Technical Assignment 3

Medical Office Building

Brendon J. Burley
Structural Option
November 15, 2004

Advisor:
Dr. Thomas Boothby

Brendon J. Burley
Structural Option
Advisor:
Dr. Thomas Boothby
Submitted: Nov. 15, 2004

Executive Summary

This report considers the lateral systems of the Medical Office Building in Malvern, Pa. The building site is in a corporate park with several low to medium rise buildings. It is an extension of an existing complex and joins the previous buildings with a sky bridge. Although mostly office space, the building does include a small auditorium that is used for conferences and stockholder meetings. The open office space and auditorium benefit from the 28' square bays that are the basis of the structural system. The gravity system is composed of filigree slabs and beams resting on cast in place concrete columns. The lateral system is made up of cast in place frames in the east-west direction, and framing beams in the north-south direction.

The frames and beams in the building were analyzed based on strength and overturning. Deflection was not considered because the building is only four (4) stories above grade and was considered to be very stiff. The lateral loads that were utilized in the analysis were those due to wind. These loads were distributed to the frames in both directions based on the relative stiffness of each frame. The wind loads were divided among the stories based on the pressure gradient. Torsion in the frame was found to be 3189'k, the effects of this moment were felt mostly by the outermost frames in the north-south direction. The combined load of torsion and direct shear was used to calculate the overturning moments for each frame. The critical frame for overturning was determined to be a combined effect of the two west most north-south frames.

The overturning moments experienced by the critical frames were 47530'k and 24362'k. These frames together generated a net uplift of 576k on the column between them. After applying the axial loads to the column from above, 205k of uplift still existed. This means that the frame will overturn unless underpinning is provided. This underpinning may be provided by the strip footing at the base of the column that also bears the weight of three other columns that are not likely experiencing a net uplift. The remaining frames experienced significantly less uplift and were not considered to suffer from uplift.

In addition to uplift, the strength of the individual members in the frames had to be tested. A spot check of a beam and column in the east most central frame was used to determine the adequacy of member strength. The beam and column considered are those on the exterior bay below the top floor. A mistake in the load calculation resulted in a moment of 657'k, well above the strength of the beam, which was found to be 367'k without the safety factor. A 30% reduction of the negative load to estimate the correct amount found the ultimate moment to be 460'k, which places the beams design within an error resulting from design assumptions.

The column was also affected by the incorrect moment generated in the beam, but no corrections were necessary to verify the design. The column experienced an axial load of 326.9k and a moment of 308'k, the actual moment would be less so the use of the incorrect moment is conservative. Using column tables it was found that a reinforcing ratio of roughly 0.010 was required and the current design provided 0.015 so the column checks out.

Overall, the lateral system check is good. The overturning of the one frame may be resolved by the foundation design. The discrepancies in the beam can be dismissed as errors in analysis. The column appears to be over designed for the top, but is probably good for the bottom of the frame.

Brendon J. Burley
Structural Option
Advisor:
Dr. Thomas Boothby
Submitted: Nov. 15, 2004

Introduction

The Medical Office Building resides in a corporate park in Malvern, PA. The park has several low to medium height buildings that are clustered on moderately spaced sites. The Medical Office Building is an extension of an existing complex, and is used mostly as office space for two departments. However, an auditorium in the basement serves as the central location for meetings and conferences for the entire complex. A sky bridge on the third floor joins the Medical Office Building to a neighboring building allowing indoor travel between the existing complex.

The structural system of the Medical Office building is predominantly concrete, although steel trusses are used to span the atrium along the southwest wall. Filigree slabs and beams resting on cast in place concrete columns serve as the gravity system in the building. The columns are aligned on square bays measuring twenty-eight (28) feet to a side, except for the southwestern curve, which uses a radial alignment for the exterior columns. The building height varies from roughly 40’ above grade to as much as 68’ above grade. The lateral system is comprised of five (5) frames in the east-west direction and eight (8) beams on columns in the north-south direction. A typical floor plan showing the location of the frames appears below.

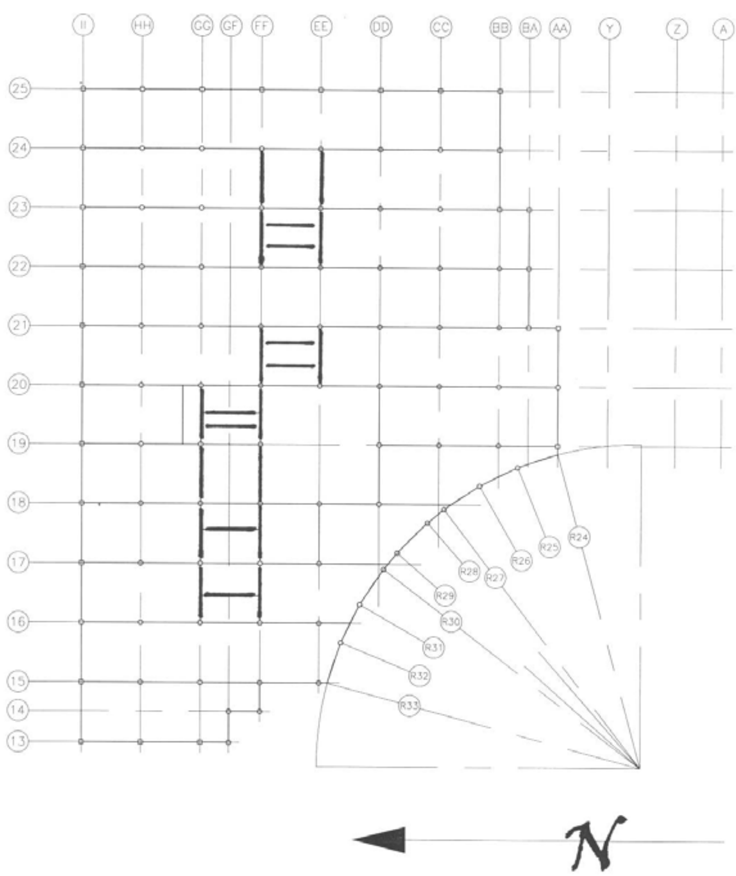


Figure 1 – Floor plan with frames

This report investigates the lateral support system of the Medical Office Building. The system was analyzed under wind loads in combination with live and dead loads. The system was tested for both strength and overturning. Spot checks of one horizontal member and one vertical member were also performed. Each component of the study is reported separately as follows:

- Loads
- Frames
- Overturning
- Beam Check
- Column Check

Loads

Wind loads, live loads, and dead loads all play a role in the analysis of the lateral system. These loads were determined using ASCE 7-02 in most cases, and the judgment of the analyzer where ASCE 7-02 did not apply. The live loads and dead loads based on these considerations were taken as:

Lobbies & 1st floor corridors	100 psf
2nd floor corridors	80 psf
Offices	50 psf
Stairwells	100 psf
Roof Live Load	20 psf
Filigree Dead Load	50 psf

For ease of calculations the live load was considered to always be 100 psf. The wind loads on the building depend on the direction of the wind and the height of the building. Because of the sloping nature of the site, and the resulting variation in the height of the building, the wind loads were estimated based on a mean height for each side of the building. The loads for the four principal directions are summarized in the following charts.

East		G=	0.81	GCpi=	0.18
		Cp(wind)=	0.80	Cp(lee)=	-0.425
Height	Kz	q	Windward	Leeward	Total
0-15ft.	0.57	11.82	5.53	-8.44	13.97
20ft.	0.62	12.86	6.02	-8.44	14.46
25ft.	0.66	13.69	6.41	-8.44	14.84
30ft.	0.70	14.52	6.79	-8.44	15.23
40ft.	0.76	15.76	7.38	-8.44	15.81
43ft.	0.78	16.09	7.53	-8.44	15.97

South		G=	0.79	GCpi=	0.18
		Cp(wind)=	0.80	Cp(lee)=	-0.5
Height	Kz	q	Windward	Leeward	Total
0-15ft.	0.57	11.82	5.34	-9.78	15.12
20ft.	0.62	12.86	5.81	-9.78	15.59
25ft.	0.66	13.69	6.19	-9.78	15.97
30ft.	0.70	14.52	6.56	-9.78	16.34
40ft.	0.76	15.76	7.12	-9.78	16.90
50ft.	0.81	16.80	7.59	-9.78	17.37
53ft.	0.82	17.01	7.69	-9.78	17.47

West		G=	0.81	GCpi=	0.18
		Cp(wind)=	0.80	Cp(lee)=	-0.425
Height	Kz	q	Windward	Leeward	Total

0-15ft.	0.57	11.82	5.53	-9.20	14.73
20ft.	0.62	12.86	6.02	-9.20	15.22
25ft.	0.66	13.69	6.41	-9.20	15.60
30ft.	0.70	14.52	6.79	-9.20	15.99
40ft.	0.76	15.76	7.38	-9.20	16.58
50ft.	0.81	16.80	7.86	-9.20	17.06
59ft.	0.85	17.55	8.21	-9.20	17.41

North		G=	0.79	GCpi=	0.18
		Cp(wind)=	0.80	Cp(lee)=	-0.5
Height	Kz	q	Windward	Leeward	Total
0-15ft.	0.57	11.82	5.34	-9.85	15.19
20ft.	0.62	12.86	5.81	-9.85	15.66
25ft.	0.66	13.69	6.19	-9.85	16.04
30ft.	0.70	14.52	6.56	-9.85	16.41
40ft.	0.76	15.76	7.12	-9.85	16.98
50ft.	0.81	16.80	7.59	-9.85	17.44
54ft.	0.83	17.13	7.74	-9.85	17.59

The maximum pressures resulting from the wind in the east-west direction and north-south direction are west and north. These two directions were used to determine the wind loads on the frames. The force due to wind was distributed to each story based on the pressure gradient (Figure 2). This force then has to be distributed among the lateral support elements before the analysis can continue.

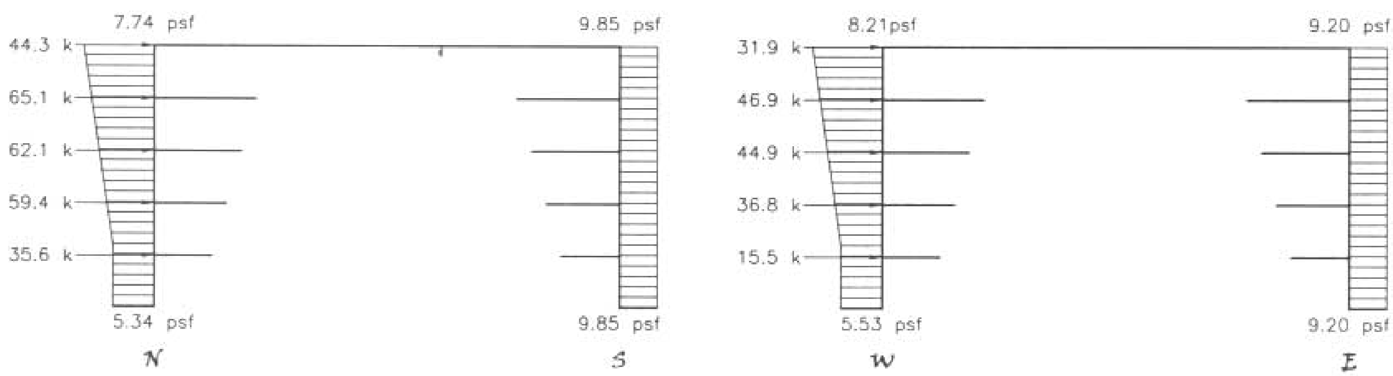


Figure 2 – Wind pressure gradients and equivalent loads.

Frames

The lateral support system of the Medical Office Buildings consists of frames in both directions. The entire system runs nearly the full length of the building in the east-west direction and remains centered in the north-south direction (Figure 3). The frames in the east-west direction include both columns and beams, but in the north-south direction the beams rest on girders before transferring the load to the columns. This results in torsion in the girders that only partially transfers to the columns. In the east-west direction the load transfers directly to the columns. Page three (3) of the appendix includes sketches of each frame; a general sketch appears in figure 4.

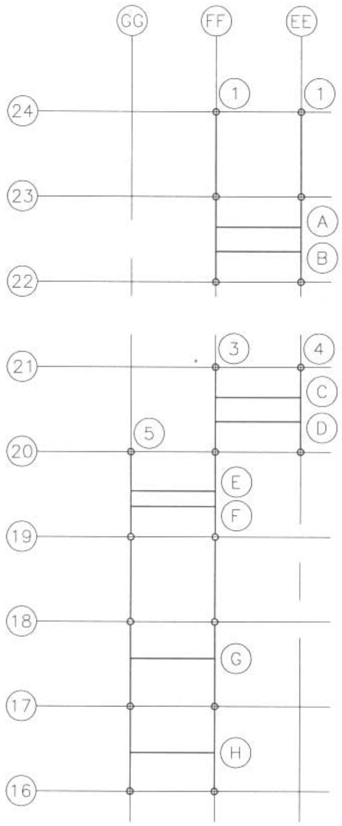


Figure 3 – Frame layout

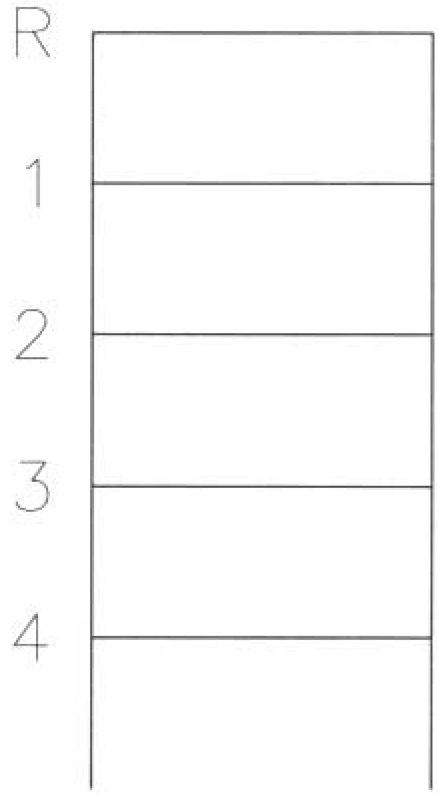


Figure 4 – Typical frame elevation

Each member in the lateral support system will take load relative to its stiffness. For the beams in the north-south direction, the stiffness is equal because each beam is the same size and has the same supports. Because there are eight (8) beams, each will take $1/8^{th}$ of the load due to wind. In the east-west direction the five (5) frames each have different properties. In order to distribute the load among the frames, each frame's stiffness must be determined. Stiffness refers to an elements ability to resist deflections and is defined by the following equation.

$$K = \frac{F}{\Delta} \tag{1}$$

The stiffness of each frame was estimated by performing a portal analysis of the exterior column of each frame due to a force at the top of the frame. The calculations of stiffness are detailed in the appendix; the results are summarized in the following table.

Frame	K
1	0.147
2	0.147
3	0.353
4	0.072
5	0.280

In addition to distributing the loads based on stiffness, it is necessary to consider torsion within the lateral system that may result from the distribution of loads. Torsion in the frame occurs when the center of mass does not coincide with the center of stiffness. For the Medical Office Building the center of mass is located 89' from the north wall and 128' from the east wall and the center of stiffness is located 82' from the north wall and 144' from the east wall. This results in eccentricities that lead to a total torsion moment of 3189'k. This moment results in additional shearing forces in the lateral system based on the stiffness and eccentricity of the individual frames. The results of this distribution are presented below.

Frame	Shear
1	26.291
2	144.480
3	63.134
4	70.766
5	226.191
A	701.854
B	569.339
C	103.290
D	55.172
E	42.059
F	61.650
G	501.226
H	1066.100

The final step in the distribution of load to the frames is to distribute the new frame shears to the stories in the frame based on the previous distribution related to the wind pressure. These results are summarized in the following tables.

Level	Wind	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
R	31.90	4.77	26.19	11.44	12.83	41.00
1	46.90	7.01	38.50	16.82	18.86	60.27
2	44.90	6.71	36.86	16.11	18.05	57.70
3	36.80	5.50	30.21	13.20	14.80	47.29
4	15.50	2.32	12.72	5.56	6.23	19.92

Level	Wind	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G	Frame H
R	44.30	134.66	109.23	17.17	9.17	8.07	11.83	96.16	177.22
1	65.10	197.88	160.52	25.23	13.48	11.86	17.38	141.32	260.42
2	62.10	188.76	153.12	24.07	12.86	11.31	16.58	134.80	248.42
3	59.40	180.55	146.46	23.02	12.30	10.82	15.86	128.94	237.62
4	35.60	0.00	0.00	13.80	7.37	0.00	0.00	0.00	142.41

Overturning

One of the possible failure methods of a lateral system is that it could overturn. Overturning occurs if the load on a column becomes tensile instead of compressive. Unless underpinning is provided, this results in the footing pulling up out of the ground and the entire frame toppling over. The overturning moment of each frame is based on the load carried by the frame at the stories. The following table summarizes the overturning moments of each frame.

	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
Mot=	855.68	4702.36	2938.67	2373.96	10528.50

	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G	Frame H
Mot=	24287.66	19702.00	3417.18	1825.28	1581.62	2318.36	24362.08	47530.47

The overturning moment can be distributed to the columns numerous ways. For this report the column loads were distributed based on the contribution of the column to the overall moment of inertia of the frame. The resultant uplift on each frame due to the overturning moment can be found in the appendix starting at page 8. In general the uplift due to wind is insignificant compared to the loading due to gravity on the column, so overturning was considered non critical to the design of most, but not all, frames

The case resulting in the most uplift is the combined effects of frames G and H on Column 17-GG. The uplift due to the wind load is 576 k. The weight of the column, the roof, and the floors all work to hold the column down. Applying the worst load combination there is a net uplift of 205 k on the column. This means that the frame will overturn unless the footing can hold it down. The footing underneath the column is part of a strip that supports three other columns. If these columns provide enough downward restraint, it may prevent overturning of column 17-EE.

Beam Check

Not only must the frames resist overturning, but they must also be able to bear load. The beam located in frame 1 between column lines 24 and 23 on level 1 was chosen as the member to be checked. The wind load was determined from a portal analysis of frame 1. The dead and live loads were patterned to result in the largest moment in the beam. Figure 5 shows the resulting moment diagrams for the wind load, dead load, maximum negative moment live load, and maximum positive moment live load on the beam.

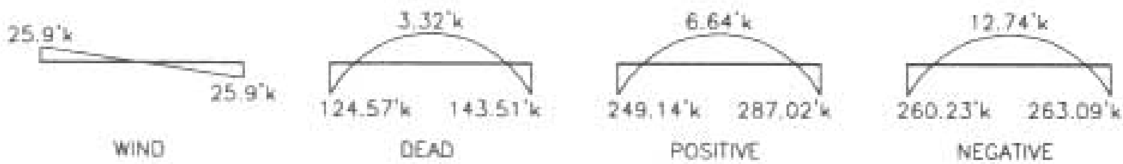


Figure 5 – Moment diagrams for beam at frame 1 level 1

The load combination resulting in the worse moment was 1.2D+1.6L+1.0W and the maximum moment for this combination was 657'k. The beam as built is a 48x18 beam with 6-#8 bars in the top strand and 6-#6 bars in the bottom strand. This results in a ρ value of 0.010. Entering a beam chart (Nilson 2004) with 40 ksi steel and 3 ksi concrete yields an R value of 367 psi. Through some manipulation of the numbers the maximum bending moment of the beam is found to be 358'k before the safety factor. This is significantly less than the required capacity of 657'k. Therefore this beam appears under designed. The estimation used for the frame system may be responsible for over estimating the negative moments and thus causing the large capacity requirement. A more thorough analysis would likely result in less required capacity.

A mistake in the calculations is responsible for the large required moment. A moment factor of 1/8 was taken instead of 1/12, the adjustment is discussed in conclusions.

Column Check

The exterior column beneath level 1 in frame 1 was checked for strength as a representative of the vertical frame elements. The same load combinations used in the bema check were used for the column check. The resulting moment diagrams for the columns are presented in figure 6.

Figure 6 – Moment diagrams for column 24-FF

The worst case load combination resulted in an axial force of 326.9 k and a moment of 308'k. This is equivalent to the load having a 10" eccentricity. Using a column chart (Nilson 2004) to plot the axial and moment strength percentages shows a required ρ of 0.01. The current column provides a ρ of 0.015 therefore it more than meets the requirements for strength.

Conclusions

The lateral system of the Medical Office Building generally supports the necessary wind loads. The frames are balanced in both directions resulting in a relatively small moment due to torsion. This moment does however lead to a rather large shear in the north-south frames that are far from the center of the building. This larger force in the frames results in very high overturning at the extreme frames that in the tested case appeared to fail the frame. The strip footing may be able to hold down the frame if the other three columns provide enough net downward force. Most of the building experiences much smaller loads and would be unlikely to overturn.

The frames themselves also have to be able to bear the stresses induced by the lateral loads. In the case of the beam in frame 1 that was investigated, there was not enough capacity to bear the required load. The estimation used in this analysis largely favored the negative moment because of a mistake in the calculation of the fixed end moments. Reducing the moments by roughly 30% to reflect a change of the moment coefficient to 1/12 from 1/8 results in a required strength of 460'k, this doesn't appear to be a large enough change to validate the beam design, but it does bring the margin of error into the range of being a poor estimation of the frame behavior. The same change would lower the loads on the column in frame 1 as well, but since it had enough strength to bear the increased loads it should be more than capable of taking the reduced loads.

Bibliography

American Society of Civil Engineers. Minimum Design Loads for Buildings and Other Structures. Reston, VA : American Society of Civil Engineers, Structural Engineering Institute, 2003.

Nilson, et. al. Design of Concrete Structures. New York, NY: McGraw Hill, 2004.

Appendix

1/11

Wind Load

$V = 90$ mph

$K_d = 1.0$ (using strength design)

Exposure "B"

$K_{zt} = 1.0$

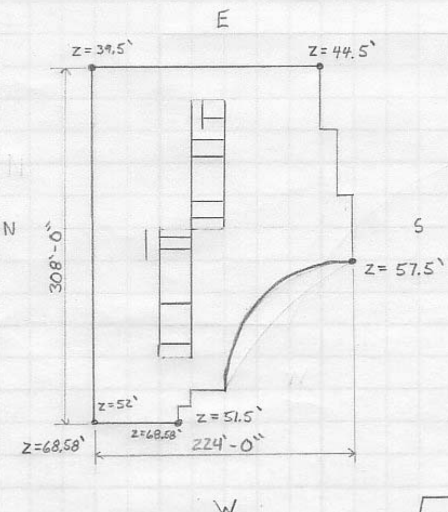
Assume Rigid Structure

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right)$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$$

$$c = 0.30$$

$$\bar{z} = .6h \geq 30'$$



East: $\frac{7}{8}\left(\frac{44.5+39.5}{2}\right) + \frac{1}{8}\left(\frac{44.5+57.5}{2}\right) = 43.125'$ $\bar{z} = 28.9'$

South: $\frac{6}{11}\left(\frac{44.5+57.5}{2}\right) + \frac{5}{11}\left(\frac{57.5+51.5}{2}\right) = 52.59'$ $\bar{z} = 31.6'$

West: $\frac{2.5}{8}(68.58) + \frac{5.5}{8}\left(\frac{57.5+51.5}{2}\right) = 58.90'$ $\bar{z} = 35.3'$

North: $1\left(\frac{39.5+68.58}{2}\right) = 54.04'$ $\bar{z} = 32.4'$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$$l = 320$$

$$\bar{\epsilon} = \frac{1}{3.0}$$

East use $\bar{z} = 30'$, B=224'	South B=308'	West B=224'	North B=308'
$L_{\bar{z}} = 309.99$	$L_{\bar{z}} = 315.41$	$L_{\bar{z}} = 327.27$	$L_{\bar{z}} = 318.05$
$Q = 0.80$	$Q = 0.77$	$Q = 0.80$	$Q = 0.77$
$I_{\bar{z}} = 0.30$	$I_{\bar{z}} = 0.30$	$I_{\bar{z}} = 0.30$	$I_{\bar{z}} = 0.30$
$G = 0.81$	$G = 0.79$	$G = 0.81$	$G = 0.79$

$$q = 0.00256\, K_z K_{zt} K_d V^2 I$$

$$I = 1.0$$

$$q = 0.00256\, K_z (1.0)(1.0)(90)^2(1.0) = 20.74\, K_z \text{ psf}$$

2/11

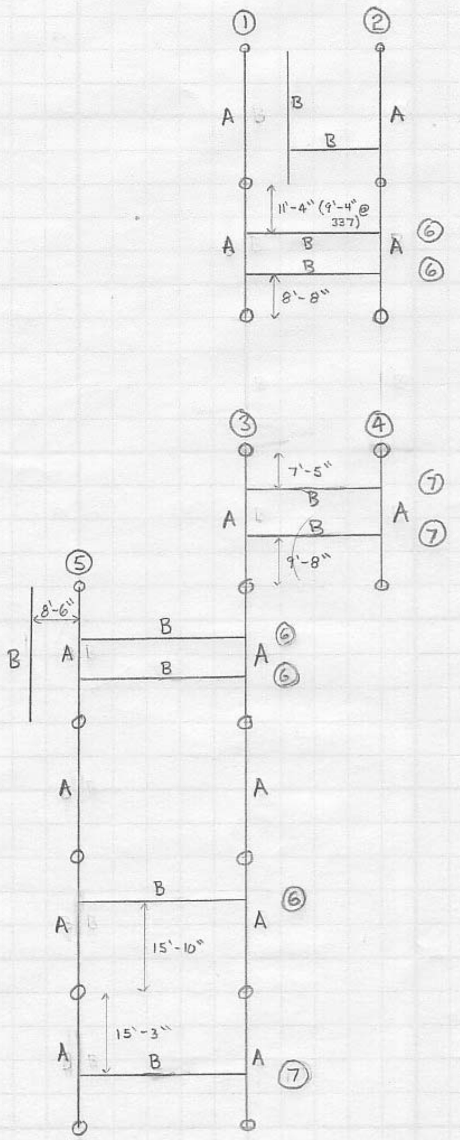
$$p = q\,GC_p - q(GC_{pi})$$

$C_p = 0.8$ Windward

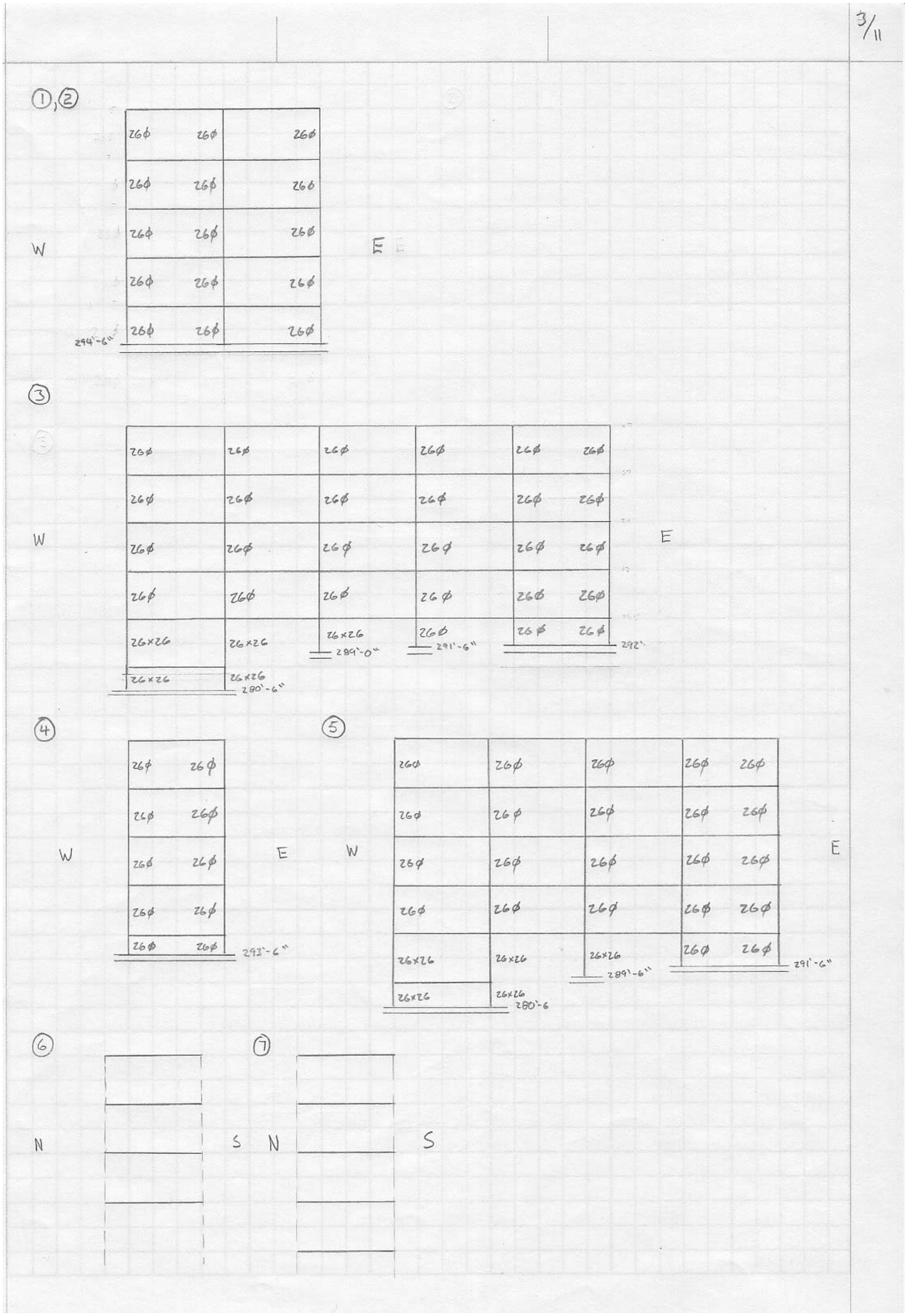
East	South	West	North
B= 224`	B = 308`	B = 224`	B = 308`
L= 308`	L = 224`	L = 308`	L = 224`
C_p = -0.425	C_p = -0.5	C_p = -0.425	C_p = -0.5

p calculated in Excel

Frame

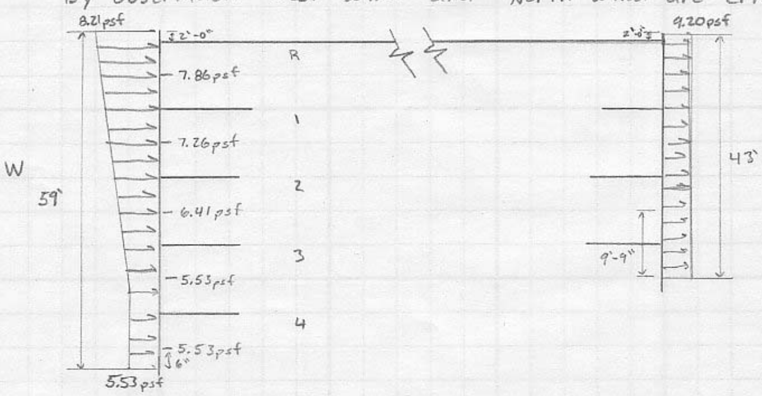


A 48 × 18
B 48 × 15 ½


3/11
1, 2
W
E
294'-6"
3
280'-6"
289'-0"
291'-6"
292'
4
293'-6"
5
289'-6"
291'-6"
280'-6
6
N
S
7

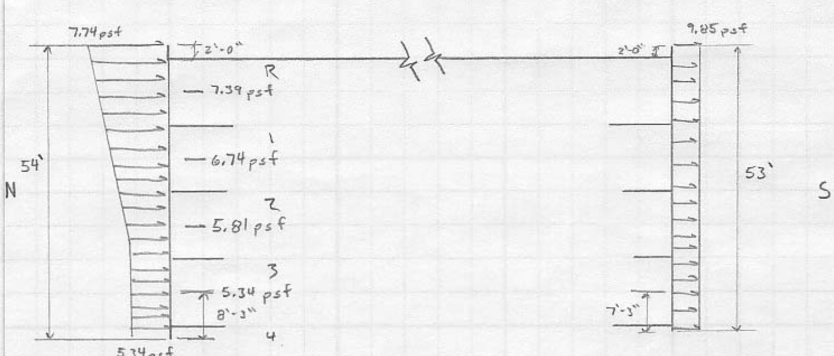
4/11

By observation West Wind and North Wind are critical



Width = 224'-0"
Story Height = 12'-6"

	Windward	Leeward	Total
R:	$P = (8.25)(224)\left(\frac{8.21+7.86}{2}\right) = 14849$ lb.	$P = (8.25)(224)(9.20) = 17002$ lb.	31.9 k
1:	$P = (12.5)(224)\left(\frac{7.86+7.26}{2}\right) = 21168$ lb.	$P = (12.5)(224)(9.20) = 25760$ lb.	46.9 k
2:	$P = (12.5)(224)\left(\frac{7.26+6.41}{2}\right) = 19138$ lb.	$P = (12.5)(224)(9.20) = 25760$ lb.	44.9 k
3:	$P = (12.5)(224)\left(\frac{6.41+5.53}{2}\right) = 16716$ lb.	$P = (9.75)(224)(9.20) = 20093$ lb.	36.8 k
4:	$P = (12.5)(224)\left(\frac{5.53+5.53}{2}\right) = 15484$ lb.	$P = 0$	15.5 k



Width = 308'-0"
Story Height = 12'-6"

	Windward	Leeward	Total
R:	$P = (8.25)(308)\left(\frac{7.74+7.39}{2}\right) = 19223$ lb.	$P = (8.25)(308)(9.85) = 25029$ lb.	44.3 k
1:	$P = (12.5)(308)\left(\frac{7.39+6.74}{2}\right) = 27200$ lb.	$P = (12.5)(308)(9.85) = 37923$ lb.	65.1 k
2:	$P = (12.5)(308)\left(\frac{6.74+5.81}{2}\right) = 24159$ lb.	$P = (12.5)(308)(9.85) = 37923$ lb.	62.1 k
3:	$P = (12.5)(308)\left(\frac{5.81+5.34}{2}\right) = 21464$ lb.	$P = (12.5)(308)(9.85) = 37923$ lb.	59.4 k
4:	$P = (8.25)(308)\left(\frac{5.34+5.34}{2}\right) = 13569$ lb.	$P = (7.25)(308)(9.85) = 21995$ lb.	35.6 k

In N-S direction lateral system does not directly include columns and all beams are the same size ∴ stiffness depends on # of beams.

Level \ #Beams	Frame 6	Frame 7	Load ⑥	Load ⑦
R:	5	3	5.54 k/frame	5.54 k/frame
1:	5	3	8.14 k/frame	8.14 k/frame
2:	5	3	7.76 k/frame	7.76 k/frame
3:	5	3	7.43 k/frame	7.43 k/frame
4:	0	3	0 k/frame	11.87 k/frame

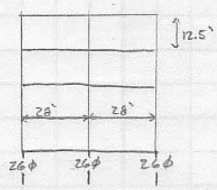
$$\text{Load} = \frac{\#\text{beams}}{\text{total beams}}(P)$$

5/11

Stiffness for E-W frames

$$K = \frac{F}{\Delta}$$

Frame 1 & 2

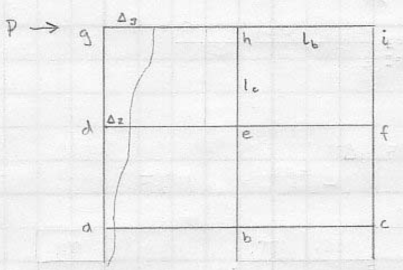


$I_{col} = \pi \frac{d^4}{64} = \pi \frac{26^4}{64} = 22432 \text{ in}^4$

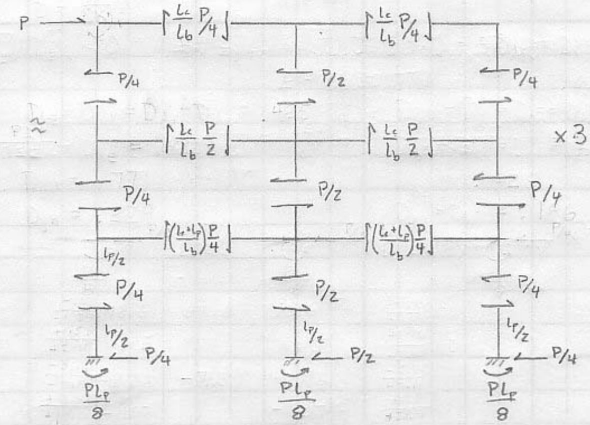
$I_{beam} = \frac{1}{12} bh^3 = \frac{1}{12}(48)(15.5)^3 = 14896 \text{ in}^4$

$K_{beam} = \frac{I_{beam}}{L_{beam}} = 44.33$

$K_{col} = \frac{I_{col}}{L_{col}} = 149.55$



$\approx$



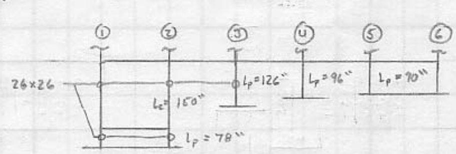
In general $FEM = \frac{PL}{8}$, for one column $\Delta = \Sigma\Delta$ and for assumed deflection bending $FEM = \frac{6EI}{L^2}\Delta \quad \therefore \Delta \approx \frac{PL^3}{48EI}$

$$\Delta = \frac{P}{E}\left(\frac{L_c^3}{48 I_{col}} \times 4 + \frac{L_p^3}{48 I_{col}}\right) = \frac{P}{48(22432)E}\left(4(150)^3 + (54)^3\right) = 12.684\frac{P}{E}$$

$$K = \frac{P}{\Delta} = 0.0788E$$

Using Portal Analysis Column shear in outer column $= \frac{P}{2n}$ where n is number of bays. The fixed end moment for the column is the column shear multiplied by half the column height $\therefore FEM = \frac{PL}{4n}$. With a fixed end columns, a good assumption for cast in place concrete, $FEM = \frac{6EI}{L^2}\Delta \quad \therefore \Delta = \frac{PL^3}{24nEI}$.

Frame 3 5 bays 4 full columns $L_c = 150''$, varied base



Stiffest column will control

$I_{26} = \frac{1}{12} bh^3 = \frac{1}{12}(26)(26)^3 = 38081 \text{ in}^4$

$K = \frac{I}{L}$

Deflection assumption is only valid for exterior columns

$$\Delta_1 = \frac{P}{24(5)E}\left(\frac{150^3}{22432} \times 4 + \frac{150^3}{38081} + \frac{78^3}{38081}\right) = 5.858\frac{P}{E}$$

$$\Delta_6 = \frac{P}{24(5)E}\left(\frac{150^3}{22432} \times 4 + \frac{90^3}{22432}\right) = 5.286\frac{P}{E} \quad \Leftarrow \text{controls}$$

$$K = \frac{P}{\Delta} = 0.1892E$$

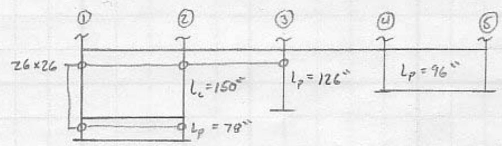
6/11

Frame 4 1 bay, 4 full columns $L_c = 150"$, 1 pier $L_p = 72"$

$$\Delta = \frac{P}{24(1)E}\left(\frac{150^3}{22432} \times 4 + \frac{72^3}{22432}\right) = 25.769\,\frac{P}{E}$$

$$K = \frac{P}{\Delta} = 0.0388E$$

Frame 5 4 bays, 4 full columns $L_c = 150"$, varied base



$$\Delta_1 = \frac{P}{24(4)E}\left(\frac{150^3}{22432} \times 4 + \frac{150^3}{38081} + \frac{78^3}{38081}\right) = 7.322\,\frac{P}{E}$$

$$\Delta_2 = \frac{P}{24(4)E}\left(\frac{150^3}{22432} \times 4 + \frac{96^3}{22432}\right) = 6.680\,\frac{P}{E} \quad \leftarrow \text{controls}$$

$$K = \frac{P}{\Delta} = 0.1497E$$

$$\text{Total Stiffness} = 2(0.0788E) + 0.1892E + 0.0388E + 0.1497E = 0.5353E$$

$$\text{Frame Loads} = \frac{K}{K_T}P$$

1 $\frac{0.0788}{0.5353}P = 0.147P$

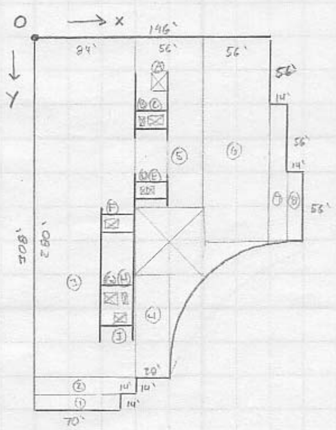
2 $\frac{0.0788}{0.5353}P = 0.147P$

3 $\frac{0.1892}{0.5353}P = 0.353P$

4 $\frac{0.0388}{0.5353}P = 0.072P$

5 $\frac{0.1497}{0.5353}P = 0.280P$

Torsion

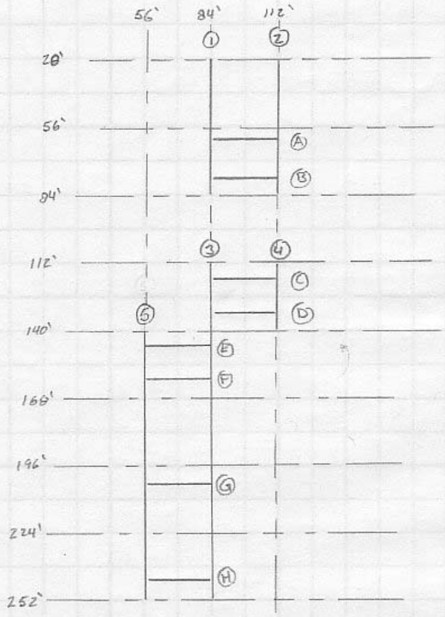


Zone	Area (ft^2)	x (ft.)	y (ft.)	Q_x	Q_y
1	980	35	301	34300	294980
2	1176	42	287	49392	337512
3	23520	42	140	987840	3292800
4	2352	98	238	230496	559776
5	7840	112	70	878080	548800
6	9408	168	84	1580544	790272
7	1568	203	112	318304	175616
8	784	217	140	170128	109760
A	-155	102.25	39.5	-15848.75	-6122.5
B	-14	87.75	71.333	-1228.5	-998.662
C	-78	100.25	71.333	-7819.5	-5564
D	-58.792	91.917	124.875	-5403.965	-7341.651
E	-58.792	100.417	124.875	-5903.716	-7341.651
F	-129.976	67.396	147.458	-8759.862	-19166.001
G	-149.139	65.583	215.083	-9780.783	-32077.264
H	-17.208	75.042	215.083	-1291.323	-3701.148
I	-86.333	72.125	232.625	-6226.768	-20083.214
	46880.760			4186820.833	6007119.909

$$\bar{x} = \frac{Q_x}{A} = 89.308'$$

$$\bar{y} = \frac{Q_y}{A} = 128.136'$$

7/11



Frame	K_{rel}	X	Y	Q
1	0.147	84		12.348
2	0.147	112		16.464
3	0.353	84		29.652
4	0.072	112		8.064
5	0.280	56		15.680
	0.999	—		82.208
A	0.125		67.667	8.458
B	0.125		75.667	9.458
C	0.125		119.417	14.927
D	0.125		130.333	16.292
E	0.125		152.917	19.115
F	0.125		159.917	19.990
G	0.125		208.167	26.021
H	0.125		239.25	29.906
	1.000		—	144.167

$x_{cr} = \frac{Q_x}{\Sigma K} = 82.290'$

$y_{cr} = \frac{Q_y}{\Sigma K} = 144.167'$

$e_x = x_{CR} - x = -7.018' \times 1.05 = -7.369'$

$e_y = y_{CR} - y = 16.031' \times 1.05 = 16.833'$

accidental torsion

Determine d_i for each frame

$d_i = |x - x_{cr}|$ or $d_i = |y - y_{cr}|$

Frame	K_{rel}	d_i	Kd_i	Kd_i^2
1	0.147	1.71	0.25	0.43
2	0.147	29.71	4.37	129.83
3	0.353	1.71	0.60	1.03
4	0.072	29.71	2.14	63.58
5	0.280	26.29	7.36	193.49
A	0.125	76.50	9.56	731.34
B	0.125	68.50	8.56	586.36
C	0.125	24.75	3.09	76.48
D	0.125	13.83	1.73	23.93
E	0.125	8.75	1.09	9.54
F	0.125	15.75	1.97	31.03
G	0.125	64.00	8.00	512.00
H	0.125	95.08	11.88	1129.67

$J = 3488.71\ ft^2$

Sum forces

W-E : Y direction

$P_y = 31.9 + 46.9 + 44.9 + 36.8 + 15.5$

$P_y = 176.0\ k$

N-S: X direction

$P_x = 44.3 + 65.1 + 62.1 + 59.4 + 35.6$

$P_x = 266.5\ k$

Torsion Moment

$M_t = P_y e_x + P_x e_y$

$M_t = 176.0(-7.369) + 266.5(16.833)$

$M_t = 3189.05\ 'k$

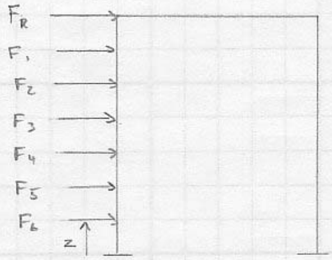
$F = \frac{k}{\Sigma K} P + \frac{kd_i}{J} M_t$ → see excel

For story

$F_i = \frac{P_i}{P} F$ → see excel

8/11

Overturning

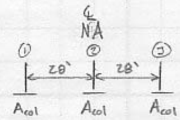


$M_{OT} = \sum F_i z \longrightarrow$ see excel

Moment Arm = Frame Length

Assume load distributes based on moment of inertia to frame nuetral axis.

Frame 1, 2



$A_{col} = \pi \frac{d^2}{4} = \pi \frac{(26)^2}{4} = 531 \text{ in}^2$

$I \approx A d^2$

$I_1 = 531\,(28 \times 12)^2 = 59.95 \times 10^6 \text{ in}^4$

$I_2 = 531\,(0)^2 = 0 \text{ in}^4$

$I_3 = 531\,(28 \times 12)^2 = 59.95 \times 10^6 \text{ in}^4$

$I_{total} = 119.90 \times 10^6 \text{ in}^4$

$P_{tot1} = \frac{855.68(12)}{56(12)} = 15.28 \text{ k}$

$P_1 = \frac{59.95}{119.90} = 7.64 \text{ k}$

$P_2 = \frac{0}{119.90} = 0 \text{ k}$

$P_3 = \frac{59.95}{119.90} = 7.64 \text{ k}$

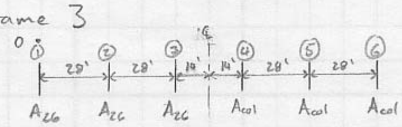
$P_{tot2} = \frac{4702.36(12)}{56(12)} = 83.97 \text{ k}$

$P_1 = 41.99 \text{ k}$

$P_2 = 0 \text{ k}$

$P_3 = 41.99 \text{ k}$

Frame 3



$A_{26} = 26(26) = 676 \text{ in}^2$

Col	x	A	Q_x
1	0	676	0
2	28	676	18928
3	56	676	37856
4	84	531	44604
5	112	531	59472
6	140	531	74340
		3621	235200

$NA = \frac{Q_x}{A} = 64.95'$

$P_{tot} = \frac{2938.67(12)}{140(12)} = 20.99 \text{ k}$

$I = A(x - NA)^2$

$I_1 = 4.11 \times 10^8 \text{ in}^4$ $P_1 = 7.31 \text{ k}$

$I_2 = 1.33 \times 10^8 \text{ in}^4$ $P_2 = 2.37 \text{ k}$

$I_3 = 7.80 \times 10^6 \text{ in}^4$ $P_3 = 0.14 \text{ k}$

$I_4 = 2.77 \times 10^7 \text{ in}^4$ $P_4 = 0.49 \text{ k}$

$I_5 = 1.69 \times 10^8 \text{ in}^4$ $P_5 = 3.01 \text{ k}$

$I_6 = 2.99 \times 10^8 \text{ in}^4$ $P_6 = 7.67 \text{ k}$

$I_{tot} = 1.18 \times 10^9$

Frame 4 : Symmetric

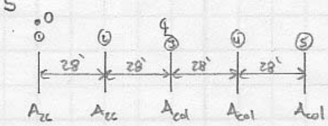
$P_{tot} = \frac{2373.96(12)}{28(12)} = 84.78 \text{ k}$

$P_1 = 42.39 \text{ k}$

$P_2 = 42.39 \text{ k}$

9/11

Frame 5



Col	x	A	Q_x
1	0	676	0
2	28	676	18928
3	56	531	29730
4	84	531	44595
5	112	531	59460
		2945	152713

$P_{tot} = \frac{10528650(12)}{112(12)} = 94.00\,k$

$NA = \frac{Q_x}{A} = 51.86$

$I = A\,(x - NA)^2$

$I_1 = 2.62 \times 10^8\ in^4$ $P_1 = 36.49$

$I_2 = 5.54 \times 10^7\ in^4$ $P_2 = 7.72$

$I_3 = 1.67 \times 10^6\ in^4$ $P_3 = 0.23$

$I_4 = 7.90 \times 10^7\ in^4$ $P_4 = 11.01$

$I_5 = 2.76 \times 10^8\ in^4$ $P_5 = 38.54$

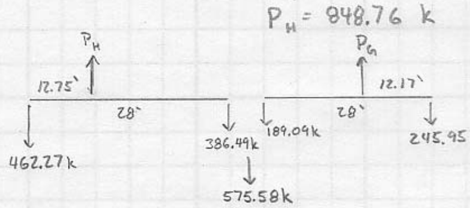
$I_{tot} = 6.74 \times 10^8$

Frame A, B, C, D, E, F, G, H are symmetric and supported on beams, width = 28`

$P = \frac{1}{2}\frac{M_{OT}}{width}$

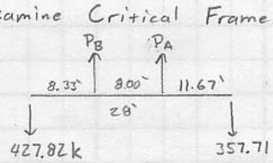
$P_A = 433.71\ k$
$P_B = 351.82\ k$
$P_C = 61.02\ k$
$P_D = 32.59\ k$
$P_E = 28.24\ k$
$P_F = 41.40\ k$
$P_G = 435.04\ k$
$P_H = 848.76\ k$

Examine Critical Frames



critical

or



Interior Column with 1 roof, 5 floors, TA = 28`×28`

wind is in opposite direction of live and dead loads

using
$L_R = 20\ psf$
$L_f = 100\ psf$
$D = 50\ psf$ ← filligree slab
$D_{col} = 600\ plf$ ← column self weight

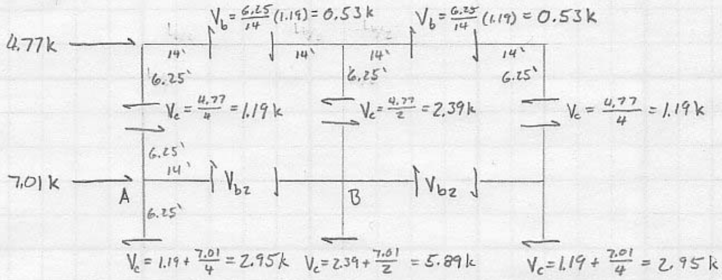
Load pattern $1.2D + 0.5L + 1.3W$ worst for overturning

$1.2(50)(28^2) = 47.04\ k/story$ dead
$0.5(100)(28^2) = 39.20\ k/story$ live
$1.0(20)(28^2) = 15.68\ k/roof$
$1.2(600)(68.5) = 49.32\ k$ column
$1.3(575.58k) = 748.25\ k$ uplift

$P_{tot} = 15.68 + 5(39.20) + 6(47.04) + 49.32 - 748.25 = \underline{-205.014}$ N.G.

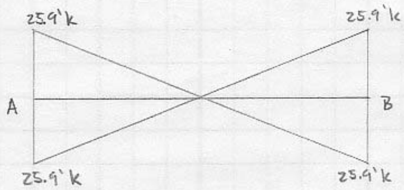
10/11

Spot Check of Frame 1 Level 1 Beam 24-23 and Column FF-24

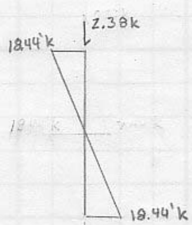


$V_{b2} = 6.25(1.19 + 2.95)/14 = 1.85\,k$

$FEM_b = 14(1.85) = 25.9'k$

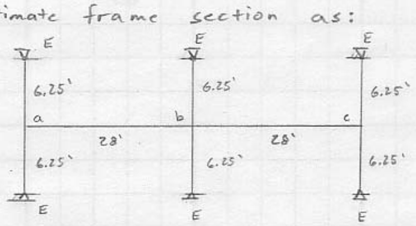


Wind on beam



Wind on column below level 1

Estimate frame section as:



$K_{col} = \frac{I_{col}}{6.25} = \frac{22432}{6.5} = 3451$

$K_{beam} = \frac{I_b}{28} = \frac{14896}{28} = 532$

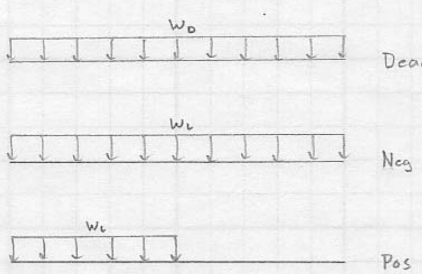
modified: $K_{aE} = K_{cE} = \frac{3}{4} K_{col} = 2588 = K_{bE}$

$D_{aE} = D_{cE} = \frac{2588}{2(2588)+532} = 0.453$

$D_{ab} = D_{cb} = \frac{532}{2(2588)+532} = 0.092$

$D_{bE} = \frac{2588}{2(2588+532)} = 0.415$

$D_{ba} = D_{bc} = \frac{532}{2(2588+532)} = 0.085$

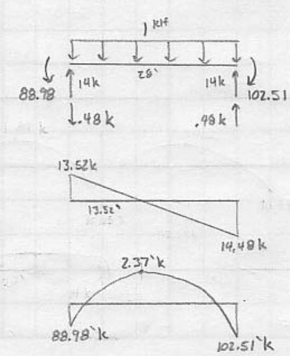


Use unit load of 1 klf

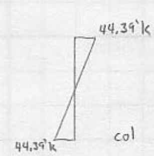
$FEM = \frac{wl^2}{8} = \frac{1(28)^2}{8} = 98'k$

	2x			2x	
1	0.453	0.092	0.085	0.415	0.085
0	0	-98	98	0	-98
	+44.39	+9.02	0	0	0
	44.39	-88.98	+4.51		-4.51
			0	0	0
			102.51		-102.51

} symmetry

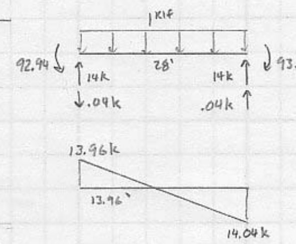


$\frac{102.51 - 88.98}{28} = .48$



11/11

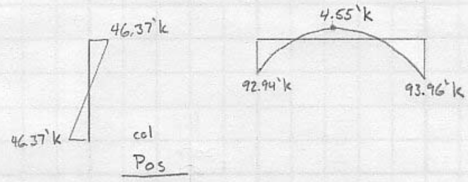
	×2			2×			×2	
1	0.453	0.092	0.085	0.415	0.085	0.092	0.453	1
0	0	−98	98	0	0	0	0	0
	+44.39	+9.02	−8.33	−40.67	−8.33	0	0	
		−4.17	+4.51		0	−4.17		
	+1.89	+.38	−.38	−1.87	−.38	+.38	+1.89	
		−.19	+.19		+.19	−.19		
	+.09	+.02	−.03	−.16	−.03	+.02	+.09	
	46.37	−92.94	93.96	−42.7	−8.55	−3.96	1.98	

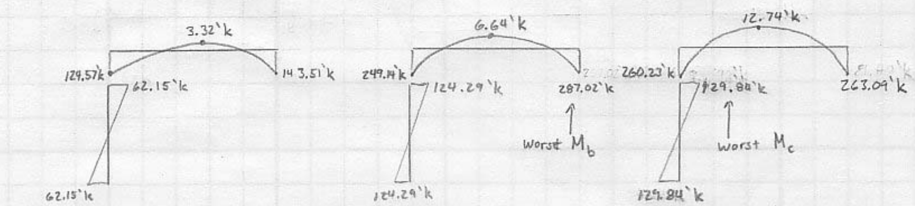


Tributary Area = 28'

$w_L = 100(28) = 2.80$ klf

$w_D = 50(28) = 1.40$ klf





For Beam

$1.2D + 1.6L + 1.0W = 1.2(143.51) + 1.6(287.02) + 1.0(25.9) = 657.344'k$ ← controls

$1.2D + .5L + 1.3W = 1.2(143.51) + .5(287.02) + 1.3(25.9) = 349.39'k$

For Column

$1.2D + 1.6L + 1.0W = 1.2(62.15) + 1.6(129.84) + 1.0(25.9) = 308.22'k$ ← controls

$1.2D + .5L + 1.3W = 1.2(62.15) + 0.5(129.84) + 1.3(25.9) = 173.17'k$

$P_u = 1.2D + 1.6L + 1.0L_R + 1.0W$

$= 1.2(47.04 \times 2) + 1.6(125.44) + 1.0(15.68) + 1.0(-2.38) = 326.9$ k

$e = \frac{M}{P_u} = .943'$

Beam: 6 #8 T, 6 #6 B, 48 × 18, #3 stirrups, $f_y = 40$ ksi, $f'_c = 3$ ksi, $d = 18 - 1.5 - \frac{3}{8} - \frac{1}{2} = 15.625$

$A_s = 6(.79) + 6(.44) = 7.38$ $a \approx 0.70$ ∴ $c \approx 0.60$ ∴ $\varepsilon_t = 0.075$ → $\phi = .9$

$\rho = \frac{A_s}{bd} = \frac{7.38}{48(15.625)} = 0.010$

from Nilson Graph A.1b $R = 367$ psi

$M_n = R\,bd^2 = 367(48)(15.625)^2 = 4300781"$ lb $= 358.40'k$

$\phi M_n = 322.56'k < 657.34k$

No good

Column: 26" ϕ, 8 #9, #3 ties @ 12"

$26 - 3 - 2\left(\frac{3}{8}\right) - 2\frac{d_b}{2} = 21.123$ $\gamma = \frac{21.123}{26} = 0.81$

$A_s = 8(1) = 8$ in² $\frac{e}{h} = \frac{.943}{26/12} = 0.435$

Adjust for $f_y = 40$ksi $R_n = \frac{308.22(12)}{3(531)(26)} = 0.089 \times 1.3 = 0.116$

$K_n = \frac{326.9}{3(531)} = 0.205 \times 1.3 = 0.267$

from Nilson Graph A.15 need $\rho_g = 0.01$

$\rho_g = \frac{8}{531} = 0.015$

$\rho_g > \rho_{g\,req}$ ∴ o.k.