TECHNICAL ASSIGNMENT 1
STRUCTURAL CONCEPTS/STRUCTURAL EXISTING CONDITIONS REPORT

THE TOWERS AT THE CITY UNIVERSITY OF NEW YORK
NEW YORK CITY, NEW YORK

ROBIN SCARAMASTRO
STRUCTURAL OPTION
OCTOBER 6, 2006
AE 481 W
DR. MEMARI

TABLE OF CONTENTS

EXECUTIVE SUMMARY

The Towers at the City University of New York is a new residence hall for CUNY students and faculty. It is the first dormitory for the Manhattan college in its 185 year history. The building is located at 130th Street and Saint Nicholas Terrace in the upper west side of New York City. The 11 story building is capable of housing 600 CUNY students and faculty in 165 apartments. The total cost of development and construction of the Towers was $54 million. Some of the features of the 185,000 square foot building are fully furnished apartments with private bedrooms, a laundry room, a fitness room, classroom spaces, administrative offices, a reception desk that is operational 24 hours a day, and numerous lounge and study spaces. Ground was broken in May 2005 and was completed in August 2006.

The objective of this report is to investigate in depth the existing conditions of the structure of the building. This will include detailed descriptions and evaluations of the concrete columns and beams, spread footing foundations and foundation walls, lateral resisting system, and flat plate floor slabs for the code prescribed gravity and lateral loads. A lateral analysis for wind and seismic loading will be performed using hand calculations to compare to the original design.

The appendices can be found at the end of this report and include more detailed calculations used in determining the forces on the building. Appendix A includes the lateral load calculations and Appendix B includes the design check calculations.

CODES

The following is a list of all applicable codes used in the original design of the Towers. All designs in New York City are governed by the city's own building code. For wind and seismic loading, the Building code references ASCE 7 and The Unified Building Code consecutively.

- *The Building Code of the City of New York*
- ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures,* by the American Society of Civil Engineers.
- ACI 318-89, *Building Code Requirements for Structural Concrete,* by the American Concrete Institute.
- ACI 530-99, *Building Code Requirements for Masonry Structures*, by the American Concrete Institute
- *Manual of Steel Construction, Load Resistance Factor Design*, 3rd Edition, by the American Institute of Steel Construction.
- *CRSI Handbook 2002*, by the Concrete Reinforcing Steel Institute.

STRUCTURAL SYSTEM

The structural system that was chosen The Towers is cast in place concrete columns and floor slabs. The slabs are a two-way flat plate system that directly transfer the floor loads to the columns. The penthouse consists of structural steel tube columns, wide flange beams and steel angle bracing.

MATERIALS

Concrete and steel was used in the construction of the towers. The following tables include the strengths of the materials.

Cast in Place Concrete	
Member	Compression Strength (at 28 days)
Elevated slabs	5000 psi NW
Columns	5000 psi NW
Slab on Grade	4000 psi NW
Walls	4000 psi NW
Footings	4000 psi NW
Matt foundation	4000 psi NW
Concrete beams	5000 psi NW

Reinforcing Steel		
Material	ASTM Standard	Fy (ksi)
Reinforcing bars	ASTM A615 Grade 60	60
Welded Wire Fabric	ASTM A487 for sizes D4 and larger	70
Deformed bar anchors	ASTM A496	70

Structural Steel			
Material	ASTM Standard	Fy (ksi)	Fu (ksi)
Wide Flanges	ASTM A992	50	65
Tubes	A500 Grade B	46	58
Plates	A36	36	58
Angles	A36	36	58

Welding electrodes for steel connections are E70XX with a tensile strength of 70ksi. Shear studs for the concrete are ¾" diameter Type B with a Fy = 50ksi and Fu = 60ksi. Anchor bolts for HSS column to concrete pier connections are ¾" diameter A36 steel.

FOUNDATION

Based on the soil borings and the geotechnical report, a shallow foundation was permissible for The Towers. The soil report indicated that solid bedrock was beneath 6' – 12' of firm soils at the site. The slabs and spread footings sit directly on top of the bedrock. Matt slab foundations that range in thickness from 36" to 42" are used to support the loads from the concrete shear walls around the stair and elevator cores. The foundation walls are cast in place reinforced concrete atop spread footings. Rectangular spread footings up to 30" in depth support the gravity load from the concrete columns.

FRAMING

A cast-in-place concrete system was chosen for The Towers. Rectangular columns are laid out on an irregular grid and large concrete beams are used in the central lobby area of the building that connects the two wings. The beams also support the cantilevered portion of the building at the third floor over the main entrance. The floor slab is tied in to the columns by studrails at each face, and reinforcing bars over the column transfer the floor loads into the columns. The thin brick prefabricated panels that make up the façade of the building are also connected to the top of the slab with steel angles. Expansion joints are used at the edges of the slab where they meet with the exterior wall panels. 2″ seismic expansion joints are also used at the corners of the building.

The penthouse of the building is structural steel. Steel tube columns are used as the columns and W-shapes are used as beams. Bracing is provided by steel angles for the beam to column connections. The penthouse consists of two levels. The floor of the first level is the cast in place roof slab. The second floor framing consists of W24x55 beams. The roof of the penthouse is framed with W12x14 beams. The exterior girders that carry the floor and roof framing are connected to the columns with moment connections, and are additionally braced with steel angle knee bracing.

FLOOR SLAB

The typical structural slab for all 11 stories of the Towers a is two way 8″ elevated flat plate concrete slab. The slab is reinforced with #4 bars at 12″ on center. Extra bars are provided at column locations for added resistance against shear forces. For the basement, a 4″ slab on grade was used. The slab on grade is reinforced with welded wire fabric and is cast over a vapor barrier and 4″ of a porous fill base. The floor system for the first level is the flat plate concrete slab. The floor system of the structural steel penthouse consists of a 4 ½″ concrete slab with metal deck.

LATERAL FORCE RESISTING SYSTEM

Lateral loads imposed on the building will be resisted by concrete shear walls located throughout the building. One wall is located in the north wing of the building, and the other walls are around the stair towers and elevator core. The typical structural layout in Figure 1 below illustrates the locations of columns and shear walls. The floor slab acts as a rigid diaphragm to transfer the toads to the lateral force resisting system. The shear walls are 10″ thick and are reinforced with two curtains of rebar.

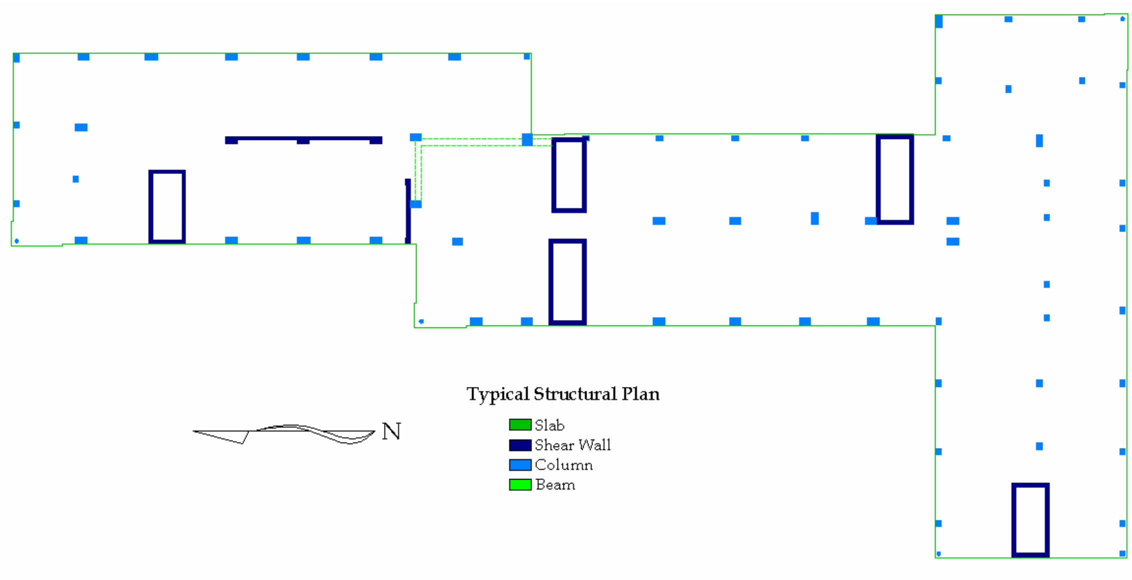


Figure 1: Typical structural framing plan

LOADING CONDITIONS

GRAVITY LOADS

The following is the list of gravity dead and live loads for each of the building occupancies used in the design of The Towers. These loads are in accordance with the Building Code of the City of New York. The loads listed do not include the weight of the structural members and the live loads are reducible per section 27-566 of the building code. The gravity loads are transferred to the columns directly by the two way flat plate slab.

DORMITORY	PSF
Construction Dead Load	
- 8″ normal weight concrete elevated slab	100
Superimposed Dead Load	
- ceiling	4
- floor finish	2
- mechanical/electrical	2
- partitions (100-200 plf)	12
Total Dead Load	120
Live Load	
- for partitioned dormitories	40

LOBBY	PSF
Construction Dead Load	
- 10″ normal weight concrete elevated slab	125
Superimposed Dead Load	
- ceiling	2
- floor finish	2
- mechanical/electrical	6
Total Dead Load	135
Live Load	100

LOUNGE	PSF
Construction Dead Load	
- 10″ normal weight concrete elevated slab	125
Superimposed Dead Load	
- ceiling	2
- floor finish	2
- mechanical/electrical	6
Total Dead Load	135
Live Load	100

ROOF (MECHANICAL)	PSF
Construction Dead Load	
- 8″ normal weight concrete elevated slab	100
Superimposed Dead Load	
- ceiling	2
- mechanical/electrical	6
- roofing and insulation	6
Total Dead Load	115
Live Load	
- weight of equipment and ponding water	150

STAIRS	PSF
Dead Load	75
Live Load	100

EXTERIOR WALL LOADS	PSF
Dead Load	
- prefabricated thin brick panels with metal stud back-up wall	24
- curtain wall system	15

LATERAL LOADS

Lateral loads imposed on The Towers are the result of wind and seismic forces. Per the City Building Code of the City of New York, the wind loads are calculated based on the methods provided in ASCE 7-98 and the seismic loads are calculated based on the UBC Section 2312-1990.

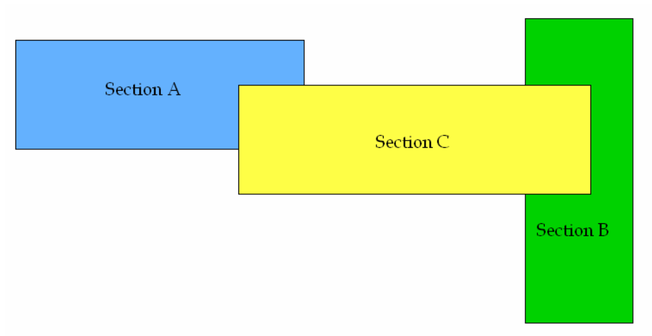


Figure 2: Building components used for lateral load calculations

To simplify the loading for wind, the building will be broken up into three components as shown in Figure 2. Section A, B and C consist of 8, 6, and 11 stories consecutively. Wind loading was calculated for both the north-south and east-west directions. However, since the loading area of the east-west direction is significantly greater than the north-south direction, the loading diagrams are only shown for the east-west direction.

The following is a summary of the wind and seismic loads, as well as diagrams to illustrate the loading patterns on the building. See Appendix B for complete loading calculations.

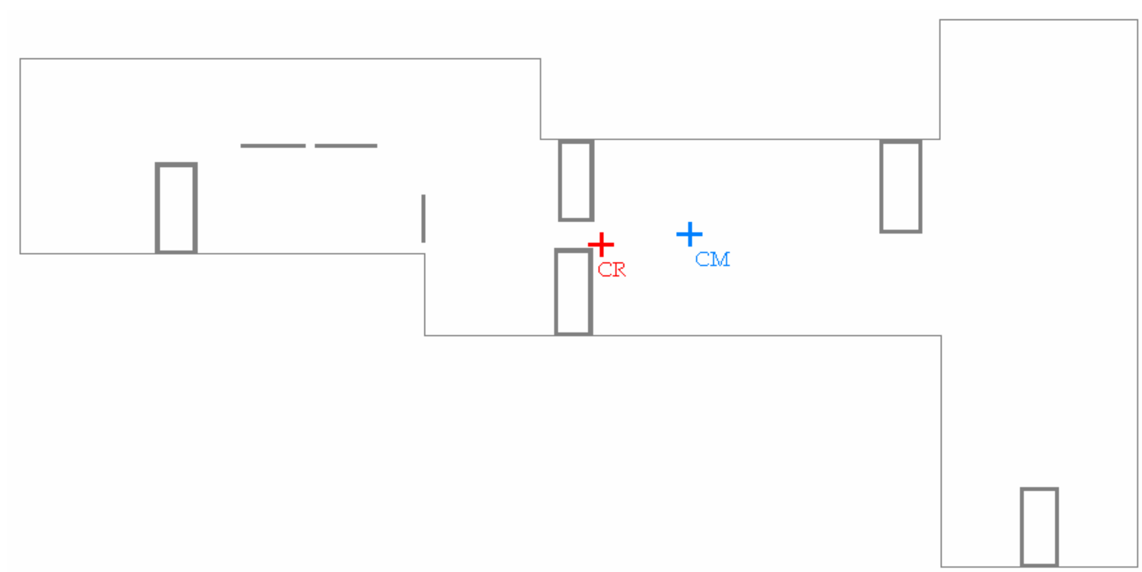


Figure 3: Center of rigidity and center of mass for the 5th floor

WIND LOADS

- Basic Wind Speed	V = 95 mph	
- Importance Factor	I_w = 1.0	
	Category 4	
- Building Exposure	D	
- Mean Roof Height	110′-0″	
- Gust Factor (Rigid Structure)	G = 0.85	
- Topographic Factor	Kzt = 1.0	
- Wind Directionality Factor	Kd = 0.85	
- Velocity Pressure Coefficients	Kh = 1.455	
	Kz = 1.03	0 – 15′
	Kz = 1.08	15 – 20′
	Kz = 1.12	20 – 25′
	Kz = 1.16	25 – 30′
	Kz = 1.22	30 – 40′
	Kz = 1.27	40 – 50′
	Kz = 1.31	50 – 60′
	Kz = 1.34	60 – 70′
	Kz = 1.38	70 – 80′
	Kz = 1.40	80 – 90′
	Kz = 1.43	90 – 100′
	Kz = 1.455	100 – 110′
- Internal Pressure Coefficient	GCpi = +/- 0.18	
- Wall Pressure Coefficients	Cp = 0.8 (windward)	
	Cp = -0.5 (leeward ⊥ 294’-8”)	
	Cp = -0.3 (leeward ⊥ 144’-4”)	
	Cp = -0.7 (sidewall)	
- Roof Pressure Coefficients	Cp = -0.9 (0 – h)	
	Cp = -0.5 (h – 2h)	
	Cp = -0.3 (>2h)	

East – West Wind

Loading Diagrams

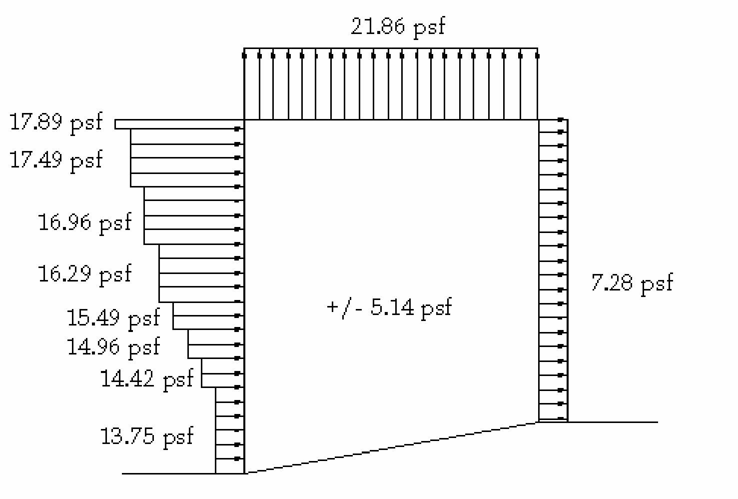


Figure 4: Wind pressure on Section A

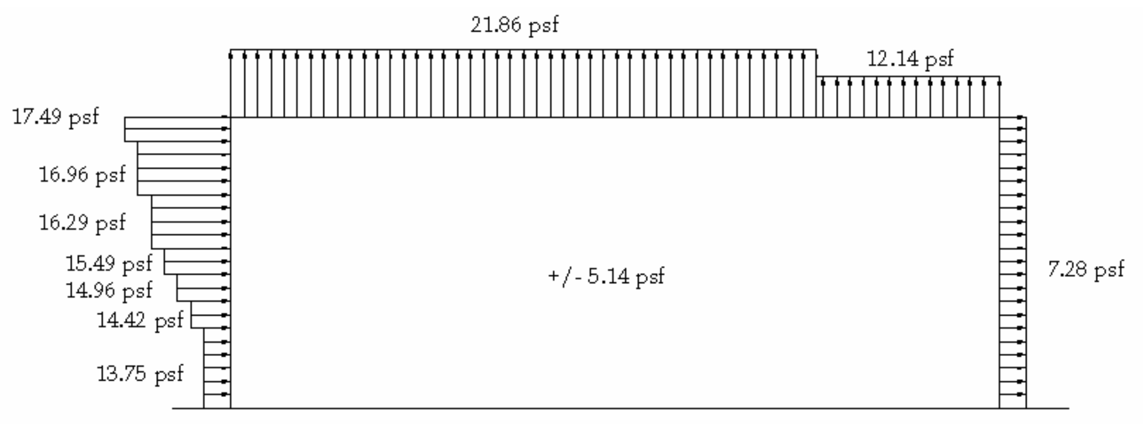


Figure 5: Wind pressure on Section B

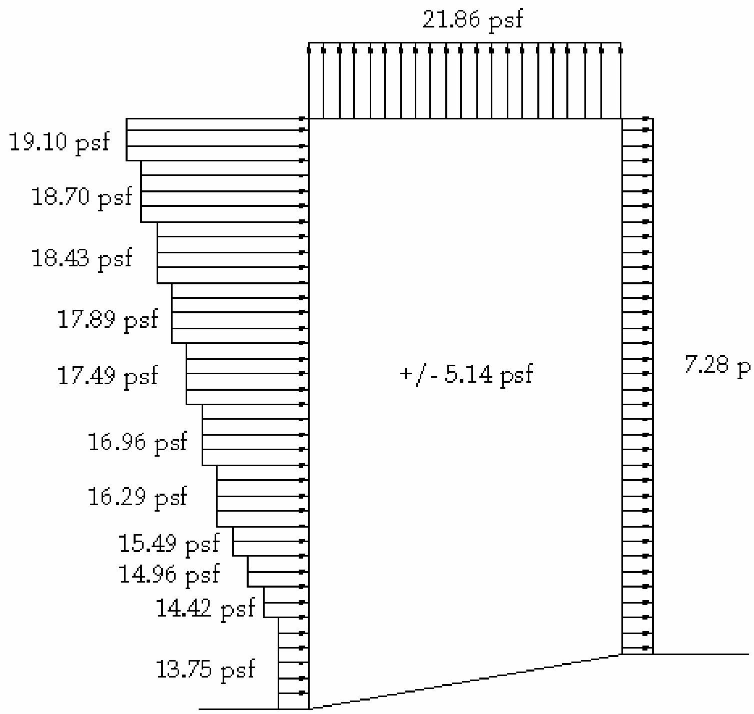


Figure 6: Wind pressure on Section C

Story Shears

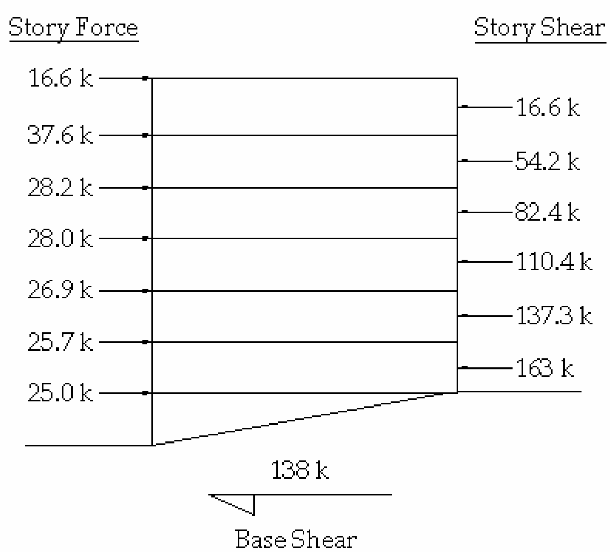


Figure 7: Story forces on Section A

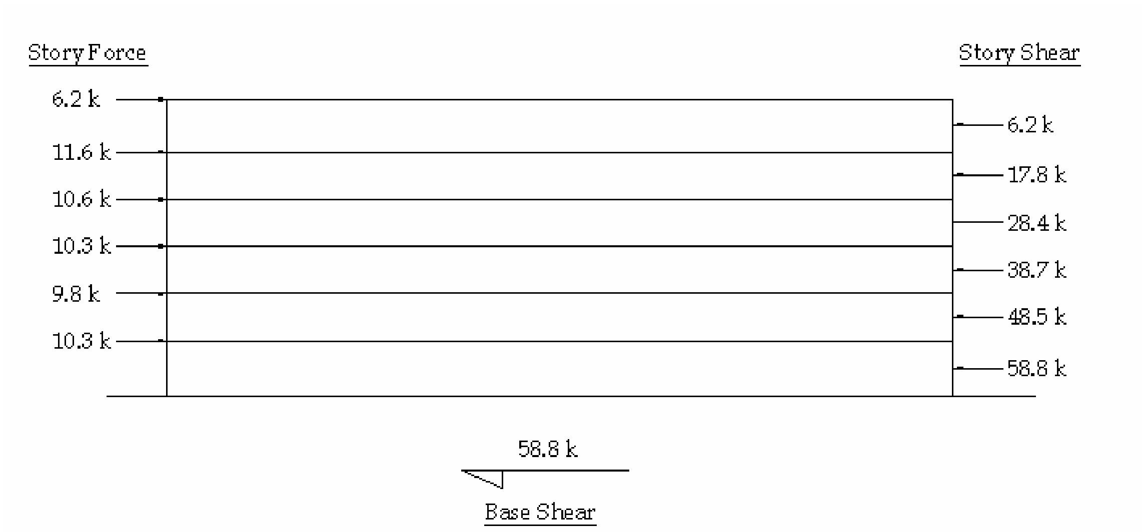


Figure 8: Story forces on section B

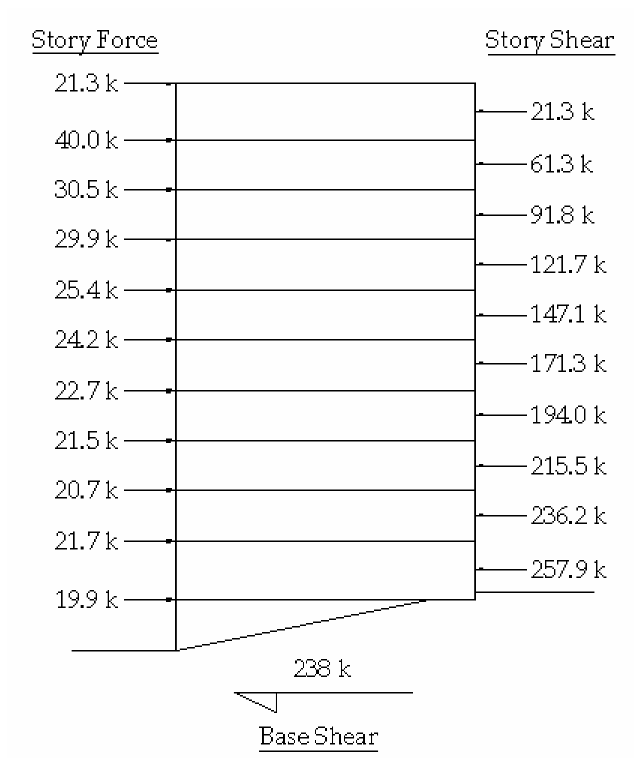


Figure 9: Story forces on section C

SEISMIC LOADS

- Seismic Zone Factor	Z = 0.15
- Site Coefficient	S_1 = 1.0
- Importance Factor	I = 1.0
	I_p = 1.0
- Analysis Procedure	Equivalent Lateral Force
- Plan Structural Irregularities	No
- Vertical Structural Irregularities	No
- Building Height	h_n = 110′
- Type of Lateral System	Typical Frame with Concrete Shear Walls R = 5 C_T = 0.020

In calculating the weight of the Towers for seismic forces, 100% of the dead load and 25% of the live load are considere. The resultant story force acts at the center of mass of the floor.

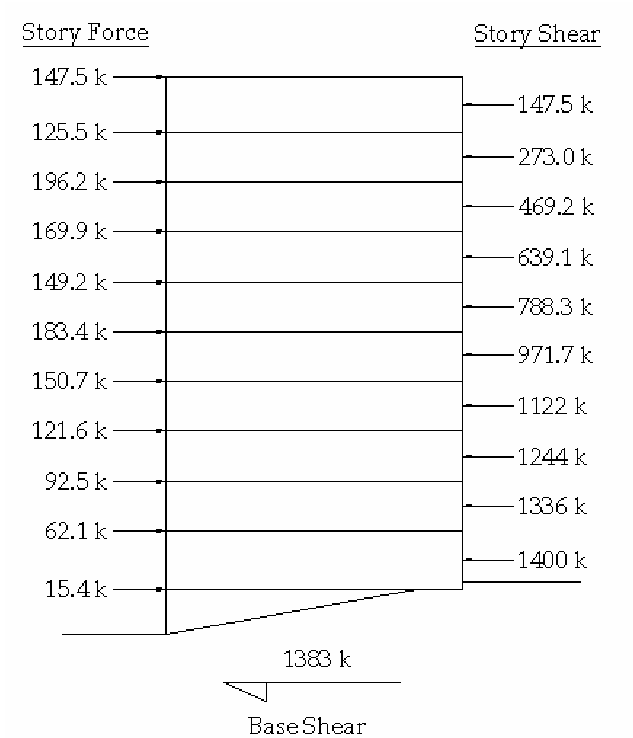


Figure 10: Seismic story forces

DESIGN CHECKS

All design checks are performed on the 5th level of the building and are compared to the actual design of the Towers. For the gravity framing check, a single bay was chosen to check the flat plate slab and a column. The design loads used in the gravity analysis is 120 psf dead load and 40 psf live load for dormitory occupancy. The load factors used are 1.2DL + 1.6LL.

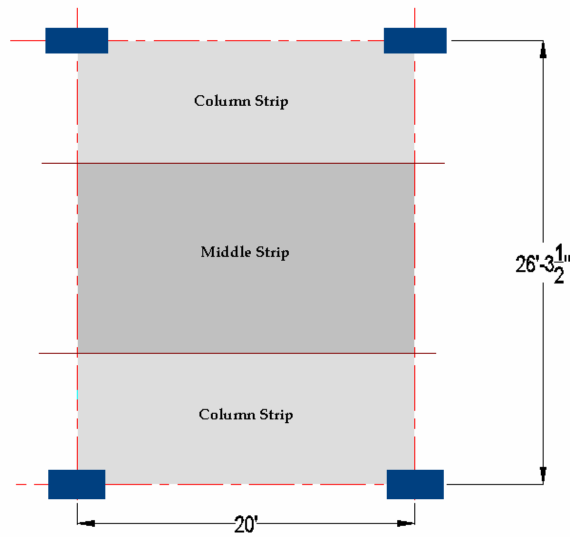


Figure 11: Typical bay used for gravity load checks

The lateral framing check will be performed on a shear wall in the stair tower. The shear wall will be checked against seismic loading and the forces will be distributed to the walls by the method of rigidity. For complete calculations of the design checks, see Appendix b.

GRAVITY LOADS

Slab Check

Using ACI 318-05, the minimum thickness of the flat plate slab based on deflection criteria is 8”. The minimum reinforcing that is allowable is #4 reinforcing bars at 12” centers. The calculated values are the same as what was built. To resist punching shear for the column through the slab, 8 #6 reinforcing bars are included over the column.

Column Check

The 20”x40” exterior column was checked using ACI 318-05. Using Microsoft Excel, an interaction diagram was developed to check the column capacity. By inspection, the maximum loads in the column fall within the limits of the interaction diagram.

LATERAL LOADS

Shear Wall Check

The wind and seismic loads analyzed at the 5th floor and are distributed as shown in Figures 11 and 12 below. The loads are distributed by rigidity and all shear walls are assumed the have equal stiffness. The story force acts at the center of mass of the floor, and the floor rotates about the center of stiffness.

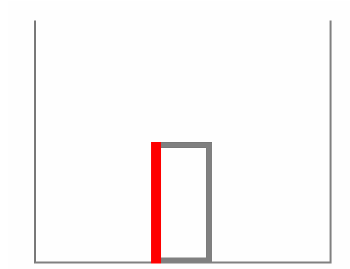


Figure 12: Shear wall used for design check calculation

For a shear wall in the stair tower with a 10″ thickness and 54′-6″ height, a design calculation was performed, using the larger forces caused by seismic loading. The reinforcing needed for the shear wall is #6 reinforcing bars at 12″ on center in the longitudinal direction and #5 reinforcing bars at 12″ on centers in the transverse direction. This is comparable to was actually designed for this shear wall. The design calls for #6 vertical bars at 12″ on center and #5 bars at 12″ on center in each face of the wall.

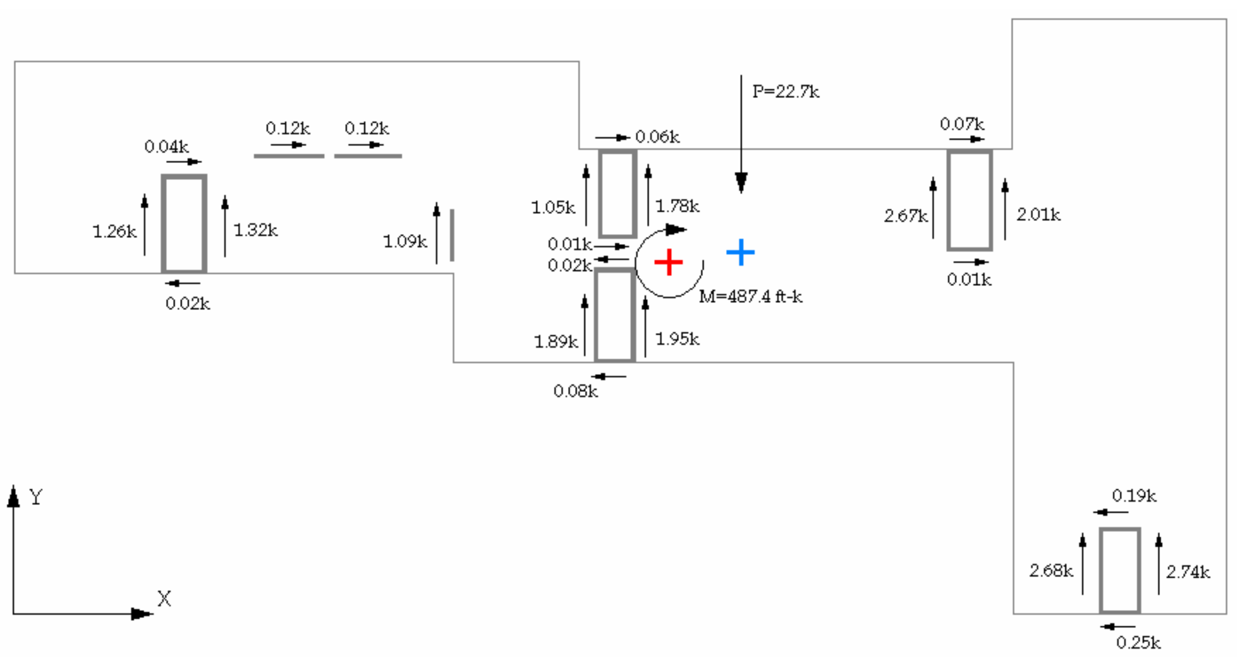


Figure 13: Wind load distribution to shear walls at the 5th floor

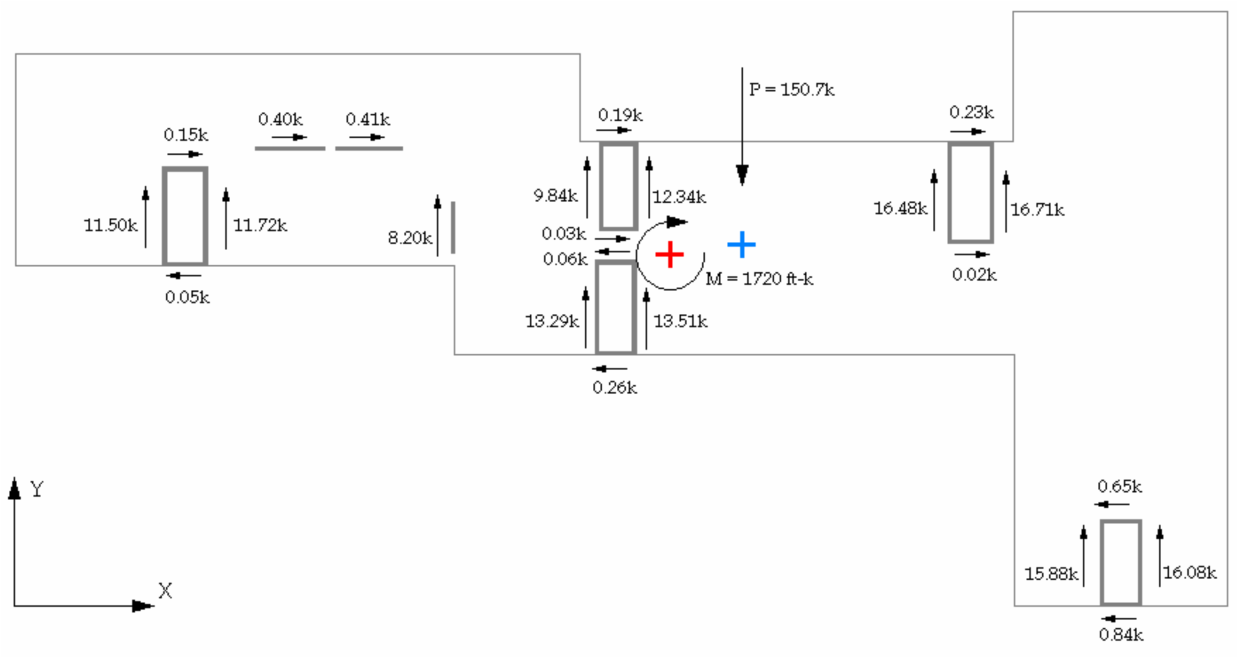


Figure 14: Seismic load distribution to shear walls at the 5th floor

APPENDIX A

LATERAL LOAD CALCULATIONS

A.1

CENTER OF RIGIDITY AND CENTER OF MASS

Floors 1 – 6

Center of Rigidity

X dimension				Y dimension			
	k	x	kx		k	y	ky
1	22.50	38.42	864.38	1	8.83	0.42	3.68
2	22.50	47.25	1063.13	2	8.83	20.58	181.82
3	14.00	107.00	1498.00	3	8.83	61.42	542.51
4	19.50	142.58	2780.37	4	8.83	81.92	723.59
5	19.50	150.58	2936.37	5	8.83	82.75	730.96
6	21.33	141.75	3024.00	6	8.83	104.42	922.34
7	21.33	151.00	3221.33	7	8.83	90.00	795.00
8	23.00	228.17	5247.83	8	8.83	112.58	994.48
9	23.00	237.42	5460.58	9	8.00	92.00	736.00
10	21.00	264.50	5554.50	10	8.00	110.67	885.33
11	21.00	273.25	5738.25	11	17.00	110.33	1875.67
SUM	228.67		37388.74	12	17.17	110.33	1894.06
				SUM	120.83		10285.44
	x =	kx = / Σk	163.51		y =	ky = / Σk	85.12

Center of Mass

X dimension			Y dimension		
Area	x	Ax	Area	y	Ay
5480	53.33	292248	5480	108.25	593210
644	122	78568	644	123.33	79424.5
9733	200.67	1953121	9733	86.92	845992
1640	268.67	440619	1640	128.5	210740
3152	268.83	847352	3152	30.5	96136
20649		3611908	20649		1825503
x =	ΣAx = / ΣA	174.919	y =	ΣAy = / ΣA	88.4064

Floors 7 – 9

Center of Rigidity

X dimension				Y dimension			
	k	x	kx		k	y	ky
1	22.50	38.42	864.38	3	8.83	61.42	542.51
2	22.50	47.25	1063.13	4	8.83	81.92	723.59
3	14.00	107.00	1498.00	5	8.83	82.75	730.96
4	19.50	142.58	2780.37	6	8.83	104.42	922.34
5	19.50	150.58	2936.37	7	8.83	90.00	795.00
6	21.33	141.75	3024.00	8	8.83	112.58	994.48
7	21.33	151.00	3221.33	9	8.00	92.00	736.00
8	23.00	228.17	5247.83	10	8.00	110.67	885.33
9	23.00	237.42	5460.58	11	17.00	110.33	1875.67
SUM	186.67		26095.99	12	17.17	110.33	1894.06
				SUM	103.17		10099.94
	x =	$\frac{kx}{\Sigma k}$ =	139.80		y =	$\frac{ky}{\Sigma k}$ =	97.90

Center of Mass

X dimension			Y dimension		
Area	x	Ax	Area	y	Ay
5480	53.33	292248	5480	108.25	593210
644	122	78568	644	123.33	79424.5
8700	190.67	1658829	8700	86.92	756204
14824		2029645	14824		1428839
x =	$\frac{\Sigma Ax}{\Sigma A}$ =	136.916	y =	$\frac{\Sigma Ay}{\Sigma A}$ =	96.3868

Floor 10 and roof

Center of Rigidity

X dimension				Y dimension			
	k	x	kx		k	y	ky
4	19.50	142.58	2780.37	3	8.83	61.42	542.51
5	19.50	150.58	2936.37	4	8.83	81.92	723.59
6	21.33	141.75	3024.00	7	8.83	90.00	795.00
7	21.33	151.00	3221.33	8	8.83	112.58	994.48
8	23.00	228.17	5247.83	9	8.00	92.00	736.00
9	23.00	137.42	3160.58	10	8.00	110.67	885.33
SUM	127.67		20370.49	SUM	51.33		4676.92
	x =	kx = / Σk	159.56		y =	ky = / Σk	91.11

Center of Mass

X dimension			Y dimension		
Area	x	Ax	Area	y	Ay
8700	190.67	1658829	8700	86.92	756204
8700		1658829	8700		756204
x =	ΣAx = / ΣA	190.67	y =	ΣAy = / ΣA	86.92

A.2

WIND PRESSURES

Velocity Wind Pressure

Windward pressure

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$
$$q_z = (0.00256)(1.03)(0.085)(1.0)(95mph)^2(1.0)$$
$$q_z = 20.23psf$$

Height	kz	qz	
0 - 15	1.03	20.23	psf
15 - 20	1.08	21.21	psf
20 - 25	1.12	22.00	psf
25 - 30	1.16	22.78	psf
30 - 40	1.22	23.96	psf
40 - 50	1.27	24.94	psf
50 - 60	1.31	25.73	psf
60 - 70	1.34	26.32	psf
70 - 80	1.38	27.10	psf
80 - 90	1.4	27.49	psf
90 - 100	1.43	28.08	psf
100 - 110	1.455	28.57	psf

Leeward pressure

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I$$
$$q_h = (0.00256)(1.455)(0.085)(1.0)(95mph)^2(1.0)$$
$$q_h = 28.57psf$$

Design Wind Pressure

Windward wall

$$p = q_z GCp - (GCpi) q_h$$
$$p = (20.23psf)(0.85)(0.8) - (\pm 0.18)(28.57)$$
$$p = 13.75 \pm 5.14psf$$

Height	qz		p			
0 - 15	20.23	psf	13.75	+/-	5.14	psf
15 - 20	21.21	psf	14.42	+/-	5.14	psf
20 - 25	22.00	psf	14.96	+/-	5.14	psf
25 - 30	22.78	psf	15.49	+/-	5.14	psf
30 - 40	23.96	psf	16.29	+/-	5.14	psf
40 - 50	24.94	psf	16.96	+/-	5.14	psf
50 - 60	25.73	psf	17.49	+/-	5.14	psf
60 - 70	26.32	psf	17.89	+/-	5.14	psf
70 - 80	27.10	psf	18.43	+/-	5.14	psf
80 - 90	27.49	psf	18.70	+/-	5.14	psf
90 - 100	28.08	psf	19.10	+/-	5.14	psf
100 - 110	28.57	psf	19.43	+/-	5.14	psf

Leeward wall with north-south wind

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\text{psf})(0.85)(-0.3) - (\pm 0.18)(28.57)$$
$$p = -7.28 \pm 5.14\text{psf}$$

Roof with north south wind

$$\frac{h}{L} = \frac{110'}{294'-8''} = 0.373$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\text{psf})(0.85)(-0.9) - (\pm 0.18)(28.57) \qquad 0 - 110'$$
$$p = -21.86 \pm 5.14\text{psf}$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\text{psf})(0.85)(-0.5) - (\pm 0.18)(28.57) \qquad 110' - 220'$$
$$p = -12.14 \pm 5.14\text{psf}$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57\text{psf})(0.85)(-0.3) - (\pm 0.18)(28.57) \qquad 220' - 294'\text{-}8''$$
$$p = -7.28 \pm 5.14\text{psf}$$

Leeward wall east-west wind

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57psf)(0.85)(-0.5) - (\pm 0.18)(28.57)$$
$$p = 12.14 \pm 5.14psf$$

Roof with east-west wind

$$\frac{h}{L} = \frac{110'}{144'-4''} = 0.762$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57psf)(0.85)(-0.9) - (\pm 0.18)(28.57) \qquad 0 - 110'$$
$$p = -21.86 \pm 5.14psf$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57psf)(0.85)(-0.5) - (\pm 0.18)(28.57) \qquad 110' - 220'$$
$$p = -12.14 \pm 5.14psf$$

$$p = q_h GCp - (GCpi)q_h$$
$$p = (28.57psf)(0.85)(-0.3) - (\pm 0.18)(28.57) \qquad 220' - 294'\text{-}8''$$
$$p = -7.28 \pm 5.14psf$$

A.3

Seismic Forces

Building Period

$$T = Ct(h_n)^{3/4}$$

$$T = (0.02)(110')^{3/4}$$

$$T = 0.679s$$

Design Base Shear

$$V = \frac{CvI}{RT}w = \frac{(0.12)(1.0)}{(5)(0.679)}(42700k)$$

$$V = 1510$$

$$Ft = 0.07VT = (0.07)(1510k)(0.679)$$

$$Ft = 71.7k$$

$$Fi = \frac{(V - Ft)w_i h_i}{\Sigma w_i h_i}$$

Level	h_i	w_i	$w_i h_i$	F_i
1	8.67	2600	22533	15.43
2	18.90	4800	90700	62.12
3	27.56	4900	135056	92.50
4	36.23	4900	177523	121.58
5	44.90	4900	219989	150.67
6	53.56	5000	267813	183.42
7	62.23	3500	217802	149.17
8	70.90	3500	248135	169.95
9	79.56	3600	286425	196.17
10	87.23	2100	183181	125.46
roof	97.90	2200	215371	147.51

$\Sigma w_i h_i$=	2064528

APPENDIX B

DESIGN CHECK CALCULATIONS

B.1

SLAB CHECK FOR GRAVITY LOADS

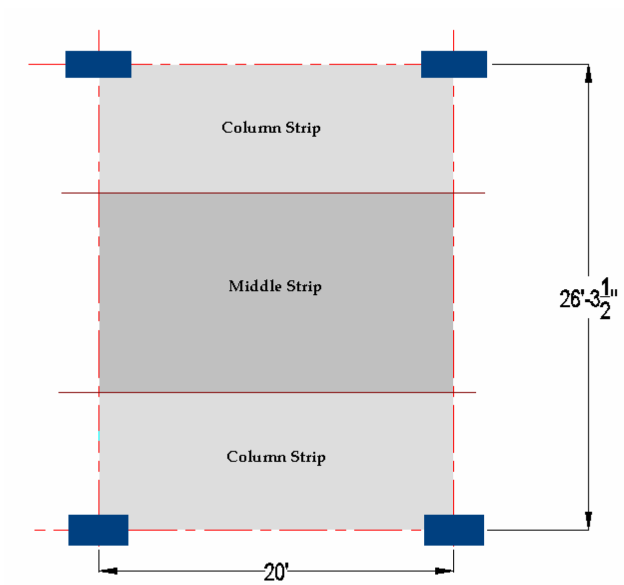


Figure 15: Typical bay used for load calculations

For a dormitory occupancy:
DL = 120 psf
LL = 40 psf

$$\omega = 1.2DL + 1.6LL$$
$$\omega = 1.2(120psf) + 1.6(40psf)$$
$$\omega = 208psf$$

l_1 = 26′-3 ½″ = 26.29′
l_2 = 20′

Clear Span:
ln = 24′-8 ½″ = 24.71′

Factored Static Moment:
Mo = 257 ′k

Column strip width:
$$\tfrac{1}{4} l_1 = \tfrac{1}{4}(26.29') = 6.57'$$

Middle strip width:
$$\tfrac{1}{2} l_1 = \tfrac{1}{2}(26.29') = 13.15'$$

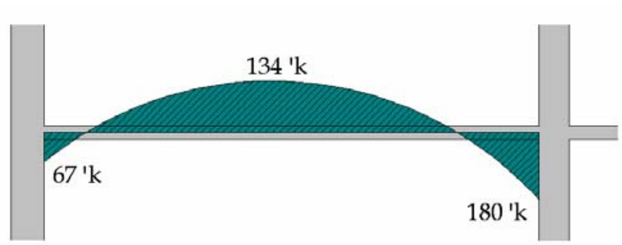


Figure16: Moment diagram for slab analysis

Using ACI 318-05:

Minimum Thickness = 5″ (two way slab without drop panels)

Minimum Thickness = $\dfrac{(20')\left(12''/1'\right)}{30} = 8'' \Rightarrow$ **use 8″ for slab thickness**

Static Moment: $$Mo = \frac{\omega l_n l_2^2}{8} = \frac{(208psf)(24.71')(20')^2}{8}$$
$$Mo = 257'^k$$

Flat plate construction, therefore no supporting interior or edge beams:

$$\alpha_{f1} = 0 \Rightarrow \alpha_{f1}\frac{l_2}{l_2} = 0$$
$$\beta_t = 0$$

End Span Static Moment Distribution per ACI 318-05			
	Distribution Factor	Moment	Percentage to Column Strip
Interior Negative Moment	0.7	180 ft-k	100%
Exterior Negative Moment	0.26	67 ft-k	100%
Positive Moment	0.52	134 ft-k	n/a

Design slab reinforcement for the maximum moment of 180 ‘k

$$180'^k\left(\frac{12"}{315.5"}\right) = 6.84'^k / ft$$
$$R = \frac{6.84'^k / ft}{(0.9)(6.5")(12")} = 97.5$$

By linear interpolation:

R	ρ
89	0.0015
97.5	0.00164
110	0.0020

$$\rho = 0.00164 < \rho_{MIN} = 0.002 \therefore \text{Use } \rho_{MIN}$$
$$\rho_{MIN} = 0.002$$
$$As_{MIN} = (0.002)bd$$
$$As_{MIN} = (0.002)(6.5")(12")$$
$$As_{MIN} = 0.192in^2 / ft$$

Use an 8” thick flat plate slab with #4 bars at 12” o.c. ***(As = 0.2 in²/ft)***

Punching Shear

$$Vc = \left(\frac{\alpha d}{b_o}\right)\sqrt{f'c}\,b_o d = \left(\frac{(30)(6.5")}{112"}\right)\sqrt{5000psi}\,(112")(6.5")$$

$$Vc = 89.6k$$

$$\phi Vc = 0.75(89.6k) = 67.2k$$

$$Vu = \omega\left(l^2 - b^2\right) = (208psf)\left(27.357^2 - 3^2\right)$$

$$Vu = 154k$$

$$Vs = \frac{Vu - \phi Vc}{\phi} = \frac{154k - 67.2k}{0.75}$$

$$Vs = 115.7k$$

$$Av = \frac{Vs}{fy(\sin\phi)} = \frac{115.7k}{(60ksi)(\sin(45))}$$

$$Av = 2.73in^2$$

Provide an additional (8) #6 reinforcing bars in the column strip to resist punching shear ***(As = 3.52 in²)***

B.2

COLUMN CHECK FOR GRAVITY LOADS

f'v =	5	ksi		
fy =	60	ksi		
As =	0.79	in2	n =	12
b =	20	in2		
d =	40	in2		

To determine if the column is adequate, the column axial load and moment must be plotted on the interaction diagram.

Pure Axial

Po =	3928.51	k

Balanced Strain

c_{bal} =	22.1939	in
Pb =	1644.27	k
Mb =	1905.89	ft-k

Pure Bending

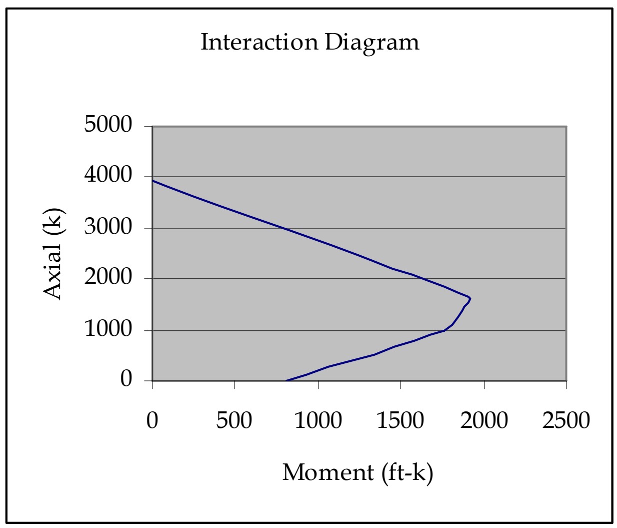
c =	4.98269	in
Mo =	812.969	ft-k

c=h

c =	20	in
Pb =	1445	k
Mb =	1887.24	ft-k

εt = 0.005 = ε4

c=	14.0625	in
Pn =	985.469	k
Mn =	1760.38	ft-k



Axial Load on Column

$$Pn = (273sf)(208psf) = 60k$$

Moment on Column

$$Mu = 67'^{k}$$

B.3

LATERAL SYSTEM CHECK

WIND LOADS

Wind Shear Wall Analysis Floors 1 - 6					
Centroid	x =	174.92	ft	Σkiy =	228.67
	y =	88.41	ft		
Load	P =	238.00	k	e =	-11.41
Moment	M =	-2715.97	ft-k		

Wall	k_{iy}	k_{ix}	x_i	y_i	d_i	d_i^2	$k_i d_i^2$	$F_{i,direct}$	$F_{i, torsion}$	$F_{i,total}$
1	22.50		38.42		136.50	18633	419242	23.42	-5.250	18.168
2	22.50		47.25		127.67	16299	366738	23.42	-4.910	18.508
3	14.00		107.00		67.92	4613	64582	14.57	-1.625	12.946
4	19.50		142.58		32.34	1046	20390	20.30	-4.753	15.543
5	19.50		150.58		24.34	592	11549	20.30	-0.811	19.485
6	21.33		141.75		33.17	1100	23471	22.20	-1.210	20.994
7	21.33		151.00		23.92	572	12205	22.20	-0.872	21.332
8	23.00		228.17		-53.25	2835	65212	23.94	2.093	26.032
9	23.00		237.42		-62.50	3906	89836	23.94	2.457	26.396
10	21.00		264.50		-89.58	8025	168519	21.86	3.216	25.073
11	21.00		273.25		-98.33	9669	203047	21.86	3.530	25.387
12		8.83		0.42	87.99	7742	68389	0.00	-1.329	-1.329
13		8.83		20.58	67.82	4600	40633	0.00	-1.024	-1.024
14		8.83		61.42	26.99	728	6435	0.00	-0.408	-0.408
15		8.83		81.92	6.49	42	372	0.00	-0.098	-0.098
16		8.83		82.75	5.66	32	283	0.00	-0.085	-0.085
17		8.83		104.42	-16.01	256	2264	0.00	0.242	0.242
18		8.83		90.00	-1.59	3	22	0.00	0.024	0.024
19		8.83		112.58	-24.18	585	5163	0.00	0.365	0.365
20		8.00		92.00	-3.59	13	103	0.00	0.049	0.049
21		8.00		110.67	-22.26	496	3964	0.00	0.304	0.304
22		17.00		110.33	-21.93	481	8173	0.00	0.637	0.637
23		17.17		110.33	-21.93	481	8254	0.00	0.643	0.643

SEISMIC LOADS

Seismic Shear Wall Analysis Floors 1 - 6										
Centroid	x =	174.92	ft	Σkiy =	228.67					
	y =	88.41	ft							
Load	P =	1383.00	k	e =	-11.41					
Moment	M =	-15782.28	ft-k							
Wall	k_{iy}	k_{ix}	x_i	y_i	d_i	d_i^2	$k_i d_i^2$	$F_{i,direct}$	$F_{i,\ torsion}$	$F_{i,total}$
1	22.50		38.42		136.50	18633	419242	136.08	-30.508	105.575
2	22.50		47.25		127.67	16299	366738	136.08	-28.534	107.549
3	14.00		107.00		67.92	4613	64582	84.67	-9.445	75.228
4	19.50		142.58		32.34	1046	20390	117.94	-27.618	90.320
5	19.50		150.58		24.34	592	11549	117.94	-4.714	113.224
6	21.33		141.75		33.17	1100	23471	129.03	-7.029	121.997
7	21.33		151.00		23.92	572	12205	129.03	-5.069	123.957
8	23.00		228.17		-53.25	2835	65212	139.11	12.165	151.272
9	23.00		237.42		-62.50	3906	89836	139.11	14.278	153.385
10	21.00		264.50		-89.58	8025	168519	127.01	18.686	145.696
11	21.00		273.25		-98.33	9669	203047	127.01	20.511	147.522
12		8.83		0.42	87.99	7742	68389	0.00	-7.720	-7.720
13		8.83		20.58	67.82	4600	40633	0.00	-5.951	-5.951
14		8.83		61.42	26.99	728	6435	0.00	-2.368	-2.368
15		8.83		81.92	6.49	42	372	0.00	-0.569	-0.569
16		8.83		82.75	5.66	32	283	0.00	-0.496	-0.496
17		8.83		104.42	-16.01	256	2264	0.00	1.405	1.405
18		8.83		90.00	-1.59	3	22	0.00	0.140	0.140
19		8.83		112.58	-24.18	585	5163	0.00	2.121	2.121
20		8.00		92.00	-3.59	13	103	0.00	0.286	0.286
21		8.00		110.67	-22.26	496	3964	0.00	1.769	1.769
22		17.00		110.33	-21.93	481	8173	0.00	3.703	3.703
23		17.17		110.33	-21.93	481	8254	0.00	3.739	3.739

Checking the shear wall for worst case, therefore earthquake load controls

Assuming the shear wall is s the same stiffness throughout the whole height:

Total Base Shear = 1380k

Shear wall takes 146k

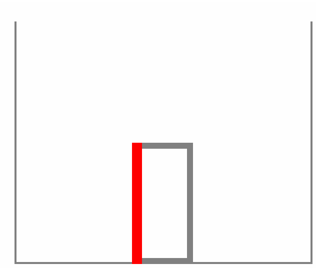


Figure 17: Shear wall used for design check calculation

$f'c = 4000psi$
$\rho_l = 0.0025$
$\rho_t = 0.0020$

$A_{sl} = (0.0025)(10")(12")$
$A_{sl} = 0.30in^2 \Rightarrow$ **Use #6 longitudinal bars at 12″ on center in both faces of wall**

$A_{st} = (0.002)(10")(12")$
$A_{st} = 0.24in^2 \Rightarrow$ **Use #5 transverse bars at 12″ on center in both faces of wall**