5.0 DESIGN CRITERIA

5.1 Design Objectives

The main objective of this project is to find an alternative structural system that will perform as well as steel in achieving long-span bays. Additionally, the chosen type of system should allow greater control in determining floor depths. Therefore, keeping these depths to a minimum will be of great importance. In order to meet this goal, the following standards must be met:

- Long bay spans must be preserved.
- Service spaces in the building's core must remain unchanged.
- Limit the overall floor depth to 24".
- Keep shearwalls in locations similar to those of the braced frames.
- Design must be in compliance with model codes set forth by ACI 318-05, IBC 2003, and ASCE 7-05.

5.2 Design Procedure

To make this project manageable, one of the main assumptions considered is that floors 2-5 are one type of typical floor and floors 6-15 are a second type of typical floor. Since the office tower is the responsibility of the structural designer, it alone will be considered for the redesign. The garage will be omitted from design because it is the responsibility of the precast manufacturer.

All schematic designs will be performed by traditional hand calculations using procedures outlined in concrete design texts by Antoine Naaman and Charles Nilson as well as ACI 318-05. Spreadsheets containing the calculations embedded in these procedures will be created to ease repetition. The preliminary designs yielded by the spreadsheets (available in *Appendix B: Preliminary Member Design*) will be entered into ENERCALC to confirm hand calculations. A structure of these preliminary members will be modeled in RAM structural design software for refinement. The gravity system will be checked and refined using RAM Concrete and post-tensioned elements including slabs and beams will use RAM Conce*pt*. Conce*pt* will be used to check concrete stresses and deflections under service conditions as well as design minimum reinforcing. The lateral system will be analyzed further using RAM Frame to scrutinize lateral loads and compare them to hand calculations, as well as obtain building drifts under various load cases and combinations.

5.3 Loading Conditions

5.3.1 Gravity Loads

5.3.1.1 Dead and Live

LOADING IN POUNDS/SQUARE FOOT	Office Floors	Mechanical Rooms	Penthouse Floor	Main roof	Penthouse Roof
Concrete Slab	100	100	100	50	50
Roof & Insulation				5	5
Ceiling	5	5	5	5	5
Collateral	5	5	5	5	5
Mechanical	5	5	5	10	10
Total Dead Load	115	115	115	65	65
Total Live Load	80	125	150	60	60

Table 1-Gravity loading information.

Dead loads include the self-weight of the structure and any additional loads accounting for sprinklers, MEP, and collateral loading. It is evident that the total dead load is greater for the concrete design than it was for the composite steel design, 115 psf compared to 65 psf. The live loading conditions comply with those set forth in IBC 2003.

A majority of the office tower is classified as office occupancy which results in a live load of 60 psf, plus an additional 20 psf for partitions. Although the loads are not consistent with those of the original designer, they are still conservative and comply with IBC 2003. There are portions on each floor considered to be service spaces which house HVAC and electrical equipment in mechanical rooms. These spaces, subjected to heavy equipment loads, will be designed for a 125 psf live load. This area is illustrated on the typical floor in the diagram above. The table above summarizes the gravity loading conditions in each type of occupancy.

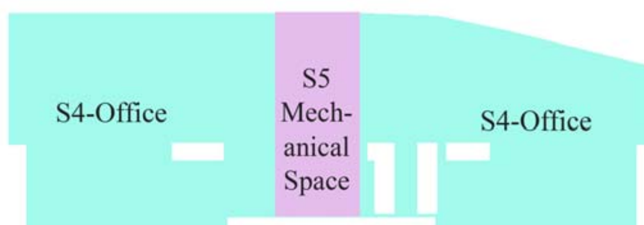


Figure 12-Diagram of loading conditions.

5.3.1.2 Snow Loads

The roof will inevitably be designed for snow loading. But since there is a building setback where the main roof meets the penthouse, drifting has the possibility of becoming an issue. Drifting can occur on the north and west sides the penthouse indicated by the lavender areas labeled 1 and 2 in the diagram to the right. A spreadsheet was developed according to the ASCE 7-02 guidelines set out in Chapter 7. In section 1, the maximum drift load was found to be 49 psf and in section 2, the load was found to be 63 psf. For further details on the calculations, please refer to *Appendix A.1: Snow Loading*.

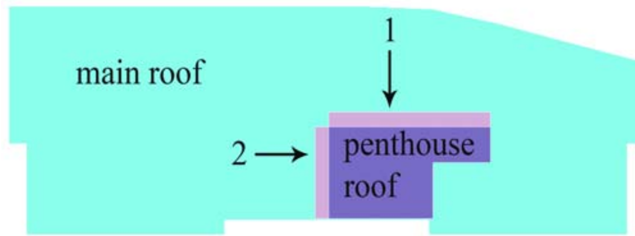


Figure 13-Areas of concern for snow drift.

5.3.2 Lateral Loads

The lateral loads, both wind and seismic, for the building were found using the guidelines set forth in IBC 2003 and ASCE7-02. Complete calculations were found using spreadsheets; please refer to *Appendix A.2: Lateral Loading* for intermediate steps. Although the wind loads for Gateway Plaza did not change due the redesign, seismic loads increased dramatically due to the increase in the structure's weight.

The wind load is distinctly greater in the north-south direction because the building dimension perpendicular to this direction is 270', which is three times larger than that in the opposite direction, and collects a great deal more pressure. In the east-west direction, however, seismic loads were found to control due to the increase in building weight. Considering both load types--seismic and all four cases of wind--and including accidental eccentricity, the only cases that resulted in unfavorable results were those that included eccentricity. In addition to the existing eccentricity between the center of rigidity and the center of mass, the accidental eccentricity created unfavorable rotations. Load combinations checked by RAM Frame include those from ASCE 7-02 in Chapter 2.0 for strength design:

1. $1.4(D)$
2. $1.2(D)+1.6(L)+0.5(L_r or S)$
3. $1.2(D)+1.6(L_r or S)+(0.5L or 0.8W)$
4. **$1.2(D)+1.6(W)+L+0.5(L_r or S)$ Controls in N-S**
5. $1.2D+1.0E+0.5L+0.2S$
6. $0.9(D)+1.6(W)$
7. $0.9(D)+1.0(E)$ **Controls in E-W**

5.3.2.1 Wind Loading

Wind Loads on the Main Wind Force Resisting System were found according to the Analytical Procedure, outlined in Section 6.5. To find the story forces and shears, a tributary area approach was taken. The pressure at each floor level was distributed over an area equal to half the floor height above and below the level. As would be expected from a building on the coastal Northeastern United States, wind is the controlling load case in the north-south direction. The basic wind loading characteristics are:

- Basic Wind Speed: 90 mph
- Wind Load Importance: 1.0
- Exposure Category: B
- Internal Pressure Coefficient: +/- 0.18
- Height: 210.5”
- Maximum wind pressure at roof: 23.3 psf

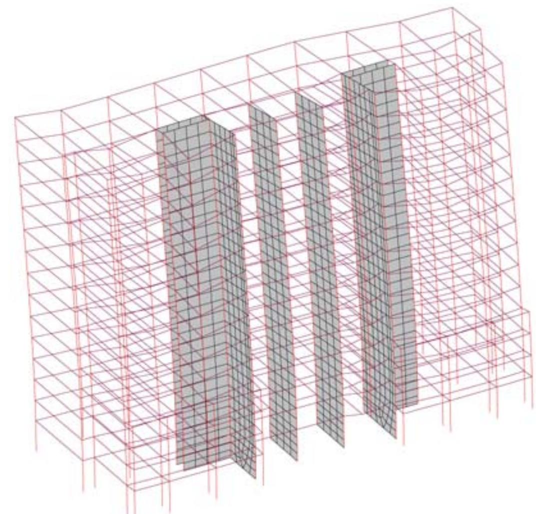


Figure 14-Structure under wind loads in north direction.

These characteristics were used to find the following loads on the building in both characteristic directions. The loads and controlling drift tabulated on page 20 are summaries of RAM output, which are similar to the hand calculations performed according to ASCE 7-02. ASCE 7’s Case 4 resulted in a building drift of 2.96”, approximately h/850, which is considered very acceptable.

5.3.2.2 Seismic Loading

Seismic Loads were found using the Equivalent Lateral Force Procedure as laid out in Section 9.5.5 of ASCE 7-02. Seismic loading characteristics include:

- Site Class: D
- Spectral Response: 0.3
- 1-second Spectral Response: 0.075
- Design Spectral Response: 0.32

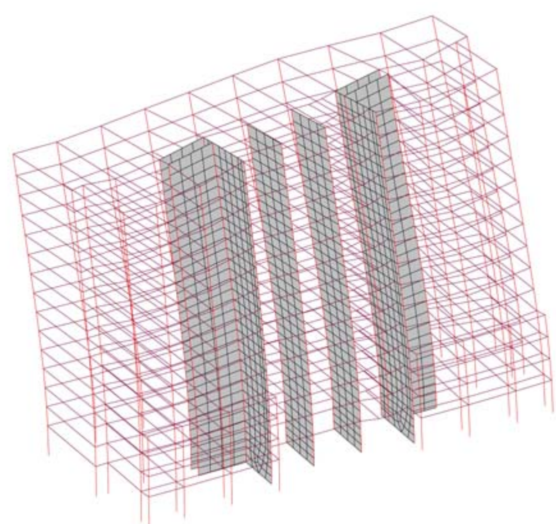


Figure 15-Structure under seismic loads in the east-west direction.

- 1-second Design Spectral Response: 0.12
- Seismic Use Group: II
- Seismic Design Category: B
- Seismic Importance Factor: 1.0
- Response Modification Factor: 3
- Base shear: 709 k

These characteristics were used to find the following loads on the building in both characteristic directions. The loads and controlling drift are tabulated on the next page. Case 7 was shown to control drift in the east-west direction, 2.25”. Again, this is an acceptable drift limit.

The table on the next page summarizes the story shears in both, north-south and east-west, directions according to RAM Frame and hand calculations performed according to ASCE 7-02. Additionally, it tabulates the maximum story drifts according to the controlling load combinations. The diagrams below depict how these forces act on the building in both directions.

Story Shear due to Wind					Story Shear due to		Story Drift	Story Drift
	X		Y		Both X & Y		**X**	**Y**
	ASCE 7	RAM	ASCE 7	RAM	ASCE 7	RAM	**1.2D+1.0E+.5L$_r$**	**1.2D+1.6W+.5L$_r$**
R	24.6	15 k	90 k	48 k	55 k	88 k	2.255 in	2.963 in
15	46.8	42 k	172 k	139 k	156 k	205 k	2.383 in	2.703 in
14	68.5	69 k	253 k	225 k	251 k	306 k	2.193 in	2.457 in
13	90	95 k	332 k	310 k	335 k	393 k	1.998 in	2.212 in
12	111.1	121 k	411 k	393 k	408 k	467 k	1.799 in	1.967 in
11	131.6	146 k	488 k	476 k	472 k	528 k	1.596 in	1.726 in
10	151.9	170 k	564 k	556 k	526 k	579 k	1.392 in	1.488 in
9	171.7	193 k	639 k	634 k	572 k	619 k	1.190 in	1.259 in
8	190.9	216 k	712 k	709 k	610 k	679 k	0.992 in	1.038 in
7	209.4	237 k	783 k	782 k	639 k	672 k	0.801 in	0.829 in
6	227.3	256 k	852 k	852 k	662 k	686 k	0.620 in	0.634 in
5	244.4	274 k	919 k	918 k	680 k	695 k	0.446 in	0.458 in
4	260.7	289 k	984 k	980 k	692 k	698 k	0.301 in	0.304 in
3	275.9	301 k	1045 k	1036 k	699 k	695 k	0.178 in	0.177 in
2	281.6	311 k	1068 k	1110 k	702 k	695 k	0.082 in	0.081 in

Table 2-Comparison of hand calculations to computer analysis results and worst case story drifts.

NS-direction wind forces EW-direction wind forces Seismic Forces

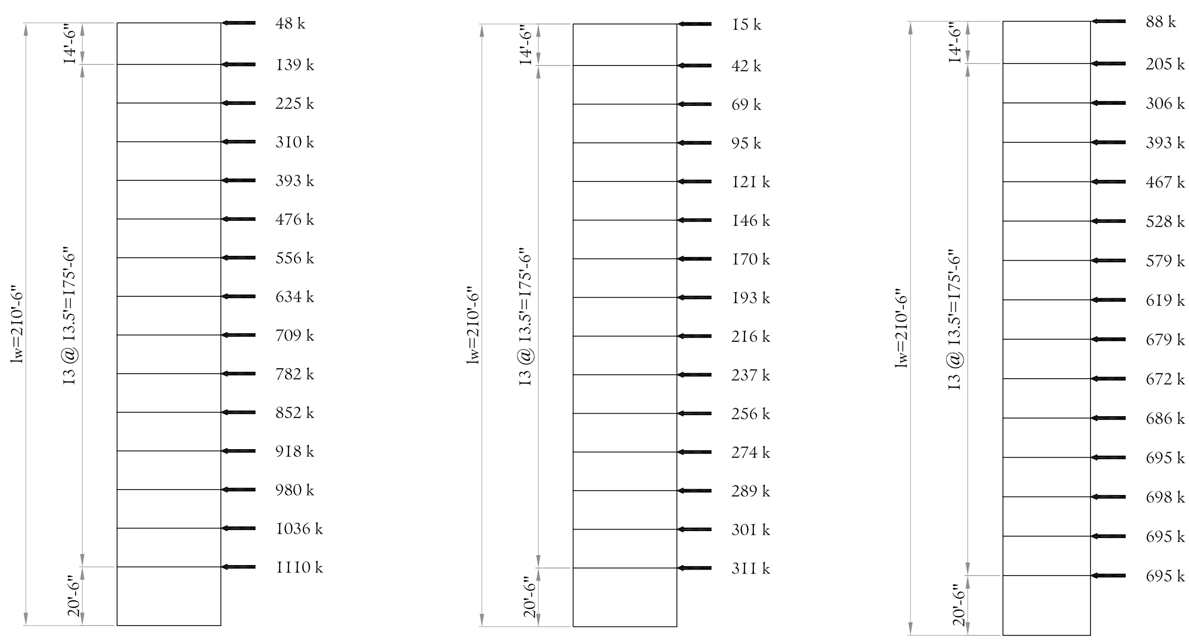


Figure 16-Story forces on the building due to lateral loads.

6.0 STRUCTURAL DEPTH

6.1 Post-tensioning

6.1.1 Slab

Spanning 52.5' with regular reinforced concrete is very difficult, and nearly impossible to stay within reasonable floor depths. For this reason, it has been concluded that a one-way post-tensioned floor system is the best candidate for redesign. The 52.5' span also requires the concrete to have a high compressive strength to withstand the large stress imposed by the prestressing tendons. For this reason, the monolithically cast slab and beam floor system will be designed using 6000 psi concrete. The slab spans the 30'-0" direction and is framed out by post-tensioned beams along column grid lines. Initial designs and hand calculations were performed following an example published by the Portland Cement Association. The example conforms to the concrete and steel stress limits provided by Chapter 18 in ACI 318-02 and is classified as Class U, uncracked. For detailed calculations, see *Appendix B.1: Post-tensioned Slab*.

An 8" thick slab was initially chosen according to an l/44 guideline set forth by the Post-tensioning Institute and accepted practice. The tendon profile was laid out in order to preserve the 2-hour fire rating of the existing system, requiring 1.75" of cover for prestressing tendons. This restricted the strands to a depth of 6.75" from the top of the slab at mid-span, 1.75" at the interior supports, and 4" at exterior supports (see **Figure 17**-Tendon profile in slab). With this profile, the effective prestressing force is found to be 1303 k. This translates into 49 tendons that need to be evenly distributed across the 52'-6" span. For constructability purposes, 8 ducts with 6 wires in each were distributed evenly across the bay. The ducts were routed around any slab openings to preserve the continuity of the prestressing force. In the angled northeast corner of the building where the slab area decreases, every other tendon was removed to prevent over-stressing the concrete (see **Figure 18**-Tendon layouts in slab).

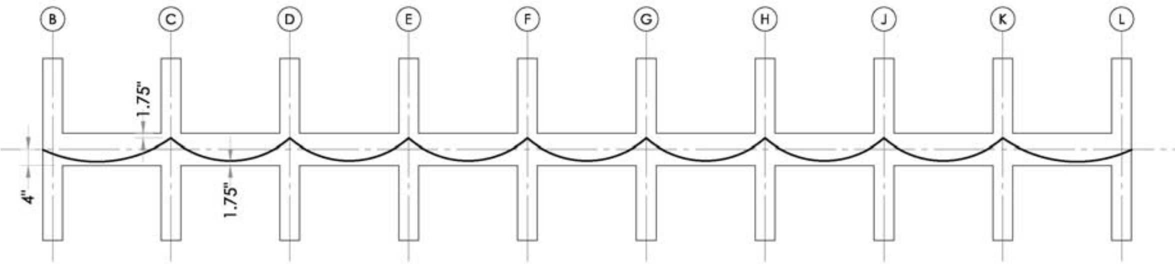


Figure 17-Tendon profile in slab.

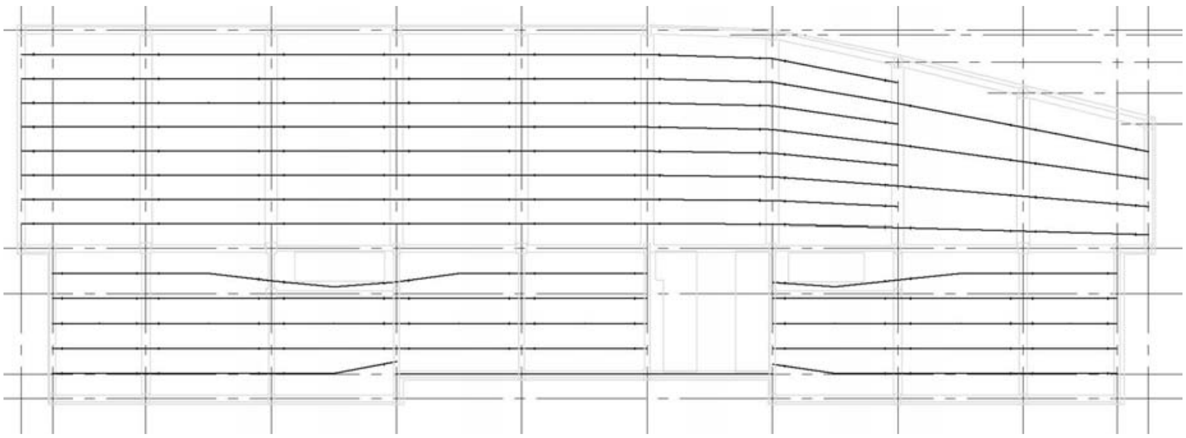


Figure 18-Tendon layouts in slab.

6.1.2 Post-tensioned Beams

The beams are included in the design because the aspect ratio of the bay is greater than the 2:1 ratio necessary for a two-way flat slab system. Therefore, post-tensioned beams will need to frame out the slab. They will need to be massive because of the large spans and heavy slab loads that they must support. To keep with the original goal of decreasing floor depth, the beams were kept to a maximum of 24" deep, including slab depth. Therefore, the beams are unconventionally wide and utilize a large amount of slab to aide in compression. The beams were initially designed according to Chapter 18 in ACI 318 and designed as Class T, the transition between uncracked and cracked. They are analyzed as T-beams to account for the additional compressive strength found in the slab. Detailed hand calculation that account for prestress losses can be found in *Appendix B.3: Beams*. These calculations consider the tendon profile to have a single drape, rather than a parabolic profile, which simplifies the calculation and provides a sufficient initial design. A feasible domain of acceptable initial forces and tendon eccentricities was constructed where an initial force, number of strands, and tendon profile were chosen.

The desired beam geometries were entered into ENERCALC to determine section properties including: area, moment of inertia, section modulus, and neutral axis. The geometry and loading for the 52.5' beam require 39 tendons with a profile of 14" at the ends and 5" at mid-span. The 36' beam requires 16 strands with the same profile. Because these calculations do not consider the beams to be continuous, they are just approximations and require closer evaluation. When the tendons from interior beams span shearwalls, they require a straight tendon profile.

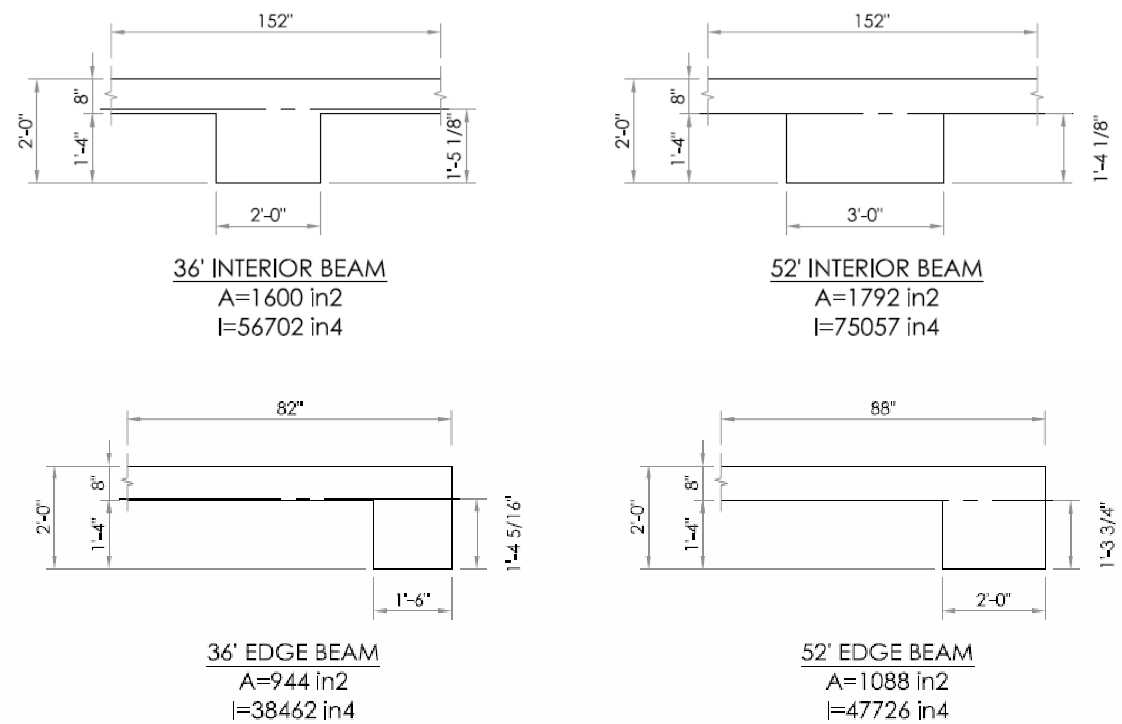


Figure 19-Cross sections of beams with section properties found in ENERCALC.

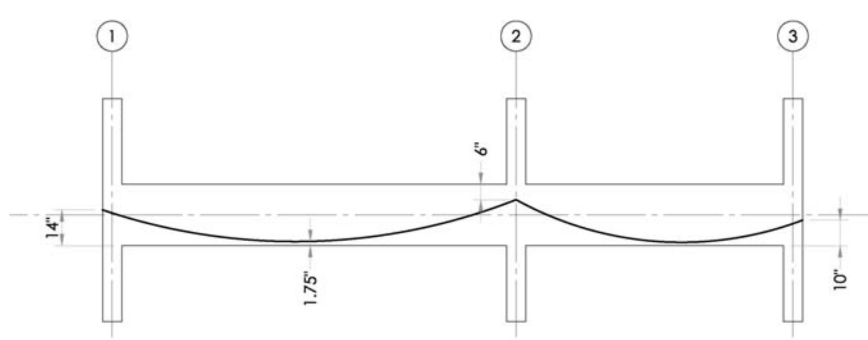


Figure 20-Tendon layout in continuous beams.

6.1.3 RAM Concept

The slab and beams, designed by hand calculations, were modeled in RAM Conce*pt* and are considered to be typical of all floors in the building. The program was used to determine concrete stresses and deflections for multiple load cases including: initial, sustained, and long-term service loading. Transverse shear reinforcement for beams and minimum required reinforcement for the slab were also determined with RAM Conce*pt*.

When modeled, column and middle design strips were generated according the ACI 318-02 for the maximum flange width of beams. Minimum reinforcement in the beams and slab was also indicated to be 0.0018 using #4 bars (see illustrations below for design strips).

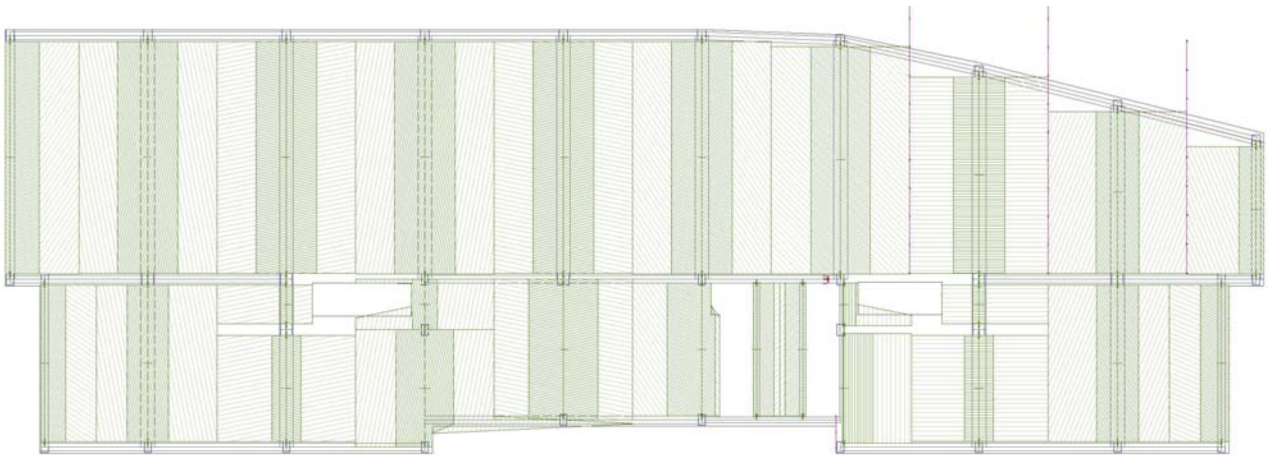


Figure 21-Longitude design strips generated by RAM Concept.

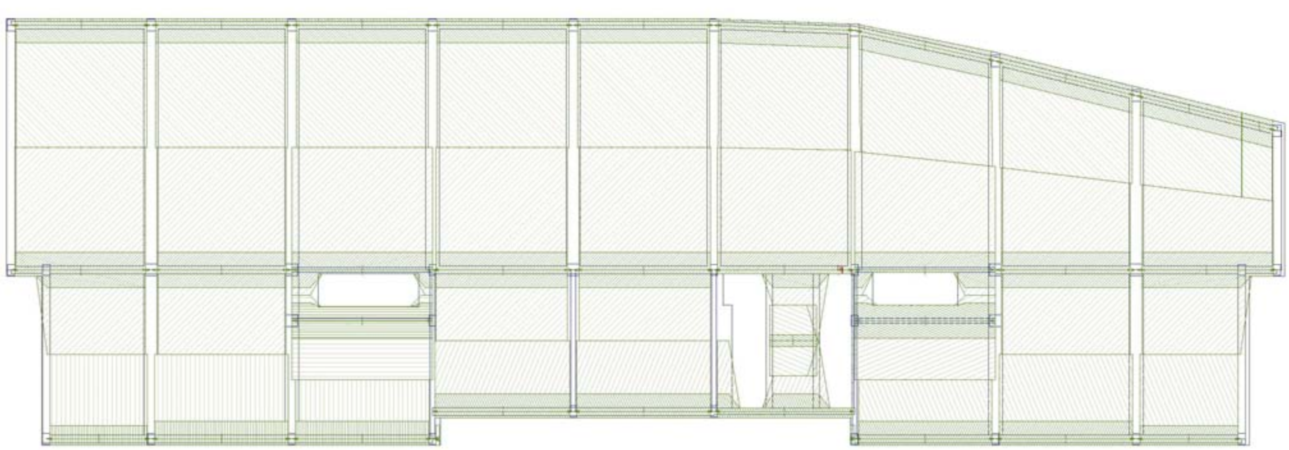


Figure 22-Latitude design strips generated by RAM Concept.

The preliminary designs worked well when modeled, and required only a few minor adjustments. The following beam schedule summarizes the location of beams, sizes, initial prestressing force, and number of strands. See the following two pages for transverse shear reinforcing details and deflection plans for various loading conditions. For minimum slab reinforcement plans, see *Appendix C: Plans*.

Properties	B-1	B-2	B-3	B-4	B-5	B-6
Dimensions	24x 24	36x 24	18x 24	24x 24	20x 24	24x 24
Shape	L	T	L	T	L	T
Fi	638 k	1064 k	266 k	1064 k	319 k	1064 k
# Strands	24	40	10	40	12	40
$d_{supports}$	17in 18in	14in 18in	18in 17in	18in 17in	17in 17in	18in 17in
d_{mid}	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in

Properties	B-7	B-8	B-9	B-10	B-11	B-12
Dimensions	18x 24	18x 24	20x 24	20x 24	20x 24	36x 24
Shape	L	L	T	T/L	L	T
Fi	319 k	319 k	319 k	319 k	319 k	1064 k
# Strands	12	12	12	12	12	40
$d_{supports}$	17in	14in 17in	17in 20in	14in 17in	14in 20in	14in 20in
d_{mid}	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in

Properties	B-13	B-14	B-15	B-16
Dimensions	36x 24	36x 24	24x 24	16x 12
Shape	T	T	L	L
Fi	718 k	718 k	559 k	0 k
# Strands	27	27	21	0
$d_{supports}$	14in 18in	14in 18in	17in 18in	-
d_{mid}	1.75 in	1.75 in	1.75 in	-

Table 3-Beam schedule including tendon profile, dimensions, and number of strands.

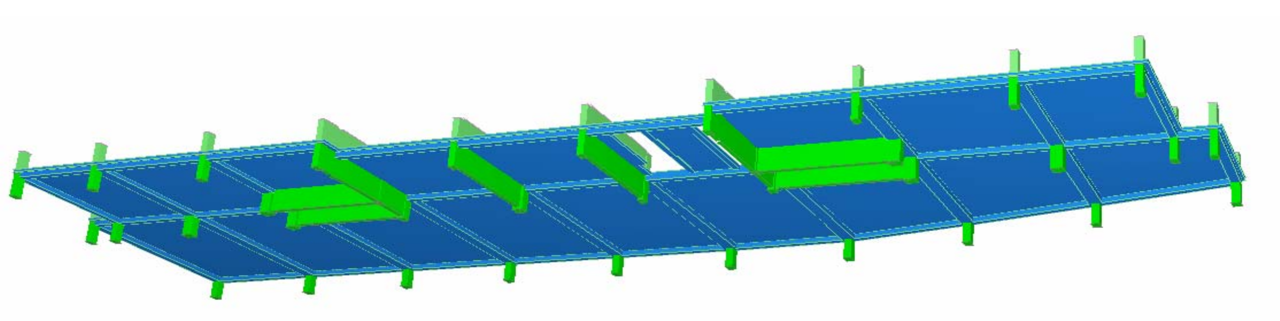


Figure 23- 3D view of underside of typical floor.

Transverse Reinforcing Details

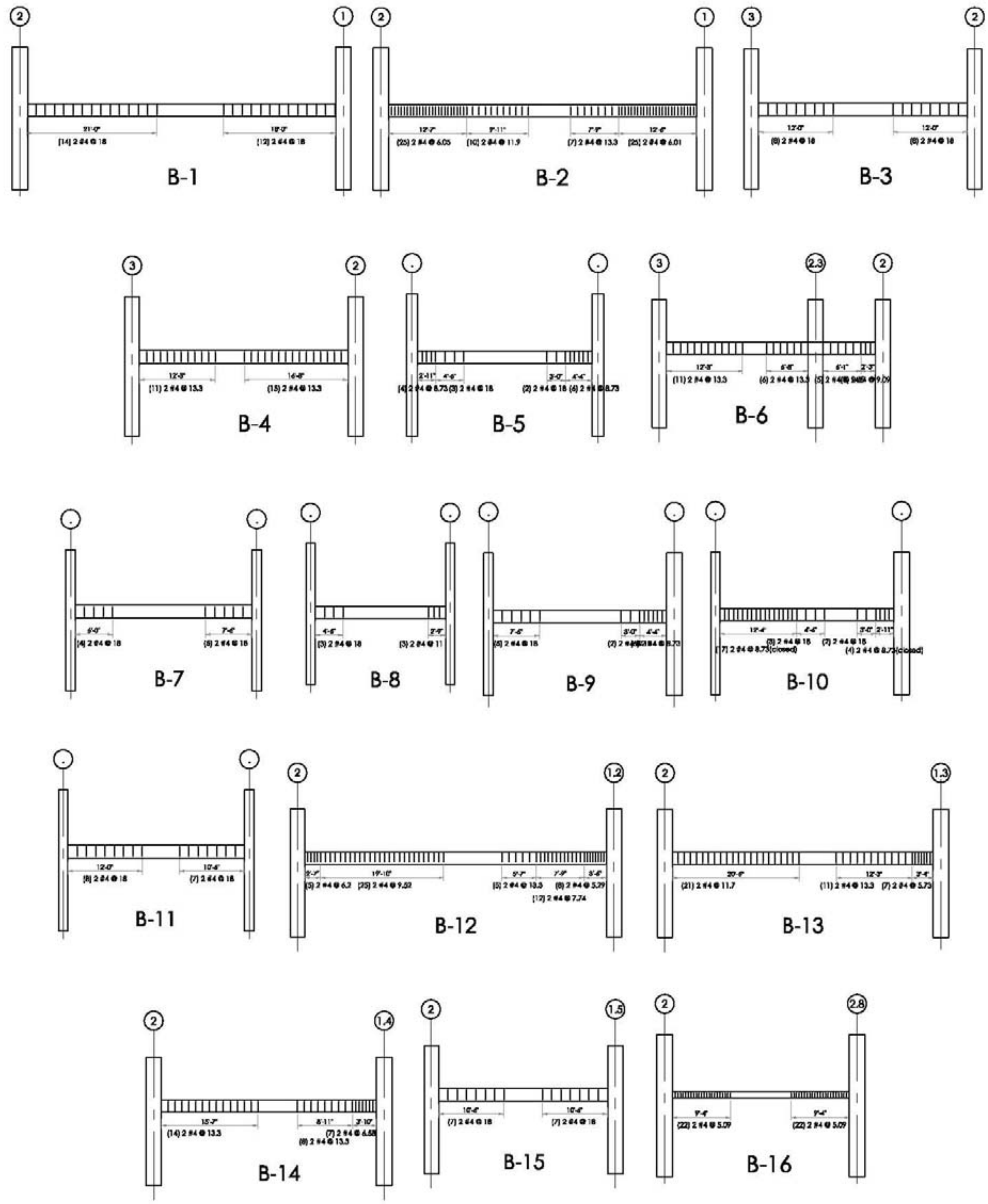
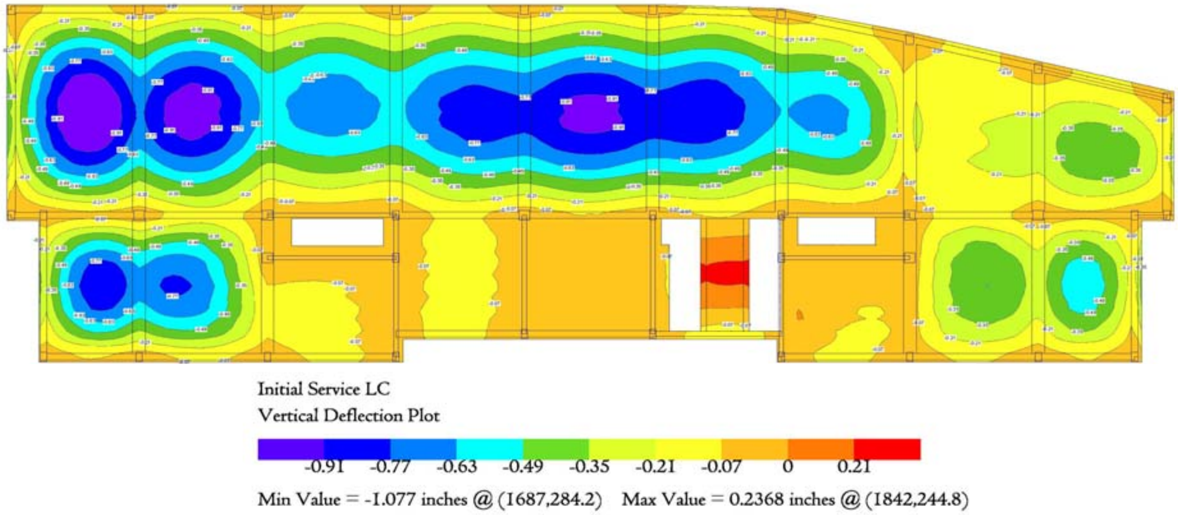
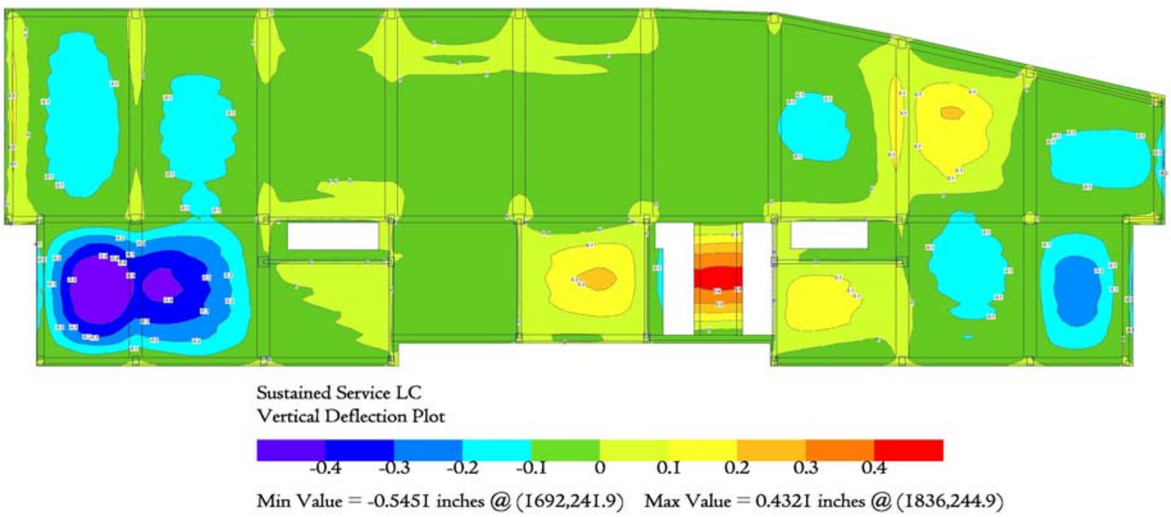


Figure 24-Transverse, shear reinforcement layouts for beams as designed by RAM Conce*pt*. See framing plan in *Appendix C.4: Post-tensioned concrete typical floor framing.*

Deflection Plans





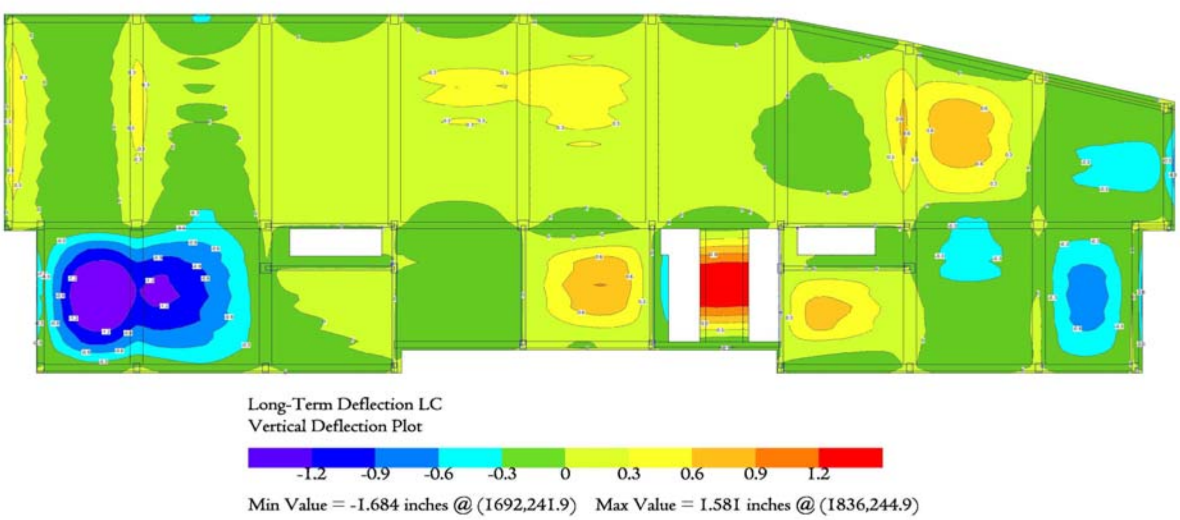


Figure 25-Deflection plans for three loading conditions from RAM Conce*pt*.

6.2 Regularly Reinforced

6.2.1 Columns

Columns were designed in a traditional manner by determining axial forces at each level and approximating moments applied to the top and bottom of the column from beams framing into it. The axial forces were found based on tributary area where live loads were reduced based on Chapter 4.8 in ASCE 7-02. Preliminary calculations can be found in *Appendix B.2: Columns*. With these axial loads and approximate moments, interaction diagrams were used to determine initial reinforcing details.

These initial column sizes were modeled in RAM Concrete and their reinforcing was analyzed more closely. The reinforcing is spliced at every other level, and patterns and bar sizes have been narrowed down for constructability purposes. All columns have transverse shear reinforcing of #3 closed bars at 9" o.c. The following page contains the final column schedule, and the foundation plan can be found in *Appendix C.3: Post-tensioned concrete foundation*.

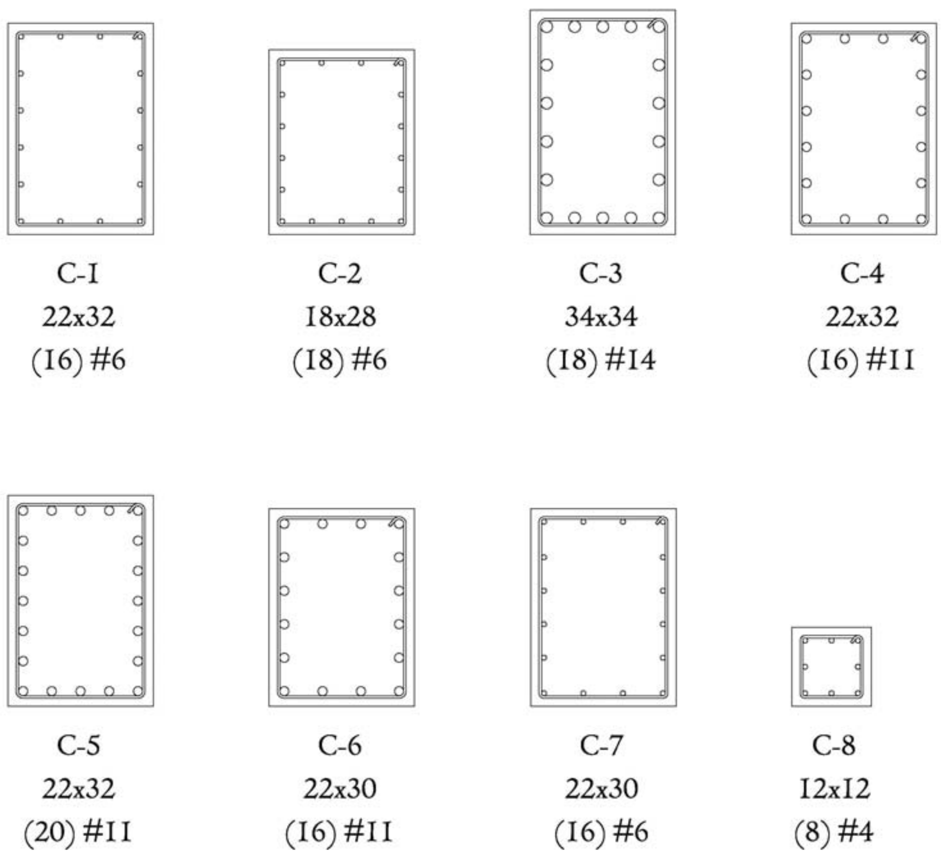


Figure 26-Reinforcing for columns (see column schedule on next page).

Floor	C-1	C-2	C-3	C-4	C-5	C-6	C-7	C-8
R	22x 22 (16)- #9 2228k	18x 18 (18)- #4 1212k	24x 24 (18)- #6 2218k	24x 24 (16)- #9 2533k	22x 22 (20)- #8 2221k	22x 26 (16)- #8 2389k	22x 26 (16)- #6 2171k	
15	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #9 2520k	22x 26 (20)- #7 2364k	22x 26 (16)- #9 2520k	22x 26 (16)- #6 2171k	
14	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #6 2171k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2171k	22x 26 (16)- #6 2171k	
13	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #6 2171k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2171k	22x 26 (16)- #6 2171k	
12	22x 26 (16)- #6 2170.740k	18x 24 (18)- #5 1647.594k	32x 32 (18)- #7 3815.760k	22x 26 (16)- #6 2170.740k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2170.740k	22x 26 (16)- #6 2170.740k	
11	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #6 2171k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2171k	22x 26 (16)- #6 2171k	
10	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
9	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
8	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
7	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
6	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
5	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	34x 34 (18)- #8 4387k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k
4	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	34x 34 (18)- #9 4534k	22x 32 (16)- #8 2827k	22x 32 (20)- #8 2950k	22x 30 (16)- #8 2681k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k
3	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	34x 34 (18)- #11 4927k	22x 32 (16)- #10 3126k	22x 32 (20)- #10 3324k	22x 30 (16)- #10 2980k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k
2	22x 32 (16)- #6 2608k	18x 28 (18)- #6 1980k	34x 34 (18)- #14 5412k	22x 32 (16)- #11 3307k	22x 32 (20)- #11 3551k	22x 30 (16)- #11 3161k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k

Table 4-Column schedule. See foundation plan in *Appendix C.3: Post-tensioned concrete-typical floor framing.*

6.2.2 Lateral System Design

Initially, the lateral system was planned to be cast-in-place shearwalls in similar locations to the braced frames in the composite steel structure. After running lateral load analyses in RAM considering all of the load combinations discussed in the section *5.3.2 Lateral Loads*, story drifts were too large in certain load combinations. Without the freedom to add more shearwalls, the concrete frames needed to be included. Because this building is designed as a cast-in-place concrete structure with connections similar to moment connections, every frame that has been designed for gravity loading can be considered in resisting lateral load. However, these frames are not enough to resist all of the lateral loads alone and need to be incorporated with the shearwalls. Concrete shearwalls are used as the main structural elements that resist lateral forces and concrete frames supplement in resisting lateral loads.

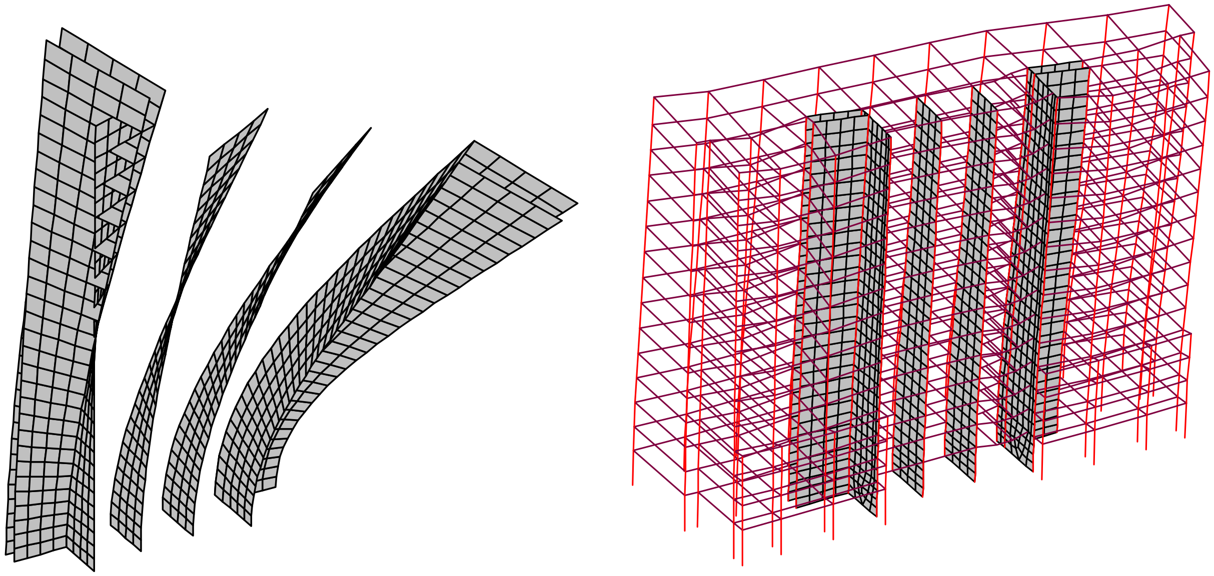


Figure 27-Lateral systems under 1.2D+0.5L+1.4E when seismic loading is in the east-west direction and considers 5% accidental eccentricity.

The shearwalls are sufficient in resisting load combinations considering dead, live, and most cases of wind loading. However, the load combinations considering seismic loading introduce large deflections, around 12” when load is applied in the east-west direction, and torsional problems, around 1.15°. Since this drift was not drastically decreased by increasing wall thickness or material strength, the introduction of concrete moment frames to help in resisting loads is necessary.

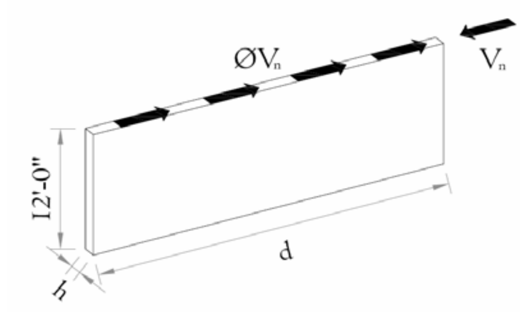


Figure 28-Shear wall dimension and loading diagram.

Shearwall Design: The reinforcement was designed to resist only shear forces, and not checked in bending for overturning. Initial designs for shearwalls were performed using the controlling load cases in each direction that were found by hand calculations. Wind loads were found to control in the north-south direction while seismic loads were found to control in the east-west direction. Shearwalls were designed using basic strength principles of $V_u \le \phi V_n$ and $V_n = V_c + V_s$ and modeled as a cantilevered beam with a series wall shears acting as point loads.

The walls were designed for the most heavily loaded level in shear and bending. The greatest loading, shear of 308 k and moment of 270 ft-k, exists in walls 1 and 4 (see diagram below) during wind loading in the north-south direction. A trial wall size of 12" thick with a compressive strength of 4000 psi were used for both directions and found to work. The steel used is #6 @ 14" in both horizontal and vertical directions.

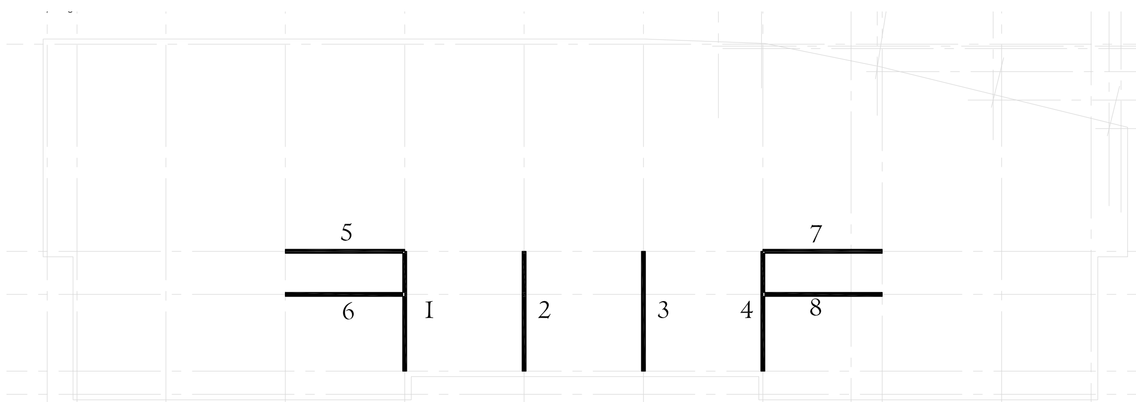


Figure 29-Location of shearwalls in plan.

To determine the distribution of lateral forces to the shearwalls, their stiffnesses were calculated by the following equation: $k = \dfrac{Et}{4(h/l)^3 + 3(h/l)}$ and the story shears were distributed accordingly. From these forces, the overturning moments and uplift at the base of each wall were found. All of these values are tabulated in the table on the next page and shown acting on the shearwalls. Refer to *Appendix B.4: Shearwalls* for further calculations on shearwalls.

Floor	N-S				E-W			
	1	2	3	4	5	6	7	8
R	29 k	17 k	17 k	29 k	14 k	14 k	14 k	14 k
15	54 k	32 k	32 k	54 k	39 k	39 k	39 k	39 k
14	80 k	47 k	47 k	80 k	63 k	63 k	63 k	63 k
13	105 k	61 k	61 k	105 k	84 k	84 k	84 k	84 k
12	130 k	76 k	76 k	130 k	102 k	102 k	102 k	102 k
11	154 k	90 k	90 k	154 k	118 k	118 k	118 k	118 k
10	178 k	105 k	105 k	178 k	132 k	132 k	132 k	132 k
9	201 k	119 k	119 k	201 k	143 k	143 k	143 k	143 k
8	223 k	133 k	133 k	223 k	152 k	152 k	152 k	152 k
7	245 k	147 k	147 k	245 k	160 k	160 k	160 k	160 k
6	265 k	161 k	161 k	265 k	166 k	166 k	166 k	166 k
5	284 k	175 k	175 k	284 k	170 k	170 k	170 k	170 k
4	300 k	191 k	191 k	300 k	173 k	173 k	173 k	173 k
3	312 k	210 k	210 k	312 k	175 k	175 k	175 k	175 k
2	308 k	226 k	226 k	308 k	176 k	176 k	176 k	176 k
Overturning	**249,327**	**150,208**	**150,208**	**249,327**	**172,127**	**172,127**	**172,127**	**172,127**
Uplift	**6,926 k**	**5,007 k**	**5,007 k**	**6,926 k**	**5,738 k**	**5,738 k**	**5,738 k**	**5,738 k**

Table 5-Shear on walls due to direct shear.

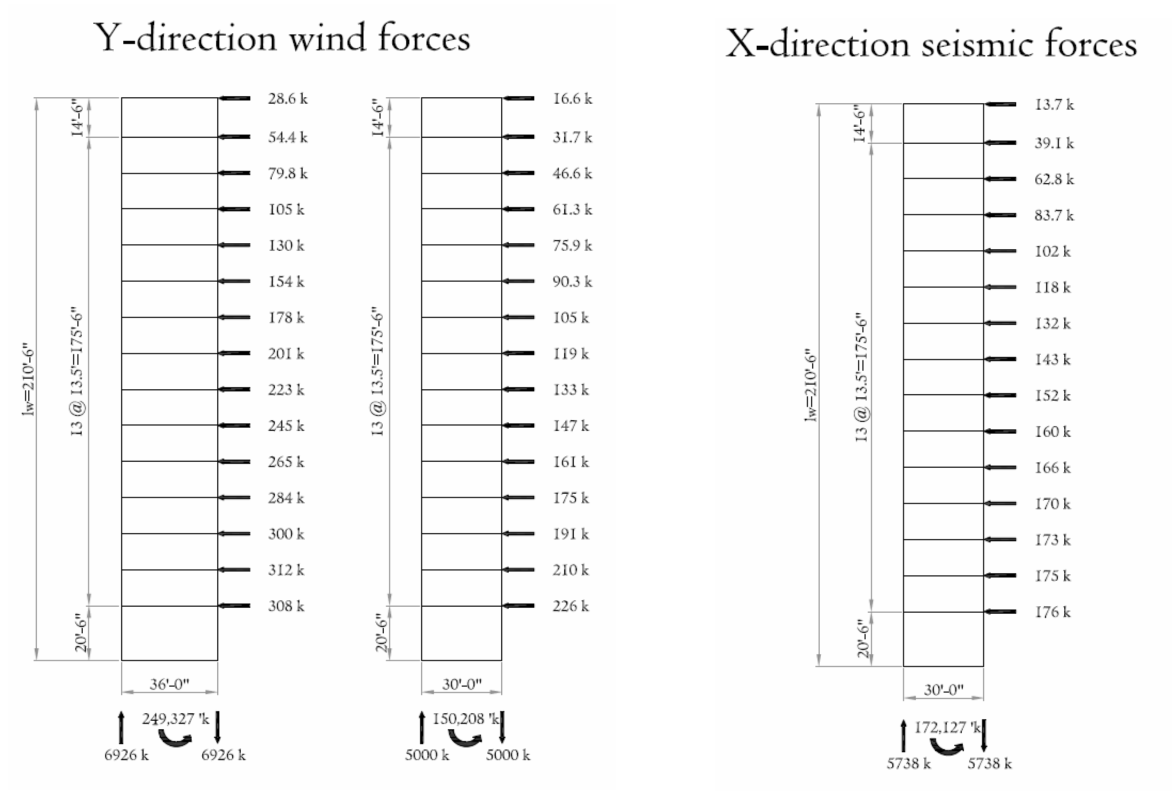


Figure 30-Direct shear acting on walls with resulting uplift and overturning moment.

6.3 Foundations

The concrete structure adds 30% more weight to the foundation when compared to the composite steel structure. The undesirable soil conditions in the city of Wilmington warrant deep foundations to support the gravity loads and overturning moment imposed on them. Since the clusters of auger-cast piles are already 10-18, increasing the size of clusters will begin to compromise the already poor soil. The objective of changing the foundations to caissons is to preserve the integrity of the soil, to limit settlement, and eliminate the need for a 60" pile cap. Their capacity was found based on the end soil bearing pressure of the caisson. For those underneath shearwalls, they will be reinforced to take tension due to overturning.

Caissons						
		Amount	**Material**	**Labor**	**Equip-ment**	**Cost**
A1020-310	4'-0" dia. x 100'	20 Ea	4358	70459		$1,496,329
	5'-0" dia. x 100'	6 Ea	8064	144990		$918,324
	6'-0" dia. x 100'	10 Ea	11730	172277		$1,840,069
					TOTAL	**$4,254,722**
Concrete Filled, Drilled Piers						
A1020-130	End Bearing Steel Piles					**Cost**
2380	4 pile cluster	5	5625	3325		$44,750
2460	6 pile cluster	8	8425	5025		$107,600
2480	7 pile cluster	7	9825	5850		$109,725
2500	8 pile cluster	5	12600	7525		$100,625
2560	12 pile cluster	9	15400	9200		$221,400
03310-240	Pile caps, incl. forms and reinf.	612	108	49	0.31	$96,309
					TOTAL	**$680,409**

Table 6-Cost estimate and comparison between caissons and concrete filled, steel piles.

In addition to the caissons, grade beams will be provided to engage all of the deep foundations when the shearwalls and frames are forced into action. Although the caissons and the grade beams have not been explicitly designed for, the impact that the structure has on them has been estimated. The foundation plan can be found in **Figure 29**-Foundation Plan.

7.0 STRUCTURAL SUMMARY

The concrete in the super-structure including: columns, girders, and slabs, will have a compressive strength of 6000 psi, but the shearwalls will be 4000 psi. The need for such high strength concrete in the gravity framing comes from the long-span feature of the office floors. In order to preserve the 52'-6" spans, the concrete needed to have enough strength to withstand the amount of stress caused by the post-tensioning force.

Post-tensioned Slab: The post-tensioned slab will be 8" thick and contribute 100 psf of dead load to the structure. The ungrouted tendons in the slab will be spaced about 6' o.c. and span the 30' direction of each bay. The tendons will be banded in groups of (6) ½" diameter strands and have yield strength of 270 ksi. They will have a parabolic profile of 6.75" above the bottom of the slab at supports and 1.75" from the bottom of the slab at mid-span.

Columns: The columns in the building range in size from 18"x28" to 30"x30". See page 28 for a full column schedule, and refer to the foundation plan in *Appendix C.3: Post-tensioned concrete-foundation* for column locations.

Beams: The beams will use post-tensioning steel with yield strength of 270 ksi. The steel strands will be grouted solid. Interior beams spanning 52'-6" will be 16"x36" and have approximately 35 strands in them, and interior beams spanning 36'-0" will be 16"x24" and have approximately 20 strands in them. In order to develop the full compressive capacity in the slab, the beams will be analyzed as T-beams. Beam designs have achieved the objective of decreasing floor depth by 7.25" over the composite steel system, when fire-proofing is considered.

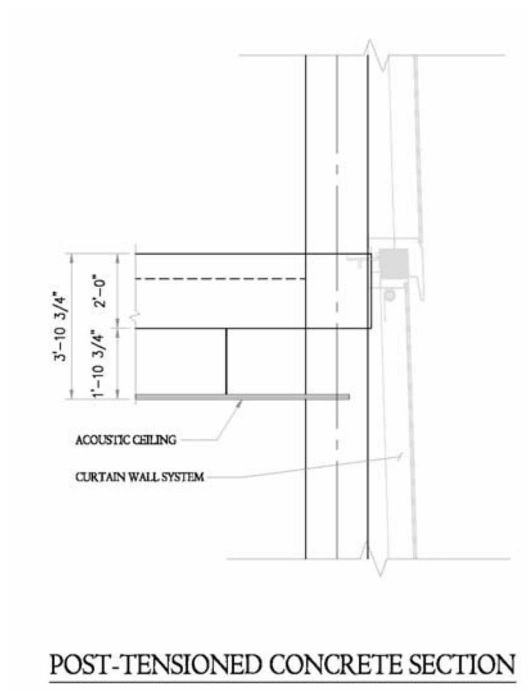


Figure 31-Section of concrete floor system.

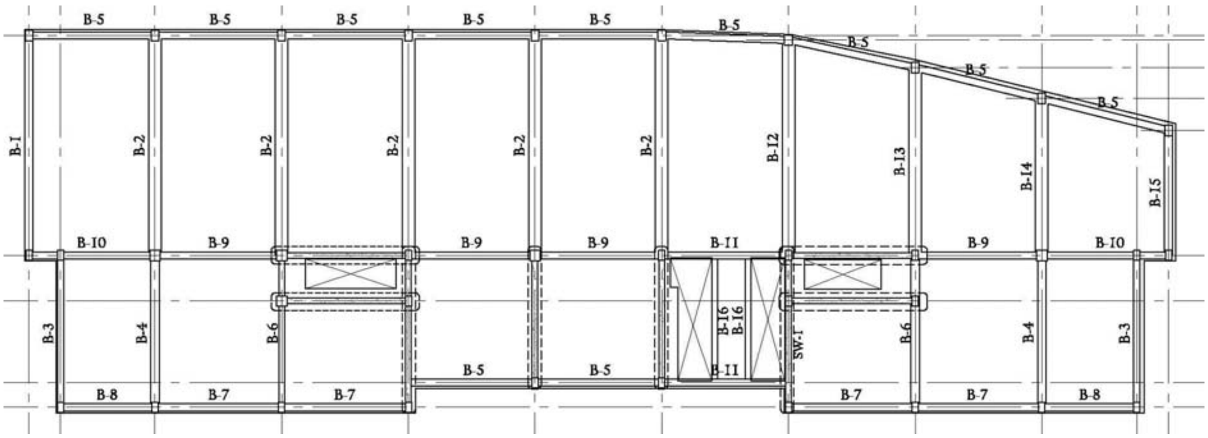


Figure 32-Post-tensioned concrete framing plan for typical floor.

Shearwalls: Shearwalls will be identical in each direction: 12” thick, 4000 psi, and #6 @ 14” both vertically and horizontally. Though shearwalls will resist a majority of the lateral loads, they are not the only lateral resisting elements.

Foundations: The foundations will be caissons to support the additional weight of the concrete frame. Their sizes range from 4’-0” to 6’-6” in diameter. The caissons under shearwalls will be reinforced at the top to prevent overturning. They will be connected by a network of grade beams to more evenly distribute load and prevent differential settlement. The foundation plan is pictured below.

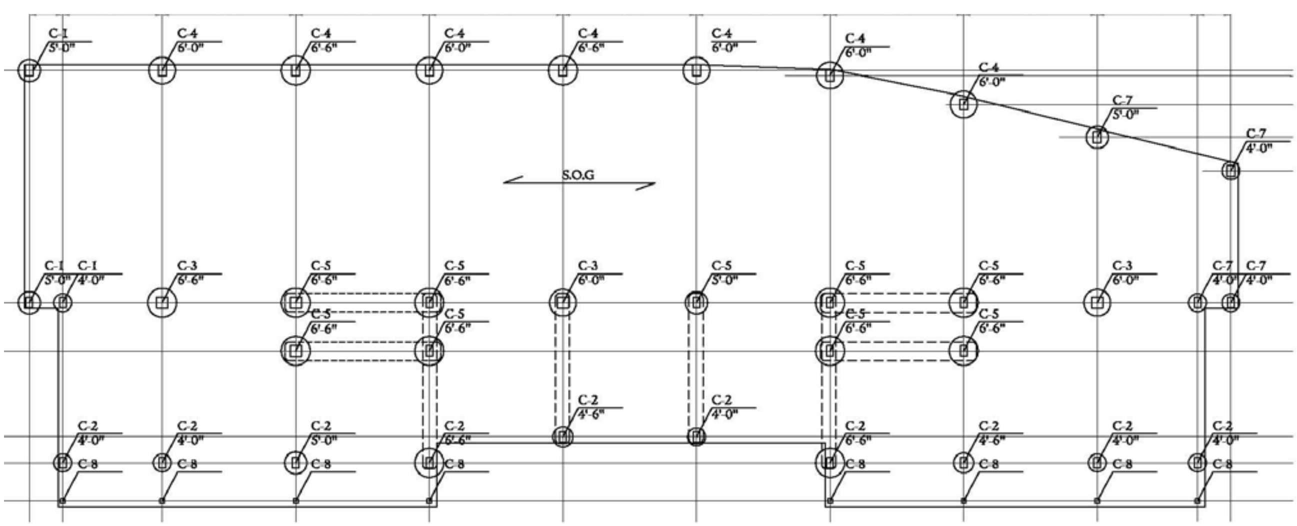


Figure 33-Foundation plan.