Structural Redesign

Floor System

Alternate 1: The first floor system that was investigated is a two-way flat plate reinforced concrete system. This system will consist of a 10" reinforced concrete slab that frames into 16" square columns. The floor to floor height will remain the same as the original design at 10'-8" which means that there is a ceiling cavity of 4" for any MEP equipment.

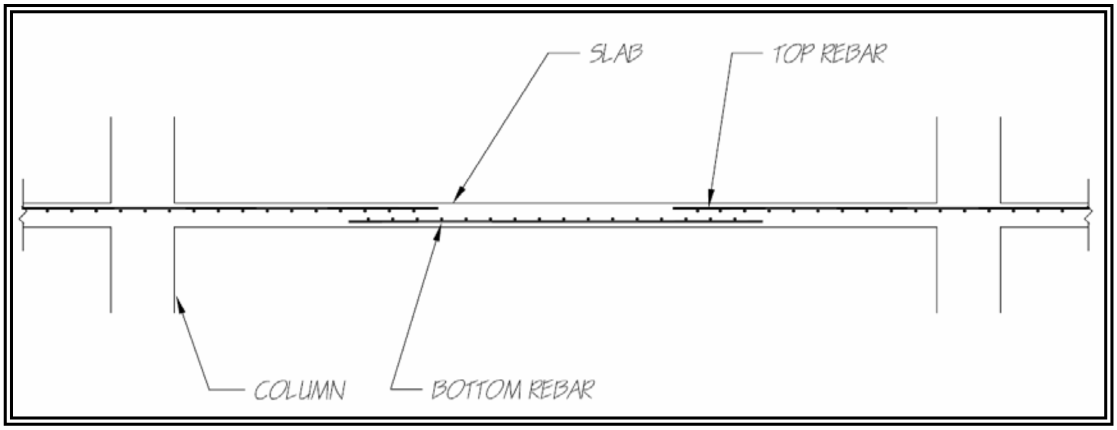


Figure 8: Cross Section of Two-way Flat Plate System

Alternate 2: The second floor system investigated was a two-way post tensioned flat plate concrete system with similar sized columns. It is the intent of this system to reduce the thickness of the floor slab to provide a larger ceiling cavity for MEP equipment. Reducing the thickness of the floor slab will reduce the dead loads into the columns and the punching shear in the slab and thus allow for smaller column sections and less overall weight that the foundation needs to support.

Design Gravity Loads

The gravity loads used for the design of both the reinforced concrete flat-slab and the post-tensioned flat-slab were gathered from ASCE7-02.

Dead Loads:

Concrete (incl. rebar)	150 PSF
Terrace Pavers	25 PSF
Ceiling - Units	10 PSF
Ceiling - Parking	3 PSF
Flooring	4 PSF
Roofing	7 PSF
Mechanical Room	250 PSF

Live Loads:

Floor – Private	40 PSF
Floor – Public	100 PSF
Balconies – if < 60 ft2	60 PSF
Balconies – if > 60 ft2	100 PSF
Fire Escapes	100 PSF
Corridors	100 PSF
Garage	40 PSF
Roof	20 PSF
Partitions	15 PSF

Snow Loads:

Roof Load	25 PSF
Drifting	25 PSF

Shear Design

Prior to beginning the reinforced concrete design, the CRSI Design Guide and the ACI code were consulted to determine preliminary sizes for the columns and the floor slabs. These preliminary estimates were based on loading, bay sizes and deflection limitations. The deflection limitations from Table 9.5(c) in the ACI dictated a 10" minimum slab depth corresponding with the 26' bays. Thinner slab sizes would be possible for the smaller bays but it was decided to stay with 10" slab for the entire floor as it would be easier during construction. Confronting the CRSI with the knowledge of a 10" slab, 26' bay size, and a superimposed load of about 50 PSF an initial column size of 14" square was determined for an interior column. It was decided to use a 15" column and concrete with an f'_c = 5000 PSI to account for the variety of loading schemes from balconies and corridors. The following equations from ACI chapter 11 were used to check and verify this decision.

Shear Loading:

$$V_u = w_u * l_1 * l_2 / 2 \quad \text{(Eq. 1a)}$$

$$V_u = w_u * l_1 * l_2 \quad \text{(Eq. 1b)}$$

Wide Beam Shear:

$$V_c = \Phi * 2 * \sqrt{(f'_c)} * b_w * d \quad \text{(Eq. 2)}$$

Punching Shear:

$$V_c = \Phi * (2+4/\beta_c) * \sqrt{(f'_c)} * b_o * d \quad \text{(Eq. 3)}$$

$$V_c = \Phi * (\alpha_s * d/b_o + 2) * \sqrt{(f'_c)} * b_o * d \quad \text{(Eq. 4)}$$

$$V_c = \Phi * 4 * \sqrt{(f'_c)} * b_o * d \quad \text{(Eq. 5)}$$

When designing the post-tensioned concrete floor system the same column sizes were used as in the reinforced concrete design but another approach was used to determine a preliminary slab thickness. Since wide beam shear rarely controls and the columns were the same dimensions in both directions equations 2 and 3 will not control, and equation 4 requires a great deal of information so it was used later as a design check. This leaves equations 5 and equation 6 (below) which were rewritten as a function of 'd' in relation to f'$_c$ of the concrete and the factored loads then solved by graphing against equation 1b for the minimum value of 'd'. This procedure is shown below using equation 5 as an example:

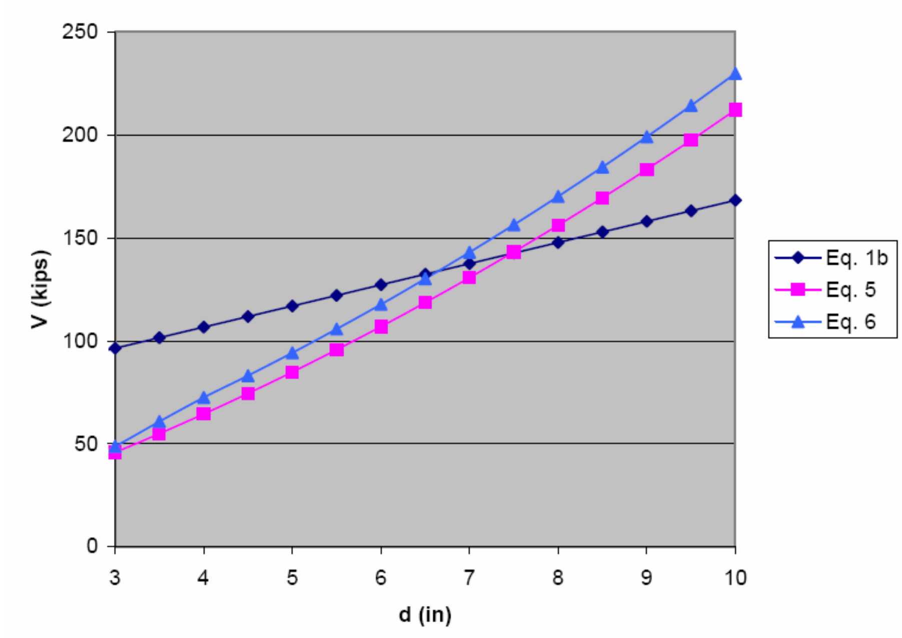
$$V_c = \Phi^* (\beta_p^*\sqrt{(f'_c)} + 0.3^* f_{pc})^*b_o^*d + V_p \qquad \text{(Eq. 6)}$$

(1) $w_u = 1.2^*(12.5^*(d+1.0)+SDL) + 1.6^*(LL)$

(2) $V_u < V_c$

(3) $w_u^*l_1^*l_2 < \Phi^*4^*\sqrt{(f'_c)}^*b_o^*d$

(4) $(15^*\mathbf{d}+22.5+1.2^*SDL+1.6^*(LL))\ ^*l_1^*l_2 < \Phi^*4^*\sqrt{(f'_c)}^*(60+4^*\mathbf{d})^*\mathbf{d}$



This approach is applicable because the post-tensioning creates forces in the slab that counter the dead loads thus greatly reducing the deflections under service loads, as well as creating a vertical force in the area around the column that opposes the shear forces which is demonstrated in the following diagram. The minimum depth of 'd' was found to be 6.5" by equation 6 so the minimum slab depth, assuming 1.0" of cover, is then 7.75" but for ease of construction and to provide greater sag in the tendons a thickness of 8.0" was used. This value was then checked against equations 2, 3, and 4 and found to be sufficient for resisting punching shear and wide beam shear.

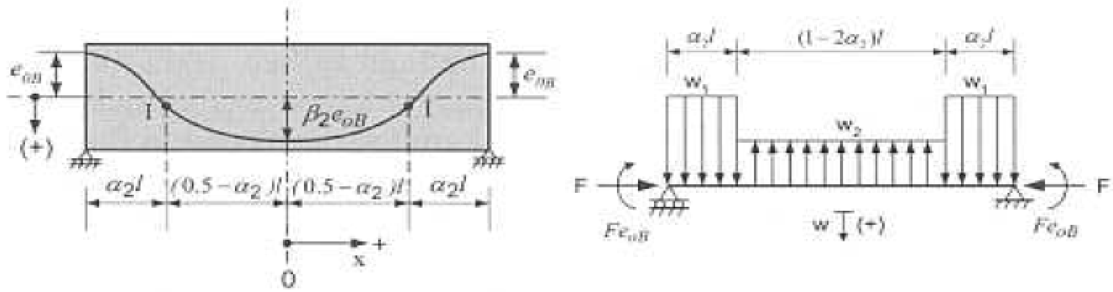


Figure 9: Parabolic tendon profile (left) and the induced forces from that profile (right).

Flexural Design

The floor system was designed a number of ways. The first two ways considered the floor slab as just a gravity force resisting element thus designing it to resist flexure from the design live and dead loads. The floor slab was first designed as a reinforced concrete flat-plate using the Direct Design method (ACI ch. 13.6) and then designed again using the Equivalent Frame analysis (ACI ch. 13.7). These two methods were then compared based on the design moments and the time to calculate each. The Direct Design method is much faster than the Equivalent Frame analysis, but it calculates lower negative moments at interior columns and therefore is un-conservative at these critical sections. The Equivalent Frame analysis, on the other hand, is un-conservative at exterior columns and mid-span of interior beams.

		Span 1		Span 2		Span 3	
		Total	Per Foot	Total	Per Foot	Total	Per Foot
Direct Design							
Interior	CS	274	21.1	254	19.5	274	21.1
	MS	92	7.1	85	6.5	92	7.1
Middle	CS	163	12.5	110	8.5	163	12.5
	MS	108	8.3	73	5.6	108	8.3
Exterior	CS	136	10.5	---	---	136	10.5
	MS	0	0.0	---	---	0	0.0
Equivalent Frame							
Interior	CS	314	24.2	293	22.5	314	24.2
	MS	105	8.1	98	7.5	105	8.1
Middle	CS	165	12.7	80	6.2	165	12.7
	MS	110	8.5	53	4.1	110	8.5
Exterior	CS	103	7.9	---	---	103	7.9
	MS	0	0	---	---	0	0

Table 1: Comparison of design moments calculated using Direct Design and Equivalent Frame Analysis

The third way the floor slab was calculated was as a post-tensioned flat-plate system. Considering the slab to be post-tensioned allowed for a thinner slab with greater section properties than a regular reinforced slab. By draping the tendons in a parabolic shape they produce a uniformly distributed load in the slab that acts in opposition to the gravity loads which reduces the deflection of the slab under service loading cases. The precompression of the slab also eliminates tension cracks, which means that the entire cross section is utilized to resist moments caused by live, dead, and even lateral loads.

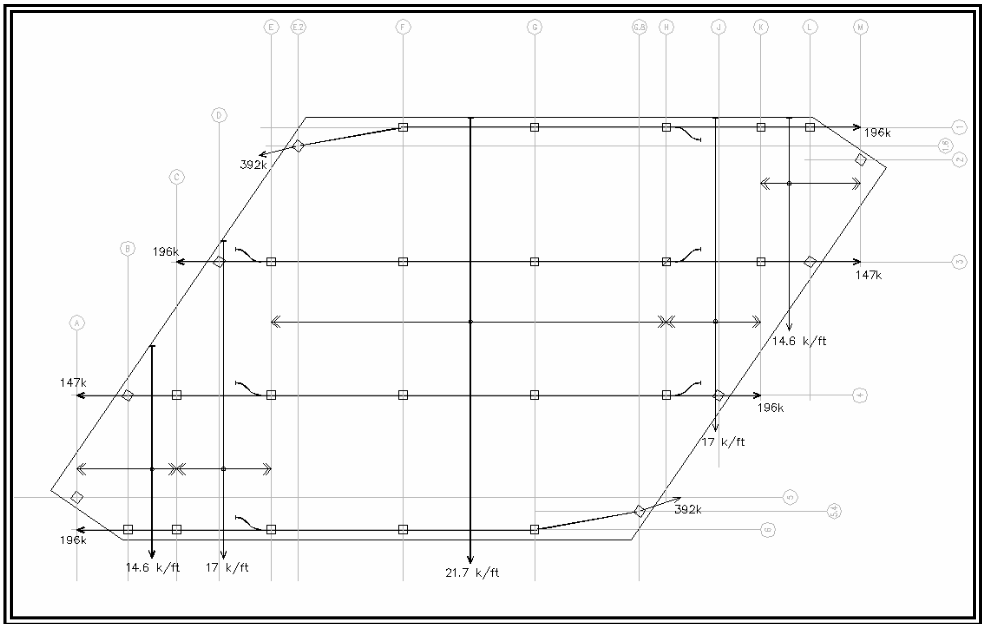


Figure 10: Typical post-tensioning tendon layout.

The fourth, and final, floor slab design considered the slab to be an integral part of the lateral force resisting frame. For this design, ADOSS was used to perform a frame analysis to determine the design moments on the slab due to the gravity loads and the controlling wind cases.

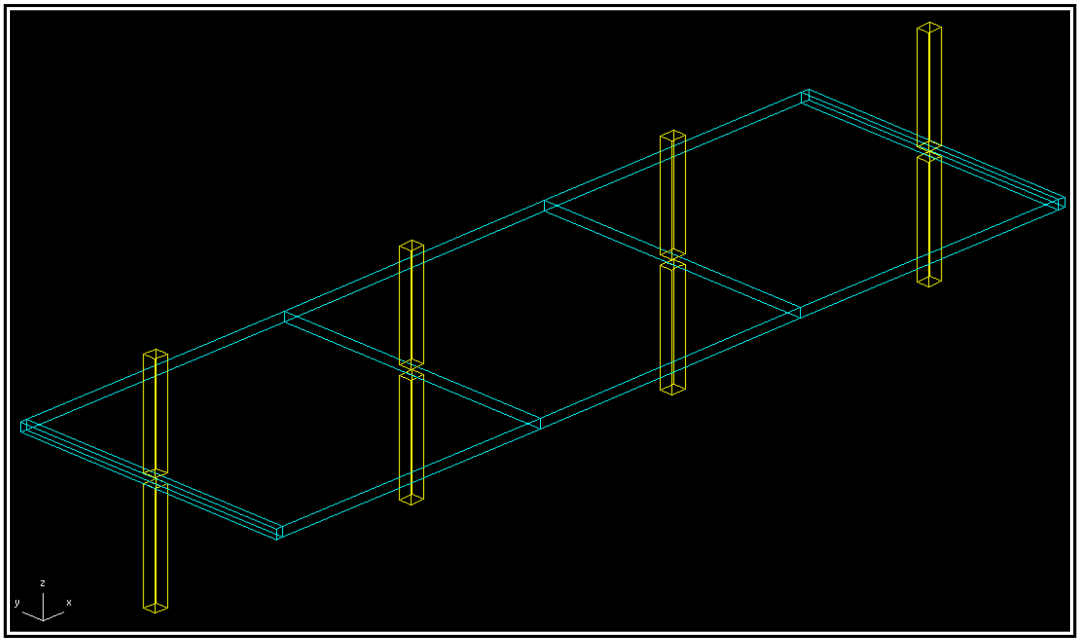


Figure 11: Isometric view of a typical N-S frame.

Columns

The initial column sizes were found during the shear design in the CRSI Handbook to be 14" square. It was decided that a 15" square column would be a better choice because of shear due to the various loading schemes from interior floor loads, terrace loads, and the loads due to the frame acting as a moment frame. The columns were designed initially for the factored axial loads that they would experience and it was found that the columns became very large and were an intrusion to the open nature of the floor plan. A second design iteration was performed using higher strength concrete and this was enough to decrease the columns to more reasonable sizes. A comparison of these sizes can be seen in Table 2 below. Due to the large axial forces on the columns towards the lower floors, they did not have to be altered to accommodate the additional moment caused by

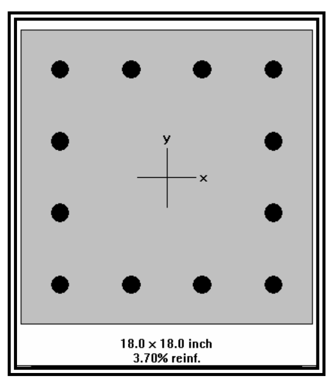


Figure 12: Column rebar design.

the frame acting to resist lateral forces. The columns, in the upper stories, where the lateral forces were greater and the axial forces lower, required additional rebar to account for the magnified moments caused by sway frames.

Column G-3, G-4, F-3, F-4 Below:	P_u (kips)	Initial Sizes		Final Sizes	
		Size	f'_c	Size	f'_c
Roof	116.3	15.0	5000	15.0	5000
Mechanical	405.3	15.0	5000	15.0	5000
24	516.6	15.0	5000	15.0	5000
23	627.8	15.0	5000	15.0	5000
22	741.8	15.0	5000	15.0	5000
21	855.9	15.0	5000	15.0	5000
20	969.9	16.0	5000	15.0	6000
19	1084.0	18.0	5000	16.0	6000
18	1198.0	18.0	5000	16.0	6000
17	1312.1	20.0	5000	16.0	6000
16	1426.2	20.0	5000	18.0	8000
15	1540.2	20.0	5000	18.0	8000
14	1654.3	22.0	5000	18.0	8000
13	1768.3	22.0	5000	18.0	8000
12	1882.4	22.0	5000	20.0	8000
11	1996.4	24.0	5000	20.0	8000
10	2110.5	24.0	5000	20.0	8000
9	2224.6	24.0	5000	20.0	8000
8	2338.6	26.0	5000	22.0	8000
7	2452.7	26.0	5000	22.0	8000
6	2566.7	26.0	5000	22.0	8000
5	2680.8	26.7	5000	22.0	8000
4	2791.5	27.3	5000	22.0	8000
3	2902.3	27.8	5000	30.0	8000
2	3013.1	28.3	5000	30.0	8000
Mezz.	3094.6	28.7	5000	30.0	8000

Table 2: Column sizes.

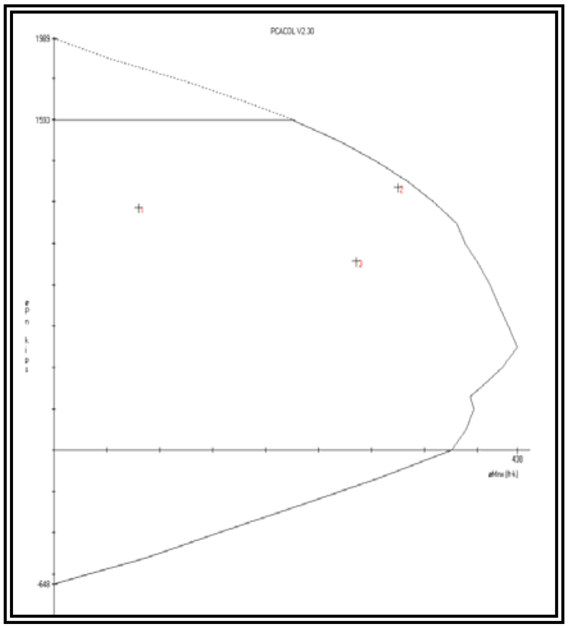


Figure 13: Column interaction diagram.

Lateral System

The first alternative lateral system that was investigated is a concrete shear wall system positioned around the elevator core that continues from the foundation through all levels of the building to the roof. The other alternative that was investigated is a system where the shear walls and slab-frame are working together to resist the lateral forces. This is possible for very little extra cost due to the fact that the columns, slabs, and shear walls are all poured monolithically which provides the moment connections needed for the frame to act integrally with the shear walls.

Lateral Design Loads

The lateral systems were analyzed for the wind and seismic loading schemes set forth by the Analytical Method from ASCE7-02 chapter 6 and the Equivalent Lateral Force Method from ASCE7-02 chapter 9, respectively. The lateral loads determined from these industry accepted procedures were then be put into an ETABS model of the building where the forces in each component will be calculated based on relative stiffness. The walls were then designed based on the worst load combination of gravity and lateral forces. Below are the lateral forces for each of the wind cases and the seismic forces for the shear wall and combined system. The criteria for calculating these loads can be found in appendices A2 through A4.

	Wind								Seismic			
	Case 1		Case 2		Case 3		Case 4		10" Slab		8" PT Slab	
Story	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	Shear Walls	Frame	Shear Walls	Frame
Roof	22.05	11.55	16.54	8.66	16.54	8.66	12.41	6.50	14.80	16.44	12.08	13.43
Mechanical	36.39	34.62	27.29	25.96	27.29	25.96	20.49	19.49	41.60	46.23	37.49	41.66
24	39.60	26.87	29.70	20.15	29.70	20.15	22.29	15.13	29.82	33.13	24.76	27.51
23	38.58	26.22	28.93	19.66	28.93	19.66	21.72	14.76	27.45	30.50	22.79	25.33
22	37.75	25.75	28.32	19.31	28.32	19.31	21.26	14.50	25.19	27.98	20.91	23.24
21	43.85	25.75	32.89	19.31	32.89	19.31	24.69	14.50	25.47	28.30	21.15	23.50
20	43.85	25.75	32.89	19.31	32.89	19.31	24.69	14.50	23.18	25.76	19.25	21.39
19	43.85	25.75	32.89	19.31	32.89	19.31	24.69	14.50	21.00	23.33	17.43	19.37
18	42.86	25.26	32.15	18.95	32.15	18.95	24.13	14.22	20.74	23.05	17.22	19.14
17	41.88	24.77	31.41	18.58	31.41	18.58	23.58	13.95	18.58	20.65	15.43	17.15
16	41.39	24.53	31.04	18.40	31.04	18.40	23.30	13.81	16.55	18.38	13.74	15.26
15	41.14	24.41	30.85	18.31	30.85	18.31	23.16	13.74	14.62	16.25	12.14	13.49
14	40.15	23.92	30.12	17.94	30.12	17.94	22.61	13.47	12.82	14.25	10.65	11.83
13	40.15	23.92	30.12	17.94	30.12	17.94	22.61	13.47	10.99	12.21	9.12	10.14
12	39.17	23.44	29.38	17.58	29.38	17.58	22.05	13.19	9.57	10.64	7.95	8.83
11	39.17	23.44	29.38	17.58	29.38	17.58	22.05	13.19	8.13	9.03	6.75	7.50
10	37.94	22.83	28.45	17.12	28.45	17.12	21.36	12.85	6.80	7.55	5.64	6.27
9	37.53	22.62	28.14	16.97	28.14	16.97	21.13	12.74	5.59	6.21	4.64	5.15
8	36.34	22.04	27.25	16.53	27.25	16.53	20.46	12.41	4.50	5.00	3.73	4.15
7	35.60	21.67	26.70	16.25	26.70	16.25	20.04	12.20	3.52	3.91	2.93	3.25
6	36.23	22.16	27.17	16.62	27.17	16.62	20.40	12.48	2.60	2.89	2.15	2.39
5	34.14	21.00	25.61	15.75	25.61	15.75	19.22	11.82	1.83	2.03	1.51	1.68
4	30.71	19.01	23.04	14.25	23.04	14.25	17.29	10.70	1.14	1.27	0.92	1.02
3	30.05	18.72	22.53	14.04	22.53	14.04	16.92	10.54	0.73	0.81	0.59	0.66
2	35.51	22.33	26.63	16.75	26.63	16.75	19.99	12.57	0.40	0.45	0.33	0.36
Mezzanine	39.85	25.39	29.89	19.04	29.89	19.04	22.43	14.29	0.03	0.04	0.03	0.03
Ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 3: Wind and seismic forces.

Shear Walls

The building was first designed as if shear walls were going to resist the lateral forces caused by the wind and seismic forces. In doing so a drift limit of $\ell/400$ was imposed to prevent cracking of the façade; this limit was found in *ASCE7-02 CB.1.2 Drift of Walls and Frames*. The original intent was to have the shear walls around just the elevator core, but not only was this not enough to limit the deflections to the drift limit but the setbacks of the upper stories created a large amount of torsion that needed to be controlled. Therefore, more shear walls were needed. The architectural plans were consulted to determine likely places to add shear walls without disrupting the current floor plan. It was not initially evident where these additional walls would go because of the variation of the floor plans. It was finally decided to put them along same column lines that some of the original braces were located with openings in them at intermittent floors for doorways between the different rooms of the condominiums. This design was enough to bring the drift limit down to $\ell/560$ which is considerably less than the required $\ell/400$.

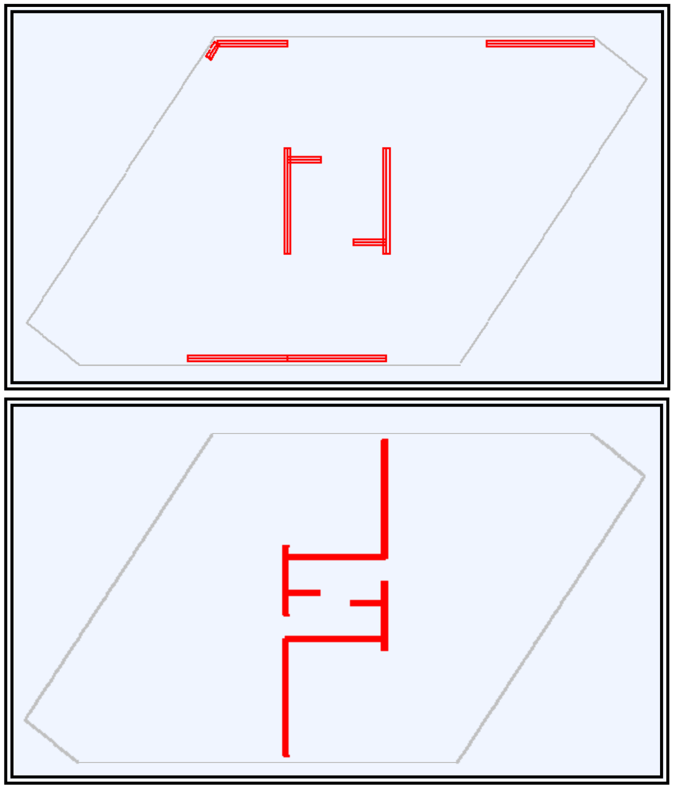


Figure 14: Final shear wall locations (top) stories 1-5 and (bottom) stories 6 to the roof.

Frame System

A frame system was initially investigated to attempt to reduce the materials needed for construction by combining two different types of lateral force resisting systems, the frame and the wall. By introducing the frame system, less shear walls may be required to satisfy the drift limit. This would be true due to the inherent nature of each of the two systems. The frame system deflects in shear and the shear walls defect in flexure. When these two systems are combined they produce a double curvature in the deflected shape of the building which is much stiffer than either of the two systems alone. This interaction is demonstrated below:

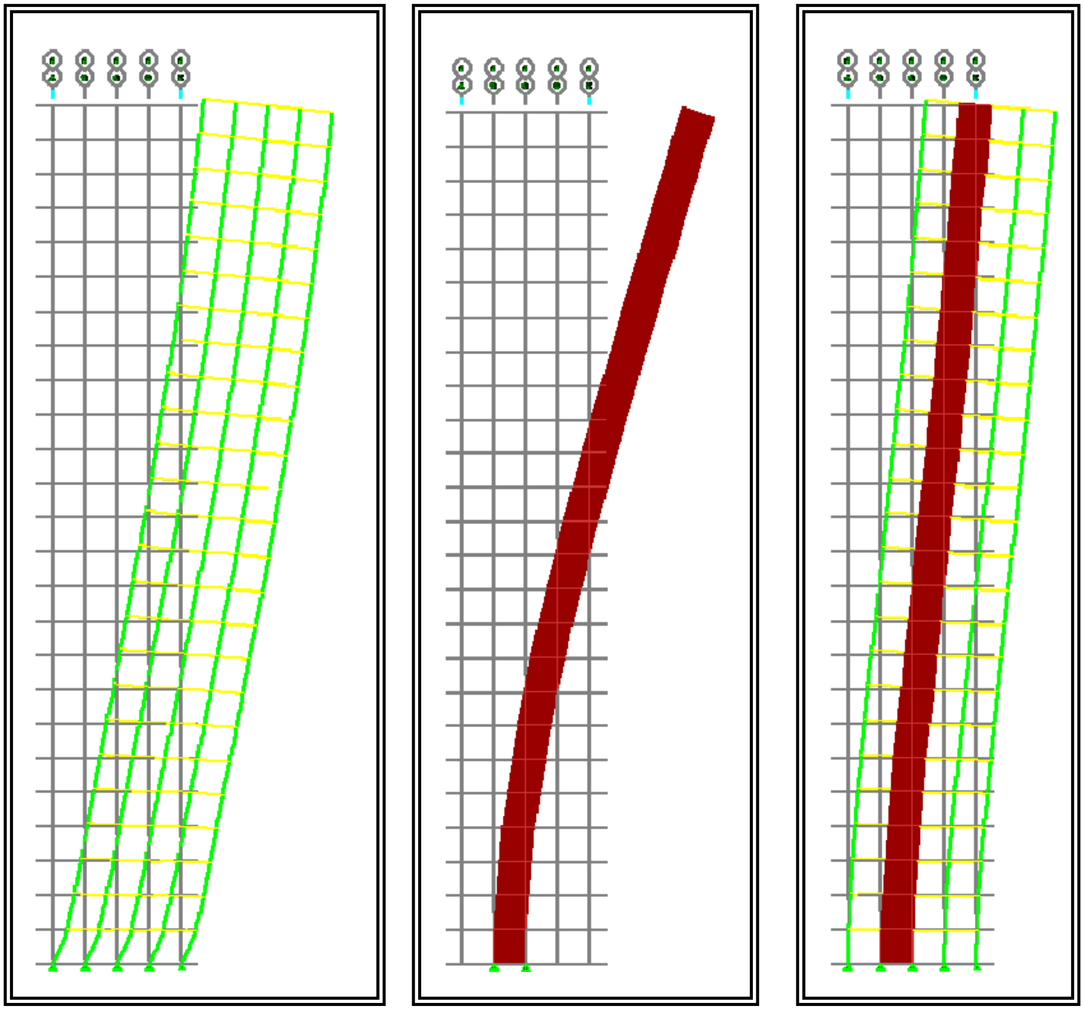


Figure 15: (Left) Frame reaction to lateral forces.
(Middle) Shear wall reaction to lateral forces.
(Right) Deflection of integrated shear wall and frame system.

This frame system was first investigated using the 10" flat-plate floor system and 15" square columns from the roof to the 5th floor and 30" columns from there to the ground floor. By incorporating the frame into the model used to determine the shear walls, the drift drastically reduced from the $\ell/560$ with just the walls to $\ell/1015$. From this point shear walls were removed until the total drift was $\ell/840$. Removing shear walls beyond this point created either excessive deflection or a severe torsional state.

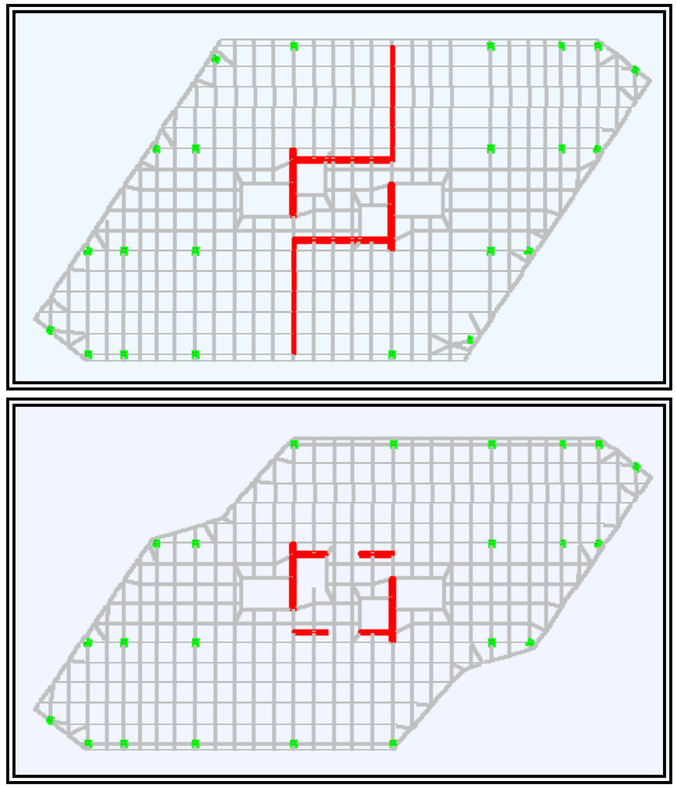


Figure 16: Final shear wall locations (top) stories 5-15 and (bottom) stories 16 to the roof.

The second frame system investigated used the 8" post-tensioned flat-plate system with the same column arrangement as the prior frame. The greatest difference between this system and the previous frame system is that the floor is much stiffer because the post-tensioning allows for the full moment of inertia of the slab to act in bending. From the shear wall model the addition of the post-tensioned floor slabs reduced the drift from the original $\ell/560$ to $\ell/1180$. This was greater than the reduction of the reinforced concrete floor system

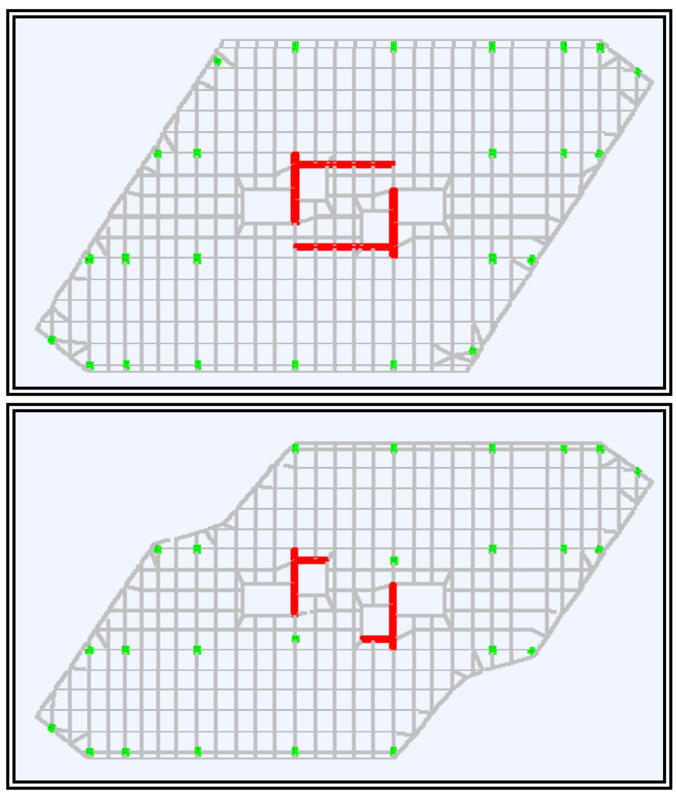


Figure 17: Final shear wall locations (top) stories 5-15 and (bottom) stories 16 to the roof.

of the previous model, as expected. In this model two entire shear walls were removed and still the overall drift was ℓ/510 which is well within the limit of ℓ/400.

The shear wall layout of the three separate designs is further demonstrated below. The image on the left shows the location of the shear walls of the model in which the shear walls alone are resisting the lateral forces. The image in the middle is of the flat-plate design where the frame and shear walls are acting together. Finally, the image on the right is of the post-tensioned, flat-plate floor system with the frame and shear walls acting integrally. Notice in the PT model how many fewer shear walls are required to meet the drift limits because of the floor slab being much stiffer than the conventionally reinforced flat-plate.

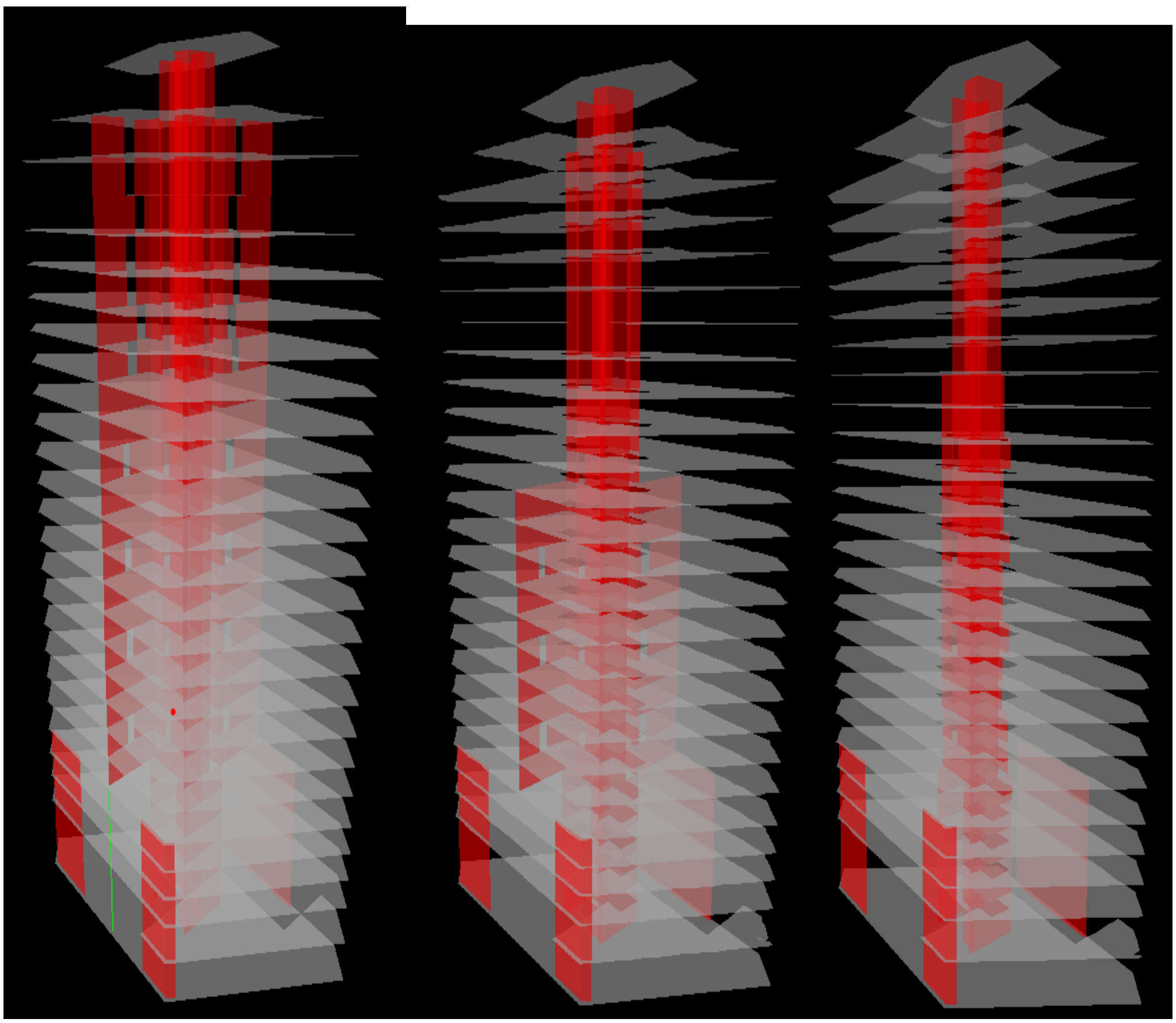


Figure 18: Shear wall layout of shear wall system (left), RC frame (middle), and PT frame system (right)