Franklin Square Hospital Center Patient Tower

Baltimore, MD

Technical Report 1

Structural Concepts / Structural Existing Conditions Report

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Structural Option

AE 481W Senior Thesis

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Executive Summary:

The intent of this report is to analyze the existing structural conditions for the Franklin Square Hospital Center Patient Tower in Baltimore, MD. As a 7 story tower addition, the structure is subject to higher constraints than found in most buildings. It is subject to much higher loading from the numerous mechanical, electrical, telecommunication and distribution systems found only in hospitals. The following technical report describes the structural concepts and existing condition of the Franklin Square Hospital Center Patient Tower including information relative to design concepts and required loading. Three objectives were met with this technical report:

- Become familiar with the buildings structural design and understand how the gravity and lateral systems were designed and function together.
- Calculate the snow, wind, and seismic loads and understand how they affect the structure.
- Provide spot checks on gravity load supporting members for the comparison of methods or analysis used by the designers.

The report begins with a detailed introduction to the main structural systems including foundation, floor system, columns, roof system, wall system and lateral system. Details and sections are copied from the plans to help explain the systems along with written descriptions.

The snow load analysis followed ASCE 7-05 and produced roof snow loads of 30 psf. A snow drift analysis was also conducted finding drift loads as high as 90 psf on some portions of the roof. Wind analysis also followed ASCE 7-05. Windward wind pressures were found to range from 7 psf on the lower floors to 13.5 psf on the upper floors. Leeward wind pressures were calculated close to 8 psf. Seismic analysis once again followed ASCE 7-05 and produced a base shear of 891 kips.

Spot checks were conducted on a portion of the building that was fairly regular and typical throughout the plan. The flat plate floor system was found to be adequately designed at all critical section except two. The reason behind these differences stems from the method of analysis used for calculating critical moments. This report used the direct design method while the designer likely used a more advanced and more accurate method. Columns were checked for pure axial strength at both the fourth floor and ground floor and found to be adequate.

Other important parameters such as gravity loading, codes used, and material strength are addressed throughout this report. Complete hand calculations are also included in the Appendix.

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."

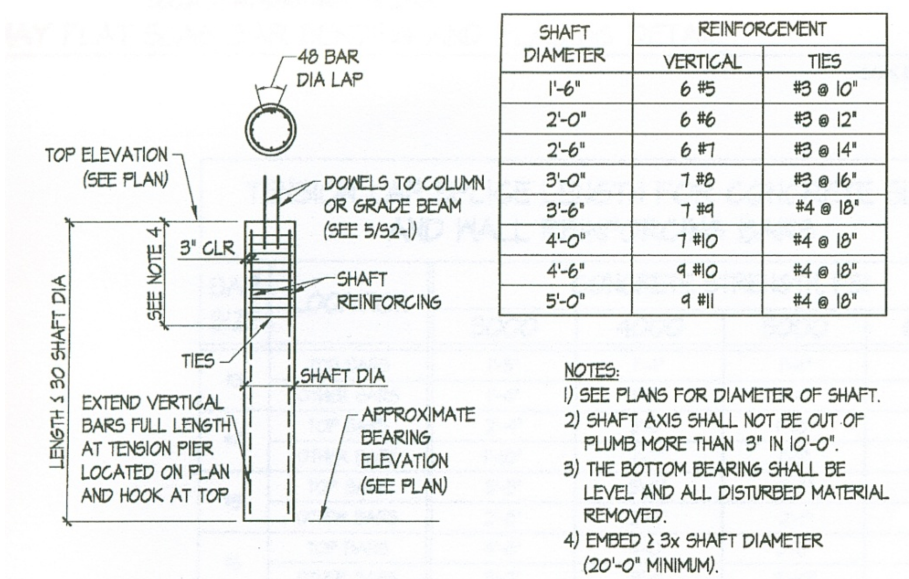


Figure 1: Drilled Pier Reinforcing

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."

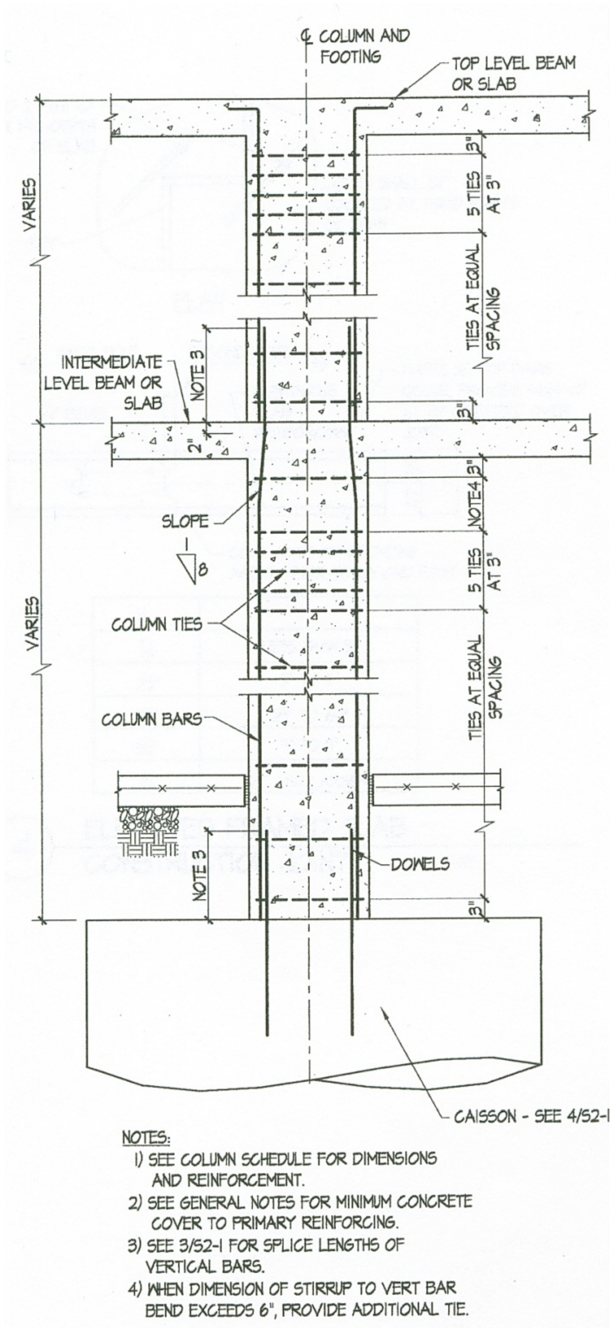


Figure 2: Typical Column Caisson Connection and Column Reinforcing

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.”

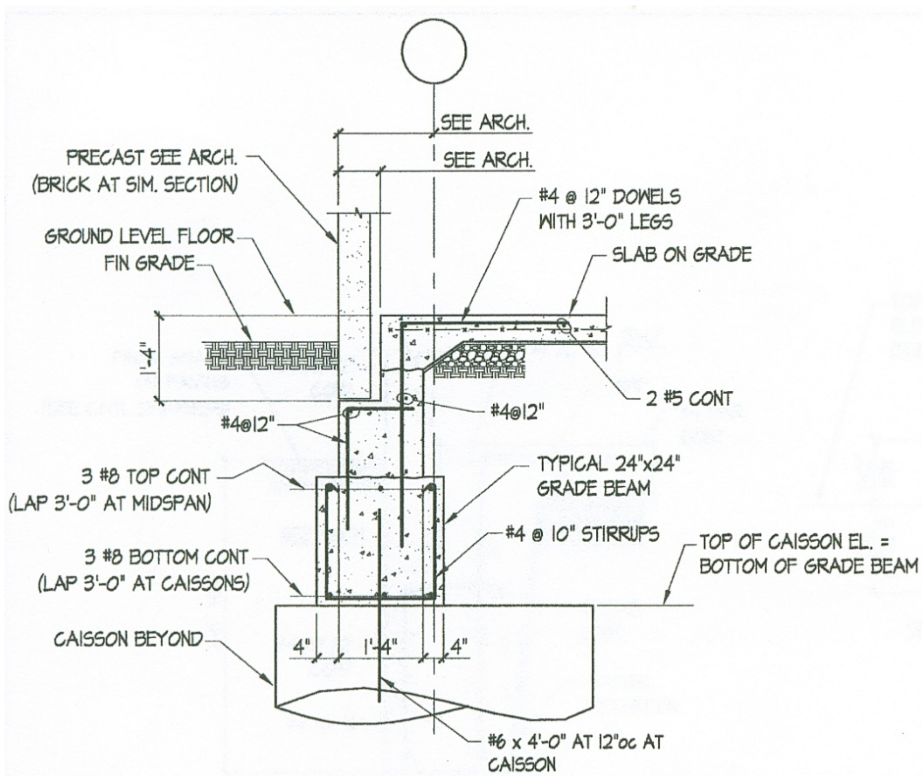


Figure 3: Typical Grade Beam Caisson Connection

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.”

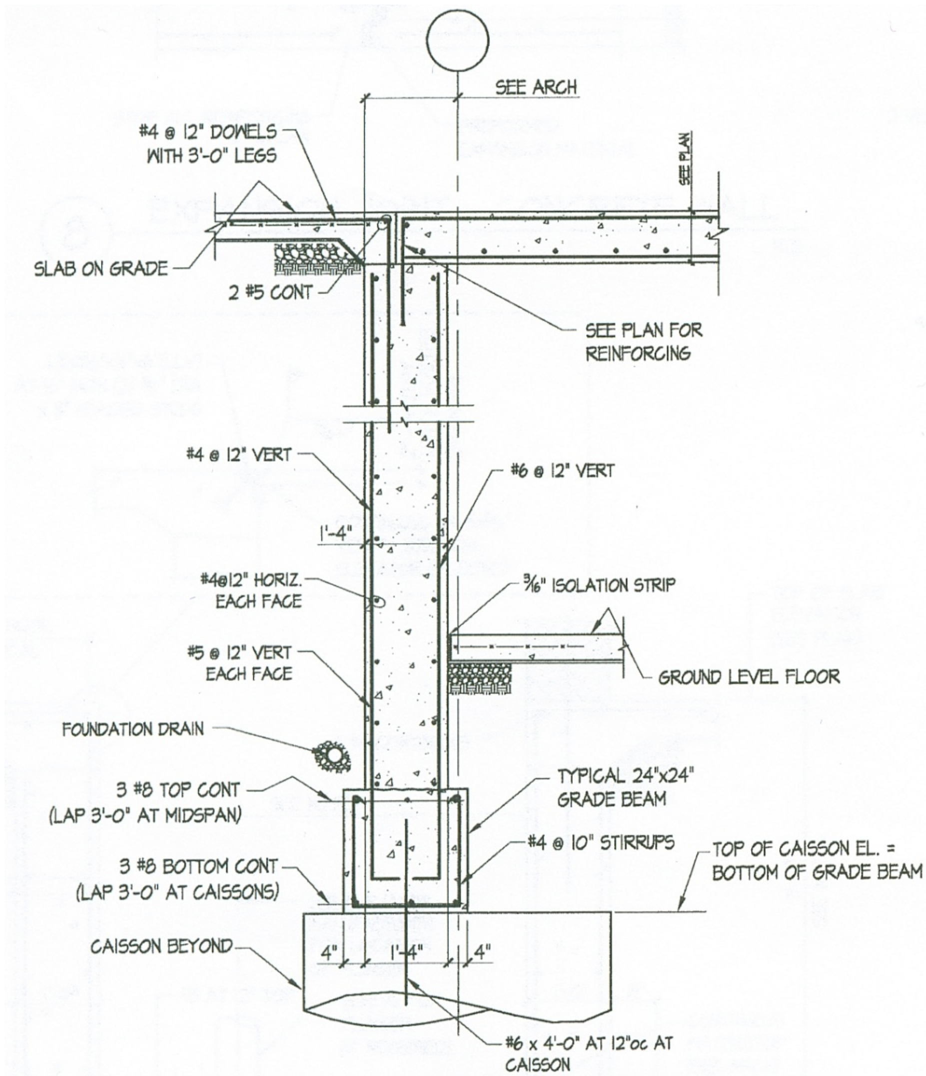


Figure 4: Typical Foundation Wall Section

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."

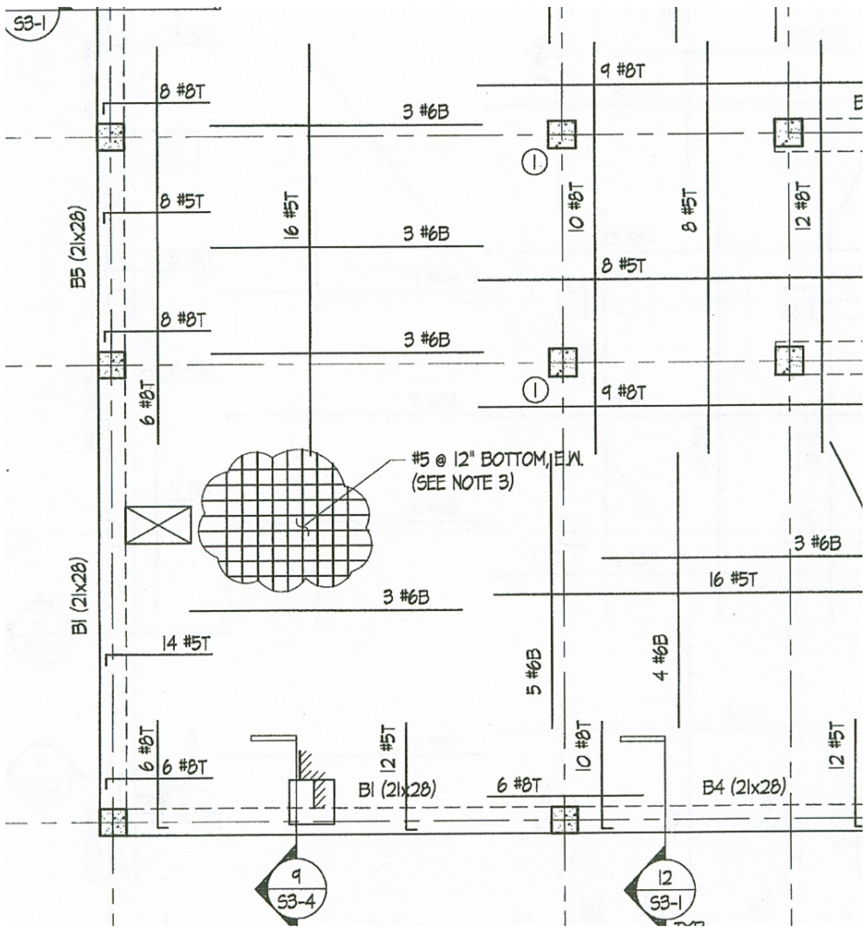


Figure 5: Slab Reinforcing Detail

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: W (INCHES)	SIZE: D (INCHES)	REINFORCING: BOTTOM BARS	REINFORCING: TOP BARS LE	REINFORCING: TOP BARS FL	REINFORCING: TOP BARS RE	STIRRUPS: SIZE	STIRRUPS: TYPE	STIRRUPS: SPACING (INCHES)	STIRRUPS: END	REMARKS
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	

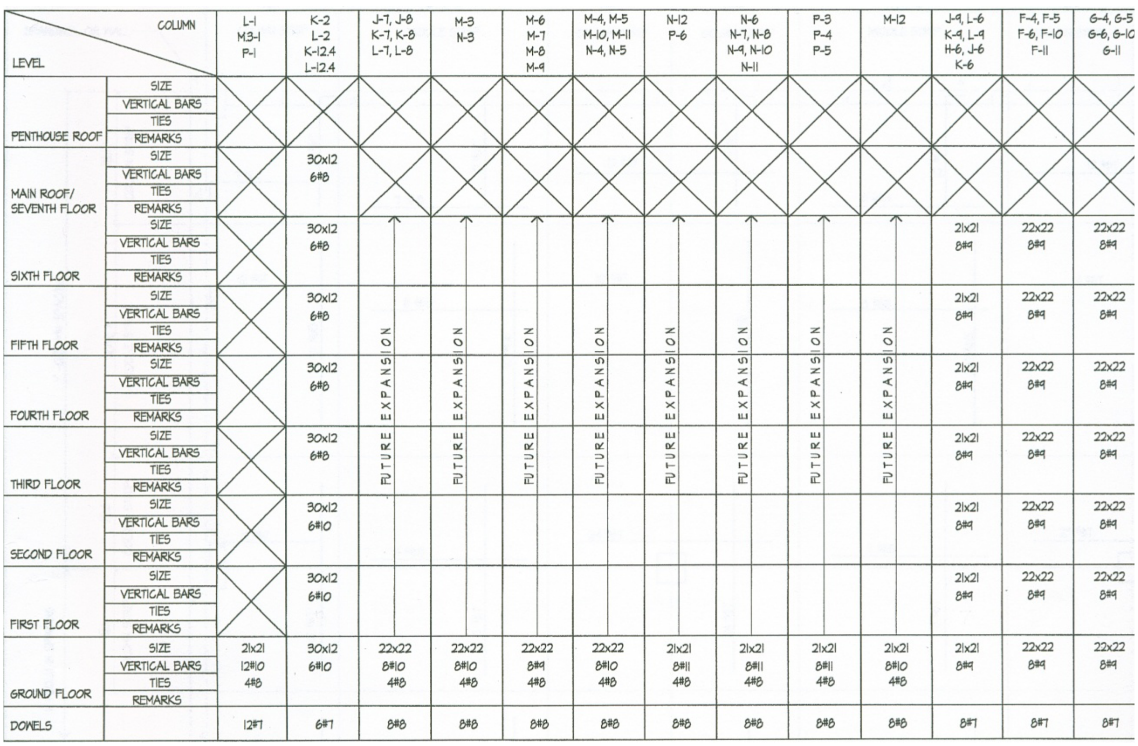
Figure 6: Portion of Concrete Beam Schedule

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 3, “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 VERTICAL BARS TIES REMARKS		30x12 6#8											
SIXTH FLOOR	SIZE VERTICAL BARS TIES REMARKS		30x12 6#8									21x21 8#9	22x22 8#9	22x22 8#9
FIFTH FLOOR	SIZE VERTICAL BARS TIES REMARKS		30x12 6#8									21x21 8#9	22x22 8#9	22x22 8#9
FOURTH FLOOR	SIZE VERTICAL BARS TIES REMARKS		30x12 6#8	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	21x21 8#9	22x22 8#9	22x22 8#9
THIRD FLOOR	SIZE VERTICAL BARS TIES REMARKS		30x12 6#8									21x21 8#9	22x22 8#9	22x22 8#9
SECOND FLOOR	SIZE VERTICAL BARS TIES REMARKS		30x12 6#10									21x21 8#9	22x22 8#9	22x22 8#9
FIRST FLOOR	SIZE VERTICAL BARS TIES REMARKS		30x12 6#10									21x21 8#9	22x22 8#9	22x22 8#9
GROUND FLOOR	SIZE VERTICAL BARS TIES REMARKS	21x21 12#10 4#8	30x12 6#10	22x22 8#10 4#8	22x22 8#10 4#8	22x22 8#9 4#8	22x22 8#10 4#8	21x21 8#11 4#8	21x21 8#11 4#8	21x21 8#11 4#8	21x21 8#10 4#8	21x21 8#9	22x22 8#9	22x22 8#9
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 are therefore expected to function as more efficient moment frames.

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

Snow Loads

The snow loads were determined based on Chapter 7 of ASCE 7-05. The ground snow load for the Baltimore area was found using the maps in ASCE 7 and was determined as 30 PSF. All assumptions regarding exposure, thermal factor, and importance factor match the designer's. Snow drift calculations follow ASCE 7's guidelines regarding leeward and windward drift heights. Table 2 shows the excel spreadsheet constructed to aid in the calculation of these drifts and Figure 8 shows the snow drift plan. See Appendix B for hand calculations.

Conclusion: The flat roof snow load came to 25.2 PSF but was rounded to 30 PSF for convenience. This will not impact loading of the structure because a minimum roof live load of 30 PSF is applied anywhere snow drifts do not accumulate. Compared to dead and live loads, the impact of snow is minimal to the structure except where drifts occur.

Table 1: Basic Snow Load Parameters	
Exposure Category C	C_e = 1.0
Thermal Factor	C_t = 1.0
Importance Factor (I)	I = 1.2
Ground Snow Load	P_g = 30 PSF
Flat Roof Snow Load	P_f = 25.2
Flat Roof Snow Load Used	30 PSF

Table 2: Snow Drift Example	
p_g (psf) =	30
p_f (psf) =	25.2
g (pcf) =	17.9
h_b=	1.407821
diff in roof height = h_c+h_b =	75
h_c =	73.59218
Leeward Drifts	
L upper roof =	41.5
h_d =	2.243974
Windward Drifts	
L lower roof =	81
h_d =	2.009186
Controlling h_d =	2.243974
w=	8.975896
w_{acutal} =	8.975896
h_d + h_b =	3.651795
max psf =	90.56713
min psf =	25.2

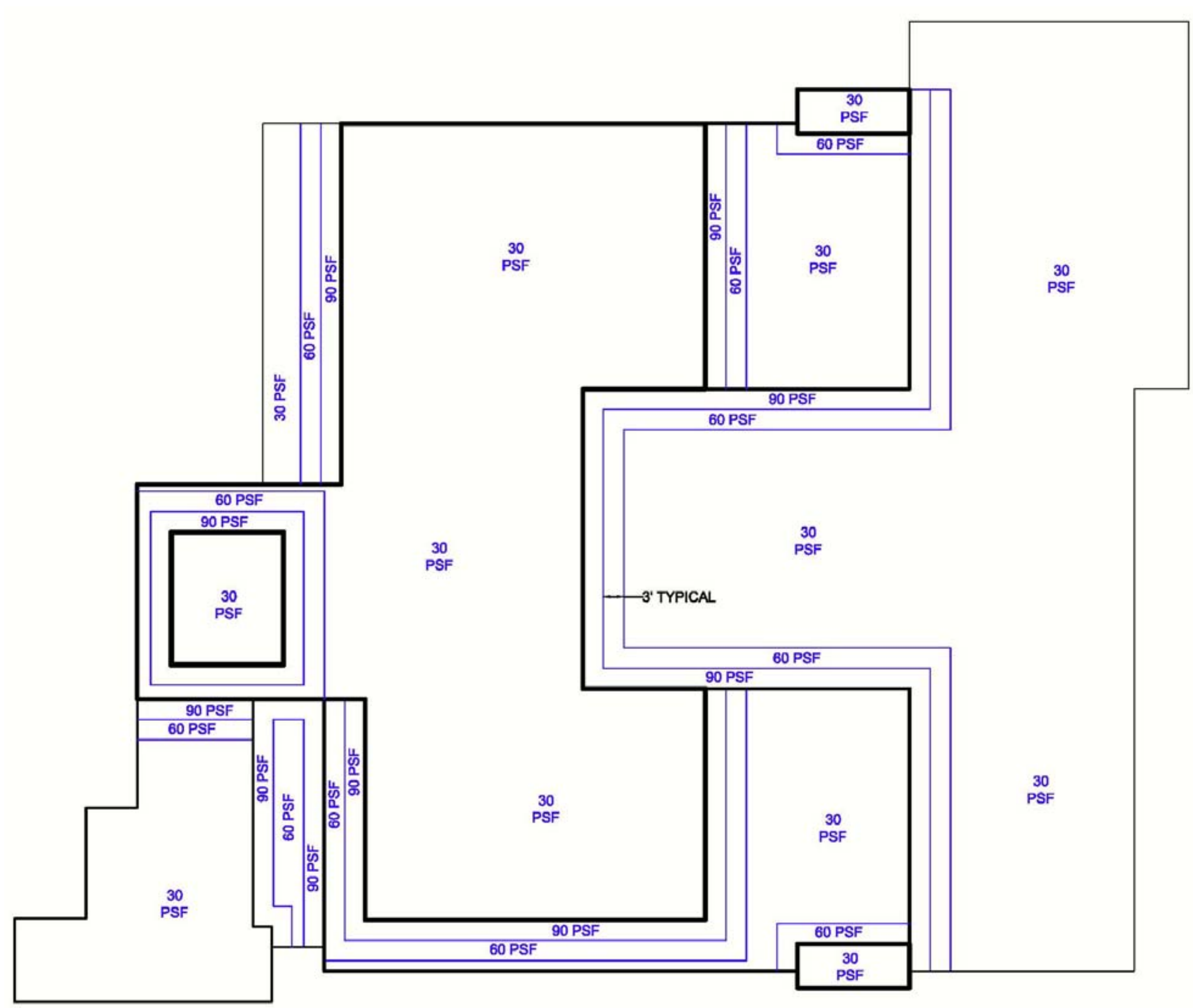


Figure 8: Snow Drift

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 9, "N-S Wind Pressure Diagram" and Figure 10, "E-W Wind Pressure Diagram." See Appendix C for hand calculations.

Conclusion: 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

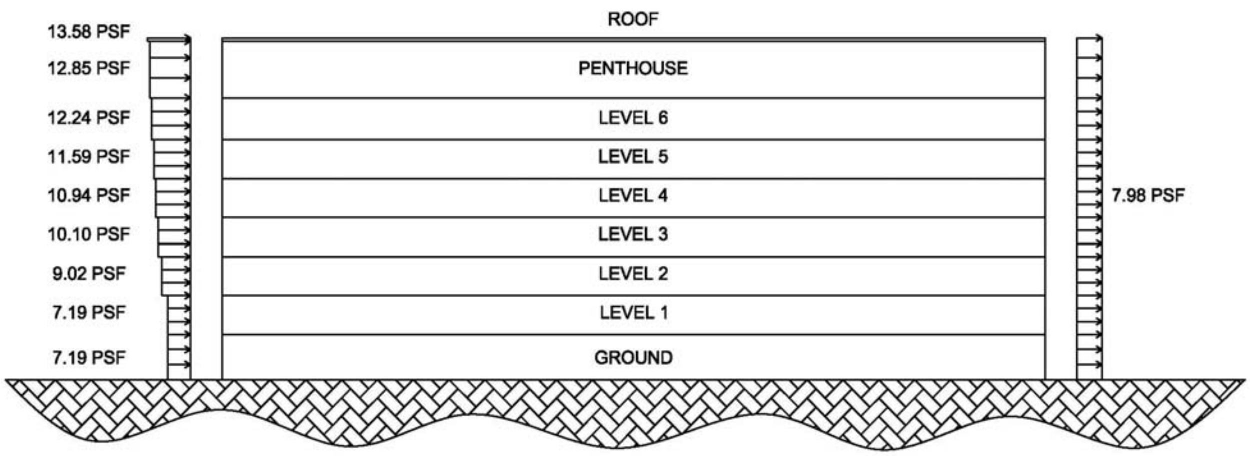


Figure 9: N-S Wind Pressure Diagram

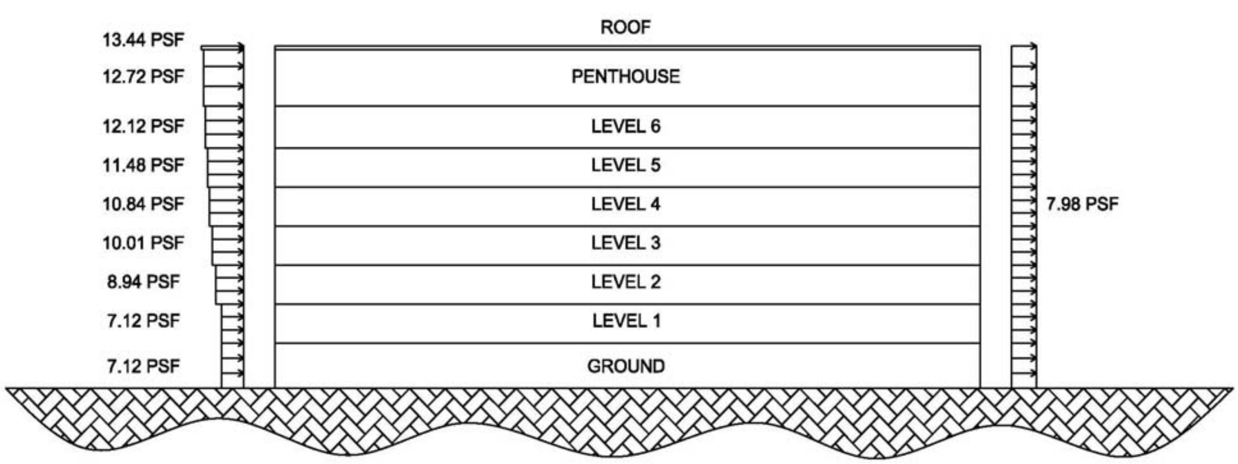


Figure 10: E-W Wind Pressure Diagram

Seismic Loads

While it may not seem to be important given its local, seismic analysis was an import consideration in the design of the Franklin Square Hospital Center. 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 11 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.

Conclusion: As calculated in this report, the seismic response coefficient is 0.016 while the designer used 0.0825. The seismic response coefficient used by the designers was based on the actual period of the structure as calculated from a building model while the calculation in this report follows the code. A reasonable conclusion would ascertain these differences stem from the codes use of trying to estimate a fictitious value. While this method can be useful, a direct building model study is far more accurate. What is more interesting is even with the large differences in seismic response coefficients, the design base shear is still close, 891k vs. 805k from the designer. This means that the building weight as calculated in this report, 55,713 k, is quite different than what the designers obtained which can be approximated as close to 10,000k. As shown in Table 13, the largest contributor to building weight is slab weight and errors in slab weight could have a big effect. The weight of the floor slabs was based on the plan dimensions of the drawings and while the drawings show large openings in the slab, smaller openings and depressions were not considered in the calculation. It could be possible the number of small openings and depressions is large enough to bring the slab weight down to match the designer's.

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

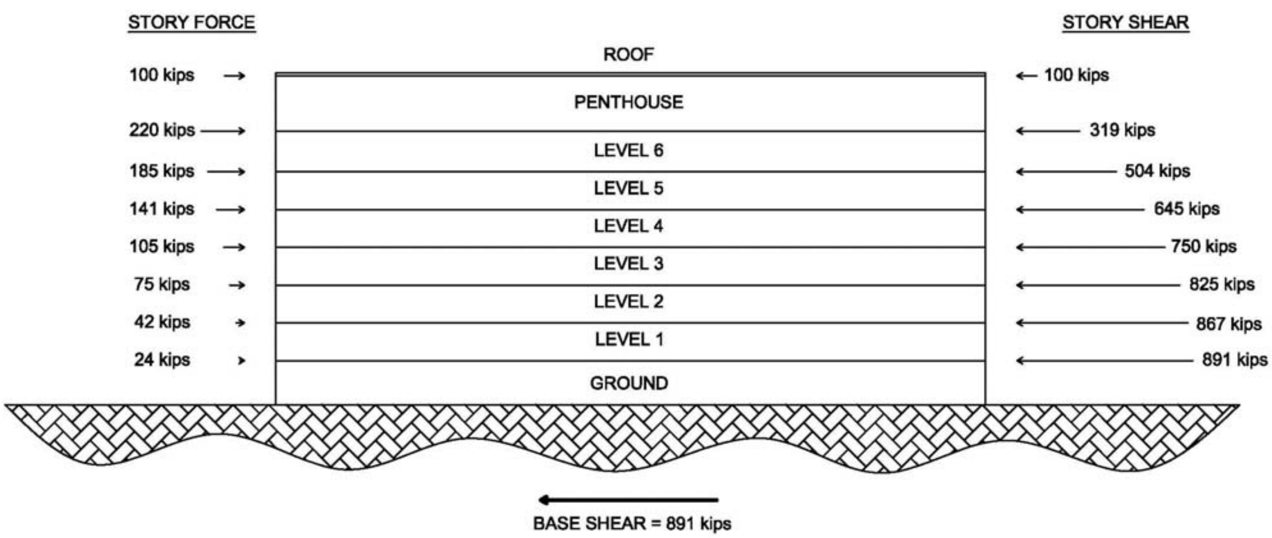


Figure 11: Seismic Load Diagram

Spot Checks

Slab and Beam Moment Checks

The 10” slab of the northwest corner of level 2 was checked for moment capacity. Direct Design Method was used to calculate the moments in the column and middle strips of the slab and edge beams. While moment capacity was not directly calculated, the steel area required to reinforce the slab was calculated and compared to the area of steel provided in the plans. The area of steel provided is sufficient for all critical sections except in the column strip at the face of the first interior column in both directions where the area of steel is insufficient by 1.16 in^2 in one direction and 0.37 in^2 in the other direction. When looking closer at the calculations it appears the designers used a different method to determine critical moments based on the area of steel they provided at the two sections where negative moments exist. They provided nearly identical amounts of steel in the exterior negative moment section as in the interior negative moment section. The frame along line F contains (10) #8 bars at each of these critical sections and the frame along line 4 contains (8) #8 bars in the exterior section and (9) #8 bars in the interior section. The direct design method assumes 70% of the total static moment is resisted by the interior support while 30 percent of the total static moment is resisted by the exterior support. A more advanced method such as the Equivalent Frame Method might provide a more exact analysis confirming the current design.

The edge beams along the same portion of slab were also checked for moment capacity. The design moments for the beams were determined in the same process as the design moments of the slab. The area of steel provided in the plans, between 2 in^2 and 3 in^2, is far greater than the area required based on gravity loading which is between 1 in^2 and 1.5in^2. This is expected as the beams are part of the lateral system and are required to resist much larger moments due to wind and seismic loads. See Appendix E for hand calculations.

Slab and Beam Shear Checks

The same slab as analyzed for moment capacity was analyzed for shear capacity, specifically punching shear at an internal column F-4. The slabs resistance to punching shear is quite adequate. The punching shear strength of the slab at 223 kips is reasonably larger than the nominal shear force on the slab at 132 kips. See Appendix E for hand calculations.

For the shear check of a beam, the shear capacity of an edge beam along the west wall was calculated, from E-5 to E-6. The beam schedule calls for #4 ties at 12” on center. Given the reinforcing and the size of the beam at 21x28, the shear capacity is a little over 79 kips. This is adequate as the shear force d away from the support is just 56 kips. See Appendix E for hand calculations.

Column Axial Checks

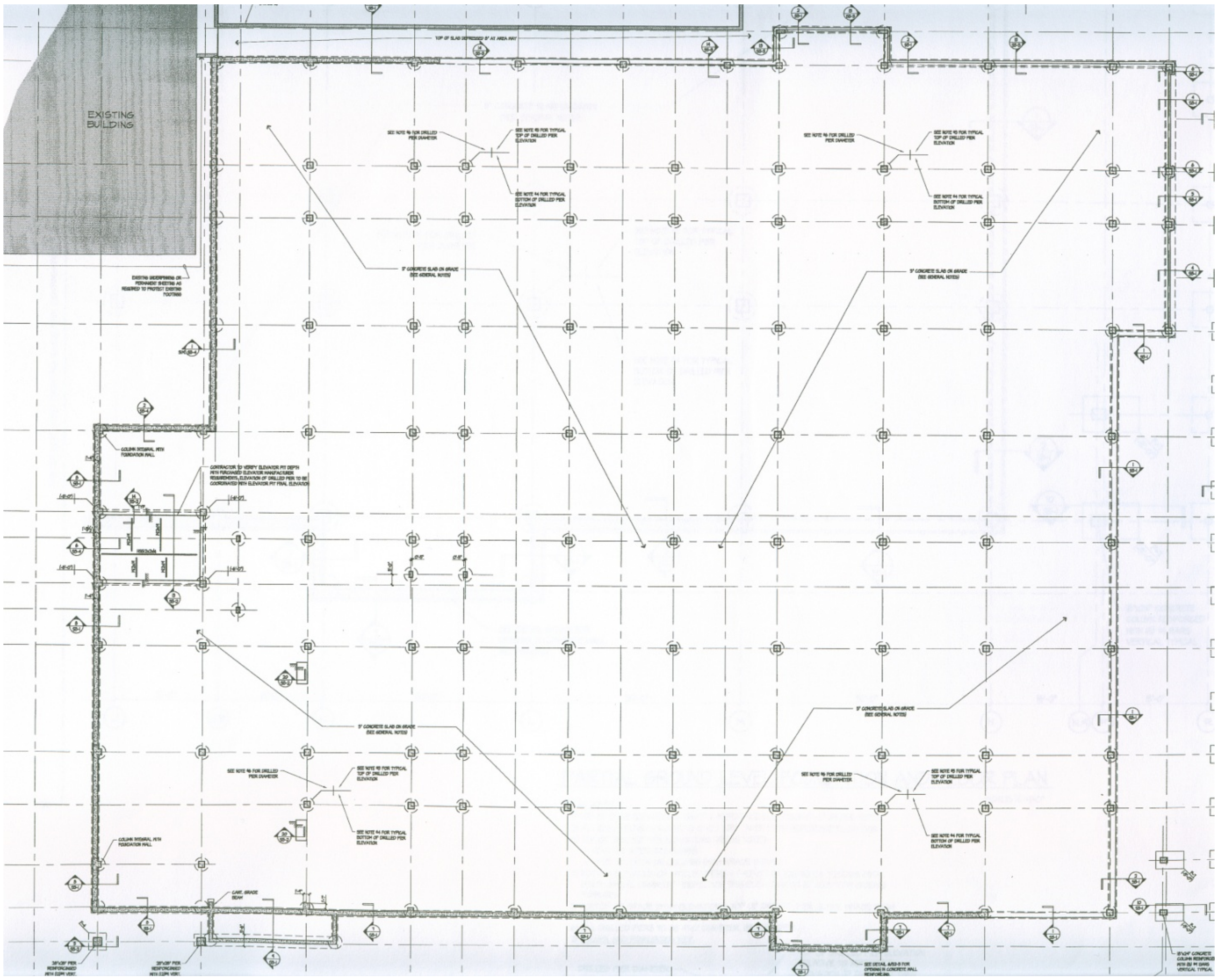
Columns were checked in two places, the 4th floor and the ground floor at F-4. The column size is continuous through the height of the building at 22”x22”, however the compressive strength of the columns change at the 3rd floor from 7000 psi to 5000 psi. Analyzed only for pure axial strength, the capacity of the column on the 4th floor is just over 2000 kips and the capacity on the ground floor is just over 2800 kips. The axial loads are 433 kips and 886 kips respectively. As expected, the column is quite adequate to carry the load given there is no applied moment for the gravity loading. Under lateral load from wind and earthquake, the column will be stressed much more likely approaching its design strength. See Appendix E for hand calculations.

Elements Needing Future Checks

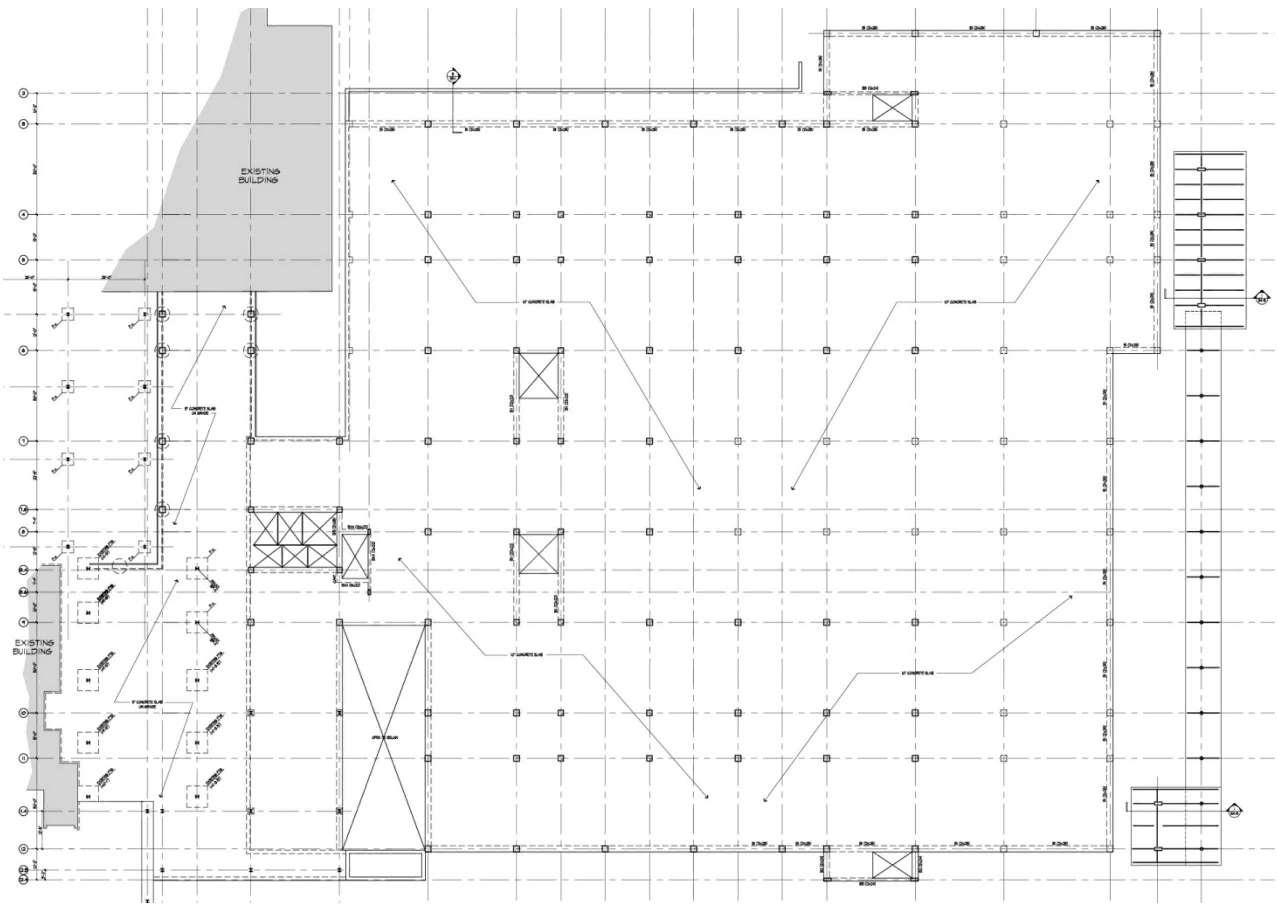
At this time, only gravity spot checks have been performed. Elements that still need checks include, but are not limited to, precast wall panels for lateral wind, roof structure for wind uplift, foundation walls for lateral soil pressure, foundation piers for axial and moment capacity, and entry canopies for wind and snow loading. However, future technical assignments will acknowledge these issues such as Technical Report 3 where a full lateral analysis of wind and lateral seismic loads will be conducted.

Appendix

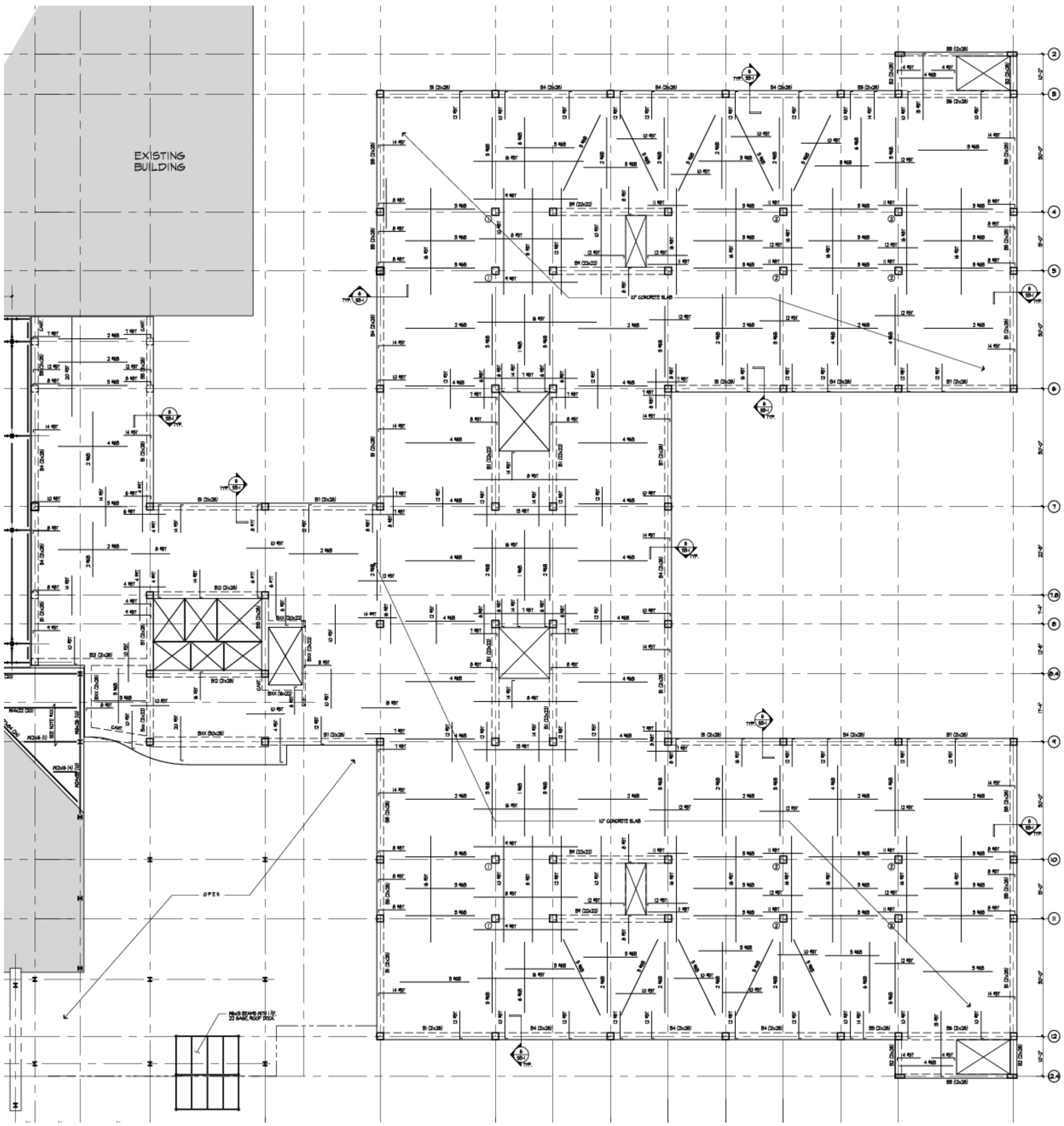
Appendix A: Building Plans



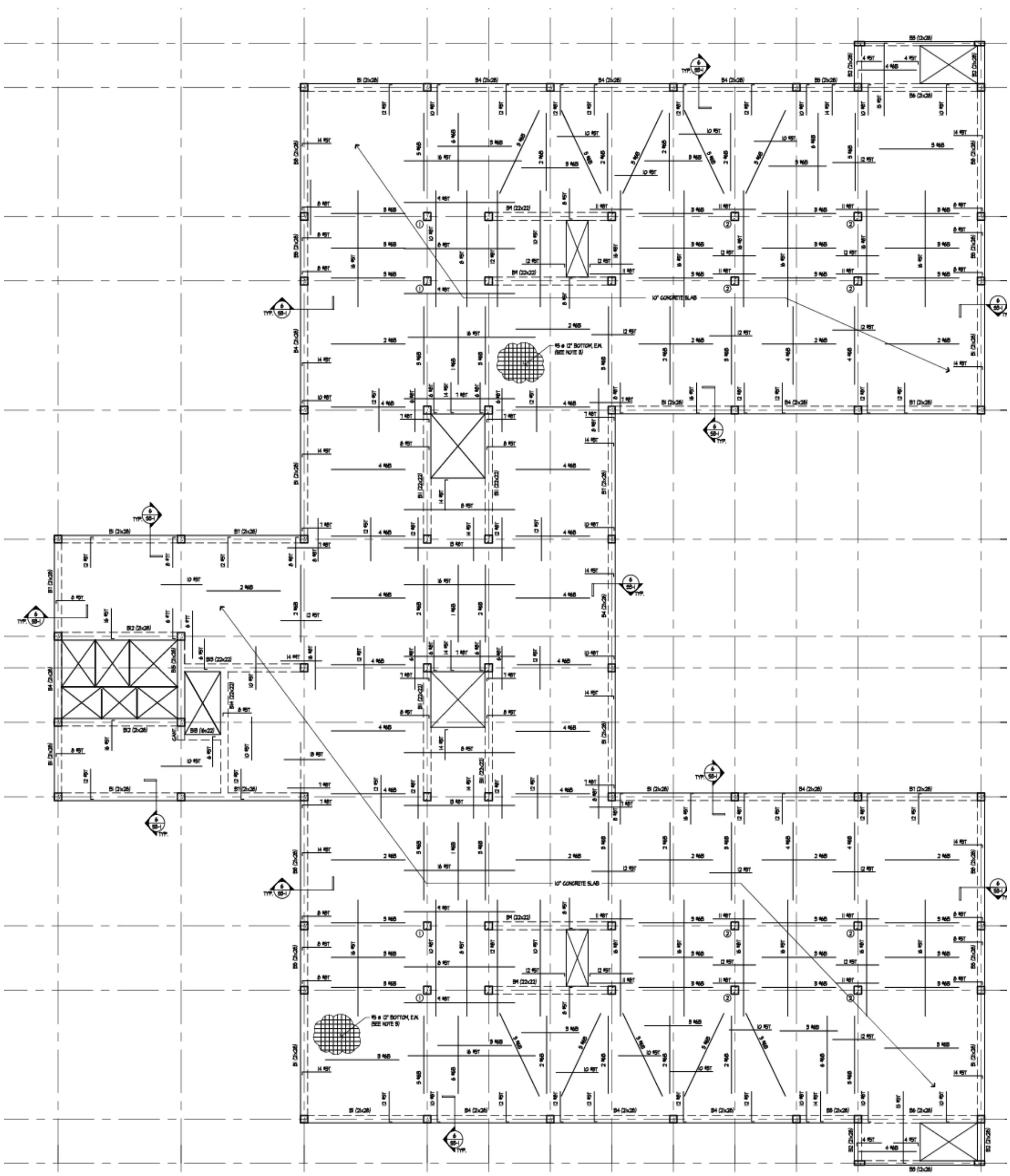
Ground Level



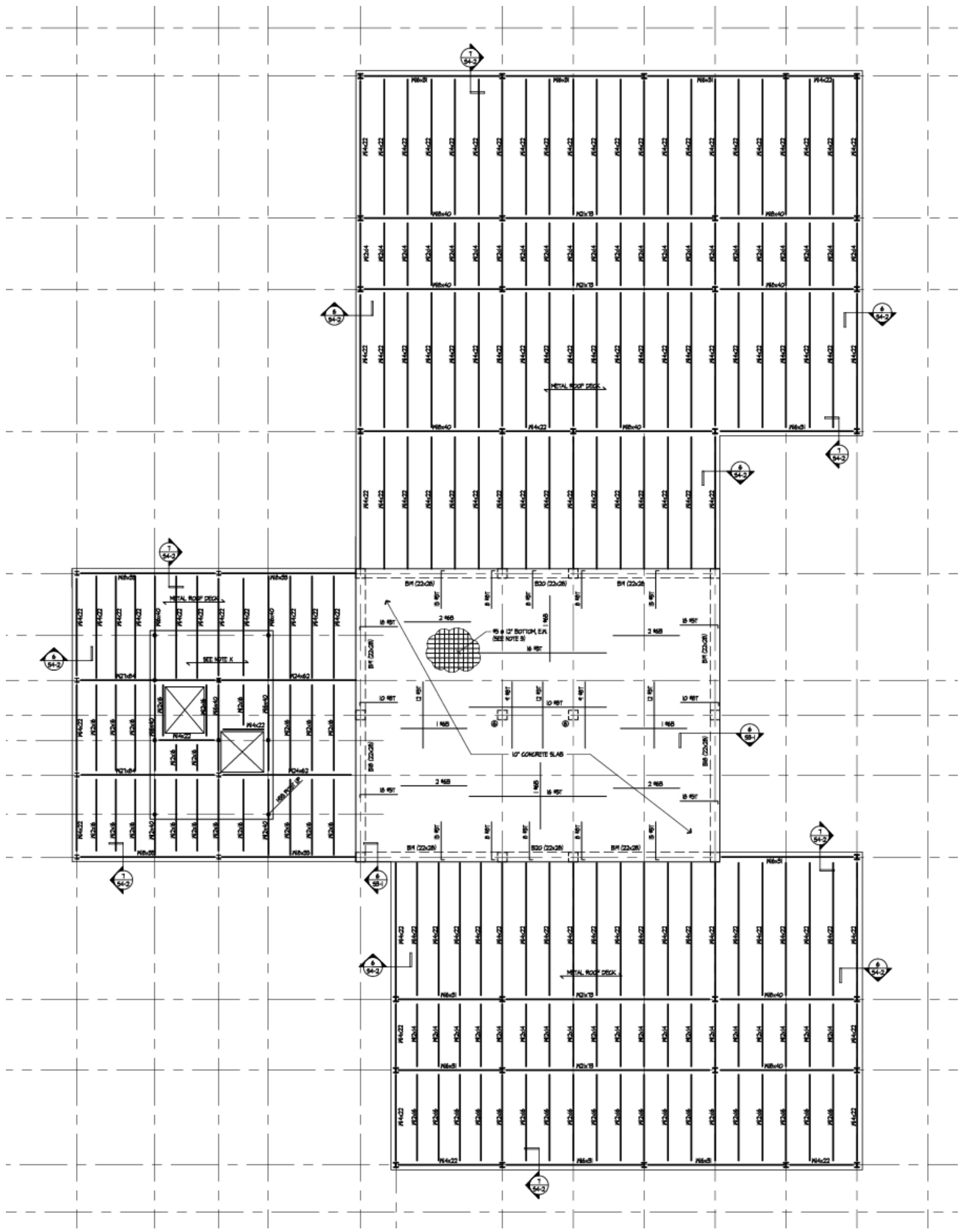
Level 1



Level 2 (Level 3 similar)



Level 4-7 (all similar)



Roof Level

Appendix B: Snow Analysis

SNOW LOAD CALCULATION

$p_f = 0.7 C_e C_t I p_g$

EXPOSURE C : $C_e = 1.0$

THERMAL FACTOR : $C_t = 1.0$

IMPORTANCE FACTOR : OCCUPANCY CATEGORY IV : $I = 1.2$

GROUND SNOW LOAD : $p_g = 30$ PSF

$p_f = 0.7(1.0)(1.0)(1.2)(30) = \boxed{25.2 \text{ PSF}}$ FLAT ROOF SNOW LOAD

SNOW DENSITY : $\gamma = 0.13 p_g + 14 \le 30 = 0.13(30) + 14 = 17.9$ PCF

$h_b = 25.2 / 17.9 = 1.41$ ft

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 = $\underline{\;1'\;}$

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

$F = -20.37(1') = -20.27$ PLF LEEWARD

BUILDING RIGID OR FLEXIBLE

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

$0.6542 < 1.0\ H_z$ → BUILDING IS FLEXIBLE

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

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

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

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

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

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

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

β = DAMPING COEFFICIENT = 1.5% = 0.015

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

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

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

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

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 $(L/B) = 260/225 = 1.156$ $\quad C_p = -0.47$ 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 $(L/B) = 225/260 = 0.865$ $\quad C_p = -0.5$ 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 $\rightarrow F_a = 1.2$

TABLE 11.4-2 $\rightarrow F_v = 1.7$

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

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

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

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

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 $\rightarrow C_t = 0.016$ $x = 0.9$

$T_a = C_t h_n^x$ $h_n = 106$ 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 $\rightarrow$ A

TABLE 11.6-2 $\rightarrow$ A SDC = A

$3.5\,T_S = 3.5(0.4105) = 1.4368 > T_a = 1.0548$ $\rightarrow$ EQUIVALENT LATERAL FORCE PROCD.

TABLE 12.2-1 $\rightarrow R = 3$ $I = 1.5 \rightarrow$ TABLE 11.5-1

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

$C_S = \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.1408}{\left(3/1.5\right)} = 0.0704$ $T_L = 8 > T = 1.808$ $C_S \le \dfrac{0.0578}{1.793\left(\frac{3}{1.5}\right)} = 0.016$

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

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

$k = 0.75 + 0.5T = 1.49$

$F_x = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \rightarrow$ 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	Totel Length of 10"x28" (ft)	Totel Length of 12"x28" (ft)	Totel Length of 21"x28" (ft)	Totel Length of 22"x24" (ft)	Totel Length of 8"x20" (ft)	Totel Length of 22"x20" (ft)	Totel Length of 24"x20" (ft)	Totel 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	Totel Length of W12x14 (ft)	Totel Length of W12x16 (ft)	Totel Length of W14x22 (ft)	Totel Length of W16x26 (ft)	Totel Length of W16x31 (ft)	Totel Length of W18x35 (ft)	Totel Length of W16x40 (ft)	Totel Length of W18x40 (ft)	Totel Length of W21x44 (ft)	Totel Length of W18x50 (ft)	Totel Length of W24x55 (ft)	Totel Length of W21x57 (ft)	Totel Length of W21x62 (ft)	Totel Length of W24x62 (ft)	Totel Length of W18x65 (ft)	Totel Length of W21x73 (ft)	Totel Length of W30x90 (ft)	Totel Length of W18x97 (ft)	Totel 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	-	-	-	-

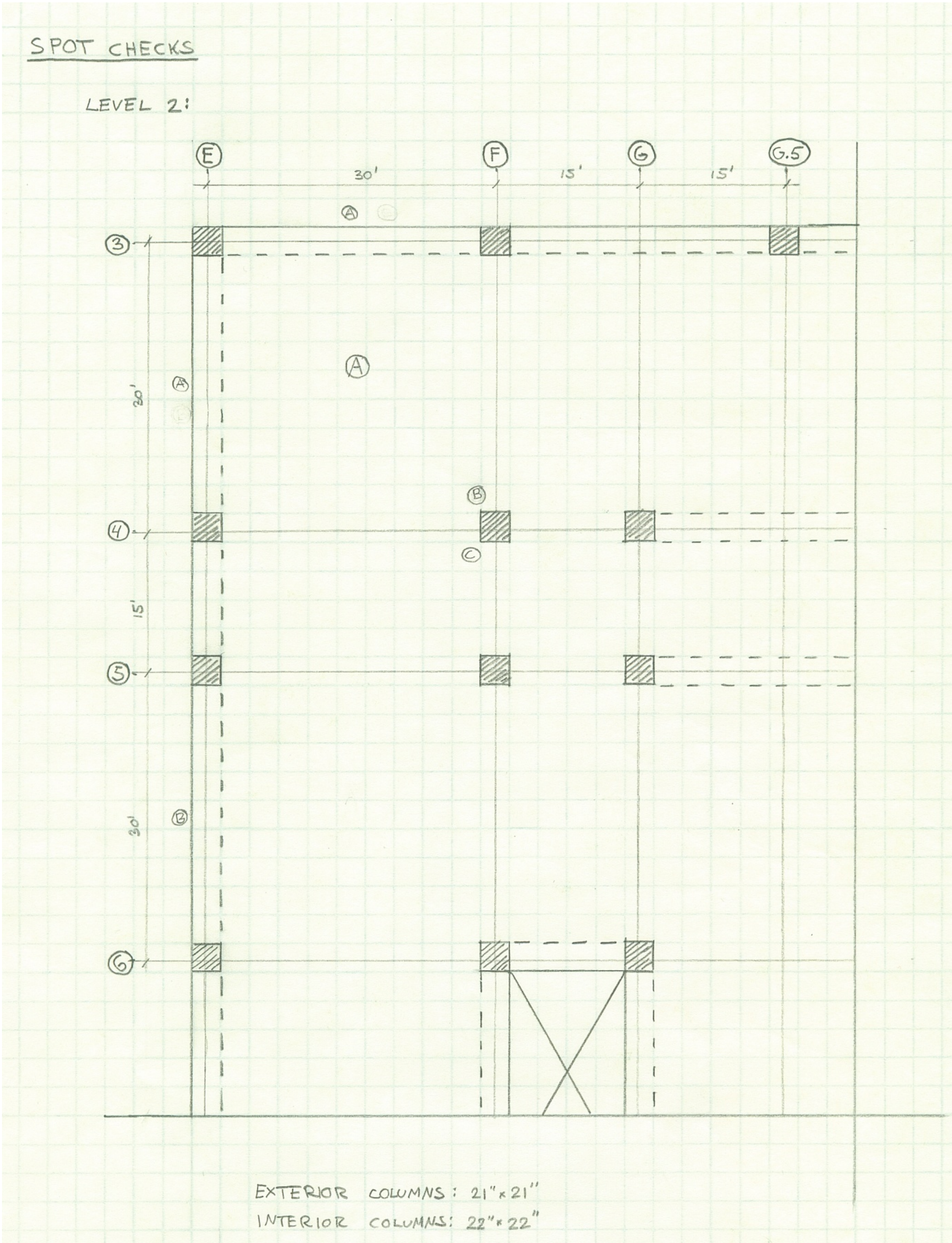
Weight of Façade						
Level	**Perimeter (ft)**	**Tributary Height (ft)**	**55% of Area (ft^2)**	**Thickness (in)**	**Wight (pcf)**	**Weight (k)**
Roof	802	8.75	7,018	7	30	123
Penthouse	802	15.25	6,727	7	80	314
Level 6	1140	12.5	7,838	7	145	663
Level 5	1140	12	7,524	7	145	636
Level 4	1140	12	7,524	7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



SLAB AND BEAM MOMENT CHECKS (A)

10" SLAB → $\omega = 145\left(\frac{10}{12}\right) = 121$ PSF

SUPERIMPOSED DEAD → $\omega = 20$ PSF

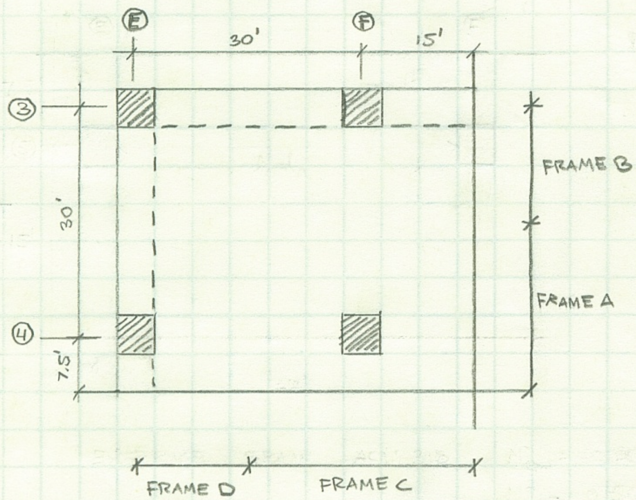
$\omega_D = 141$ PSF

PATIENT ROOMS → $\omega = 40$ PSF

PARTITIONS → $\omega = 20$ PSF

$\omega_L = 60$ PSF

$\omega_u = 1.2(141) + 1.6(60) = 265$ PSF



$\ell_n = 30 - \frac{21}{2}\left(\frac{1}{12}\right) - \frac{21}{2}\left(\frac{1}{12}\right) = 28.25'$

$s_n = 28.25'$

$f'_c = 5000$ PSI

$f_y = 60{,}000$ PSI

FRAME A: $M_o = \frac{1}{8}\omega_u \ell_2 \ell_n^2 = \frac{1}{8}(0.265)(22.5)(28.25)^2 = 594.8'^k$

FRAME B: $M_o = \frac{1}{8}(0.265)(15)(28.25)^2 = 396.5'^k$

FRAME C: $M_o = \frac{1}{8}(0.265)(22.5)(28.25)^2 = 594.8'^k$

FRAME D: $M_o = \frac{1}{8}(0.265)(15)(28.25)^2 = 396.5'^k$

$M^-_{ext} = 0.30\ M_o$

$M^+ = 0.50\ M_o$

$M^-_{int} = 0.70\ M_o$

FRAME A: $M^-_{ext} = 0.30(594.8) = 178.4'^k$

$M^+ = 0.50(594.8) = 297.4'^k$

$M^-_{int} = 0.70(594.8) = 416.4'^k$

FRAME B: $M^-_{ext} = 0.30(396.5) = 119.0'^k$

$M^+ = 0.50(396.5) = 198.3'^k$

$M^-_{int} = 0.70(396.5) = 277.6'^k$

FRAME C: $M^-_{ext} = 0.30(793.1) = 237.9'^k$

$M^+ = 0.50(793.1) = 396.6'^k$

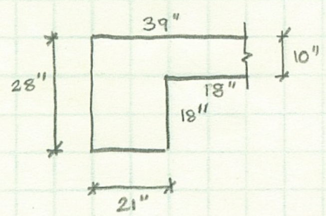
$M^-_{int} = 0.70(793.1) = 555.2'^k$

FRAME D: $M^-_{ext} = 0.30(396.5) = 119.0'^k$

$M^+ = 0.50(396.5) = 198.3'^k$

$M^-_{int} = 0.70(396.5) = 277.6'^k$

EDGE BEAM TORSIONAL CONSTANTS



$b_E = b_w + h_w = 21 + (18) = 39'' \leftarrow$
$\leq b_w + 4t = 21 + 4(10) = 61''$

$$C_1 = \left[1 - 0.63\left(\frac{21}{28}\right)\right]\left[\frac{21^3(28)}{3}\right] + \left[1 - 0.63\left(\frac{10}{18}\right)\right]\left[\frac{10^3(18)}{3}\right] = 49{,}495\ in^4$$

$$C_2 = \left[1 - 0.63\left(\frac{21}{18}\right)\right]\left[\frac{21^3(18)}{3}\right] + \left[1 - 0.63\left(\frac{10}{39}\right)\right]\left[\frac{10^3(39)}{3}\right] = 25{,}625\ in^4$$

USE $C = 49{,}495\ in^4$

$I_b = k\, b_w h^3/12$ $\quad b_E/b_w = 39/21 = 1.86,\ t/h = 10/28 = 0.36 \rightarrow k = 1.33$

$I_b = 1.33(21)(28^3)/12 = 51{,}093\ in^4$

TRANSVERSE DISTRIBUTION OF LONGITUDINAL MOMENTS SLAB

ITEM	DESCRIPTION	FRAME A	FRAME B	FRAME C	FRAME D
1.	TOTAL WIDTH (in)	270"	180"	270"	180"
2.	COLUMN STRIPWIDTH (in)	135"	90"	135"	90"
3.	MIDDLE STRIP WIDTH (in)	112.5"	90"	112.5"	90"
4.	TORSIONAL CONST. C(in^4)	55,495	55,495	55,495	55,495
5.	$I_s = \frac{l t^3}{12}$	$\frac{270(10)^3}{12} = 22{,}500$	15,000	22,500	15,000
6.	$\beta_t = \frac{C}{2I_s}$	1.23	1.85	1.23	1.85
7.	$\alpha_1 = I_b / I_s$	0	3.41	0	3.41
8.	l_2 / l_1	1.0	1.0	1.0	1.0
9.	$\alpha_1 \, l_2 / l_1$	0	3.41	0	3.41
10.	M_{ext}^- % TO CS	88%	82%	88%	88%
11.	M^+ % TO CS	60%	75%	60%	75%
12.	M_{int}^- % TO CS	75%	75%	75%	75%

FRAME A:

	M_{ext}^-	M^+	M_{int}^-
TOTAL MOMENT	-178.4	297.4	-416.4
% TO CS	88	60	75
MOMENT CS	-157	178.4	-312.3
MOMENT MS	-21.4	119	-104.1

FRAME B:

	M_{ext}^-	M^+	M_{int}^-
TOTAL MOMENT	-119	198.3	-277.6
% TO CS	82	75	75
MOMENT CS	-97.6	148.7	-208.2
% TO CS BEAM	85	85	85
MOMENT CS BEAM	-83.0	126.4	-177.0
MOMENT CS SLAB	-14.6	22.3	-31.2
MOMENT MS	-21.4	49.6	-69.4

FRAME C:

	M_{ext}^-	M^+	M_{int}^-
TOTAL MOMENT	-178.4	297.4	-416.4
% TO CS	88	60	75
MOMENT CS	-157	178.4	-312.3
MOMENT MS	-21.4	119	-104.1

FRAME D:

	M_{ext}^-	M^+	M_{int}^-
TOTAL MOMENT	-119.0	198.3	-277.6
% TO CS	82	75	75
MOMENT CS	-97.6	148.7	-208.2
% TO CS BEAM	85	85	85
MOMENT CS BEAM	-83.0	126.4	-177.0
MOMENT CS SLAB	-14.6	22.3	-31.2

FRAME A COLUMN STRIP SLAB

ITEM	DESCRIPTION	M_{ext}^-	M^+	M_{int}^-
1.	M_u ('k)	-157	178.4	-312.3
2.	CS WIDTH, b	135"	135"	135"
3.	EFFECTIVE DEPTH	8.75"	8.88"	8.75"
4.	$M_n = M_u/\phi = M_u/0.9$	-174.4	198.2	-347.0
5.	$R = M_n(12000)/bd^2$	202.5	223.4	402.9
6.	ρ	0.0035	0.0038	0.0070
7.	$A_{s\,req} = \rho bd$	4.13	4.56	8.27
8.	$A_{s\,min} = 0.0018bt$	2.43	2.43	2.43
9.	$A_{s\,prov}$	6.32	4.73	7.11
		6.32 > 4.13 ✓	4.73 > 4.56	7.11 < 8.27 X

FRAME A MIDDLE STRIP SLAB

ITEM	DESCRIPTION	M_{ext}^-	M^+	M_{int}^-
1.	M_u ('k)	-21.4	119	-104.1
2.	MS WIDTH, b	112.5"	112.5"	112.5"
3.	EFFECTIVE DEPTH	8.94"	8.94	8.94"
4.	$M_n = M_u/\phi = M_u/0.9$	-23.8	132.2	-115.7
5.	$R = M_n(12000)/bd^2$	31.7	176.5	154.4
6.	ρ	0.0005	0.0030	0.0026
7.	$A_{s\,req} = \rho bd$	0.54	3.03	2.65
8.	$A_{s\,min} = 0.0018bt$	2.025	2.025	2.025
9.	$A_{s\,prov}$	2.48	3.10	2.79
		2.48 > 2.025 ✓	3.10 ≥ 3.03 ✓	2.79 > 2.65 ✓

$A_{s\,prov}$ $M_{ext}^- = \left(\frac{112.5}{112.5+90}\right)(14) = 7.78 = 8$ $\quad 8(0.31) = 2.48\ in^2$ (#5)

$A_{s\,prov}$ $M^+ = \left(\frac{112.5}{12''o.c}\right) = 9.375 = 10$ $\quad 10(0.31) = 3.10\ in^2$

$A_{s\,prov}$ $M_{int}^- = \left(\frac{112.5}{112.5+90}\right)(16) = 8.89 = 9$ $\quad 9(0.31) = 2.79\ in^2$ (#6)

FRAME B COLUMN STRIP SLAB

ITEM	DESCRIPTION	M_{ext}^-	M^+	M_{int}^-
1.	M_u (14)	-14.6	22.3	-31.2
2.	CS WIDTH, b - BEAM (21")	69"	69"	69"
3.	EFFECTIVE DEPTH	8.81"	8.94"	8.81"
4.	$M_n = M_u/\phi = M_u/0.9$	-16.2	24.8	-34.7
5.	$R = M_n(12000)/bd^2$	36.3	53.9	77.7
6.	ρ	0.0006	0.0009	0.0013
7.	$A_{sreq} = \rho bd$	0.38	0.57	0.81
8.	$A_{smin} = 0.0018bt$	1.24	1.24	1.24
9.	A_{sprov} =	4.80	1.86	4.80

4.80 > 1.24 ✓ 1.86 > 1.24 ✓ 4.80 > 1.24 ✓

FRAME B COLUMN STRIP BEAM

ITEM	DESCRIPTION	M_{ext}^-	M^+	M_{int}^-
1.	M_u (14)	-83.0	126.4	-177.0
2.	BEAM WIDTH	21"	21"	21"
3.	EFFECTIVE DEPTH	26.69	26.69	26.69
4.	$M_n = M_u/\phi = M_u/0.9$	-92.2	140.4	-196.7
5.	$R = M_n(12000)/bd^2$	74.0	112.7	157.8
6.	ρ	0.0013	0.0019	0.0027
7.	$A_{sreq} = \rho bd$	0.71	1.08	1.51
8.	$A_{smin} = 0.0018bt$	1.06	1.06	0.38
9.	A_{sprov}	3.0	2.0	3.0

3.0 > 1.06 ✓ 2.0 > 1.08 ✓ 3.0 > 1.51 ✓

FRAME B MIDDLE STRIP SLAB

ITEM	DESCRIPTION	M_{ext}^-	M^+	M_{int}^-
1.	M_u (14)	-21.4	49.6	-69.4
2.	MS WIDTH, b	90"	90"	90"
3.	EFFECTIVE DEPTH	8.94"	8.94"	8.94"
4.	$M_n = M_u/\phi = M_u/0.9$	-23.8	55.1	-77.1
5.	$R = M_n(12000)/bd^2$	39.7	91.9	128.6
6.	ρ	0.0006	0.003	0.0026
7.	$A_{sreq} = \rho bd$	0.48	2.41	2.09
8.	$A_{smin} = 0.0018bt$	1.62	1.62	1.62
9.	A_{sprov}	1.86	2.48	2.17

1.86 > 1.62 ✓ 2.48 > 2.41 ✓ 2.17 > 2.09 ✓

A_{sprov} M_{ext}^- = $(14-8)(0.31) = 1.86 \text{ in}^2$

A_{sprov} M^+ = $\left(\frac{90''}{12'' o.c.}\right) = 7.5 = 8$ $8(0.31) = 2.48 \text{ in}^2$

A_{sprov} M_{int}^- = $(16-9)(0.31) = 2.17 \text{ in}^2$

FRAME C COLUMN STRIP SLAB

ITEM	DESCRIPTION	M_{ext}^-	M^+	M_{int}^-
1.	M_u (1k)	-157	178.4	-312.3
2.	CS WIDTH, b	135"	135"	135"
3.	EFFECTIVE DEPTH	8.75"	8.88"	8.75"
4.	$M_n = M_u/\phi = M_u/0.9$	-174.4	198.2	-347.0
5.	$R = M_n(12000)/bd^2$	202.5	223.4	402.9
6.	ρ	0.0035	0.0038	0.0070
7.	$A_{s_{req}} = \rho bd$	4.13	4.56	8.27
8.	$A_{s_{min}} = 0.0018\,bt$	2.43	2.43	2.43
9.	$A_{s_{prov}}$	7.9	5.61	7.9

$7.9 > 5.67$ ✓ $5.61 > 4.56$ ✓ $7.9 < 8.27$ ✗

FRAME C MIDDLE STRIP SLAB

ITEM	DESCRIPTION	M_{ext}^-	M^+	M_{int}^-
1.	M_u (1k)	-21.4	119.0	-104.1
2.	MS WIDTH, b	112.5"	112.5"	112.5"
3.	EFFECTIVE DEPTH	8.94"	8.94"	8.94"
4.	$M_n = M_u/\phi = M_u/0.9$	-23.8	132.2	-115.7
5.	$R = M_n(12000)/bd^2$	31.7	176.5	154.4
6.	ρ	0.0006	0.003	0.0026
7.	$A_{s_{req}} = \rho bd$	0.54	3.03	2.65
8.	$A_{s_{min}} = 0.0018\,bt$	2.025	2.025	2.025
9.	$A_{s\,prov}$	2.17	3.10	2.79 in^2

$2.17 > 2.025$ ✓ $3.10 > 3.03$ ✓ $2.79 > 2.65$ ✓

$A_{s_{prov}}$ $M_{ext}^- = \left(\frac{112.5}{112.5+90}\right)(12) = 6.67 = 7$ $7(0.31) = 2.17\ in^2$

$A_{s_{prov}}$ $M^+ = \left(\frac{112.5}{12''\,o.c.}\right) = 9.375 = 10$ $10(0.31) = 3.10\ in^2$

$A_{s_{prov}}$ $M^- = \left(\frac{112.5}{112.5+90}\right)(16) = 8.89 = 9$ $9(0.31) = 2.79\ in^2$

FRAME D SAME AS FRAME B W.R.T. MOMENTS AND REINFORCING ∴ OK ✓

ALL REINFORCING IS ADEQUATE EXCEPT FOR M_{int}^- ON BOTH FRAME A AND C

$\boxed{\text{SLAB AND BEAM SHEAR CHECKS (B)}}$

PUNCHING SHEAR COLUMN F-4

$d \approx 8.9"$ $\quad d/2 = 4.45"$ $\quad b_o = 4(22 + 2(4.45)) = 123.6\ in$

$A_{trib} = \left(\frac{30}{2} + \frac{15}{2}\right)\left(\frac{30}{2} + \frac{15}{2}\right) - \frac{(22 + 2(4.45))^2}{144} = 499.6\ ft^2$

$V_u = w_u A_{trib} = 0.265(499.6) = 132.4^k$

$V_c = 4\sqrt{f'_c}\, b_o d$ GOVERNS BY INSPECTION

$= 4\sqrt{5000}(123.6)(8.9) = 311^k$

$\phi V_c = 0.75(311) = 233^k > 132.4^k$ OK✓

BEAM SHEAR E5 TO E6

$w = 0.265\ k/ft^2 \left(14.13\ ft + \frac{28}{12}\ ft\right) = 4.36\ k/ft$

$w_{beam} = \frac{(21)(28-10)}{144}\left(145\ lb/ft^3\right) = 0.38\ k/ft$

$w_u = 4.36 + 0.38 = 4.74\ k/ft$ $\quad V_u = \frac{w_u l}{2} = \frac{4.74(28.25)}{2} = 66.96^k$

$V_u @ d = 66.96 - 4.74(26.69)/12 = 56.42^k$

$V_s =$ #4 @ 12" $= 0.20(60)(26.69)/12 = 26.69^k$

$V_c = 2\sqrt{5000}(21)(26.69)/1000 = 79.27^k$

$\phi V_n = 0.75\left[79.27 + 26.69\right] = 79.47^k > 56.42^k$ OK✓

$\boxed{\text{COLUMN SPOT CHECKS ©}}$

COLUMN F4

FOURTH FLOOR: $A_{trib} = 503\ ft^2/\text{LEVEL}$

22" × 22" W/ (8) #9

$w_L = (\underbrace{40}_{\text{ROOM}} + \underbrace{80}_{\text{HALLWAY}})/2 = 60\ PSF + \underbrace{20\ PSF}_{\text{PARTITIONS}} = 80\ PSF$

$f'_c = 5000\ PSI$

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: $w_D = 141\ PSF$ $w_L = 80\ PSF$

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

SELF WEIGHT $= \frac{(22")(22")}{144"/1}(12' + 13' + 17.5')\frac{(145\ lb/ft^3)}{1000} = 20.7^k$

$P_D = \frac{527(503)}{1000} + 20.7 = 285^k$

LIVE LOAD REDUCTION: $K_{LL} = 4$ $A_T = 503(4) = 2012\ ft$

$L_o = (30 + 80 + 80 + 80)(503) = 136^k$

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

$P_u = 1.2\ P_D + 1.6\ P_L = 1.2(285) + 1.6(57) = 433^k$

$P_u = 1.4D$ DOES NOT GOVERN

PURE AXIAL STRENGTH

$P_o = 0.85\ f'_c\ A_c + A_s f_y = 0.85(5000)(22 \times 22 - 8) + 8(60)$

$P_o = 2023^k$

$P_o = 2023^k > P_u = 433^k$ OK✓

COLUMN F4

GROUND LEVEL: $A_{trib} = 503\ ft^2/\text{LEVEL}$

ROOM HALLWAY PARTITIONS

22" x 22" W/ (8) #9 $\quad w_L = (40 + 80)/2 = 60\ PSF + 20\ PSF = 80\ PSF$

$f'_c = 7000\ PSI$

ROOF: $w_D = 80\ PSF$ $\quad w_L = 30\ PSF$

PENTHOUSE: $w_D = 165\ PSF$ $\quad w_L = 80\ PSF$

LEVEL 6: $w_D = 141\ PSF$ $\quad w_L = 80\ PSF$

LEVEL 5: $w_D = 141\ PSF$ $\quad w_L = 80\ PSF$

LEVEL 4: $w_D = 141\ PSF$ $\quad w_L = 80\ PSF$

LEVEL 3: $w_D = 141\ PSF$ $\quad w_L = 80\ PSF$

LEVEL 2: $w_D = 141\ PSF$ $\quad w_L = 80\ PSF$

LEVEL 1: $w_D = 141\ PSF$ $\quad w_L = 80\ PSF$

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

$$\text{SELF WEIGHT} = \frac{(22")(22")}{144\ "/'}(12 + 12 + 12 + 12 + 12 + 13 + 17.5)\frac{(145\ lb/ft^3)}{1000} = 44.1^k$$

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

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

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

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

$$P_u = 1.2 P_D + 1.6 P_L = 1.2(593) + 1.6(109) = 886^k$$

$P_u = 1.4 P_D$ DOES NOT GOVERN

PURE AXIAL STRENGTH

$$P_o = 0.85(7000)(22 \times 22 - 8) + 8(60) = 2{,}833^k$$

$$P_o = 2833^k > P_u = 886^k \quad \underline{OK} \checkmark$$