Mathew Nirenberg
Structural Option
Metropolis at Dadeland
Adviser: Schneider
Tech 1 Exec Summary
10/5/05

Technical Report 1 – Executive Summary
Structural Concepts and Existing Conditions

This report contains an analysis of the overall structure of the Metropolis at Dadeland, phase 1 tower. Metropolis is a 28 story building that is 313 feet tall, meaning that there is no taller tower south of it in Florida. It is the tallest of a rebuilding effort around the east end of Kendall to create a “downtown” atmosphere in that area. The bottom level is entirely commercial and public space. Up to the seventh level there is a parking garage on the interior with loft spaces around the perimeter. The eighth floor is a communal space for the buildings residents with spaces like a gym and pool. Above that there are just condominiums and penthouses. The top two floors and roof are dedicated to mechanical equipment.

The structure of Metropolis is entirely concrete. The foundation consists of piles, most of which lie under a 5 foot thick mat. The columns are reinforced concrete and the slabs are all post-tensioned aside from the roof. The strengths of concrete used range from 4,000 psi to 10,000 psi.

The focus of this report was on the loading of this structure. Analysis of wind, seismic, and gravity loads were performed. As expected, the wind loading was far more critical than the seismic loading. Loading diagrams and plans of the structure are included throughout the report to clarify the loads calculated. Spot checks were also performed in order to ensure that my analysis matches the design of the original engineer. Wind calculations can be found in appendix A, seismic in appendix B, shear wall analysis in appendix C, and the spot check in appendix D.

In the column and shear wall check my calculations resulted in the need for less reinforcement than what was originally designed into the building. This is most likely caused by my simplified assumptions to be able to design everything by hand in a reasonable time frame. My calculation of the slab, however, required more reinforcing than the real design. I attribute this to the challenges I had trying to follow the procedure layed out in *Design of Post-Tensioned Slabs,* Post-Tensioning Institute, 2nd edition.

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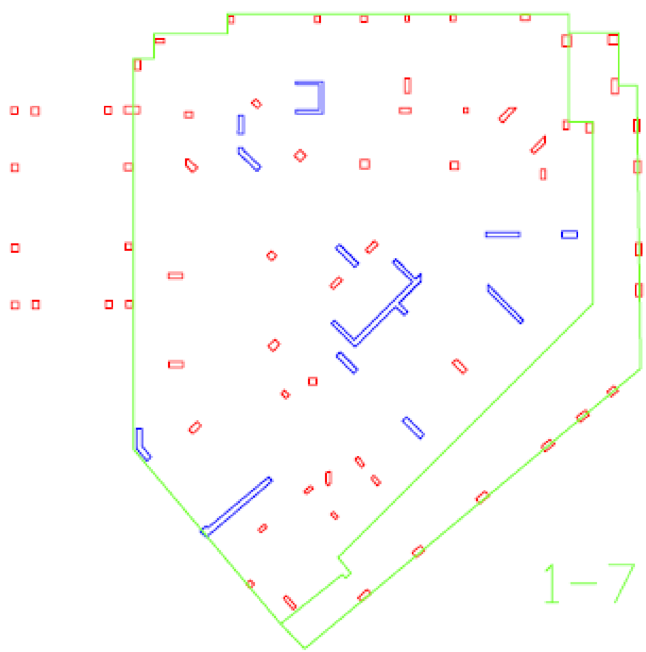
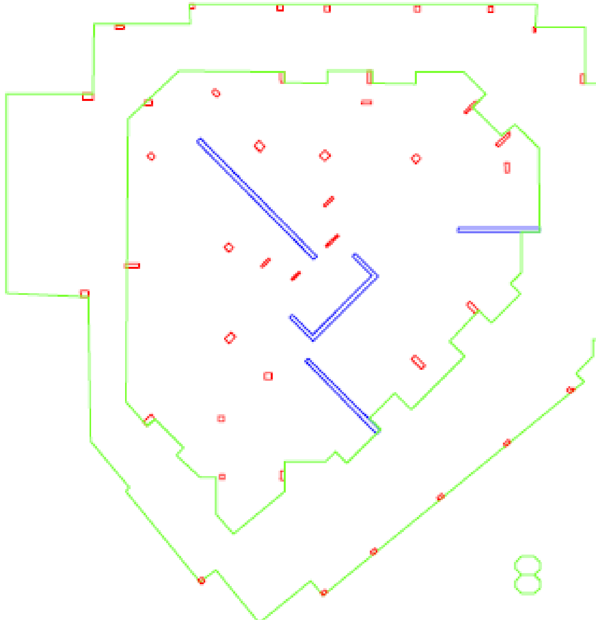
Structural System

The overall structure of the Metropolis at Dadeland Condominium towers is entirely concrete. The foundation and columns are reinforced concrete, and the slabs are post-tensioned concrete. The foundation is a 5' thick Mat that sits atop auger-cast bearing piles that reach 31'6" to 45'6" below the bottom of the mat with a minimum of 5' rock socket. The grout used is 7500 psi in the 24" piles and 6500 psi in the 16" piles. The columns and shear walls are primarily 10,000 psi up to level 4, 8,000 psi up to level 8, 6,000 psi up to level 21, and 5,000 psi up to the roof. The post-tensioned floor slabs consist of 5000 psi concrete and are 8" in the parking deck and 9.5" thick in residential areas. The CMU walls are typically reinforced with #7 bars at 32" o.c. and have an f'c=1500 psi.

The only code applicable to the structure is the Florida Building Code (FBC), since it encompasses every aspect of the building. The loading is based on ASCE 7-98, and concrete design comes from ACI 318-99. The assumed superimposed dead loads are 20 psf in the living units and 65 psf (vertical) from CMUs. The assumed superimposed live loads are 40 psf in the units, 60 psf on balconies, and 100 psf in stairs and public areas.

The framing is irregular since it must mimic the unique shape of the building and the fact that the shape of the building changes as it goes higher. There are columns that rotate by 45˚ at the eighth floor.

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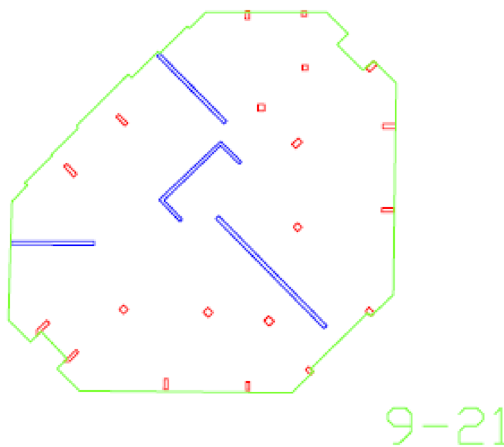
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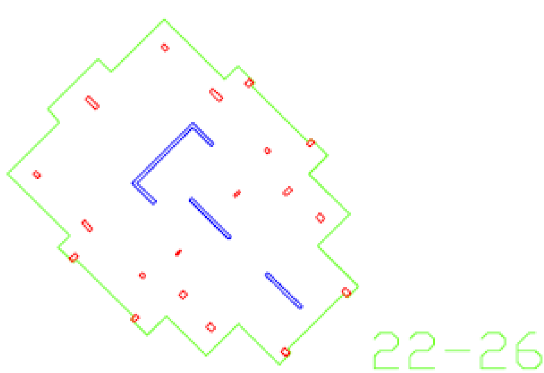
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Lateral Loading / Lateral Resisting System

The fact that shear walls are relied on for lateral load resistance makes the analysis of the rest of the structure easier since it is not expected to be supporting the bending moments from the entire wind load. The greatest ease of this is that I will have to do less with post-tensioned floors in irregular bending patterns. The floors are act as a rigid diaphragm in lateral load distribution.

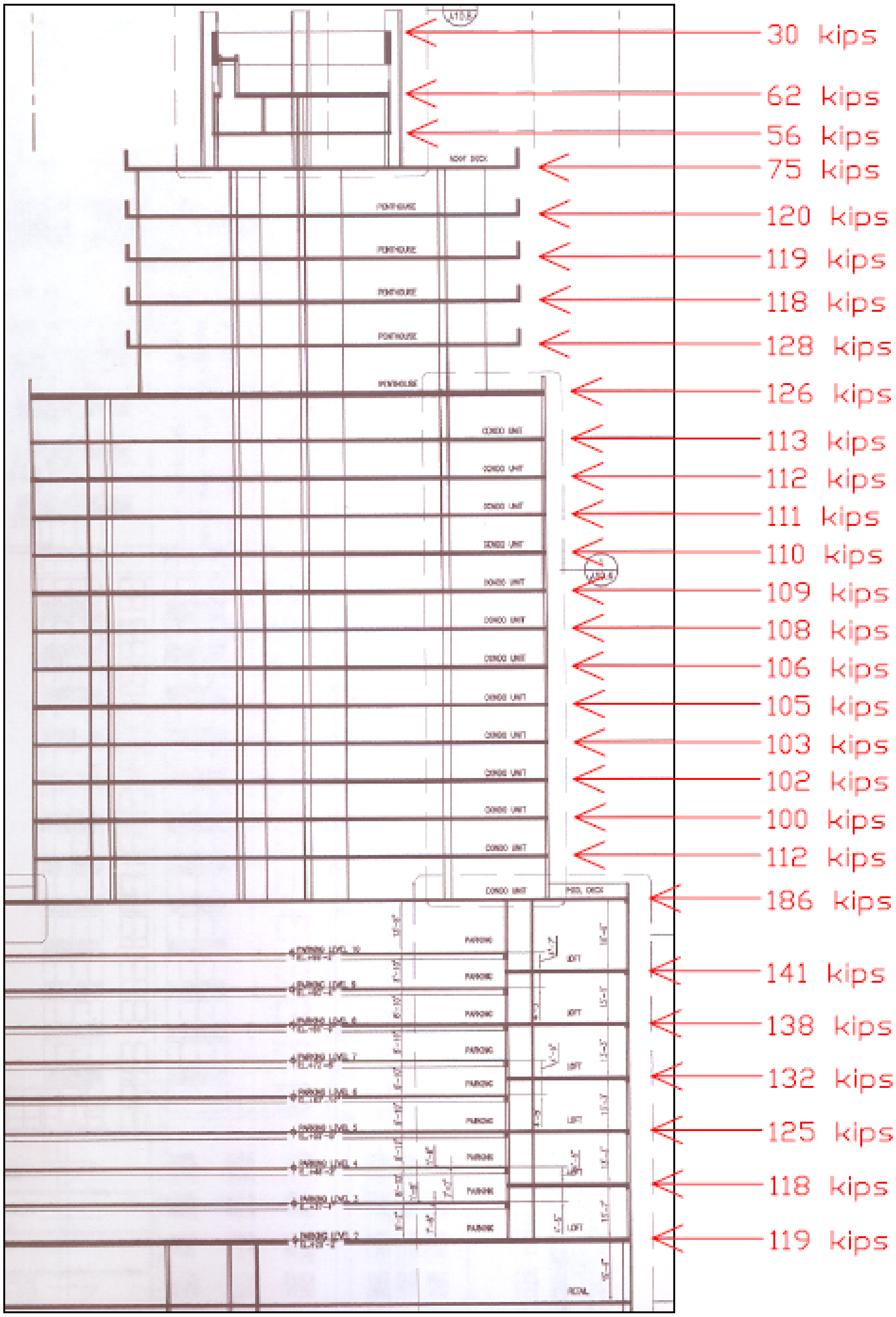
Seismic loads were never calculated in the design of the structure because through very brief inspection as well as local experience the designer knew that wind loads easily controlled. My calculation of wind loads should be slightly different; they should be higher, than what was used in design because the designers used a wind tunnel to test the actual loads on the structure. Overall, my calculations should be more conservative.

Wind vs Seismic Loading

		Wind		Seismic	
	Shear (E-W)	Shear (N-S)		Shear	(accumulated values to that level)
ground	3083.13	2292.01		1974	
2	3083.13	2292.01		1971	
3	2963.75	2179.60		1964	
4	2845.77	2068.51		1950	
5	2721.23	1951.25		1930	
6	2589.37	1827.10		1903	
7	2451.45	1697.24		1867	
8	2310.24	1564.28		1817	
9	2124.06	1445.72		1770	
10	2011.31	1373.93		1717	
11	1911.03	1310.07		1659	
12	1809.16	1245.20		1593	
13	1705.78	1179.37		1512	
14	1600.98	1112.64		1442	
15	1494.82	1045.04		1356	
16	1387.37	976.61		1263	
17	1278.68	907.40		1162	
18	1168.80	837.43		1053	
19	1057.78	766.73		937	
20	945.66	695.33		813	
21	832.47	623.26		678	
22	706.15	532.10		594	
23	578.48	439.96		510	
24	460.66	354.93		420	
25	341.79	269.14		324	
26	221.89	182.61		223	
27	146.92	87.13		133	
28	90.86	57.92		59	
roof	29.02	25.70		0	

Wind E-W clearly controls design

ESE wind is the most critical direction.



These Loads were then applied to the shear walls. The loads were distributed by relative rigidities among the shear walls that ran in the same direction as wall A, which is the one that was checked. Since wall A lies very near the centroid of the structure torsional forces were ignore for simplification. They would not be a great variation in load due to the lack of eccentricity on wall A.

floor #	On SW "A" (k)
1	0.0
2	25.1
3	24.8
4	26.2
5	27.7
6	29.0
7	29.7
8	39.1
9	47.4
10	42.1
11	42.8
12	43.4
14	44.0
15	44.6
16	45.1
17	45.7
18	46.1
19	46.6
20	47.1
21	47.5
22	53.1
23	80.4
24	74.2
25	74.9
26	75.5
27	47.2
28	24.1
29	26.6
30	17.4

I analyzed this shear wall at level 10. The results yielded a need for # 6 bars spaced at 12 inches running horizontally and #5 bars spaced at 12 inches running vertically. This is a little less reinforcing in the horizontal direction and a little more reinforcing in the vertical direction than what is shown in shear wall detail done by the engineer on this building. Our actual capacities are probably very similar. Calculations can be found in appendix C.

Spot Check

Post-Tensioned Slab Check

Floor 6	Loft Space			Cols 9&10		
fpc	175	psi		L = 36'	36	L
Fe	19.95			w = 25'-5"	25.42	w
force/tendon	24.33				9.5	t
# tendons	20.91					
use	21					
Fe	20.034					
fpc	0.18	OK				
				Self	118.75	
				Super		
Tendon Profile				Dead	30	
a	3.5			live	40	
Wbal	0.04			w (unfact)	0.19	ksf
Wnet	0.15					

This Force of 24.33 kips in 21 tendons yields a total force of 511 kips. From here I designed slab with regular reinforcing based on the Wnet that remained after the application of these tendons. This resulted in 25 #8 bars in the bottom, 15 #6 bars in the top at the exterior columns, and 19 #6 bars in the top at the interior column.

The preliminary result is within a reasonable value of the actual design of the building. The post-tensioning, with the same geometry, has a force of 594 kips. The rebar is 30 #4 bars in the bottom, 15 #4 bars in the top at the exterior support, and 5 #5 bars in the top at the interior support. The discrepancies are most likely due to the fact that the actual building was design using computer models and programs, and my numbers are based on an attempt to follow an example from *Design of Post-Tensioned Slabs*, Post-Tensioning Institute, 2^{nd} edition. The results of this attempt as well as the rebar calculations can be found in appendix D.

Column Check

My design of the column resulted in the need for 14 #7 bars which would be places along the two long sides of the column and #3 ties at 18 inches. This is noticeably less than was designed into the actual column. Most of the reinforcing is likely required to bending in the weak direction since very little was needed in the strong direction, and none needed for the pure axial load. However, since the spaces were symmetrical my analysis did not yield any moment in that direction. It is likely that some unbalanced condition would have attributed to that design. The differential in lateral ties can also be attributed to the fact that the designer of the actual structure has a uniform tie schedule based on the rebar size that is uniform across the building.

Conclusions

My building has a rather unique structure. To begin with the overall shape is roughly triangular. This leads to the lateral load resisting system, shear walls since this is an entirely concrete structure, running in many different directions. They do primarily run either parallel or perpendicular to the larges face, but that does not always hold true, nor does the same side of the building maintain as the largest face throughout the height of the structure. The columns also refuse to run in any completely regular pattern since they were more determined by architectural feasibility than by engineering simplification. Most of the floors are also post-tensioned, all but the ground and 22^{nd} floor, which adds to the complexity of checking a portion of the building by hand.

After analyzing both wind and seismic loading on the structure the wind load easily controlled design, as should be expected in a hurricane prone and seismically inactive region such as Miami, Florida, where the building is located. This is so often the case in Miami that seismic was never calculated for the building in reality. It is also hard to compare wind loading numbers since the wind loads were actually the result of wind tunnel testing instead of using a design method such as ASCE 7-98. My calculations of shear distribution were slightly simplified which may have lessened the design requirements of the shear wall I checked. Also, a major reason for the difference is that the shear wall was designed to remain prismatic and match up other portions of that wall, which were not continuous on many other levels.

The spot checks of the slab and column also had their difficulties. Trying to design post-tensioning by hand proved to be very difficult. The challenge with the column was what to do with weak axis bending. While in the strong direction design was controlled by minimum reinforcement, the weak direction may be controlled by an imbalance in slab loads, but the space around the column is effectively uniform, which minimizes any moment in that direction. Once again this may have also been an over-designed member in order to achieve structure simplification.

Enough of my calculations are close enough to what was actually put into the building for me to be fairly comfortable with my results. If I were to have made a few less simplifications our designs would likely be even closer.

Appendix A:
Wind Loads

floor #	height	kh	kz	alpha	zg	qz	qh	P (psf) (E-W)	F (klf) (E-W)	P (N-S)	F (N-S)	Story Shear (N-S)	Story Shear (E-W)
1	0.00	0.85	0.00	9.5	900	0.00	78.80	0.00	0.00	0.00	0.00	3447.30	3502.93
2	15.67	0.90	0.86	9.5	900	41.95	78.80	41.18	0.65	40.53	0.64	3447.30	3502.93
3	29.25	0.98	0.98	9.5	900	47.84	78.80	46.96	0.64	46.22	0.63	3334.90	3388.71
4	42.50	1.05	1.06	9.5	900	51.75	78.80	50.81	0.67	50.00	0.66	3223.81	3275.83
5	55.75	1.11	1.12	9.5	900	54.79	78.80	53.79	0.71	52.94	0.70	3106.55	3156.68
6	69.00	1.16	1.17	9.5	900	57.31	78.80	56.26	0.75	55.37	0.73	2982.39	3030.52
7	82.08	1.22	1.21	9.5	900	59.44	78.80	58.36	0.76	57.43	0.75	2852.54	2898.57
8	100.08	1.26	1.27	9.5	900	61.98	78.80	60.84	1.10	59.88	1.08	2719.58	2763.46
9	110.75	1.28	1.29	9.5	900	63.31	78.80	62.16	0.66	61.17	0.65	2528.80	2569.61
10	120.08	1.31	1.32	9.5	900	64.40	78.80	63.22	0.59	62.22	0.58	2413.28	2452.22
11	129.41	1.33	1.34	9.5	900	65.42	78.80	64.23	0.60	63.21	0.59	2310.53	2347.81
12	138.74	1.35	1.36	9.5	900	66.39	78.80	65.18	0.61	64.14	0.60	2206.15	2241.75
14	148.07	1.37	1.37	9.5	900	67.30	78.80	66.08	0.62	65.03	0.61	2100.23	2134.12
15	157.40	1.38	1.39	9.5	900	68.17	78.80	66.93	0.62	65.87	0.61	1992.84	2025.00
16	166.73	1.40	1.41	9.5	900	69.01	78.80	67.75	0.63	66.67	0.62	1884.07	1914.47
17	176.06	1.42	1.43	9.5	900	69.80	78.80	68.53	0.64	67.44	0.63	1773.97	1802.59
18	185.39	1.44	1.44	9.5	900	70.56	78.80	69.28	0.65	68.18	0.64	1662.60	1689.42
19	194.72	1.45	1.46	9.5	900	71.30	78.80	70.00	0.65	68.89	0.64	1550.01	1575.02
20	204.05	1.46	1.47	9.5	900	72.00	78.80	70.69	0.66	69.57	0.65	1436.25	1459.43
21	213.38	1.48	1.48	9.5	900	72.68	78.80	71.36	0.67	70.23	0.66	1321.37	1342.69
22	225.05	1.49	1.50	9.5	900	73.50	78.80	72.16	0.84	71.02	0.83	1205.39	1224.85
23	236.72	1.51	1.52	9.5	900	74.29	78.80	72.93	0.85	71.78	0.84	1058.70	1075.79
24	247.39	1.53	1.53	9.5	900	74.98	78.80	73.62	0.79	72.45	0.77	910.44	925.13
25	258.06	1.53	1.55	9.5	900	75.65	78.80	74.27	0.79	73.09	0.78	773.62	786.10
26	268.73	1.55	1.56	9.5	900	76.30	78.80	74.91	0.80	73.72	0.79	635.58	645.83
27	280.40	1.56	1.57	9.5	900	76.99	78.80	75.58	0.88	74.38	0.87	496.35	504.36
28	289.07	1.58	1.58	9.5	900	77.48	78.80	76.07	0.66	74.86	0.65	342.71	348.24
29	298.57	1.59	1.59	9.5	900	78.01	78.80	76.59	0.73	75.37	0.72	227.83	231.51
30	313.57	1.60	1.61	9.5	900	78.82	78.80	77.38	0.58	76.15	0.57	101.09	102.72

Appendix B: Seismic

Building Location :			**Miami, FL**		
Number of Stories :	N		28		
Inter-story Height	h_s		11	ft	
Building Height :	h_n		313	ft	
Seismic Use Group :	I		I		Residential
Occupancy Importance Factor :			1.00		
Site Classification :			C		
0.2s Acceleration :	S_S		0.06	g-s	
1s Acceleration :	S_1		0.02	g-s	
Site Class Factor :	F_a		1.20		
Site Class Factor :	F_v		1.70		
Adjusted Accelerations :	S_{MS}	$= F_a S_S$	0.072	g-s	
	S_{M1}	$= F_v S_1$	0.037	g-s	
Design Spectral Response Accelerations :	S_{DS}	$= (2/3)S_{MS}$	0.048	g-s	
	S_{D1}	$= (2/3)S_{M1}$	0.025	g-s	
Seismic Design Category :			**A**		
			Equivalent Lateral Load Method can be used		

a. Seismic Base Shear Coefficient (9.5.3.2)

i. N-S Direction

Response Modification Factor :	R_{N-S}		4
Seismic Response Coefficient :	$C_{s,\,N-S}$	$= S_{DS}/(R_{N-S}/I)$	**0.012**
	$C_{T,\,N-S}$		0.02
	x		0.75
Approximate Period of Structure :	T_{N-S}	$= C_{T,\,N-S} h_n^{\,x}$	1.49
but Seismic Response Coefficient need not be greater than	$C_{S\,max,\,N-S}$	$S_{D1}/T(R_{N-S}/I)$	**0.004**
and	$C_{S\,min}$	$= 0.044 I S_{DS}$	**0.0021**
		Therefore, the Seismic Response Coefficient ($C_{s,\,N-S}$) used is	**0.004**

ii. E-W Direction

Response Modification Factor :	R_{E-W}		4
Seismic Response Coefficient :	$C_{s,\,E-W}$	$= S_{DS}/(R_{E-W}/I)$	**0.012**
	$C_{T,\,E-W}$		0.02
	x		0.75
Approximate Period of Structure :	T_{E-W}	$= C_{T,\,E-W} h_n^{\,x}$	1.49
but Seismic Response Coefficient need not be greater than	$C_{S\,max,\,E-W}$	$S_{D1}/T(R_{E-W}/I)$	**0.004**
and	$C_{S\,min}$	$= 0.044 I S_{DS}$	**0.0021**
		Therefore, the Seismic Response Coefficient ($C_{s,\,E-W}$) used is	**0.004**

Level	height	slab thickness	self weight (psf)	total dead load	floor area (SF)	weight (k)	perimeter (ft)	w/ walls (k)
1	7.84	5.00	62.50	82.50	30,000	2,475	600	41,475
2	6.79	8.75	109.38	129.38	30,000	3,881	630	44,831
3	6.63	8.75	109.38	129.38	30,000	3,881	630	44,831
4	6.63	8.75	109.38	129.38	30,000	3,881	630	44,831
5	6.63	8.75	109.38	129.38	30,000	3,881	630	44,831
6	6.54	8.75	109.38	129.38	30,000	3,881	630	44,831
7	9.00	8.75	109.38	129.38	30,000	3,881	630	44,831
8	5.34	13.00	162.50	252.50	30,000	7,575	600	46,575
9	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
10	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
11	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
12	4.66	8.00	100.00	120.00	14,000	1,680	550	37,430
13	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
14	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
15	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
16	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
17	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
18	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
19	4.67	8.00	100.00	120.00	14,000	1,680	550	37,430
20	5.84	8.00	100.00	120.00	14,000	1,680	550	37,430
21	5.83	8.00	100.00	120.00	14,000	1,680	550	37,430
22	5.33	22.00	275.00	295.00	7,000	2,065	300	21,565
23	5.34	8.00	100.00	120.00	7,000	840	300	20,340
24	5.34	8.00	100.00	120.00	7,000	840	300	20,340
25	5.83	8.00	100.00	120.00	7,000	840	300	20,340
26	4.34	8.00	100.00	120.00	7,000	840	300	20,340
27	4.75	8.00	100.00	120.00	7,000	840	250	17,090
28	7.50	10.00	125.00	145.00	3,500	508	200	13,508
29	7.50	9.00	112.50	132.50	2,000	265	150	10,015
								987,165

Exponent $k_{N\text{-}S}$ = 1+ ($T_{N\text{-}S}$ - 0.5)/(2.5 - 0.5) = 1.494

			(k)	(k)	(ft-k)
	$w_x h_x^{\ k}$	C_{vx}	F_x	V_x	M_x
1	0	0.0000	0.00	1974.33	0.00
2	2736456.929	0.0015	3.01	1971.32	47.19
3	6953239.81	0.0039	7.65	1963.67	223.83
4	12151557.26	0.0068	13.37	1950.29	568.36
5	18227451.11	0.0102	20.06	1930.23	1118.34
6	25066297.03	0.0140	27.59	1902.65	1903.45
7	32488619.47	0.0181	35.75	1866.89	2934.75
8	45390411.87	0.0253	49.95	1816.94	4999.36
9	42439298.52	0.0237	46.71	1770.23	5172.67
10	47890898.64	0.0267	52.71	1717.53	6328.87
11	53556044.11	0.0299	58.94	1658.59	7627.44
12	59426791.95	0.0331	65.40	1593.19	9073.75
13	65496019.24	0.0365	72.08	1521.11	10672.95
14	71757291.15	0.0400	78.97	1442.13	12430.07
15	78204756.91	0.0436	86.07	1356.07	14349.92
16	84833066.57	0.0473	93.36	1262.71	16437.23
17	91637303.47	0.0511	100.85	1161.86	18696.54
18	98612928.74	0.0550	108.53	1053.33	21132.32
19	105755735.4	0.0590	116.39	936.94	23748.88
20	113061809.7	0.0630	124.43	812.51	26550.48
21	122424551.3	0.0682	134.73	677.78	30321.47
22	76068268.77	0.0424	83.72	594.07	19817.15
23	76632627.55	0.0427	84.34	509.73	20864.05
24	81623308.54	0.0455	89.83	419.90	23181.29
25	86717027.5	0.0483	95.43	324.47	25646.21
26	92403643.49	0.0515	101.69	222.77	28514.76
27	81253169.87	0.0453	89.42	133.35	25849.13
28	67399354.73	0.0376	74.18	59.18	22146.47
roof	53769946.54	0.0300	59.18	0.00	18555.68
sums	1793977876	1.00	1974.33		398912.59

Appendix C

Shear Wall

Simplified Load Distribution: I assumed all walls have same thickness, height, and material strength on each floor, which means that length and orientation will be the only factor that will affect the amount of load each shear wall carries.

SW "A" Loading

@ Roof:
A = 15'
B = 10' => A carries 60%

@ 28:
A = 15'
B = 20' => A carries 43%

@ 23–27: A = 34.75'
B = 20' => A " 63%

@ 9–22:
A = 31.58'
B = 20'
C = 31.65' => 42%
E = 28.08 @ 45° = 19.85'

@ 1–8:
A = 24.5' => 21%
B = 20'
C = 15.3'
D = 15.8'
E (45°) = 14.92' => 10.55'
F (45°) = 26.25' => 18.92
G (5°) = 26.92' => 2.35'
H (70°) = 11' => 10.34'

The strength was then analyzed at level 10:

Shear Wall "A"

$V_u = 970^k$

$V_c = 2\sqrt{f'_c}\,hd = 2\sqrt{5000}\,(16)(41.3) = 93^k$

$\cancel{\phi V_c = 953^k}$ $\phi V_c = 79.4$

$V_s = V_u - V_c = 890^k$

$S_{max} = 18''$

$A_v = \frac{V_s s}{\phi f_y d} = \frac{890(18)}{(0.9)(60)(41.3)(12)} = 0.60''^2/$

$\rho = \frac{.60}{1554''^2} = 0.000386 \neq 0.0025$

$A_v = .0025 \cdot 1554''^2 = 3.9''^2$

112" tall... 18" spacing => 6 bars => #8 @ 18"

12" spacing => 9 bars => $\boxed{\#6 @ 12''}$

$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_h - .0025)$

$= .0025 + 0.5\left(\left(2.5 - \frac{9.33}{51.6'}\right)(.00255 - .0025)\right)$

$= .0025 + .00058$

$\rho_n = 0.00256$

$\rho = \frac{A_s}{Area} = \frac{.11}{(8)(12)} = .00115$ ×

$\frac{.31}{8(12)} = 0.0032$

$\boxed{\#5 @ 12''}$ Vertical

Appendix D

Post Tensioned Slab Design

I col 9	17749.3	in^4			I =	13.17	ft
I col 10	21384	in^4					
Kc 9	6127.5		∑Kc =	12255.01			
Kc 10	7382.3		∑Kc =	14764.56			
Is	21794.5						
Ks (9)	8717.8						
Ks (10)	6227.0						

Slab Distribution Factors

at 9	0.416	
at 10	0.297	
FEM =	16.4895	ft-k

Moment Distribution

	9		10	
Ends	9-c	9-10	10-9	10-c
DF	0.584	0.416	0.297	0.703
FEM	0	-16.675	16.675	0
		16.675		
	9.74	6.94	3.47	
		-2.99	-5.98	-14.16
	1.75	1.24	0.62	
			0.184	0.436
		-8.5	10.9	

Net Stress Check

Mmax =	-8.50	ft-k
S =	180.50	in^3
ft =	-0.74	ksi
6√f'c =	0.42	ksi
X		
fc	0.12790312	
.6f'c	3	
√		

midspan

M	19.68	ft-k
ft	1.30	ksi
2√f'c =	0.14	
X		
fc	1.32	ksi
y	4.71	in
Nc	36.66	k/ft
As	1.22	in^2/ft
	#7 @ 6"	for tensile control
length	10.67	ft

These results were not yielding rational numbers.

Other Reinforcing in Slab

$\omega = 0.208$ ksf

$d = 9.5'' - \underset{\text{cover}}{1.5''} - \underset{\text{centroid of bar}}{\tfrac{1}{2}''} = \underline{7.5''}$

$M_{int} = 8.5'^{k}$

$M_{ext} = 10.9'^{k}$

$M_{mid} = 24'^{k}$

$A_s = \dfrac{M_u}{\phi f_y (d - a/2)} = \dfrac{24'^k \times 12''/'}{(0.9)(60)(6.75)} = 0.79''^2/ft$

$a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{(.79)(60)}{.85(5)(12)} = \underline{.929''}$

$A_s = \dfrac{288}{54(7.5 - \frac{.929}{2})} = 0.76''^2/ft$

$25'-5''$ slab $\Rightarrow \underline{19.4''^2}$ $\boxed{25\#8 \text{ bars}}$ Bottom

$A_s = \dfrac{8.5'^k(12)}{(0.9)(60)(6.75)} = 0.28''^2$

$a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{.28(60)}{.85(5)(12)} = 0.33''$

$A_s = \dfrac{8.5(12)}{.9(60)(7.5 - \frac{.33}{2})} = \underline{0.26''^2/ft}$

$6.5''^2 \Rightarrow \boxed{15\#6 \text{ bars}}$ Top Interior

$A_s = \dfrac{10.9(12)}{.9(60)(6.75)} = .36''^2$

$a = \dfrac{.36(60)}{.85(5)(12)} = .424''$

$A_s = \dfrac{10.9(12)}{.9(60)(7.5 - \frac{.424}{2})} = \underline{0.33''^2/ft}$

$8.4''^2 \Rightarrow \boxed{19\#6 \text{ bars}}$

Top Exterior

Then Check the Column Supporting the Slab:

Column Check

Column 9 @ Level 6

Levels 22-26: $A = 24' \times 24' = 576'^2$

22: $LL = 45_{psf}$ — $F_u = 743^k$ — $(1.2 + 1.6)$ 666

$DL = 20 + 275 = 295_{psf}$

23-26: $LL = 45_{psf}$

$DL = 120_{psf}$

Levels 8-21: $A = 24' \times 28' = 672'^2$

9-21: $LL = 45_{psf}$

$DL = 120_{psf}$ — $F_u = 1741 + 244^k = 1985^k$ — $(1.2 + 1.6)$ 1702

8: $LL = 162_{psf}$

$DL = 87_{psf}$

Levels 6-7: $A = 25.5' \times 36' = 915'^2$

$LL = 40_{psf}$

$DL = 170_{psf}$ — $F_u = 245^k$ — $(1.2 + 1.6)$ 223

$(1.2D + 1.6L)$

Axial Load = 2973^k

Moment = $0'^k$

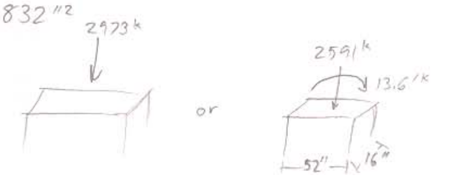
$(1.2D + 1.0L + 1.6W)$

Axial Load = 2591^k

Moment = $13.6'^k$

(based on Moment Distribution above)

$A_{col} = 52'' \times 16'' = 832''^2$



$f'_c = 8k$..

$E = 4.2$ ksi

$f_c = 0.85 f'_c bh$

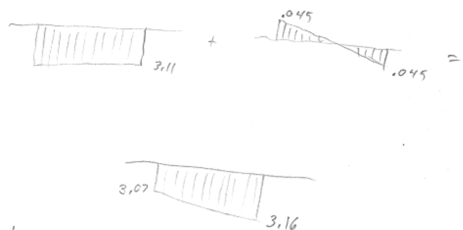
$= 0.85(8)(832) = \underline{5657^k} \overset{\phi}{(0.65)} = 3677^k > 2973^k$

OK in pure axial

$d = 52'' - 1.5'' - \cancel{1/2''} - 1/2'' = 49.5''$

$f_b = \frac{M}{S} = \frac{My}{I} = \frac{13.6'^k (52)(12''/')}{187477.3} = 0.045$ ksi

$f_c = \frac{F}{A} = \frac{2591^k}{832''^2} = 3.11$ ksi



3.16 ksi < 3.57 ksi

(pure axial condition)

$A_{s\,min} = 0.01 A_g$

$= (0.01)(832) = 8.32''^2$

$\boxed{14\ \#7\ \text{bars}}$ $\quad A_s = 8.4''^2$

#3 ties @ 18" to hold rebar in place