Franklin Square Hospital Center Patient Tower

Baltimore, MD

Technical Report 3

Lateral System Analysis and Confirmation Design

Thomas Weaver

Structural Option

AE 481W Senior Thesis

Consultant: Professor M. Kevin Parfitt
12/7/2009

Table of Contents:

Executive Summary

The goal of this technical assignment was to perform an analysis and confirmation design study of the lateral system of the Franklin Square Hospital Center Patient Tower. Loads obtained from Technical Assignment 1 were used for this study. For this analysis a 2D frame analysis was conducted in SAP along with a full 3D building model in ETABS to determine lateral force distribution, controlling load combinations, member forces and lateral displacements.

The lateral system of the structure consists of a total of 26 moment frames, with 13 in each orthogonal direction. Figure 8 and 9 show the layout of theses frames and their naming convention. It was determined that Torsion of the structure needed investigation under lateral loading and was investigated in relation to effect on member forces. Ultimately torsion was determined to impact the design very little as the center of mass and center of rigidity are very close in each level. Lateral loads were calculated and applied according to ASCE 7-05 and, while unusual for East Coast buildings, seismic has controlled the design of the lateral system in both directions due to the large mass of the building when compared to a lighter steel structure that would likely have been controlled by wind.

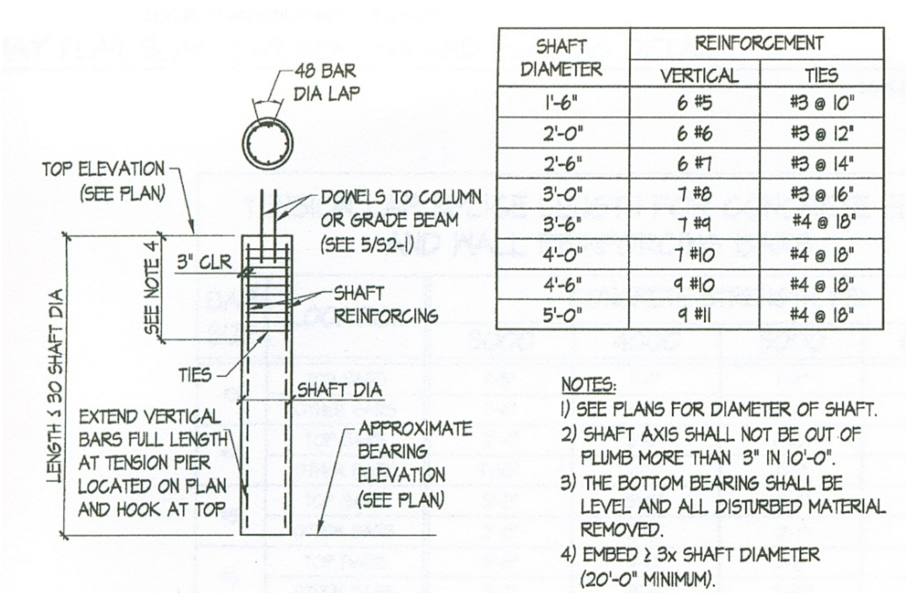
In addition, serviceability checks were completed and found to not control the design of the lateral system in any way as all drift values were well below code maximums or common allowable limits. The foundation reactions were analyzed for overturning and uplift and small uplift forces were found in some areas. These uplift forces are of no concern given the large weight of the caisson foundations which completely balance the uplift forces.

Along with serviceability and overturning spot checks, strength checks were conducted on a section of slab, perimeter edge beam, and column under the combination of dead, live and seismic loading. The slab and beam proved to be sufficient in two of the three critical moment sections while one the sections in each was overloaded. It had been concluded that the analysis method used in this report to determine dead and live load moments in the slabs and beams differs from the more advanced moment distribution method used by the designer. Had such a method been used, the slab and beam would have proven adequate. A column was checked on the ground level and level 4 and found to be adequate on the 4th floor for both seismic loading in both directions while the column on the ground floor was found to be adequate in seismic loading from only one direction. This is due to the differences in seismic load applied to the structure in this report from the designer as the roof was included as a level in load calculation while the designer most likely omitted this level.

Structural Systems

Foundation System

The foundation system of the Franklin Square Hospital Patient Tower consists of drilled piers or caissons 4 feet in diameter and centered under columns or slightly offset under perimeter grade beams. The piers range in size from 1.5 feet in diameter to 5 feet in diameter. They are embedded a minimum of 20 feet into bedrock. The total typical depth of the piers is around 42 feet below grade pending geotechnical engineer inspection. See Figure 1, "Drilled Pier Reinforcing."



SHAFT DIAMETER	REINFORCEMENT	
	VERTICAL	TIES
1'-6"	6 #5	#3 @ 10"
2'-0"	6 #6	#3 @ 12"
2'-6"	6 #7	#3 @ 14"
3'-0"	7 #8	#3 @ 16"
3'-6"	7 #9	#4 @ 18"
4'-0"	7 #10	#4 @ 18"
4'-6"	9 #10	#4 @ 18"
5'-0"	9 #11	#4 @ 18"

Figure 1: Drilled Pier Reinforcing

The piers are required to be a normal weight concrete with a concrete compressive strength (f'_c) of 3000 psi. As previously mention, the piers directly support interior columns. See Figure 2, "Column Caisson Connection and Column Reinforcing."

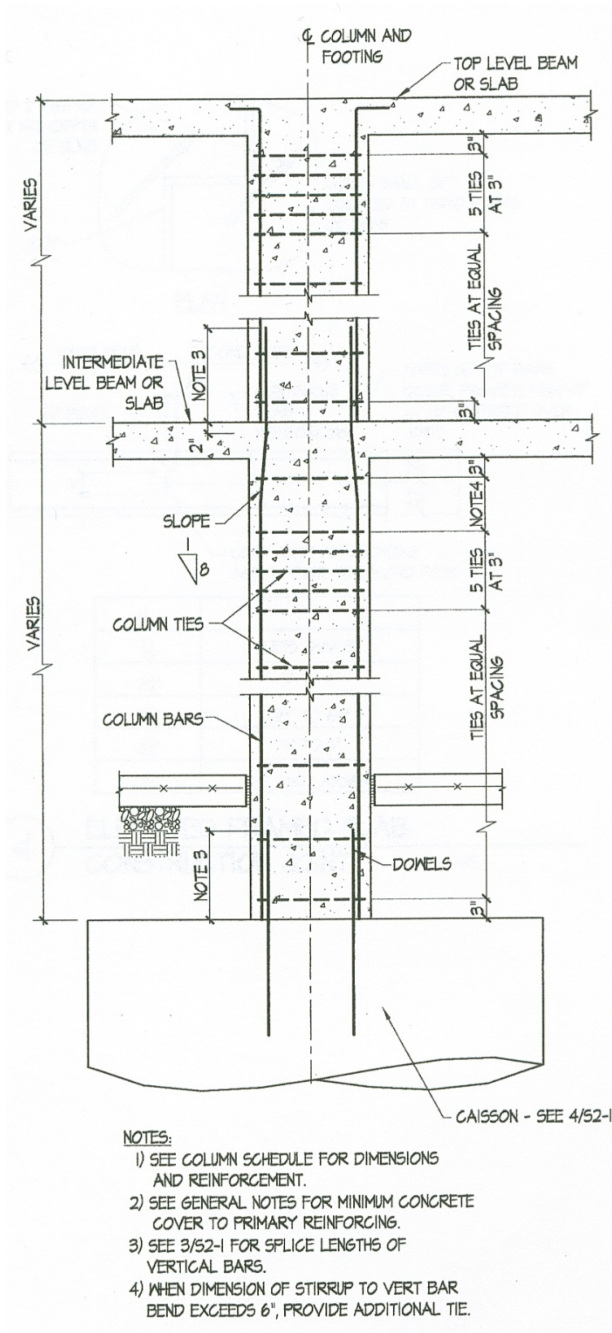


Figure 2: Typical Column Caisson Connection and Column Reinforcing

The piers also directly support perimeter grade beams. The typical grade beam is 24"x24" with some that are 36"x24". See Figure 3, "Typical Grade Beam Caisson Connection."

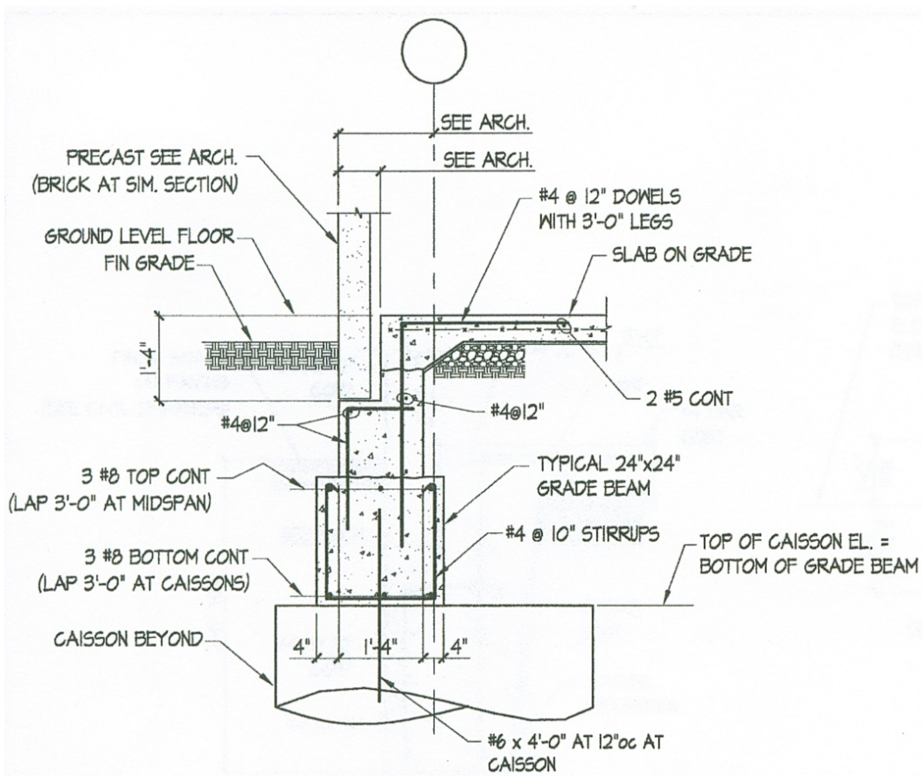


Figure 3: Typical Grade Beam Caisson Connection

While there are no sub grade levels in the structure, the west side of the ground floor can be considered below grade because the ground has been filled to provide on grade access to the first floor lobby. The existing hospital ground floor also resides on the level corresponding to the patient tower's first floor. Lateral soil pressures from the foundation of the existing building are resisted by a 16" thick foundation wall in these areas. See Figure 4, "Typical Foundation Wall Section."

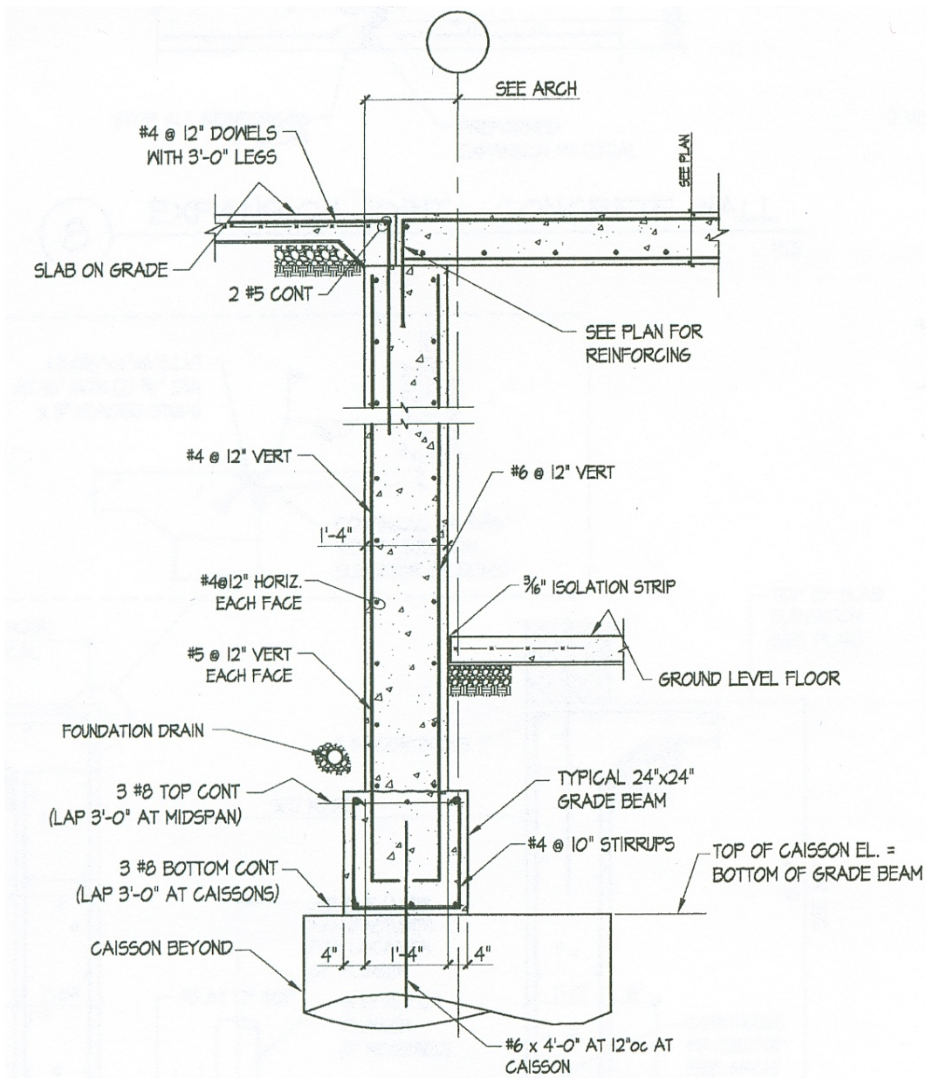


Figure 4: Typical Foundation Wall Section

The rest of the foundation consists of a 5 inch ground floor slab on grade of compressive strength equal to 3000 psi. The slab on grade is reinforced with 6x6-W2.9xW2.9 welded wire fabric over a 4 inch layer of clean, well-graded gravel or crushed stone.

Floor System

The buildings typical floor system is a 10” reinforced two way slab, or flat plate, spanning a typical 30’x30’ bay. The reinforcing varies a great deal depending on location and span but for the most part there is a continuous bottom mat of #5 or #6 bars at 12” each way with continuous top reinforcing within the column strips with mostly #6 or #8 bars. See Appendix A for Floor Plans and Figure 5, “Slab Reinforcing Detail.”

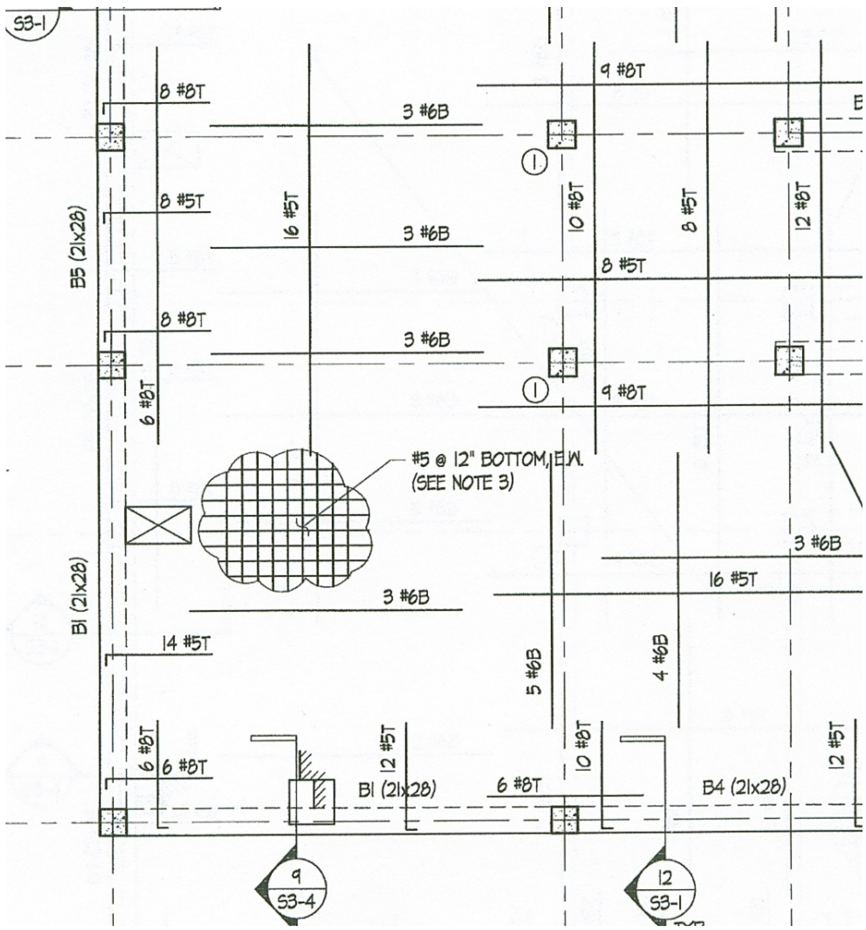


Figure 5: Slab Reinforcing Detail

The floor system also consists of edge beams that wrap the perimeter of the slab and surround openings such as stairs, elevators, and mechanical shafts. The typical edge beam is 21”x28” reinforced with #9 bars top and bottom. See Figure 6, “Portion of Concrete Beam Schedule.”

CONCRETE BEAM SCHEDULE

MARK	SIZE		REINFORCING				STIRRUPS				REMARKS
	W (INCHES)	D (INCHES)	BOTTOM BARS	TOP BARS			SIZE	TYPE	SPACING (INCHES)	END	
				LE	FL	RE					
B1	21	28	3#9	-	2#9	-	#4	S2	1@2, 12@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B2	12	28	3 #9	-	3#9	-	#4	S2	1@2, R@10	EE	
B3	10	28	3 #8	-	3#8	-	#4	S2	1@2, R@12	EE	
B4	26	20	3 #9	-	3#9	-	#4	S3	1@2, R@8 CANT. 1@2, R@8	EE	
B5	21	28	2#9	-	2#9	-	#4	S2	1@2, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B6	21	28	4#9	-	3#9	-	#4	S2	1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B7	21	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 18@8, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B8	21	28	3#9	-	2#9	3#9	#4	S2	1@2, 16@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B9	26	20	3#9	3#9	2#9	3#9	#4	S3	1@2, 20@8, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B10	22	20	4#9	5#10	2#10	5#10	#4	S3	1@2, 12@4, R@6	EE	
B11	26	20	3#9	3#9	2#9	3#9	#4	S3	1@2, 20@8, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B12	21	28	3#9	2#9	2#9	2#9	#4	S2	1@2, 14@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B13	26	20	5#9	5#9	-	7#10	#4	S3	1@2, 12@4, R@8	EE	
B14	20	20	3#9	6#9	-	6#9	#4	S3	1@2, R@6	EE	
B15	12	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 6@8, R@12 CANT. 1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B16	20	20	2#9	-	2#9	-	#4	S2	1@2, 6@8, R@12	EE	
B17	12	20	2#9	3#9	-	3#9	#4	S2	1@2, 16@6, R@12	EE	
B18	22	24	4#9	1#9	2#9	1#9	#4	S2	1@2, 15@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B19	22	24	4#9	-	2#9	-	#4	S2	1@2, 15@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B20	22	24	3#9	-	2#9	-	#4	S2	1@2, 5@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B21	21	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 12@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B22	21	28	5#9	-	2#9	-	#4	S2	1@2, R@10	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B23	21	16	2#9	-	2#9	1#9	#4	S2	1@2, 16@6, R@12	EE	
B24	21	28	5#9	2#9	2#9	2#9	#4	S2	1@2, R@12	EE	
B25	30	28	3#9	4#9	4#9	-	#4	S3	1@2, 12@12, R@18 CANT. 1@2, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B26	21	28	5#9	2#9	2#9	-	#4	S2	1@2, 10@6, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B27	21	28	3#9	2#9	2#9	-	#4	S2	1@2, 10@6, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B28	21	28	2#9	-	2#9	2#9	#4	S2	1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B29	21	28	5#9	1#9	2#9	1#9	#4	S2	1@2, 12@6, R@10	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B30	21	28	3#9	5#9	2#9	-	#4	S2	1@2, 16@4, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B31	21	28	3#9	-	2#9	5#9	#4	S2	1@2, 16@4, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B32	21	28	5#9	2#9	2#9	2#9	#4	S2	1@2, 10@6, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B33	26	22	2#9	3#9	-	3#9	#3	S2	1@2, R@6	EE	

Figure 6: Portion of Concrete Beam Schedule

Columns

The columns are for the most part 21"x21" and 22"x22 with (8) #9 bars. Instead of changing column sizes as the building rises, the engineers specified different concrete compressive strengths for different levels and reduced the reinforcing to (8) #8's in spots. The ground to 3rd floor columns have a 28 day compressive strength of 7000 psi and the columns from the 3rd floor to the roof have a 28 day compressive strength of 5000 psi.

Portions of the penthouse are supported by steel columns. For continuity and moment resisting strength, these steel columns are embedded in the full length of the concrete columns from the floor below. This results in steel columns that are 2 levels tall and fully integrated in the moment frame of the rest of the building.

The portion of the tower that does not rise past the ground floor has oversized columns designed for future expansion. The Franklin Square Hospital Center Patient Tower was realized because the existing hospital had no capacity left for additional floors. Desperately needing space, the hospital commissioned the Patient Tower and supporting spaces. In the future when such a situation arises, the new Patient tower will be able to grow with the needs of the hospital. See Figure 2, "Typical Column Caisson Connection and Column Reinforcing" and see Figure 7, "Portion of Concrete Column Schedule."

LEVEL	COLUMN	L-1 M3-1 P-1	K-2 L-2 K-12.4 L-12.4	J-7, J-8 K-7, K-8 L-7, L-8	M-3 N-3	M-6 M-7 M-8 M-9	M-4, M-5 M-10, M-11 N-4, N-5	N-12 P-6	N-6 N-7, N-8 N-9, N-10 N-11	P-3 P-4 P-5	M-12	J-9, L-6 K-9, L-9 H-6, J-6 K-6	F-4, F-5 F-6, F-10 F-11	G-4, G-5 G-6, G-10 G-11
PENTHOUSE ROOF	SIZE													
	VERTICAL BARS													
	TIES													
	REMARKS													
MAIN ROOF/ SEVENTH FLOOR	SIZE		30x12											
	VERTICAL BARS		6#8											
	TIES													
	REMARKS													
SIXTH FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
FIFTH FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
FOURTH FLOOR	SIZE		30x12	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
THIRD FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
SECOND FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#10									8#9	8#9	8#9
	TIES													
	REMARKS													
FIRST FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#10									8#9	8#9	8#9
	TIES													
	REMARKS													
GROUND FLOOR	SIZE	21x21	30x12	22x22	22x22	22x22	22x22	21x21	21x21	21x21	21x21	21x21	22x22	22x22
	VERTICAL BARS	12#10	6#10	8#10	8#10	8#9	8#10	8#11	8#11	8#11	8#10	8#9	8#9	8#9
	TIES	4#8		4#8	4#8	4#8	4#8	4#8	4#8	4#8	4#8			
	REMARKS													
DOWELS		12#7	6#7	8#8	8#8	8#8	8#8	8#8	8#8	8#8	8#8	8#7	8#7	8#7

Figure 7: Portion of Concrete Column Schedule

Roof System

The main roof system consists of cambered steel beams ranging from W12x14 to W21x73 and 1.5” deep, wide rib, 20 gauge galvanized metal deck with 3 ¼” lightweight concrete. Many of these beams are moment connected to the steel columns supporting them. A center portion of the roof contains a 10” reinforced concrete slab with concrete columns extending 2’ above the surface for future placement of the helipad deck. See Appendix A for “Roof Framing Plan.”

Wall System

The exterior façade is for the most part 7” precast concrete panels. Loads bearing connections occur at each level, with two per panel. The connections permit horizontal movement parallel to the panel except for a single non-load bearing connection which is fixed. Precast panel loads are supported only by the columns.

Lateral System

The Franklin Square Hospital Center Patient Tower utilizes the entire structure to resist lateral forces. Every column, slab and beam acts as an ordinary reinforced concrete moment frame resisting forces in both the North-South direction and the East-West direction. The large moments are carried down the building through the columns and directly into the drilled piers. The piers, with depths of 42 feet, are quite substantial and help greatly to give the building a rigid, fixed base.

In the case of wind, the force exerted on the precast panels is directly transferred to the columns and not the floor diaphragm. Once this occurs, the force is carried down the column and across the floor diaphragm to the remaining columns. The columns are expected to resist the lateral force through their moment capacity. The perimeter edge beams are stiffer than the diaphragm and function as more efficient moment frames. There are a total of 13 moment frames acting in each direction for a total of 26 moment frames in the structure. Some are very rigid and take much of the load while others are very flexible and do little in terms of lateral force resistance. The frames that reside on the perimeter of the building have beam elements consisting of substantial 21”x28” edge beams. These are the frames that take the majority of the lateral loads compared to the rest of the frames that have beam elements consisting of the slab cross-section. Figure 9, “Moment Frames Level 4” shows the typical floor and moment frame layout. The layout of the frames changes slightly on lower floors when the plan extents expand as shown in Figure 8, “Moment Frames Ground Level”. The frame designations 1 through 12.4 and A through P are referred to heavily throughout this report and are visually identifiable on Figures 8 and 9 below. For full elevation views of each moment frame, see Appendix B.

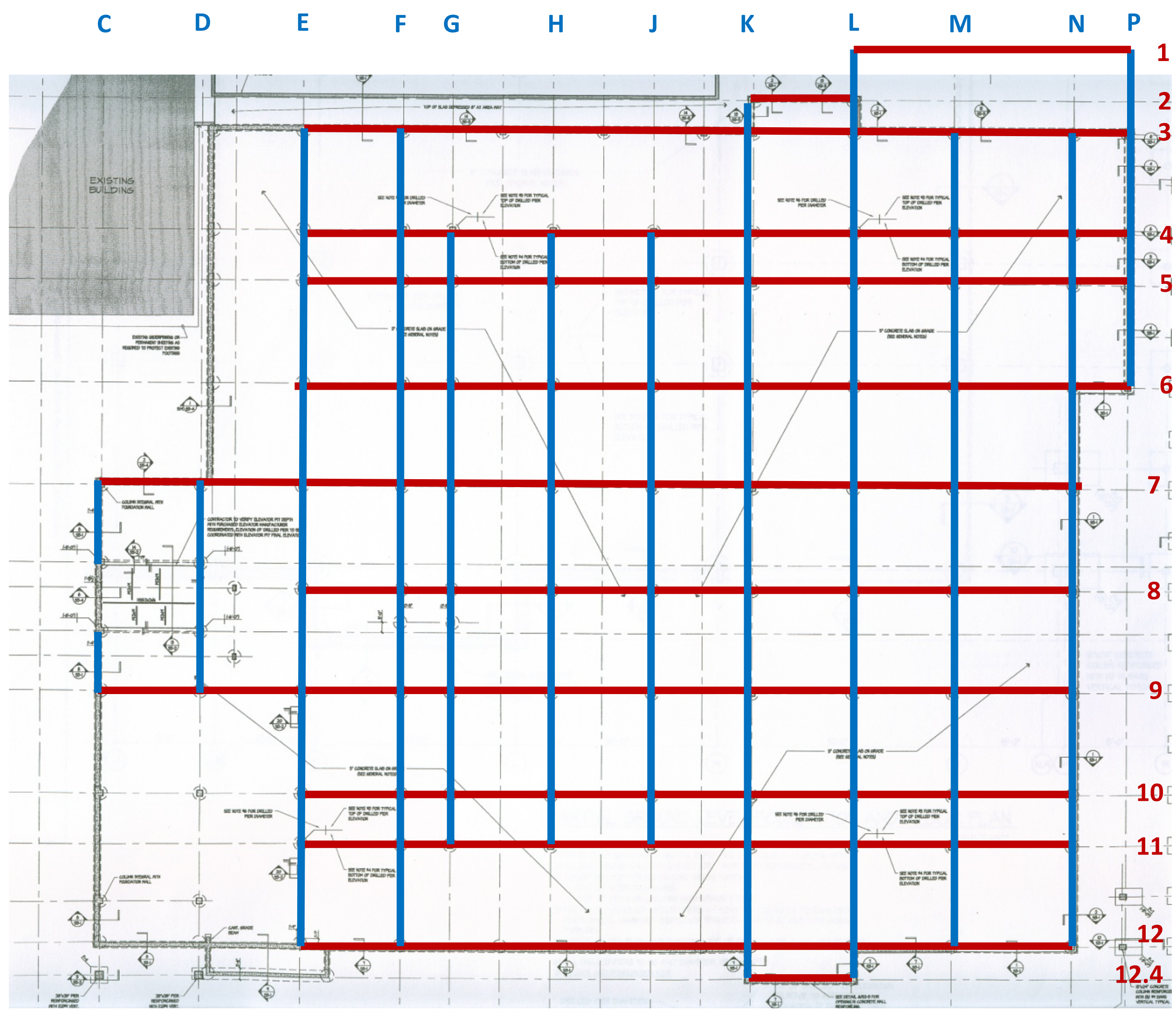


Figure 8: Moment Frames Ground Level

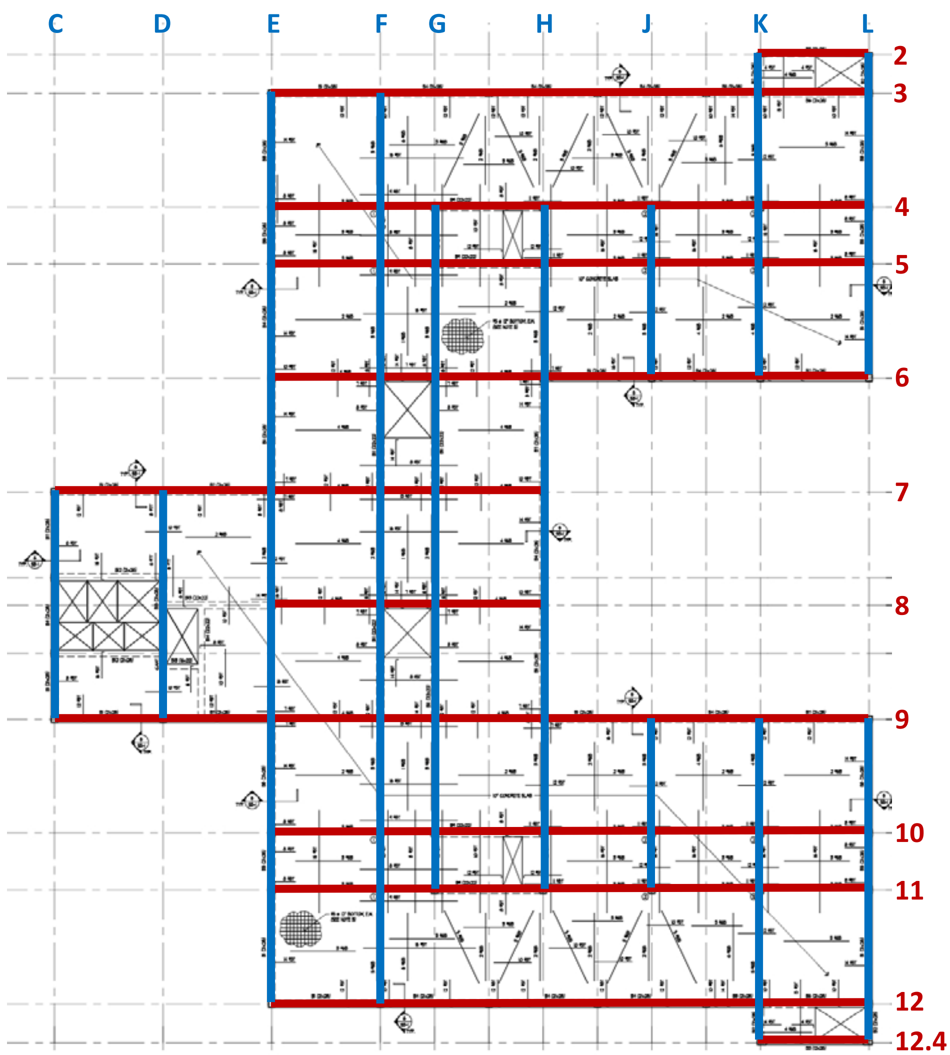


Figure 9: Moment Frames Level 4

Codes and Design Standards

General Codes and Standards

- "International Building Code 2006", International Code Council with Baltimore County Amendments
- "Minimum Design Loads for Buildings and Other Structures, ASCE 7-05", American Society of Civil Engineers

Concrete

- "Building Code Requirements for Reinforced Concrete, ACI 318", American Concrete Institute
- "ACI Manual of Concrete Practice – Parts 1 through 5"
- "Manual of Standard Practice", Concrete Reinforcing Steel Institute
- "PCI Design Handbook – Precast and Prestressed Concrete", Prestressed Concrete Institute

Structural Steel

- "Manual of Steel Construction – Allowable Stress Design", Ninth Edition
- "Manual of Steel construction – Load and resistance Factor Design", Third Edition
- "Manual of Steel Construction, Volume II Connection", ASD 9th Edition/LRFD 3rd Edition
- "Detailing for Steel construction", American Institute of Steel Construction
- "Structural Welding Code ANSI/AWS D1.1, American Welding Society

Steel Deck

- "Design Manual Floor Decks and Roof Decks", Steel Deck Institute

Material Specification

Concrete

Application	f'c @ 28 days	Weight (PCF)
Slabs-On-Grade (Interior)	3000	145
Slabs-On-Grade (Exterior)	3500	145
Reinforced Slabs	5000	145
Reinforced Beams	5000	145
Fill on Metal Deck	4000	110
Columns (Ground to 3rd Floor)	7000	145
Columns (3rd Floor to Roof)	5000	145
Walls	4000	145
Grade Beams	3000	145
Footings	3000	145
Caissons	3000	145
Topping	3000	145

Structural Steel

Application	
Deformed Reinforcing Bars	ASTM A615, Grade 60
Rolled Shapes	ASTM A992, Grade 50
Channels, Angles and Plates	ASTM A36
Structural Pipe	ASTM A53, Grade B, F_y = 35 ksi
Round HSS Shapes	ASTM A500, Grade B, F_y = 42 ksi
Structural Tubing (Square and Rectangular HSS)	ASTM A500, Grade B, F_y = 46 ksi
High Strength Bolts	ASTM A325-N typical
Anchor Rods	ASTM F1554 Grade 36
Smooth & Threaded Rod	ASTM A36
Headed Shear Studs	ASTM A108
Welding Electrodes	AWS A5.1 OR A5.5, E70XX
Galvanized Metal Deck	ASTM A653
Painted Phosphated Metal Floor Deck	ASTM A611

Gravity and Lateral Loads

Live and Dead Loads

Live Loads (LL)		
Area	**ASCE 7-05 Load**	**Design Load**
Patient Rooms	40 PSF	40 PSF
Lobbies and 1st Floor Corridors	100 PSF	100 PSF
Corridors above 1st Floor	80 PSF	80 PSF
Stairs and Exits	100 PSF	100 PSF
Mechanical	-	As Noted On Plans
Partitions	20 PSF	20 PSF
Roof	20 PSF	30 PSF Minimum (Snow Load is used when greater than 30 PSF)

Dead Loads (DL)		
Material	**ASCE 7-05 Load**	**Design Load**
Superimposed	-	20 PSF
Normal Weight Concrete	-	145 PCF
Lightweight Concrete		110 PCF
Concrete on Metal Deck	-	63 PSF
Precast Façade	-	85 PSF
Curtain Wall	-	3 PSF

Wind Loads

The wind loads were determined based on Chapter 6 of ASCE 7-05. Method 2: Analytical Procedure was used to determine loads for the main wind-force resisting system. The height of the building was taken as the top of the penthouse roof. While the penthouse covers a slightly smaller area than the floor below, the full width of the building at the seventh floor was taken as the building width and length in the calculations except where noted such as calculating story force and story shear. During the calculation of the gust factors, an assumption was made concerning the damping coefficient of the building and 1.5% was assumed after reading commentary C in ASCE 7-05 relating to damping coefficient ranges for common building types. Table 4 summarizes assumptions concerning wind directionality, exposure, and topographical influences. Table 5 summarizes Gust factors in both directions. Tables 6 and 7 summarize design wind pressures in both directions while Tables 8 and 9 summarize design wind forces in both directions. For wind pressure diagrams, see Figure 10, "N-S Wind Pressure Diagram" and Figure 11, "E-W Wind Pressure Diagram." See Appendix C for hand calculations.

Conclusions: The wind analysis below obtained a base shear force of 437.4 kips for wind in the North-South direction and 518.6 kips in the East-West direction. These two values are expected to be similar as the building sits on a rather square footprint.

Table 4: Basic Wind Pressure Parameters	
Basic Wind Speed (MPH)	90
Wind Directionality Factor (K_d)	0.85
Importance Factor (I)	1.15
Exposure Category	B
Topographic Factor (K_{zt})	1
Building Height	106 ft
N-S Building Length	260 ft
E-W Building Length	225 ft
L/B in N-S Direction	1.156
L/B in E-W Direction	0.865

Table 5: C_p, Gust Factors, GC_{pi} Factors

Wind Direction	C_p (Windward)	C_p (Leeward)	C_p (Sidewall)	Gust Factor	GC_{pi}
N-S	0.8	-0.47	-0.7	0.833	±0.18
E-W	0.8	-0.5	-0.7	0.825	±0.18

Table 6: Design Wind Pressures in the N-S Direction					
Location	**Height above ground level, z (ft)**	**k_z**	**q (psf)**	**External pressure qGC_p (psf)**	**Internal pressure $q_h(GC_{pi})$ (psf)**
Windward	106	1.005	20.370787	13.575093	±3.67
	87	0.951	19.276237	12.845685	±3.67
	74	0.906	18.364113	12.237845	±3.67
	62	0.858	17.39118	11.589482	±3.67
	50	0.81	16.418246	10.941119	±3.67
	38	0.748	15.161541	10.103651	±3.67
	26	0.668	13.539986	9.0230466	±3.73
	14	0.532	10.783342	7.1860192	±3.67
Leeward	All	1.005	20.370787	-7.975367	±3.67
Side	All	1.005	20.370787	-11.87821	±3.67

Table 7: Design Wind Pressures in the E-W Direction					
Location	**Height above ground level, z (ft)**	**k_z**	**q (psf)**	**External pressure qGC_p (psf)**	**Internal pressure $q_h(GC_{pi})$ (psf)**
Windward	106	1.005	20.370787	13.44472	±3.67
	87	0.951	19.276237	12.722317	±3.67
	74	0.906	18.364113	12.120314	±3.67
	62	0.858	17.39118	11.478178	±3.67
	50	0.81	16.418246	10.836043	±3.67
	38	0.748	15.161541	10.006617	±3.67
	26	0.668	13.539986	8.9363907	±3.73
	14	0.532	10.783342	7.1170058	±3.67
Leeward	All	1.005	20.370787	-8.40295	±3.67
Side	All	1.005	20.370787	-11.76413	±3.67

Table 8: Design Wind Forces in the N-S Direction									
Level	**Height (ft)**	**Tributary Height (ft)**	**Tributary Width (ft)**	**Windward (psf)**	**Leeward (psf)**	**Total (psf)**	**Story Force (kips)**	**Story Shear (kips)**	**Overturning Moment (ft-kips)**
Roof (8)	106	10	165	13.58	-7.98	21.55	35.6	35.6	3769
7	87	15.5	225	12.85	-7.98	20.82	72.6	108.2	9411
6	74	12.5	225	12.24	-7.98	20.21	56.8	165.0	12212
5	62	12	225	11.59	-7.98	19.56	52.8	217.8	13506
4	50	12	225	10.94	-7.98	18.92	51.1	268.9	13446
3	38	12	225	10.10	-7.98	18.08	48.8	317.7	12074
2	26	12	225	9.02	-7.98	17.00	45.9	363.6	9454
1	14	13	225	7.19	-7.98	15.16	44.3	408.0	5712
Ground	0	7	277	7.19	-7.98	15.16	29.4	437.4	0
Total							437 k		79,584 ftk

Table 9: Design Wind Forces in the E-W Direction									
Level	**Height (ft)**	**Tributary Height (ft)**	**Tributary Width (ft)**	**Windward (psf)**	**Leeward (psf)**	**Total (psf)**	**Story Force (kips)**	**Story Shear (kips)**	**Overturning Moment (ft-kips)**
Roof (8)	106	10	240	13.44	-8.40	21.85	52.4	52.4	5558
7	87	15.5	260	12.72	-8.40	21.13	85.1	137.6	11969
6	74	12.5	260	12.12	-8.40	20.52	66.7	204.3	15116
5	62	12	260	11.48	-8.40	19.88	62.0	266.3	16511
4	50	12	260	10.84	-8.40	19.24	60.0	326.3	16316
3	38	12	260	10.01	-8.40	18.41	57.4	383.8	14583
2	26	12	260	8.94	-8.40	17.34	54.1	437.9	11384
1	14	13	260	7.12	-8.40	15.52	52.5	490.3	6864
Ground	0	7	260	7.12	-8.40	15.52	28.2	518.6	0
Total							518 k		98,301 ftk

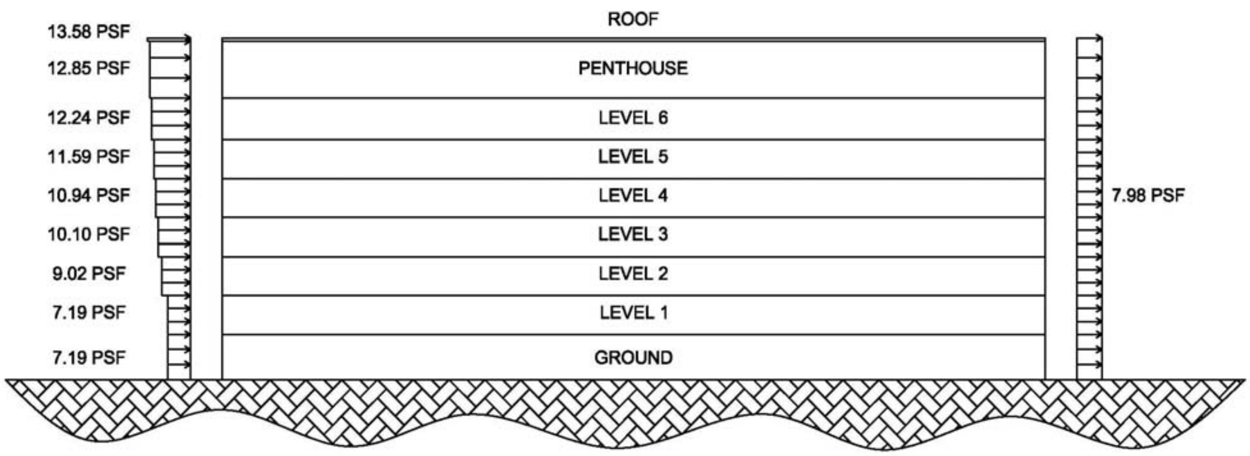


Figure 10: N-S Wind Pressure Diagram

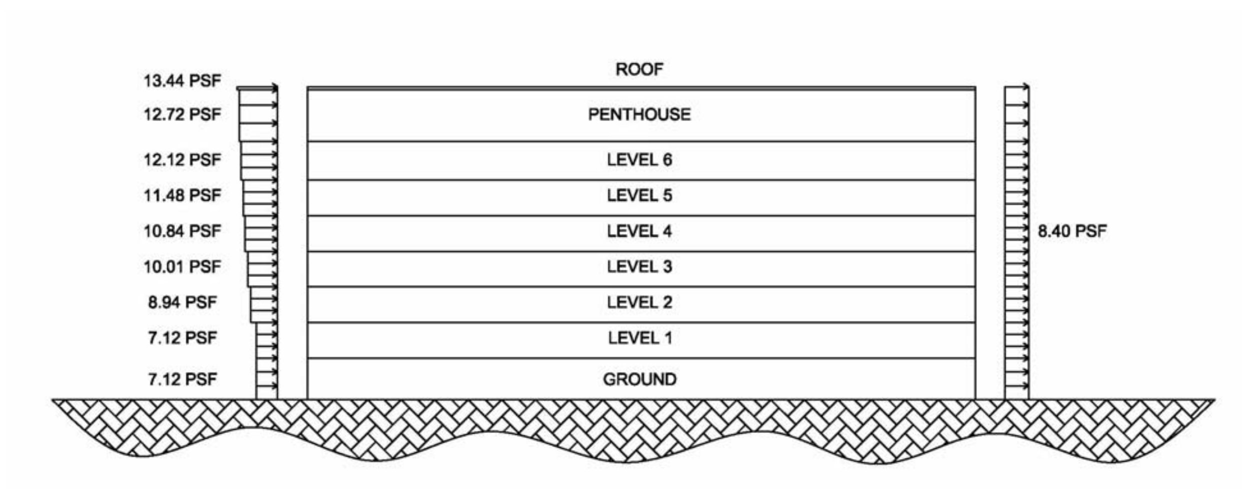


Figure 11: E-W Wind Pressure Diagram

Seismic Loads

While it may not seem to be important given its location, seismic analysis was an import consideration in the design of the Franklin Square Hospital Center due mainly to the high weight of the building. Loads were determined based on Chapter 8 of ASCE 7-05 and the Equivalent Lateral Force Procedure was used. The spectral response coefficients were determined from the USGS Earthquake Hazard Program providing higher accuracy than the map in ASCE7 can. Table 10 details the basic seismic Parameters. Table 11 details the seismic load at each level and the overturning moment at the base. Table 12 shows the components that contribute to building weight. Figure 12 shows the seismic load diagram on the building's elevation. More tables showing each component of building weight are available in Appendix D along with seismic hand calculations.

Conclusions: As calculated in this report, the seismic response coefficient is 0.016 while the designer used 0.0825. Through the use of an ETABS building model, the design period was found to be 2.45 seconds in the E-W Direction and 2.29 seconds in the N-S direction. Because the period derived from the code was lower at 1.79 seconds, it was used as the governing design period in both directions. From this, the design seismic response coefficient of 0.016 was calculated resulting in a base shear of 891.4 kips. This resulting base shear of 891k is slightly different than the base shear of 805k from the designer but still very close considering the complexity of the calculations. The differences in seismic response coefficient and base shear values are most likely from differing assumption made by the designer from this report. This report included the penthouse roof in the determination of seismic loads due to its size compared to the typical floor while the designer most likely did not include this level.

Table 10: Basic Seismic Parameters	
Spectral Response Coeff. S_s	0.176
Spectral Response Coeff. S_1	0.051
Soil Site Class	C
Seismic Design Category	A
Response Modification Factor	3
Importance Factor	1.5
Seismic Response Coeff. Cs	0.016
Total Building Weight	55,713 k
Design Base Shear	891 k

Table 11: Seismic Load							
Level	**Height, h_x (ft)**	**Story Weight, w_x (kips)**	**$w_x h_x^k$**	**$w_x h_x^k$ / $\sum w_i h_i^k$**	**Lateral Force (kips)**	**Story Shear (kips)**	**Overturning Moment (ft-k)**
Roof	105	2221	2280488	0.112	100	100	10451
Penthouse	87	6485	5032389	0.246	220	319	27769
Level 6	74	6955	4241148	0.208	185	504	37318
Level 5	62	6881	3223504	0.158	141	645	39990
Level 4	50	7092	2411323	0.118	105	750	37512
Level 3	38	7617	1720432	0.084	75	825	31363
Level 2	26	7438	954480	0.047	42	867	22542
Level 1	14	10966	559451	0.027	24	891	12480
Ground	0	59	0	0	0	891	0
Total					**891 k**		**219,424 ftk**

Table 12: Total Building Weight Calculation (kips unless otherwise noted)								
Level	**Floor (Conc. Slab + Conc. on Metal Deck)**	**Beams (Conc. + Steel)**	**Columns (Conc. + Steel)**	**Façade**	**Curtain Wall**	**Superimposed Dead**	**Partitions**	**Total**
Roof	1882	216	-	123	-	-	-	2221
Penthouse	5129	451	68	314	14	509	-	6485
Level 6	4274	507	78	663	18	707	707.46	6955
Level 5	4274	505	32	636	18	707	707.46	6881
Level 4	4456	534	32	636	18	707	707.46	7092
Level 3	4827	579	32	636	18	762	761.92	7617
Level 2	4665	539	35	636	18	772	772.14	7438
Level 1	8302	373	36	689	20	772	772.14	10966
Ground	-	-	59	-	-	-	-	59
								55,713 k

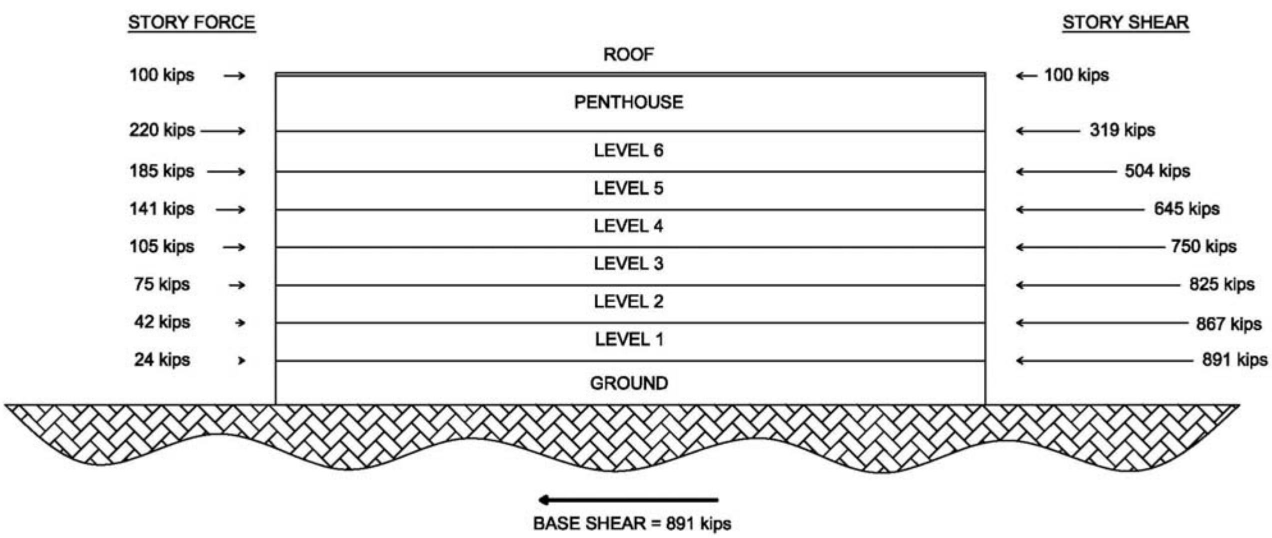


Figure 12: Seismic Load Diagram

Load Cases and Combinations

Wind Load Cases

To begin the determination of the story forces derived from wind pressures, the projected width of the building on each level was tabulated along with the resulting center of pressure and the calculated eccentricity from the center of rigidity as shown in Table 13, "Story Widths and Center of Pressure Locations." These calculated eccentricities between application of force and rigidity are what cause building torsion due to the existence of load application at a distance from rigidity and is different than the applied torsion due to the requirements of the load cases.

Table 13: Story Widths and Center of Pressure Locations							(from c. of rigidity)	
Story	Bx	By	Location of Bx/2	X CR	Location of By/2	Y CR	ΔX (ft)	ΔY (ft)
8	1980	2760	1350	1564.9	1622	1395.3	-17.9	18.9
7	2700	3124	1710	1678.2	1562	1549.2	2.7	1.1
6	2700	3124	1710	1643.1	1562	1554.8	5.6	0.6
5	2700	3124	1710	1589.0	1562	1561.5	10.1	0.0
4	2700	3124	1710	1497.9	1562	1573.7	17.7	-1.0
3	3060	3124	1530	1376.5	1562	1600.1	12.8	-3.2
2	3060	3124	1530	1285.0	1562	1648.7	20.4	-7.2
1	3684	3362	2202	1659.0	1681	1625.9	45.3	4.6

Four main load cases were looked at for wind. The first case involved the wind pressure acting along only one direction at a time analyzed separately in the N-S and E-W directions. See Figure 13, "Wind Case 1." Tables 14 and 15 show the forces applied to the diaphragms on each level of the building.

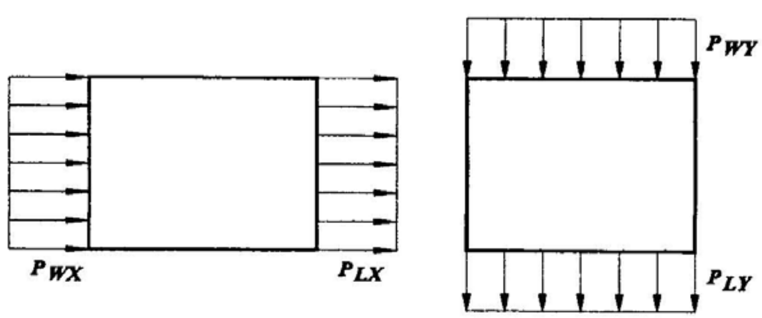


Figure 13: Wind Case 1

Table 14: Wind Case 1 (N-S)			
Story	Fx	Fy	Mz
8	0	35.6	0
7	0	72.6	0
6	0	56.8	0
5	0	52.8	0
4	0	51.1	0
3	0	48.8	0
2	0	45.9	0
1	0	44.3	0

Table 15: Wind Case 1 (E-W)			
Story	Fx	Fy	Mz
8	52.4	0	0
7	85.1	0	0
6	66.7	0	0
5	62.0	0	0
4	60.0	0	0
3	57.4	0	0
2	54.1	0	0
1	52.5	0	0

The second wind case investigated was similar to wind case 1 except only 75% of the wind pressure was applied to the building but an additional eccentric moment is added. See Figure 14, "Wind Case 2," and Tables 16, 17, 18, and 19 for applied forces and moments.

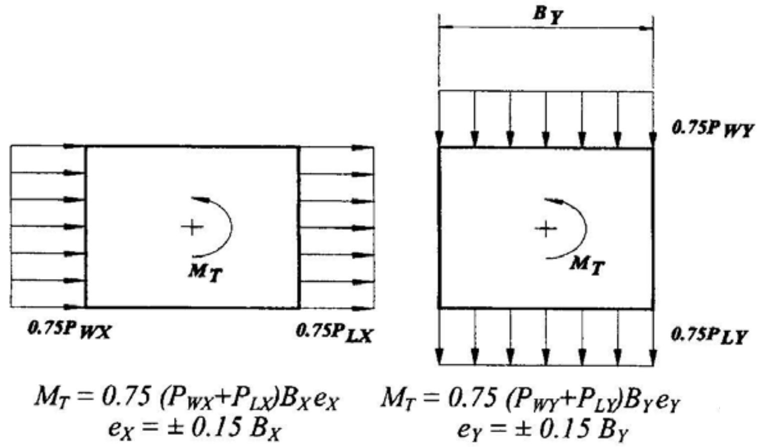


Figure 14: Wind Case 2:

Table 16: Wind Case 2 (0.75N-S, ex = +1.5 Bx)			
Story	Fx	Fy	Mz
8	0.0	26.7	7929.9
7	0.0	54.5	22052.3
6	0.0	42.6	17253.0
5	0.0	39.6	16038.0
4	0.0	38.3	15521.6
3	0.0	36.6	16799.4
2	0.0	34.4	15801.1
1	0.0	33.2	18360.1

Table 17: Wind Case 2 (0.75N-S, ex = -1.5 Bx)			
Story	Fx	Fy	Mz
8	0.0	26.7	-7929.9
7	0.0	54.5	-22052.3
6	0.0	42.6	-17253.0
5	0.0	39.6	-16038.0
4	0.0	38.3	-15521.6
3	0.0	36.6	-16799.4
2	0.0	34.4	-15801.1
1	0.0	33.2	-18360.1

Table 18: Wind Case 2 (0.75E-W, ey = +1.5 By)			
Story	**Fx**	**Fy**	**Mz**
8	39.3	0.0	16270.2
7	63.8	0.0	29908.4
6	50.0	0.0	23441.7
5	46.5	0.0	21789.9
4	45.0	0.0	21087.0
3	43.1	0.0	20173.2
2	40.6	0.0	19013.4
1	39.4	0.0	19856.8

Table 19: Wind Case 2 (0.75E-W, ey = -1.5 By)			
Story	**Fx**	**Fy**	**Mz**
8	39.3	0.0	-16270.2
7	63.8	0.0	-29908.4
6	50.0	0.0	-23441.7
5	46.5	0.0	-21789.9
4	45.0	0.0	-21087.0
3	43.1	0.0	-20173.2
2	40.6	0.0	-19013.4
1	39.4	0.0	-19856.8

For the third wind case, 75% of the wind pressure was applied in each direction simultaneously with no additional eccentric moment. See Figure 15, "Wind Case 4," and Table 20 for applied forces.

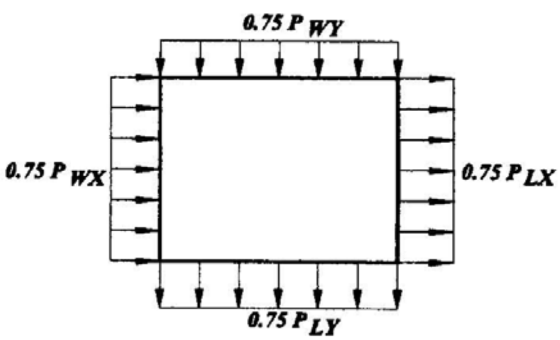
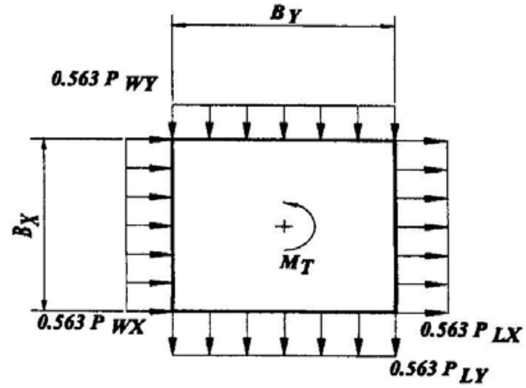


Figure 15: Wind Case 3

Table 20: Wind Case 3 (0.75N-S + 0.75E-W)			
Story	**Fx**	**Fy**	**Mz**
8	39.3	26.7	0.0
7	63.8	54.5	0.0
6	50.0	42.6	0.0
5	46.5	39.6	0.0
4	45.0	38.3	0.0
3	43.1	36.6	0.0
2	40.6	34.4	0.0
1	39.4	33.2	0.0

The fourth and final wind case investigated involved applying 56.3% of the wind pressure in each direction plus an eccentric moment with four different combinations of eccentricity. See Figure 16, "Wind Case 4," and Tables 21, 22, 23 and 24 for the applied forces and moments.



$$M_T = 0.563\ (P_{WX}+P_{LX})B_X e_X + 0.563\ (P_{WY}+P_{LY})B_Y e_Y$$
$$e_X = \pm\ 0.15\ B_X \qquad e_Y = \pm\ 0.15\ B_Y$$

Figure 16: Wind Case 4

Table 21: Wind Case 4 (0.563N-S + 0.536E-W, ex=+1.5, ey = +1.5)

Story	Fx	Fy	Mz
8	29.5	20.0	17044.4
7	47.9	40.8	38523.3
6	37.5	32.0	30166.9
5	34.9	29.7	28041.8
4	33.8	28.7	27138.1
3	32.3	27.5	27683.0
2	30.4	25.8	26066.6
1	29.5	24.9	28885.5

Table 22: Wind Case 4 (0.563N-S + 0.536E-W, ex=+1.5, ey = -1.5)

Story	Fx	Fy	Mz
8	29.5	20.0	463.7
7	47.9	40.8	250.4
6	37.5	32.0	223.3
5	34.9	29.7	207.0
4	33.8	28.7	199.4
3	32.3	27.5	1956.9
2	30.4	25.8	1869.3
1	29.5	24.9	3752.4

Table 23: Wind Case 4 (0.563N-S + 0.536E-W, ex=-1.5, ey = +1.5)

Story	Fx	Fy	Mz
8	29.5	20.0	-463.7
7	47.9	40.8	-250.4
6	37.5	32.0	-223.3
5	34.9	29.7	-207.0
4	33.8	28.7	-199.4
3	32.3	27.5	-1956.9
2	30.4	25.8	-1869.3
1	29.5	24.9	-3752.4

Table 24: Wind Case 4 (0.563N-S + 0.536E-W, ex=-1.5, ey = -1.5)

Story	Fx	Fy	Mz
8	29.5	20.0	-17044.4
7	47.9	40.8	-38523.3
6	37.5	32.0	-30166.9
5	34.9	29.7	-28041.8
4	33.8	28.7	-27138.1
3	32.3	27.5	-27683.0
2	30.4	25.8	-26066.6
1	29.5	24.9	-28885.5

Seismic Load Cases

Compared to wind, the load cases for seismic loading are much more straightforward. There are only two cases, one from each orthogonal direction. Table 25, "Center of Mass and Rigidity," details the eccentricity from the center of mass of each level to the center of rigidity of that same level. Just as in the case of wind, these calculated eccentricities between application of force and rigidity are what cause building torsion due to the existence of an imaginary leaver arm and is different than the applied torsional moment due to the requirements of the load cases. In the case of seismic loading, the code mandates an applied accidental eccentricity of 5% which was applied through an option in ETABS. Tables 26 and 27 tabulate the applied diaphragm forces due to seismic response of the structure.

Table 25: Center of Mass and Rigidity					(from c. of rigidity)	
Story	X CM	Y CM	X CR	Y CR	ΔX (ft)	ΔY (ft)
8	1527.0	1613.1	1564.9	1395.3	-3.2	18.2
7	1845.7	1543.9	1678.2	1549.2	14.0	-0.4
6	1845.7	1543.9	1643.1	1554.8	16.9	-0.9
5	1845.7	1543.9	1589.0	1561.5	21.4	-1.5
4	1845.7	1543.9	1497.9	1573.7	29.0	-2.5
3	1735.1	1556.7	1376.5	1600.1	29.9	-3.6
2	1735.1	1556.7	1285.0	1648.7	37.5	-7.7
1	2441.0	1633.5	1659.0	1625.9	65.2	0.6

Table 26: Seismic N-S		
Story	Fx	Fy
8	0.0	100.0
7	0.0	220.0
6	0.0	185.0
5	0.0	141.0
4	0.0	105.0
3	0.0	75.0
2	0.0	42.0
1	0.0	24.0

Table 27: Seismic E-W		
Story	Fx	Fy
8	100.0	0.0
7	220.0	0.0
6	185.0	0.0
5	141.0	0.0
4	105.0	0.0
3	75.0	0.0
2	42.0	0.0
1	24.0	0.0

Load Combinations

The load combinations determined to apply to the structure came from ASCE 7-05. The combinations, listed below, were not all analyzed at this time but will need to be checked with further investigation of the structure.
ASCE 7-05 Load Combinations

1.2D + 1.6S + (L or 0.8 W)
1.2D + 1.6W + L + 0.5 S
1.2D + 1.0E + L + 0.2S
0.9 D + 1.6 W + 1.6H
0.9D + 1.0E + 1.6H

As lateral analysis is the main focus of this report, dead and live loading were not considered in the ETABS building model at this time. Therefore seismic forces were multiplied by a factor of 1.0 while wind forces were multiplied by a factor of 1.6. The following load combinations were input into the ETABS building model for assessment.

1.6(WIND1ONE)	*Wind Case 1 in the N-S direction (Table 14)*
1.6(WIND1TWO)	*Wind Case 1 in the E-W direction (Table 15)*
1.6(WIND2ONE)	*Wind Case 2 in the N-S direction with positive eccentricity (Table 16)*
1.6(WIND2TWO)	*Wind Case 2 in the N-S direction with negative eccentricity (Table 17)*
1.6(WIND2THREE)	*Wind Case 2 in the E-W direction with positive eccentricity (Table 18)*
1.6(WIND2FOUR)	*Wind Case 2 in the E-W direction with negative eccentricity (Table 19)*
1.6(WIND3)	*Wind Case 3 (Table20)*
1.6(WIND4ONE)	*Wind Case 4 with positive X and positive Y eccentricity (Table 21)*
1.6(WIND4TWO)	*Wind Case 4 with positive X and negative Y eccentricity (Table 22)*
1.6(WIND4THREE)	*Wind Case 4 with negative X and positive Y eccentricity (Table 23)*
1.6(WIND4FOUR)	*Wind Case 4 with negative X and negative Y eccentricity (Table 24)*
1.0(SEISMIC 1)	*Seismic in the N-S direction (Table 26)*
1.0(SEISMIC 2)	*Seismic in the E-W direction (Table 27)*

Conclusions: After investigation, it appears the lateral system of the structure is controlled by the load combination for seismic, **1.2D + 1.0E + L + 0.2S**, and not the load combination for wind 1.2D + 1.6W + L + 0.5S. Wind Case 1 was the most critical wind load case for frames in both directions, but not as critical as seismic. While it seems strange that seismic could control the design in Baltimore, it should be noted that the Franklin Square Hospital Center Patient Tower weighs very much. Because of the large weight, even with the very low seismic response coefficient, the story shears are very high; high enough to control over wind forces even when load factors are applied.

Distribution of Lateral Loads

The relative stiffness of the frames in the N-S and E-W directions were assessed along their primary line of action with the aid of SAP. The relative stiffness of the ground level and level 4 were assessed by applying a 100 kip load at each respective floor separately and then the corresponding lateral displacement at that level was recorded. The stiffness, k, for each frame was then calculated by taking the 100 kip load and dividing it by that frames displacement. It should be noted that for the purposes of this report the procedure followed is a reasonable and acceptable way to determine the relative stiffness of each frame.

SAP Modeling Assumption:

- All Columns are Fixed at their bases
- Members not participating in Lateral Resistance were not modeled
- ACI 318-08 Modified Moments of Inertia for Columns, Beams, and Slabs
- All Column and Beam Connections were modeled with Rigid End Offsets equal to 1.0
- Beam Insertion Points were modeled correctly with Modified Stiffness from offsets
- Panel Zone's were Explicitly Modeled
- Equal Constraints at each level were used to model diaphragm constraints

Below, Tables 28, 29, 30, and 31 show the relative stiffness of each frame along their line of action at levels 1 and 5. The labeling of the frames corresponds to the labeling in Figure 8, "Moment Frames Ground Level" and Figure 9, "Moment Frames Level 4". In total there are 21 moment frames resisting lateral load from the 5th level and 25 moment frames resisting lateral load from the 1st level.

<u>Conclusions:</u> As expected, there are a number of frames in both directions that are significantly more rigid than others. Because the structure has perimeter edge beams of 21"x28," those frames that consist entirely of edge beams or have a large percentage of edge beams are far more rigid than those frames that have only the slab cross section acting as beams.

The distribution of loads at level 5, shown in Tables 30 and 31 show this very clearly as the two frames that consist entirely of perimeter edge beams in the East-West direction, Frames 3 and 12, have much larger relative stiffness's than the other frames at 18.47% and 18.39%. The next stiffest frames in the East-West direction consist of frames that are for the majority made up of perimeter edge beams, Frames 6, 7, and 9. These all have three bays where only the slab contributes resistance therefore they have relative stiffness's of only 11.79%, 10.27%, and 15.53%. The remaining 7 frames in the East-West direction are either very short or have beam elements predominantly composed of slab sections. The highest relative stiffness of this group is 5.33%. This means, that at a minimum, the moment frames with perimeter edge beams are 3.3 times stiffer than the moment frames consisting only of slab elements. The frames in the N-S direction have very similar stiffness ratios but the data is more difficult to interpret at first glance because many of the frames are of greatly differing lengths and beam/slab compositions although the same principals of relative stiffness apply.

The calculations regarding relative stiffness of the moment frame portions on the 1st level, Tables 28 and 29 are even more difficult to discern without also carefully looking at the structural plan of the level and understanding how the frames on this level relate to the levels above. The 1st level differs from the horseshoe shape of the levels above by expanding east and forming an almost square footprint. Therefore, some moment frames that mostly consisted of 21"x28" beam elements above, now only consist of 10" slab elements. Looking at the calculated relative stiffness's, once again, the exterior frames that have 21"x28" beam elements are stiffer than the those frames with slab elements but the magnitude of the differences is less when compared to the upper levels.

Table 28: Level 1 Relative Frame Stiffness N-S				
Frame Line	**P (Kips)**	**Δp (in)**	**k (K/in)**	**Percent of Total Stiffness**
C	100	0.0964	1037.344	8.36%
D	100	0.2787	358.8088	2.89%
E	100	0.0697	1434.72	11.56%
F	100	0.0769	1300.39	10.48%
G	100	0.089	1123.596	9.06%
H	100	0.1307	765.1109	6.17%
J	100	0.146	684.9315	5.52%
K	100	0.0714	1400.56	11.29%
L	100	0.0676	1479.29	11.92%
M	100	0.1332	750.7508	6.05%
N	100	0.0677	1477.105	11.90%
P	100	0.168	595.2381	4.80%
Total			12407.85	

Table 29: Level 1 Relative Frame Stiffness E-W				
Frame Line	**P (Kips)**	**Δp (in)**	**k (K/in)**	**Percent of Total Stiffness**
1	100	0.3493	286.2869	2.43%
2	100	0.2918	342.7005	2.90%
3	100	0.0674	1483.68	12.57%
4	100	0.1153	867.3027	7.35%
5	100	0.1086	920.8103	7.80%
6	100	0.0864	1157.407	9.80%
7	100	0.0678	1474.926	12.49%
8	100	0.1398	715.3076	6.06%
9	100	0.0873	1145.475	9.70%
10	100	0.1281	780.6401	6.61%
11	100	0.1108	902.5271	7.65%
12	100	0.0722	1385.042	11.73%
12.4	100	0.2918	342.7005	2.90%
Total			11804.81	

Table 30: Level 5 Relative Frame Stiffness N-S				
Frame Line	**P (Kips)**	**Δp (in)**	**k (K/in)**	**Percent of Total Stiffness**
C	100	0.6916	144.5922	13.31%
D	100	4.5066	22.18968	2.04%
E	100	0.4213	237.3606	21.85%
F	100	1.0879	91.92021	8.46%
G	100	1.1956	83.64001	7.70%
H	100	0.9509	105.1635	9.68%
J	100	4.1041	24.36588	2.24%
K	100	0.6929	144.321	13.29%
L	100	0.43	232.5581	21.41%
Total			1086.111	

Table 31: Level 5 Relative Frame Stiffness E-W				
Frame Line	**P (Kips)**	**Δp (in)**	**k (K/in)**	**Percent of Total Stiffness**
2	100	3.6203	27.62202	2.62%
3	100	0.5143	194.439	18.47%
4	100	2.1376	46.78144	4.44%
5	100	2.1073	47.45409	4.51%
6	100	0.8058	124.1003	11.79%
7	100	0.9254	108.0614	10.27%
8	100	5.5042	18.16794	1.73%
9	100	0.6119	163.4254	15.53%
10	100	2.2067	45.31654	4.31%
11	100	1.7837	56.06324	5.33%
12	100	0.5166	193.5734	18.39%
12.4	100	3.6203	27.62202	2.62%
Total			1052.627	

ETABS 3D Building Model

It was determined early in the analysis of the Franklin Square Hospital Center Patient Tower that an advanced 3D model would be necessary to accurately determine member forces and stresses given lateral loading. This is in part due to the numerous frames in each direction, the complex interaction of varying member sizes to the overall rigidity, the lengthy calculations needed for torsion effects on a building of this complexity and the numerous load combinations required by code. The lateral system of the structure was modeled given the assumption below, and all thirteen load combinations were input in ETABS for detailed and accurate analysis. See Figure 17, "ETABS 3D Model" for a rendering of the lateral system.

ETABS Modeling Assumption:

- All Columns are Fixed at their bases
- Members not participating in Lateral Resistance were not modeled
- ACI 318-08 Modified Moments of Inertia for Columns, Beams, and Slabs
- All Column and Beam Connections were modeled with Rigid End Offsets equal to 1.0
- Beam Insertion Points were modeled correctly with Modified Stiffness from offsets
- Panel Zone's were Explicitly Modeled
- Rigid Diaphragms were created on Each Level
- Diaphragms were given Mass calculated from Story Weight

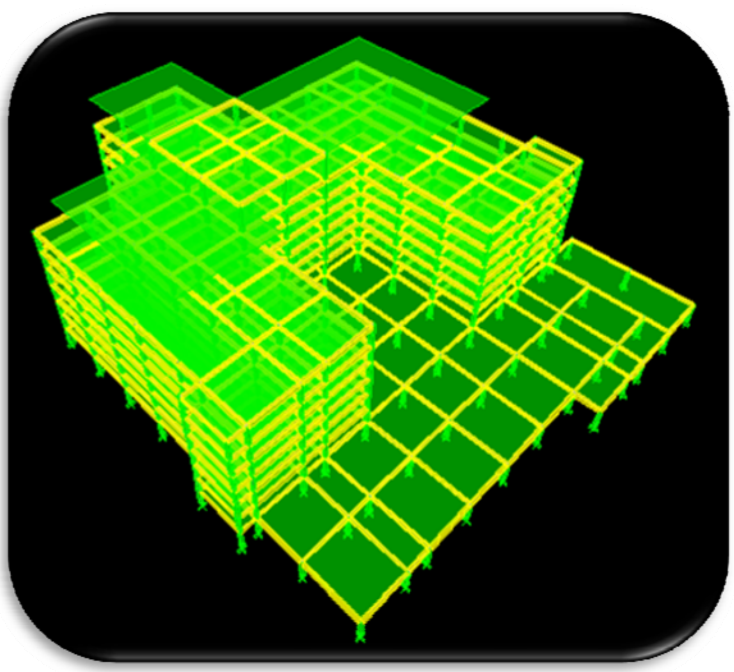


Figure 17: ETABS 3D Model

With the entire lateral system modeled and the diaphragm masses assigned, a modal analysis was conducted resulting in the fundamental periods of the building. As mentioned earlier in this report, the period of vibration in the East-West direction was 2.45 seconds while the period of vibration in the North-South Direction was 2.29 seconds, and the period of vibration in torsion was 1.78 seconds. Additionally, the center of mass and rigidity were calculated in ETABS and is tabulated above in Table 25 “Center of Mass and Rigidity” and displayed visually below for a typical level in Figure 18, “Center of Mass, Pressure, and Rigidity Level 5”

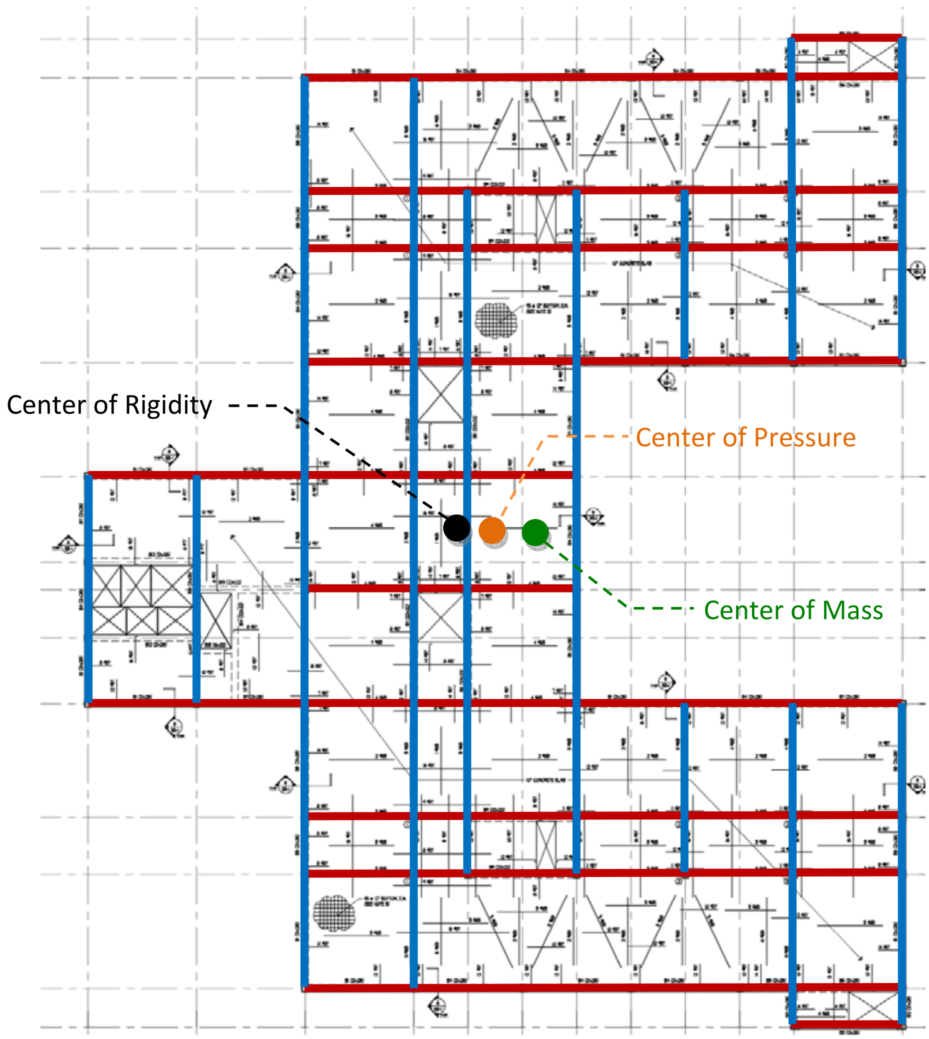


Figure 18: Center of Mass, Pressure, and Rigidity Level 5

<u>Conclusions:</u> As seen in Figure 18, torsion effects are very small for loading in the East-West Direction but have moderate effect for loading in the North-South direction. This is most clearly show when seismic loads are applied in the North-South direction and the frames on the east side of the building, namely Frame L, take a larger percentage of the load then the frames on the west side of the building, namely Frame C. Table 36, "Level 4 Column Shear, Seismic North-South" shows this clearly as the distribution of loading from N-S seismic is noticeably eccentric when compared to the effective direct non-eccentric loading from SAP. Frame L takes 9% more load while frames C, D, and E take on average 2.69% less load. This same effect from loading in the N-S direction from wind is less noticeable because less eccentricity exists between the center of rigidity and the center of pressure.

For the load cases in the East-West direction where eccentricity is almost negligible, the distribution of loads from ETABS and SAP can be more directly compared for accuracy. The analysis in ETABS appears to make more of a difference in rigidity between the stiffest and softest frames when compared to the relative stiffness procedure using SAP. For instance, Table 37, "Level 4 Column Shear, Seismic East-West" shows ETABS sending 23.9% of the story shear to frame 12, while SAP's analysis only sends 18.39% of the load to that same frame. It appears the analysis in ETABS views the stiffer frames to be more stiff than the relative stiffness method does. The differences are very small though with differences on average of 6%. These small differences are small enough that the method used to determine lateral distribution of loads is very close to the analysis computed in ETABS.

Table 32: Ground Level Column Shear, Seismic North-South				
Frame	**Seismic Induced Shear (1.0 Seismic N-S)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
C	-13.27	2.26%	8.36%	-6.10%
D	-15.08	2.57%	2.89%	-0.32%
E	-49.56	8.44%	11.56%	-3.12%
F	-45.23	7.71%	10.48%	-2.77%
G	-39.16	6.67%	9.06%	-2.38%
H	-13.62	2.32%	6.17%	-3.85%
J	-26.99	4.60%	5.52%	-0.92%
K	-48.61	8.28%	11.29%	-3.01%
L	-39.17	6.67%	11.92%	-5.25%
M	-74.02	12.61%	6.05%	6.56%
N	-157.42	26.82%	11.90%	14.92%
P	-64.77	11.04%	4.80%	6.24%
Total	-586.90			

Table 33: Ground Level Column Shear, Seismic East-West				
Frame	**Seismic Induced Shear (1.0 Seismic E-W)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
1	-22.11	3.62%	2.43%	1.19%
2	-12.59	2.06%	2.90%	-0.84%
3	-80.49	13.17%	12.57%	0.60%
4	-40.68	6.66%	7.35%	-0.69%
5	-46.01	7.53%	7.80%	-0.27%
6	-53.53	8.76%	9.80%	-1.05%
7	-48.15	7.88%	12.49%	-4.62%
8	-48.29	7.90%	6.06%	1.84%
9	-40.32	6.60%	9.70%	-3.11%
10	-39.58	6.48%	6.61%	-0.14%
11	-45.96	7.52%	7.65%	-0.13%
12	-116.59	19.07%	11.73%	7.34%
12.4	-16.94	2.77%	2.90%	-0.13%
Total	-611.24			

Table 34: Ground Level Column Shear, Wind Case 1 North-South				
Frame	**Wind Induced Shear (1.6 Wind1 N-S)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
C	-10.52	2.62%	8.36%	-5.74%
D	-12.13	3.02%	2.89%	0.13%
E	-39.49	9.84%	11.56%	-1.72%
F	-36.16	9.01%	10.48%	-1.47%
G	-30.93	7.71%	9.06%	-1.35%
H	-13.44	3.35%	6.17%	-2.82%
J	-20.68	5.15%	5.52%	-0.37%
K	-35.67	8.89%	11.29%	-2.40%
L	-29.81	7.43%	11.92%	-4.49%
M	-44.16	11.01%	6.05%	4.96%
N	-91.19	22.73%	11.90%	10.82%
P	-37.06	9.24%	4.80%	4.44%
Total	-401.24			

Table 35: Ground Level Column Shear, Wind Case 1 East-West				
Frame	**Wind Induced Shear (1.6 Wind1 E-W)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
1	-20.29	3.72%	2.43%	1.29%
2	-12.44	2.28%	2.90%	-0.62%
3	-74.55	13.65%	12.57%	1.08%
4	-38.32	7.02%	7.35%	-0.33%
5	-42.81	7.84%	7.80%	0.04%
6	-48.99	8.97%	9.80%	-0.83%
7	-43.23	7.92%	12.49%	-4.58%
8	-41.86	7.67%	6.06%	1.61%
9	-37.57	6.88%	9.70%	-2.82%
10	-35.42	6.49%	6.61%	-0.13%
11	-40.48	7.41%	7.65%	-0.23%
12	-95.00	17.40%	11.73%	5.66%
12.4	-15.11	2.77%	2.90%	-0.14%
Total	-546.07			

Table 36: Level 4 Column Shear, Seismic North-South				
Frame	**Seismic Induced Shear (1.0 Seismic N-S)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
C	-70.11	10.93%	13.31%	-2.38%
D	-13.39	2.09%	2.04%	0.04%
E	-161.62	25.19%	21.85%	3.34%
F	-35.45	5.53%	8.46%	-2.94%
G	-31.79	4.96%	7.70%	-2.75%
H	-69.45	10.83%	9.68%	1.14%
J	-9.52	1.48%	2.24%	-0.76%
K	-55.12	8.59%	13.29%	-4.70%
L	-195.04	30.40%	21.41%	8.99%
Total	-641.49			

Table 37: Level 4 Column Shear, Seismic East-West				
Frame	**Seismic Induced Shear (1.0 Seismic E-W)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
2	-14.24	2.22%	2.62%	-0.41%
3	-129.70	20.17%	18.47%	1.70%
4	-20.55	3.20%	4.44%	-1.25%
5	-20.41	3.17%	4.51%	-1.33%
6	-69.90	10.87%	11.79%	-0.92%
7	-47.80	7.44%	10.27%	-2.83%
8	-4.89	0.76%	1.73%	-0.97%
9	-117.42	18.26%	15.53%	2.74%
10	-23.25	3.62%	4.31%	-0.69%
11	-23.68	3.68%	5.33%	-1.64%
12	-153.67	23.90%	18.39%	5.51%
12.4	-17.37	2.70%	2.62%	0.08%
Total	-642.88			

Table 38: Level 4 Column Shear, Wind Case 1 North-South				
Frame	**Wind Induced Shear (1.6 Wind N-S)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
C	-46.15	13.31%	13.31%	0.00%
D	-8.09	2.33%	2.04%	0.29%
E	-96.73	27.91%	21.85%	6.05%
F	-19.67	5.67%	8.46%	-2.79%
G	-17.54	5.06%	7.70%	-2.64%
H	-37.08	10.70%	9.68%	1.01%
J	-4.48	1.29%	2.24%	-0.95%
K	-26.57	7.67%	13.29%	-5.62%
L	-90.33	26.06%	21.41%	4.65%
Total	-346.64			

Table 39: Level 4 Column Shear, Wind Case 1 East-West				
Frame	**Wind Induced Shear (1.6 Wind E-W)**	**% of Total**	**Relative Stiffness from SAP**	**% Difference**
2	-10.56	2.49%	2.62%	-0.14%
3	-97.56	22.99%	18.47%	4.52%
4	-14.33	3.38%	4.44%	-1.07%
5	-13.93	3.28%	4.51%	-1.23%
6	-48.51	11.43%	11.79%	-0.36%
7	-31.19	7.35%	10.27%	-2.92%
8	-2.41	0.57%	1.73%	-1.16%
9	-75.30	17.74%	15.53%	2.22%
10	-13.82	3.26%	4.31%	-1.05%
11	-13.81	3.25%	5.33%	-2.07%
12	-92.94	21.90%	18.39%	3.51%
12.4	-10.03	2.36%	2.62%	-0.26%
Total	-424.39			

Lateral System Load Path Confirmation

While, in principle the moment frame lateral system of the Franklin Square Hospital Center Patient Tower is simple, the distribution of lateral load between the many frames is complicated. This is partly due to the differences in interior vs. exterior column sizes, but it is mostly due to the difference in the horizontal beam elements of the moment frames and the length of the frame along its primary line of action. While the exterior moment frames all consist of beam elements 21" wide by 28" deep, the interior "beam elements" are nothing more than the 10" flat plate floor system with, in some spots, beams of larger sizes framing around mechanical, elevator and stair openings. The relative stiffness procedure conducted with the aid of SAP was very effective in determining the percentage of load being resisted by each moment frame. These results largely confirm the output from ETABS with differences in relative stiffness below 8% when eccentricity is not considered. Therefore it is safe to say the ETABS 3D model is an accurate representation of the Franklin Square Hospital Centers Patient Tower's lateral system and the output from ETABS is indeed accurate and can be used for more detailed analysis in further reports.

Lateral System Checks

Strength

A strength check of the slab, edge beams, and columns was conducted under combined gravity and lateral loading to verify the current design. The checks were performed on Level 4 in the North West corner; the same area checked previously for gravity loading in Technical Assignment 1. When the slab in this area was checked previously under just gravity loading, a minor issue was found that was the cause of differing analysis methods used by this report and the designer. In that report, Direct Design Method was used to determine critical moments in the slab. These calculated moments resulted in some areas of perceived weakness and some areas of perceived overdesign. These same issues were found again in this report and a logical reason behind these differences has been uncovered. The designer likely used a more advanced equivalent frame method or moment distribution method that more accurately determined critical moments for slab design. With this understood, the following strength checks of the slab, while not yet acceptable in this report through calculations, can be proven acceptable through logical reasoning. For all hand calculations of strength checks, see Appendix E.

Slab: The slab portion of moment frame K was analyzed for strength between frames 3 and 4. Lateral strength requirements were assessed so load combination 1.2D+1.0E+L was used and the critical moments from dead, live, and seismic were calculated and compared to the strength of the current design. The exterior negative moment was calculated as worst case, 153 ft kips, while the mid span positive moment was 168 ft kips and the negative interior moment was 299 ft kips. At this point it was obvious the method used to distribute moments to the slab differed from the designers as the area of steel provided by the designer was identical in the two negative moment sections. With this known, the capacity of the slab was checked and ΦM_n for the negative moment sections was found to be 296 ft kips while the positive moment section was found to be 206 ft kips. This resulted in two of the three sections checking ok while one section was over stressed. Had the negative moments distributed equally between the interior and exterior faces, the factored Mu would have been close to 226 ft kips and would have checked ok compared to the capacity of the section at 299 ft kips.

Edge Beam: As previously discussed, the perimeter edge beams of the structure offer far more rigidity to the moment frames they compose than the moment frames using slab cross-sections. As a result the lateral load induced moments in the beams are much larger than the induced moments in the slab. When combined with factored gravity loading, the negative exterior moment acting on the beam was 188 ft kips while the factored negative interior moment was 285 ft kips. The same issues experienced in the slab check became issues in the beam check. The positive induced moment was a lowly 119 ft kips. The moment capacity of the beam was checked and found to be 227 ft kips in the negative moment regions and 336 ft kips in the positive moment regions. As in the slab check, the factored negative moment, if averaged, would have been close to 237 ft kips. In this case, even with the negative moments being distributed evenly, the section still does not have enough strength. This leads to the second difference in assumptions between this report and the designers. The base shear was calculated at 891 kips compared to the design base shear of 805 kips. This difference means that the story shear on every floor is slightly more in this report than as designed which results in higher induced moments in the lateral moment frames. With a lesser base shear, and a more advanced procedure for distributing gravity load moments in the slabs and beams, the beam would have likely proved adequately strong.

Columns: The column on the grid line intersection of frame F and frame 4 was also checked for strength against combined gravity and lateral loading. First, the column was checked on level 4 and a factored axial force of 409 kips was found along with a factored moment of 221 kips. Figure 19, "Column F4 Strength Check Level 4", shows the column is perfectly adequate to carry the factored axial load and moment from combined gravity and seismic lateral loading.

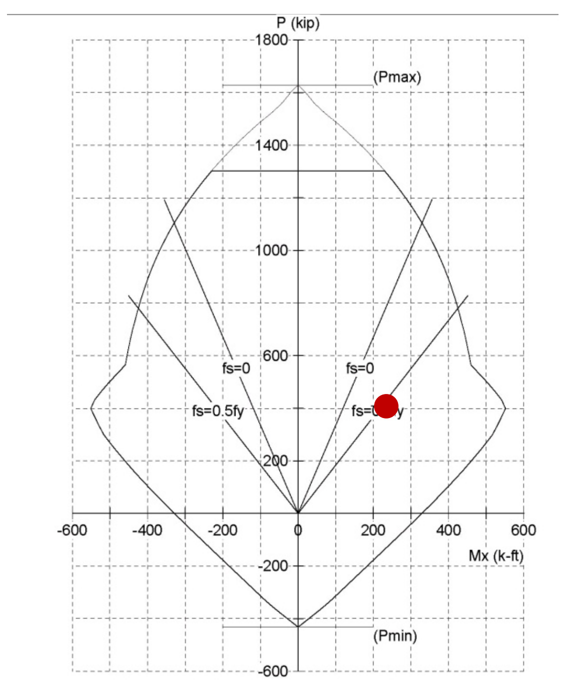


Figure 19: Column F4 Strength Check Level 4

The same column was then checked on the ground floor but in this case it was not immediately clear which seismic load case controlled the design, therefore factored seismic loading in the North-South direction(**1** on Figure 20) was assessed as well as factored seismic loading in the East-West direction (**2** on Figure 20), both with gravity loading. As seen in Figure 20, "Column F4 Strength Check Ground Level" the column is severely overstressed from the load combination with seismic loading in the East-West Direction.

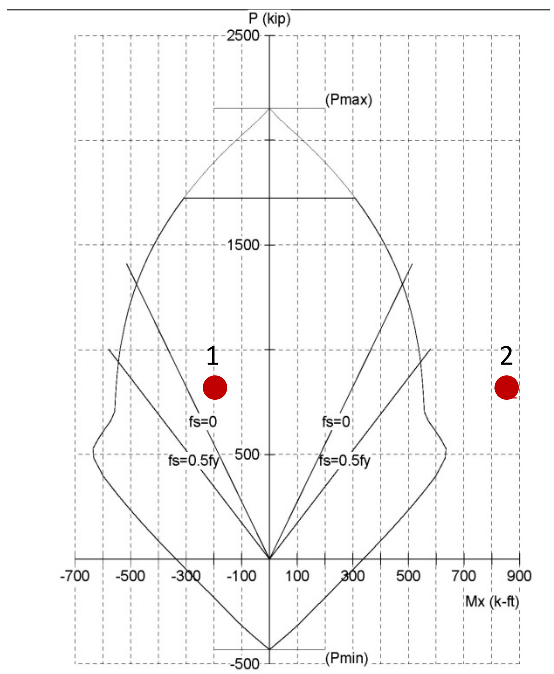


Figure 20: Column F4 Strength Check Ground Level

It is of concern, with respect to seismic loading in the East-West Direction, that this column is so severely overloaded. This loading must be the result of an error in the modeling of the ETABS model or an error in how the rigid diaphragm is distributing loads when compared to reality. In future analysis this loading will be looked at much more carefully and resolved.

Drift and Story Drift

As part of the design check for the existing structure, the building was analyzed for drift under lateral loading. While there are code requirement for seismic drift, there are no firm code requirements for lateral wind drift. As such, common accepted allowable drift limits were assessed.

Wind: Total drift due to wind as well as story drift due to wind were compared to the common allowable drift of H/400. While wind strength checks were governed by factored wind in the orthogonal directions without added eccentric moments, serviceability wind drift requirements were governed by un-factored wind case 4 involving 56.25% of the wind pressure acting in each direction including an added eccentric moment. What is interesting about the calculated deflections is how small they are. This is because the column and beam sizes of the moment frames were sized for seismic strength requirements which are far larger than sizes needed for wind serviceability. Table 40, "Wind Drift (Un-Factored Wind Case 4 One)" tabulates these calculated drift values. While most of the critical displacements occurred along the same column line in all plans, level 8 and level 1 had critical displacements in different column locations. Level 8 has smaller plan extents while level 1 had larger plan extents. In all cases, the critical displacement occurred in the same relative position on each plan, the south-east corner. All levels were determined to be in accordance with the standard allowable drift limit of H/400.

Table40 : Wind Drift (Un-Factored Wind Case 4 One)

Story	Total Height (ft)	X-Displacement (in)	Y-Displacement (in)	Total Drift (in)	Standard Allowable Total Drift (H/400) (in)	Story Height (ft)	Story Drift (in)	Standard Allowable Story Drift (H/400) (in)
8	105	0.662	0.493	0.825	3.15	18	0.187	0.54
7	87	0.654	0.504	0.826	2.61	13	0.083	0.39
6	74	0.589	0.452	0.743	2.22	12	0.097	0.36
5	62	0.513	0.392	0.646	1.86	12	0.121	0.36
4	50	0.417	0.319	0.525	1.50	12	0.142	0.36
3	38	0.304	0.233	0.383	1.14	12	0.153	0.36
2	26	0.181	0.141	0.230	0.78	12	0.147	0.36
1	14	0.066	0.051	0.083	0.42	14	0.083	0.42

Seismic: Seismic drift values were measured from the center of mass of each level according to ASCE 7-05 Section 12. Of the two seismic cases analyzed, seismic loading in the East-West was the critical load case for lateral drift. Seismic allowable drift was calculated in accordance with ASCE 7-05 Section 12 as 0.010hsx. Table 41, "Seismic Drift (Factored Seismic E-W)" displays all appropriate data concerning code compliance. All levels were determined to be in accordance with the code allowable drift limits.

Table 41: Seismic Drift (Factored Seismic E-W)

Story	Total Height (ft)	X-Displacement (in)	Y-Displacement (in)	Total Drift (in)	Total δx = 2.5δxe/1.5	Story Height (ft)	Story Drift (in)	Story δx = 2.5δxe/1.5	0.010 hsx
8	105	2.203	-0.006	2.203	**3.67**	18	0.428	**0.71**	2.16
7	87	1.775	0.025	1.775	**2.96**	13	0.199	**0.33**	1.56
6	74	1.576	0.024	1.576	**2.63**	12	0.235	**0.39**	1.44
5	62	1.341	0.023	1.342	**2.24**	12	0.287	**0.48**	1.44
4	50	1.055	0.002	1.055	**1.76**	12	0.322	**0.54**	1.44
3	38	0.732	0.015	0.732	**1.22**	12	0.319	**0.53**	1.44
2	26	0.413	0.011	0.414	**0.69**	12	0.262	**0.44**	1.44
1	14	0.151	0.011	0.152	**0.25**	14	0.152	**0.25**	1.68

Conclusions: Seismic and Wind displacements are of similar importance with some floors being controlled by wind requirements and others by seismic requirements. However, serviceability requirements relating to drift are of no concern given the current design of the lateral system of the Franklin Square Hospital Center Patient Tower as all drift values are well below their allowable limits. Seismic strength requirements control over any drift serviceability requirement for any loading in any direction.

Overturning and Impact on Foundations

Overturning moment caused by the factored seismic loads required checking for overall stability of the structure and impacts on the foundation system. The load combination 0.9D+1.0E+1.6H was determined to be the critical load combination for uplift and overturning. The load combination was analyzed for seismic in both orthogonal directions and uplift was of no concern for seismic load in the East-West direction but was of concern in the North-South direction. Figure 19, "Frame K Uplift Load Combination Reactions" shows the forces on the base of Frame K resulting from this load combination. While almost every column does not experience uplift forces, one column along line 12.4 does have a slight uplift force. The magnitude of this uplift force is 14.9 kips.

Conclusions: The typical caisson or drilled pier is 3ft in diameter and 42 feet deep with an un-factored self weight of 44.5 kips. Most uplift forces in the structure have magnitudes ranging from 2 to 15 kips with the exception of two strange locations where there are uplift forces of 35 kips. This is of no concern given the factored self weight of the caisson at 40 kips. Therefore, the uplift force of 14.9 kips, results in a factored unbalanced downward force of 25 kips. Even in the case of the two larger uplift forces of 35 kips, there is still a net downward force of 5 kips. Therefore it is concluded that the uplift cause by lateral forces to the foundations of the Franklin Square Hospital Center are of no concern. The moment capacity of these caissons, while not investigated in this report, will need detailed investigation in further reports.

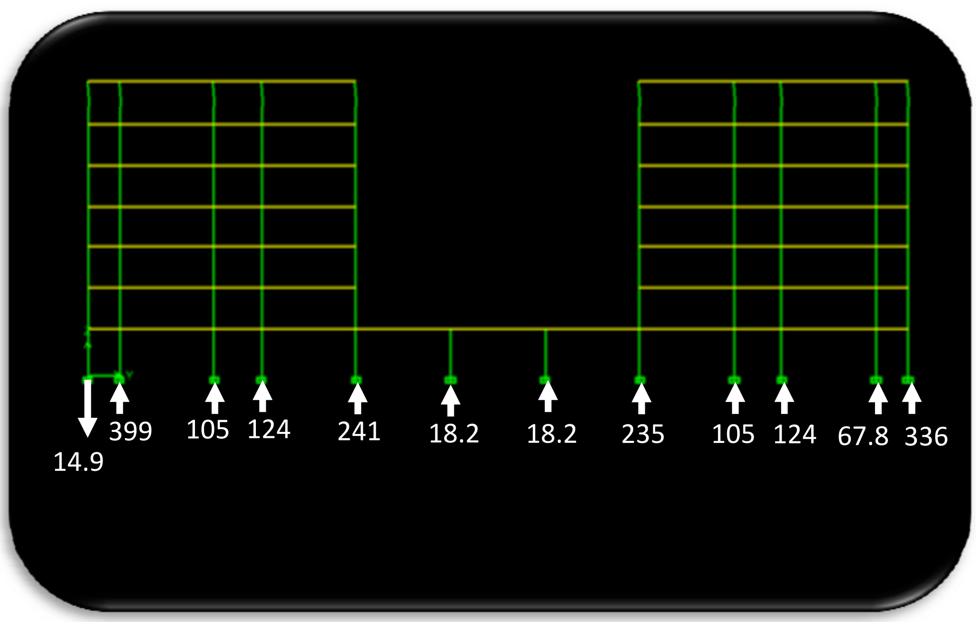


Figure 19: Frame K Uplift Load Combination Reactions

Conclusion

After investigation, it has been determined that as a whole, the lateral system of the Franklin Square Hospital Center Patient Tower is adequate in many areas but there are some areas of concern. Torsion of the structure under these lateral loads was also investigated and determined to impact the design very little as the center of mass and center of rigidity are very close.

The relative stiffness of each moment frame was calculated with the aid of SAP to determine the distribution of lateral loads and checked against the output from ETABS. The results from each method proved to be very similar which validates the output of the full 3D ETABS model suggesting the exterior moment frames with larger beam sections take the majority of the lateral load.

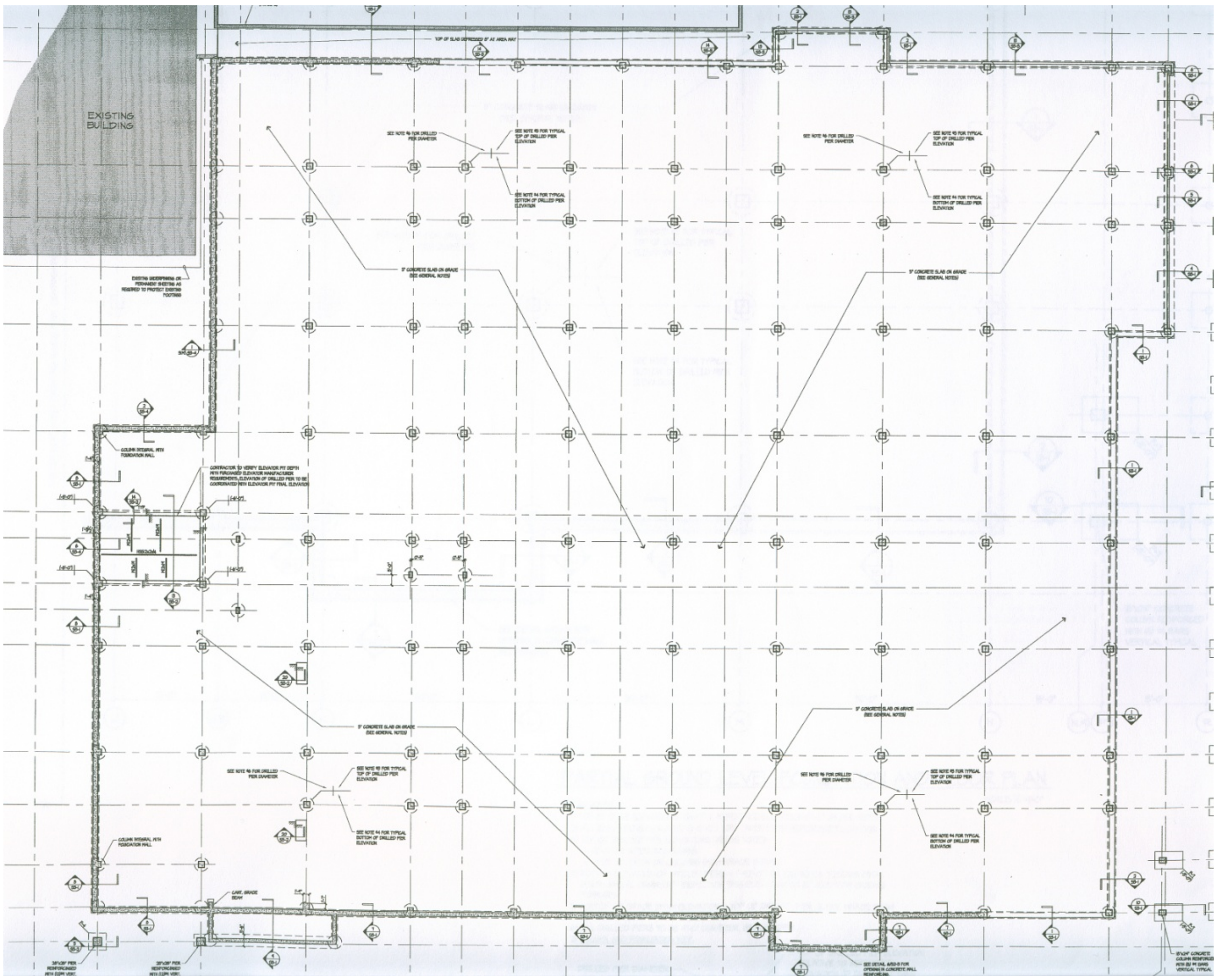
Lateral loads were calculated and applied according to ASCE 7-05 and, while unusual for East Coast buildings, seismic has controlled the design of the lateral system in both directions due to the large mass of the building when compared to a lighter steel structure that would likely have been controlled by wind.

In addition, serviceability checks were completed and found to not control the design of the lateral system in any way as all drift values were well below code maximums or common allowable limits. The foundation reactions were analyzed for overturning and uplift and small uplift forces were found in some areas. These uplift forces are of no concern given the large weight of the caisson foundations which completely balance the uplift forces.

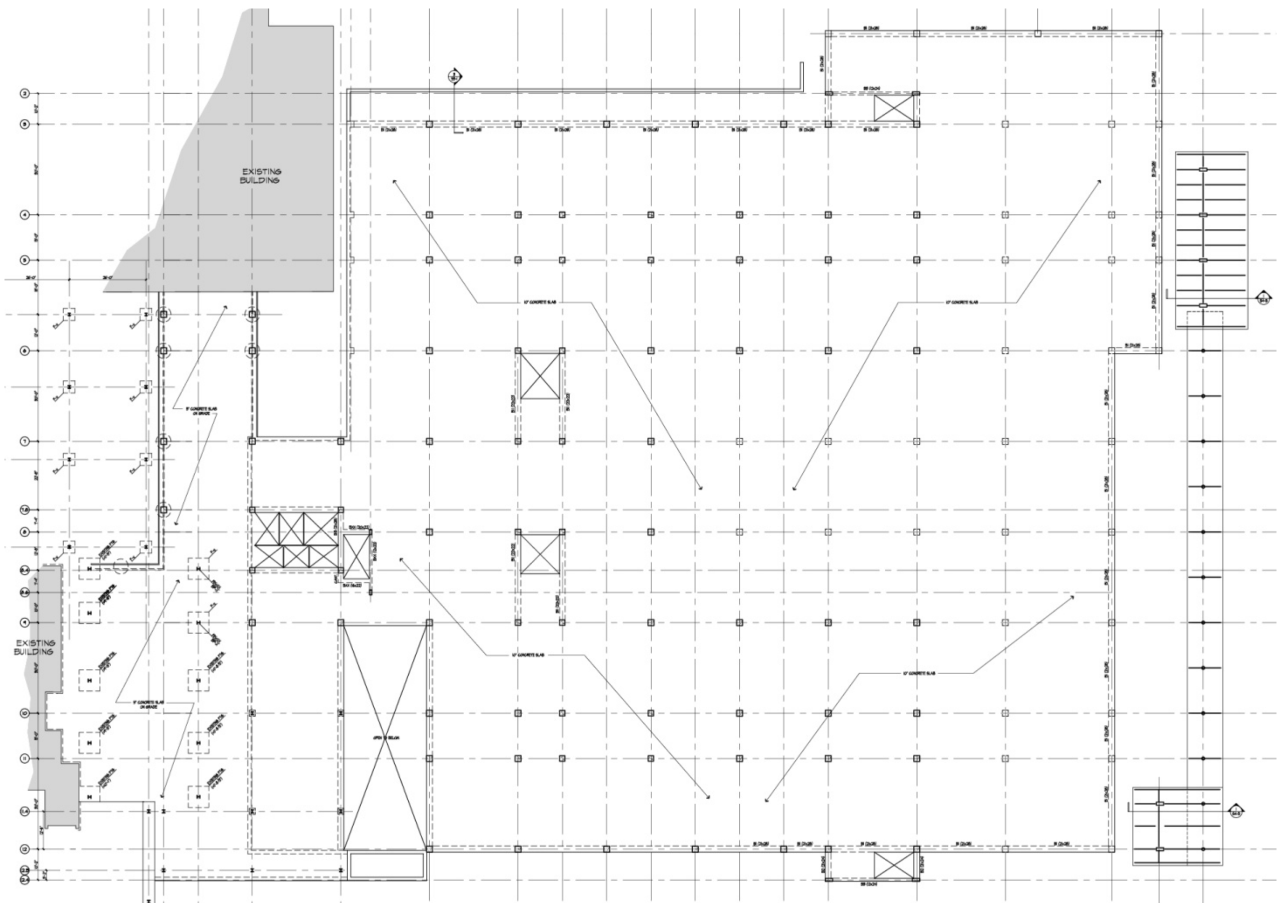
Along with serviceability and overturning spot checks, strength checks were conducted on a section of slab, perimeter edge beam, and column under the combination of dead, live and seismic loading. The slab and beam proved to be sufficient in two of the three critical moment sections while one the sections in each was overloaded. It had been concluded that the analysis method used in this report to determine dead and live load moments in the slabs and beams differs from the more advanced moment distribution method used by the designer. Had such a method been used, the slab and beam would have proven adequate. A column was checked on the ground level and level 4 and found to be adequate on the 4th floor for both seismic loading in both directions while the column on the ground floor was found to be adequate in seismic loading from only one direction. This is due to the differences in seismic load applied to the structure in this report from the designer as the roof was included as a level in load calculation while the designer most likely omitted this level.

Appendix

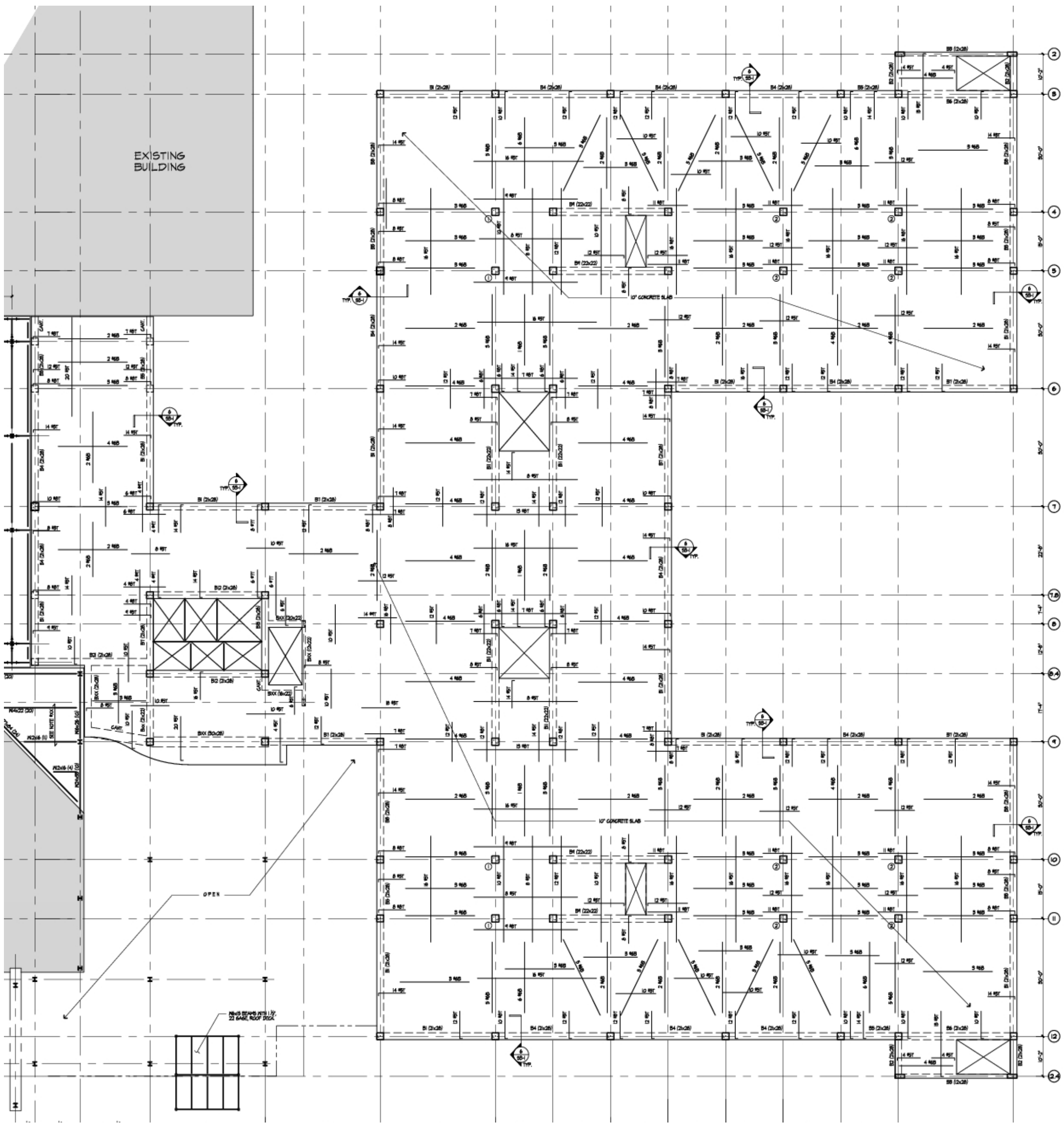
Appendix A: Typical Floor Plans



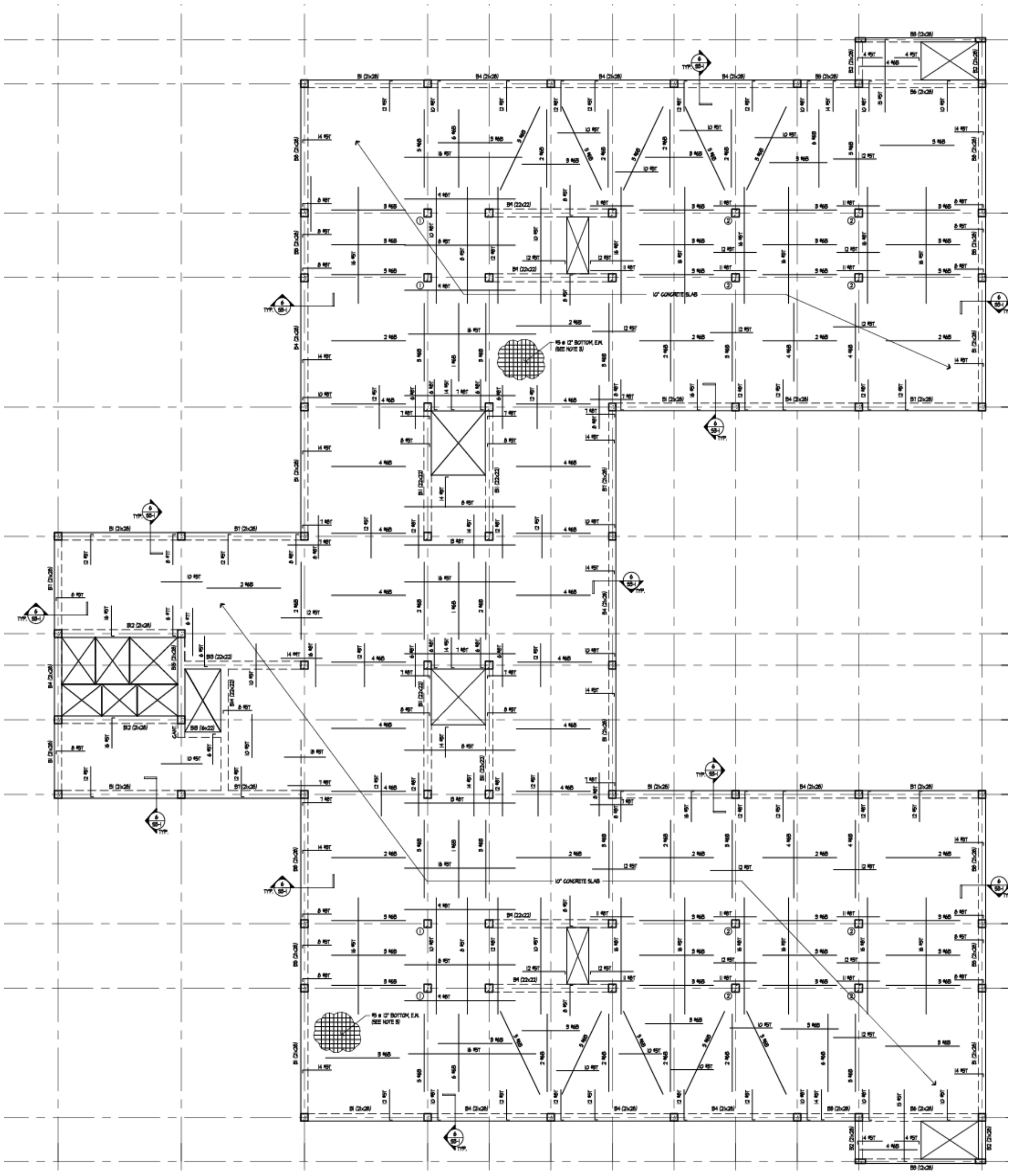
Ground Level



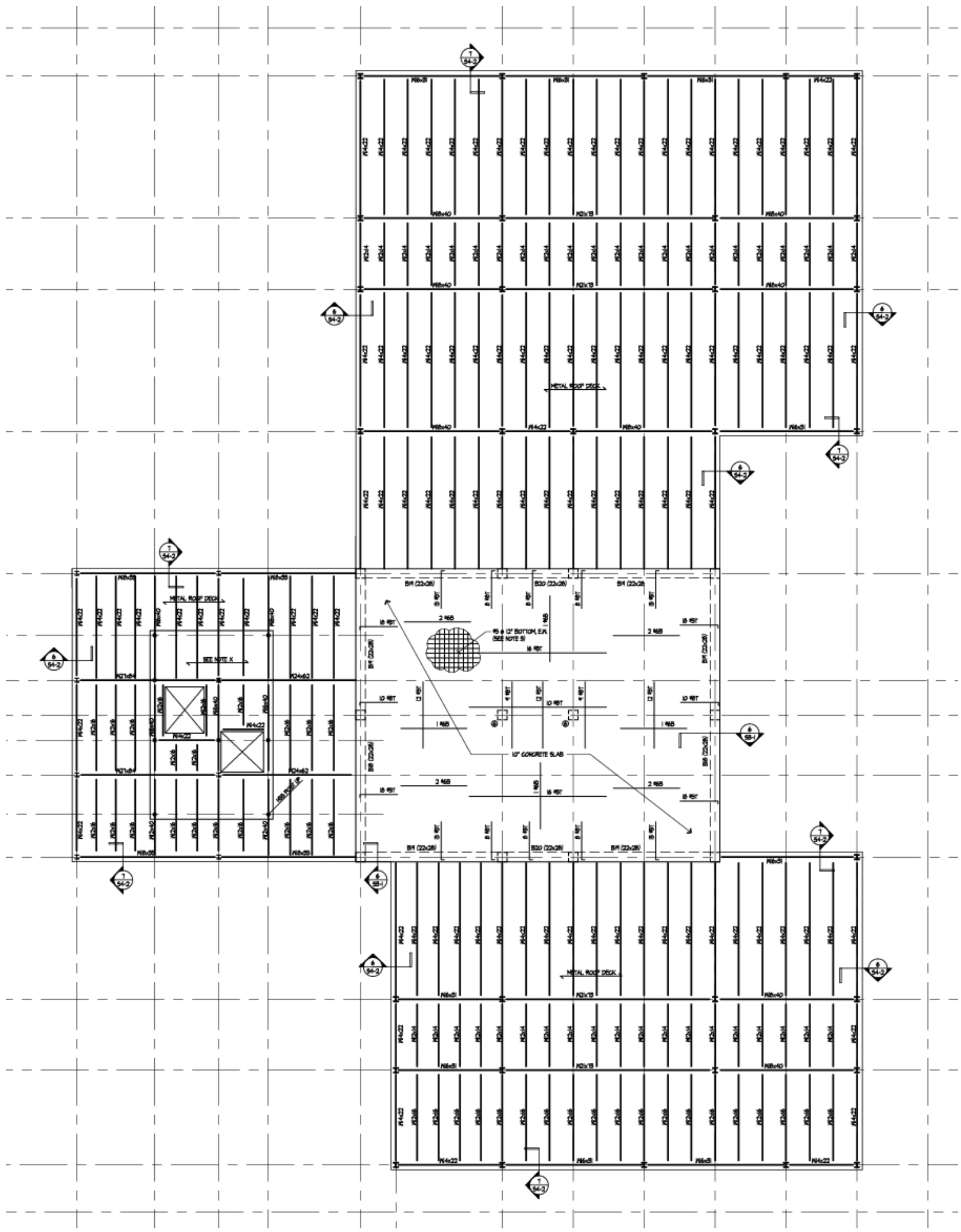
Level 1



Level 2 (Level 3 similar)



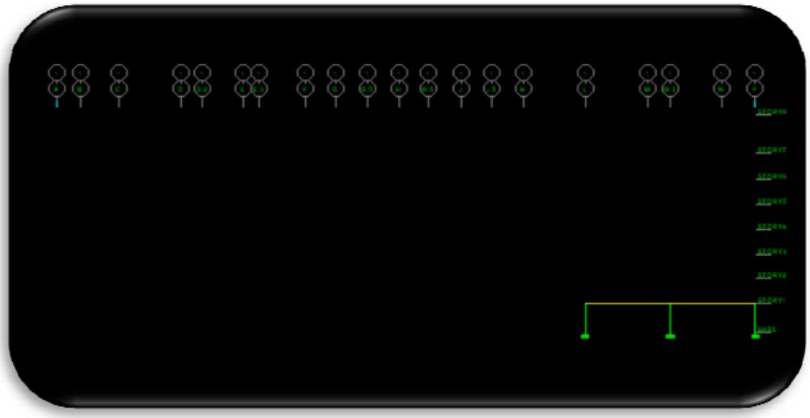
Level 4-7 (all similar)



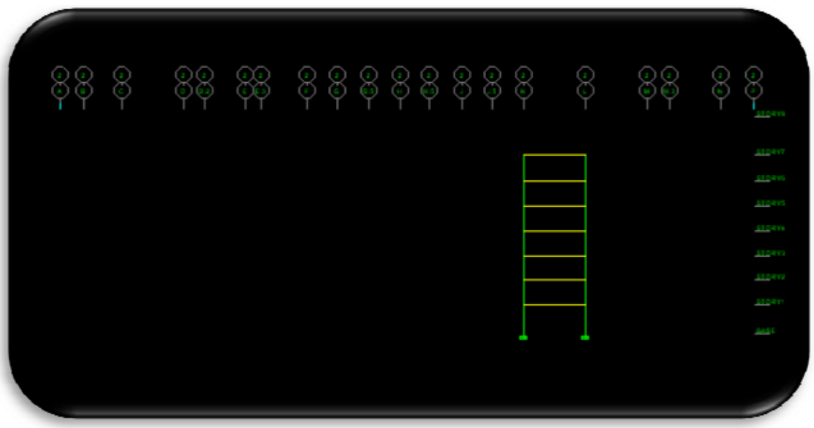
Roof Level

Appendix B: Lateral Frame Elevations

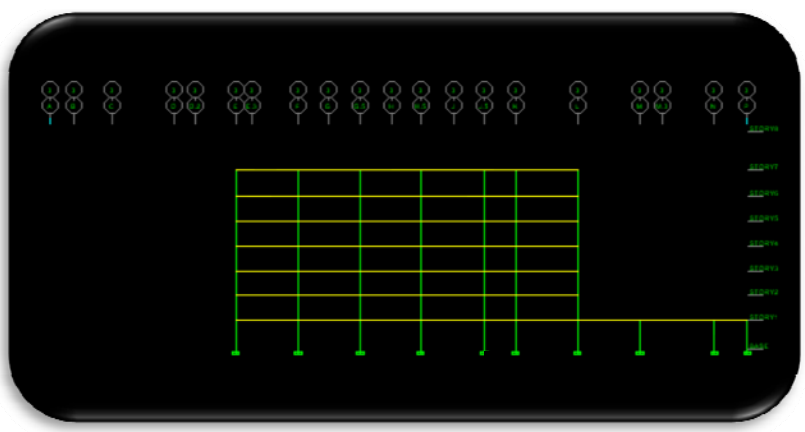
Due to the number of moment frames, 26 in total, frames that looked similar are grouped together below to save space and paper. It should be noted that while two or more frames may look similar, they each have unique column and/or beam elements causing them to respond differently when loaded.



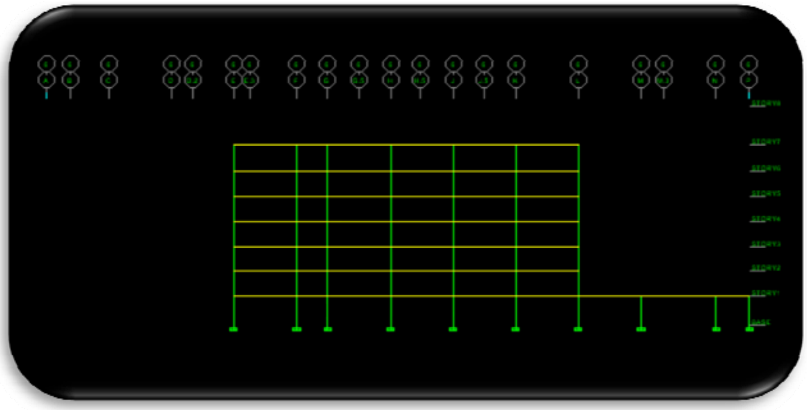
Frame 1



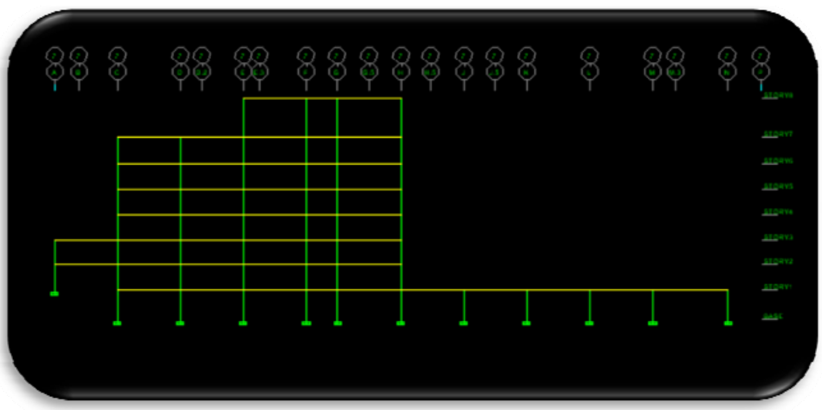
Frame 2 and Frame 12.4



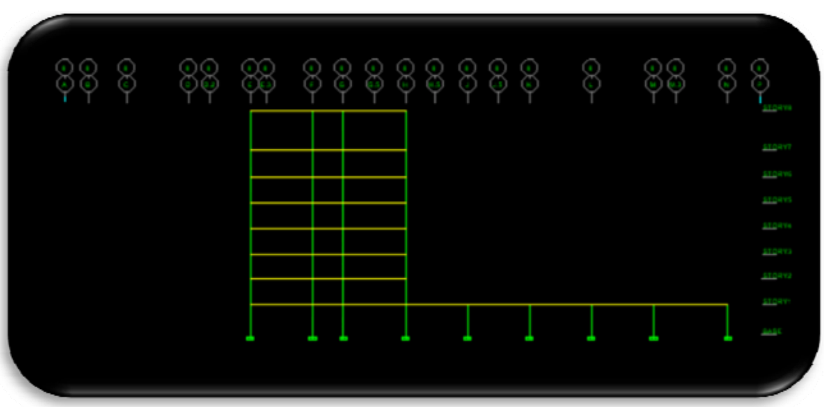
Frame 3



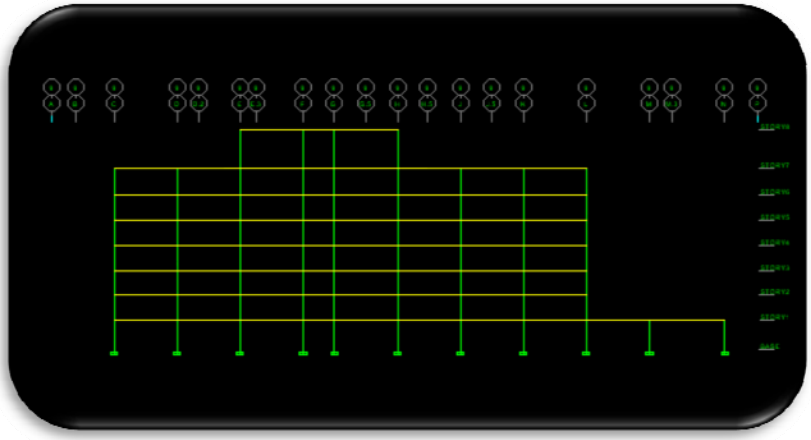
Frame 4, Frame 5, and Frame 6



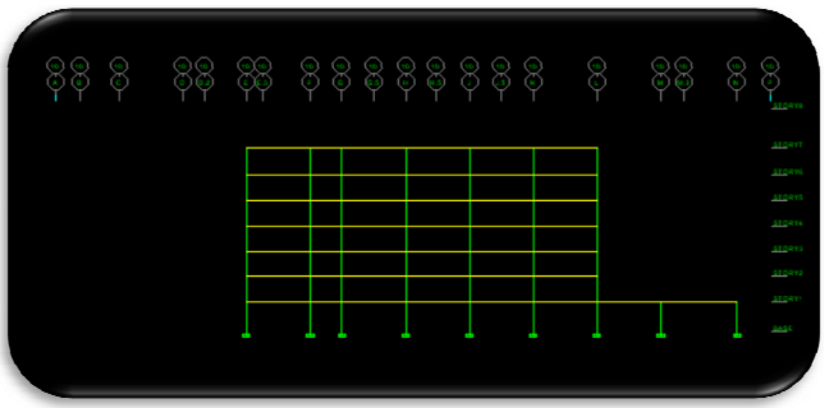
Frame 7



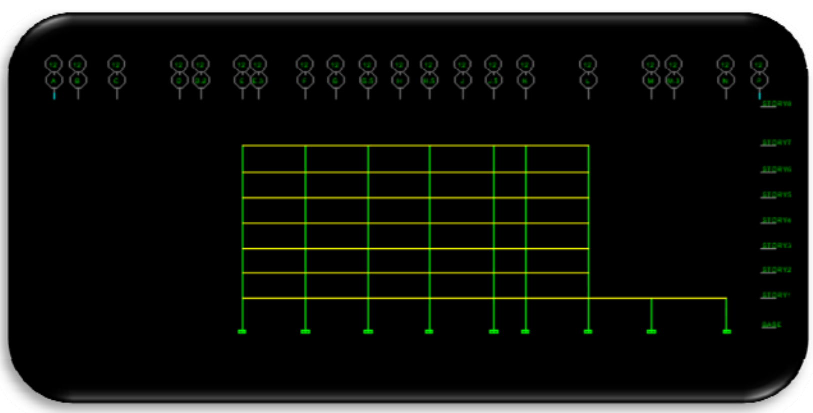
Frame 8



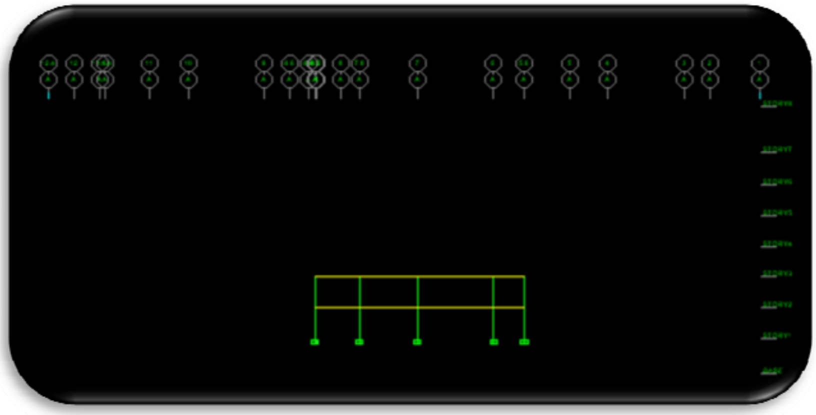
Frame 9



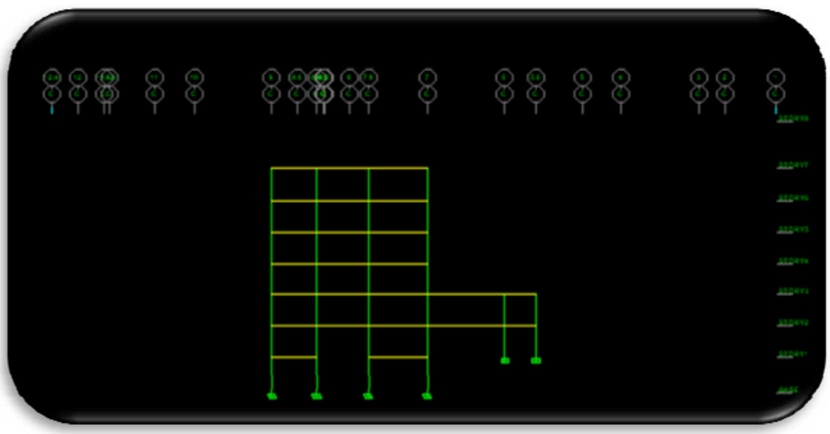
Frame 10, Frame 11



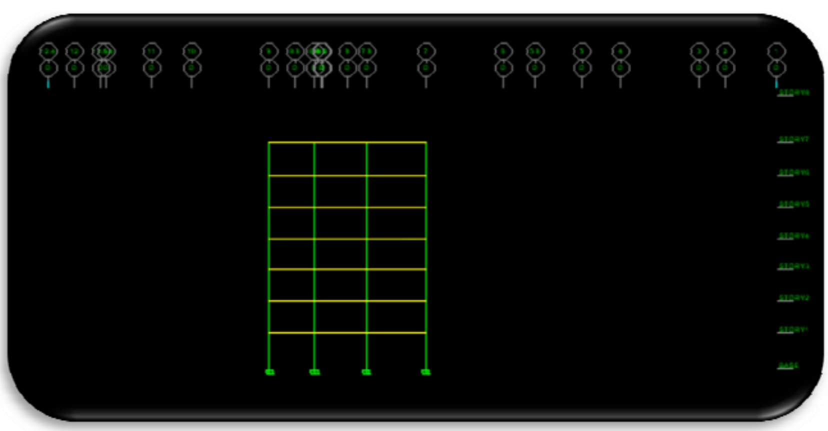
Frame 12



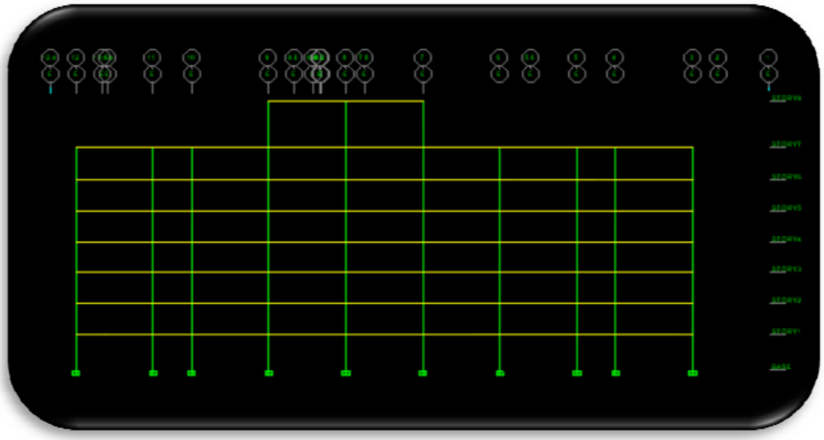
Frame A



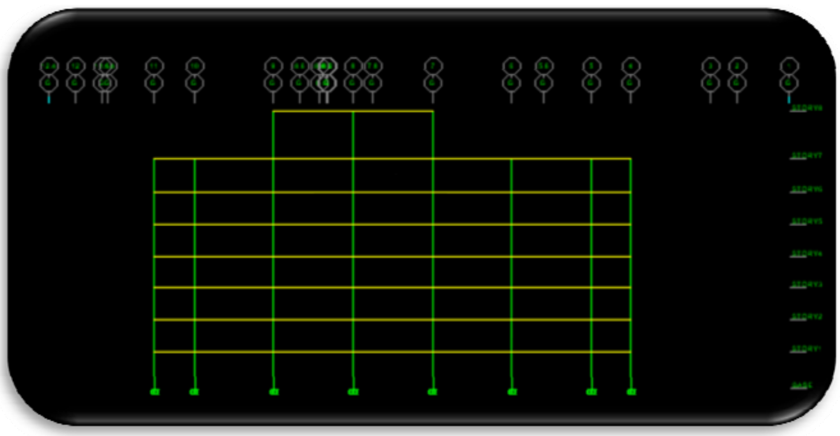
Frame C



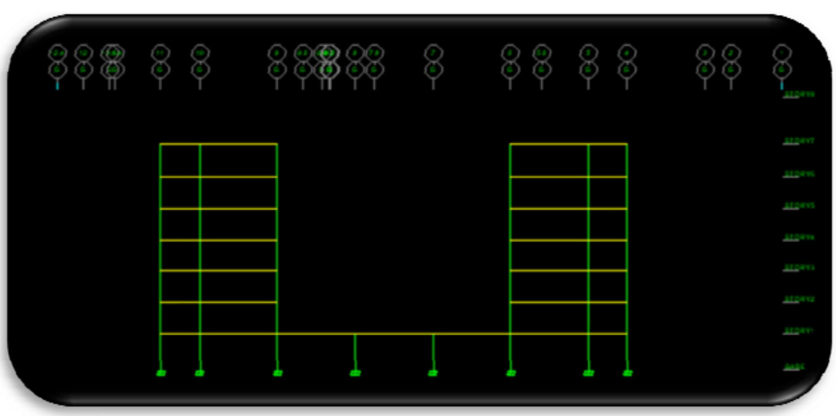
Frame D



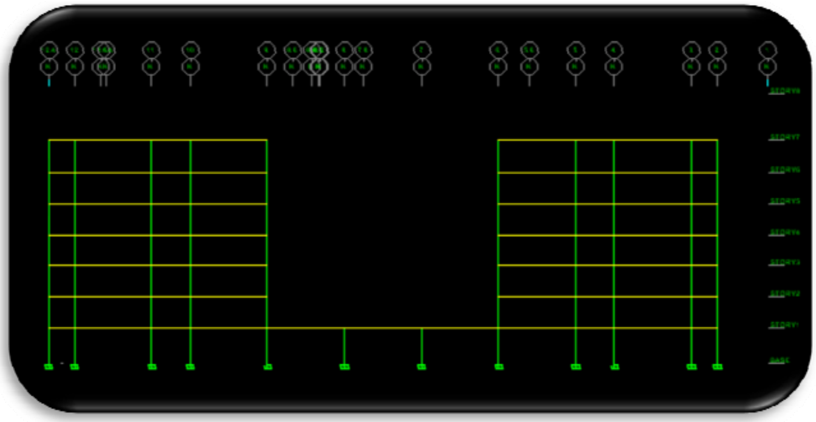
Frame E and Frame F



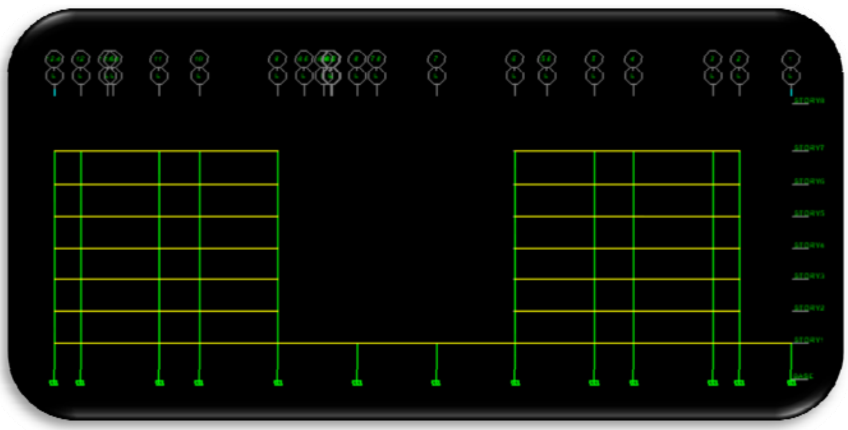
Frame G and Frame H



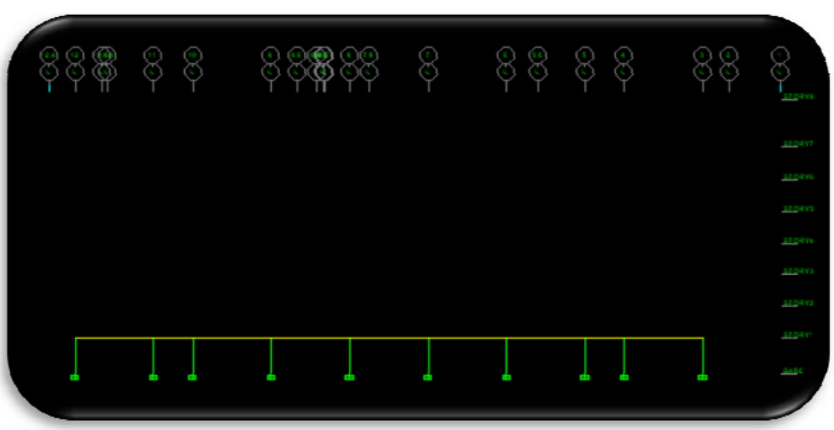
Frame J



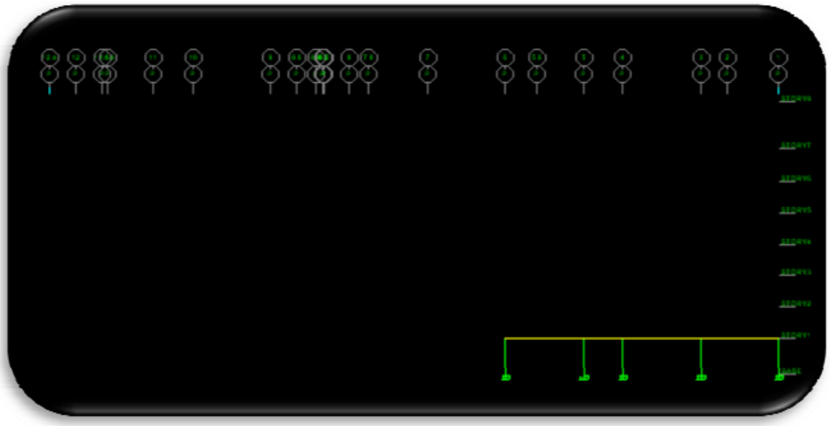
Frame K



Frame L



Frame M and Frame N



Frame P

Appendix C: Wind Analysis

WIND LOAD CALCULATION

BASIC WIND SPEED = 90 MPH → FIG. 6-1

$K_d = 0.85$ → BUILDING → TABLE 6-4

CATEGORY IV → $I = 1.15$ → TABLE 1604.5 AND TABLE 6-1

EXPOSURE CATEGORY B → 6.5.6

TOPOGRAPHIC FACTOR $K_{zt} = 1.0$

VELOCITY PRESSURE EXPOSURE COEFFICIENTS

HEIGHT OF BUILDING = 105' (TOP OF SLAB PENTHOUSE ROOF)

HEIGHT OF PARAPET = 1'

106 ft

LINEAR INTERPOLATION ⇒ $K_h = 1.005$ → TABLE 6-3

VELOCITY PRESSURE AT PARAPETS : $q_p = 0.00256(1.005)(1.0)(0.85)(90)^2(1.15)$
$= 20.37$ PSF

COMBINED NET PRESSURE COEFFICIENT $GC_{pn} = 1.5$ WINDWARD PARAPET
$GC_{pn} = -1.0$ LEEWARD PARAPET

COMBINED NET PRESSURE ON PARAPET

$P_p = 20.37(1.5) = 30.56$ PSF WINDWARD PARAPET

$= 20.37(-1.0) = -20.37$ PSF LEEWARD PARAPET

FORCES ON PARAPET $F = 30.56(1') = 30.56$ PLF WINDWARD

$F = -20.37(1') = -20.37$ PLF LEEWARD

BUILDING RIGID OR FLEXIBLE

CONCRETE MOMENT RESISTING FRAME $n_1 = 43.5/H^{0.9} = 43.5/(106)^{0.9} = 0.6542$ Hz

$0.6542 < 1.0$ Hz → BUILDING IS FLEXIBLE

$g_Q = g_v = 3.4$ $\quad g_R = \sqrt{2\ln(3{,}600(0.6542))} + \dfrac{0.577}{\sqrt{2\ln(3{,}600(0.6592))}} = 4.09$

$\bar{z} = 0.6h = 0.6(106) = 63.6 > z_{min} = 30$ ✓

$I_{\bar{z}} = 0.3\left(\dfrac{33}{63.6}\right)^{1/6} = 0.269$

$L_{\bar{z}} = 320\left(\dfrac{63.6}{33}\right)^{1/3} = 398.23$

N-S: $B = 225'$ ↓ $Q = 0.80$

E-W: $B = 260'$ ↓ $Q = 0.79$

$\bar{V}_{\bar{z}} = 0.45\left(\dfrac{63.6}{33}\right)^{1/4}(90)\left(\dfrac{88}{60}\right) = 70.0$

WIND LOAD CALCULATION CONTINUED

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.6542(348.23)}{70.0} = 3.72$$

$$R_n = \frac{7.47(3.72)}{(1+10.3(3.72))^{5/3}} = 0.061$$

$$n = 4.6(0.6542)(106)/70 = 4.56 > 0$$

$$R_h = \frac{1}{4.56} - \frac{1}{2(4.56)^2}\left(1 - e^{-2(4.56)}\right) = 0.195$$

$n = 4.6(0.6542)B/70$

N-S ↓ $n = 9.67$

E-W ↓ $n = 11.18$

N-S

$$R_B = \frac{1}{9.67} - \frac{1}{2(9.67)^2}\left(1 - e^{-2(9.67)}\right) = 0.098$$

E-W

$$R_B = \frac{1}{11.18} - \frac{1}{2(11.18)^2}\left(1 - e^{-2(11.18)}\right) = 0.085$$

N-S ↓ $L = 260'$

E-W ↓ $L = 225'$

N-S: $n = 15.4(0.6542)(260)/70 = 37.42$

E-W: $n = 15.4(0.6542)(225)/70 = 32.38$

$$\text{N-S: } R_L = \frac{1}{37.42} - \frac{1}{2(37.42)^2}\left(1 - e^{-2(37.42)}\right) = 0.026$$

$$\text{E-W: } R_L = \frac{1}{32.38} - \frac{1}{2(32.38)^2}\left(1 - e^{-2(32.38)}\right) = 0.030$$

β = DAMPING COEFFICIENT = 1.5% = 0.015

$$\text{N-S: } R = \sqrt{\frac{1}{0.015}(0.061)(0.195)(0.098)(0.53 + 0.47(0.026))} = 0.205$$

$$\text{E-W: } R = \sqrt{\frac{1}{0.015}(0.061)(0.195)(0.085)(0.53 + 0.47(0.030))} = 0.192$$

$$\text{N-S: } G_f = 0.925\left[\frac{1 + 1.7(0.269)\sqrt{(3.4)^2(0.80)^2 + (4.09)^2(0.205)^2}}{1 + 1.7(3.4)(0.269)}\right] = 0.833$$

$$\text{E-W: } G_f = 0.925\left[\frac{1 + 1.7(0.269)\sqrt{(3.4)^2(0.79)^2 + (4.09)^2(0.192)^2}}{1 + 1.7(3.4)(0.269)}\right] = 0.825$$

WIND LOAD CALCULATION CONTINUED

SEE EXCEL TABLES FOR q_z and q_h VELOCITY PRESSURE

DETERMINE PRESSURE COEFFICIENTS C_p FOR WALLS AND ROOF

FOR WIND IN THE NORTH-SOUTH DIRECTION

WINDWARD WALL: $C_p = 0.8$ FOR USE WITH q_z

LEEWARD WALL $(L/B) = 260/225 = 1.156$ $C_p = -0.47$ FOR USE WITH q_h

SIDE WALL: $C_p = -0.7$ FOR USE WITH q_h

FOR WIND IN THE EAST-WEST DIRECTION

WINDWARD WALL: $C_p = 0.8$ FOR USE WITH q_z

LEEWARD WALL $(L/B) = 225/260 = 0.865$ $C_p = -0.5$ FOR USE WITH q_h

SIDE WALL: $C_p = -0.7$ FOR USE WITH q_h

$q_i = q_h = q_p = 20.37$

$GC_{pi} = \pm 0.18$

DESIGN WIND PRESSURES p_z AND p_h

WINDWARD WALLS: $p_z = q_z G_f C_p - q_h (GC_{pi})$

N-S: $p_z = q_z (0.833)(0.8) - 20.37(\pm 0.18) = 0.666 q_z \pm 3.67$ PSF

E-W: $p_z = q_z (0.825)(0.8) - 20.37(\pm 0.18) = 0.660 q_z \pm 3.67$ PSF

LEEWARD AND SIDE WALLS $p_h = q_h G_f C_p - q_h GC_{pi}$

N-S: $p_h = (20.37)(0.833)C_p - 20.37(\pm 0.18) = 16.97 C_p \pm 3.67$ PSF

E-W: $p_h = (20.37)(0.825)C_p - 20.37(\pm 0.18) = 16.81 C_p \pm 3.67$

Appendix D: Seismic Analysis

SEISMIC LOAD CALCULATION

DETERMINE SEISMIC GROUND MOTION VALUES

FROM USGS EARTHQUAK HAZARDS PROGRAM: ZIP: 21237

$\hookrightarrow S_s = 0.176\ g$

$S_1 = 0.051\ g$

SITE CLASS C

TABLE 11.4-1 → $F_a = 1.2$

TABLE 11.4-2 → $F_v = 1.7$

$S_{MS} = F_a S_S = 1.2(0.176) = 0.2112$

$S_{M1} = F_v S_1 = 1.7(0.051) = 0.0867$

$S_{DS} = 2 S_{MS}/3 = 2(0.2112)/3 = 0.1408$

$S_{D1} = 2 S_{M1}/3 \quad 2(0.0867)/3 = 0.0578$

DETERMINE SDC

OCCUPANCY CATEGORY IV

$\hookrightarrow$ SDC = A

$T_S = S_{D1}/S_{DS} = 0.0578/0.1408 = 0.4105$

$0.8T_S = 0.3284$ TABLE 12.8.2 → $C_t = 0.016$ $x = 0.9$

$T_a = C_t h_n^x$ $h_n = 106$ ft $T_a = 0.016(105)^{0.9} = 1.0548 > 0.8T_S = 0.3284$

$\hookrightarrow$ TABLE 11.6-1 & 11.6-2

TABLE 11.6-1 → A

TABLE 11.6-2 → A SDC = A

$3.5T_S = 3.5(0.4105) = 1.4368 > T = 1.0548$ → EQUIVALENT LATERAL FORCE PROCD.

TABLE 12.2-1 → R = 3 I = 1.5 → TABLE 11.5-1

TABLE 12.8-1 → $C_u = 1.7$ $C_u T_a = 1.7(1.0548) = 1.793 = T$

$C_S = \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.1408}{\left(\frac{3}{1.5}\right)} = 0.0704$ $T_L = 8 > T$ $C_S \le \dfrac{0.0578}{1.793\left(\frac{3}{1.5}\right)} = 0.016$

$\therefore\ \underline{C_S = 0.016}$

$V = C_S W = 0.016(55713) = \underline{\underline{891.4^k}}$

$k = 0.75 + 0.5T = 1.49$

$F_x = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$ → SEE SPREAD SHEATS

Weight of Concrete Floor Slabs				
Level	**Area (ft^2)**	**Slab Thickness (in)**	**Weight (pcf)**	**Weight (k)**
Roof	4,789	10	145	579
Penthouse	35,373	12	145	5129
Level 6	35,373	10	145	4274
Level 5	35,373	10	145	4274
Level 4	35,373	10	145	4274
Level 3	38,096	10	145	4603
Level 2	38,607	10	145	4665
Level 1	68,710	10	145	8302
Ground	-	-		-

Weight of Conc. on Metal Deck Floor Slabs			
Level	**Area (ft^2)**	**Weight (psf)**	**Weight (k)**
Roof	20,680	63	1303
Penthouse	-	-	-
Level 6	-	-	-
Level 5	-	-	-
Level 4	2,886	63	182
Level 3	3,548	63	224
Level 2	-	-	-
Level 1	-	-	-
Ground	-	-	-

Weight of Concrete Beams

Level	Total Length of 10"x28" (ft)	Total Length of 12"x28" (ft)	Total Length of 21"x28" (ft)	Total Length of 22"x24" (ft)	Total Length of 8"x20" (ft)	Total Length of 22"x20" (ft)	Total Length of 24"x20" (ft)	Total Length of 26"x20" (ft)	∑ Area (ft^3) (minus slab depth)	Weight (pcf)	Weight (k)
Roof	0	0	0	252.5	0	0	0	0	540	145	78
Penthouse	18.25	90.17	1076.5	0	0	43.83	28.25	254.25	3111	145	451
Level 6	18.25	90.17	1076.5	0	18.25	28.25	0	254.25	3496	145	507
Level 5	74.75	33.67	1076.5	0	18.25	28.25	0	254.25	3482	145	505
Level 4	74.75	33.67	1076.5	0	18.25	0	28.25	254.25	3486	145	505
Level 3	74.75	33.67	1195.1	0	18.25	0	28.25	254.25	3797	145	551
Level 2	18.25	90.17	1213.3	0	0	0	0	206	3715	145	539
Level 1	0	97.18	758.5	0	18.25	194.5	0	72.33	2575	145	373
Ground	-	-	-	-		-	-	-	-	-	-

Weight of Steel Beams																				
Level	**Total Length of W12x14 (ft)**	**Total Length of W12x16 (ft)**	**Total Length of W14x22 (ft)**	**Total Length of W16x26 (ft)**	**Total Length of W16x31 (ft)**	**Total Length of W18x35 (ft)**	**Total Length of W16x40 (ft)**	**Total Length of W18x40 (ft)**	**Total Length of W21x44 (ft)**	**Total Length of W18x50 (ft)**	**Total Length of W24x55 (ft)**	**Total Length of W21x57 (ft)**	**Total Length of W21x62 (ft)**	**Total Length of W24x62 (ft)**	**Total Length of W18x65 (ft)**	**Total Length of W21x73 (ft)**	**Total Length of W30x90 (ft)**	**Total Length of W18x97 (ft)**	**Total Length of W24x103 (ft)**	**Weight (k)**
Roof	891	571	2488	15	158	-	60	75	90	180	-	30	225	60	-	180	60	-	-	138
Penthouse	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Level 6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Level 5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Level 4	-	-	870	-	-	150	-	-	-	-	-	-	-	-	-	-	-	45	-	29
Level 3	-	195	120	185	-	-	-	-	20	185	20	-	-	-	35	-	-	-	40	28
Level 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Level 1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ground	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Weight of Concrete Columns									
Level	**# of 21x21**	**# of 22x22**	**# of 30x12**	**# of 12x20**	**# of 21x27**	**∑ Area (ft^2)**	**Height (ft)**	**Weight (pcf)**	**Weight (k)**
Roof	-	-	-	-	-	-	-	-	-
Penthouse	12	0	4	0	0	46.75	16.67	145	9
Level 6	43	29	4	0	0	239.16	12.00	145	35
Level 5	43	29	4	0	0	239.16	11.17	145	32
Level 4	43	29	4	0	0	239.16	11.17	145	32
Level 3	43	29	4	0	0	239.16	11.17	145	32
Level 2	50	29	4	0	0	260.60	11.17	145	35
Level 1	50	29	4	4	0	267.26	11.17	145	36
Ground	62	47	4	4	1	368.45	13.17	145	59

Weight of Steel Columns				
Level	**# of W10x49**	**∑ W (k/ft)**	**Height (ft)**	**Weight (k)**
Roof	-	-	-	-
Penthouse	68	3.33	17.50	58
Level 6	68	3.33	13.00	43
Level 5	-	-	-	-
Level 4	-	-	-	-
Level 3	-	-	-	-
Level 2	-	-	-	-
Level 1	-	-	-	-
Ground	-	-	-	-

Weight of Façade						
Level	**Perimeter (ft)**	**Tributary Height (ft)**	**55% of Area (ft^2)**	**Thickness (in)**	**Wight (pcf)**	**Weight (k)**
Roof	802	8.75	7,018	7	30	123
Penthouse	802	15.25	6,727	7	80	314
Level 6	1140	12.5	7,838	7	145	663
Level 5	1140	12	7,524	7	145	636
Level 4	1140	12	7,524	7	145	636
Level 3	1140	12	7,524	7	145	636
Level 2	1140	12	7,524	7	145	636
Level 1	1140	13	8,151	7	145	689
Ground	-		-	-		-

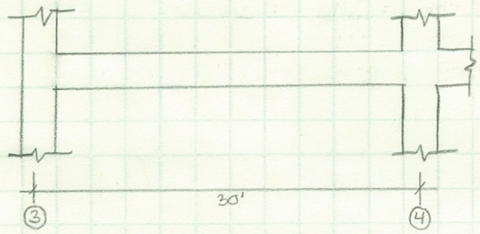
Weight of Curtain Wall					
Level	**Perimeter (ft)**	**Tributary Height (ft)**	**45% of Area (ft^2)**	**Wight (psf)**	**Weight (k)**
Roof	-	-	-	-	-
Penthouse	802	12.5	4,511	3	14
Level 6	1140	12	6,156	3	18
Level 5	1140	12	6,156	3	18
Level 4	1140	12	6,156	3	18
Level 3	1140	12	6,156	3	18
Level 2	1140	12	6,156	3	18
Level 1	1140	13	6,669	3	20
Ground	-		-		-

Superimposed DL			
Level	**Area (ft^2)**	**Superimposed DL (psf)**	**Weight (k)**
Roof	-	-	-
Penthouse	25,469	20	509
Level 6	35,373	20	707
Level 5	35,373	20	707
Level 4	35,373	20	707
Level 3	38,096	20	762
Level 2	38,607	20	772
Level 1	38,607	20	772
Ground	-	-	-

Weight of Partitions			
Level	**Area (ft^2)**	**Partition Load (psf)**	**Weight (k)**
Roof	-	-	-
Penthouse	-	-	-
Level 6	35,373	20	707
Level 5	35,373	20	707
Level 4	35,373	20	707
Level 3	38,096	20	762
Level 2	38,607	20	772
Level 1	38,607	20	772
Ground	-	-	-

Appendix E: Lateral System Check Hand Calculations

SLAB MOMENT CHECK   LINE F BETWEEN 3 AND 4   4th LEVEL



LOAD COMBINATION $1.2D + 1.0E + L$

SLAB DEAD LOAD MOMENTS: $w_D = 121 + 20 = 141$ PSF $w_L = 80$ PSF

$w_U = 1.2(141) + 1.0(80) = 249.2$

$M_o = \frac{1}{8}(0.249)(22.5)(28.25)^2 = 559^{\prime k}$

% M_o TO CS SAME AS TECH 1:   % M_o TO CS $M^-_{ext} = 88\%$

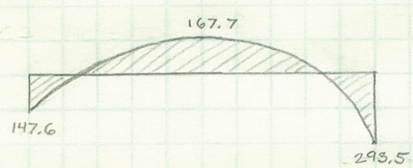
% M_o TO CS $M^+ = 60\%$

% M_o TO CS $M^-_{int} = 75\%$

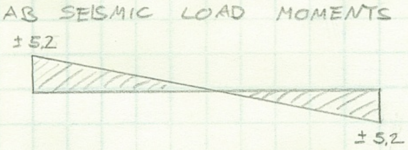
$M^-_{ext} = 0.30(559)(0.88) = 147.6^{\prime k}$

$M^+ = 0.50(559)(0.60) = 167.7^{\prime k}$

$M^-_{int} = 0.70(559)(0.75) = 293.5^{\prime k}$



SLAB SEISMIC LOAD MOMENTS



WORST CASE DESIGN MOMENTS

$M^-_{ext} = -152.8^{\prime k}$

$M^+ = 167.7^{\prime k}$

$M^-_{int} = -298.7^{\prime k}$

COLUMN STRIP REINFORCMENT:

COLUMN STRIP WIDTH = 135"

NEGATIVE MOMENT REINFORCEMENT: (10) #8 TOP → $d = 10 - 3/4" - 1.00"/2 = 8.75"$

POSITIVE MOMENT REINFORCEMENT: (5) #6 BOTTOM → $d = 10 - 3/4" - 0.75"/2 = 8.88"$

#5 @ 12" o.c. ⇒ 11 #5 BARS IN COLUMN STRIP → $d = 10 - 3/4 - 0.75" - 0.625/2$
$= 8.19"$

$f'_c = 5{,}000$ psi $\quad f_y = 60{,}000$ psi

NEGATIVE MOMENT CAPACITY:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{7.9(60)}{0.85(5)(135)} = 0.826" \qquad c = 0.826/0.8 = 1.03"$$

$$\varepsilon_s = \frac{0.003}{1.03}(8.75 - 1.03) = 0.022 \geq 0.005 \quad \therefore \phi = 0.9$$

$$\phi M_n = 0.9 A_s f_y (d - a/2) = 0.9(7.9)(60)(8.75 - 0.826/2) = 3556.6 \text{ k·in} = 296.4 \text{ 'k}$$

$\phi M_n > M^-_{ext} = 153.1$ 'k $\quad$ OK ✓

$\phi M_n < M^-_{int} = 298.7$ 'k $\quad$ NOT OK ✗

POSITIVE MOMENT CAPACITY:

$$a = \frac{5.61(60)}{0.85(5)(135)} = 0.587" \qquad c = 0.587/0.8 = 0.734"$$

$$\varepsilon_s = \frac{0.003}{0.734}(8.88 - 0.734) = 0.033 \geq 0.005 \quad \therefore \phi = 0.9$$

$$\phi M_n = 0.9\left[(2.2)(60)(8.88 - 0.587/2) + (3.41)(60)(8.19 - 0.587/2)\right]$$

$$= 2474 \text{ in·k} = 206 \text{ 'k}$$

$\phi M_n > M^+ = 167.7$ $\quad$ OK ✓

BEAM MOMENT CHECK LINE (E) BETWEEN (3) AND (4) 4th LEVEL

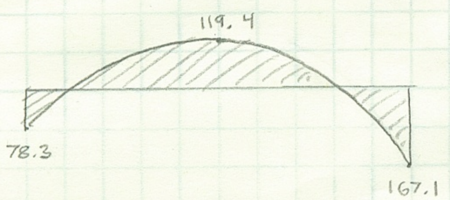
LOAD COMBINATION 1.2D + 1.0E + L

$M_o = \frac{1}{8}(0.249)(15)(28.25)^2 = 372.6^{'k}$

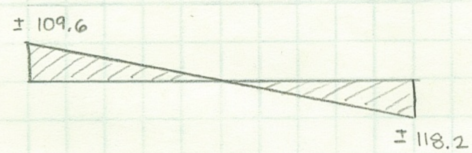
% M_o TO CS BEAM SAME AS TECH 1:

% M_o TO CS BEAM M^-_{ext} = 70%
% M_o TO CS BEAM M^+ = 64%
% M_o TO CS BEAM M^-_{int} = 64%

$M^-_{ext} = 0.30(373)(0.70) = 78.3^{'k}$
$M^+ = 0.50(373)(0.64) = 119.4^{'k}$
$M^-_{int} = 0.70(373)(0.64) = 167.1^{'k}$



SLAB SEISMIC LOAD MOMENTS



WORST CASE DESIGN MOMENTS

$M^-_{ext} = -187.9^{'k}$

$M^+ = 119.4^{'k}$

$M^-_{int} = 285.3^{'k}$

BEAM REINFORCMENT:

BEAM WIDTH = 21"

NEGATIVE MOMENT REINFORCMENT: (2) #9 TOP → $d = 28 - 1.5'' - \frac{1.128''}{2} = 25.9''$

POSITIVE MOMENT REINFORCMENT: (3) #9 BOTTOM → $d = 28 - 1.5'' - \frac{1.128''}{2} = 25.9''$

$f'_c = 5000$ psi $\quad f_y = 60{,}000$ psi

NEGATIVE MOMENT CAPACITY:

$a = \frac{2.0(60)}{0.85(5)(21)} = 1.345'' \qquad c = 1.345/0.8 = 1.68''$

$\varepsilon_s = \frac{0.003}{1.68}(25.9 - 1.68) = 0.043 \geq 0.005 \therefore \phi = 0.9$

$\phi M_n = 0.9(2)(60)(25.9 - 1.345/2) = 2725 \text{ ink} = 227 \text{ ftk}$

$\phi M_n > M^-_{ext} = 187.9$ 'k $\underline{\text{OK}}$ ✓

$\phi M_n < M^-_{int} = 285.3$ 'k $\underline{\text{NOT OK}}$ ✗

POSITIVE MOMENT CAPACITY

$a = \frac{3.0(60)}{0.85(5)(21)} = 2.017'' \qquad c = 2.017/0.8 = 2.52''$

$\varepsilon_s = \frac{0.003}{2.52}(25.9 - 2.52) = 0.028 \geq 0.05 \therefore \phi = 0.9$

$\phi M_n = 0.9(3)(60)(25.9 - 2.017/2) = 4032 \text{ ink} = 336 \text{ ftk}$

$\phi M_n > M^+ = 119.4$ 'k $\underline{\text{OK}}$ ✓

$\boxed{\text{COLUMN SPOT CHECK}}$ FRAME F LINE 4

COLUMN F4 FOURTH LEVEL

LOAD COMBINATION $1.2D + 1.0E + L$ (SEISMIC E-W CONTROLS)

$A_{trib} = 503 \text{ FT}^2/\text{LEVEL}$ $22'' \times 22''$ W/ (8) #9

$w_L = 80$ PSF

ROOF: $w_D = 80$ PSF $w_L = 30$ PSF
PENTHOUSE: $w_D = 165$ PSF $w_L = 80$ PSF
LEVEL 6: $w_D = 141$ PSF $w_L = 80$ PSF
LEVEL 5: $w_D = 141$ PSF $w_L = 80$ PSF

$f'_c = 5000$ PSI

$w_D = 80 + 165 + 141 + 141 = 527$ PSF

$$\text{SELF WEIGHT} = \frac{(22'')(22'')}{144^{in^2}/_{ft^2}} (12 + 13 + 17.5) \left(\frac{145 \text{ lb/ft}^3}{1000}\right) = 20.7^k$$

$$P_D = \frac{527(503)}{1000} + 20.7 = 285.8^k$$

LIVE LOAD REDUCTION $K_{LL} = 4$ $A_T = 503(4) = 2012$ FT

$$L_o = (30 + 80 + 80 + 80)(503) = 135^k$$

$$L = 136 \left[0.25 + \frac{15}{\sqrt{4(2012)}}\right] = 57^k = P_L$$

$P_E = 4.91^k$

$P_U = 1.2(285.8) + 1.0(4.91) + 1.0(57) = 409^k$

$M_U = 221'^k$

PCA COLUMN OUTPUT OK ✓

COLUMN F4 GROUND LEVEL

LOAD COMBINATION $1.2D + 1.0E + L$

$A_{trib} = 503\ ft^2/\text{LEVEL}$ $22'' \times 22''$ w/ (8) #9

$\omega_L = 80$ PSF

$f'_c = 7000$ PSI

ROOF: $\omega_D = 80$ PSF $\omega_L = 30$ PSF
PENTHOUSE: $\omega_D = 165$ PSF $\omega_L = 80$ PSF
LEVEL 6: $\omega_D = 141$ PSF $\omega_L = 80$ PSF
LEVEL 5: $\omega_D = 141$ PSF $\omega_L = 80$ PSF
LEVEL 4: $\omega_D = 141$ PSF $\omega_L = 80$ PSF
LEVEL 3: $\omega_D = 141$ PSF $\omega_L = 80$ PSF
LEVEL 2: $\omega_D = 141$ PSF $\omega_L = 80$ PSF
LEVEL 1: $\omega_D = 141$ PSF $\omega_L = 80$ PSF

$\omega_D = 80 + 165 + 141 + 141 + 141 + 141 + 141 + 141 = 1091$ PSF

$$\text{SELF WEIGHT} = \frac{(22)(22)}{144}(12+12+12+12+12+13+17.5)\left(\frac{145}{1000}\right) = 44.1^k$$

$$P_D = \frac{1091\,(503)}{1000} + 44.1 = 593^k$$

LIVE LOAD REDUCTION $K_{LL} = 4$ $A_T = 503(8) = 4024\ ft^2$

$$L_o = (30 + 80 + 80 + 80 + 80 + 80 + 80 + 80)(503) = 297^k$$

$$L = 297\left[0.25 + \frac{15}{\sqrt{4(4024)}}\right] = 109^k = P_L$$

$P_{E_1} = 9.24^k$ $P_{E_2} = -8.24^k$

$P_{u_1} = 1.2(593) + 1.0(9.24) + 1.0(109) = 829.8^k$ $P_{u_2} = 812.4^k$

$M_{E_1} = -202.3^{'k}$ $M_{E_2} = 845.9^{'k}$

SEISMIC NORTH-SOUTH OK ✓

SEISMIC EAST-WEST NOT OK ✗