Technical Report I

University of Marlyand College Park Dorm Building 7

College Park, MD

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Executive Summary

Technical Report 1 is a structural existing conditions report. The purpose of this report is to describe and analyze the structure of The University of Maryland College Park Dorm Building 7. This report will explain the design and loading by taking both current standards and the governing standards based on the project's start date and location into consideration.

The University of Maryland College Park Dorm Building 7 (Building 7) is the final stage of the south campus master plan at the University of Maryland. Building 7 is the corner stone of the south campus entrance for all to take part of as they approach the campus. Building 7 is an eight story residential dorm in the shape of an unsymmetrical-U that compliments the adjacent two existing dorm buildings in architectural styles with its shape and material usage. This eight story-133,000 square feet residential building, houses 370 bedrooms, study lounges, seminar spaces and resident life offices. The average floor to floor height is 10 feet on each floor with an average floor area of 12,000-15,500 square feet per floor, depending on shifts in the vertical plane. The layout of each floor is such that all of the rooms have an exterior view of the surrounding campus with a central corridor running the length of the building. The roof level houses the mechanical equipment along with the elevator and stair towers.

The wind loads were calculated using the analytical procedure outlined in ASCE 7-05, Sect. 6.5. Due to Building 7's unsymmetrical floor plan, additional wind loading had to be considered at orthogonal directions to find the worse case. From here Wind Pressure Step Diagrams were formed and it was found that a 45 degree direction gave a worse case direction. Wind was determined to be the controlling lateral force, which the design engineer found to also be the controlling lateral force. There is considerable differences between my values and the engineers, mine being larger. A possible reason for this difference is the assumptions on the buildings rigidity and also the directions they considered in determining the pressures.

The Seismic loads were calculated using the equivalent lateral force procedure (ELF) as outlined in ASCE 7-05, Sect. 12.8 as well as the simplified design procedure outlined in ASCE 7-05, Sect. 11.7. After the base shear was determined, Eq. 12.18-12 was used to determine the seismic shear contribution of each floor. Building 7's site is very stiff to hard silty clays at the deep foundation level, resulting in a Site Class C. Based on the site class, S_{D1}, and occupancy category the Seismic Design Category was determined to be A. Once all seismic loads were determined, I compared them to the loads given by the design engineer. The seismic base shear that I calculated was only off by 28 kips if the simplified design procedure is used. A possible reason for this difference is in the calculated weight of the building.

Finally gravity spot checks were performed on a first floor and the second floor of building 7. A typical beam and a typical column was chosen with a similar layout and loading throughout the building. Using the loads determined by this report and following ACI 318-05, the concrete members were found to adequate to carry the loads. The beam being sized adequately for the loads while the column was over sized based my results. This is further explained in the report.

Structural Systems

Foundation

The foundation system is composed of reinforced concrete grade beams 24”x30” with 3#8’s on the top and bottom with number #4 stirrups placed every 14”. The deep foundation portion is auger cast grout piles 16” in diameter. These piles are to be 65’ below elevation and are to be able to carry at 65 ton allowable load capacity. The pile configurations range from 2-4 piles per cap. The slab on grade for the foundation is 4” thick normal weight concrete reinforced with 6x6-1.4xW1.4 welded wire fabric. All foundation concrete is 4ksi except for the SOG which is 3.5 ksi. Due to the site’s soil conditions it was necessary that the differential settlement over the entire building was limited, because of this the allowable soil bearing capacity was held to 500 psf.

Floor Systems

Lower 2 Floors

The lower two floors are made of reinforced concrete beams spanning between the columns. The intermediate members between these beams are made up of the Hambro Floor System, which includes the steel joists and the slab system. The concrete beams range from 16x36 to 18x18 to 24x36 with the reinforcing ranging in each from 3#5’s to 6#10’s for longitudinal bars with #4 stirrups spaced from 8” to 16” O.C.

The Hambro Floor System in Building 7 is not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed criteria that the contractor must follow. The following is the criteria: are overall depth of the members is 16” deep typically throughout except in the corridors which it drops to 8”deep, the slab on top is to be 5” thick reinforced with 6x6-W4.0xW4.0 welded wire fabric.

Upper 6 Floors

The floor system is made of the same Hambro Floor System but instead of them bearing on concrete girders they bear on light-gage stud bearing walls. This Hambro Floor System is also to be designed by the contractor instead of the Engineer. Here are the criteria for these 7 stories: overall depth of the members is 16” deep typically throughout except in the corridors which it drops to 8”deep with a 3” thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric.

Column and Bearing Wall Systems

The concrete columns support the lower two floors of Building 7. They arranged to form a typical bay of 15’x20’. These columns are gravity bearing only due to the type of lateral system in the building. The typical size of the columns range from 18x14 to 64x14 with the reinforcing ranging in each from 4#9’s to 10#9’s for vertical bars with #4 stirrups spaced at 14” O.C.. The concrete compressive strength for the columns is 6 ksi. The column layout can be seen in Figure 1 and are highlighted in red.

The bearing walls in Building 7 support the upper 6 floors and run along the outside perimeter of the building as well as along the corridors. The typical spans for the floor joists are 20’. Dealing with the concerns that the joists may not line up with the studs causing the header to buckle, this problem was solved by placing a distribution tube across the tops of all bearing walls. These walls are also to be designed by the contractor who is given general criteria to follow along with a loading diagram for all the different bearing walls. The general criteria are: a maximum stud spacing of 16” O.C., a minimum G90 galvanized coating, and have a minimum 16 gage thickness. The Bearing walls on a typical floor plan have been highlighted green in Figure 2 to show their exact locations.

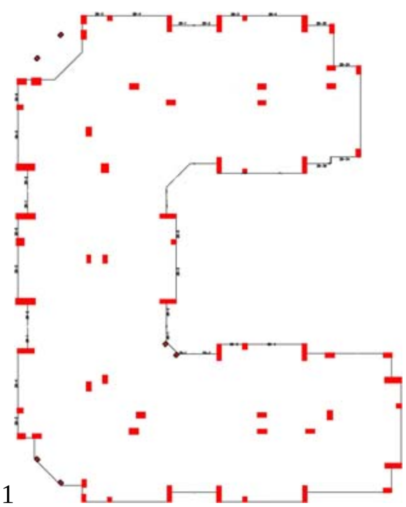


Figure 1

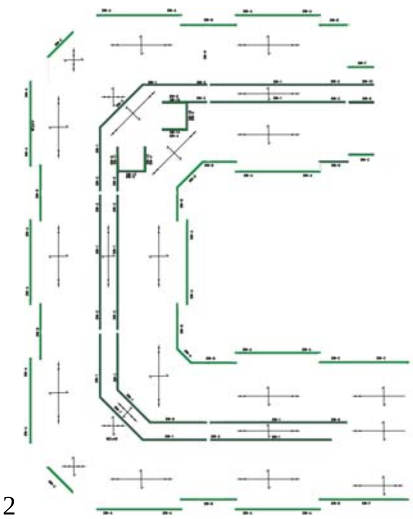


Figure 2

Roof System

The roof system is made of the same Hambro Floor System bearing on light gage walls. This Hambro Floor System is also to be designed by the contractor instead of the Engineer just as the other floors are to be designed. Here are the criteria for the roof: overall depth of the members is 16” deep typically throughout except in the corridors which it drops to 8”deep with a 3” thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric. The mechanical unit weights are listed and are placed close to the corridors for they are formed by the bearing walls. The elevator towers and stair towers are made of the same light gage studs.

Lateral Systems

The primary lateral system for Building 7 is shear walls. On each floor there are 16 shear walls spanning both directions of the building, 9 in the north-south direction and 7 in the east-west direction. The exact location of the shear walls can be found in blue in Figure 3. The lower two stories shear walls are 10” thick reinforced concrete with 10#5’s on each end for flexure and for shear reinforcement there is #5@12” each way, each face. All concrete shear walls are 6 ksi normal weight concrete. The upper floors shear walls are to be light gage studs with maximum stud spacing of 16” O.C. they are also have a minimum G90 galvanized coating and have a minimum gage of 16 for the studs while the tracks are permitted to have a 20 gage minimum. There is to be bridging at 4’ spacing throughout the shear walls. Since these are light gage it was determined that steel strapping was needed and is being provided in an X pattern connecting to the farthest opposite ends. The light-gage shear walls not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed loading diagrams of each load and the type of load on every shear wall.

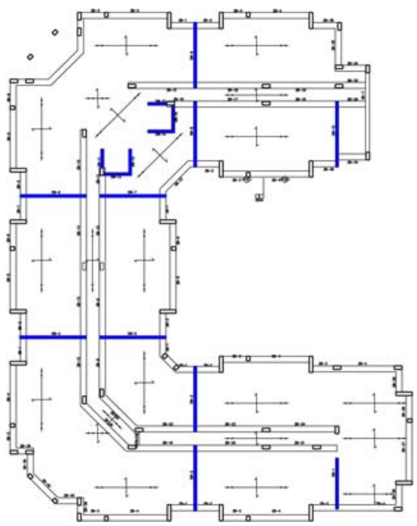


Figure 3

Design Codes

1. AISC Unified Manual 13th Edition
2. ACI 318-05
3. ASCE 7-05
4. International Building Code (IBC) 2006
5. SJI Design Manual
6. ACI 530.1/ASCE 6 “Specification for Masonry Structures”

Deflection Criteria

Typical live load deflections limited to: L/360
Typical total deflections limited to: L/240
Maximum deflection limited to ¾”

Material Specification

These materials, their grades, and strengths were the materials that the current Building 7 is utilizing. All materials were listed on the drawings, general notes, or the specifications. These materials are summarized in the table below.

Material Properties		
Material	Grade	Strength
Concrete		
Foundation	-	f'c = 4000 psi
Slab on grade	-	f'c = 3500 psi
Column	-	f'c = 6000 psi
Shear walls	-	f'c = 6000 psi
Floor Slab	-	f'c = 3000 psi
HSS Rect and Circular	A500-Gr. B	fy = 46 ksi
Steel W-Shapes	A992	Fy = 50ksi
HP Shapes/Angles	A36	fy = 36 ksi
Reinforcing Bars	Gr. 60	Fy = 60 ksi
Light Gage Studs	A1003-Gr. ST50H	fy = 50 ksi
CMU	ASTM C90 Lit Wt.	-
Grout	C270 Type S	-
Masonry walls	-	f'm = 1500 psi

Gravity and Lateral Loads

Live Loads

The live loads for Building 7 were calculated in accordance with IBC 2006 which references ASCE 7-05, Chapter 6. In the event that ASCE did not list loads needed a close equivalent was chosen to meet that space.

Live Loads			
Occupancy	Design Load	Code Required Loads	
		Load	Code
Corridors	100 psf	100 psf	ASCE 7
Offices	100 psf	50 psf	ASCE 7
Seminar Room	100 psf	40 psf	ASCE 7
Mechanical Room	250 psf	125 psf	Light manufacturing
Partition	15 psf	-	-
Roof	30 psf	20 psf	ASCE 7
Dormitory Rooms	40 psf	40 psf	ASCE 7
Lobby	100 psf	100 psf	ASCE 7

Dead Loads

The dead loads for Building 7 were determined by referencing various standards and textbooks to find the corresponding values of their weights. Approximate values were assumed when ranges were listed depending on how dense the layouts were.

Dead Loads		
Roof Dead Load	Material	Design Weight
	Rigid Insulation	4 psf
	3" Hambro Slab	38 psf
	M/E/P	5 psf
	Ceiling Finishes	3 psf
	Roofing Finish	4 psf
	Total Dead Load	54 psf
Typ. Floor Dead Load	Material	Design Weight
	3" Hambro Slab	38 psf
	5" Hambro Slab	63 psf
	M/E/P	5 psf
	Ceiling Finishes	3 psf
	Total Dead Load	46-71 psf

Snow Loads

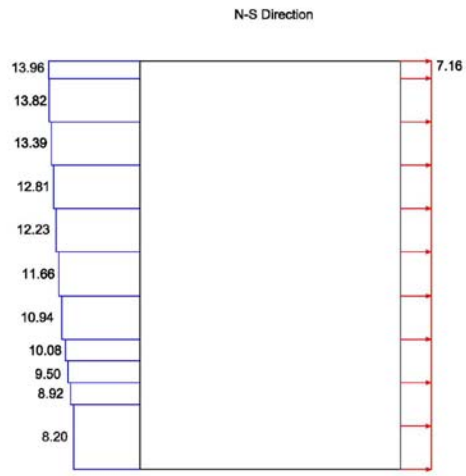
The snow load for Building 7 was calculated in accordance ASCE 7-05, Chapter 7. Figure 7-1 was used to determine the ground snow load. Then all acceptable factors listed in Chapter 7 were used to determine the flat roof snow load. Drift was not taken into consideration for this technical report due to the lack of data regarding the heights of the mechanical units which would affect the drift load. The Drift snow load though would have a significant impact near the mechanical units and along the parapets.

Snow Load Criteria	
Ground Snow Load (Pg)	25
Thermal Factor (Ct)	1
Exposure Factor (Ce)	1
Importance Factor (I)	1
Flat Roof Snow Load(pf)	17.5
Minimum Required Pf	20 psf

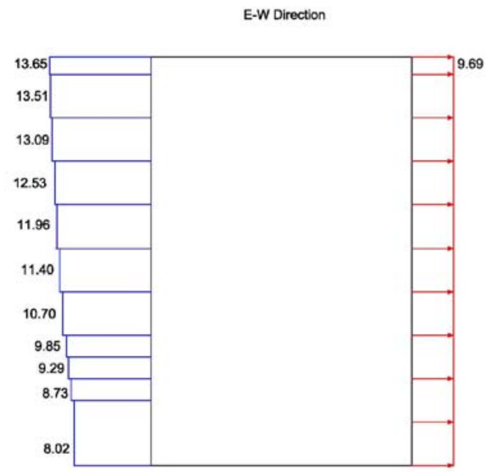
Wind Loads

All wind loads were calculated in accordance with ASCE 7-05, Chapter 6. The analytical method 2 was used to examine lateral wind loads in the North/South direction as well as the East/West direction. Also due to the irregular shape of the building it was necessary to look at the most critical orthogonal for it could possibly control. The Figure in appendix A shows the direction and the projected area. Since the floor was made of reinforced concrete it was assumed that the building was acting rigid. Building 7 is categorized as Exposure B due to its urban setting and in College Park, MD the basic wind speed was found to be 90 mph per Figure 6-1 in ASCE 7. The building is not quite a square relative to the four directions, with the N/S direction (169’-8”) slightly longer than the E/W direction (133’-6”). Thus, wind controlled in the NE/SW direction. The Appendix contains detailed spreadsheets of calculations and determined criteria. Wind pressures and forces are summarized in the diagrams below.

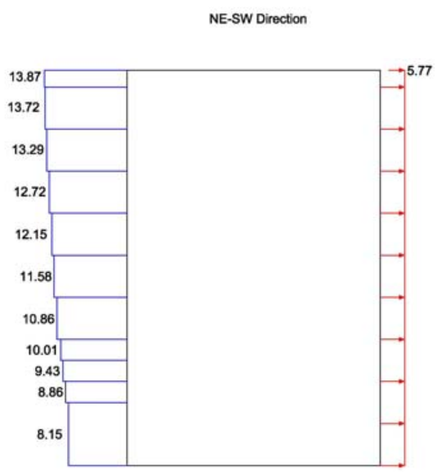
Wind Pressures



Wind Pressure Distribution in the North-South Direction



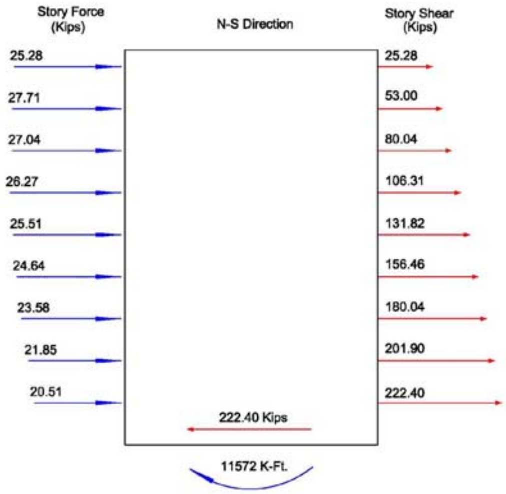
Wind Pressure Distribution in the East-West Direction



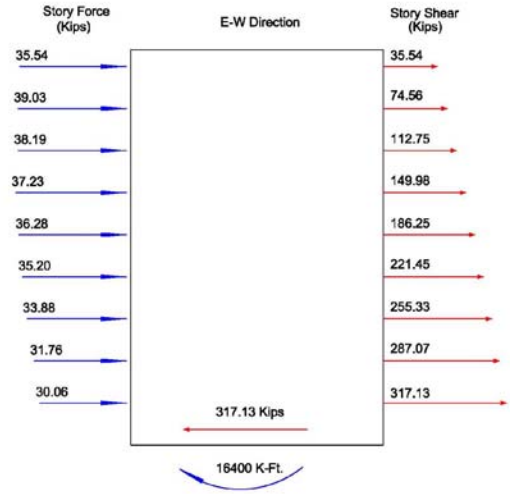
All Values on Wind Pressure Step Diagrams are in pounds per square foot (psf). The Blue indicates windward and the red indicate leeward pressures.

Wind Pressure Distribution in the Northeast-Southwest Direction

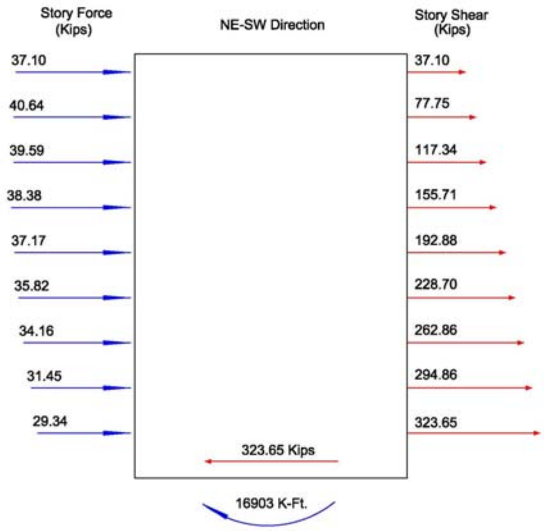
Wind Story Forces and Story Shears



Story Force and Shear in the North-South Direction



Story Force and Shear in the East-West Direction



Story Force and Shear in the Northeast-Southwest Direction

As the diagrams show, the NE-SW Direction controls for wind loading and resulting forces are:

Base shear: 323 Kips
Overturning Moment: 16903 K-Ft

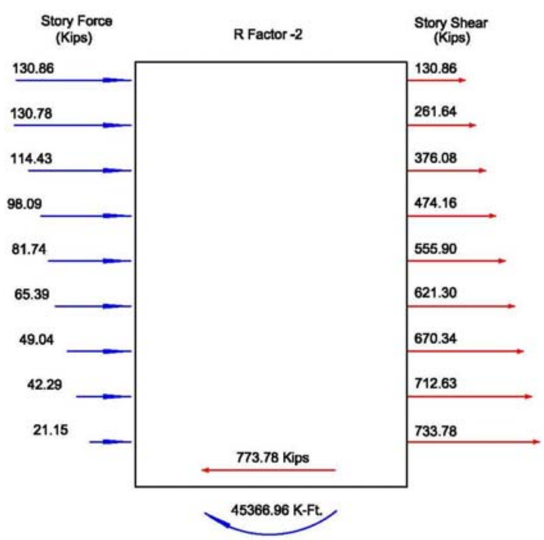
When comparing the wind loads, the base shear that I calculated was significantly off from the base shears the engineer determined. The engineer's values were 175 kips in the N-S direction and 240 kips in the E-W direction. After looking at my criteria and what was listed on the drawing they seemed to match, so a possible reason for it being off could be my assumptions of the building height for part of the first story is below grade on the one side. Also these values that the engineer gave could be taken from a computer model which may take different assumptions into account. Finally I didn't resolve my orthogonal load into an equivalent N-S and E-W load for this technical report.

Seismic Loads

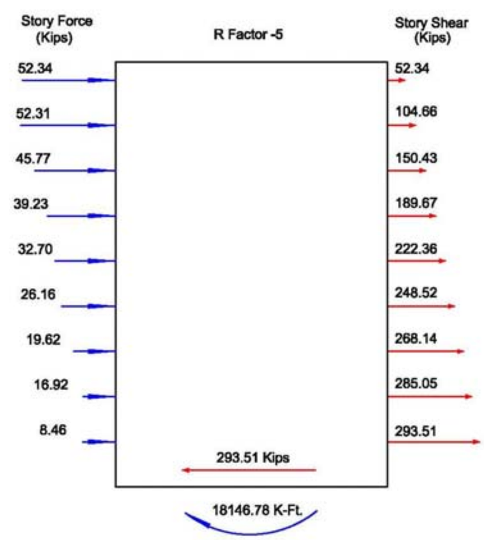
The seismic loads were calculated in accordance with ASCE 7-05, Chapter 12 and referencing Chapter 22. After looking at the geotechnical report, it was concluded that the building site is very stiff to hard silty clays at the deep foundation level, resulting in a Site Class C. it was also determined to be Seismic Design Category A. All other factors and accelerations were obtained from ASCE 7-05 figures, tables, and equations. Two simplification assumptions have been made for these calculations: the building is regular in shape and the building is rigid.

ASCE 7 Sect. 11.7 Allows for a simplified procedure because the factors of the site and response allow for a Seismic Design Category A. after looking at both equivalent lateral force procedure (ELF) and the simplified method there are significant differences. If ELF is used then the base shear is high and as you will see controls over wind. If simplified is used them the base shear is low and wind will control, also this is the procedure that the structural designer of Building 7 used.

The R-Factor in Building 7 was determined by ASCE 7. An R of 2 corresponding to light framed walls with shear panels, because this is the primary system on the upper floors. On the lower 2 floors though they system changes to ordinary reinforced concrete shear walls which has an R of 5. Both calculations were completed to show the differences but if looking at the worse case the R-Factor of 2 would generate this. Below, shown in the diagrams are the results from the different R-Factors and also the simplified code. Refer to Appendix B for more detailed spreadsheets and criteria.



Story Force and Shear with a R-Factor of 2



Story Force and Shear with a R-Factor of 5

Simplified Base Shear = 1% weight = 119.1 Kips

Controlling Lateral Loads

After completing the wind load analysis and seismic load analysis it can be concluded that if the results from the equivalent lateral force procedure are used, then the seismic controls in both base shear and overturning moment if the values for the R-Factor of 2 is used. If an R-Factor of 5 is considered the wind controls with base shear but not overturning moment. However if the simplified design procedure, allowed per ACSE 7 Sect. 11.7, is used, then wind is controlled for both shear and overturning moment. The Structural Engineer did use this section resulting in their calculations to have wind control. So in keeping with the thought process of the Engineer, further developments and analysis should be looked at from a wind controlled prospective.

N-S base shear..222.4 Kips
E-W base shear..317.1 Kips
NE-SW base shear..323.6 Kips (controlling wind)
ELF seismic base shear (R=5)..293.5 Kips
ELF seismic base shear (R=2)..733.8 Kips (controlling seismic)
Simplified seismic base shear..119.1 Kips

N-S wind overturning moment...11,572 K-Ft
E-W wind overturning moment..16,400 K-Ft
NE-SW wind overturning moment...16,903 K-Ft (controlling wind)
ELF seismic overturning moment (R=5)................................18,147 K-Ft
ELF seismic overturning moment (R=2)................................45,367 K-Ft (controlling seismic)

When comparing the seismic loads the based shear that I calculated was on 28 kips off of the base shear the engineer determined. This is relatively good considering that for the simplified procedure the base shear is determined by 1% of the building weight. The reason for it being off could be due to my calculation and assumption of the weight.

Distribution of Lateral Loads

The lateral loads for Building 7 are distributed by the method of relative stiffness. The reason for using relative stiffness is due to the concrete slab and how it acts rigidly. The controlling wind force would need to be resolved into X and Y forces and be applied to the floor. From here the center of mass and center of rigidity would need to be calculated to determine how those forces went into each shear wall. Most likely there will be torsion on the structure that the shear walls need to carry as well.

Spot Checks

Beam Spot Check

A typical interior reinforced concrete beam was chosen on the first floor to be analyzed to see if it was capable of carrying the determined loads. ACI 318-05 was used to determine the results. The beam was reinforced differently on the left side, the middle and the right side. I chose the two most critical sections to look at. The beam is an 18”x18” with a #4 stirrup spaced at 8” O.C. and with a closed stirrup.

ACI moment coefficients were used in place of a moment distribution to gather the shears and moments in the beam. As my calculations show (found in appendix C) the beam is adequate to carry the forces I determined in flexure and also in shear. The only inconsistence is that my maximum calculated stirrup spacing is less than what the designer used. Possible reasons could be forces or the method of design on the designer’s end. Also torsion was not considered and could have played a critical role.

Column Spot Check

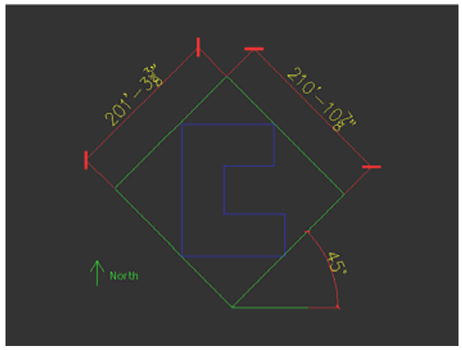
A typical interior reinforced concrete column was chosen on the second floor to be analyzed to see if it was capable of carrying the determined loads. ACI 318-05 was used to determine the results. The concrete column is 30”x14” so as to fit inside of the wall and framing into the column are concrete transfer girders that bear and distribute the loads from the bearing walls above. My calculations show and describe the member in detail about shape and reinforcing bar.

I chose three points to check the interaction diagram along the long direction of the column though due to only a compressive force the other points are not needed. PCA Column was used to draw the interaction diagram and plot the load of the column. I concluded that since the girders on each side of the column carry the same loads that the moments they transfer into the column cancel each other out. This assumption may be different from what the designer used. After doing my calculations (found in appendix D) I found that the column is adequate to carrying the loads. My load seemed rather low compared to the capacity of the column therefore the designer most likely had moments in the column and could have distributed the loads other than by tributary area, which I used.

Appendix

Appendix A: Wind Anaylsis

Orthogonal Projection for Wind Pressure Analysis



Wind Criteria & Calculated Variables

Wind Criteria		
Basic Wind Speed (V)	90 mph	figure 6-1C
Wind Exposure Category	B	
Occupancy Factor	II	
Importance Factor	1	table 6-1
Wind Directionality Factor (K_d)	0.85	table 6-4
Topographic factor (K_{zt})	1	sect 6.5.7.1-2
Number of Stories	9	
Building Height (Ft.)	94	
N-S Building Length (Ft.)	169.75	
E-W Building Length (Ft.)	133.5	
NE-SW Building Length (Ft.)	200.75	
NW-SE Building Length (Ft.)	210.75	
L/B in N-S Direction	1.27	
L/B in E-W Direction	0.79	
L/B in NE-SW Direction	1.05	

Gust Factor			
Variable	Wind Direction		
	N-S	E-W	NE-SW
Stiffness	Rigid	Rigid	Rigid
B (Feet)	133.50	169.75	210.75
L (Feet)	169.75	133.50	200.75
h (Feet)	94.00	94.00	94.00
c	0.30	0.30	0.30
Z	56.40	56.40	56.40
I_z	0.27	0.27	0.27
Lz	202.95	159.61	240.02
∈	0.33	0.33	0.33
Q	0.77	0.73	0.76
gQ & gv	3.40	3.40	3.40
G	0.80	0.77	0.79

	Wind Direction			
	N-S Direction	E-W Direction	NE-SW Direction	
C_p, Windward	0.8	0.8	0.8	figure 6-6
C_p, Leeward	-0.3	-0.5	-0.2	
Gust Factor	0.796	0.773	0.789	
GCpi	0.18	0.18	0.18	figure 6-5

Wind Pressure Spreadsheets

Height (Feet)	K_z	q_z	Wind Pressures (psf) N-S Windward	N-S Leeward	N-S Total
94	0.97	17.10	13.96	-7.16	21.12
90	0.96	16.92	13.82	-7.16	20.98
80	0.93	16.39	13.39	-7.16	20.54
70	0.89	15.69	12.81	-7.16	19.97
60	0.85	14.98	12.23	-7.16	19.39
50	0.81	14.28	11.66	-7.16	18.82
40	0.76	13.40	10.94	-7.16	18.10
30	0.7	12.34	10.08	-7.16	17.23
25	0.66	11.63	9.50	-7.16	16.66
20	0.62	10.93	8.92	-7.16	16.08
0-15	0.57	10.05	8.20	-7.16	15.36

Height (Feet)	K_z	q_z	Wind Pressures (psf) E-W Windward	E-W Leeward	E-W Total
94	0.97	17.10	13.65	-9.69	23.34
90	0.96	16.92	13.51	-9.69	23.20
80	0.93	16.39	13.09	-9.69	22.78
70	0.89	15.69	12.53	-9.69	22.21
60	0.85	14.98	11.96	-9.69	21.65
50	0.81	14.28	11.40	-9.69	21.09
40	0.76	13.40	10.70	-9.69	20.38
30	0.7	12.34	9.85	-9.69	19.54
25	0.66	11.63	9.29	-9.69	18.98
20	0.62	10.93	8.73	-9.69	18.41
0-15	0.57	10.05	8.02	-9.69	17.71

Height (Feet)	K_z	q_z	Wind Pressures (psf) NE-SW Windward	NE-SW Leeward	NE-SW Total
94	0.97	17.10	13.87	-5.77	19.64
90	0.96	16.92	13.72	-5.77	19.50
80	0.93	16.39	13.29	-5.77	19.07
70	0.89	15.69	12.72	-5.77	18.50
60	0.85	14.98	12.15	-5.77	17.92
50	0.81	14.28	11.58	-5.77	17.35
40	0.76	13.40	10.86	-5.77	16.64
30	0.7	12.34	10.01	-5.77	15.78
25	0.66	11.63	9.43	-5.77	15.21
20	0.62	10.93	8.86	-5.77	14.64
0-15	0.57	10.05	8.15	-5.77	13.92

Note these spread sheets use all relevant equations located and described in ASCE 7

Wind Story Force, Shear and Overturning Moment Spreadsheets

Wind (North-South) Level	Height (Feet)	Tributary Area (Feet)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (k-ft)	
roof	90	9	21.00	25.28	25.28	2275.40	
8	80	10	20.54	27.71	53.00	4492.57	
7	70	10	19.97	27.04	80.04	6385.40	
6	60	10	19.39	26.27	106.31	7961.77	
5	50	10	18.82	25.51	131.82	9237.03	
4	40	10	18.10	24.64	156.46	10222.79	
3	30	10	17.23	23.58	180.04	10930.27	
2	20	10	16.37	21.85	201.90	11367.35	
1	10	10	15.36	20.51	222.40	11572.41	
ground	0	0	0.00	0.00	222.40	11572.41	Total

Wind (East-West)							
Level	Height (Feet)	Tributary Area (Feet)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (k-ft)	
roof	90	9	23.28	35.54	35.54	3198.50	
8	80	10	22.78	39.03	74.56	6320.54	
7	70	10	22.21	38.19	112.75	8993.51	
6	60	10	21.65	37.23	149.98	11227.08	
5	50	10	21.09	36.28	186.25	13040.86	
4	40	10	20.38	35.20	221.45	14448.76	
3	30	10	19.54	33.88	255.33	15465.23	
2	20	10	18.7	31.73	287.07	16099.92	
1	10	10	17.71	30.06	317.13	16400.55	
ground	0	0	0	0.00	317.13	16400.55	Total

Wind (East-West)							
Level	Height (Feet)	Tributary Area (Feet)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (k-ft)	
roof	90	9	19.57	37.10	37.10	3339.42	
8	80	10	19.07	40.64	77.75	6590.87	
7	70	10	18.5	39.59	117.34	9362.13	
6	60	10	17.92	38.38	155.71	11664.78	
5	50	10	17.35	37.17	192.88	13523.07	
4	40	10	16.64	35.82	228.70	14955.75	
3	30	10	15.78	34.16	262.86	15980.62	
2	20	10	14.92	31.45	294.31	16609.71	
1	10	10	13.92	29.34	323.65	16903.08	
ground	0	0	0	0.00	323.65	16903.08	Total

Appendix B: Seismic Analysis

Building Weight

The effective weight of the building was first calculated by determining the weight of each of the building's 8 floors and roof. This included the exact weights of all slabs, bearing walls, partitions, exterior brick façade, and the superimposed dead loads. Adding the weights of the floors resulted in the building's effective weight. From here the seismic base shear was calculated.

Roof Weight						
	Dead Load	Snow Load	Partition Load	Perimeter Wall Load	Perimeter Length	Roof Area
	54	20<30 so NA.	-	47	760	14750
	Total Roof Weight =	1117.98	kips			

Typical Floor Weight Upper Floors						
	Dead Load	Snow Load	Partition Load	Perimeter Wall Load	Perimeter Length	Floor Area
	46	-	15	47	760	14750
	Total Floor Weight =	1256.95	kips			

Typical Floor Weight Lower Floors						
	Dead Load	Snow Load	Partition Load	Perimeter Wall Load	Perimeter Length	Floor Area
	71	-	15	47	760	14750
	Total Floor Weight =	1625.7	kips			

Seismic Criteria and Forces Spreadsheets

Seismic Criteria is all based on ASCE 7, IBC 2006, Geotechnical Report and www.usgs.org. All relevant codes were imputed into excel and the following charts were created to determine the seismic base shear.

Calculations with R=5

Seismic Criteria		
Occupancy Category	I	
Importance Factor	1.000	Table 11.5-1
Seismic Category	A	ASCE 7 Section 11.6
Site Class	C	geo-tech report
Spectral Acceleration for Short Periods (Ss)	0.154	usgs.org
Spectral Acceleration for 1 Second Periods (S1)	0.050	usgs.org
Site Coefficient, Fa	1.200	ASCE 7 Table 11.4-1
Site Coefficient, Fv	1.700	ASCE 7 Table 11.4-2
Seismic Design Category		ASCE 7 Table 11.6-1,2
R Factor	5.000	ordinary reinforced shear walls
S_{MS}	0.185	
S_{M1}	0.085	
S_{DS}	0.123	
S_{D1}	0.057	
Cs	0.025	

floor	Height (Ft.)	Weight (Kips)	Cvx	Fx (kips)
roof	90	1118.00	0.18	52.34
8	80	1257.00	0.18	52.31
7	70	1257.00	0.16	45.77
6	60	1257.00	0.13	39.23
5	50	1257.00	0.11	32.70
4	40	1257.00	0.09	26.16
3	30	1257.00	0.07	19.62
2	20	1626.00	0.06	16.92
1	10	1626.00	0.03	8.46
Total Weight		11912	kips	

Seismic Base Shear	293.51	kips
Overturning Moment	18146.78	kip-ft

<u>Calculation with R=2</u>

Seismic Criteria		
Occupancy Category	I	
Importance Factor	1.000	Table 11.5-1
Seismic Category	A	ASCE 7 Section 11.6
Site Class	C	geo-tech report
Spectral Acceleration for Short Periods (Ss)	0.154	usgs.org
Spectral Acceleration for 1 Second Periods (S1)	0.050	usgs.org
Site Coefficient, Fa	1.200	ASCE 7 Table 11.4-1
Site Coefficient, Fv	1.700	ASCE 7 Table 11.4-2
Seismic Design Category		ASCE 7 Table 11.6-1,2
R Factor	2.000	light-framed wall with shear panels
S_{MS}	0.185	
S_{M1}	0.085	
S_{DS}	0.123	
S_{D1}	0.057	
Cs	0.062	

floor	Height (Ft.)	Weight (Kips)	Cvx	Fx (kips)
roof	90	1118.00	0.18	130.86
8	80	1257.00	0.18	130.78
7	70	1257.00	0.16	114.43
6	60	1257.00	0.13	98.09
5	50	1257.00	0.11	81.74
4	40	1257.00	0.09	65.39
3	30	1257.00	0.07	49.04
2	20	1626.00	0.06	42.29
1	10	1626.00	0.03	21.15
Total Weight		11912	kips	

Seismic Base Shear	733.78	kips
Overturning Moment	45366.96	kip-ft

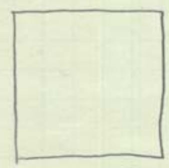
Appendix C: Beam Spot Check

Typical Beam Spot Check

Technical Report I.

Beam Spot Check

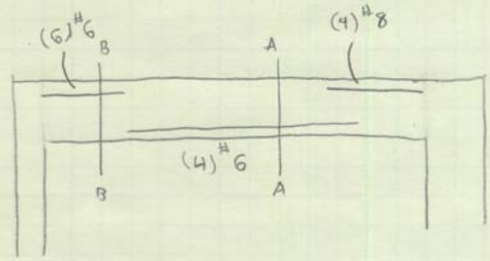
Verify Beam B1-12



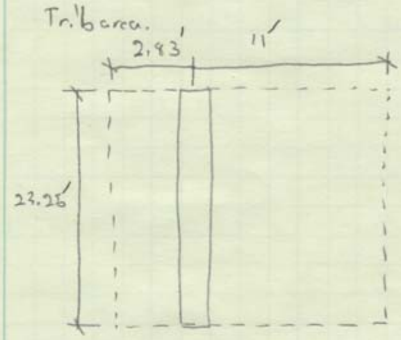
#4 stirrup at 8"

- Loads

DL= 71 psf includes slab.
LL= 40 on Apartment side
100 on Corridor side.



Trib area.



beam/girder weight:

$$\frac{(18"\times 18")}{144}(150\,pcf) = 337.5\,plf$$

load on beam

$$w_L = 40(11) + 100(2.83) = 723.0\,plf$$

$$w_D = 71(13.83) = 982$$

$$w_u = 1.2(337.5 + 723.0) + 1.6(982) = 2.84\ klf$$

Using ACI Moment Coe

Interior Span

$$M^{+} = \frac{w_u l_n^2}{16} \qquad M^{-} = \frac{w_u l_n^2}{11}$$

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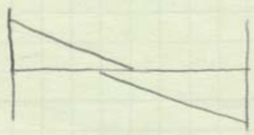
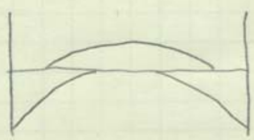
$V = \frac{w_u l_n}{2}$

V



$V = \frac{2.84(23.25)}{2} = 33^{K}$

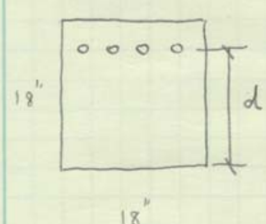
M



$M^- = \frac{w_u l_n^2}{11} = \frac{2.84(23.25)^2}{11} = 139.6^{k-ft}$

$M^+ = \frac{w_u l_n^2}{16} = \frac{2.84(23.25)^2}{16} = 95.9^{k-ft}$

Section A



$d = 18 - 1.5 - \frac{1}{2} - \frac{0.75}{2} = 15.6''$

$A_s = 4(0.44) = 1.76\ in^2$

$A_{s\,min} = \begin{cases} \frac{3\sqrt{f'_c}\,bd}{f_y} = \frac{3\sqrt{6000}(18)(15.6)}{60{,}000} = 1.08\ in^2 \\ \frac{200\,bd}{f_y} = \frac{200(18)(15.6)}{60{,}000} = 0.936\ in^2 \end{cases}$

$\beta_1 = 0.75$

$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85(0.75)\left(\frac{6}{60}\right)\left(\frac{0.003}{0.007}\right) = 0.0273$

$A_{s\,max} = 0.0273(18)(15.6) = 7.67\ in^2$

$A_s \quad 0.936 \le 1.76 \le 7.67$ So good

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$\underline{M_n}$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.76(60{,}000)}{0.85(6000)(18)} = 1.15$$

$$C = \frac{a}{\beta_1} = \frac{1.15}{0.75} = 1.53$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{1.53}(15.6-1.53) = 0.0275 \geq 0.004$$

so $\phi = 0.9$

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$= \frac{0.9(1.76)(60)(15.6 - \frac{1.15}{2})}{12} = 119^{k\text{-}ft}$$

$\phi M_n = 119 > 95.9$ so section Works.

$\underline{\text{Section B}}$

$A_s = 6(0.44) = 2.64 \text{ in}^2$

$A_{s\,min} = 0.936$ same as before
$A_{s\,max}\ 7.67$ same as before.

$A_s = 0.936 \leq 2.64 \leq 7.67$ so good.

$\underline{M_n}$

$$a = \frac{2.64(60)}{0.85(6)(18)} = 1.72$$

$$C = \frac{1.72}{0.75} = 2.3$$

$$\varepsilon_s = \frac{0.003}{2.3}(15.6-2.3) = 0.017 > 0.004 \text{ so:}$$

$\phi = 0.9$

3/4

$$\phi M_n = \frac{0.9(2.64)(60)(15.6 - 1.72/2)}{12} = 175.1^{k\text{-}ft}$$

$\phi M_n = 175.1 > 139.6^{k\text{-}ft}$ So good.

The beam passes Moment Capacity for both Critical Sections.

Shear Strength Check.

$$V_c = 2\sqrt{f'_c}\, bd = 2\sqrt{6000}(18)(15.6) = 43.5^k$$

$$\phi V_n = 0.5\phi V_c = 0.5(0.75)(43.5) = 16.31^k$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{33}{0.75} - 43.5 = 0.5^k$$

$$V_s \leq 4\sqrt{6000}(18)(15.6) = 87^k$$

$$S_{max} = \min \begin{cases} d/2 = 15.6/2 = 7.8 = 7.5'' \text{ controls} \\ 24 = 24'' \end{cases}$$

$$A_{v\,min} = \max \begin{cases} \frac{0.75\sqrt{6000}(18)(7.5)}{60{,}000} = 0.13 \\ \frac{50(18)(7.5)}{60{,}000} = 0.11 \end{cases}$$

a #4 stirrup

$2(0.2) = 0.4 > 0.11$ So good.

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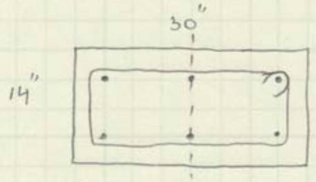
Appendix D: Column Spot Check

Typical Column Spot Check

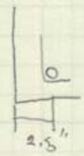
Technical Report I

Column Spot Check

Spot Checking Typ Interior Column.



1.5" clear cover
$f'_c = 6$ ksi
6 #9 bars
#4 Stirrup @ 14"
$f_y = 60$ ksi



$A_s = 6(1) = 6.0\,''^2$

– Pure axial Strength.

$$P_o = 0.85\, f'_c A_c + A_s f_y$$
$$= 0.85(6)(30 \times 14 - 6) + 6(60)$$
$$= 2471.4^{k}$$

– Balanced Condition

$$\varepsilon_y = \frac{60}{29{,}000} = 0.0021 \qquad c = \frac{0.003}{0.003 + 0.0021}(27.5) = 16.18$$

$$\varepsilon_{s1} = \frac{0.003}{16.18}(16.18 - 2.5) = 0.0025 \qquad f_{s1} = 60\text{ ksi}$$

$$\varepsilon_{s2} = \frac{0.003}{16.18}(16.18 - 15) = 2.19 \times 10^{-4} \qquad f_{s2} = 6.34$$

$$\varepsilon_{s3} = \frac{0.003}{16.18}(16.18 - 27.5) = 0.0021 \qquad f_{s3} = -60\text{ ksi}$$

$$M_b = 0.85 f'_c b \beta_1 c \left(\frac{w}{2} - \frac{\beta_1 c}{2}\right)$$

$$0.85(6)(14)(0.75)(16.18)\left(\frac{30}{2} - \frac{0.75(16.18)}{2}\right)/12 = 645^{k\text{-}ft}$$

$$P_b = 0.85 f'_c b \beta_1 c + 2f_1 + 2f_2 + 2f_3$$

$$= 0.85(6)(14)(0.75)(60) + 2(6.34) + 2(-60) = 258.8^{k}$$

1/3

- Pure Bending

$f_{s_1} = \frac{0.003}{c}(c-2.75)(29000)$, $f_{s_2} = -60ksi$, $f_{s_3} = -60ksi$

$0.85(6)(14)(0.75)c + 2\left(\frac{0.003}{c}((c-2.75)(29000))\right) - 2(60) - 2(60)$

$53.55c^2 + 170c - 478.5 - 360c$

$c = 5.25 in$

$f_{s_1} = 41.4 ksi$

$\varepsilon_{s_2} = 0.007 >$ so ok

$\varepsilon_{s_3} = 0.014$ so ok.

$Mb = 0.85(6)(14)(0.75)(5.25)\left(\frac{30}{2} - \frac{0.75(5.25)}{2}\right) + 2(41.4)(15-2.75)$

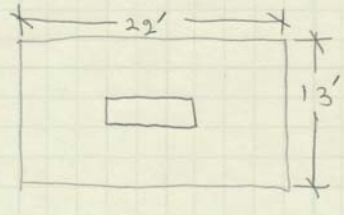
$+ 0 + 2(-60)(15-27.5)$

$Mb = 514.8^{k-ft}$

$\phi M_n = 0.9(514.8)\ 463.32\ k\text{-}ft$
$\phi P_n = 0.65(2471.4) = 1606.2^{k-ft}$
$\phi M_b = 0.9(645) = 580.15^{k-ft}$
$\phi P_b = 0.68(259.) = 178\ K$

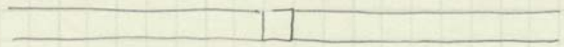
due to the column being equally spaced around, the beam moments in the column cancel out with each other leaving only the axial load Pu.

Tributary of the column



2/3

weight of upper floors due to bearing loads



$1.2(6.3) = 7.56$ klf
$1.6(4.2) = 6.72$ klf
14.28 klf

314.9^{K} upper floors

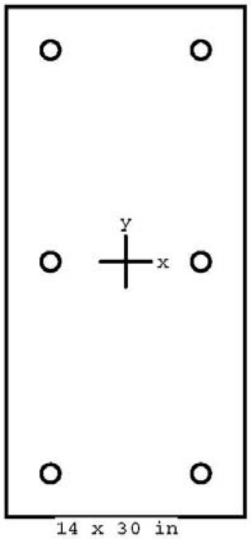
$w_{LL} = 723.0$ plf
$w_{DL} = 337.5$ plf $+ 71(13.83) = 1319.4$ plf

see beam spot checks
(same weights)

$P_u = 1.2(1319.4)(22) + 1.6(723)(22)$

$P_u = 60$ kips.

3/3



Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

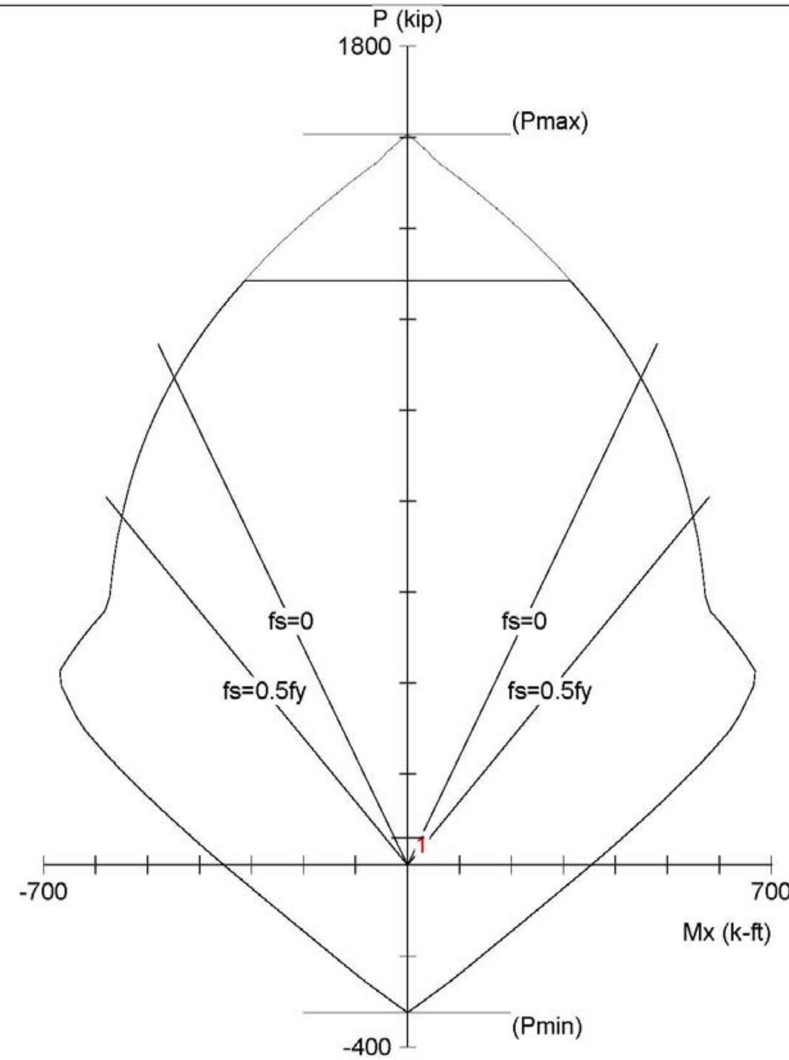
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 09/28/08

Time: 17:36:54



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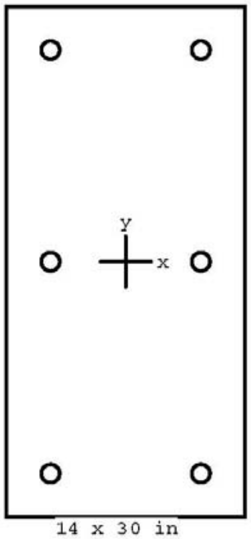
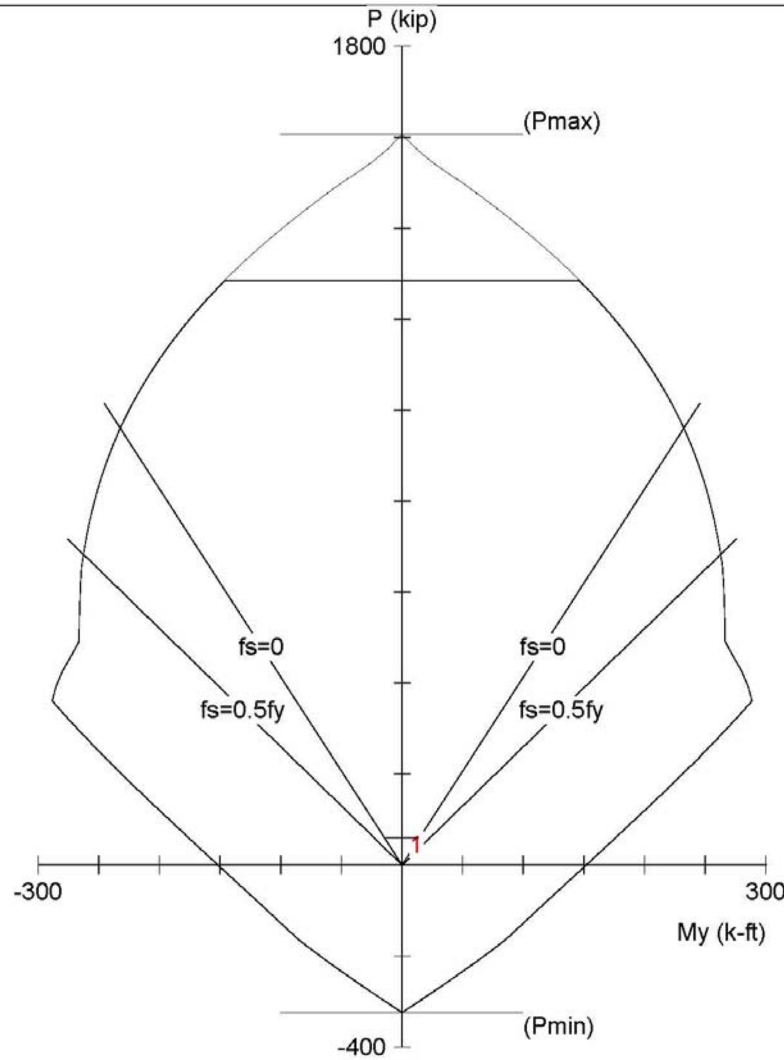
File: untitled.col

Project:

Column:		Engineer:	
f'c = 6 ksi	fy = 60 ksi	Ag = 420 in^2	6 #9 bars
Ec = 4415 ksi	Es = 29000 ksi	As = 6.00 in^2	Rho = 1.43%
fc = 5.1 ksi	fc = 5.1 ksi	Xo = 0.00 in	Ix = 31500 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 6860 in^4
Beta1 = 0.75		Clear spacing = 7.74 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		





Code: ACI 318-02

Units: English

Run axis: About Y-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 09/28/08

Time: 17:36:13

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File: untitled.col

Project:

Column:

Engineer:

f'c = 6 ksi	fy = 60 ksi	Ag = 420 in^2	6 #9 bars
Ec = 4415 ksi	Es = 29000 ksi	As = 6.00 in^2	Rho = 1.43%
fc = 5.1 ksi	fc = 5.1 ksi	Xo = 0.00 in	Ix = 31500 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 6860 in^4
Beta1 = 0.75		Clear spacing = 7.74 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		