Technical Assignment 1

Medical Office Building

Brendon J. Burley
Structural Option
October 6, 2004

Advisor:
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Medical Office Building
Malvern, PA

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Executive Summary

The structural system of the Medical Office Building is explained and analyzed in this report. The structural system itself is a concrete moment frame on strip footings. Flat one-way slabs span the beams and serve as the floor system. Loads from the floor are transferred to the beams and then to the columns where they are carried to the footings and then the ground. The loads used in the original design of this building have been reevaluated to take into account more recent information.

The original code this building was designed under was BOCA 1997. As a reference to a more modern code, BOCA 1999 was chosen as the major building code. Many of the sections in BOCA refer to ASCE 7. In most cases the data in ASCE7-02 outdates BOCA 1999 and therefore was substituted as necessary. Live loads, roof loads, snow loads and earthquake loads were all calculated in accordance with ASCE 7-02. Wind loads were calculated per BOCA 1999. These results were unexpectedly low and deemed unacceptable for design by the code, therefore a wind pressure of 10 psf was used for the entire building's wind load. The other loads seemed relatively accurate with a net earthquake shear force of 3060 k, a live roof load of 12 psf and a snow load of 19 k.

These loads were used to generate sketches of load distributions. An analysis of these distributions was used to check the agreement between the older code and newer code and spot mistakes in determining the loads. Three system components were tested; a slab, a beam, and a column. The slab had good agreement in both load magnitude and design. The ultimate moments came out to be 17.3'k and 16.8'k for old and new respectively. The new slab was 2" thicker and used more rebar despite having less capacity demand. The beam check did not yield as good results as the slab.

The beam ultimate moment under the new loads was 848.4'k, well above the old 490'k. To further complicate the matter the new beam is also smaller and uses less steel than the original design. It is possible that architecture controlled the decision on beam size, but that does not explain the large discrepancy in the ultimate moment. The column design also had a large size discrepancy, but this is likely due to the analysis of the column only including the gravity loads. The new design was a 16" diameter column with 8 #8's and the old design was a 26" diameter column with 8 #10's. The use of live load reduction factors in the new design may have pushed the column down in size as well.

Since the beam did not check out it will be very important to look at the calculations leading to the ultimate moment under the new code. Once the loads are fixed design of alternate structural systems for the Medical Office Building will commence.

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Introduction

The following report describes the structural system of the Medical Office Building and presents the results of a preliminary analysis of that system. The report is divided into four sections:

- Structural System
- Relevant Codes
- Loads
- Spot Checks

The structural systems section describes in detail the systems used to resist both gravity and lateral loads. The relevant codes section lists those codes used to determine the loads and handle the calculations for the spot checks. The loads section provides information on the application and magnitude of the design loads. The spot checks section describes the results of rudimentary designs performed on typical members. Details of all the calculations used to obtain this data can be found in the appendices.

Structural System

The structural system of the Medical Office Building is composed of several concrete moment frames. These moment frames are comprised of cast-in-place concrete columns and beams. The beams mostly run parallel to each other and are spanned by monolithically cast concrete slabs. At the ground a slab on grade serves as the floor, but the columns carry the load below the slab on grade to strip footings, which run perpendicular to the girders. A schematic of the system can be seen in figure 1. A detailed description of each component follows below.

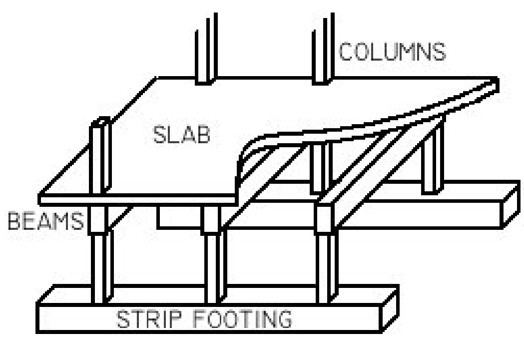


Figure 1 – Schematic of Structural System

Foundation

The foundation of the medical office building consists of strip footings and foundation walls. The footings vary in width from as narrow as 3'-6" to as wide as 9'-0" and are typically 3'-0" deep. The notes call for the concrete in the footings to have a compressive strength of 4000 psi and the steel to be grade 60. The foundations carry loads directly from the columns at all parts of the building, but due to slope variations around the building foundation walls are placed to retain the earth.

The foundation walls are also made of 4000 psi concrete with grade 60 steel reinforcement. These walls are typically 1'-0" thick at the base and are keyed into the footing. The foundation walls have a 4" brick ledge cut into them once they rise above grade.

Columns

Concrete columns carry all the loads of the building to the foundation. The columns are lined up on column lines which are spaced at 28'-0" in both the longitudinal and latitudinal directions. Three column lines are placed between the grids to follow the architectural taper of the building. There are also ten radial column lines that trace the curve of the southwest face of the Medical Office Building.

Each of the columns is designed for 4000 psi concrete reinforced with grade 60 steel reinforcement. Typically, all of the internal columns are round with a 26" diameter. Their reinforcing consists of several vertical bars that are tied by a circumferential bar. The exterior columns tend to be rectangular in shape and are reinforced by vertical bars tied cross wise and along the perimeter. In general a columns does not change shape between stories, but there are some exceptions.

Along the southwest face of the Medical Office Building, the columns change shape along the curved façade. Columns at the base are rectangular while the columns above are circular. This transition also happens at parts of the building where the brick veneer rises more than one story. At the lower story, the columns are square, but once the brick veneer ends these columns become round.

Beams

The concrete beams are also composed of 4000 psi concrete with grade 60 steel reinforcing. Most of the beams run from north to south (longitudinal). However, near the mechanical rooms and stairways additional beams run between the longitudinal beams. These latitudinal beams are identified in figure 2. The latitudinal beams are usually smaller than the longitudinal beams they span, but they may provide additional stiffness for lateral loads.

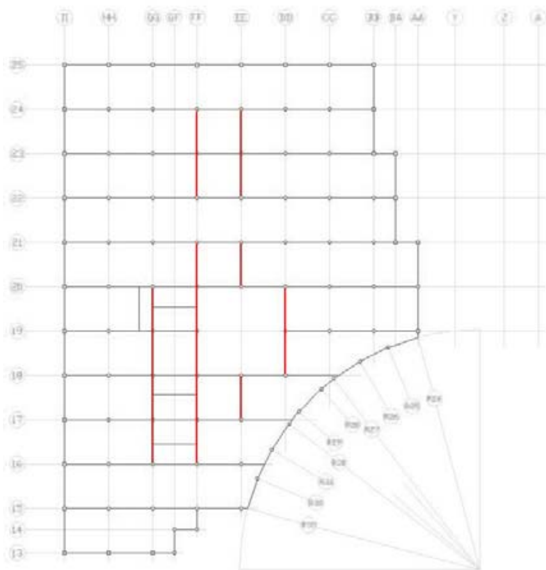


Figure 2 – Typical Plan: Latitudinal beams highlighted

The longitudinal beams themselves are very wide. Each beam has a width of 8'-0" and a depth of only 18". The top reinforcing in the beam is typically #9 bars. The beam details show bottom reinforcing in the beams, however the plans do not call for any bottom reinforcement. The details also call for the beams to be cambered ¾”.

Slabs

There are two types of slabs in the Medical Office Building. The first type is the slab on grade that serves as the flooring for the lowest level and the part of the second level of the building. The slab on grade is designed for 3000 psi concrete and grade 60 steel reinforcement. The typical reinforcement is #5 bars every 12” in the top part of the slab. The other slabs are those cast with the beams.

The slabs above grade were designed as one-way slabs. These slabs consist of 4000 psi concrete with grade 60 steel reinforcing. In general these slabs are 9” or 10” thick. They are reinforced by welded wire fabric and by rebar. The rebar typically only extends slightly past the edge of the beam and are usually #8 bars.

Conclusions

Monolithic concrete structures always absorb some lateral load due to the inherent stiffness in the connections. However, this does not mean that another system could not be used. Given that the slabs spanning the beams are thick and no joist system was used, it seems unlikely that the latitudinal beams are part of a shear wall system. The small size of these beams relative to their longitudinal partners also suggests that their purpose is local support. Despite this, they still create sections of the building that are relatively stiffer than other sections of the same building. This may cause a concentration of wind load along the frames containing these beams and result in a design where these latitudinal beams did not appear.

Relevant Codes

The Medical Office Building was constructed in East Whiteland Township in 1999. At that time, the township recognized BOCA 1997 with local modifications as their building code. For the purposes of comparison, BOCA 1999 was chosen as the building code for this report. The analyses in this report were performed using Load Resistance Factored Design (LRFD). However, given that the building was designed in the mid 90’s and the relative newness of LRFD, it seems likely that the original designers used Allowable Stress Design (ASD). The use of these different techniques will result in certain size differences. It is also important to note that BOCA 1999 references ASCE 7 for load calculations. For convenience ASCE 7-02 was used instead of the relevant BOCA 1999 for load calculations.

The following table summarizes the codes used in creating this report.

Code \ Standard	Section	Description
BOCA 1999	-	Model Code
	1609	Wind Load
ASCE 7-02	2.3.2	Load Combinations
	4.2.1	Live Load Requirements
	4.8	Live Load Reduction
	4.9.1	Roof Live Load Calculation
	7.3	Snow Loads
	9	Earthquake Loads
AIC 2002	-	Code for concrete design
ASTM	-	Standards for testing materials

Loads

The American Society of Civil Engineers has set several standards for the loads that are to be applied when designing buildings. These codes take into account the layout of the site and the use of the building. The Medical Office Building is a mixed-use facility that is mostly open office space. The basement of the facility houses a flexible Auditorium where people gather for conferences or large company meetings. The building itself is located in a hilly corporate complex with 4 other buildings nearby. The other buildings provide some shelter from the wind, but are not closely packed enough to create a cityscape. Using this information the loads on the Medical Office Building are as follows:

Live Load

The following live loads are the minimum required by ASCE for the spaces in the Medical Office Building.

Lobbies & 1st floor corridors	100 psf
2nd floor corridors	80 psf
Offices	50 psf
Stairwells	100 psf

Because the majority of the floor is open office space, the corridors could be placed anywhere. For convenience, all floors could be treated as corridors. Also, movable partitions can be taken as an additional 10 psf. In effect the load on floors above ground becomes 90 psf. For the sake of simplicity in calculations, this can be rounded to 100 psf of live floor load.

Roof Live Load

Minimum roof live load is defined by ASCE 7-02 equation 4.9.1:

$$L_r = 20R_1R_2$$

Where R_1 and R_2 are based on the characteristics of the roof. The Medical Office Building has wide spans and a flat roof, therefore R_1=0.6 and R_2=1.0. This makes the total roof live load equal to 12 psf.

Snow Load

The snow load on a building is a function of the ground snow load. ASCE 7-02 modifies the ground snow load by factors for wind, heat of the building, and importance of the building. The ground snow load on site is approximately 30 psf. Because the building is normally heated and not exceptionally important the only modifier it gets is for wind, which is based on the site. Using the equation in section 7.3 of ASCE 7-02:

$$p_f = 0.7C_eC_tIp_g$$

the snow load is determined to be 19 psf.

Wind Load

The wind loads were calculated based on BOCA 1999's reporting of ASCE 7-95. There are several factors influencing the windward pressure on a building. Figures 3 and 4 represent the wind pressure and wind loads on two building sections. The full calculations are viewable in the appendix.

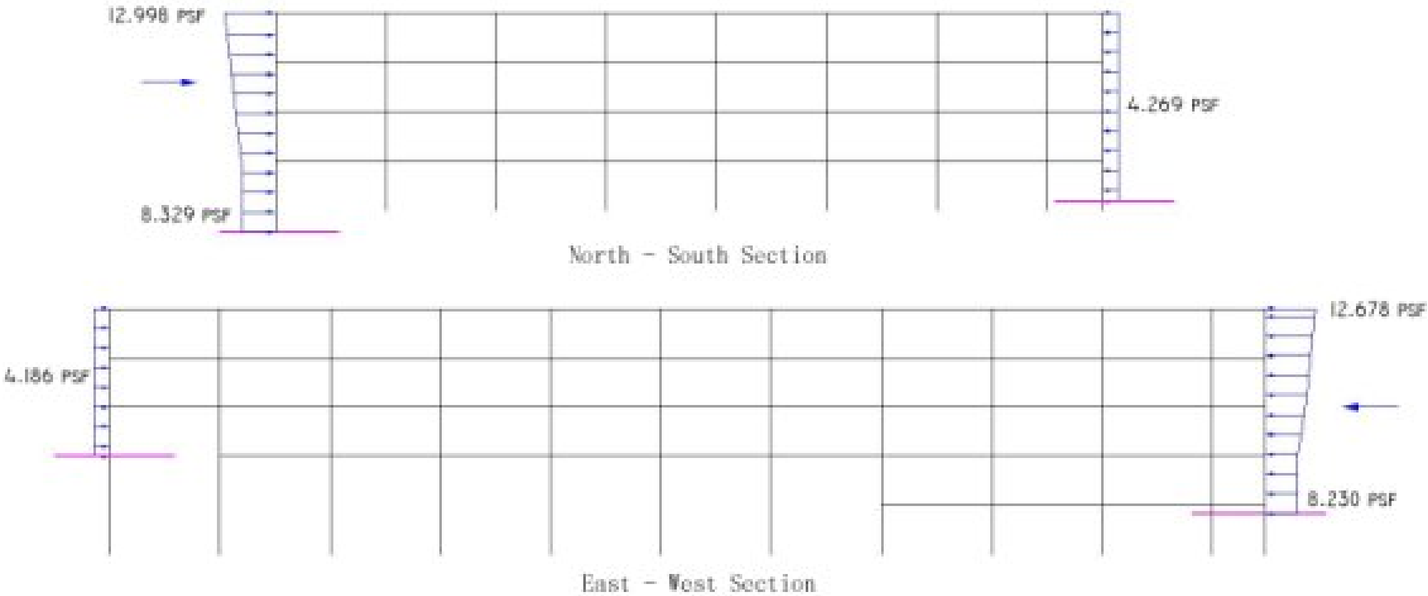


Figure 3 – Wind Pressure Diagrams

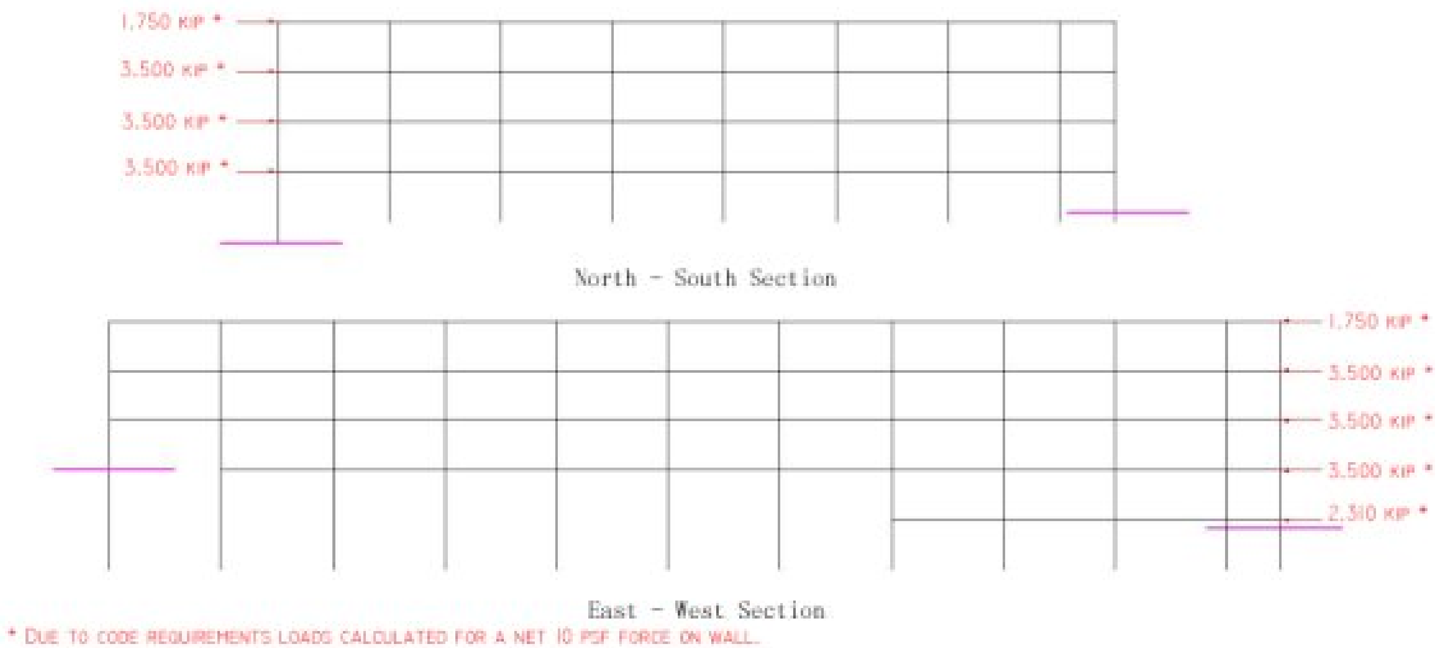


Figure 4 – Wind Load Diagram

Earthquake Loads

According to BOCA 1999, the occupancy level and low seismic accelerations associated with the Medical Office Building do not require design for earthquakes. The Simplified Analysis method described in Section 9.5.4 of ASCE 7-02 was used to perform an analysis of the earthquake load.

Based on observation of the boring logs, the Site Class C was chosen for the earthquake calculations. The base shear was calculated as 3160 kips. Considering that there are 115 columns, each column would have to take 27.5 k if the load were distributed evenly. This value is substantially higher than the wind load shear, which raises suspicions regarding the validity of the wind pressure profile. The earthquake loads are summarized in figure 5.

Figure 5 – Earthquake Load Diagram

Spot Checks

Using the loads derived in the previous section and the codes and standards stated earlier, typical structural elements were redesigned to check for matching with the existing conditions. The elements checked were:

- Slab that spans column lines 21 and 22 on level 337
- Beam on column line 22 between column lines FF and EE on level 337
- The column located at the intersection of column line 22 and column line FF

Slab

Analysis of the one-way slab began by treating the slab as a 1' wide strip. In this form the slab can be treated as a beam. A self-weight of 125 plf was assumed and the live load was taken as 100 psf (80 psf 2nd story corridor + 20 psf partitions & miscellaneous). Next, adjacent slabs were placed side by side as part of a continuous beam. Ignoring the effects from slabs further down the frame, the slabs on either side of the slab of interest were used to develop load patterns.

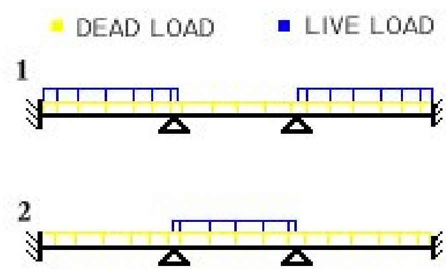


Figure 6 – Load Patterns

The moments of the central span were determined through a moment distribution analysis. The stiffness of the slabs was assumed to be equal for this purpose. Separate moments were calculated for the dead load and the two live load conditions so that load factors could be combined later. After calculating the gravity loads the moments due to wind were calculated from a portal analysis (see Wind Load in Appendix) and load factors were applied to maximize the combined moments.

The combined moments were compared to determine the worst-case positive and negative moments in the slab. The calculations showed M_-=16.8'k and M_+= 1.414'k. Compared to the original design, which shows M_u=17.3'k/ft, the ultimate loads seem accurate. The discrepancy in the numbers is most likely due to estimations in the current analysis. Despite this good agreement in load, the design is somewhat different.

Based on AIC Section 9.5.2 the minimum slab thickness for continuous spans is 1/28th of the clear span. Since the girder that the slab rested on was not designed, the clear span was taken to be 28', resulting in a 1' deep slab instead of the 10" slab in the original design. The reinforcing was similarly mismatched with the new design requiring 28 #3's on top instead of 4 #8's on top. This discrepancy may have to do with a change in the concrete code, but a portion of it may be due to the shorter span of the original slab.

Even with the differences in the final design, the agreement between the ultimate moments signifies that the location checks ok.

Beam

The beam was designed using the same technique as the slab. The beam's load was determined by tributary area using the previously designed slab to calculate dead load. In addition to the dead load, the beam self-weight was assumed to be 500 plf. Using the same load patterns as above (Figure 6) and assuming equal stiffness in the girders the moments of the central span were calculated. Once again a portal analysis (see Wind Load in Appendix) was used to determine the moment contribution of wind on the beam.

Summing all the moments with their appropriate load factors yielded M_{-}= 848.4'k and M_{+}= -145.8'k. This is wildly different than the M_u=490'k calculated by the original designers. The cause of such a large discrepancy is unlikely something predictable. An error in calculations may be the cause of such a difference. Despite this failed comparison, the beam was designed anyways. Because the positive moment is negative, it was assumed that bottom reinforcing was unnecessary. Top reinforcing was determined to be 10 #10's, which easily fit into the 60" wide and 20" deep beam. The original beam is 96" wide and 18" deep and has 13 #9's as its top supports. Interestingly, the new beam design only needs to be 76" wide if the height is limited to 18". This suggests that the original beam may have been over designed for architectural reasons.

In this case neither the moment nor the design match, so the check is not ok.

Column

The column was analyzed as a gravity only system. The actual design of a column would require that uplift on the roof be considered and that overturning moments be considered on the exterior walls. It would also be important to take into account drift that would cause eccentricity in the columns. In addition to these over sights, live load reductions from ASCE 7-02 section 4.8 were used to calculate the maximum axial load. Since the column being tested is an interior column that, excluding beam heights, is only 11' tall and is doubly fixed, it was assumed to be a short column.

After the appropriate load factors and live load reductions, the ultimate axial strength was 548.5k. Based on ACI 10.9.1 a steel to concrete ratio of .04 to 1 was chosen to enter design. The analysis yielded a 16" diameter column with 8 #8's. Although the new column does not match the 26" diameter column with 8 #10's that was originally designed, the result may still be good. There is no way of knowing if the original designer used live load reductions themselves. Also by neglecting any lateral loads, the new column design should be expectedly smaller.

Given the margin of error from the assumptions made, it should be okay to say that the column checks out.

Conclusions

This report establishes the fundamental of an ongoing design process. Now that the loads have been determined a serious design process can proceed. The foundation system still needs to be designed, which involves designing the column connections and the footings. Wind effects such as overturning and uplift must be considered. The bad check on the beam suggests that a closer look be taken at the calculations leading to the design moments. Another consideration that was overlooked in this report is that of the atrium roof, which uses a steel truss structural system. Additional codes and analysis will be required to work on the atrium. With the knowledge gained from this report, work can begin on designing alternate systems to support the Medical Office Building.

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 A

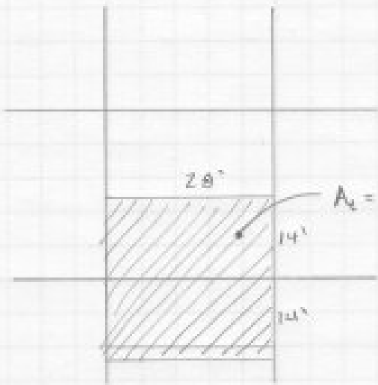
Roof Loads | | Brendon J. Burley | 1/1

Live Load:

$L_r = 20 R_1 R_2$

R_1 - based on tributary Area
R_2 - roof pitch

pitch = 0 (flat roof) $R_2 = 1.0$



$A_t = 28 \times (14+14) = 784 \text{ ft}^2 \quad > 600 \text{ ft}^2 \quad \therefore R_1 = .6$

$L_r = 20(.6)(1.0) = \boxed{12 \text{ psf}}$

Snow Load:

$p_f = 0.7 C_e C_t I p_g$

$p_g = 30$ psf
$I = 1.0$ (no extra danger to humans)
$C_T = 1.0$ (Building heated above freezing)
$C_e = 0.9$ (Exposure Class B)

$p_f = 0.7(0.9)(1.0)(1.0)(30) = 18.9 = \boxed{19 \text{ psf}}$

Earthquake Load | Brendon J. Burley | 1/1

$V = \frac{1.2\ S_{DS}}{R} W$ Ordinary reinforced concrete frame

$R = 3$

$F_x = \frac{1.2\ S_{DS}}{R} W_x$ $S_{DS} = 2/3\ S_{MS}$

$S_{MS} = F_a S_S$

Choose Class C - Dense soil and soft rock

* Site has a lot of rock near surface

$S_S \approx .20g$ - from map

From table 9.4.1.2.4a ASCE 7-02 $F_a = 1.2$

$S_{MS} = 1.2\ (0.20g) = 0.24$

$S_{DS} = 2/3\ (0.24g) = 0.16$

Level	Weight	Area	Dead Load	Partition
349.5	9275 k	54558 ft²	150 psf	20 psf
337.0	9275 k	54558 ft²	150 psf	20 psf
324.5	9753 k	57372 ft²	150 psf	20 psf
312.0	9796 k	57624 ft²	150 psf	20 psf
299.5	9796 k	57624 ft²	150 psf	20 psf
287.0	1482 k	8720 ft²	150 psf	20 psf
Total	49378 k			

Partition Load is 20 psf from live load assumptions.

W = Area (DL + Partition)

$V = \frac{1.2\ (.16)}{3} (49378) = 3160.192\ k = 3160\ k$

$F_{349.5} = \frac{1.2(.16)}{3} (9275) = 593.6\ k = 594\ k$

$F_{337.0} = F_{349.5} = 594\ k$

$F_{324.5} = \frac{1.2(.16)}{3} (9753) = 624.192\ k = 624\ k$

$F_{312.0} = \frac{1.2(.16)}{3} (9796) = 626.944\ k = 627\ k$

$F_{299.5} = F_{312} = 627\ k$

$F_{287.0} = \frac{1.2(.16)}{3} (1482) = 94.948\ k = 95\ k$

Wind Load | Brendon J. Burley | 1/3

Sample Wind Loads for Column Lines FF and 22
BOCA 1999

Windward Pressure : $P = P_v I (K_z G_h C_p - K_h (GC_{pi}))$

Leeward Pressure : $P = P_v I (K_h G_h C_p - K_h (GC_{pi}))$

$V = 70$ mph , Exposure B , $I = 1.15$

Ceiling Elev = 351'-6"
West grade @ FF = 299'-6" ← controls $B = 224'$
East grade @ FF = 312'-0" $L = 308'$
South grade @ 22 = 301'-6" $B = 308'$
North grade @ 22 = 295'-11" ← controls $L = 224'$

$H_{west} = 351'\text{-}6" - 299'\text{-}6" = 52'\text{-}0" \rightarrow \frac{H}{L} = .17 \; ; \frac{H}{B} = .23$
$H_{north} = 351'\text{-}6" - 295'\text{-}11" = 55'\text{-}7" \rightarrow \frac{H}{L} = .25 \; ; \frac{H}{B} = .18$

West
Windward $C_p = 0.8$
Leeward $C_p = -0.5$
Roof $C_p = -0.7$
$K_h = .64$
$P_v = 12.5$ psf
$G_h = 1.41$
$GC_{pi} = 0.25$ or -0.25

North
Windward $C_p = 0.8$
Leeward $C_p = -0.5$
Roof $C_p = -0.7$
$K_h = .66$
$P_v = 12.5$ psf
$G_h = 1.40$
$GC_{pi} = 0.25$ or -0.25

West Leeward Wall
$P = 12.5(1.15)[(.64)(1.41)(-0.5) - .64(-.25)]$ or $P = 12.5(1.15)[(.64)(1.41)(-0.5) - .64(.25)]$
$P = -4.186$ psf $P = -8.786$ psf

West Windward Wall @ 52'
$P = 12.5(1.15)[(.64)(1.41)(0.8) - .64(-.25)]$ or $P = 12.5(1.15)[(.64)(1.41)(0.8) - .64(.25)]$
$P = 12.678$ psf $P = 8.078$ psf
$GC_{pi} = -0.25$ is critical

@ 50'	12.515 psf
@ 40'	11.543 psf
@ 30'	10.408 psf
@ 25'	9.759 psf
@ 20'	9.110 psf
@ ≤15'	8.230 psf

Net pressure < 10psf
use 10 psf for design

North Leeward Wall
$P = 12.5(1.15)[.66(1.40)(-0.5) - .66(-.25)]$ or $P = 12.5(1.15)[.66(1.40)(-0.5) - .66(.25)]$
$P = -4.269$ psf $P = -9.013$ psf

North Windward Wall @ 55'-7"
$P = 12.5(1.15)[.66(1.40)(0.8) - .66(-.25)]$ or $P = 12.5(1.15)[.66(1.40)(-0.5) - .66(.25)]$
$P = 12.998$ psf $P = 8.254$ psf
$GC_{pi} = -0.25$ is critical

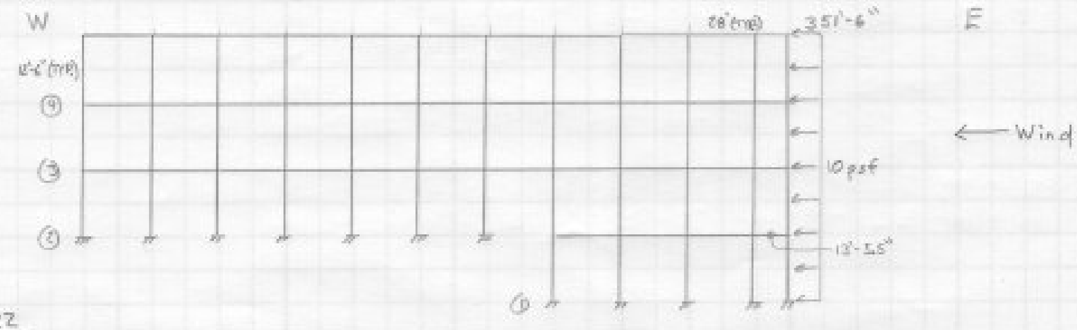
Wind Load | | Brendon J. Burley | 2/3

North Windward Wall

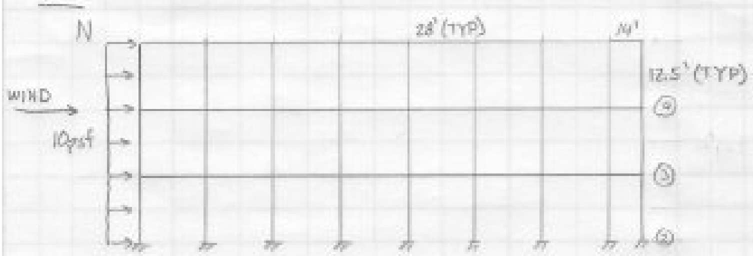
@ 50'	12.515 psf
@ 40'	11.549 psf
@ 30'	10.422 psf
@ 25'	9.778 psf
@ 20'	9.134 psf
@ ≤15'	8.329 psf

Net pressure < 10 psf
use 10 psf for design

FF



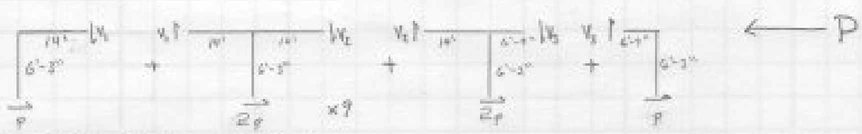
ZZ



Approximate Tributary Width = 28' for both frames

Portal Analysis

FF Top :



$P = 10 \times 28 \times 6.25 = 1750$ lbs

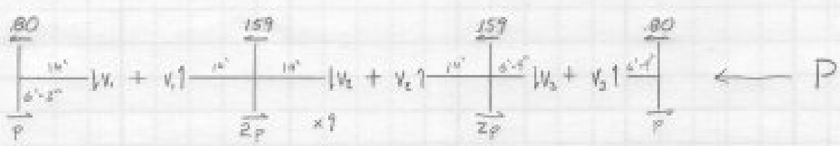
$p = P/22 = 80$ lbs

$\circlearrowleft + \Sigma M = 0 = 14V_1 - 6.25P \Rightarrow V_1 = 36$ lbs

By Symmetry $V_1 = V_2$

$\circlearrowleft + \Sigma M = 0 = 14V_2 + 6.75V_3 - 2P \cdot 6.25 \Rightarrow V_3 = 74$ lbs

FF ④ :



$P = 10 \times 28 \times 12.5 = 3500$

$p = P/22 + 80 = 239$ lbs

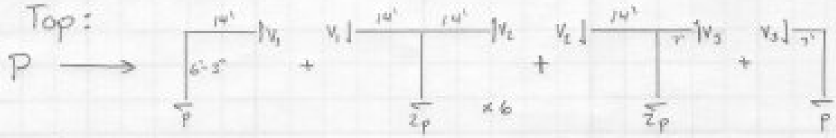
$\circlearrowleft + \Sigma M = 0 = 14V_1 - 6.25P - 6.25(80) \Rightarrow V_1 = 142$ lbs

$\circlearrowleft + \Sigma M = 0 = 6.75V_3 - 6.25P - 6.25(80) \Rightarrow V_3 = 295$ lbs

Symmetry $V_1 = V_2$

Wind Load Brendon J. Burley $3/3$

ZZ Top:



$P = 10 \times 28 \times 6.25 = 1750$ lbs

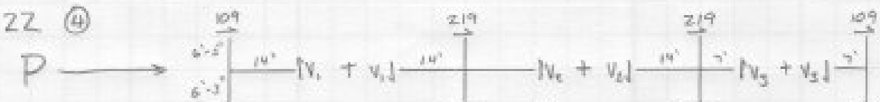
$p = P/16 = 109$ lbs

$\circlearrowleft + \Sigma M = 0 = 6.25P - 14V_1 \Rightarrow V_1 = 49$ lbs

Symmetry $V_1 = V_2$

$\circlearrowleft + \Sigma M = 0 = 6.25P - 7V_3 \Rightarrow V_3 = 98$ lbs

ZZ ④



$P = 10 \times 28 \times 12.5 = 3500$ lbs

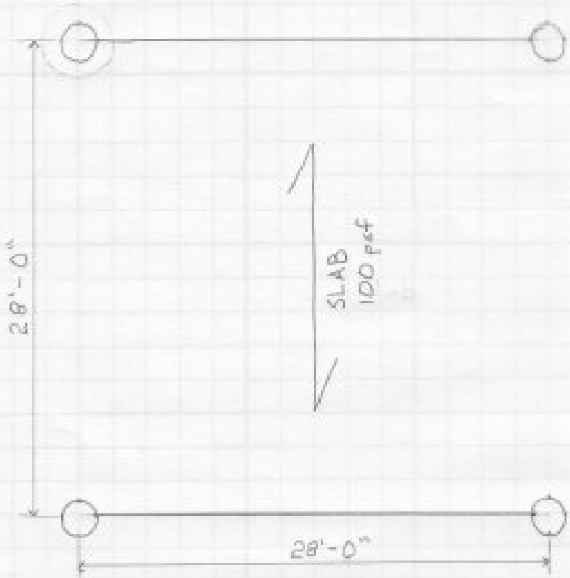
$p = P/16 + 109 = 328$ lbs

$\circlearrowleft + \Sigma M = 0 = 109(6.25) - 14V_1 + 6.25P \Rightarrow V_1 = 195$ lbs

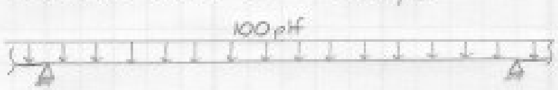
Symmetry $V_1 = V_2$

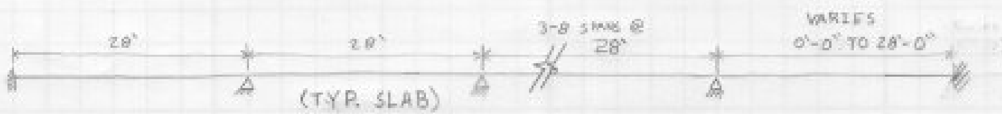
$\circlearrowleft + \Sigma M = 0 = 109(6.25) + 6.25P - 7V_3 \Rightarrow V_3 = 391$ lbs

Appendix B



Estimate slab as 1' strips



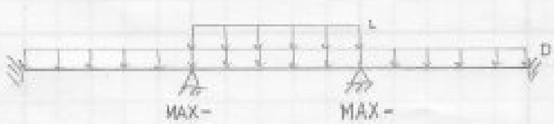


Estimate As :

Approximate Self Wgt. as 125 plf

Load Patterns

①



②

Typical Floor | Brendon J. Burley | 2/3

Assume Slabs are all equally stiff ∴ $D_{BA} = D_{BC} = D_{CB} = D_{CD} = .5$

Dead

	A	B		C		D
DF	0	.5	.5	.5	.5	0
	-8.167	8.167	-8.167	8.167	-8.167	8.167

$M_D = \frac{wL^2}{12} = \frac{.125(28)^2}{12} = 8.167$'k

$M_L = \frac{wL^2}{12} = \frac{.100(28)^2}{12} = 6.533$'k

① Live

DF	0	.5	.5	.5	.5	0
	-6.533	6.533	0	0	-6.533	6.533
		-3.267	-3.267			
	-1.633			-1.633		
				4.083	4.083	
			2.042			2.042
		-1.021	-1.021			
	-0.510			-0.510		
				0.255	0.255	
			0.128			0.128
		-0.064	-0.064			
	-0.032			-0.032		
				0.016	0.016	
			0.008			-0.008
		-0.004	-0.004			
	-0.002			-0.002		
				0.001	0.001	
	-8.710	8.177	-2.178	2.178	-2.178	8.711

Possible Load Combinations

1.4D

1.2D + 1.6L + 1.0W

1.2D + 0.5L + 1.3W

From Wind Load: FF ④

$V_2 = 142$ lbs.

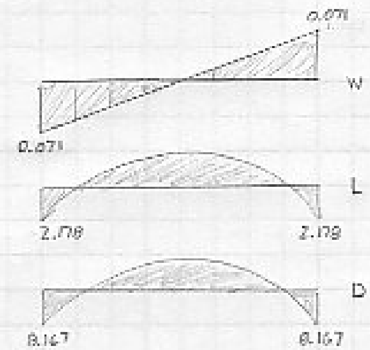
∴ $M_{overturn} = 14 V_2 = 1.988$'k

Since slab acts as beam, the 1' strip takes

$M_{slab} = 0.071$'k

Assume Wind can be inversed.

①

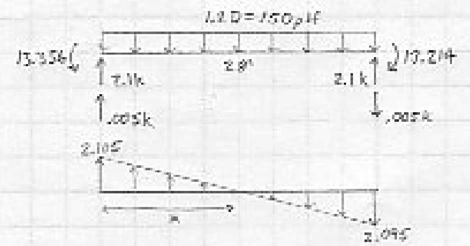


② Live

DF	0	.5	.5	.5	.5	0
	0	0	-6.533	6.533	0	0
		3.267	3.267			
	1.633			1.633		
				-4.083	-4.083	
			-2.042			-2.042
		1.021	1.021			
	0.510			0.510		
				-0.255	-0.255	
			-0.128			-0.128
		0.064	0.064			
	0.032			0.032		
				-0.016	-0.016	
			-0.008			-0.008
		0.004	0.004			
	0.002			0.002		
				-0.001	-0.001	
	2.177	4.356	-4.355	4.355	-4.355	-2.178

By observation 1.2D + 1.6L + 1.0W controls

$M_L = -13.356$'k $M_R = 13.214$'k

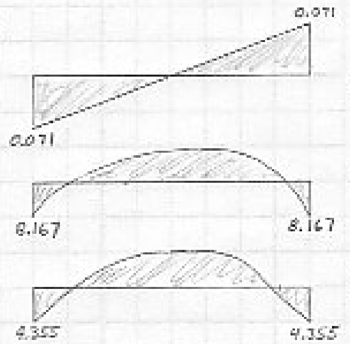


$V = 2.105 - .150x = 0 \Rightarrow x = 14.033$'

$M_+ = \frac{1}{2}(2.105)x - 13.356 = 1.414$'k

Typical Floor | Brendon J. Burley | 3/3

②



Use $1.2D + 1.6L + 1.0W$

$M_L = -16.839$'k $\quad M_R = 16.697$'k

Slab Design

ACI 9.5.2 Minimum Slab Thickness

for both ends continuous $h_{min} = \frac{l}{28}$

$h_{min} = \frac{28(12)}{28} = 12"$ $\quad d \approx 11"$

Using Grade 60 Rebar and 4000 psi concrete

$\rho_{max} = .85\beta_1 \frac{f'_c}{f_y} \frac{.003}{.003+.004} = .85(.85)\frac{4}{60}(\frac{3}{7}) = 0.0206$

Assume $\phi = .9$

$M_u = \phi M_n = \phi \rho f_y bd^2(1 - .59\rho \frac{f_y}{f'_c}) = .9\rho(60)(12)(11)^2(1 - .59\rho\frac{60}{4}) = 16.839 \times 12$

$78408\rho - 693910.8\rho^2 = 202.068$

$\rho = 0.0026 < 0.0206$

$\rho = \frac{A_s}{bd} = \frac{A_s}{12(11)} \Rightarrow A_s = .348 \text{ in}^2$ $\quad$ Try #3's @ 1'-0" spacing Top

For bottom bars $M_u = 1.414$'k $= 16.968$"k

$\therefore 78408\rho - 693910.8\rho^2 = 16.968$

$\rho = 0.0002$

$A_s = \rho bd = 0.0002(12)(11) = 0.029 \text{ in}^2$ $\quad$ Try #3's @ 12'-0" spacing Bottom

but spacing $< 3h = 36" \not< 18"$

Try #3's @ 18" spacing Bottom

Girder

Brendon J. Burley 1/2

Analysis as Slab (see Typical Floor) except as follows:

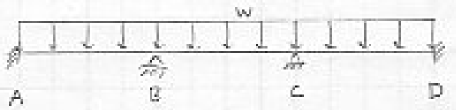
Assume Self Weight = 500 plf

TA = 28' ∴ L = 2800 plf

Slab Weight = 28' × 1' × 150 = 4200 plf

$M_D = \frac{wl^2}{8} = \frac{(4.200 + .500)(28)^2}{8} = 460.6'k$

$M_L = \frac{wl^2}{8} = \frac{2800\,(28)^2}{8} = 274.4'k$



	A	B	B	C	C	D
DF	0	.5	.5	.5	.5	0
Dead	-460.6	460.6	-460.6	460.6	-460.6	460.6
①	-274.4	274.4	0	0	-274.4	274.4
		-137.2	-137.2	137.2	137.2	
	-68.6		68.6	-68.6		68.6
		-34.3	-34.3	34.3	34.3	
	-17.2		17.2	-17.2		17.2
		-8.6	-8.6	8.6	8.6	
	-4.3		4.3	-4.3		4.3
		-2.1	-2.1	2.1	2.1	
	-1.1		1.1	-1.1		1.1
		-.5	-.5	.5	.5	
	-.3		.3	-.3		.3
		-.1	-.1	.1	.1	
	-365.9	91.6	-91.3	91.3	-91.6	365.9
②	0	0	-274.4	274.4	0	0
		137.2	137.2	-137.2	-137.2	
	68.6		-68.6	68.6		-68.6
		34.3	34.3	-34.3	-34.3	
	17.2		-17.2	17.2		-17.2
		8.6	8.6	-8.6	-8.6	
	4.3		-4.3	4.3		-4.3
		2.1	2.1	-2.1	-2.1	
	1.1		-1.1	1.1		-1.1
		.5	.5	-.5	-.5	
	.3		-.3	.3		-.3
		.1	.1	-.1	-.1	
	91.5	182.8	-183.1	183.1	-182.8	-91.5

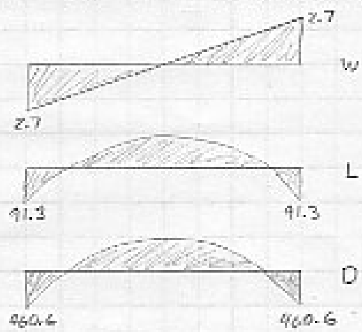
From Wind Load: 22 ④

$V_t = 195$ lbs

$M_W = .195\,(14) = 2.730'k$

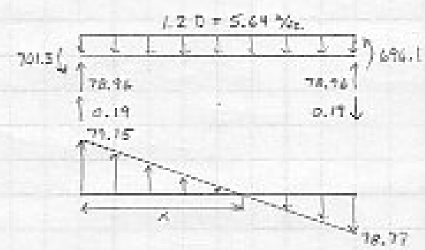
Assume wind can be inversed

①



By observation use 1.2D + 1.6L + 1.0W

$M_L = -701.5'k$ $M_R = 696.1$



$79.15 - 5.64x = 0 \Rightarrow x = 14.034$

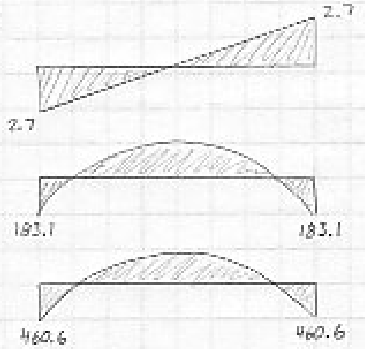
$M_+ = \frac{1}{2}(79.15)(14.034) - 701.5 = -145.8'k$

Girder | Brendon J. Burley | 2/2

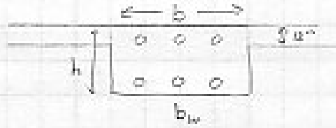
(2)

Using $1.2D + 1.6L + 1.0W$

$M_L = -848.4'k$ $M_R = 843.0'k$



Girder Design



$b = \frac{L}{4} = \frac{28}{4}(12) = 84"$ ⇐ controls

$b = b_n = 28(12) = 336"$

$b = b_w + 16h_s = b_w + 16(12) = b_w + 192"$

Design for $M_u = -848.4'k$ first using Grade 60 steel and 4000 psi concrete.

$\rho_{max} = .85\beta_1 \frac{.003}{.003+.004}\frac{f'_c}{f_y} = .85(.85)\frac{3}{7}\frac{4}{60} = 0.0206$

$\rho = .6\rho_{max} = 0.0124$

Assume $\phi = .9$

$M_u = \phi M_n = \phi \rho f_y bd^2\left(1 - .59\rho\frac{f_y}{f'_c}\right) = .9(.0124)(60)bd^2\left(1 - .59(.0124)\frac{60}{4}\right) = 848.4 \times 12$

$bd^2 = 17078.5 \text{ in}^3$

Say that $d = 15" \Rightarrow b = 75.9" = 76"$

if $d = 17" \Rightarrow b = 59.1" = 60"$ ← choose this one

if $d = 21" \Rightarrow b = 38.7" = 39"$

$A_s = \rho bd = 0.0124(17)(60) = 12.648 \text{ in}^2$

Try 10 #10's Top

Assume 3" cover to center of bar

$b_{min} = 2(3) + 10(1.27) + 9(1.27) = 30.13" < 60"$ ∴ ok

$T = C$

$A_s f_y = .85 f'_c b a \Rightarrow a = \frac{A_s f_y}{.85 f'_c b} = \frac{12.7(60)}{.85(4)(84)} = 2.67" < 12"$

∴ $M_n = A_s f_y\left(d - \frac{a}{2}\right) = 12.7(60)\left(17 - \frac{2.67}{2}\right) = 11937.47"k = 994.8'k$

$\rho = \frac{A_s}{bd} = \frac{12.7}{60(17)} = 0.0125$

$\rho_{.005} = .85\beta_1 \frac{.003}{.003+.005}\frac{f'_c}{f_y} = .85(.85)\frac{3}{8}\left(\frac{4}{60}\right) = 0.0181 > 0.0125$

∴ $\phi = .9$

$\phi M_n = .9(994.8) = 895.3'k > 848.4'k$ o.k.

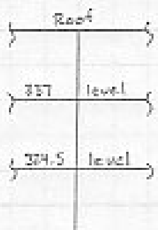
No bottom reinforcing needed because $M_+ < 0'k$

Column | | Brendon J. Burley | 1/1

Choose Column FF-22

Assume every story has a 150 psf D.L.
Use column tributary area to estimate gravity loads.
Neglect lateral loads

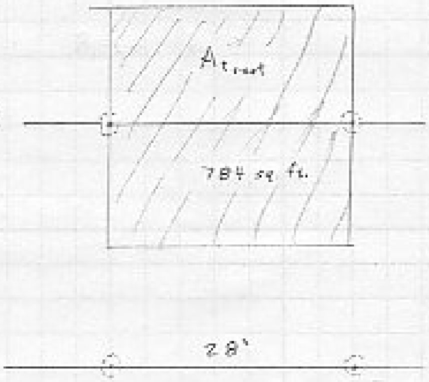
$A_t = 28' \times 28' = 784$ sq. ft.
$K_{LL} = 4$ for interior columns



Roof LL: $L_r = 20 R_1 R_2$

$A_{t\,roof} > 600$ sq. ft. => $R_1 = .6$
Flat roof $R_2 = 1$

$L_r = 12$ psf



Snow: $p_f = .7 C_e C_t I p_g$
$p_g = 30$ psf
Category B => $C_e = 0.9$
$C_t = 1.0$
Category II => $I = 1.0$

$p_f = 18.9$ psf $= 19$ psf

Level	Total TA	Dead	Live	Adjusted Live
Roof	784	117600	9408	9408
337	1568	235200	125440	50176
324.5	2352	352800	188160	75264

← Cannot Adjust L_r < 100 psf
$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_t}}\right) \geq .4$

Using $1.2D + 1.6L + .5L_r$ → $P_{324.5} = 548.5\ k$ ← controls
Using $1.4D$ → $P_{324.5} = 493.9\ k$

ACI 10.9.1 $0.08 \geq \frac{A_s}{A_g} \geq 0.01$ Use $A_s = 0.04 A_g$
Choose spirally reinforced column, Grade 60 Steel, 4000 psi conc.

$P_u = \phi P_n = .85\,\phi(.85 f'_c (A_g - A_s) + f_y A_s)$
$548.5 = .85(.7)(.85(4)(A_g - .04A_g) + 60(.04A_g))$
$A_g = 162.76\ in^2$
If round $A_g = \pi d^2/4$ => $d = 14.4"\ \phi$ use 16"
$A_s = .04(162.76) = 6.51\ in^2$
Choose 8 #8's spaced evenly
$A_g = 201\ in^2$ $A_s = 6.32\ in^2$
$\phi P_n = .85(.7)(.85(4)(201 - 6.32) + 60(6.32)) = 619.6\,k > 548.5\,k$ o.k.

16" ϕ column w/ 8 #8's