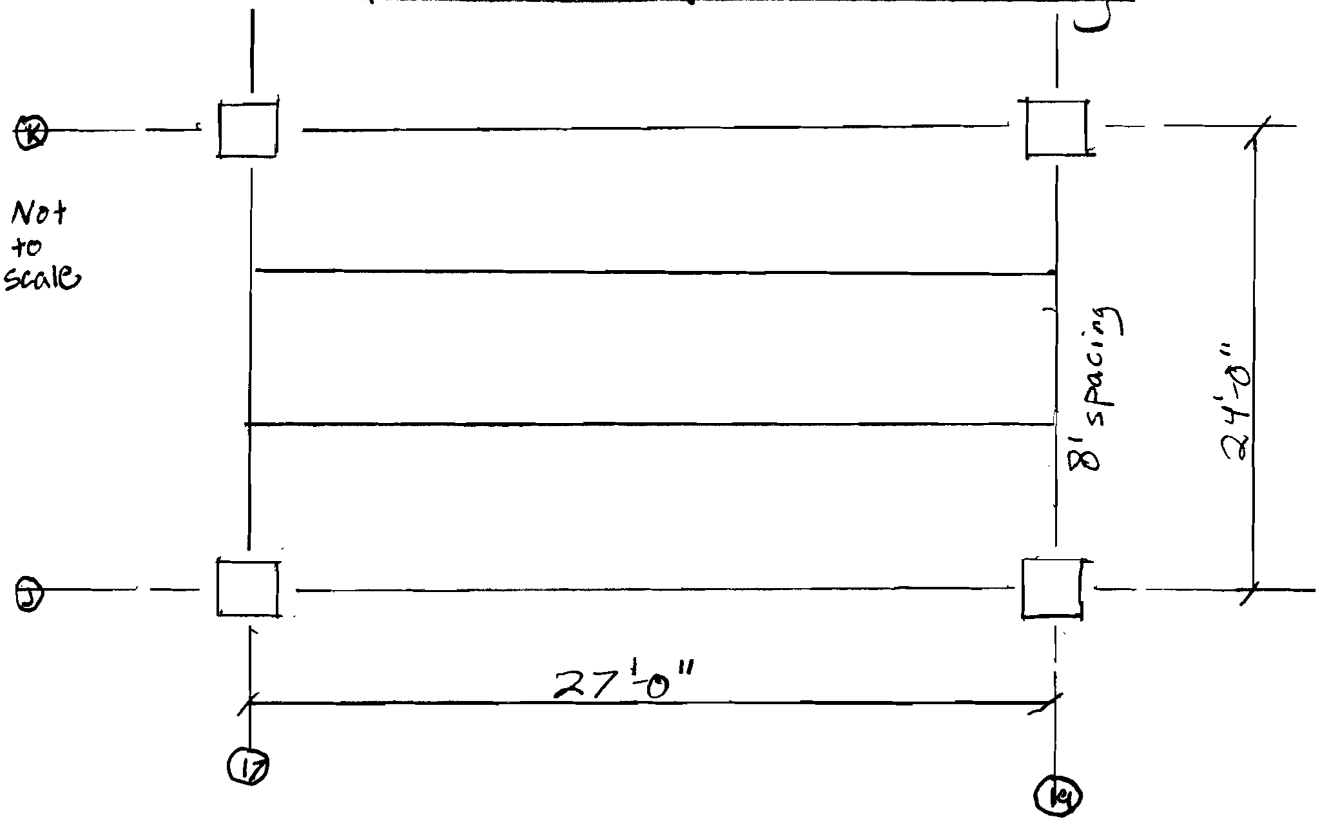


8 Design Alternate 3: Non-Composite Steel Beam

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.

Mark Bland | Alternate #3 | Notebook Submission B

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
(15 + 5 superimposed, 3 collatoral)

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

$w_u = 1.29$ KIF

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

Strength

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

Mark Bland | Alternate #3 | Notebook Submission B

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 $L/360$

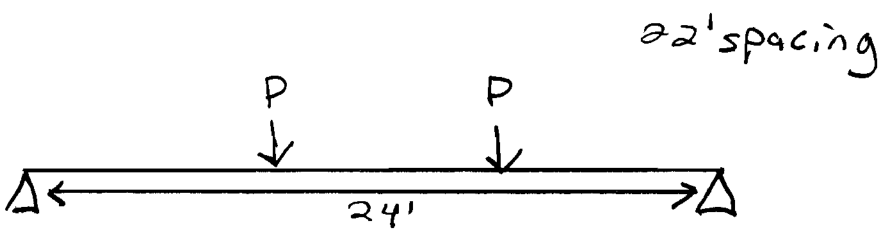
$$I_{req} = \frac{5(360)(55)(8)(27\times12)^3}{384(29{,}000)\ 12\ ^{in}/_{ft}} = 201.6\ in^4$$

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

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

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

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

$$= 543.3\ in^4$$

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

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

$$= 387.4\ in^4$$

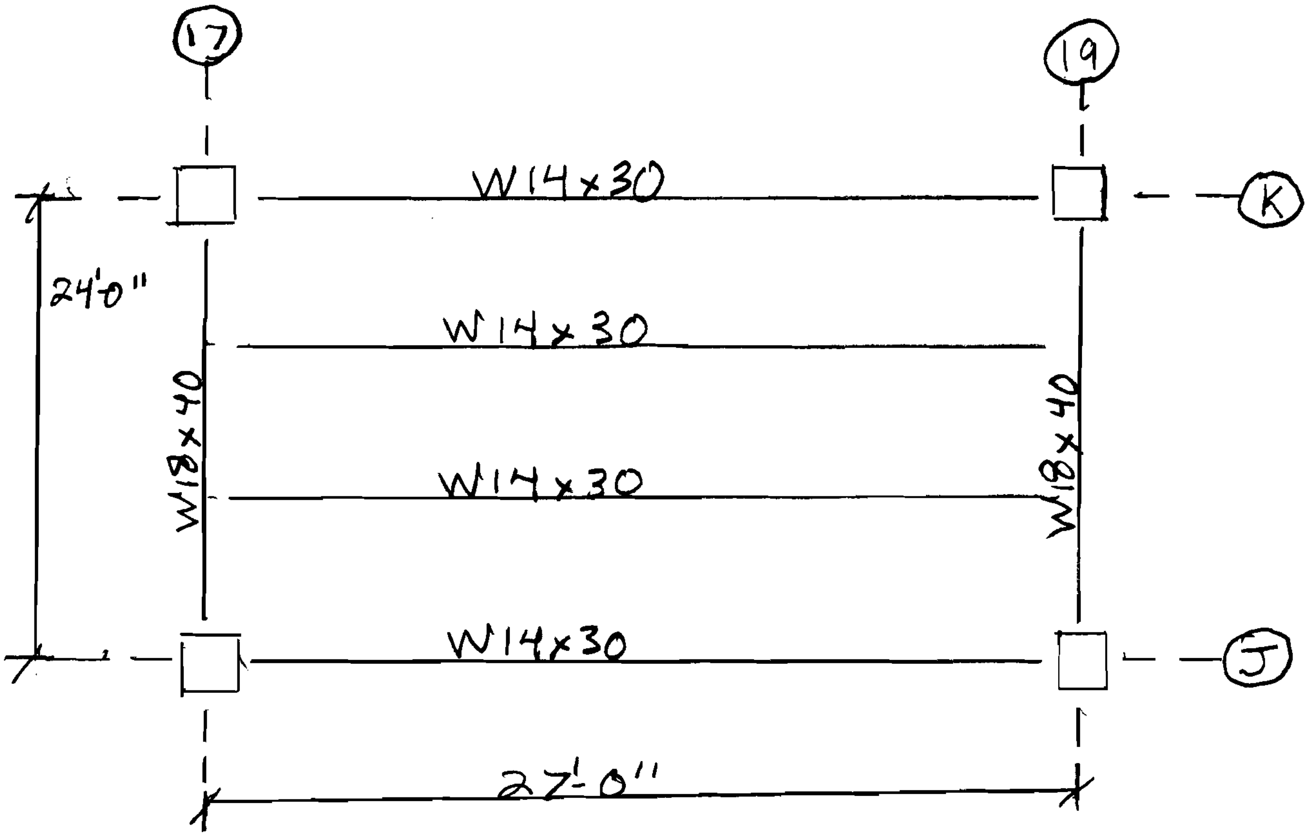
Use W18x40

$\phi M_n = 294\ ftK > 178.4\ ftK$

$I_x = 612\ in^4 > 543.3\ in^4$

Design #3 layout

Deck-1.5C20, 3.5" N.W concrete



9 Closing

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.