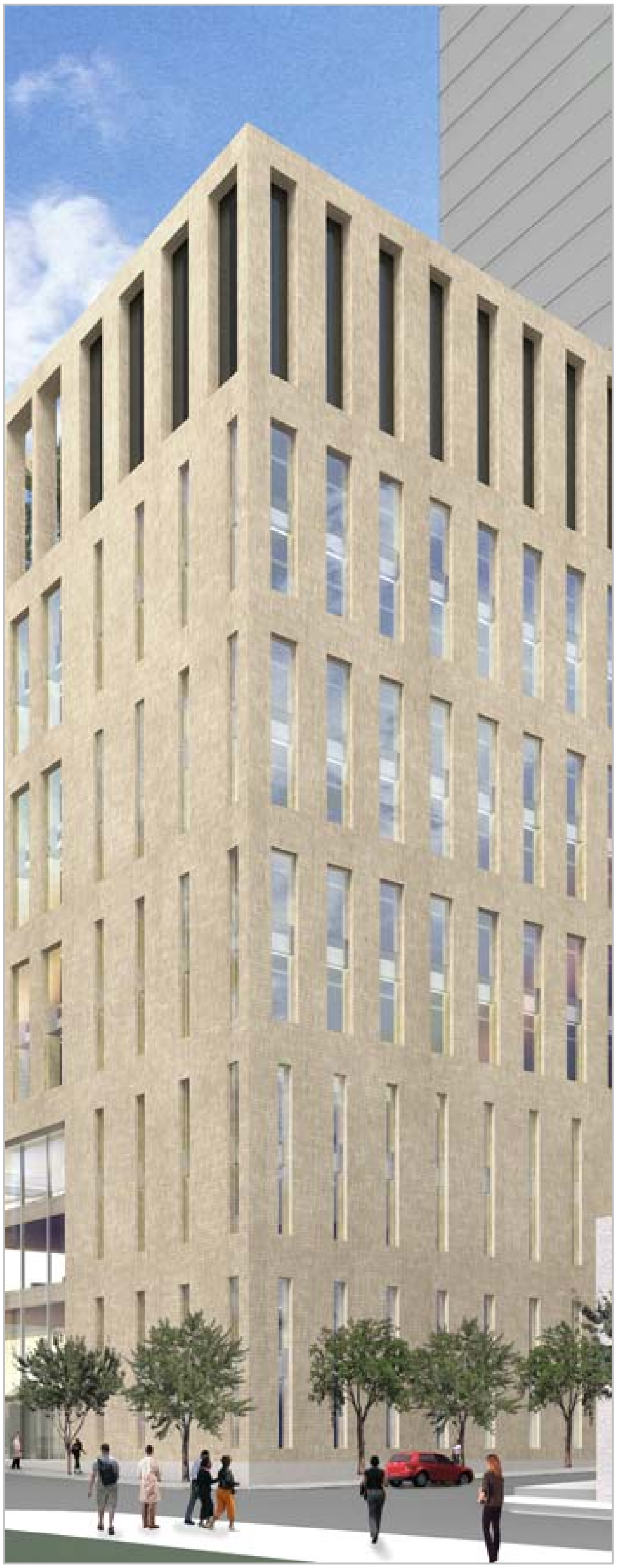
center for science & medicine

new york, ny



Technical Assignment 3

December 3, 2007

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Structural Option
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Executive Summary

This report is an investigation of the existing lateral system of the Center for Science & Medicine. The purpose of the study is to gain an understanding of how lateral loads are distributed among load resisting elements, to confirm that a logical load path exists for distribution of these forces, and to verify that lateral resisting structural members have been designed sufficiently for strength and serviceability.

First, a preliminary investigation of the lateral system was conducted by determining the relative stiffness of each lateral load resisting frame in the building. These hand calculations concluded that braced frames resist the majority of lateral load in each direction, while perimeter moment frames resist the small remainder of lateral forces in each direction. Next, a computer model of the lateral system was built in E-Tabs, and wind loads were applied to the building since they had been found to control over seismic in both directions. E-Tabs output was used to re-calculate the relative stiffness of each frame, and results were comparable to those of the hand calculations. In preliminary calculations, each moment frame was found to resist less than 10% of the total lateral load in each direction. The computer analysis found each moment frame to resist about 15%-20% of the lateral load in each direction. Although analysis results did not match exactly, it is still valid to conclude that the building's moment frames are less stiff and therefore take less lateral load than the braced frames at the core. This is probably due to the fact that each moment frame is two stories in height (a total of 30 feet), while each braced frame is only one story in height (a total of 15 feet) and thus better able to resist later load.

In addition to calculating the direct shear distribution to each frame, calculations were performed to determine shear due to torsion for each frame. The majority of calculated torsional shear was reasonably small in value, and therefore not a concern, but a few instances of high torsional shears occurred where eccentricities were large. This will require a further investigation to determine whether calculations are erred or if these frames actually need to be checked and possibly re-designed for such high torsional shears.

A check of total building drift and interstory drift was also performed using E-Tabs output. Since this is a serviceability check, loads were applied without LRFD load factors. The removal of load factors changed the governing case in the East-West direction, so both wind and seismic load cases were checked. Limitations for total building drift and interstory drift due to wind and seismic were not exceeded by actual drift values, confirming the lateral system's ability to meet serviceability drift requirements.

Finally, spot checks were performed on select elements of a typical braced frame and a typical moment frame to confirm their ability to carry the applied loads. Both the double-tee brace from Braced Frame 1 and a column from Moment Frame A were checked and found to have enough capacity to resist lateral and gravity loads (as applicable).

All back-up calculations are included in the Appendix or have already been recorded in Technical Report 1.

Introduction

The Center for Science & Medicine is a research laboratory designed for scientific investigation, discovery, and treatment. Located in New York City's Upper Manhattan, the building is organized and shaped by its architectural program. On the north and south edges of the site, two linear lab bars encompass a core of support spaces. The building's east edge links the inside to the outside with a window-covered, multi-story atrium. Situated within the building are 6 additional floors of wet lab research space, 1½ floors of clinical space, a clinical trial area, and space for research imaging. The building is 11 stories above grade with a typical floor to floor height of 15'-0", giving a total building height of 184'-0." A 40-story residential tower will also rise on the site adjacent to the lab, but the buildings are clearly defined as two separate entities. Below is a site plan showing the CSM research center, the adjacent residential tower, outdoor service areas, and surrounding buildings.

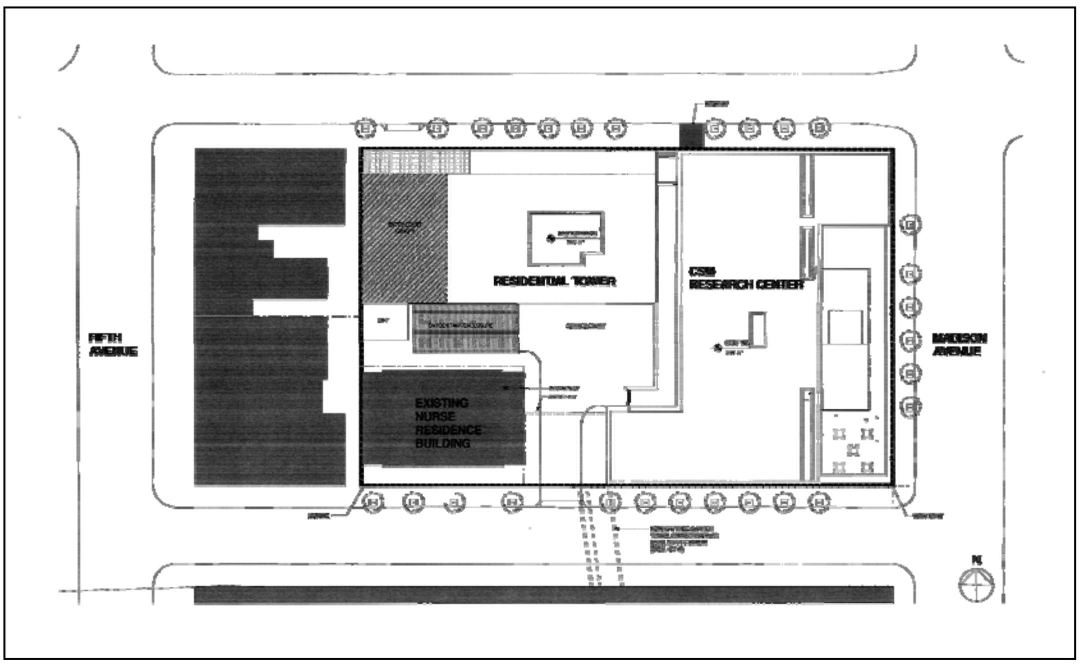


Figure 1: Site Plan

It is important to note that the Center for Science & Medicine, or CSM, is only at the 50% design development phase. Thus, the existing structural design and calculated quantities are not absolute or finalized.

This report will examine the existing lateral force-resisting system currently implemented in the design of CSM. The analysis includes a combination of SAP, E-Tabs, and hand calculations. Spot checks are also performed on various lateral elements to verify their adequacy in resisting the applied loads.

Existing Structural System

Foundation

The foundation consists of reinforced concrete spread footings ranging from 4'x4'x2' to 8'x8'x4' (l x w x h) in size, with a concrete compressive strength of $f'_c = 5000$ psi. Maximum footing depth is 49'-0" below grade, and all footings bear on sound bedrock (Class 2-65 rock with bearing capacity 40TSF or Class 1-65 rock with bearing capacity 60TSF, according to New York City Building Code). Seven (7) of the total forty-three (43) footings have been designed to support columns from both the research center and the residential tower, as dictated by their location at the CSM / tower interface. Foundation loads vary from 400 to 3200 kips.

Below grade perimeter walls consist of cast-in-place, reinforced concrete ($f'_c = 5000$ psi) braced by the below-grade floor slabs. The walls stand 48 ft in height (equivalent to 4 basement levels). These walls have been designed to resist lateral loads from soil and surcharge in addition to the vertical loads transferred from perimeter columns above. On the north and south perimeter walls, reinforced concrete pilasters support perimeter columns above. A continuous grade beam ($f'_c = 5000$ psi) supports these perimeter basement walls.

The lowest level basement floor is an 8" concrete slab on grade with a compressive strength of $f'_c = 4000$ psi, typically reinforced with #5 bars@12" each way. At typical columns, additional slab reinforcement is provided with (4)#4 bars oriented diagonally in the horizontal plane around the column base. At lateral columns located around the building core, the slab is reinforced with (12)#5 bars oriented diagonally with additional longitudinal bars arranged in a grid pattern around the column base.

Floor Framing System

CSM's existing floor system uses composite metal deck. The floor slabs typically consist of 3" metal deck with 4 ¾" normal-weight concrete topping, giving a total slab depth of 7 ¾". Thicker, normal-weight concrete slabs will be provided in spaces such as mechanical floors to meet acoustic and vibration criteria. These thickened slabs will be designed with 3" metal deck and 8" NWT concrete topping with reinforcement, giving a total slab depth of 11". Full composite action is created by 6" long, ¾" diameter shear studs, and concrete compressive strength is to be $f'_c = 4000$ psi. The composite metal deck is supported by wide flange steel beams ranging from W12x14 to W36x150 in size and spaced approximately 10'-6" on center.

There are two typical bay sizes used throughout the building, 21'-0"x 21'-0" and 43'-8" x 21'-0." Square bays typically occur within the building core, and rectangular, longer span bays typically occur around the building perimeter where research labs and clinical spaces are located. All floor framing has been designed to meet stringent vibration limits, due to the sensitivity of laboratory equipment located throughout the building, and these requirements are outlined further into the body of this report.

Lateral System

Lateral resistance to wind and seismic loads is provided by a combination of braced and moment resisting steel frames. Refer to the plan on the right for the location of each lateral element and its label. Braced frames are shown in red, and moment frames are shown in blue.

Braced Frames. In both the North-South and East-West directions, lateral loads are resisted by diagonally-braced frames located around the building core. The majority of the braced frames are braced concentrically, but some of the frames are eccentrically braced due to architectural needs (space for doors, etc.). The core is made up of (6) column bays spaced at approximately 20'x20' and using W14 column sections. Heavy double tee sections serve as diagonal braces at the core and vary from WT6x39.5 to WT6x68 in size.

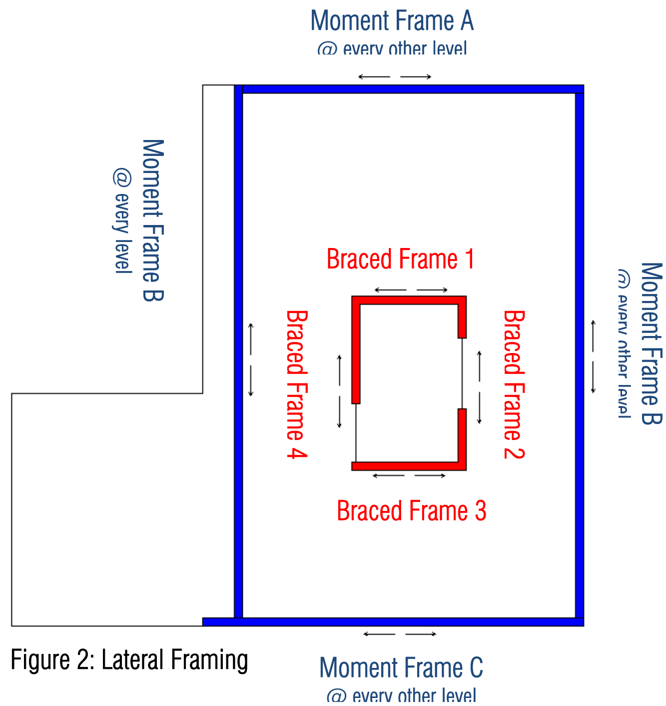


Figure 2: Lateral Framing

North-South Direction

Braced Frame 2

Braced Frame 4

East-West Direction

Braced Frame 1

Braced Frame 3

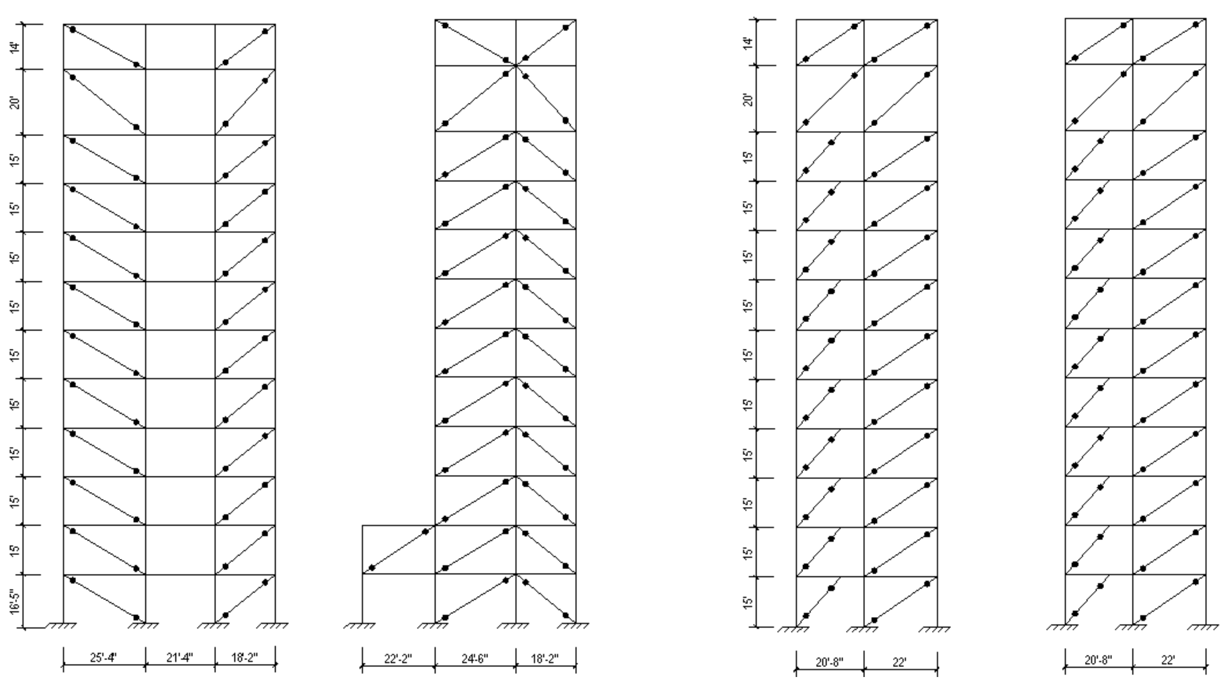


Figure 3: Braced Frames

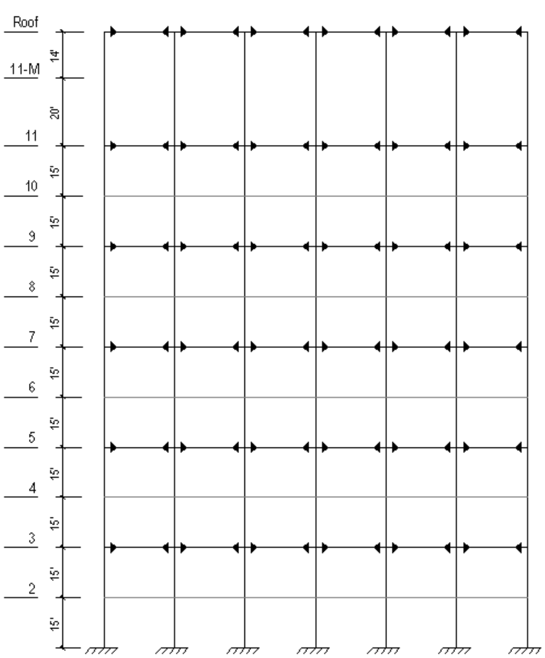
Moment Frames. In both the North-South and East-West directions, remaining lateral loads are taken by a system of beam/column moment frames located at the perimeter of the building (or just inside of it, see Moment Frame D). These moment frames have been designed to use W14 or W24 column sections spaced approximately 21'-0" on center and W30 and W24 wide flange beams. What makes these frames unique is their double-heighted configuration. The first moment connections occur on the third level and then alternate levels up through the building's roof (a total of six floors with moment connections). Thus, instead of each moment frame being 15'-0" in height (as they would have been if occurring at each floor), the moment frames are actually 30'-0" in height. Shear connections occur on even-numbered levels, and spandrel breams are set back (framing into girders), thus providing no contribution to lateral resistance at these locations.

Such a double-heighted frame configuration was necessary for CSM because of architectural design. The exterior cladding is a "perforated" system, meaning that the aesthetic pattern spans the height of two floors and the framing of every other level is visible through the windows. In other words, the exterior appears to be punched, or perforated, by alternating floor levels. For this reason, moment connections had to be placed at every other level, with intermediate levels framed by spandrel beams set back from the frame. Although this is not a desirable design from a structural point of view, it seemed to be the best solution that would satisfy both the structural integrity and the aesthetic appeal of the building.

The diagrams below depict moment frames with dark lines and arrow heads, while intermediate levels are grayed.

<u>East-West Direction</u>

Moment Frame A



Moment Frame C

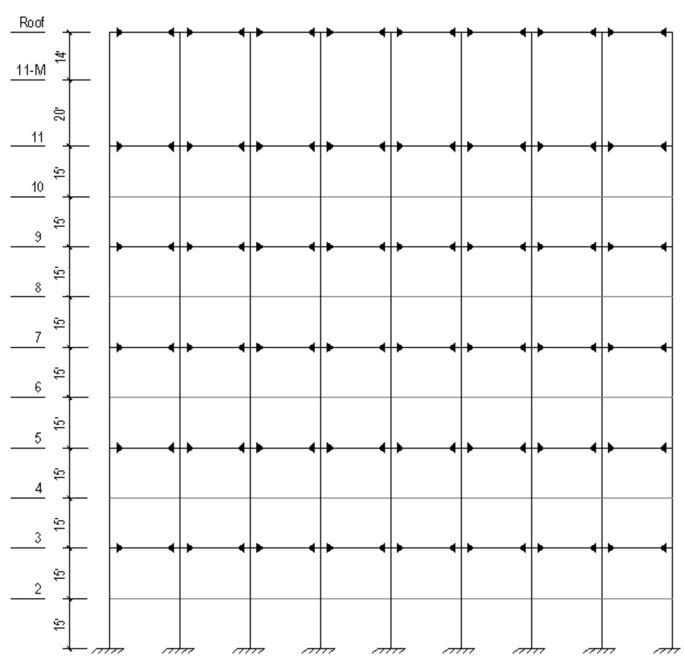
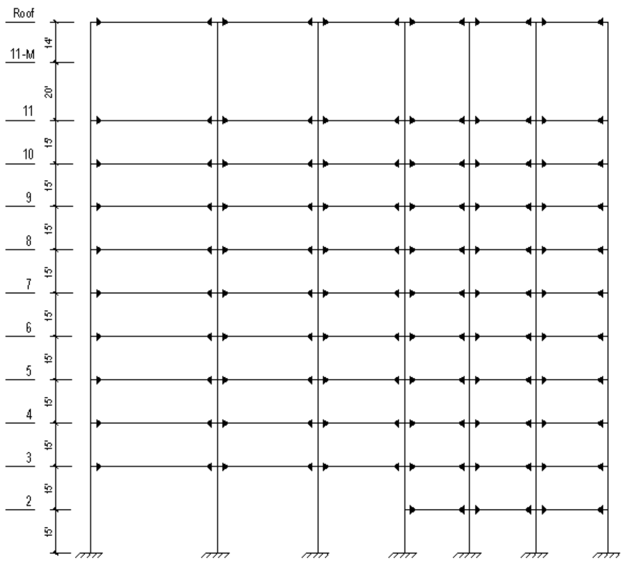


Figure 4a: Moment Frames

North-South Direction

Moment Frame B



Moment Frame D

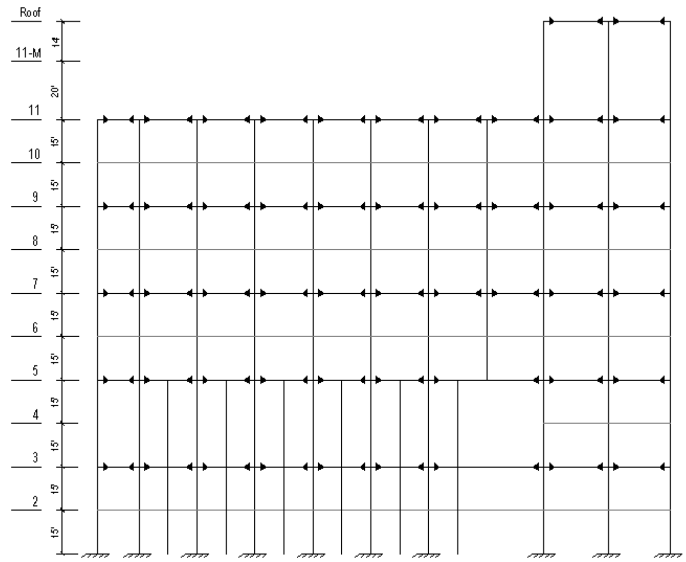


Figure 4b: Moment Frames

Roof System

The flat roof system is similar to a typical floor slab, consisting of 3" metal roof deck with 4 ¾" normal weight reinforced concrete topping and 6"x ¾" shear studs. Supporting this deck are wide flange steel beams ranging from W12x14 to W36x150 in size and spaced approximately 10'-6" on center. It is also important to note that a portion of the roof will be a green roof, but design has not progressed enough to gather significant detail at this time.

Typical Floor Plans

Architectural

Below is the architectural floor plan for the first level of CSM. Colored zones indicate the functions of each area. The building footprint stays basically the same with increasing height, except for a slight decrease in area on the southwest corner beginning on the 3rd floor.

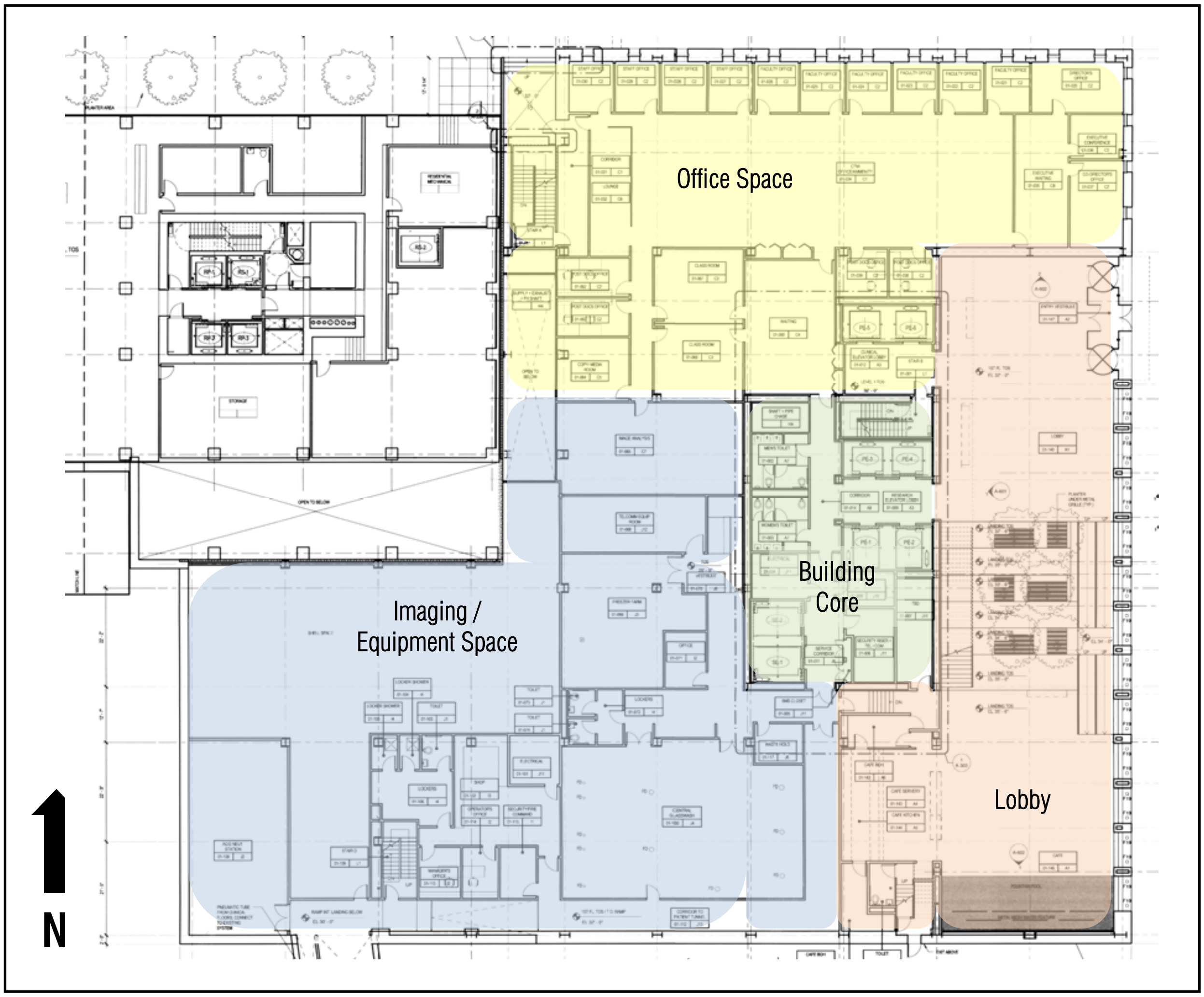


Figure 5: Level 1, Architectural Plan

Framing

Typical floor framing is shown in the figure below (laboratory floor). Composite metal deck spans the floor in the east-west direction in most areas and in the north-south direction above the atrium. Perimeter columns are spaced approximately 20'-0" to 22'-3" on center, and the longest span is 43'-8" (located on the north side of the building). A typical bay is noted with a dashed line and enlarged below.

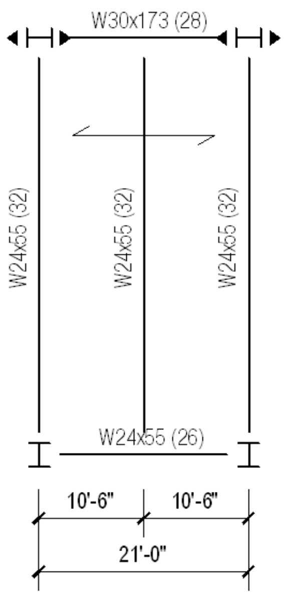


Figure 6: Typical Bay

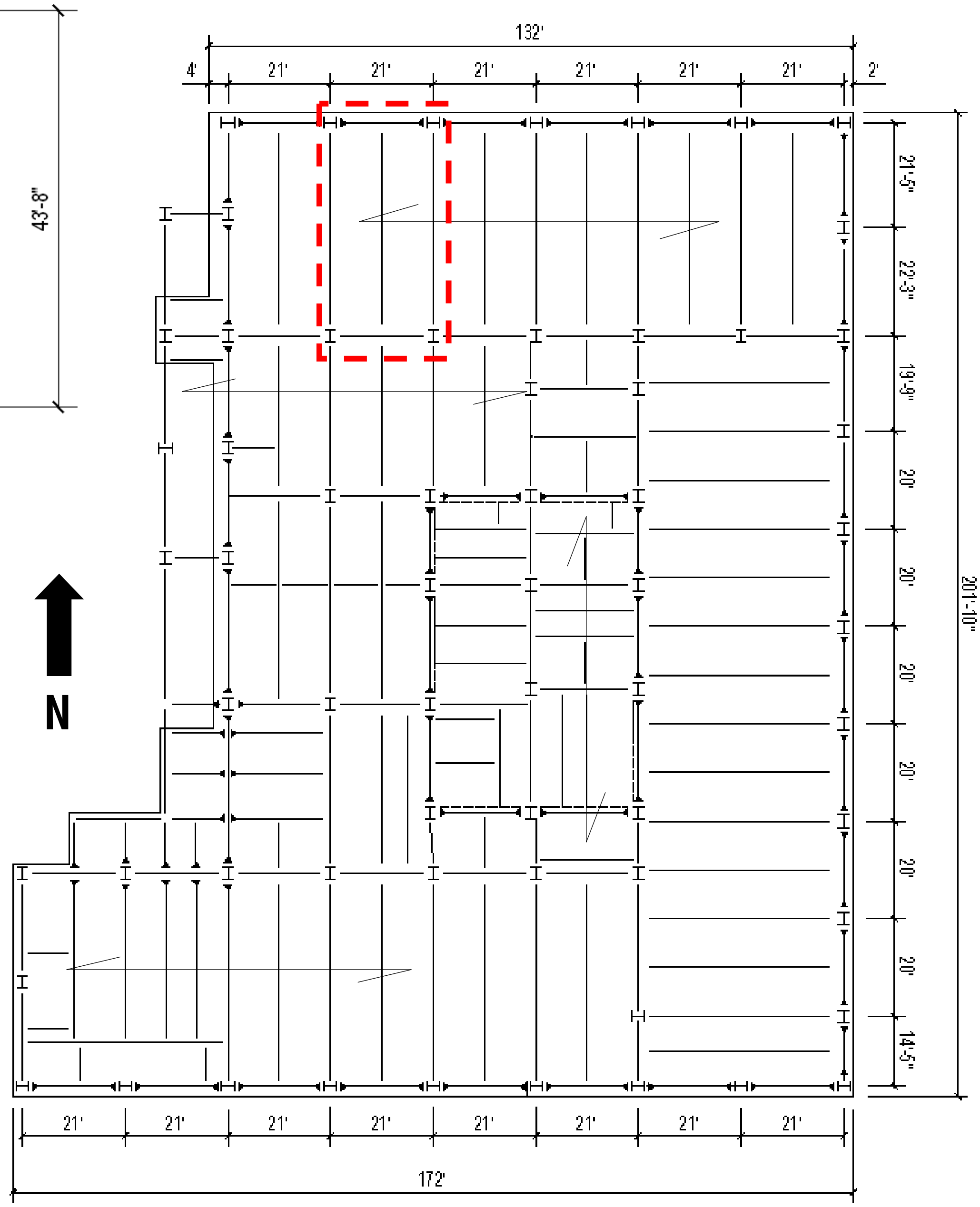


Figure 7: Level 5, Floor Framing Plan

Code & Design Requirements

Applicable Design Standards

International Building Code 2006

AISC LRFD-2005, 13th Edition (Structural Steel)

ASCE 7-05

Deflection Criteria

Floor to Floor Deflection

Typical live load deflection	L/360
Typical total deflection	L/240
Typical exterior spandrel deflection	½"

Drift Limits

Allowable Building Drift	H/400
Interstory Drift, Wind	h/400 to h/600 …… ASCE 7-05 (Section CC.1.2)
Interstory Drift, Seismic	0.015h ……………. ASCE 7-05 (Table 12.12-1)

Load Combinations

The following load combinations should be considered when combining factored loads using strength design. In the case of gravity loads only, equation 2 usually governs. When both lateral and gravity loads are carried by a member, equations 4 or 5 may govern depending on the nature of the lateral load (wind vs. seismic).

Basic Load Combinations (LRFD), ASCE7-05

1.) 1.4(D+F)

2.) 1.2(D+F+T) + 1.6(L+H) + 0.5(L_r or S or R)

3.) 1.2D + 1.6(Lr or S or R) + (L or 0.8W)

4.) 1.2D + 1.6W + L + 0.5(Lr or S or R)

5.) 1.2D + 1.0E + L + 0.2S

6.) 0.9D + 1.6W + 1.6H

7.) 0.9D + 1.0E + 1.6H

Gravity Loads

Below is a table summarizing the load values of the structural designer and of IBC 2006 (which references ASCE 7-05).

Floor / Description		Superimposed Dead Load	Design Live Load	IBC Live Load	Vibration Velocity
SC1 & SC 2					
·	Vivarium	30 psf	50 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
SC1 & SC2 Interstitial					
·	Mechanical Service	10 psf	50 psf	-	-
·	Stair	5 psf	100 psf	100 psf	-
Level 1					
·	Lobbies, Corridors	110 psf	100 psf	100 psf	-
·	Office	30 psf	50 psf	50 psf	8000 μin/s
·	Glass Wash	10 psf	125 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Level 2					
·	Wet Lab	25 psf	100 psf	-	2000 μin/s
·	Loading Dock	75 psf	250 psf	250 psf	-
·	Auditorium	40 psf	60 psf	60 psf	-
·	Stair	5 psf	100 psf	100 psf	-
Level 3					
·	Wet Lab	25 psf	100 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Level 4					
·	Lobbies, Corridors	110 psf	100 psf	100 psf	-
·	Office	30 psf	50 psf	50 psf	8000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Levels 5 - 10					
·	Office	30 psf	50 psf	50 psf	8000 μin/s
·	Wet Lab	25 psf	100 psf	-	2000 μin/s
·	Stair	5 psf	100 psf	100 psf	-
Level 11					
·	Roof Terrace	235 psf	100 psf	100 psf	-
·	Mechanical	80 psf	125 psf	-	-
·	Stair	5 psf	100 psf	100 psf	-
Roof					
·	Green Roof	60 psf	100 psf	100 psf	-
·	Snow Load	-	30 psf	22 psf (see calcs)	-
Superimposed Loads					
·	Partitions	10-20 psf	-	-	-
·	CMEP	10 psf	-	-	-
·	Finishes / Screed	5-15 psf	-	-	-
·	Roofing Membrane / Insul.	10 psf	-	-	-

Figure 8: Gravity Loads

Lateral Loads

Seismic Loads.

Seismic loads were calculated in accordance with ASCE 7-05, Chapter 12. Although previously calculated in Technical Report 1, all calculations were revised using more accurate values for areas and loadings. This yielded a much lower effective seismic weight, and, consequently, a lower base shear. The procedure and results are outlined below.

After careful study of the geotechnical report, it was concluded that the building subterranean site is primarily rock and falls under Site Class B. All other factors and accelerations were obtained from ASCE 7-05 figures, tables, and equations. The response modification factor, R, was found by assuming a dual system of moment frames in braced frames in both directions (e.g., moment frames are able to take at least 25% of the load). This assumption will be checked later in the report, after the lateral analysis has been performed. To determine the effective weight of the building, the weight of each of the building's twelve floors above grade was calculated, accounting for all slabs and columns, an approximation for beams / connections / bracing elements, and the superimposed dead loads listed in the table on the previous page. Summing the weights of each floor generated the building's effective weight, and in turn, seismic base shear. More extensive calculations and diagrams are shown in the Appendix.

Seismic Design Values, ASCE 7-05						
Occupancy	III	Table 1-1	Frames in N/S Direction: Dual System	Response Modification Coefficient	$R = 7$	Table 12.2-1
Importance Factor	$I = 1.25$	Table 11.5-1		Coefficient C_u	$C_u = 1.7$	Table 12.8-1
Site Class	B	Table 20.3-1		Fundamental Period, T	$T = 1.68$	Sec. 12.8.2
Spectral Response Acceleration, short	$S_s = 0.35$	Figure 22-1		Seismic Response Coefficient	$C_s = 0.042$	Eq. 12.8-3
Spectral Response Acceleration, 1 sec	$S_1 = 0.06$	Figure 22-2		Building Height (above grade)	$h = 184'$	
Site Coefficient, F_a	$F_a = 1.0$	Table 11.4-1				
Site Coefficient, F_v	$F_v = 1.0$	Table 11.4-2	Frames in E/W Direction: Dual System			
MCE Spectral Response Acceleration, short	$S_{MS} = 0.35$	Eq. 11.4-1		Response Modification Coefficient	$R = 7$	Table 12.2-1
MCE Spectral Response Acceleration, 1 sec	$S_{M1} = 0.06$	Eq. 11.4-2		Coefficient C_u	$C_u = 1.7$	Table 12.8-1
Design Spectral Acceleration, short	$S_{DS} = 0.233$	Eq. 11.4-3		Fundamental Period, T	$T = 1.68$	Sec. 12.8.2
Design Spectral Acceleration, 1 sec	$S_{D1} = 0.04$	Eq. 11.4-4		Seismic Response Coefficient	$C_s = 0.042$	Eq. 12.8-3
Seismic Design Category	B	Table 11.6-1		Building Height (above grade)	$h = 184'$	

Figure 9: Seismic Design Values

Conclusions. The revised base shear was calculated to be V = 1,123 kips, which is significantly less than the value obtained in Technical Report 1. The table on the following page breaks down the story forces, shears, and overturning moments at each level.

Floor	w_x (k)	h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
1							1,123.6	
2	2,328.5	15.0	74.1	172,610	0.004	4.1	1,119.5	61.4
3	2,003.0	30.0	223.2	446,987	0.009	10.6	1,108.9	318.2
4	1,875.7	45.0	425.2	797,590	0.017	18.9	1,089.9	851.8
5	2,121.2	60.0	671.8	1,425,111	0.030	33.8	1,056.1	2,029.3
6	2,121.2	75.0	958.0	2,032,056	0.043	48.2	1,007.9	3,617.0
7	2,121.2	90.0	1,280.1	2,715,400	0.057	64.4	943.5	5,800.0
8	2,121.2	105.0	1,635.7	3,469,599	0.073	82.3	861.1	8,646.1
9	2,121.2	120.0	2,022.5	4,290,288	0.091	101.8	759.3	12,218.5
10	2,121.2	135.0	2,439.1	5,173,911	0.109	122.8	636.5	16,576.9
11	3,955.6	150.0	2,883.9	11,407,669	0.241	270.7	365.8	40,610.6
Roof	3,861.8	184.0	3,990.8	15,411,530	0.326	365.8		67,300.0
		$\sum w_i h_i^k =$	47,342,753	$\sum F_x = V =$	1,123.6		$\sum M =$	158,030.1

Figure 10: Seismic Design Calculations

Effective Seismic Weight, W = 26,752.0 kips

Calculated Base Shear, V = 1,123.6 kips

Factored Base Shear, (1.0)V = 1,123.6 kips

Wind Loads.

Wind loads were calculated in accordance with ASCE 7-05, Chapter 6, using the analytical method. Although a residential tower will eventually rise adjacent to the Center for Science & Medicine on its south side, wind pressures were calculated based on the absence of this tower to account for the time CSM will be standing alone on the site. The fundamental frequency of the building was found to be less than one (period greater than one), indicating that the structure is flexible rather than rigid. It is categorized as Exposure B due to its urban location. The building is not quite a square, with the North-South direction (200’-0”) slightly longer than the East-West direction (172’-0”). Calculations are summarized below and detailed in the Appendix.

Wind Design Values, ASCE 7-05		
Occupancy	III	Table 1-1
Importance Factor	I = 1.15	Table 6-1
Basic Wind Speed	100 mph	Figure 6-1
Wind Directionality Factor	Kd = 0.85	Table 6-4
Topographic Factor	kzt = 1	Sec. 6.5.7.2
Gust Effect Factor	N-S: G_f = 0.81	Sec. 6.5.8
	E-W: G_f =0.54	
Internal Pressure Coefficient	Gcpi = +/- 0.18	Figure 6-5
External Pressure Coefficients	Windward, Cp = 0.8	Figure 6.6
	Leeward, Cp = -0.46	

Figure 11: Wind Design Values

Wind Pressures on North-South Frame
B = 172 ft, L = 200 ft

Floor	Height (ft)	hx	Kz	qz	Pressures (psf) N/S windward		Total	Force (kips)	Factored Force (x1.6)	Shear (kips)	Factored Shear	Moment (ft-k)	Factored Moment (x1.6)
					N/S windward	N/S leeward	Total						
Roof	34	184	1.18	29.53	19.13 ± 5.32 = 24.4	-11.00 ± 5.32 = -16.3	40.8	119.2	190.7	119.2	190.7	4,052.9	6,484.6
11	15	150	1.11	27.78	18.00 ± 5.32 = 23.3	-11.00 ± 5.32 = -16.3	39.6	170.3	272.5	289.5	463.2	2,554.9	4,087.8
10	15	135	1.08	27.03	17.51 ± 5.32 = 22.8	-11.00 ± 5.32 = -16.3	39.1	101.6	162.6	391.2	625.8	1,524.3	2,438.9
9	15	120	1.04	26.02	16.86 ± 5.32 = 22.2	-11.00 ± 5.32 = -16.3	38.5	100.2	160.3	491.3	786.1	1,502.4	2,403.8
8	15	105	1.00	25.02	16.22 ± 5.32 = 21.5	-11.00 ± 5.32 = -16.3	37.8	98.5	157.6	589.8	943.7	1,477.3	2,363.6
7	15	90	0.96	24.02	15.57 ± 5.32 = 20.9	-11.00 ± 5.32 = -16.3	37.2	96.8	154.9	686.6	1098.6	1,452.2	2,323.5
6	15	75	0.91	22.77	14.76 ± 5.32 = 20.1	-11.00 ± 5.32 = -16.3	36.4	94.9	151.9	781.5	1250.5	1,423.9	2,278.3
5	15	60	0.85	21.27	13.78 ± 5.32 = 19.1	-11.00 ± 5.32 = -16.3	35.4	92.6	148.2	874.2	1398.7	1,389.4	2,223.1
4	15	45	0.785	19.64	12.73 ± 5.32 = 18.0	-11.00 ± 5.32 = -16.3	34.4	90.0	144.0	964.2	1542.7	1,350.2	2,160.3
3	15	30	0.70	17.52	11.35 ± 5.32 = 16.7	-11.00 ± 5.32 = -16.3	33.0	86.9	139.0	1051.0	1681.7	1,303.1	2,085.0
2	15	15	0.57	14.26	9.24 ± 5.32 = 14.6	-11.00 ± 5.32 = -16.3	30.9	82.4	131.8	1133.4	1813.5	1,235.7	1,977.1
1	0	0						39.8	63.7	1173.3	1877.2	0.0	0.0
							Base Shear =	1,173.3	1,877.2		M =	19,266.2	30,826.0

Figure 12: Calculated Wind Pressures, N-S

Wind Pressures on East-West Frame
B = 200 ft, L = 172 ft

Floor	Height (ft)	hx	Kz	qz	Pressures (psf)			Force (kips)	Factored Force (x1.6)	Shear (kips)	Factored Shear	Moment (ft-k)	Factored Moment (x1.6)
					E/W windward	E/W leeward	Total						
Roof	34	184	1.18	29.53	12.76 ± 5.32 = 18.1	-7.33 ± 5.32 = -12.6	30.7	104.5	167.1	104.5	167.1	3,551.4	5,682.2
11	15	150	1.11	27.78	12.00 ± 5.32 = 17.3	-7.33 ± 5.32 = -12.6	30.0	149.4	239.0	253.9	406.2	2,241.0	3,585.6
10	15	135	1.08	27.03	11.68 ± 5.32 = 17.0	-7.33 ± 5.32 = -12.6	29.6	89.4	143.1	343.3	549.2	1,341.1	2,145.8
9	15	120	1.04	26.02	11.24 ± 5.32 = 16.6	-7.33 ± 5.32 = -12.6	29.2	88.3	141.2	431.5	690.4	1,324.1	2,118.5
8	15	105	1.00	25.02	10.81 ± 5.32 = 16.1	-7.33 ± 5.32 = -12.6	28.8	87.0	139.2	518.5	829.6	1,304.6	2,087.4
7	15	90	0.96	24.02	10.38 ± 5.32 = 15.7	-7.33 ± 5.32 = -12.6	28.3	85.7	137.1	604.2	966.7	1,285.2	2,056.3
6	15	75	0.91	22.77	9.84 ± 5.32 = 15.2	-7.33 ± 5.32 = -12.6	27.8	84.2	134.7	688.4	1101.4	1,263.3	2,021.2
5	15	60	0.85	21.27	9.19 ± 5.32 = 14.5	-7.33 ± 5.32 = -12.6	27.2	82.4	131.9	770.8	1233.3	1,236.5	1,978.4
4	15	45	0.785	19.64	8.49 ± 5.32 = 13.8	-7.33 ± 5.32 = -12.6	26.5	80.4	128.7	851.2	1362.0	1,206.1	1,929.8
3	15	30	0.70	17.52	7.57 ± 5.32 = 12.9	-7.33 ± 5.32 = -12.6	25.5	78.0	124.8	929.2	1486.7	1,169.6	1,871.4
2	15	15	0.57	14.26	6.16 ± 5.32 = 11.5	-7.33 ± 5.32 = -12.6	24.1	74.5	119.2	1003.7	1605.9	1,117.3	1,787.7
1	0							36.2	57.9	1039.9	1663.8	0.0	0.0
							Base Shear =	1,039.9	1,663.8		M =	17,040.2	27,264.3

Figure 13: Calculated Wind Pressures, E-W

Conclusions. The base shear was calculated to be 1,173.3 kips (1,877.2, factored) on the North-South frames and 1,039.9 kips (1,663.8, factored) on the East-West frames. Once load factors are applied (1.6 for wind, 1.0 for seismic), it is observed that wind loads will control over seismic:

Seismic, Vb = 1,123.6 k < Wind E-W, Vb = 1,663.8 k < Wind N-S, Vb = 1,877.2 k

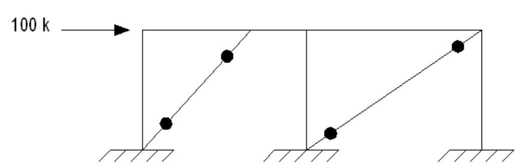
Preliminary Lateral Investigation

A simplified analysis of the lateral system was performed using a combination of computer modeling and hand calculations. The purpose of this analysis is to provide insight into how loads are distributed to each lateral element. The results show indicate how much of the total load each braced frame and moment frame will take, per level.

Relative Stiffness.

To find the relative stiffness of each braced frame and moment frame in the building, the frames were modeled individually in SAP. The frames were separated by level, and columns were fixed at their bases. With all braces pinned and moment frames properly restrained at the connection points, a 100 kip horizontal load was applied at the top of each frame. The deflection of each frame was read from computer output, and a simple calculation of P/Δ yielded the stiffness of each frame. Finally, frame stiffness was summed at each level, and the relative stiffness of each brace was found. Below is a representative example of how each one-story frame was loaded.

Braced Frame 1, Level 10



Moment Frame A, Level 3

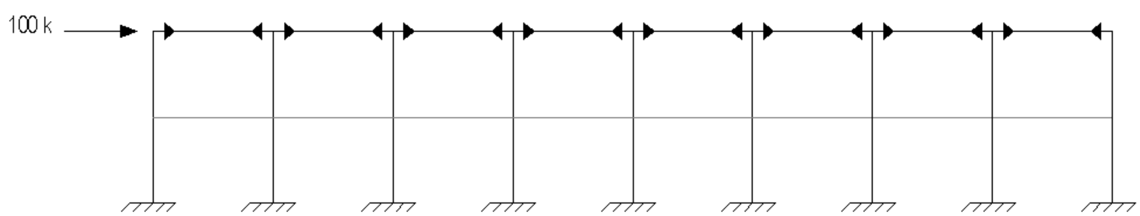


Figure 14: Frame Loading for Relative Stiffness

The following page presents a summary of the relative stiffness of each frame in each direction, at each level of the building.

SUMMARY: EAST-WEST DIRECTION				
Level	BRACED FRAME 1 Relative Stiffness	BRACED FRAME 3 Relative Stiffness	MOMENT FRAME A: Relative Stiffness	MOMENT FRAME C: Relative Stiffness
Roof	0.52	0.44	0.018	0.02
Level 11 Mezz	0.53	0.47	-	-
Level 11	0.48	0.40	0.05	0.04
Level 10	0.55	0.45	0.00	0.00
Level 9	0.48	0.39	0.06	0.04
Level 8	0.55	0.45	0.00	0.00
Level 7	0.47	0.39	0.06	0.05
Level 6	0.55	0.45	0.00	0.00
Level 5	0.44	0.44	0.06	0.03
Level 4	0.50	0.50	0.00	0.00
Level 3	0.44	0.44	0.06	0.04
Level 2	0.50	0.50	0.00	0.00

Figure 15: Relative Stiffness, Frames in the East-West Direction

SUMMARY: NORTH-SOUTH DIRECTION				
Level	BRACED FRAME 2 Relative Stiffness	BRACED FRAME 4 Relative Stiffness	MOMENT FRAME D: Relative Stiffness	MOMENT FRAME B: Relative Stiffness
Roof	0.51	0.48	0.004	0.00
Level 11 Mezz	0.50	0.50	-	-
Level 11	0.51	0.38	0.07	0.03
Level 10	0.60	0.33	0.06	0.00
Level 9	0.53	0.35	0.07	0.04
Level 8	0.54	0.38	0.08	0.00
Level 7	0.51	0.37	0.08	0.05
Level 6	0.54	0.38	0.08	0.00
Level 5	0.53	0.35	0.09	0.03
Level 4	0.57	0.34	0.09	0.00
Level 3	0.48	0.42	0.08	0.03
Level 2	0.64	0.32	0.04	0.00

Figure 16: Relative Stiffness, Frames in the North-South Direction

Conclusions. This analysis indicates that the braced frames around the core are much stiffer than the two-story moment frames at the perimeter. While each braced frames takes anywhere from 32% - 64% of the lateral force on a given level, each moment frame only takes from 2% - 5% of the lateral force on a given level. These results make sense because the braced frames are braced at every level (every 15 feet) with heavy double-tee sections, while the moment frames only provide resistance at every other level and stand 30 feet in height. One would expect the braced frames to be more rigid than the moment frames in this case. However, if the calculations above are accurate, then the moment frames CANNOT be considered to act in a dual system with the braced frames, since they do not carry at least 25% of the load. This theory will be tested by a computerized analysis, which is summarized in a later section of this report.

Center of Rigidity.

The center of rigidity (COR) location was determined using the relative stiffness of each frame and using a zero reference point at the South-West corner of the building. Since the centers of rigidity were relatively close in value on each level, an average of all centers was taken to get one center of rigidity for the entire building. COR values for each level are shown below, and the average value is located on the basic building floor plan (Level 1). Hand calculated values are compared to values computed by E-Tabs and are found to be very accurate.

Center of Rigidity							
	Hand Calculations					E-Tabs Calculations	
Level	∑R	∑Rx	∑Ry	x	y	x	y
2	1.0	1,835	1,088	1834.6	1088.0	1812.5	1135.7
3	1.0	1,783	1,103	1782.7	1102.8	1829.4	1113.3
4	1.0	1,774	1,088	1773.8	1088.0	1826.2	1097.3
5	1.0	1,785	1,103	1785.4	1102.8	1821.2	1092.8
6	1.0	1,765	1,270	1765.0	1270.4	1809.9	1090.8
7	1.0	1,817	1,111	1817.4	1110.7	1794.9	1092.8
8	1.0	1,765	1,125	1765.0	1125.1	1774.3	1093.4
9	1.0	1,791	1,125	1791.1	1125.3	1760.2	1100.2
10	1.0	1,789	1,125	1789.1	1125.1	1759.5	1106.7
11	1.0	1,771	1,108	1771.4	1108.2	1749.6	1112.4
11 mezz	1.0	1,790	1,110	1790.0	1110.3	1787.1	1117.0
Roof	1.0	1,783	1,510	1783.2	1510.1	1722.2	1177.1
			Average:	1787.4	1155.6	1787.3	1110.8

Figure 17: Center of Rigidity (inches), by floor

X: East-West

Y: North-South

The location of the average COR makes sense, since the layout of the lateral elements is basically symmetrical, and the sharing of lateral loads is not heavily concentrated to one side or another. COR values will be used later in this report when determining eccentricities for seismic loads resulting in torsional shear.

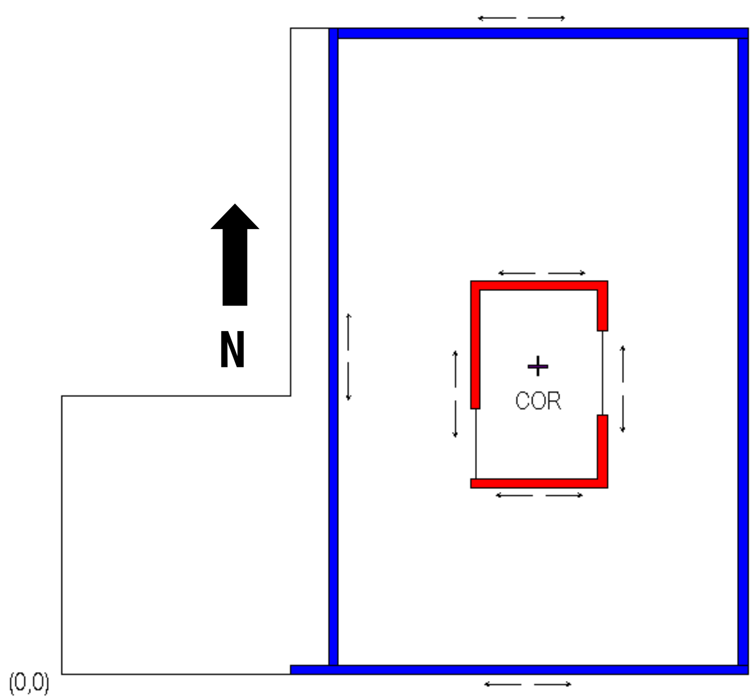


Figure 18: Level 1 Floor Plan with COR shown

Center of Mass.

The center of mass (COM) can be determined in a similar manner as the COR by using the same (0,0) reference point and by accounting for the masses of the floor system, framing systems, and façade materials. Due to time constraints, the COM was not calculated by hand. Instead, the values determined by E-Tabs will be used (without a check by hand).

Center of Mass (E-Tabs Output)		
Level	x	y
2	1467.14	1194.47
3	1951.62	1033.39
4	1648.34	1133.91
5	1645.15	1133.09
6	1651.78	1131.64
7	1647.82	1132.82
8	1652.91	1131.98
9	1647.31	1132.50
10	1651.48	1129.63
11	1534.19	1221.27
11 mezz	1619.14	1124.91
Roof	1299.55	1070.22

Figure 19: Center of Mass (in), by floor

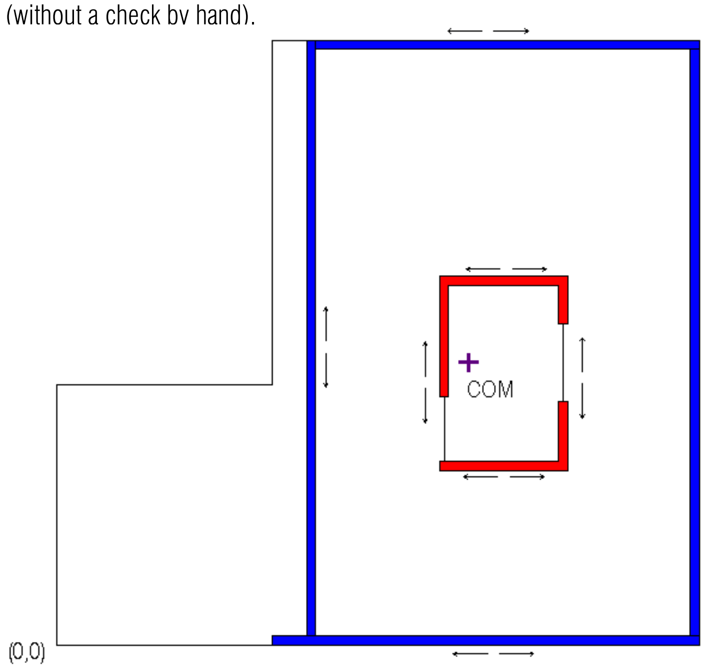


Figure 20: Center of Mass, shown on typical lab floor

Center of Geometry.

The center of geometry (COG) corresponds to the geometric centroid of the floor diaphragm at each level. Determining these locations involves a simple calculation involving areas and distances from the (0,0) reference point. Hand calculations are summarized in the table below indicating the geometric center of each level of the building. The COG will be used for finding eccentricities of wind loads when torsion is examined later in this report.

Center of Geometry							
Level	∑A	∑Ax	∑Ay	x (ft)	x (in)	y (ft)	y (in)
1	33,429	4,073,158	2,966,131	121.8	1462.1	88.7	1064.8
2	25,921.6	2,512,954	2,042,005	96.9	1163.3	78.8	945.3
3	26,991	3,610,668	2,573,072	133.8	1605.3	95.3	1144.0
4	24,270	3,103,402	2,476,539	127.9	1534.4	102.0	1224.5
5	28,592	3,920,355	2,698,968	137.1	1645.4	94.4	1132.8
6	28,593	3,920,355	2,698,968	137.1	1645.3	94.4	1132.7
7	28,594	3,920,355	2,698,968	137.1	1645.2	94.4	1132.7
8	28,595	3,920,355	2,698,968	137.1	1645.2	94.4	1132.6
9	28,596	3,920,355	2,698,968	137.1	1645.1	94.4	1132.6
10	28,597	3,920,355	2,698,968	137.1	1645.1	94.4	1132.6
11	28,598	3,920,355	2,698,968	137.1	1645.0	94.4	1132.5
11-M	4,800	781,105	413,023	162.7	1952.8	86.0	1032.6
Roof	22,117	2,696,580	2,199,853	121.9	1463.1	99.5	1193.6

Figure 21: Center of Geometry, by floor

Computerized Lateral Analysis

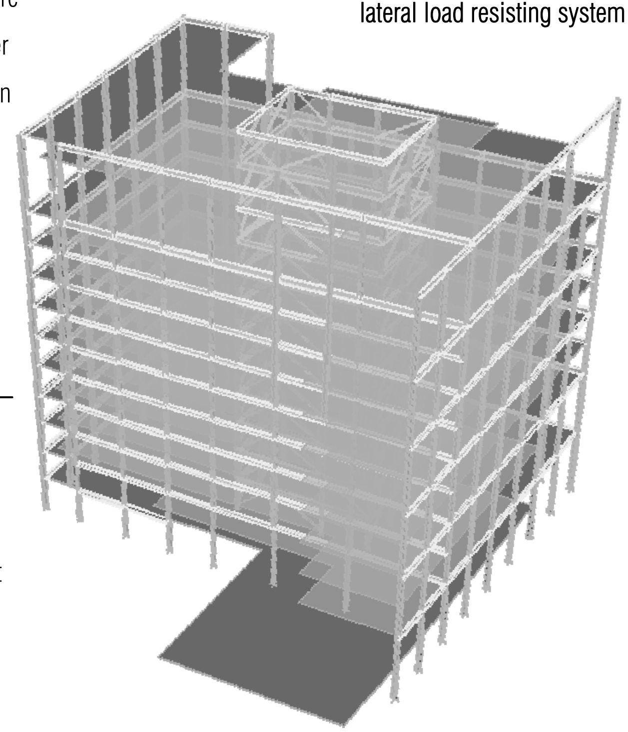
E-Tabs Model.

To carry out the lateral analysis, a basic model of CSM was constructed using E-Tabs. Only lateral elements were modeled, since gravity members would have no effect on the distribution of lateral loads. After inserting the double-height perimeter moment frames and the braced frames at the core, a floor diaphragm was modeled on each level (with property "none"). The mass of each floor was assigned to these diaphragms at their centers of mass (the masses of each floor was previously calculated in Technical Assignment 1 and shown in the Appendix of this report).

It was decided to refrain from modeling the four basement levels in E-Tabs as a part of the lateral system. Perimeter moment frames run from the roof to the ground level, but obviously do not continue below ground. The frames sit on concrete perimeter walls, which have been designed for gravity loads and lateral soil loads only, thus unable to be considered part of the lateral load resisting system. The braced frames, however, actually run all the way from the roof to the bottom level basement, 48 feet underground. This leads one to question whether these braced frames in the basement should be accounted for in analyzing lateral load distribution. To resolve this question, the E-Tabs model simply ignores the sub-grade levels and their framing systems for the purposes of this report. If it is found that these frames do in fact play a crucial role in lateral load resistance, they will be factored in accordingly.

After the lateral elements had been modeled in E-Tabs, horizontal wind loads were applied in both the North-South and East-West directions, which were previously calculated in Technical Assignment 1 and found to control over seismic design loads. Four load combinations were set up: the application of 1.6W in the positive X, positive Y, negative X, and negative Y directions (where X is East-West and Y is North-South). A simple reading of resulting forces in each member indicated how the lateral load was distributed to each floor and to each frame.

Figure 22: E-Tabs Model, lateral load resisting system



Distribution of Direct Shear.

The results from running this analysis were reasonably similar to the preliminary calculations done by hand. In both analyses, the majority of the lateral loads were distributed to the braced frames, while any remaining load was distributed to the moment frames. However, the exact distributions of these loads were slightly different between analyses.

In preliminary hand calculations, each moment frame was found to resist less than 10% of the total lateral load in each direction. The computer analysis, however, found each moment frame to resist about 15%-20% of the lateral load in each direction. Although analysis results did not match exactly, it is still valid to conclude that the building's braced frames at the core are stiffer and therefore take more lateral load than the perimeter moment frames. The tables below display the relative stiffness of each frame on each level as calculated by E-Tabs and as calculated by hand. See the Appendix for more detailed data and calculations.

SUMMARY: EAST-WEST DIRECTION (X)								
Level	BRACED FRAME 1 Relative Stiffness		BRACED FRAME 3 Relative Stiffness		MOMENT FRAME A: Relative Stiffness		MOMENT FRAME C: Relative Stiffness	
	By Hand	E-Tabs	By Hand	E-Tabs	By Hand	E-Tabs	By Hand	E-Tabs
Roof	0.52	0.40	0.44	0.80	0.018	0.310	0.02	0.22
Level 11-M	0.53	0.90	0.47	0.10	0.00	0.00	0.00	0.00
Level 11	0.48	0.30	0.40	0.26	0.05	0.21	0.04	0.23
Level 10	0.55	0.47	0.45	0.53	0.00	0.00	0.00	0.00
Level 9	0.48	0.31	0.39	0.37	0.06	0.15	0.04	0.17
Level 8	0.55	0.44	0.45	0.56	0.00	0.00	0.00	0.00
Level 7	0.47	0.31	0.39	0.40	0.06	0.15	0.05	0.14
Level 6	0.55	0.43	0.45	0.57	0.00	0.00	0.00	0.00
Level 5	0.44	0.38	0.44	0.35	0.06	0.13	0.03	0.14
Level 4	0.50	0.52	0.50	0.48	0.00	0.00	0.00	0.00
Level 3	0.44	0.40	0.44	0.38	0.06	0.12	0.04	0.10
Level 2	0.50	0.34	0.50	0.32	0.00	0.19	0.00	0.14

Figure 23: Comparison of relative stiffness in E-W direction

SUMMARY: NORTH-SOUTH DIRECTION (Y)								
Level	BRACED FRAME 2 Relative Stiffness		BRACED FRAME 4 Relative Stiffness		MOMENT FRAME D: Relative Stiffness		MOMENT FRAME B: Relative Stiffness	
	By Hand	E-Tabs	By Hand	E-Tabs	By Hand	E-Tabs	By Hand	E-Tabs
Roof	0.51	0.26	0.48	0.65	0.004	0.060	0.00	0.03
Level 11 Mez	0.50	0.00	0.50	1.00	0.00	0.00	0.00	0.00
Level 11	0.51	0.44	0.38	0.24	0.07	0.26	0.03	0.06
Level 10	0.60	0.56	0.33	0.21	0.06	0.17	0.00	0.06
Level 9	0.53	0.55	0.35	0.21	0.07	0.17	0.04	0.06
Level 8	0.54	0.58	0.38	0.20	0.08	0.15	0.00	0.06
Level 7	0.51	0.56	0.37	0.22	0.08	0.14	0.05	0.08
Level 6	0.54	0.23	0.38	0.69	0.08	0.07	0.00	0.01
Level 5	0.53	0.45	0.35	0.33	0.09	0.16	0.03	0.07
Level 4	0.57	0.19	0.34	0.74	0.09	0.06	0.00	0.00
Level 3	0.48	0.43	0.42	0.46	0.08	0.08	0.03	0.03
Level 2	0.64	0.37	0.32	0.59	0.04	0.00	0.00	0.03

Figure 24: Comparison of relative stiffness in N-S direction

Explanation of Error. Of course, there are endless possible errors and incorrect assumptions that could have been made in either analysis. Within E-Tabs, it is possible that there was an error in modeling floor diaphragms. Since a diaphragm is what allows lateral load to travel to load resisting elements (i.e., braced frames and moment frames), it is possible that this element was not modeled correctly and thus distributed load improperly. Or, there may be error within the calculations done by hand. The method of analysis, which required a 100 k load to be applied to each frame separately to measure deflection, is only an approximate method and therefore may not yield the most accurate results. Overall, however, the hand calculations were able to provide a reasonable prediction of how the lateral system would behave, and the computer model was able to both confirm and sharpen these observations to give a more accurate representation of structural behavior.

Shear Due to Torsion.

Eccentricities of the resultant shear forces, from wind and seismic loads, result in torsion acting on the building. Torsion from seismic loads is caused by the eccentricity of the center of mass with the center of rigidity. Torsion from wind forces is caused by the eccentricity of the geometric center of the building with the center of rigidity. These torsional moments can be resolved into shear forces acting on the braced frames and moment frames. The following equation is used to determine these resultant shear forces due to torsion in each frame in each direction,

$$F_i = VeR_iC / \sum RC^2$$

where V is the base shear acting on the building in that direction, Ri is the relative stiffness of the frame, and C is the perpendicular distance from the frame to the center of rigidity or geometric center. This equation was applied to find torsion resulting from both wind and seismic loads. It cannot be assumed that wind controls as it does for direct shear because of different eccentricities of the CORs and COGs. The torsional shear effects are summarized below, with expanded tables located in the Appendix.

Torsional Shear (kips), Wind: North-South Direction (Y)

				Braced Frame 2	Braced Frame 4	Moment Frame D	Moment Frame B
Level	$V_{factored}$	COG, X	e_x	Torsional Shear	Torsional Shear	Torsional Shear	Torsional Shear
Roof	190.7	1463.1	163.53	14.35	225.43	4.40	0.842
11	272.5	1645.0	110.83	34.70	78.77	12.94	1.951
10	162.6	1645.1	515.44	122.57	191.15	23.47	5.413
9	160.3	1645.1	2.18	0.50	0.79	0.10	0.023
8	157.6	1645.2	513.22	122.55	175.47	19.98	5.225
7	154.9	1645.2	2.57	0.58	0.95	0.09	0.034
6	151.9	1645.3	6.47	0.59	7.35	0.11	0.011
5	148.2	1645.4	0.21	0.04	0.11	0.01	0.002
4	144	1534.4	113.90	6.31	653.92	1.98	0.000
3	139	1605.3	571.88	80.86	699.48	11.21	2.462
2	131.8	1163.3	303.80	17.13	60.63	0.00	0.852

Figure 25

Torsional Shear (kips), Wind: East-West Direction (X)

				Braced Frame 1	Braced Frame 3	Moment Frame A	Moment Frame C
Level	$V_{factored}$	COG, Y	e_y	Torsional Shear	Torsional Shear	Torsional Shear	Torsional Shear
Roof	167.1	1193.6	123.35	31.06	34.60	5.20	3.799
11	239	1132.5	88.76	19.49	13.27	3.45	4.308
10	143.1	1132.6	2.92	0.60	0.53	0.00	0.000
9	141.2	1132.6	0.09	0.01	0.01	0.00	0.002
8	139.2	1132.6	0.66	0.12	0.12	0.00	0.000
7	137.1	1132.7	0.15	0.02	0.02	0.00	0.003
6	134.7	1132.7	1.07	0.19	0.20	0.00	0.000
5	131.9	1132.8	0.34	0.05	0.04	0.00	0.006
4	128.7	1224.5	90.58	25.85	11.03	0.00	0.000
3	124.8	1144.0	110.58	17.52	12.28	1.30	1.206
2	119.2	945.3	249.16	19.66	41.63	3.82	4.398

Figure 26

Torsional Shear (kips), Seismic: North-South Direction (Y)

				Braced Frame 2	Braced Frame 4	Moment Frame D	Moment Frame B
Level	$V_{factored}$	COR, X	e_x	Torsional Shear	Torsional Shear	Torsional Shear	Torsional Shear
Roof	365.8	1722.2	422.68	131.48	593.86	13.56	5.446
11	270.7	1749.6	215.46	92.20	71.18	21.31	4.245
10	122.8	1759.5	108.04	27.67	13.49	3.13	0.977
9	101.8	1760.2	112.89	23.60	11.65	2.71	0.847
8	82.3	1774.3	121.38	22.84	9.03	2.04	0.749
7	64.4	1794.9	147.06	22.75	8.61	1.75	0.972
6	48.2	1809.9	158.12	8.04	20.47	0.69	0.100
5	33.8	1821.2	176.06	12.95	7.32	1.22	0.553
4	18.9	1826.2	177.89	3.17	9.11	0.26	0.000
3	10.6	1829.4	122.18	2.80	2.16	0.13	0.052
2	4.1	1812.5	345.35	2.43	3.22	0.00	0.056

Figure 27

Torsional Shear (kips), Seismic: East-West Direction (X)

				Braced Frame 1	Braced Frame 3	Moment Frame A	Moment Frame C
Level	$V_{factored}$	COR, Y	e_y	Torsional Shear	Torsional Shear	Torsional Shear	Torsional Shear
Roof	365.8	1177.1	106.86	55.46	67.97	9.73	7.306
11	270.7	1112.4	108.85	25.51	19.37	4.73	6.092
10	122.8	1106.7	22.90	3.75	3.82	0.00	0.000
9	101.8	1100.2	32.31	2.84	3.18	0.37	0.508
8	82.3	1093.4	38.62	3.82	4.73	0.00	0.000
7	64.4	1092.8	40.06	2.18	2.75	0.29	0.331
6	48.2	1090.8	40.84	2.30	3.00	0.00	0.000
5	33.8	1092.8	70.34	2.47	2.21	0.23	0.305
4	18.9	1097.3	36.61	0.99	0.87	0.00	0.000
3	10.6	1113.3	79.92	0.98	0.81	0.08	0.076
2	4.1	1135.7	58.76	0.25	0.18	0.04	0.030

Figure 28

Conclusions. Torsional shear values are reasonably small for the majority of frames. However, in the N-S direction, there are a few unusually high torsional shear values, in both the wind and seismic load cases (highlighted in yellow). These large torsional shears are due to higher eccentricities between the COR/COM and COG/COM. Despite these higher eccentricities, the torsional shear still should probably not be that high at these locations. This issue will need to be investigated further and checked for error in calculation before any conclusions can be made.

Total Building Drift.

Total building drift was taken as the maximum deflection at the top of the lateral force resisting frames in each direction, as calculated by the E-Tabs analysis. These deflections were compared to an industry standard drift limitation of H/400. Since drift is a serviceability check, no load factors need to be applied to lateral loads. Thus, wind still controls over seismic in the North-South direction, but seismic now controls over wind in the East-West direction. Because of this, a seismic load case was added to the E-Tabs model, and new output was generated to calculate building drift. Total deflections, recorded in the table below, are less than the standard H/400 (where H = 184'or 2208") and are therefore acceptable.

H/400 (in)	Δ_{top} E-W (in)		Δ_{top} N-S (in)	
	Seismic	Wind	Seismic	Wind
5.52	1.84	1.06	2.4	3.23

Figure 29: Building Drifts, E-W and N-S

Interstory Drift.

Interstory drift was also calculated by E-Tabs analysis. Drift between stories was checked for both wind and seismic load cases, since wind controls in the North-South direction (for serviceability checks only, no load factors applied) and seismic controls in the East-West direction. These calculated drifts were compared to ASCE 7-05 standards for wind interstory drift (h/400 to h/600) and seismic interstory drift (0.015h), where h is the story height. Total interstory displacements, recorded in the table below, are significantly less than the allowable limits for both loading cases.

Inter-Story Drift, Wind			
Story	Allowable Drift h/600 (in)	Actual Drift X (East-West)	Actual Drift Y (North-South)
ROOF	0.68	0.000487	0.00102
STORY 11-M	0.4	0.000441	0.00122
STORY 11	0.3	0.000426	0.00160
STORY 10	0.3	0.00049	0.00191
STORY 9	0.3	0.000524	0.00206
STORY 8	0.3	0.000544	0.00220
STORY 7	0.3	0.000567	0.00216
STORY 6	0.3	0.000576	0.00156
STORY 5	0.3	0.000695	0.00163
STORY 4	0.3	0.000711	0.00160
STORY 3	0.3	0.000708	0.00150
STORY 2	0.3	0.000385	0.00112

Figure 30: Interstory Drift, Wind

Inter-Story Drift, Seismic			
Story	Allowable Drift 0.015h (in)	Actual Drift X (East-West)	Actual Drift Y (North-South)
ROOF	6.12	0.00114	0.00088
STORY 11-M	3.6	0.000981	0.00104
STORY 11	2.7	0.000904	0.00139
STORY 10	2.7	0.000998	0.00159
STORY 9	2.7	0.001007	0.00163
STORY 8	2.7	0.000994	0.00163
STORY 7	2.7	0.000965	0.00150
STORY 6	2.7	0.000922	0.000944
STORY 5	2.7	0.000967	0.000926
STORY 4	2.7	0.000975	0.000871
STORY 3	2.7	0.000897	0.000759
STORY 2	2.7	0.000452	0.000565

Figure 31: Interstory Drift, Seismic

Spot Checks.

To verify the capacity of lateral framing elements, spot checks were performed on typical load resisting elements. First, a strength check was performed on a typical diagonal brace (a double-tee shape). Next, a strength check of a typical beam and column within a moment frame was carried out. The procedure followed is outlined below.

Diagonal Brace Check. To check a typical diagonal brace for strength in axial compression, a frame was chosen on level 7 (a typical lab floor), shown below. Table 4-7 of the AISC Steel Construction Manual was consulted to give allowable axial compression for the WT shape. Since the WT braces are doubled (two members are installed) in the existing design, each WT shape must be able to carry half of the axial load seen in the brace modeled in E-Tabs.

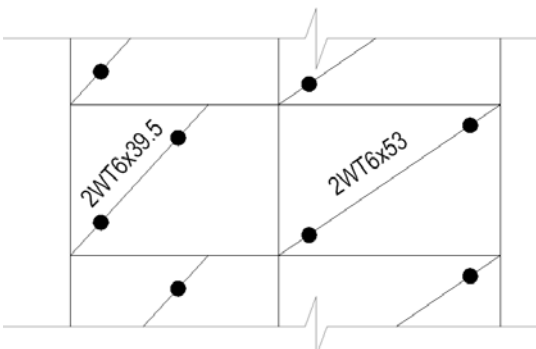


Figure 32: Level 7, Braced Frame 1

Member	Length	Axial Load, Pu	Pu in single WT	Allowable, ΦPn
2WT6x39.5	20'-6"	57.02 k	28.51 k	74.8 k, for a WT6x29
2WT6x53	25'-10"	99.3 k	49.65 k	43.4 k, for a WT6x25

Figure 33: Axial Load Check

Unfortunately, the specific WT shapes used as diagonal braces are not listed in Table 4-7 to give allowable axial compression. To work around this, the strength of the next smallest shape (corresponding to the correct length) was chosen and compared to the actual load. The 2WT6x39.5 is clearly capable of taking the axial load it is under, as a smaller section actually exceeds the required capacity. The 2WT6x53 was compared to the only shape listed in Table 4-7 for a length greater than 25'-10," and the available strength of this member is slightly under the required capacity of the brace. However, it is reasonable to assume that the brace will, in fact, be able to carry its axial load since it is a much larger shape than that it was compared to.

Moment Frame Check.

To check a typical moment frame for strength in combined bending and axial loads, a frame was chosen on level 7 (a typical lab floor), shown below.

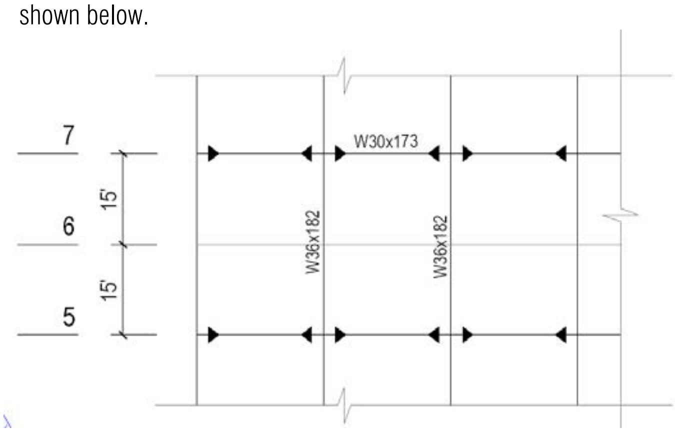


Figure 34: Level 7, Moment Frame A

For a W36x182 column with KL = 30', Table 6-1 gives:

$p = 1.65 \times 10^3$, $bx = 0.642 \times 10^3$

$p^*Pu + bx^*Mux = (1.65E-3)^*(335.2\ k) + (0.642E-3)^*(145.7\ ft\text{-}k) = 0.65 < 1$ …… OK

(Pu was previously calculated in Technical Assignment 1, and Mux was given by E-Tabs analysis)

Conclusions. Although only a representative example from each frame type was chosen for analysis, the members in these frames passed strength checks with no problem, indicating that the majority of lateral framing elements of similar size and loading would behave in the same way.

Conclusion

After conducting a lateral analysis of the Center for Science & Medicine, a better understanding of lateral load distribution has been gained and a general knowledge of how resisting structural elements work together has been established. When lateral loads are applied in the form of seismic or wind forces, shears at each story are resisted by braced frames at the core and moment frames at the perimeter. The floor diaphragm allows the loads to travel through the structure and into these lateral resisting elements. It has been concluded that each moment frame carries approximately 15%-20% of the total lateral load in each direction, and each braced frames carries the remainder of the lateral load not resisted by moment frames (which is the majority of the load). Interestingly, since each moment frame does not carry over 25% of the lateral load, the system cannot be considered a "dual system," as defined in ASCE 7-05.

In general, torsional shear does not seem to be an issue of concern for the system. A further investigation of torsional shear may need to be carried out for specific frames on specific levels, but this will be done later if found necessary. Deflection between stories and the total building drift is satisfactory according to industry standard, and satisfactory spot checks performed on lateral elements can attest to the integrity of the structure's design.

Appendix

Appendix A: Lateral Loads

Seismic.

Typical Calculations of Floor Weight:

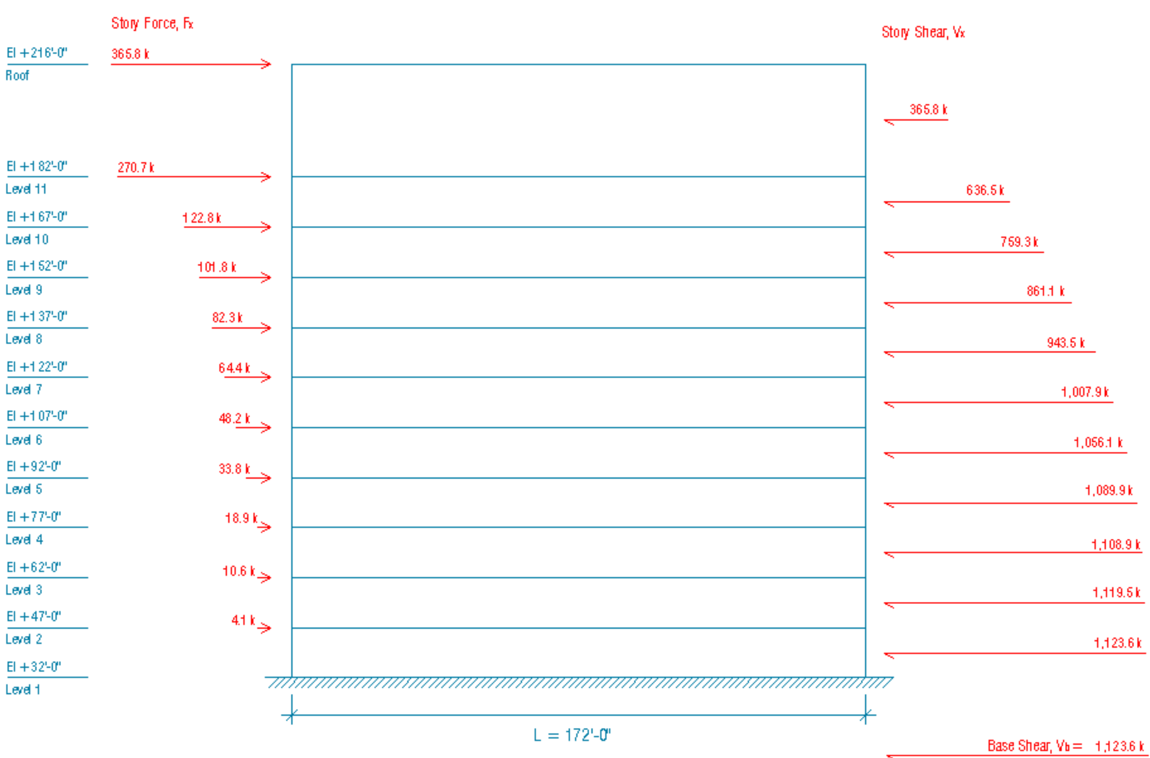
Floor 5					
Approx Area:	28,487	ft^2	Floor to Floor Height:	15 ft	
Slab:					
thickness =	4.75	in			
unit weight =	150	pcf			
total weight =	1,691.4	kips			
Columns:					
Shape	Quantity	Unit Weight (lb/ft)	Column Height (ft)	Total Weight	
W14x61	9	61	15	8.2	kips
W14x68	1	68	15	1.0	kips
W14x90	6	90	15	8.1	kips
W14x74	3	74	15	3.3	kips
W14x109	1	109	15	1.6	kips
W14x120	4	120	15	7.2	kips
W14x145	1	145	15	2.2	kips
W14x176	1	176	15	2.6	kips
W14x211	10	211	15	31.7	kips
W24x117	9	117	15	15.8	kips
W24x146	7	146	15	15.3	kips
W36x135	4	135	15	8.1	kips
W36x150	5	150	15	11.3	kips
total weight =	116.5	kips			
Beams, Connections, Bracing, etc:					
allowance =	11.0	psf			
total weight =	313.4	kips			
TOTAL FLOOR WEIGHT:			2,121.2 kips	or	74 psf

Floor 11					
Approx Area:	28,488	ft^2	Floor to Floot Height:	34 ft	
(Mezzanine additional 5,123 ft^2)					
Slab (Flr 11):					
thickness =	8	in			
unit weight =	150	pcf			
total weight =	2,848.8	kips			
Slab (Mezz):					
thickness =	8	in			
unit weight =	150	pcf			
total weight =	512.3	kips			
Columns:					
Shape	Quantity	Unit Weight (lb/ft)	Column Height (ft)	Total Weight	
W14x61	18	61	34	37.3	kips
W14x82	1	82	34	2.8	kips
W14x120	5	120	34	20.4	kips
W14x145	1	145	34	4.9	kips
W14x176	1	176	34	6.0	kips
W14x211	10	211	34	71.7	kips
W24x117	2	117	34	8.0	kips
W24x146	6	146	34	29.8	kips
W36x135	4	135	34	18.4	kips
W36x150	5	150	34	25.5	kips
total weight 11-M =					
	35.448	kips			
total weight 11 =					
	189.3	kips			
Beams, Connections, Bracing, etc:					
allowance =	11.0	psf			
total weight =	369.7	kips			
TOTAL FLOOR 11 WEIGHT:			3,407.8 kips	or	120 psf
TOTAL FLOOR 11-M WEIGHT:			547.7 kips	or	107 psf

Applied Seismic Forces:
(used in drift analysis and torsional shear analysis only)

Floor	w_x (k)	h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
1							1,123.6	
2	2,328.5	15.0	74.1	172,610	0.004	4.1	1,119.5	61.4
3	2,003.0	30.0	223.2	446,987	0.009	10.6	1,108.9	318.2
4	1,875.7	45.0	425.2	797,590	0.017	18.9	1,089.9	851.8
5	2,121.2	60.0	671.8	1,425,111	0.030	33.8	1,056.1	2,029.3
6	2,121.2	75.0	958.0	2,032,056	0.043	48.2	1,007.9	3,617.0
7	2,121.2	90.0	1,280.1	2,715,400	0.057	64.4	943.5	5,800.0
8	2,121.2	105.0	1,635.7	3,469,599	0.073	82.3	861.1	8,646.1
9	2,121.2	120.0	2,022.5	4,290,288	0.091	101.8	759.3	12,218.5
10	2,121.2	135.0	2,439.1	5,173,911	0.109	122.8	636.5	16,576.9
11	3,955.6	150.0	2,883.9	11,407,669	0.241	270.7	365.8	40,610.6
Roof	3,861.8	184.0	3,990.8	15,411,530	0.326	365.8		67,300.0
		$\sum w_i h_i^k =$	47,342,753	$\sum F_x = V =$	1,123.6		$\sum M =$	158,030.1

Loading Diagram:



Wind.

Wind Pressures on North-South Frame
B = 172 ft, L = 200 ft

Floor	Height (ft)	hx	Kz	qz	Pressures (psf)			Force (kips)	Factored Force (x1.6)	Shear (kips)	Factored Shear	Moment (ft-k)	Factored Moment (x1.6)
					N/S windward	N/S leeward	Total						
Roof	34	184	1.18	29.53	19.13 ± 5.32 = 24.4	-11.00 ± 5.32 = -16.3	40.8	119.2	190.7	119.2	190.7	4,052.9	6,484.6
11	15	150	1.11	27.78	18.00 ± 5.32 = 23.3	-11.00 ± 5.32 = -16.3	39.6	170.3	272.5	289.5	463.2	2,554.9	4,087.8
10	15	135	1.08	27.03	17.51 ± 5.32 = 22.8	-11.00 ± 5.32 = -16.3	39.1	101.6	162.6	391.2	625.8	1,524.3	2,438.9
9	15	120	1.04	26.02	16.86 ± 5.32 = 22.2	-11.00 ± 5.32 = -16.3	38.5	100.2	160.3	491.3	786.1	1,502.4	2,403.8
8	15	105	1.00	25.02	16.22 ± 5.32 = 21.5	-11.00 ± 5.32 = -16.3	37.8	98.5	157.6	589.8	943.7	1,477.3	2,363.6
7	15	90	0.96	24.02	15.57 ± 5.32 = 20.9	-11.00 ± 5.32 = -16.3	37.2	96.8	154.9	686.6	1098.6	1,452.2	2,323.5
6	15	75	0.91	22.77	14.76 ± 5.32 = 20.1	-11.00 ± 5.32 = -16.3	36.4	94.9	151.9	781.5	1250.5	1,423.9	2,278.3
5	15	60	0.85	21.27	13.78 ± 5.32 = 19.1	-11.00 ± 5.32 = -16.3	35.4	92.6	148.2	874.2	1398.7	1,389.4	2,223.1
4	15	45	0.785	19.64	12.73 ± 5.32 = 18.0	-11.00 ± 5.32 = -16.3	34.4	90.0	144.0	964.2	1542.7	1,350.2	2,160.3
3	15	30	0.70	17.52	11.35 ± 5.32 = 16.7	-11.00 ± 5.32 = -16.3	33.0	86.9	139.0	1051.0	1681.7	1,303.1	2,085.0
2	15	15	0.57	14.26	9.24 ± 5.32 = 14.6	-11.00 ± 5.32 = -16.3	30.9	82.4	131.8	1133.4	1813.5	1,235.7	1,977.1
1	0	0						39.8	63.7	1173.3	1877.2	0.0	0.0
							Base Shear =	1,173.3	1,877.2		M =	19,266.2	30,826.0

Wind Pressures on East-West Frame
B = 200 ft, L = 172 ft

Floor	Height (ft)	hx	Kz	qz	Pressures (psf)			Force (kips)	Factored Force (x1.6)	Shear (kips)	Factored Shear	Moment (ft-k)	Factored Moment (x1.6)
					E/W windward	E/W leeward	Total						
Roof	34	184	1.18	29.53	12.76 ± 5.32 = 18.1	-7.33 ± 5.32 = -12.6	30.7	104.5	167.1	104.5	167.1	3,551.4	5,682.2
11	15	150	1.11	27.78	12.00 ± 5.32 = 17.3	-7.33 ± 5.32 = -12.6	30.0	149.4	239.0	253.9	406.2	2,241.0	3,585.6
10	15	135	1.08	27.03	11.68 ± 5.32 = 17.0	-7.33 ± 5.32 = -12.6	29.6	89.4	143.1	343.3	549.2	1,341.1	2,145.8
9	15	120	1.04	26.02	11.24 ± 5.32 = 16.6	-7.33 ± 5.32 = -12.6	29.2	88.3	141.2	431.5	690.4	1,324.1	2,118.5
8	15	105	1.00	25.02	10.81 ± 5.32 = 16.1	-7.33 ± 5.32 = -12.6	28.8	87.0	139.2	518.5	829.6	1,304.6	2,087.4
7	15	90	0.96	24.02	10.38 ± 5.32 = 15.7	-7.33 ± 5.32 = -12.6	28.3	85.7	137.1	604.2	966.7	1,285.2	2,056.3
6	15	75	0.91	22.77	9.84 ± 5.32 = 15.2	-7.33 ± 5.32 = -12.6	27.8	84.2	134.7	688.4	1101.4	1,263.3	2,021.2
5	15	60	0.85	21.27	9.19 ± 5.32 = 14.5	-7.33 ± 5.32 = -12.6	27.2	82.4	131.9	770.8	1233.3	1,236.5	1,978.4
4	15	45	0.785	19.64	8.49 ± 5.32 = 13.8	-7.33 ± 5.32 = -12.6	26.5	80.4	128.7	851.2	1362.0	1,206.1	1,929.8
3	15	30	0.70	17.52	7.57 ± 5.32 = 12.9	-7.33 ± 5.32 = -12.6	25.5	78.0	124.8	929.2	1486.7	1,169.6	1,871.4
2	15	15	0.57	14.26	6.16 ± 5.32 = 11.5	-7.33 ± 5.32 = -12.6	24.1	74.5	119.2	1003.7	1605.9	1,117.3	1,787.7
1	0							36.2	57.9	1039.9	1663.8	0.0	0.0
							Base Shear =	1,039.9	1,663.8		M =	17,040.2	27,264.3

Appendix B: Relative Stiffness

Hand Calculations: Relative stiffness based on deflection analysis

EAST-WEST DIRECTION (X)

		BRACED FRAME 1:			BRACED FRAME 3:			MOMENT FRAME A:			MOMENT FRAME C:			
Level	P (kips)	Δ (in)	K_{LE}	$K_{LE}/\sum K$	Δ (in)	K_{LH}	$K_{LH}/\sum K$	Δ (in)	K_{MFS}	$K_{MFS}/\sum K$	Δ (in)	K_{MFN}	$K_{MFN}/\sum K$	$\sum K$
Roof	100	0.0189	5291.0	0.52	0.0223	4484.3	0.44	0.5559	179.9	0.02	0.4984	200.6	0.02	10155.8
Level 11-M	100	0.0395	2531.6	0.53	0.0454	2202.6	0.47	-	-	-	-	-	-	4734.3
Level 11	100	0.0315	3174.6	0.48	0.038	2631.6	0.40	0.301	332.2	0.05	0.2445	409.0	0.04	6547.4
Level 10	100	0.033	3030.3	0.55	0.0405	2469.1	0.45	-	0.0	0.00	-	0.0	0.00	5499.4
Level 9	100	0.033	3030.3	0.48	0.0405	2469.1	0.39	0.25	400.0	0.06	0.2169	461.0	0.04	6360.5
Level 8	100	0.033	3030.3	0.55	0.0405	2469.1	0.45	-	0.0	0.00	-	0.0	0.00	5499.4
Level 7	100	0.033	3030.3	0.47	0.0405	2469.1	0.39	0.2432	411.2	0.06	0.2027	493.3	0.05	6404.0
Level 6	100	0.033	3030.3	0.55	0.0405	2469.1	0.45	-	0.0	0.00	-	0.0	0.00	5499.4
Level 5	100	0.024	4166.7	0.44	0.024	4166.7	0.44	0.1861	537.3	0.06	0.2015	496.3	0.03	9367.0
Level 4	100	0.024	4166.7	0.50	0.024	4166.7	0.50	-	0.0	0.00	-	0.0	0.00	8333.3
Level 3	100	0.024	4166.7	0.44	0.024	4166.7	0.44	0.1821	549.1	0.06	0.152	657.9	0.04	9540.4
Level 2	100	0.024	4166.7	0.50	0.024	4166.7	0.50	-	0.0	0.00	-	0.0	0.00	8333.3

NORTH-SOUTH DIRECTION (Y)

		BRACED FRAME 2:			BRACED FRAME 4:			MOMENT FRAME D:			MOMENT FRAME B:			
Level	P (kips)	Δ (in)	K_{L9}	$K_{L9}/\sum K$	Δ (in)	$K_{L6.9}$	$K_{L6.9}/\sum K$	Δ (in)	K_{MFW}	$K_{MFW}/\sum K$	Δ (in)	K_{MFE}	$K_{MFE}/\sum K$	$\sum K$
Roof	100	0.0147	6802.7	0.51	0.0158	6329.1	0.48	2.0329	49.2	0.004	1.866	53.6	0.00	13234.6
Level 11-M	100	0.0384	2604.2	0.50	0.0385	2597.4	0.50	-	-	-	-	-	-	5201.6
Level 11	100	0.0191	5235.6	0.51	0.0256	3906.3	0.38	0.1469	680.7	0.07	0.2899	344.9	0.03	10167.5
Level 10	100	0.0169	5917.2	0.60	0.0306	3268.0	0.33	0.1604	623.4	0.06	-	0.0	0.00	9808.6
Level 9	100	0.0169	5917.2	0.53	0.0258	3876.0	0.35	0.1242	805.2	0.07	0.2045	489.0	0.04	11087.3
Level 8	100	0.0185	5405.4	0.54	0.0259	3861.0	0.38	0.1242	805.2	0.08	-	0.0	0.00	10071.6
Level 7	100	0.0185	5405.4	0.51	0.0259	3861.0	0.37	0.1242	805.2	0.08	0.2027	493.3	0.05	10564.9
Level 6	100	0.0185	5405.4	0.54	0.0259	3861.0	0.38	0.1242	805.2	0.08	-	0.0	0.00	10071.6
Level 5	100	0.0130	7692.3	0.53	0.020	5000.0	0.35	0.0759	1317.5	0.09	0.2596	385.2	0.03	14395.0
Level 4	100	0.0118	8474.6	0.57	0.020	5000.0	0.34	0.0759	1317.5	0.09	-	0.0	0.00	14792.1
Level 3	100	0.0118	8474.6	0.48	0.0135	7407.4	0.42	0.0747	1338.7	0.08	0.2079	481.0	0.03	17701.7
Level 2	100	0.0118	8474.6	0.64	0.0236	4237.3	0.32	0.1743	573.7	0.04	-	0.0	0.00	13285.6

SUMMARY: EAST-WEST DIRECTION

Level	BRACED FRAME 1 Relative Stiffness	BRACED FRAME 3 Relative Stiffness	MOMENT FRAME A: Relative Stiffness	MOMENT FRAME C: Relative Stiffness
Roof	0.52	0.44	0.018	0.02
Level 11 Mezz	0.53	0.47	-	-
Level 11	0.48	0.40	0.05	0.04
Level 10	0.55	0.45	0.00	0.00
Level 9	0.48	0.39	0.06	0.04
Level 8	0.55	0.45	0.00	0.00
Level 7	0.47	0.39	0.06	0.05
Level 6	0.55	0.45	0.00	0.00
Level 5	0.44	0.44	0.06	0.03
Level 4	0.50	0.50	0.00	0.00
Level 3	0.44	0.44	0.06	0.04
Level 2	0.50	0.50	0.00	0.00

SUMMARY: NORTH-SOUTH DIRECTION

Level	BRACED FRAME 2 Relative Stiffness	BRACED FRAME 4 Relative Stiffness	MOMENT FRAME D: Relative Stiffness	MOMENT FRAME B: Relative Stiffness
Roof	0.51	0.48	0.004	0.00
Level 11 Mezz	0.50	0.50	-	-
Level 11	0.51	0.38	0.07	0.03
Level 10	0.60	0.33	0.06	0.00
Level 9	0.53	0.35	0.07	0.04
Level 8	0.54	0.38	0.08	0.00
Level 7	0.51	0.37	0.08	0.05
Level 6	0.54	0.38	0.08	0.00
Level 5	0.53	0.35	0.09	0.03
Level 4	0.57	0.34	0.09	0.00
Level 3	0.48	0.42	0.08	0.03
Level 2	0.64	0.32	0.04	0.00

Appendix C: Distribution of Direct Shear

NORTH-SOUTH DIRECTION (Y)									
		BRACED FRAME 1:		BRACED FRAME 3:		MOMENT FRAME A:		MOMENT FRAME C:	
Level	∑V	V (kips)	V/∑V	V (kips)	V/∑V	V (kips)	V/∑V	V (kips)	V/∑V
Roof	363.15	94.34	0.26	235.6	0.65	22.61	0.06	10.6	0.03
Level 11 Mezz	147.97	0.07	0.00	147.9	1.00	0	0.00	0	0.00
Level 11	450.75	197.8	0.44	107.2	0.24	117.4	0.26	28.35	0.06
Level 10	622.91	349.8	0.56	128.9	0.21	106.75	0.17	37.46	0.06
Level 9	789.9	435.4	0.55	168.9	0.21	135.7	0.17	49.9	0.06
Level 8	948.35	553.4	0.58	191.05	0.20	143	0.15	60.9	0.06
Level 7	1072.1	604.1	0.56	234.4	0.22	151.8	0.14	81.8	0.08
Level 6	1296.09	292.2	0.23	899.2	0.69	89.7	0.07	14.99	0.01
Level 5	1359.4	605.7	0.45	452.4	0.33	210.8	0.16	90.5	0.07
Level 4	1600.56	310.9	0.19	1184.6	0.74	98.3	0.06	6.76	0.00
Level 3	1669.59	726	0.43	765.69	0.46	128.5	0.08	49.4	0.03
Level 2	1806.9	668.4	0.37	1074.7	0.59	2.8	0.00	61	0.03

EAST-WEST DIRECTION (X)									
		BRACED FRAME 1:		BRACED FRAME 3:		MOMENT FRAME A:		MOMENT FRAME C:	
Level	∑V	V (kips)	V/∑V	V (kips)	V/∑V	V (kips)	V/∑V	V (kips)	V/∑V
Roof	97.7	38.99	0.40	7.43	0.08	30.24	0.31	21.08	0.22
Level 11-M	39.2	35.45	0.90	3.76	0.10	0	0.00	0	0.00
Level 11	204.6	61.64	0.30	52.55	0.26	42.73	0.21	47.65	0.23
Level 10	190.9	89.22	0.47	101.66	0.53	0	0.00	0	0.00
Level 9	349.8	108.6	0.31	130.84	0.37	51.71	0.15	58.69	0.17
Level 8	286.2	126.4	0.44	159.78	0.56	0	0.00	0	0.00
Level 7	487.0	151.17	0.31	196.44	0.40	71.41	0.15	68.02	0.14
Level 6	415.0	177.27	0.43	237.73	0.57	0	0.00	0	0.00
Level 5	618.3	236.34	0.38	217.28	0.35	79.6	0.13	85.03	0.14
Level 4	528.1	272.82	0.52	255.25	0.48	0	0.00	0	0.00
Level 3	714.4	285.88	0.40	268.61	0.38	85.55	0.12	74.34	0.10
Level 2	777.4	265.34	0.34	251.37	0.32	148.5	0.19	112.14	0.14

Appendix D: Torsional Shear

				Braced Frame 2			Braced Frame 4			Moment Frame D			Moment Frame B		
Level	$V_{factored}$	COG, X	e_x	Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear
Roof	190.7	1463.1	163.53	0.26	564.92	14.35	0.65	89.92	225.43	0.060	425.08	4.40	0.03	1110.9	0.842
11	272.5	1645.0	110.83	0.44	382.98	34.70	0.24	92.02	78.77	0.26	607.02	12.94	0.06	929.0	1.951
10	162.6	1645.1	515.44	0.56	382.92	122.57	0.21	92.08	191.15	0.17	607.08	23.47	0.06	928.9	5.413
9	160.3	1645.1	2.18	0.55	382.87	0.50	0.21	92.13	0.79	0.17	607.13	0.10	0.06	928.9	0.023
8	157.6	1645.2	513.22	0.58	382.81	122.55	0.20	92.19	175.47	0.15	607.19	19.98	0.06	928.8	5.225
7	154.9	1645.2	2.57	0.56	382.75	0.58	0.22	92.25	0.95	0.14	607.25	0.09	0.08	928.8	0.034
6	151.9	1645.3	6.47	0.23	382.69	0.59	0.69	92.31	7.35	0.07	607.31	0.11	0.01	928.7	0.011
5	148.2	1645.4	0.21	0.45	382.64	0.04	0.33	92.36	0.11	0.16	607.36	0.01	0.07	928.6	0.002
4	144	1534.4	113.90	0.19	493.56	6.31	0.74	18.56	653.92	0.06	496.44	1.98	0.00	1039.6	0.000
3	139	1605.3	571.88	0.43	422.72	80.86	0.46	52.28	699.48	0.08	567.28	11.21	0.03	968.7	2.462
2	131.8	1163.3	303.80	0.37	864.67	17.13	0.59	389.67	60.63	0.00	125.33	0.00	0.03	1410.7	0.852

Braced Frame 1			Braced Frame 3			Moment Frame A			Moment Frame C		
Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear
0.40	265.43	31.06	0.80	476.57	34.60	0.310	1228.43	5.20	0.22	1193.6	3.799
0.30	326.49	19.49	0.26	415.51	13.27	0.21	1289.49	3.45	0.23	1132.5	4.308
0.47	326.45	0.60	0.53	415.55	0.53	0.00	1289.45	0.00	0.00	1132.6	0.000
0.31	326.41	0.01	0.37	415.59	0.01	0.15	1289.41	0.00	0.17	1132.6	0.002
0.44	326.37	0.12	0.56	415.63	0.12	0.00	1289.37	0.00	0.00	1132.6	0.000
0.31	326.33	0.02	0.40	415.67	0.02	0.15	1289.33	0.00	0.14	1132.7	0.003
0.43	326.29	0.19	0.57	415.71	0.20	0.00	1289.29	0.00	0.00	1132.7	0.000
0.38	326.25	0.05	0.35	415.75	0.04	0.13	1289.25	0.00	0.14	1132.8	0.006
0.52	234.51	25.85	0.48	507.49	11.03	0.00	1197.51	0.00	0.00	1224.5	0.000
0.40	315.03	17.52	0.38	426.97	12.28	0.12	1278.03	1.30	0.10	1144.0	1.206
0.34	513.69	19.66	0.32	228.31	41.63	0.19	1476.69	3.82	0.14	945.3	4.398

Braced Frame 2			Braced Frame 4			Moment Frame D			Moment Frame B		
Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear
0.26	305.77	131.48	0.65	169.23	593.86	0.060	684.23	13.56	0.03	851.8	5.446
0.44	278.35	92.20	0.24	196.65	71.18	0.26	711.65	21.31	0.06	824.4	4.245
0.56	268.48	27.67	0.21	206.52	13.49	0.17	721.52	3.13	0.06	814.5	0.977
0.55	267.80	23.60	0.21	207.20	11.65	0.17	722.20	2.71	0.06	813.8	0.847
0.58	253.71	22.84	0.20	221.29	9.03	0.15	736.29	2.04	0.06	799.7	0.749
0.56	233.13	22.75	0.22	241.88	8.61	0.14	756.88	1.75	0.08	779.1	0.972
0.23	218.11	8.04	0.69	256.89	20.47	0.07	771.89	0.69	0.01	764.1	0.100
0.45	206.79	12.95	0.33	268.21	7.32	0.16	783.21	1.22	0.07	752.8	0.553
0.19	201.77	3.17	0.74	273.23	9.11	0.06	788.23	0.26	0.00	747.8	0.000
0.43	198.56	2.80	0.46	276.44	2.16	0.08	791.44	0.13	0.03	744.6	0.052
0.37	215.51	2.43	0.59	259.49	3.22	0.00	774.49	0.00	0.03	761.5	0.056

Braced Frame 1			Braced Frame 3			Moment Frame A			Moment Frame C		
Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear	Ri	C	Torsional Shear
0.40	281.91	55.46	0.80	460.09	67.97	0.310	1244.91	9.73	0.22	1177.1	7.306
0.30	346.58	25.51	0.26	395.42	19.37	0.21	1309.58	4.73	0.23	1112.4	6.092
0.47	352.27	3.75	0.53	389.73	3.82	0.00	1315.27	0.00	0.00	1106.7	0.000
0.31	358.81	2.84	0.37	383.19	3.18	0.15	1321.81	0.37	0.17	1100.2	0.508
0.44	365.65	3.82	0.56	376.36	4.73	0.00	1328.65	0.00	0.00	1093.4	0.000
0.31	366.24	2.18	0.40	375.76	2.75	0.15	1329.24	0.29	0.14	1092.8	0.331
0.43	368.20	2.30	0.57	373.80	3.00	0.00	1331.20	0.00	0.00	1090.8	0.000
0.38	366.25	2.47	0.35	375.75	2.21	0.13	1329.25	0.23	0.14	1092.8	0.305
0.52	361.70	0.99	0.48	380.30	0.87	0.00	1324.70	0.00	0.00	1097.3	0.000
0.40	345.69	0.98	0.38	396.31	0.81	0.12	1308.69	0.08	0.10	1113.3	0.076
0.34	323.29	0.25	0.32	418.71	0.18	0.19	1286.29	0.04	0.14	1135.7	0.030