STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

Background

1100 Broadway's current floor system is composite metal deck supported by composite steel beams. The assembly consists of a 3", 18 gage, W3 Verco Formlok deck with 3 ¼" lightweight concrete topping for a total slab depth of 6 ¼". The controlling parameter for the design of gravity members supporting the composite deck is deflection due to total load as determined in Technical Report 2. This required the member capacity to be significantly higher than the gravity load demands. The total depth of the composite metal deck system and supporting composite steel beams and girders amounts to 30.25". After investigating alternative types of floor systems it's been determined the depth of the floor system can be reduced.

Solution

Technical Report 2 provided an alternative system study of a 2-way post-tensioned concrete slab. The analysis yielded a 9" total system depth, reducing the current floor depth by approximately one-third. Another advantage of post-tensioned systems is very limited deflections due to the upward force exerted by the post-tensioning tendons. With closer observation, the rectangular geometry of most bays will result in a one-way behavior. Therefore, a one-way mild steel reinforced concrete slab with post-tensioned concrete beams was proposed for study. Concrete gravity columns were designed in place of the current steel columns.

A post-tensioned slab was not considered for study. A post-tensioned slab system would be very costly especially due to 1100 Broadway's 20-story building and therefore it is more economical to post-tension only the beams and have a mild steel reinforced slab. Another disadvantage of a post-tensioned slab is opening locations are critical, limiting the placement of openings throughout the entire structure. Openings locations for a mild steel reinforced slab are not nearly as critical and can accommodate most plans.

The one-way slab and post-tensioned beam system will most likely be deeper than the 2-way post-tensioned slab previously studied, but the depth of the floor system should still be significantly reduced. Although the floor system depth will be reduced, concrete systems are usually heavier than steel systems and the impact of the proposed system on the foundations was also investigated. The current lateral system of steel moment and braced frames is no longer a viable system for the proposed concrete slab and post-tensioned beams. A change of lateral system was necessary and concrete shear walls make for the best alternative lateral system due to the 20-story height of the building.

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Senior Thesis Project Goals

One goal of my senior thesis project is to reduce the depth of the floor system. This could have many economical benefits such as reduced floor to floor height amounting in an overall reduction in building height and potential savings related to the facade and building envelope. The second goal was to become familiar with the design of post-tensioned systems.

Building Relocation

Early in the spring semester it was brought to my attention that my original thesis proposal to design a concrete system with shear walls for the 260 ft tall 1100 Broadway in Oakland, California, was not a feasible option. According to ASCE 7-05, Table 12.2-1 and section 12.2.5.4, special reinforced concrete shear walls are limited to structures of 240 ft or less in locations corresponding to Seismic Design Category D. By moving the building out of Seismic Design Category D to a location with less seismic activity, the building height is no longer limited.

Therefore, 1100 Broadway will be designed for relocation in Columbus, Ohio, which corresponds to Seismic Design Category B. Ordinary reinforced concrete shear walls are permitted for the seismic force-resisting system. The site selection is somewhat arbitrary. The only goal was to remove the building from a Seismic Design Category D location. This change allows for a focus on the post-tensioning design of the gravity system rather than heavy seismic detailing of the lateral system.

MAE Topics

An ETABS model was created to analyze the new lateral composed of ordinary reinforced concrete shear walls arranged around the core of the building. The lateral analysis section details the ETABS model's role in the design process. This portion of the study is an extension of AE 597A, Computer Modeling, and is intended to fulfill the MAE requirement for the senior thesis project.

The breadth studies focus on the complete design of a green roof system and are an extension of AE 542, Building Enclosures. They are also intended to fulfill the MAE requirement.

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Design Criteria

Design Loads

A superimposed dead load of 20 psf for mechanical systems, floor finishes, and other miscellaneous loads was used in calculations. A live load of 80 psf for office floors and a roof live load of 20 psf were used in the design. ASCE 7-05 requires a minimum live load of 100 psf for lobbies and first floor corridors and a live load of 80 psf for corridors above the first floor. Typical floors are open office plans with no designated corridors and therefore a live load of 80 psf was used in calculations in lieu of the 50 psf office load to be conservative since partition layout in the offices is subject to change.

Software

PCA Slab was used to check deflections and design reinforcing for the one-way mild steel reinforced slab. PCA Column was used to design column reinforcing and confirm shear wall reinforcing designed by hand methods. RAM Concept was the only software program available capable of post-tension design and was used to model the post-tensioned beams. An ETABS model of the lateral system was created to assist with the drift analysis. It was necessary to use a variety of software programs because no program was capable of modeling the entire structural system as one entity. Only components of the structural system could be modeled or designed by each program.

Codes

ASCE 7-05 and IBC 2006 were referenced to determine the minimum design loads on the structure. ACI 318-08 was referenced for the design of concrete elements. Each software program refers to a specific edition of the above codes. See Table 1 below for each software program’s use and the applicable code edition it references.

Table 1: Software program use and code reference

Program	Use	Code Edition
PCA Slab	One-way slab design	ACI 318-02
PCA Column	Shear wall reinforcing	ACI 318-02
	Column reinforcing	
RAM Concept	Post-tensioned beam design	ACI 318-02
ETABS	Lateral analysis	ACI 318-05

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Gravity System Design

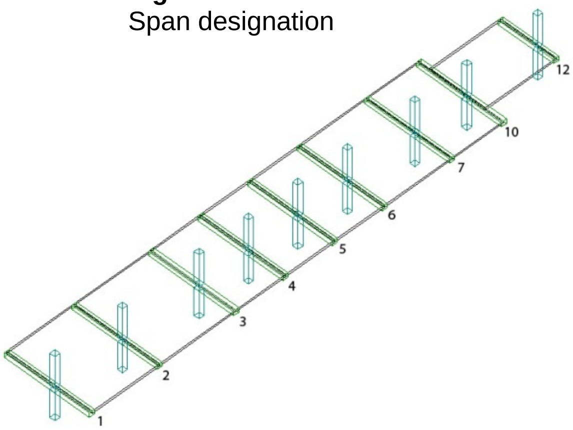
One-way Slab Design

A one-way mild steel reinforced slab was designed and spans the North/South direction. According to ACI 318-05 Chapter 9.5(a) the minimum thickness of one-way slabs unless deflections are calculated is l/24 for slabs with one end continuous and l/28 for both ends continuous. See Table 2 below for minimum thicknesses per span according to ACI. See span designations in Figure 18.

Table 2:
Minimum slab thickness (h) according to ACI

Span	Length (ft)	h min	h min (in)
1-2	27.33	l/24	13.7
2-3	31	l/28	13.3
3-4	20	l/28	8.6
4-5	20	l/28	8.6
5-6	20	l/28	8.6
6-7	27.33	l/28	11.8
7-10	20.95	l/38	9.0
10-12	28.7	l/24	14.4

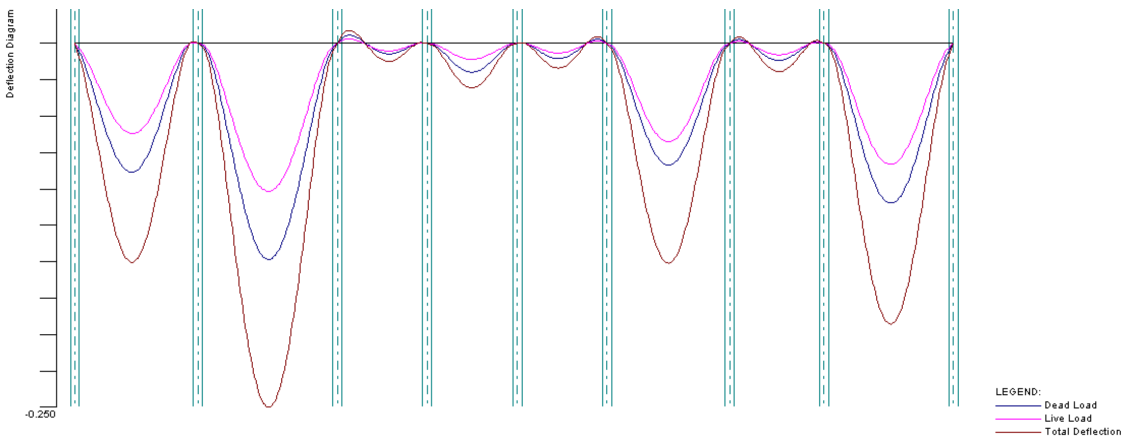
Figure 18:
Span designation



Minimum thicknesses varied significantly from 8.6" to 14.4" and therefore instead of designing a slab with multiple thicknesses or a uniform slab with the minimum 14.4" depth it was beneficial to check deflections with the objective of achieving a more uniform and shallower slab. A 10" slab thickness was chosen for design which is slightly less than the average of the minimum thicknesses in Table 2.

Deflections for the 10" slab were calculated in PCA Slab. See Figure 19 below.

Figure 19: Slab deflections from PCA Slab



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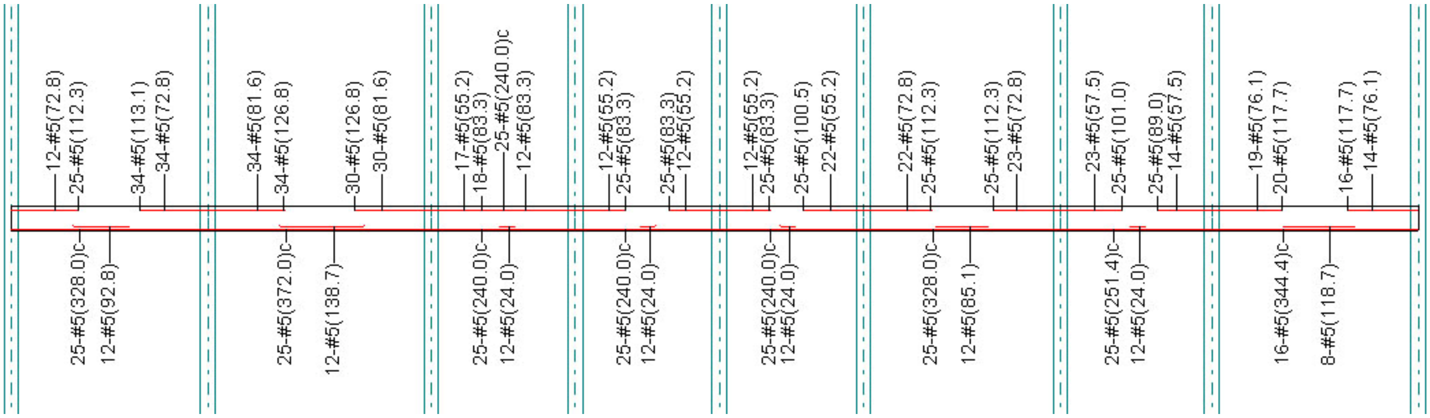
Long term deflections were calculated conservatively by multiplying the deflection due to dead load by 3 and adding it to the live load deflection. This value was compared with l/240 to determine if the 10” slab thickness was sufficient for spans that were less than the minimum thickness according to ACI. See Table 3 for a comparison. All long-term deflections were less than the allowable deflection.

Table 3: Deflection Check

		Deflections from PCA Slab		Long-term deflection	Allowable Δ
Span	Length (ft)	LL Δ (in)	DL Δ (in)	LL Δ + 3DL Δ (in)	l/240
1-2	27.33	0.062	0.088	0.326	1.4
2-3	31	0.102	0.149	0.549	1.6
3-4	20	-	-	-	-
4-5	20	-	-	-	-
5-6	20	-	-	-	-
6-7	27.33	0.067	0.084	0.319	1.4
7-10	20.95	-	-	-	-
10-12	28.7	0.083	0.11	0.413	1.5

An interior column line was modeled in PCA Slab and reinforcing for the 10” slab was designed. See Figure 20 below for reinforcing design. 60ksi reinforcing steel was used with #5 bars being typical for both top and bottom reinforcement.

Figure 20: Slab reinforcing details. Bar length indicated in parenthesis



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Post-tensioned Beam Design

Post-tensioned beams were designed using RAM Concept and span across the column lines in the East/West direction. Post-tensioning applies a precompression to the beams which reduces the tensile stresses that often cause cracking once service loads are applied to the structure. In the original composite steel design deflections controlled the size of the supporting beams and girders. By post-tensioning the beams, deflections can be limited or even eliminated with the combination of service loads and the prestress force exerted by the tendons.

The drape of the tendons can be adjusted to create a vertical force on the beam. The force exerted by the tendon drape along with the applied prestress force creates an upward force on the beam. The best tendon profile is one that exerts an upward force on the beam equal to the downward force of the applied loads. After the concrete has been placed and has achieved a strength of 3000 psi the tendons are tensioned using jacks that react against the beams.

Four floors of 1100 Broadway were chosen to design which are meant to be representative of the entire structure. Level 2 is a non-typical level which acts as a mezzanine to the retail floor below. A floor typical of Levels 3-8 was designed which encompasses the entire footprint of the building. Level 9 features a green roof on the Key System portion and was chosen to design because it sees higher loads than the other typical office floors. Lastly, a floor typical of Levels 10-Roof was designed which covers a reduced floor area as a result of the setback in the geometry of the building.

A trial beam depth was chosen based on a ratio of span length divided by 22. The interior span of 37' is the longest span and based on the ratio of l/22 a trial beam depth of 20" was chosen for the preliminary design. All beams were designed using twelve ½" diameter unbonded tendons. The tendons were encased in a plastic sheathing and greased to prevent them from bonding to the concrete. The tendons are anchored at mid depth of the beam ends. In RAM Concept the tendon drape is measured from the bottom of the beam to the centroid of the tendon group. A 1.5" cover is required on prestressed cast-in-place concrete beams not exposed to weather or in contact with the ground and therefore the tendon profile at mid span of the beams was set at 1.5" and tendon profile over the column supports was set at 18.5".

The Concept model was initially run with the preliminary beam sizes and tendon drape. From the preliminary run the drape of the tendon and beam sizes were adjusted until a successful run was completed. Beams were analyzed as T or L sections to achieve their largest capacity. Many of the beams initially did not meet the serviceability requirements for flexural members according to ACI Chapter 18.4 for prestressed Class T members or they failed in shear according to ACI Chapter 11.4.

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To keep the report clear to read only a sampling of the floor plans will be provided in the body of the report and additional plans can be reproduced upon request. See Figure 21 below for beam locations and designations for a typical office floor for Levels 3-8. Mild-steel reinforced transverse beams 2, 3, and 4 were added (in blue) because the columns did not line up and the span was too long for a single beam (in yellow). Beam dimensions can be seen in Table 4 and 5. See Figure 22 for a perspective view of the floor plan.

Figure 21: Beam designations and locations for typical Levels 3-8 and Level 9

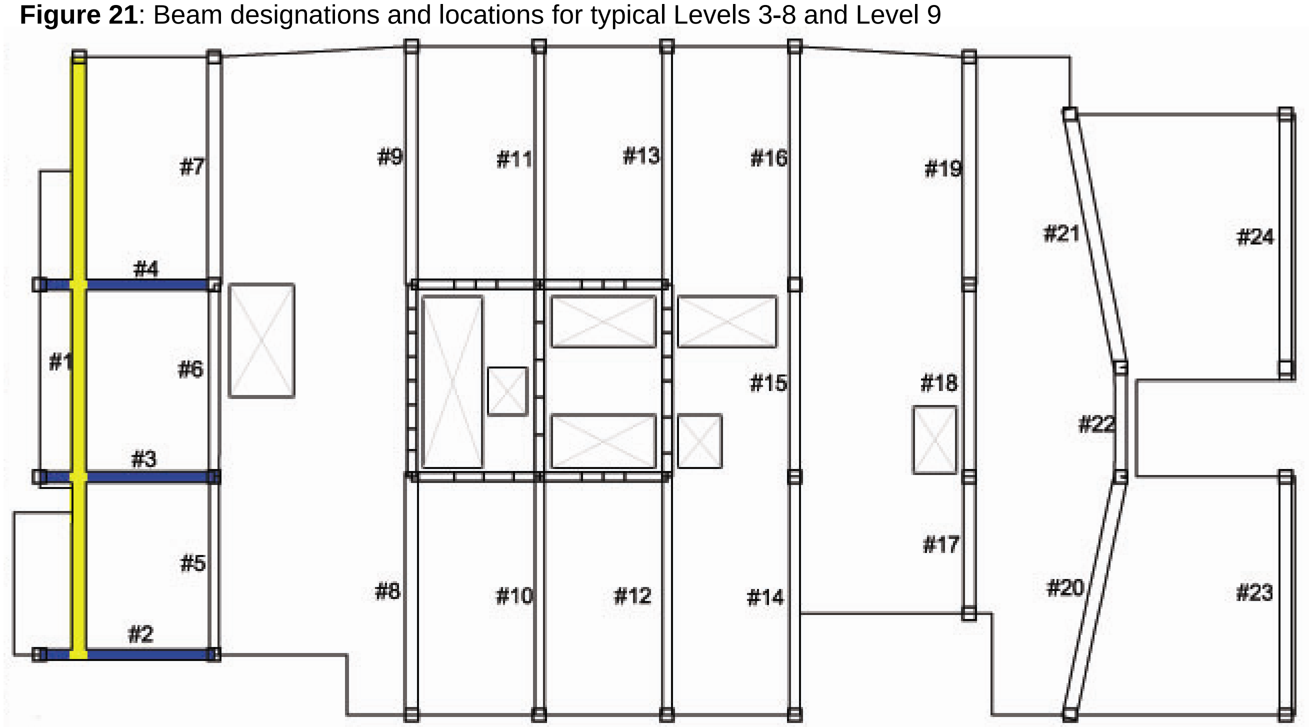

Table 4: Beam sizes for typical Levels 3-8

Number	Thickness (inches)	Width (inches)
1	22	26
2	22	20
3	22	20
4	22	20
5	18	18
6	20	20
7	22	26
8	20	24
9	20	22
10	18	18
11	16	18
12	16	18
13	16	18
14	18	20
15	16	20
16	16	20
17	18	22
18	20	22
19	22	26
20	22	26
21	22	26
22	22	22
23	18	22
24	18	22

Figure 22: Levels 3-8 perspective view

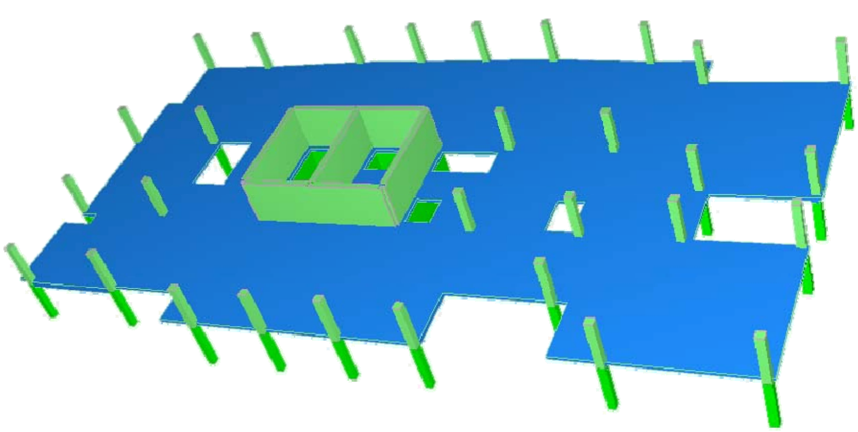

Table 5: Beam sizes for Level 9

Number	Thickness (inches)	Width (inches)
1	22	26
2	22	20
3	22	20
4	22	22
5	18	20
6	18	18
7	22	26
8	22	24
9	22	24
10	18	20
11	16	18
12	16	18
13	16	18
14	18	20
15	16	20
16	16	20
17	18	22
18	18	22
19	22	26
20	24	30
21	24	30
22	22	24
23	18	22
24	22	24

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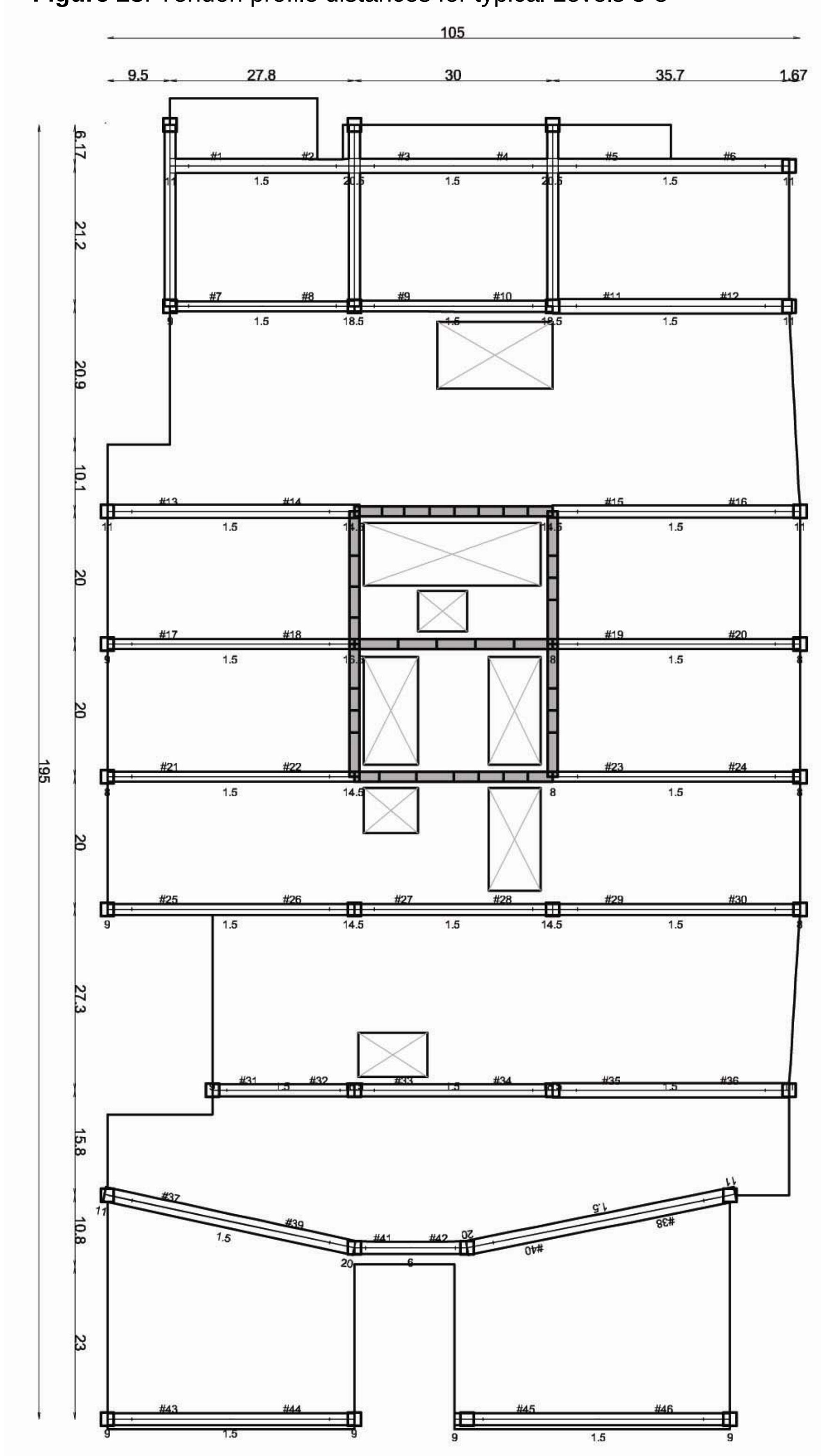
Tendon ends are numbered and their profile distance is given at midpoint of the beams and over supports. See Figure 23 below.

Figure 23: Tendon profile distances for typical Levels 3-8



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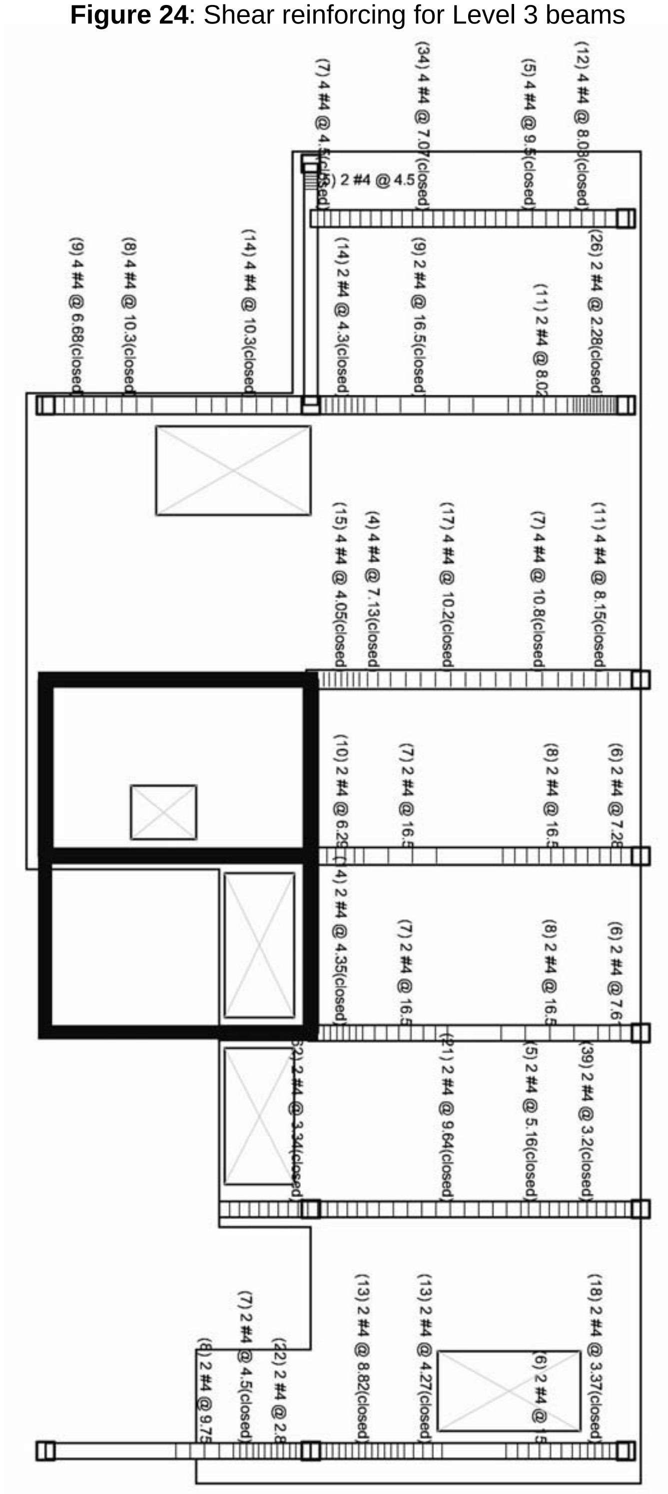
Additional mild-steel was also required for the beams. #4 bars were used for shear and #8 for top and bottom reinforcing when necessary. See Figure 24 below for shear reinforcing for Level 2.

Figure 24: Shear reinforcing for Level 3 beams



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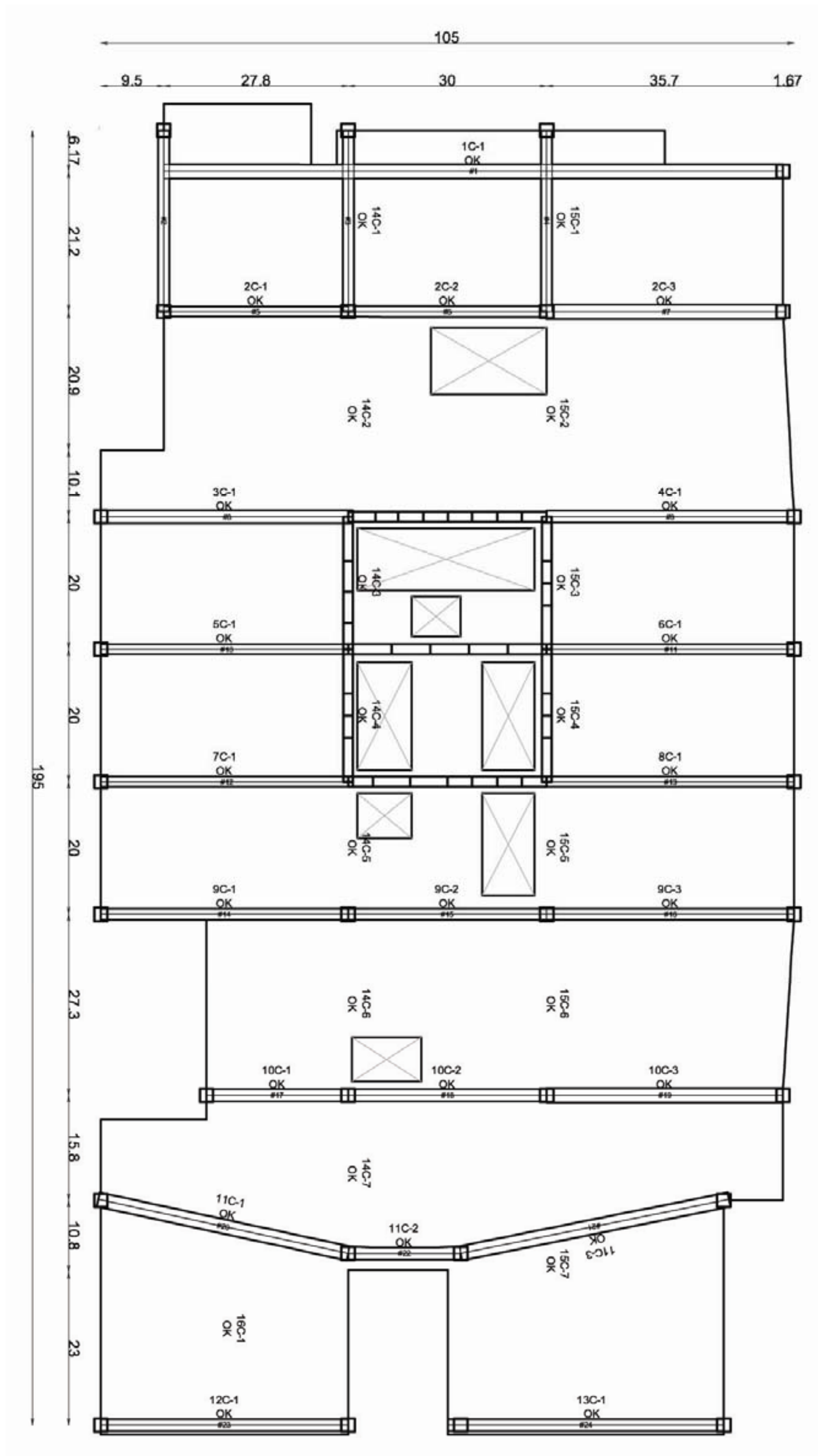
See Figure 25 below for a status plan confirming the post-tensioned beam design meets provisions set forth in ACI 318-02.

Figure 25: Status Plan for typical Levels 3-8



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Column Design

Columns were designed to handle the demands of the gravity system and were not members of the lateral system. The redesign of the gravity system resulted in an increase in gravity loads that the columns see. Columns are composed of concrete with a compressive strength of 6000 psi. Two critical columns were checked using PCA Column. A check on an exterior column can be seen in Figure 26 and a check on an interior column can be seen in Figure 27 below.

Figure 26: Check on column #8 (32x32 with (20) #10 bars)

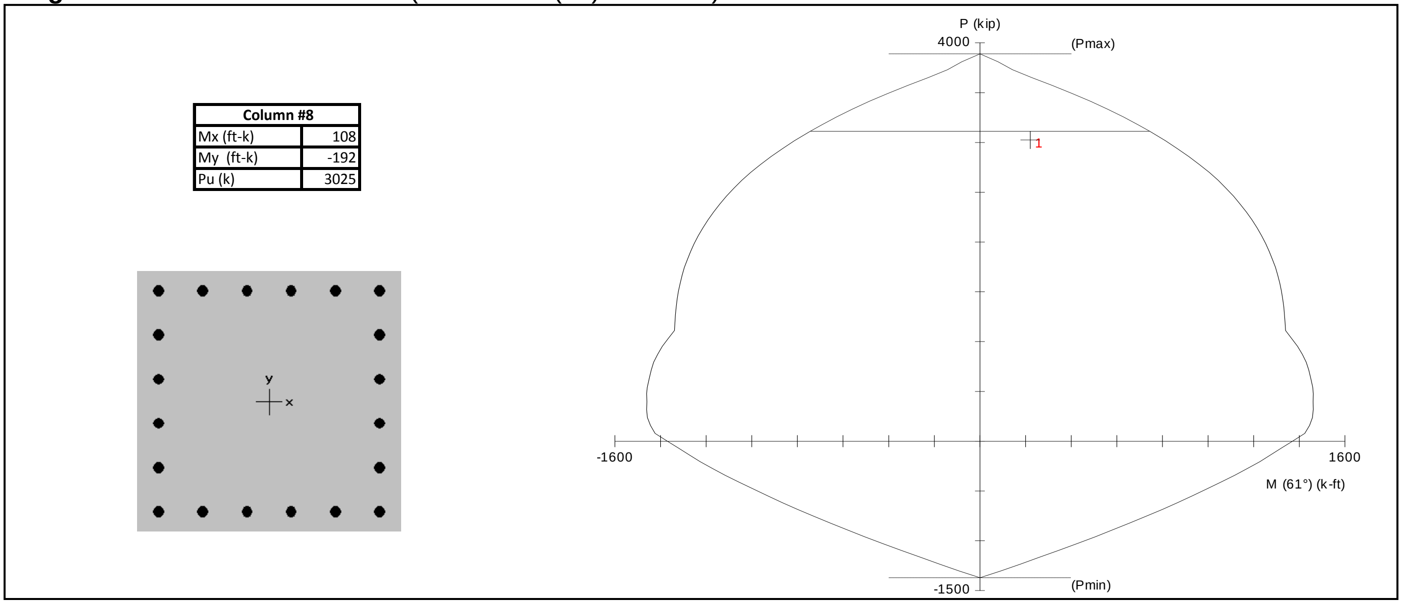
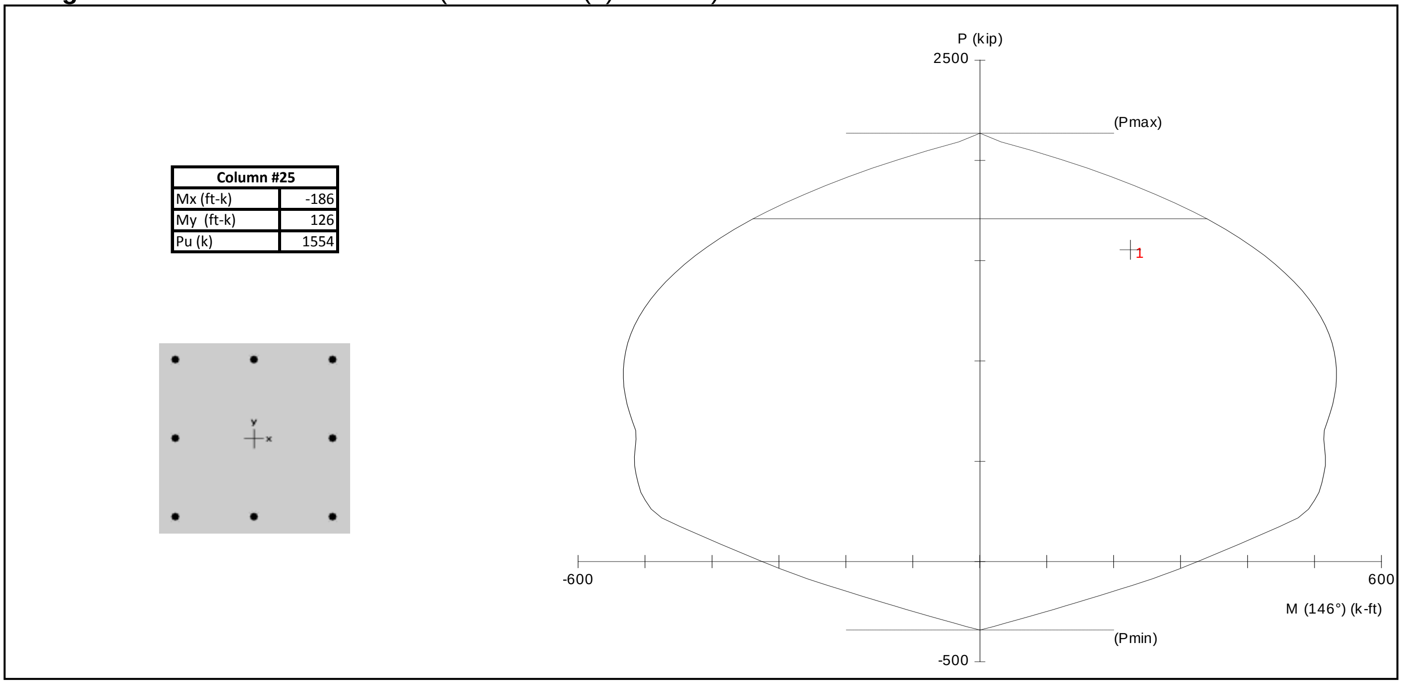


Figure 27: Check on column #25 (24x24 with (8) #8 bars)



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Gravity System Design Summary and Conclusions

The project goal of reducing the total depth of the floor system was met by switching from the existing composite metal deck and composite steel beam system to a one-way concrete slab and post-tensioned beam system. The original design was 30.25” deep and the largest beam size for the new system is 26x22 for all levels except for Level 9 which supports the green roof and has a maximum beam size of 30x24. This yields a total reduction of 8.25” in most areas and a 6.25" reduction for the portion supporting the green roof.

When checking live and dead load deflections many areas of the slab were on the high side, very close to the allowable limit. Most of the difficulty occurred in areas where the aspect ratio of the bays was relatively low. After designing a one-way system with post-tensioned beams it is possible that many of the design challenges that occurred may have been solved if a 2-way post-tensioned flat plate system were designed.

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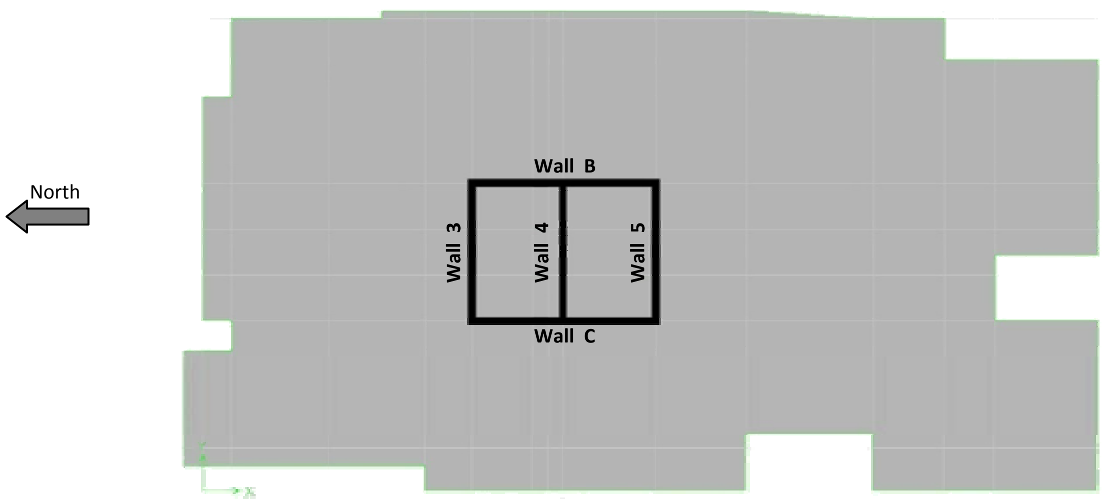
Lateral System Design

Ordinary reinforced concrete shear walls were chosen for the new lateral system. The first step in the design process was to determine a layout for the shear walls. The building is skinned from ground to roof in a glass curtain wall. This ruled out the option of placing shear walls at the perimeter of the building without requiring significant architectural changes. The existing structure utilized a core of steel special moment and braced frames. Drawing from the previous design, the concrete shear walls were placed at the same locations around the core for the preliminary design. Two 40’ long shear walls will resist lateral forces in the North/South direction and three 30’ long shear walls will resist lateral forces in the East/West direction. See Figure 28 below for the preliminary shear wall configuration.

Figure 28: Shear wall configuration



The next step in the design process was determining a preliminary thickness for the shear walls. The minimum thickness of the shear walls was limited by the shear strength of the concrete. Concrete with an f’c equal to 6000 psi was chosen for the shear walls. A required shear strength of 232 psi was calculated using a conservative estimate of shear strength equal to 3 (f’c). Using wind and seismic loads calculated according to ASCE 7-05, the total shear at each story was divided by phi factors of 0.75 for wind and 0.6 for seismic. The larger shears at each level were divided by the required shear strength of 232 psi to determine the area of concrete necessary to handle the shear forces. The required area in shear was then distributed to each wall and divided by its length to give a preliminary thickness. The required thicknesses

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based on wind loads were larger than those based on seismic loads and are provided below in Table 6 for reference. The minimum thickness required to resist shear forces is approximately 7" as highlighted below in the table. It is not advised to use a shear wall thickness less than 12" and to be conservative an 18" thickness was chosen for the design.

Table 6: Determination of preliminary shear wall thicknesses to resist wind forces

	Story Force (K)		Total Shear (lbs)		Total shear/.75 (lbs.)		Required area in shear (in^2)	
Level	E/W	N/S	E/W	N/S	E/W	N/S	E/W	N/S
Roof	32.21	16.26	32207	16262	42943	21683	185	93
20	64.00	32.28	96203	48541	128270	64722	552	279
19	63.97	32.27	160177	80808	213569	107744	919	464
18	63.97	32.27	224151	113075	298869	150767	1286	649
17	63.72	32.12	287876	145196	383835	193595	1652	833
16	61.52	30.84	349400	176032	465867	234710	2005	1010
15	61.00	30.53	410405	206565	547207	275420	2355	1185
14	60.58	30.28	470981	236847	627974	315796	2702	1359
13	59.38	29.58	530361	266432	707149	355242	3043	1529
12	58.78	29.24	589146	295667	785527	394223	3380	1696
11	58.16	28.87	647301	324536	863068	432715	3714	1862
10	56.70	28.02	704003	352556	938670	470075	4039	2023
9	56.00	27.61	759998	380164	1013330	506885	4361	2181
8	54.50	26.74	814502	406900	1086003	542534	4673	2335
7	53.25	26.00	867751	432904	1157002	577205	4979	2484
6	51.62	25.05	919373	457957	1225830	610609	5275	2628
5	50.11	24.17	969487	482130	1292650	642840	5563	2766
4	48.32	23.12	1017805	505254	1357074	673672	5840	2899
3	44.70	21.06	1062508	526314	1416677	701752	6096	3020
2	37.82	17.55	1100330	543866	1467107	725155	6313	3121
Ground	16.92	7.84	1117249	551706	1489665	735608	6410	3166

	Required area in shear per wall (in^2)					Preliminary thickness (in)				
	33% to each wall E/W			50% to each wall N/S		E/W			N/S	
Level	Wall 3	Wall 4	Wall 5	Wall B	Wall C	Wall 3	Wall 4	Wall 5	Wall B	Wall C
Roof	62	62	62	47	47	0.17	0.17	0.17	0.19	0.19
20	184	184	184	139	139	0.51	0.51	0.51	0.58	0.58
19	306	306	306	232	232	0.85	0.85	0.85	0.97	0.97
18	428	428	428	324	324	1.19	1.19	1.19	1.35	1.35
17	550	550	550	417	417	1.53	1.53	1.53	1.74	1.74
16	668	668	668	505	505	1.85	1.85	1.85	2.10	2.10
15	784	784	784	593	593	2.18	2.18	2.18	2.47	2.47
14	900	900	900	679	679	2.50	2.50	2.50	2.83	2.83
13	1013	1013	1013	764	764	2.81	2.81	2.81	3.18	3.18
12	1126	1126	1126	848	848	3.13	3.13	3.13	3.53	3.53
11	1237	1237	1237	931	931	3.44	3.44	3.44	3.88	3.88
10	1345	1345	1345	1011	1011	3.74	3.74	3.74	4.21	4.21
9	1452	1452	1452	1091	1091	4.03	4.03	4.03	4.54	4.54
8	1556	1556	1556	1167	1167	4.32	4.32	4.32	4.86	4.86
7	1658	1658	1658	1242	1242	4.61	4.61	4.61	5.17	5.17
6	1757	1757	1757	1314	1314	4.88	4.88	4.88	5.47	5.47
5	1852	1852	1852	1383	1383	5.15	5.15	5.15	5.76	5.76
4	1945	1945	1945	1450	1450	5.40	5.40	5.40	6.04	6.04
3	2030	2030	2030	1510	1510	5.64	5.64	5.64	6.29	6.29
2	2102	2102	2102	1560	1560	5.84	5.84	5.84	6.50	6.50
Ground	2135	2135	2135	1583	1583	5.93	5.93	5.93	**6.59**	**6.59**

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Material takeoffs were obtained from the RAM concept model and converted to kips to determine the total weight of the building for use in seismic calculations. The total weight of each floor was converted to a mass for input into the ETABS model. Each floor was modeled in ETABS as a rigid diaphragm which alleviated the need to model the gravity system. See Table 7 below for determination of building weight and diaphragm mass.

Table 7: Determination of building weight and diaphragm mass

	Level 2	typical lower Level (3-8)	green roof Level 9	typical upper level (10-20, roof)		Level 2	typical lower Level (3-8)	green roof Level 9	typical upper level (10-20, roof)
Concrete (cu. yds.)	254.1	596.6	592.9	461	Conversion to lbs.	1029105	2416230	2401245	1867050
Post-tensioning (lbs.)	2041	4857	4857	3847		2041	4857	4857	3847
mild-steel reinforcing (tons)	13.21	38.97	34.79	25.39		29062	85734	76538	55858
S.I. Dead (psf)	20	20	20	20		148960	356200	351000	275000
Facade Weight (plf)	195	195	195	195		85995	117000	117000	97890

	Level 2	typical lower Level (3-8)	green roof Level 9	typical upper level (10-20, roof)
Area (sq. ft.)	7448	17810	17550	13750
Perimeter (ft.)	441	600	600	502

	Level 2	typical lower Level (3-8)	green roof Level 9	typical upper level (10-20, roof)
Total floor diaphragm load (lbs)	1295163	2980021	2950640	2299645
Total floor diaphragm load (k)	1295	2980	2951	2300
area load (ksf)	0.174	0.167	0.168	0.167
diaphragm mass	**3.125E-06**	**3.007E-06**	**3E-06**	**3.0058E-06**

Total Building Weight (k)	**49722**

Shear reinforcing for the walls was designed by hand methods and it was determined that only the minimum amount of reinforcing according to ACI 318-08 was required for all of the walls. See Table 8 below for a sample calculation for Wall B.

Table 8: Determination of shear reinforcing for Wall B

WALL B

thickness (h) (in)	18	
hw (in)	156	
lw (in)	480	
f'c (psi)	6000	
fy (psi)	60000	
d (in)	384	
Max. permitted shear: Vu<ϕVn		
ϕVn (k)	4016	
Shear Strength by Vc:		
a=min of< lw/2		240
hw/2		78
a=		**78**
Vc (k)	1071	
or		
Vc (k)	**1767**	
or		
Vc (k)	-3503	
Required Horizontal Shear Reinforcing:		
Is Vu>1/2ϕVc Vu		264
1/2ϕVc		663

No. Therefore, provide minimum reinforcement

Bar Size	Area
3	0.11
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79

Level	Vu (k)
Roof	35.42
20	65.28
19	93.51
18	119.15
17	142.09
16	162.42
15	180.3
14	195.88
13	209.31
12	220.76
11	230.39
10	238.39
9	246.4
8	252.94
7	258.13
6	262.04
5	**263.77**
4	260.95
3	237.79
2	249.3

Horizontal:

smax=min of <	lw/5	96
	3h	54
	or	18
s,max=	**18**	
pt,min=Av/(s*h)	0.0025	
Av required	0.81	

Bar Size	# of Bars	s	Av	pt
3	2	4	0.22	0.003055556
4	2	6	0.4	0.003703704
5	2	10	0.62	0.003444444

Vertical:

ρl,min=max of <	0.002464583
	0.0025
ρl	0.0025
smax=min of <	160
	54
	18
s,max=	**18**
Av required	0.81

Bar Size	# of Bars	s	Av	pt
3	2	4	0.22	0.003055556
4	2	6	0.4	0.003703704
5	2	10	0.62	0.003444444

STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

The shear reinforcing design for all of the walls consists of #5 bars at a minimum spacing of 10". After the shear reinforcing was designed by hand methods it was entered into PCA Column to check under flexural loads. A check on the shear wall reinforcing design at Level 4 was performed using PCA Column. Level 4 was checked because it is the most critical typical floor. After placing openings in the walls they were grouped into two piers as seen in Figures 29 and 30 below and were entered into PCA Column to determine their flexural capacity. Axial loads on each pier were determined using RAM Concept and applied to each pier. 1.2D+1.6L+0.5Lr was the critical load combination as highlighted in red. See Table 9 below. Moments due to lateral forces were determined using ETABS and were applied simultaneously to the piers. See Table 10 for flexural loads applied to each pier.

Figure 29: Pier 1

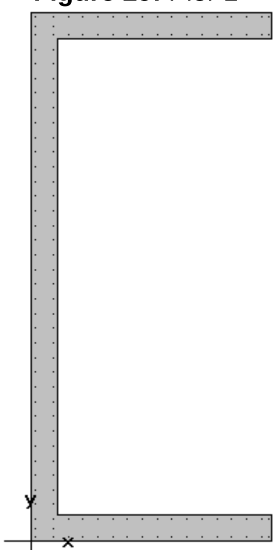


Figure 30: Pier 2

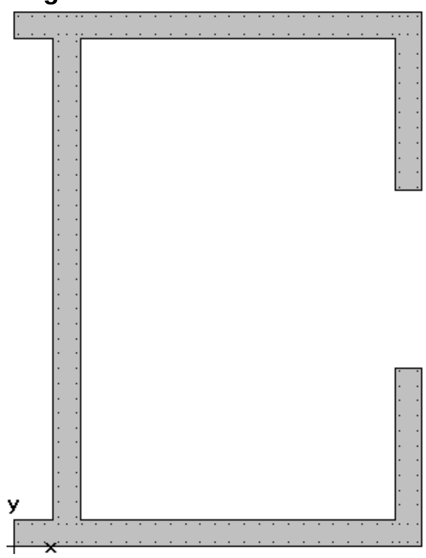


Table 9:
Axial load on shear walls supporting typical floors

For a typical floor: Level 10-Roof					
1.4D			1.2D+1.6L+0.5Lr		
	PIER 1	PIER 2		PIER 1	PIER 2
Wall	axial load (k)	axial load (k)	Wall	axial load (k)	axial load (k)
3	226		3	324	
4		30	4		14.8
5		129	5		190
B	59	164	B	92.5	250.5
C	58.5	167.5	C	89.5	254.5
total	343.5	490.5	total	506	709.8
For a typical floor: Level 4-9					
1.4D			1.2D+1.6L+0.5Lr		
	PIER 1	PIER 2		PIER 1	PIER 2
Wall	axial load (k)	axial load (k)	Wall	axial load (k)	axial load (k)
3	199		3	277	
4		86.8	4		110
5		115	5		165
B	70.5	211.5	B	82.75	248.25
C	52.25	156.75	C	80.75	242.25
total	321.75	570.05	total	440.5	765.5
Shear walls supporting Level 4 support 18 floors:					
total axial (k)	6053	9306		**8715**	**13111**

Table 10:
Ultimate factored moments from ETABS

Pier 1	
Mu (y-axis) ft-k	Mu (x-axis) ft-k
23809	49211
Pier 2	
Mu (y-axis) ft-k	Mu (x-axis) ft-k
23809	85367

STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

The ultimate factored moments and axial loads were plotted on interaction diagrams to check if they were within the shear wall's capacity. Reinforcing in both piers 1 and 2 is sufficient to carry the applied loads as seen in Figures 31 and 32 respectively. Notice the interaction diagram is not symmetrical. This is a result of biaxial loading on the shear walls due to their geometry.

Figure 31: Pier 1 Interaction Diagram

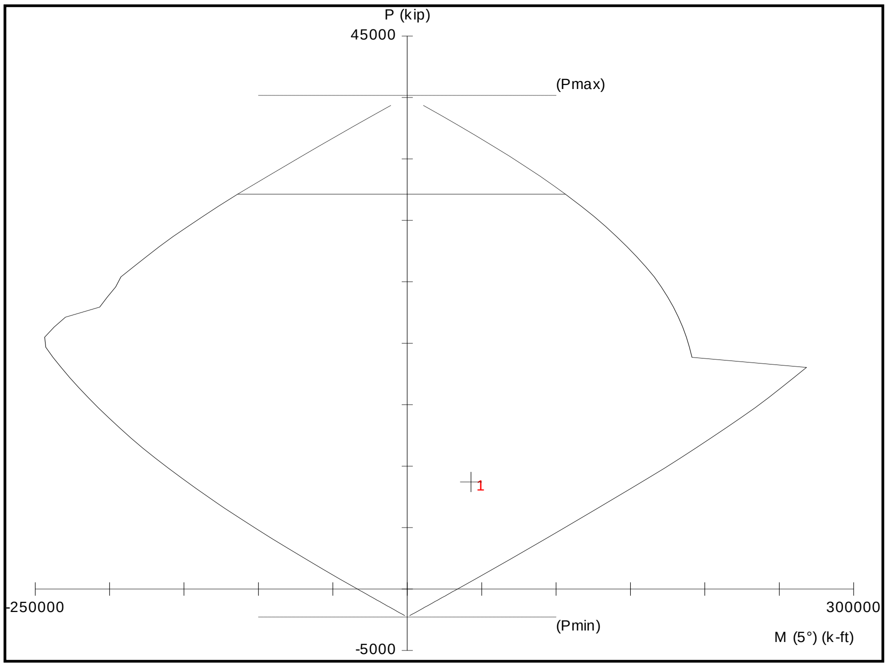
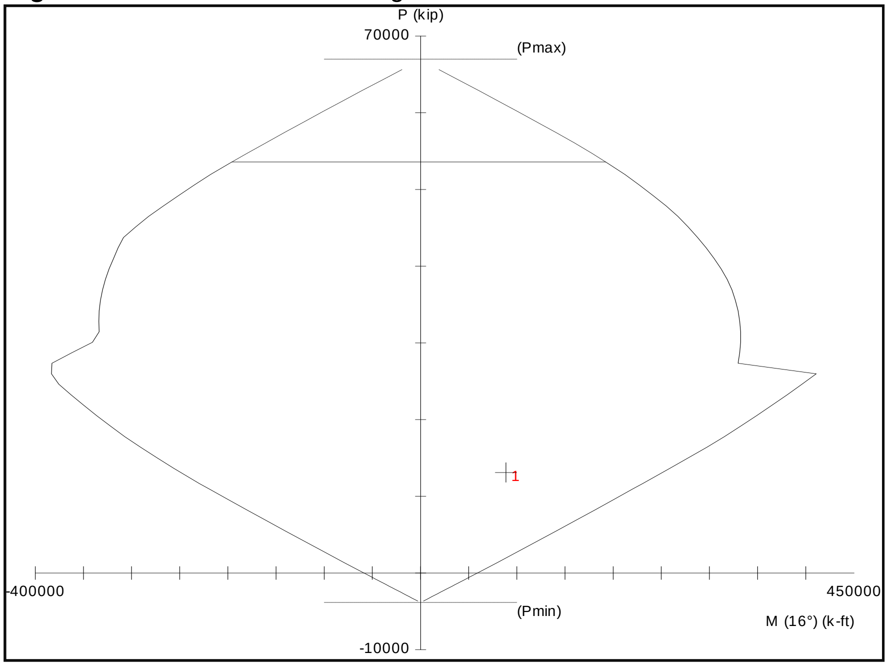


Figure 32: Pier 2 Interaction Diagram



STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

Shear wall design summary

The final lateral system design consists of 18" ordinary reinforced concrete shear walls arranged around the core of the building. Walls 3 through 5 are 30' long and resist lateral forces in the East/West direction. Walls B and C are 40' long and resist lateral forces in the North/South direction. See Figure 33 below for shear wall elevations and Figure 34 for their corresponding locations on the plan. Horizontal and vertical reinforcing consists of two rows of #5 bars spaced at 10" O.C. See Figure 35 for a section view of the reinforcing.

Figure 33: Shear wall elevations

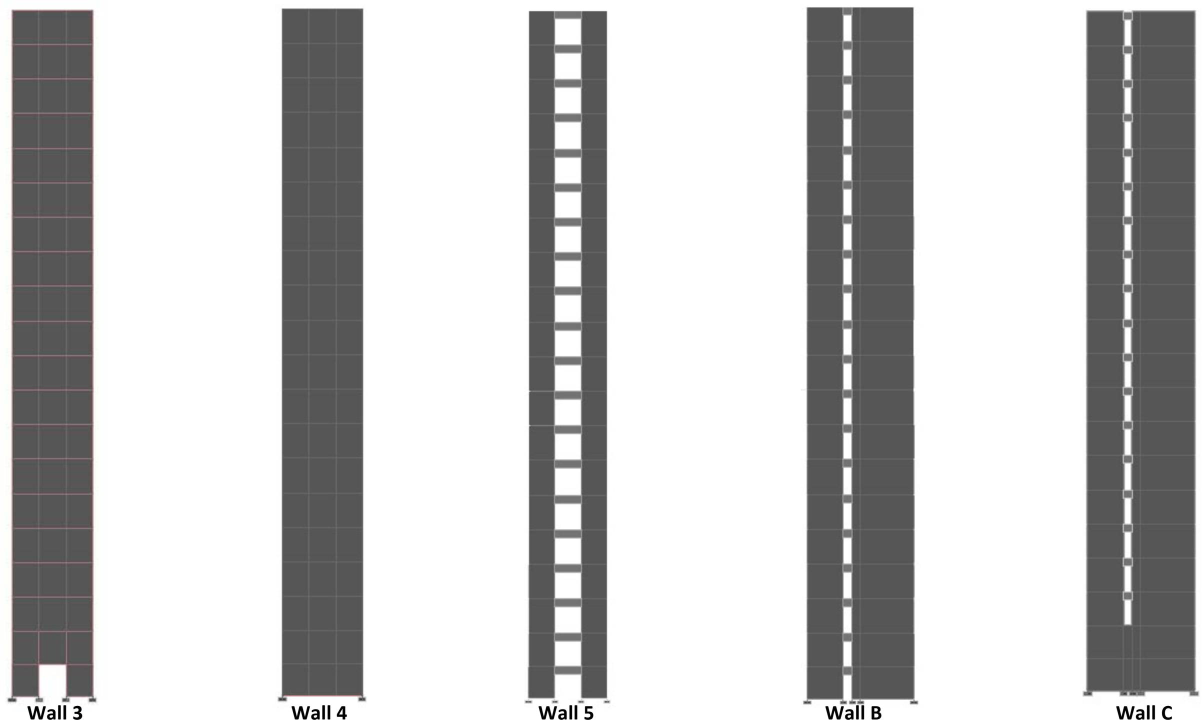


Figure 34: Plan of shear wall locations

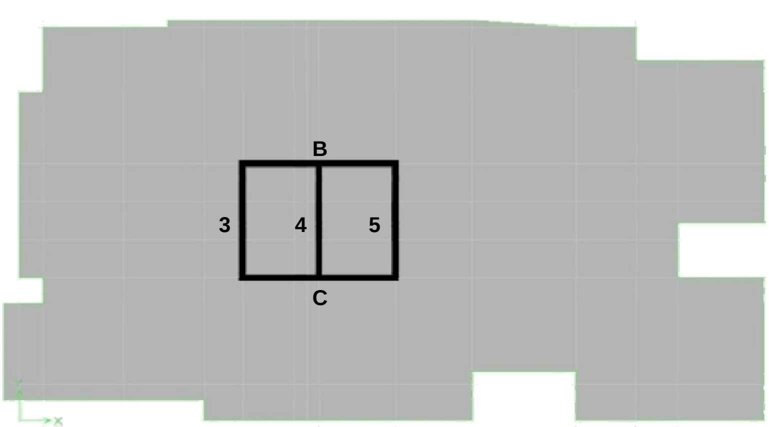
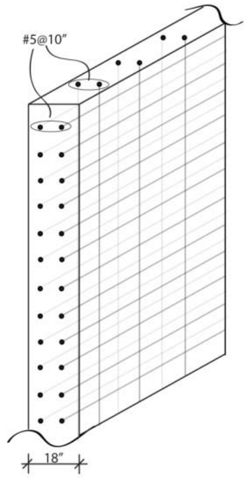


Figure 35: Reinforcing section



STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

Lateral Analysis

A drift analysis was performed to determine whether the structure meets the appropriate deflection criteria when subjected to lateral loads. It was necessary to recalculate wind and seismic loads for the building's relocation to Columbus, Ohio. Loads were determined in accordance with ASCE 7-05 and applied to the structure in ETABS. According to ACI 318-08 section 8.8.2 Lateral deflections shall be computed using 50 percent of the stiffness values of lateral elements based on gross section properties. Therefore the modulus of elasticity of the lateral elements was reduced by 50 percent to directly affect flexure, axial, and shear stiffness.

Wind

Wind forces seen in Figures 36 and 37 below were applied at the center of pressure of the structure in ETABS.

Figure 36:
Wind story forces x-direction (E/W)

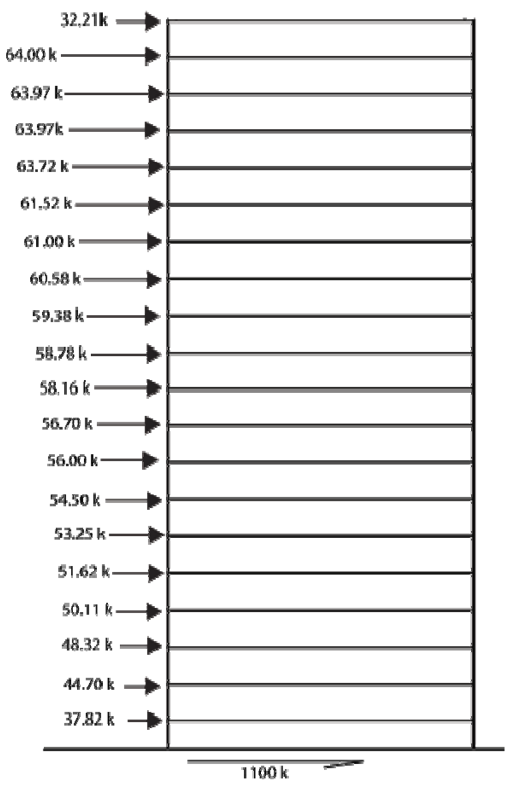
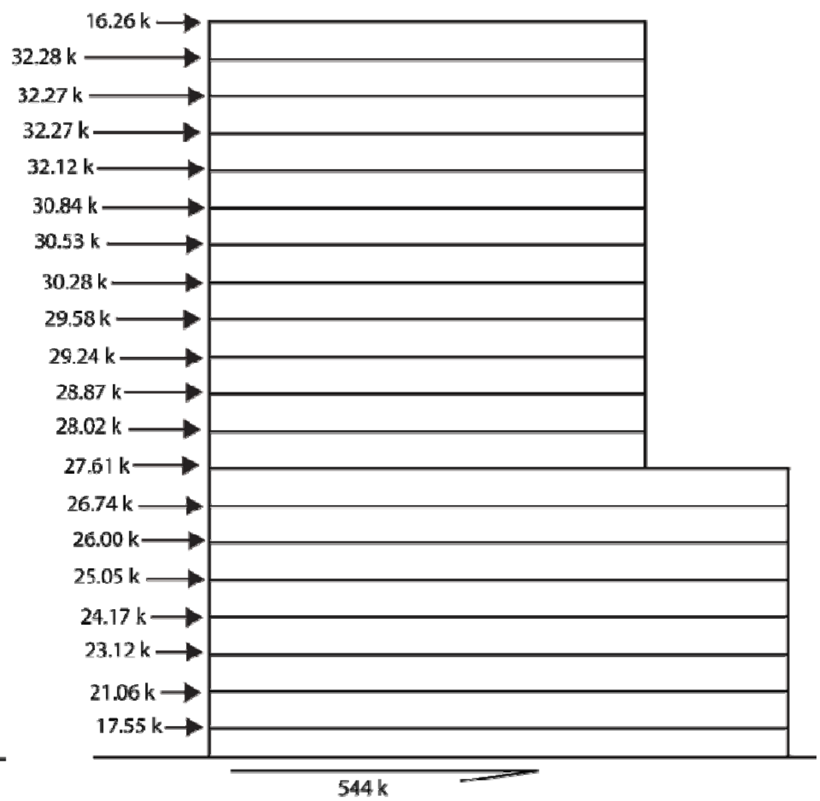


Figure 37:
Wind story forces y-direction (N/S)



STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

The maximum displacement at each level was compared with the industry standard serviceability criterion of h/400. The total building drift in both the x and y directions were within the allowable building drift limits as seen in Table 11 below.

Table 11: Total drift at each level due to wind

			Wind		
Level	Height (ft)	Floor to Floor H (ft)	**Allow. drift (in)**	disp. WX (in)	disp. WY (in)
Roof	258.50	13.00	**7.755**	1.416	4.916
20	245.50	13.00	**7.365**	1.336	4.695
19	232.50	13.00	**6.975**	1.255	4.470
18	219.50	13.00	**6.585**	1.173	4.240
17	206.50	13.00	**6.195**	1.091	4.004
16	193.50	13.00	**5.805**	1.007	3.761
15	180.50	13.00	**5.415**	0.922	3.512
14	167.50	13.00	**5.025**	0.837	3.257
13	154.50	13.00	**4.635**	0.752	2.997
12	141.50	13.00	**4.245**	0.667	2.733
11	128.50	13.00	**3.855**	0.584	2.465
10	115.50	13.00	**3.465**	0.502	2.196
9	102.50	13.00	**3.075**	0.421	1.931
8	89.50	13.00	**2.685**	0.345	1.601
7	76.50	13.00	**2.295**	0.272	1.268
6	63.50	13.00	**1.905**	0.203	0.942
5	50.50	13.00	**1.515**	0.141	0.631
4	37.50	13.00	**1.125**	0.087	0.350
3	24.50	12.50	**0.735**	0.041	0.117
2	12.00	12.00	**0.360**	0.014	0.039

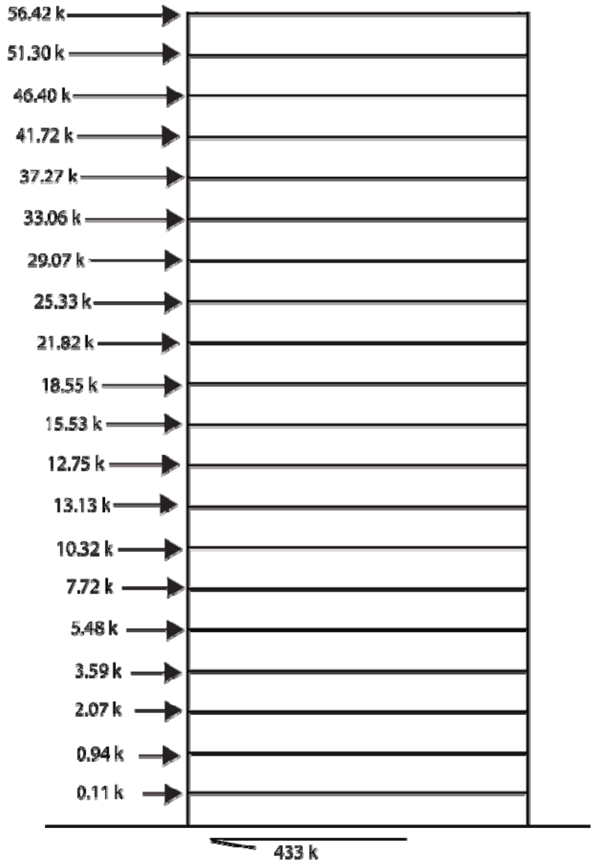
Displacement values taken from ETABS

STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

Seismic

Seismic forces seen in Figure 38 below were applied to the ETABS model at the center of mass. The resulting displacements were taken from ETABS and compared with the allowable values. Accidental torsion was taken into account by assuming a displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces. Determination of an amplification factor was not necessary due to the structure's location in Seismic Design Category B.

Figure 38: Seismic forces at each level



STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

Deflections computed at the center of mass were used to calculate the seismic story drift. The story drifts were determined by multiplying the values from ETABS by the deflection amplification factor (Cd) which is 4.5 for ordinary reinforced concrete shear walls and dividing by an importance factor of 1.0. The values were compared to the allowable story drift due to seismic forces according to ASCE 7-05 equal to 0.02 times the story height. The story drift in both the x and y directions were acceptable as seen in Table 12 below.

Table 12: Story drifts due to seismic forces

			Seismic						
Level	Height (ft)	Floor to Floor H (ft)	**all. Story drift (in)**	x-disp. (in)	x-story drift (in)	**δ_x (in)**	y-disp. (in)	y-story drift (in)	**δ_y (in)**
Roof	258.50	13.00	**3.12**	1.694	0.104	**0.466**	2.029	0.130	**0.585**
20	245.50	13.00	**3.12**	1.590	0.105	**0.473**	1.899	0.130	**0.586**
19	232.50	13.00	**3.12**	1.485	0.106	**0.478**	1.769	0.131	**0.590**
18	219.50	13.00	**3.12**	1.379	0.107	**0.482**	1.638	0.132	**0.592**
17	206.50	13.00	**3.12**	1.272	0.108	**0.486**	1.506	0.132	**0.593**
16	193.50	13.00	**3.12**	1.164	0.108	**0.486**	1.374	0.131	**0.590**
15	180.50	13.00	**3.12**	1.056	0.108	**0.484**	1.243	0.130	**0.585**
14	167.50	13.00	**3.12**	0.948	0.106	**0.479**	1.113	0.128	**0.576**
13	154.50	13.00	**3.12**	0.842	0.105	**0.470**	0.985	0.125	**0.563**
12	141.50	13.00	**3.12**	0.737	0.102	**0.458**	0.860	0.121	**0.546**
11	128.50	13.00	**3.12**	0.636	0.098	**0.441**	0.738	0.117	**0.524**
10	115.50	13.00	**3.12**	0.538	0.097	**0.438**	0.622	-0.017	**-0.076**
9	102.50	13.00	**3.12**	0.440	0.087	**0.393**	0.639	0.125	**0.564**
8	89.50	13.00	**3.12**	0.353	0.081	**0.365**	0.513	0.116	**0.524**
7	76.50	13.00	**3.12**	0.272	0.073	**0.330**	0.397	0.106	**0.478**
6	63.50	13.00	**3.12**	0.198	0.065	**0.290**	0.291	0.094	**0.422**
5	50.50	13.00	**3.12**	0.134	0.054	**0.244**	0.197	0.079	**0.355**
4	37.50	13.00	**3.12**	0.080	0.042	**0.189**	0.118	0.061	**0.276**
3	24.50	12.50	**3**	0.038	0.021	**0.095**	0.057	0.043	**0.195**
2	12.00	12.00	**2.88**	0.016	0.016	**0.074**	0.013	0.013	**0.059**

Displacement values taken from ETABS

Cd	4.5
I	1.0

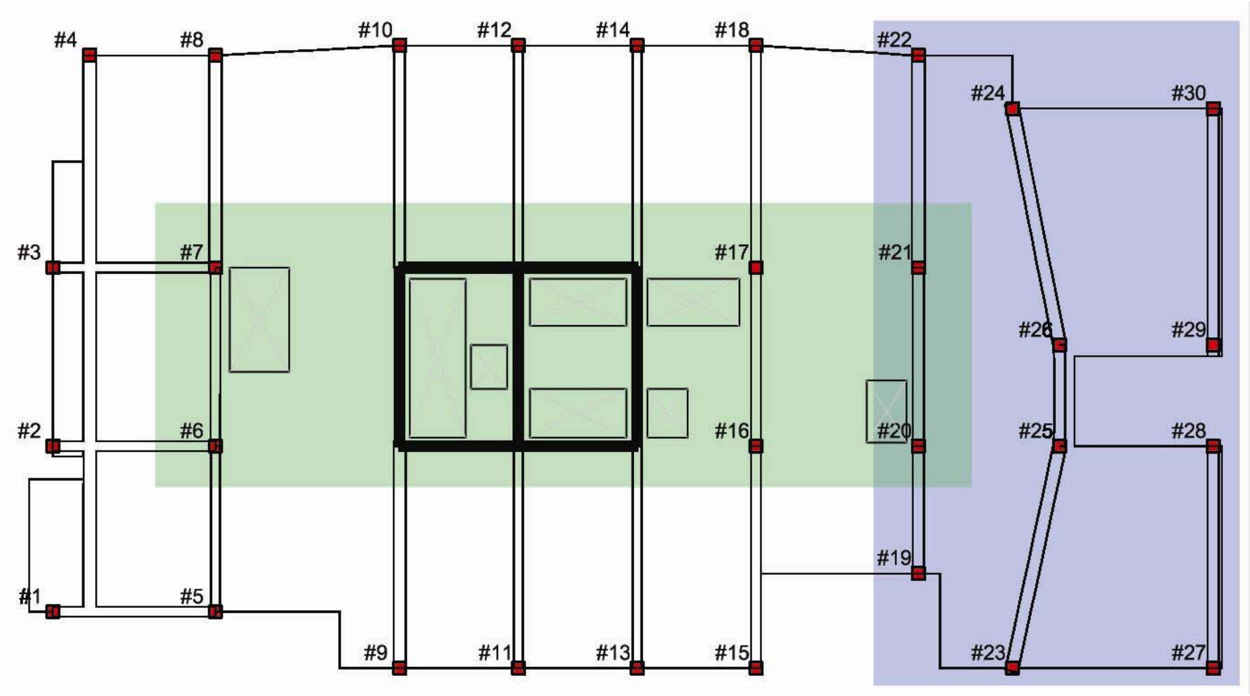
STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

Impact on Foundations

To evaluate the impact of the redesign on the foundations, the required number of piles to support the new concrete structural system was compared to the number of piles used in the original design to support the steel system.

Floor loads to each column were determined using RAM Concept and totaled to give the load on each column at the foundation level. See Figure 39 for column numbers and locations.

Figure 39: Plan of lower level indicating column numbers and locations. The key system portion of the building is highlighted in blue and the mat foundation in green.



The original design utilized 110 ton, 14"-square, driven prestresed precast concrete piles. The load on each column was divided by the 110 ton capacity of the piles to determine the required number of piles to support each column load. This figure was compared with number of piles required to support the original steel columns and a percent increase in the number of piles necessary to support each column was determined. See Table 13 for a summary of the comparison. On average 33.4% more piles are required to support each column in the concrete system than those used in the original design of the steel system. Concrete systems are generally heavier than steel systems and it's expected that the foundations would need to be increased to be able to handle the higher loads.

STRUCTURAL SYSTEM REDESIGN FOR 1100 BROADWAY

Table 13: Comparison of the number of piles required to support concrete system and original design

Column #	Ultimate load per floor to each column (k) Level 2	Levels 3-8	Level 9	Levels 10-Roof	Total load on each column (k)	# of piles required to support column for concrete system	# of piles in original design	% Increase in piles required
1		88.1	66.6	68.2	1413.6	6	6	0.0%
2		161	151	133	2713	12	8	50.0%
3	91	158	158	156	3069	13	8	62.5%
4	112	84.9	84.2	79.3	1657.2	7	8	-12.5%
5		127	130	192	3196	13	8	62.5%
8	143	151	152	152	3025	13	14	-7.1%
9		134	136	137	2584	11	8	37.5%
10	118	148	150	148	2932	12	8	50.0%
11		116	115	115	2191	9	6	50.0%
12	107	113	111	114	2264	10	8	25.0%
13		111	111	113	2133	9	8	12.5%
14	102	115	114	115	2286	10	6	66.7%
15		87.4	87.4	109	1919.8	8	6	33.3%
18	111	131	131	133	2624	11	8	37.5%
19		111	120	88.5	1848	8	Columns 19-30 support the Key System portion of the structure and not enough information is available from the original design to compare with the new concrete system	
22	81.4	118	115	91.4	2001.2	9		
23		156	188		1124	5		
24		198	236		1424	6		
25		217	252		1554	7		
26		193	191		1349	6		
27		107	123		765	4		
28		93.1	106		664.6	3		
29		109	122		776	4		
30		114	135		819	4		

Average increase in # of piles required to support each column= 33.4%

The central area of the structure is supported on piles beneath a 5'-9" reinforced concrete mat foundation. The loads on the columns and shear walls that are supported by the mat foundation were totaled and divided by the 110 ton capacity of the piles to give a total of 145 piles required beneath the mat foundation. This figure was compared to the original design which consisted of 121 piles supporting the mat foundation yielding an approximate increase in the number of piles required to support the columns and shear walls above the mat foundation of 20%. See Table 14 below for a breakdown of the comparison.

Table 14: Comparison of the number of piles required beneath the mat foundation

Column #	Ultimate load per floor to each column (k) Level 2	Levels 3-8	Level 9	Levels 10-Roof	Total load on each column (k)	Shear wall	Weight (k)
6	57.4	249	242	245	4733.4	3	1744.88
7	186	265	266	264	5210	4	1744.88
16		228	230	233	4394	5	1744.88
17	137	224	223	233	4500	B	2326.50
20	14.5	184	197	118	2731.5	C	2326.50
21	124	269	285	185	4243		

Total load to be supported by mat foundation (k)	35700
# Piles required under mat foundation for concrete system	145
# Piles supporting mat foundation for original design	121
Increase in # of piles required to support mat foundation	**19.8%**