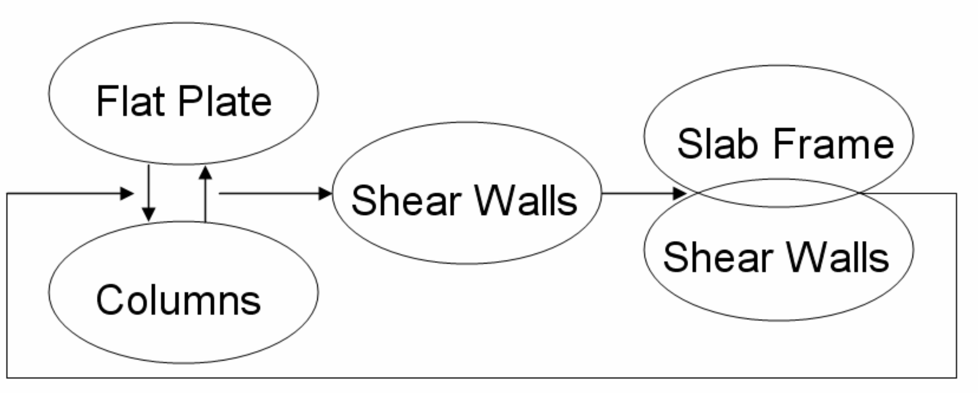
Structural Redesign

Design Criteria

I redesigned floor system with a conventionally reinforced flat plate design without significant alterations to existing architecture and building systems. The flat plate was the typical floor system used throughout the residential towers of the Odyssey. The tower structure and corresponding levels will therefore be the focus of the redesign encompassing the gravity and lateral systems.

Design objectives of the redesign include maintaining existing ceiling heights within residential units without exceeding the maximum building height limitation. The proposed redesign will be investigated through alterations of the flat plate system and corresponding adjustments in the column and lateral system. The design loads for proposed structural redesign will be in accordance with provisions of ASCE 7-02.

The design of the flat plate was a cyclical process and was preliminarily designed for gravity loading then redesigned into the lateral system. The flat plate was integrated into the lateral design as a slab frame system with the shear walls. A diagram of the process is shown below with alterations to each system described in their respective design sections throughout the report.



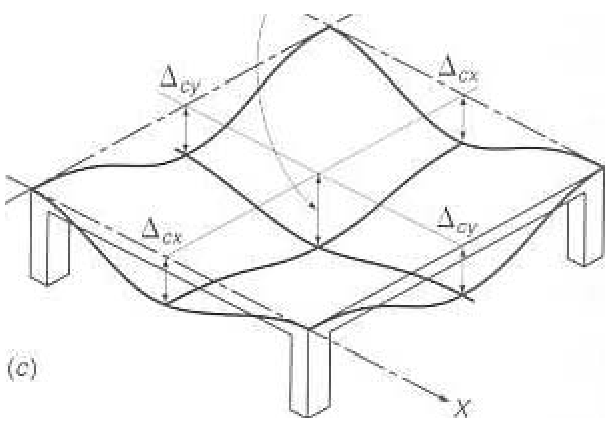
Flat Plate Design

Deflection

The minimum plate thickness was calculated based on developed methods for slab deflection control under service loads of the ACI 318-05 Building Code Requirements for Structural Concrete. Exterior and interior flat plate panels with reinforcement strength of 60,000psi are limited to a minimum thickness equal to $\ell_n/30$ and $\ell_n/33$. I determined the maximum design span length between adjacent offset columns to be 28'-6". The minimum slab thicknesses were calculated for as 11" for exterior panels and 10" for interior panels.

I carried out a design check to ensure a minimum thickness of 11" for 5000 psi exterior panels. A control panel with a size of 28'-6" x 24'-6" was analyzed for maximum column and middle strip deformations resolved in either span direction. The overall deflection was limited to $\ell/480$ for long-term deflection due to all dead loads and short term deflection due to live loads. The design check concluded that the minimum slab thickness for exterior panels would remain 11". The figure below depicts two-way flat plate deflection from superimposed strip deflections.

$$\boldsymbol{\Delta_{max} = \Delta col_{\,x,y} + \Delta mid_{\,y,x}}$$



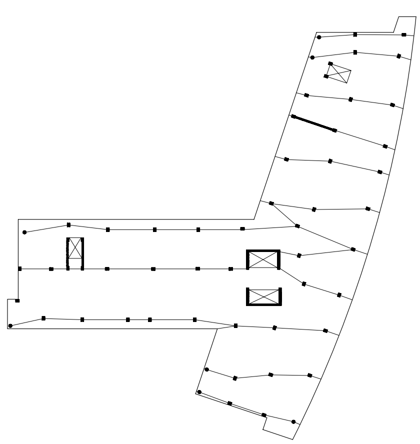
Flat Plate Deflection – *(Reference: Design of Concrete Structures, Nilson)*

Punching Shear

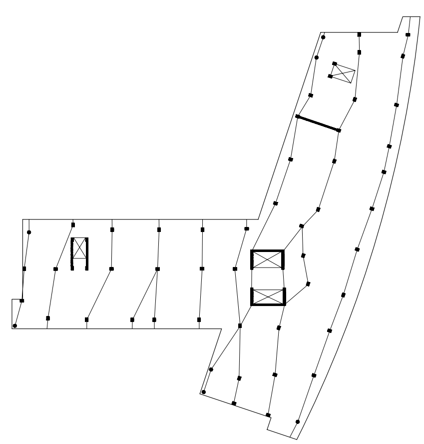
Shear design considerations were also likely to control the slab thickness for the flat plate system. I investigated the minimum design thickness of 11" under both beam shear and punching shear failure to determine which condition would control the design. Several columns sized for the 11" slab weight by axial loads from a load take-down were considered for the shear design limitations. These columns included interior, exterior, and corner locations on the floor plan. Punching shear was found to control over beam shear for loading on the tributary area at each column location. Deflection and punching shear ultimately limit the minimum design slab thickness to 11"after considering the concrete nominal shear strength capacities in accordance to ACI 318-05 (11.12). The minimum design thickness was sufficient for punching shear failure and would not need additional shear reinforcement at column interfaces. However, additional shear reinforcement may be required to resist the unbalanced moment transfer through shear and will be addressed in the frame analysis and reinforcement design sections.

Gravity Design

The Equivalent Frame method was chosen for the design of the proposed 11" flat plate system. The Direct Design Method was not used based on design limitations resulting from offset column locations and uneven span orientations in each frame direction Load path configurations were created for the frames throughout the floor plan in both grid directions. Support lines spanning between bays indicate the assumed load path from the slab into reinforcement placed at the columns. The figures below depict the assumed support lines of the flat plate system.

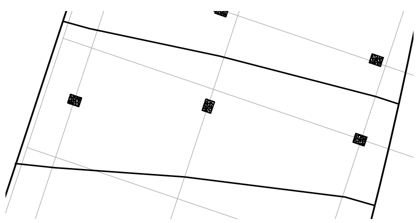


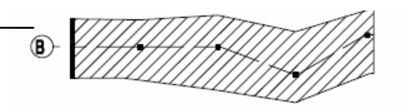
Horizontal Support Line Configuration



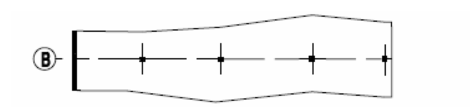
Vertical Support Line Configuration

Design strips for each support line were created by expanding a tributary width about the midpoint of each span. The design strips will be used in the frame analysis to determine design moments by the transfer of factored loads through each frame. The typical floor plan contains both straight and offset column arrangements creating numerous design strips for the flat plate system. The straight frame arrangements have relatively rectilinear design strips which suite input protocol for most computer analysis programs. As a result, the offset strips designed in PCA ADOSS were reconfigured and idealized to specified widths for a straight frame arrangement. The following figures depict an overview of the idealization process for offset design strips.

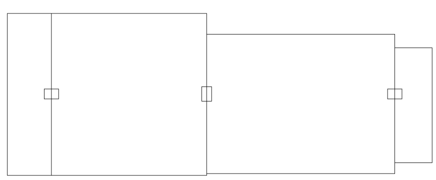


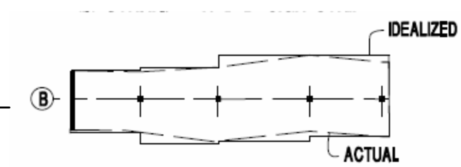


(a) DESIGN STRIP IN PROTOTYPE



(b) STRAIGHTENED DESIGN STRIP





(c) IDEALIZED TRIBUTARY FOR DESIGN

I first analyzed a rectilinear frame with the ADOSS program for design moments of 1.2D + 1.6L load combination. This frame is relatively straight when compared to the offset columns located in the skewed tower section and is an easier design check for the proposed computer analysis. A concrete strength of 5000 psi was analyzed with imposed residential level dead load and live load patterns.

Dead :		**Live**:	
Roof	50 psf	Roof	30 psf
Mechanical	150 psf	Mechanical	150 psf
Residential	27 psf	Residential	40 psf
Façade	32 psf	Public Space	100 psf

I checked the computer design moments for the strip using the Equivalent Frame method with torsion members developing an equivalent stiffness for moment distribution to supporting columns. I distributed moments over the frame using the calculated member stiffness. Multiple live load patterns were also analyzed for the distribution to determine maximum negative and positive design moments. The results of the hand check were conservative compared to the computer analysis. I believe the difference was a result of the span to column width ratios assumed for rotated columns when calculating member stiffness. I felt the computer analysis results were accurate and properly accounted for the column orientations in the frame. I concluded that ADOSS was an appropriate means of developing the design moments in the remaining frames of the flat plate for the reinforcement design.

Reinforcement

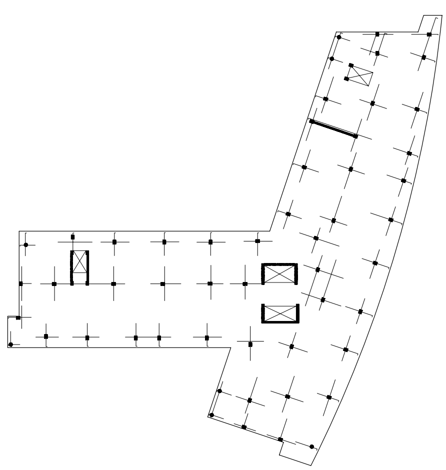
The design criterion of the positive and negative reinforcement in the flat plate was based on material efficiency. Several bar sizes were investigated for overall material quantity required to resist the distributed design moments, specifically #4, #5, #6, #7 bars. I decided that alternating positive and negative reinforcement bar sizes would limit errors during placement, increase efficiency per required spacing, and decrease excessive bar clustering. The two series of reinforcement I decided to analyze were #4 / #6 bars and #5 / #7 bars.

Column and middle strip distribution percentages were calculated in accordance with ACI 318-05 (13.6) and checked against ADOSS strip distribution percentages. Reinforcement was designed for minimum shrinkage and temperature limitations and to resist the distribution of design moments within the designated strips. A portion of the negative reinforcement was designed within effective column width to resist the flexural transfer of unbalanced moment at supports. Offset strips were designed with column strip reinforcement spaced over the entire panel to ensure adequate load path distribution into the supports. Additional shear stresses caused by the unbalanced moments at supports were under the allowable limit. The design thickness was sufficient for punching shear failure and would not require additional reinforcement.

Both sets of reinforcement were designed in each frame to compare the quantity of bars used in the design. The larger reinforcement set required fewer bars to resist the design moments, however it was necessary to consider the tonnage of each design for comparison of material quantity. The approximate reinforcement weights were calculated using minimum development lengths for two-way flat plates in accordance with ACI 318-05 (13.3.8).

Minimum Development Lengths

The overall weight of steel for #5 / #7 bars was 46.3 tons, compared to only 41.1 tons for #4 / #6 bars. Potential cost savings in material alone suggest that the smaller bar pattern with tighter spacing a more viable option. The lighter reinforcement is also preferable for distribution and placement in the field over heavier reinforcement. The design of the flat plate will use #4 bars for positive reinforcement throughout spans and #6 bars for negative reinforcement at the columns as depicted in the floor plan below.

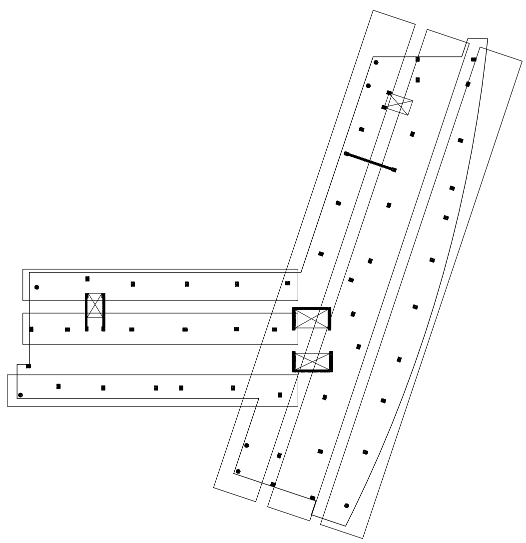


Negative Reinforcement Configuration

Lateral Design

The proposed design of the lateral system was originally for shear walls to contribute 100% of the lateral force resistance. The design was altered to incorporate the flat plate system in combination with the shear walls in a slab frame action. Details of the design alteration are covered in the Lateral System Design section. As a result of the alteration, the flat plate must be considered as a lateral resisting element and designed to resist lateral load effects.

The flat plate was redesigned with the main lateral contribution from the larger frame sections. The frames resist direct loading and torsion effects in combination with shear walls oriented in the same principle directions. An assumed distribution of 10% of the total lateral story force was applied to each frame aiding the shear walls in resisting lateral loads. The frames and accompanying shear walls are depicted in the floor plan below.

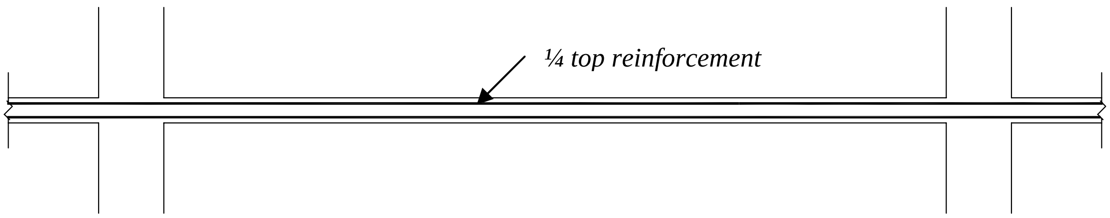


Slab Frames / Shear walls

The frames were analyzed in ADOSS under the lateral load combination 1.2D+1.0L+1.0E. Lateral forces were applied to the frame as were live load patterns. Design moments increased as well as the unbalanced moments caused by lateral force dissipation in the frame by shear transfer at the columns. The induced stress from additional shear transfer of the unbalanced moments did not exceed the allowable stress of the flat plate.

Reinforcement

Column and middle strip reinforcement was designed with the same moment distribution procedure used for the gravity load analysis. The minimum development lengths for the flat plate design were adjusted for lateral loading. A minimum of one quarter of the negative reinforcement is required to extend the full length of the span. The adjustment increased the overall weight of #4 / #6 bars to 42.4 tons. I developed a series design tables to develop column and middle strip reinforcement and to calculate the total weight of steel for the design. An example of a design table for a lateral resisting frame is shown below with designated reinforcement for column and middle strips and total calculated steel weights.



Lateral Minimum Development Lengths

Frame_6.3

Location MAX / U Moment (ft-k)		Strip	% (Gamma)	Design Moment (ft-k)	Total width (ft)	Moment/foot of width (ft-k/ft)	Rebar Design #Size (#)	S (in)	Quantity Total (# of bars)	C.S./M.S. (# of bars)	Length (ft)	Weight (plf)	Total Wt. (lbs.)
Support	1	Column Strip	0.98	118.9	13.1	9.1	6	18	9	13	8	1.502	175.2
167.3		*Unbalanced M_f*	*0.60*	*100.4*	*3.3*	*30.4*	*6*	*6*	*7*		28	1.502	138.1
121.3		Middle Strip	0.02	2.4	7.1	0.3	6	18	5	5			
Span	2	Column Strip	0.60	144.8	13.1	11.1	4	8	20	20	28	0.668	633.1
241.4		Middle Strip	0.40	96.6	7.1	13.6	4	6	14	14			
Support	2	Column Strip	0.75	235.1	13.1	17.9	6	12	13	17	18	1.502	468.6
173.7		*Unbalanced M_f*	*0.62*	*107.7*	*3.7*	*29.1*	*6*	*6*	*7*		24	1.502	151.4
313.5		Middle Strip	0.25	78.4	7.1	11.0	6	18	5	5			
Span	3	Column Strip	0.60	44.8	13.1	3.4	4	12	13	13	24	0.668	323.8
74.6		Middle Strip	0.40	29.8	7.1	4.2	4	12	7	7			
Support	3	Column Strip	0.75	182.0	13.1	13.9	6	12	13	15	15	1.502	359.3
158.9		*Unbalanced M_f*	*0.62*	*98.5*	*3.7*	*26.6*	*6*	*8*	*6*		26	1.502	146.0
242.6		Middle Strip	0.25	60.7	7.1	8.5	6	18	5	5			
Span	4	Column Strip	0.60	108.2	13.1	8.3	4	12	13	13	26	0.668	412.5
180.4		Middle Strip	0.40	72.2	7.1	10.2	4	8	11	11			
Support	4	Column Strip	0.75	229.7	13.1	17.5	6	12	13	14	15	1.502	340.5
127.1		*Unbalanced M_f*	*0.62*	*78.8*	*3.7*	*21.3*	*6*	*10*	*4*		26	1.502	135.1
306.2		Middle Strip	0.25	76.6	7.1	10.8	6	18	5	5			
Span	5	Column Strip	0.60	91.3	13.1	7.0	4	12	13	13	26	0.668	375.5
152.2		Middle Strip	0.40	60.9	7.1	8.6	4	10	9	9			
Support	5	Column Strip	0.75	242.3	13.1	18.5	6	10	16	17	15	1.502	391.0
133.9		*Unbalanced M_f*	*0.62*	*83.0*	*3.7*	*22.4*	*6*	*8*	*6*		26	1.502	164.3
323		Middle Strip	0.25	80.8	7.1	11.4	6	18	5	5			
Span	6	Column Strip	0.60	118.1	13.1	9.0	4	10	16	16	26	0.668	458.0
196.9		Middle Strip	0.40	78.8	7.1	11.1	4	8	11	11			
Support	6	Column Strip	0.75	80.3	13.1	6.1	6	18	9	13	16	1.502	348.1
167.7		*Unbalanced M_f*	*0.60*	*100.6*	*3.2*	*31.4*	*6*	*6*	*6*		26	1.502	126.9
107		Middle Strip	0.25	26.8	7.1	3.8	6	18	5	5			

A_{min} = .0018bd, .24 in^2/ft A_{max} = .008bd, 1.056 in^2/ft

#6 1.5 tons
#4 1.1 tons

Summary

The proposed flat plate system was designed with a structural thickness of 11” to meet a $\ell/480$ deflection limitation and to resist punching shear failure. I designed the reinforcement for the system as a combination of #4 / #6 bars with a total design weight of 42.4 tons. A smaller combination of reinforcement was selected to decrease associated material and labor costs based on the overall weight of the reinforcement design.

The flat plate also allowed the existing architectural program to remain throughout the entire building. Column locations were undisturbed throughout the floor plan ensuring the layout of residential units remained consistent. The adjusted structural depth of the flat plate increased the overall building height to 179’ from the average site elevation, meeting requirements of the zoning height limitation of 180’.

The adjusted thickness of the flat plate design added a significant amount of dead load to the structure. As a result, the columns and lateral system of the structure must be designed with consideration of the imposed loads. The following section further develops the column design for the flat plate system.

Column Design

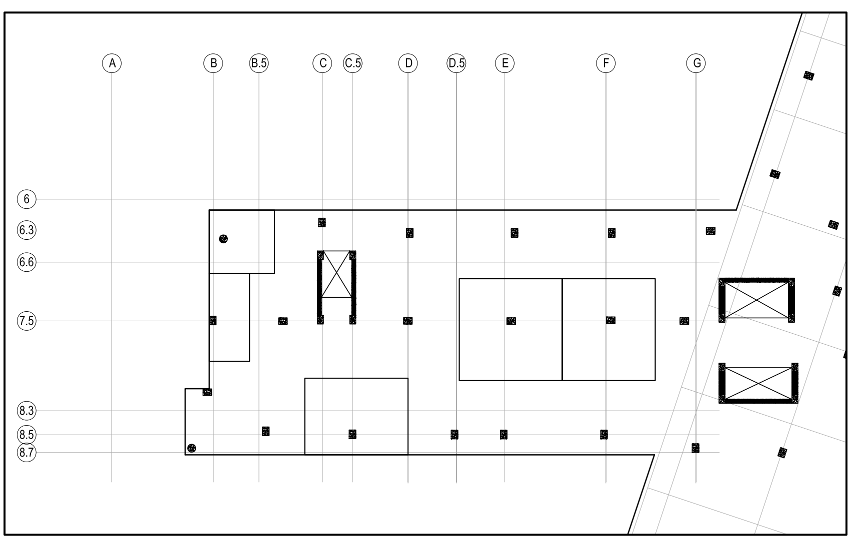
Design Criteria

The design objectives for the supporting columns were similar to the design of the flat plate system.   Minimum column dimensions would need to be considered to ensure the architectural integrity of the residential spaces. The design of a uniform column size would promote a faster schedule by construction of repeatable floors. The columns were subjected to the gravity loads listed below.

Dead :		**Live:**	
Roof	50 psf	Roof	30 psf
Mechanical	150 psf	Mechanical	150 psf
Residential	27 psf	Residential	40 psf
Façade	32 psf	Public Space	100 psf

Column Design

I started the by selecting a series of columns that provide a good representation of critical loading at different locations in the floor plan. The columns had large tributary areas positioned at interior, exterior, and corner locations. The same columns were analyzed for punching shear failure in the frame analysis. The partial floor plan below depicts the selected columns and their respective tributary areas.



Critical Columns & Tributary Areas

Axial Load

Axial loads were developed on each column by performing a load take-down for levels 2 –16 of the tower structure. Each level was designated dead and live loads depending on the use of the space within each designated tributary area. The loading from the 2nd- 15th level is entirely residential and public space and the 16th level is mechanical. The interior columns were located along corridors with a portion of the tributary area residing in designated public space loading. I decided to design the columns conservatively by assuming the public space live load of 100psf for the residential levels.

I created a column load take-down design table to accumulate the distributed dead and live loads throughout the levels. The total tributary self weight of the flat plate and columns located above a particular level were added into the accumulated dead load calculation. Live load reduction was also considered for the accumulated tributary areas with reduction factors applied to each column based on the specified location. The accumulated factored axial forces are listed in the design table below for an interior column located at column line E / 7.5.

Column E/7.5 Interior — K_{LL} = 4

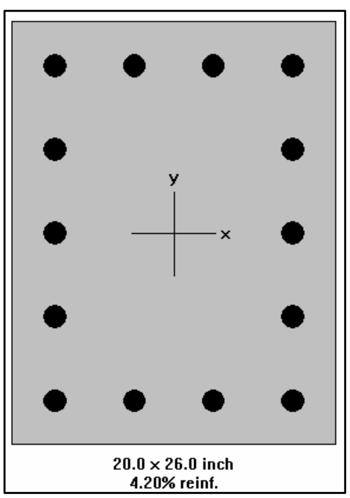
Level/Column:	Live (PSF)	Dead (PSF)	Area (ft2)	A_T (ft2)	Reduction	Live Load (kip)	Reducd LL (kip)	Dead Load (kip)	Factored Load (kip)
Roof	30	50	700	700	1.000	21.0	21.0	139.9	201.5
16	150	150	700	1400	0.450	126.0	56.8	347.5	507.8
15	100	27	700	2100	0.414	196.0	81.1	468.6	692.0
14	100	27	700	2800	0.400	266.0	106.4	588.9	877.0
13	100	27	700	3500	0.400	336.0	134.4	709.3	1066.2
12	100	27	700	4200	0.400	406.0	162.4	829.6	1255.4
11	100	27	700	4900	0.400	476.0	190.4	950.0	1444.6
10	100	27	700	5600	0.400	546.0	218.4	1070.3	1633.8
9	100	27	700	6300	0.400	616.0	246.4	1190.6	1823.0
8	100	27	700	7000	0.400	686.0	274.4	1311.0	2012.2
7	100	27	700	7700	0.400	756.0	302.4	1431.3	2201.4
6	100	27	700	8400	0.400	826.0	330.4	1551.7	2390.6
5	100	27	700	9100	0.400	896.0	358.4	1672.0	2579.8
4	100	27	700	9800	0.400	966.0	386.4	1792.3	2769.1
3	100	27	700	10500	0.400	1036.0	414.4	1912.7	2958.3
2	100	27	700	11200	0.400	1106.0	442.4	2033.0	3147.5

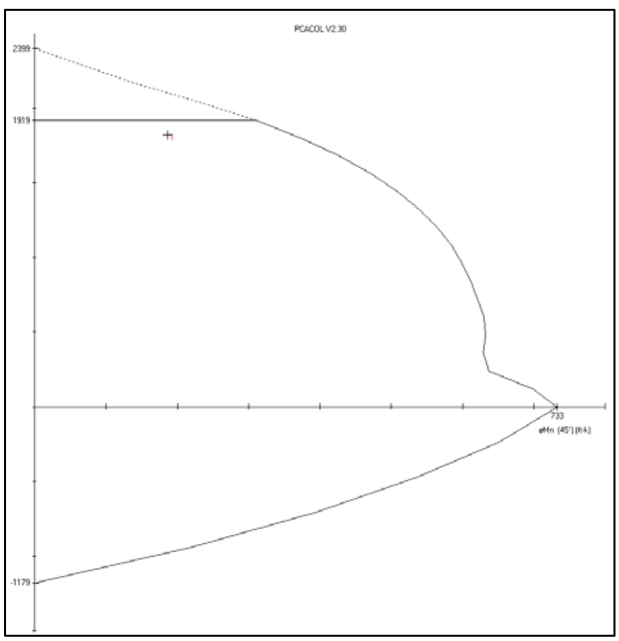
Column load take-down table – Interior column

Biaxial Bending

The columns were designed under biaxial loading conditions from the unbalanced moments found in the ADOSS frame analyses. The maximum unbalanced moments were used to determine the required column reinforcement. I used PCA Column to design sizes and reinforcement with the specified factored axial loads and bending moments obtained from the previous analyses. The effects of slenderness were neglected for the design in accordance with ACI 318-05 (10.13.2). I used a range of concrete strengths to establish the minimum column size that will be constructed uniformly over the entire flat plate. A 20"x 26" column was found as a sufficient minimum uniform design size with 14 #11 bars. The column sizes on the 1st and 2nd levels were increased to 22"x 28" to accommodate the accumulated axial forces. Concrete strengths of the columns are listed below by level along with the typical reinforcement layout and accompanying interaction diagram.

Level	Concrete Strength
1-5	8000 psi
6-7	6000 psi
8-16	5000 psi





Summary

The columns supporting the flat plate system were designed with the intention of limiting overall design size to ensure minimal architectural impacts on residential spaces. A uniform column size was desired throughout the floor plan in order to promote a faster building schedule by repeatable floor construction. The location of the columns would remain unchanged without the interruption of open spaces.

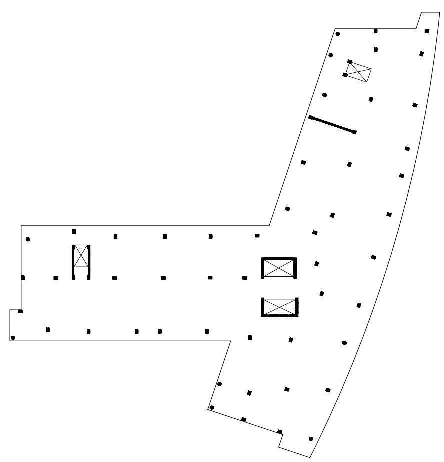
The columns were designed with a uniform size of 20”x 26”. The column design will not significantly affect residential spaces as most of the columns are integrated into protruding corners and wall spaces within the units. Material for column construction adjusted as a result of the redesign including concrete, alterations in strength, and reinforcing steel. The increase in the column design, as well as the flat plate thickness, add significant dead load to the structure resulting in alterations to the imposed lateral loads. The next section investigates the lateral implications of the gravity system redesign and will develop an analysis of the lateral system.

Lateral System Design

Design Criteria

The lateral system redesign will consider full lateral resistance by shear wall structures alleviating lateral resistance by the flat plate system. I originally assumed a long span design without post-tensioning would limit the lateral capability of the slab frame. The flat plate would act as a cracked section limiting structural stiffness opposed to the post-tensioned system designated Class U with un-cracked gross section properties.

The proposed design would adhere to the architectural program throughout a typical floor plan. Shear wall locations would remain at the central elevator shafts, the West stair tower, and an interior wall within a residential unit located in the skewed tower. The central shear walls will be extended from the existing design at the 4th level through the tower structure to resist the lateral loads from the increase in building height and weight. The locations of the shear walls are depicted below.



Typical Floor Plan

The lateral forces applied to the building by wind and seismic loading conditions will be calculated in accordance with provisions of the IBC 2003 and ASCE 7-02 design codes. A maximum displacement of H/600 was set as the design limit to control cracking in the brick veneer. The story drift was checked against the maximum limit $.02h_{sx}$ for Seismic Use Group I in accordance with ASCE7-02 (9.5.2.8). The concrete strength of the shear walls will be 4000 psi throughout the entire structure with the specified existing dimensions.

Seismic Loads

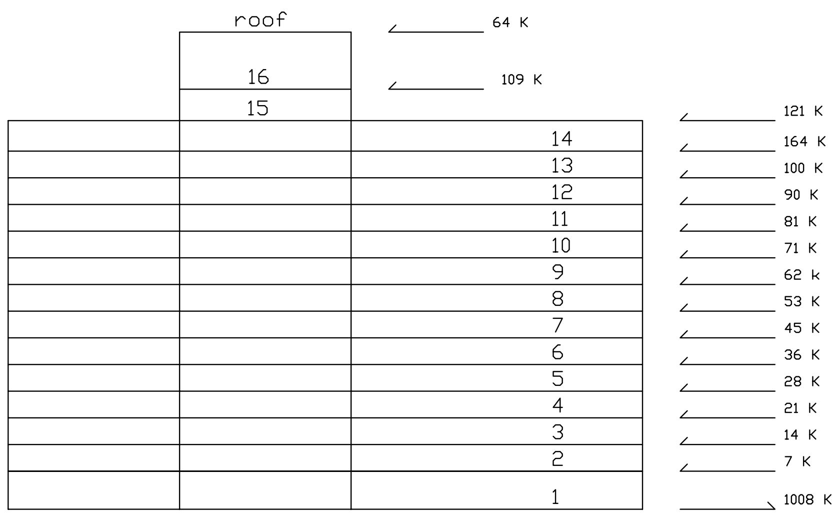
Seismic loads were calculated to account for the increased building height and weight as a result of the gravity system designs. The equivalent lateral force procedure was used to calculate the seismic forces on the building. Dead load for each level was calculated to include the added structural self-weight and super-imposed dead loads over the net floor area of each level. The resulting seismic story forces were calculated under the following design parameters in accordance with provisions of ASCE 7-02 Section 9. Full design calculations are found in Appendix A.

Design Parameters

Location:	Arlington, Virginia	
Number of Stories:	N = 16	
Inner Story Height:	hs = varies - 9'-7" typ.	
Building Height:	hn = 171	
Seismic Use Group:	1	Table: 9.1.3
Occupancy Importance:	I = 1.0	Table: 9.1.4
Site Classification:	C	9.4.1.2
Accelerations:		
0.2 s	Ss = 0.179	Figure: 9.4.1.1(a)
1.0 s	S1 = 0.063	Figure: 9.4.1.1(b)
Site Class Factor:	Fa = 1.2	Table: 9.4.1.2(a)
	Fv = 1.7	Table: 9.4.1.2(b)
Adjusted Accelerations:	Sms = 0.2148	9.4.1.2.4-1
(max.)	Sm1 = 0.1071	9.4.1.2.4-2
Design Spectral Response	S_{DS} = 0.143	9.4.1.2.5-1
Accelerations:	S_{D1} = 0.0714	9.4.1.2.5-2
Seismic Design Category:	B	9.4.2.1(a/b)
Response Modification:	R = 5	Table: 9.5.2.2
Deflection Modification:	Cd = 4.5	

Vertical Distribution of Seismic Forces

	Load	Shear	Moment
N-S	Fx	Vx	Mx
Level, x	(kips)	(kips)	(ft-kips)
Roof	64	0	10,736
16	109	64	16,502
15	121	174	16,867
14	104	295	13,391
13	100	399	11,888
12	90	499	9,870
11	81	589	8,057
10	71	670	6,444
9	62	741	5,025
8	53	803	3,798
7	45	857	2,757
6	36	902	1,894
5	28	938	1,206
4	21	967	683
3	14	988	317
2	7	1001	97
1	-	1008	-
			Σ = 109534



Seismic Force Distribution

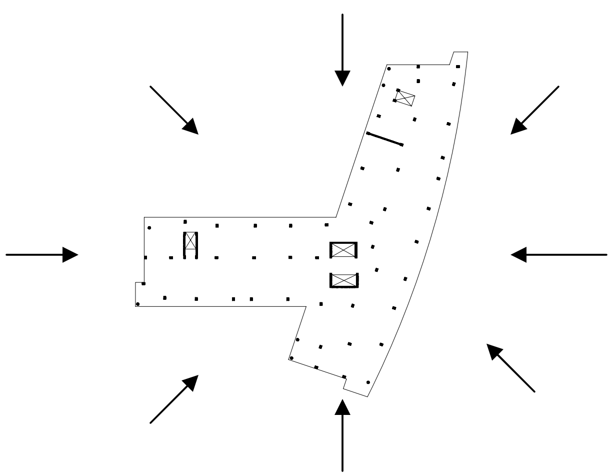
Wind Loads

Wind loads were calculated to account for the increase in building height. Design wind pressures were calculated by the Analytical Procedure in accordance with provisions of ASCE 7-02 Section 6. Full design parameters and calculations are found in the Appendix A.

Wind Loading (N-S)

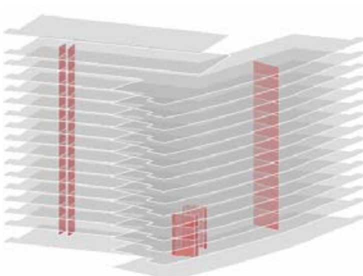
Level	Story Height (ft.)	Elevation (ft.)	Tributary Height (ft.)	Tributary Width (ft.)	Tributary Area (ft^2)	Wind Load (psf)	Wind Load (k)	Shear (k)	Moment (ft - k)
Roof	4	166.74	12.00	183	2196	**17.6**	**39**	39	-
16	16	150.74	13.63	183	2493	**17.3**	**43**	82	617.3
15	11.25	139.49	11.08	183	2028	**17.1**	**35**	117	1537.8
14	10.91	128.58	10.25	224	2295	**16.9**	**39**	155	2809.6
13	9.58	119.00	9.58	224	2147	**16.7**	**36**	191	4297.8
12	9.58	109.41	9.58	224	2147	**16.4**	**35**	226	6129.0
11	9.58	99.83	9.58	224	2147	**16.2**	**35**	261	8297.7
10	9.58	90.25	9.58	224	2147	**15.9**	**34**	295	10799.8
9	9.58	80.66	9.58	224	2147	**15.6**	**34**	329	13629.1
8	9.58	71.08	9.58	224	2147	**15.3**	**33**	362	16779.9
7	9.58	61.50	9.58	224	2147	**14.9**	**32**	394	20245.1
6	9.58	51.91	9.58	224	2147	**14.6**	**31**	425	24017.5
5	9.58	42.33	9.58	224	2147	**14.1**	**30**	455	28089.3
4	9.58	32.75	9.58	224	2147	**13.6**	**29**	484	32451.9
3	9.58	23.16	9.58	224	2147	**12.9**	**28**	512	37094.2
2	9.58	13.58	11.58	224	2594	**12.2**	**32**	544	42002.6
1	13.58	0.00	6.79	224	1521	**-**	**-**	-	49388.4

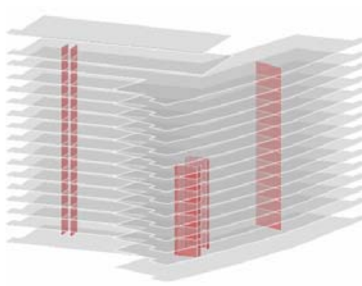
Through the initial lateral force calculations I found that seismic forces control the lateral design. To ensure this assumption, both loading conditions were applied in accordance with ASCE 7-02 to a model of the shear wall system created in ETABs. Full wind loads were applied in all principle and intermediate directions on the building represented in the figure below. This was to account for any design oversights of using a rectilinear simplification of the projected tributary widths in the Analytical Procedure. Equivalent design forces and moments for wind load cases 1-4 were also calculated and applied to the model for a complete wind load analysis.

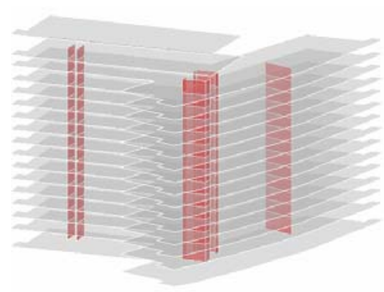


Shear Wall Analysis

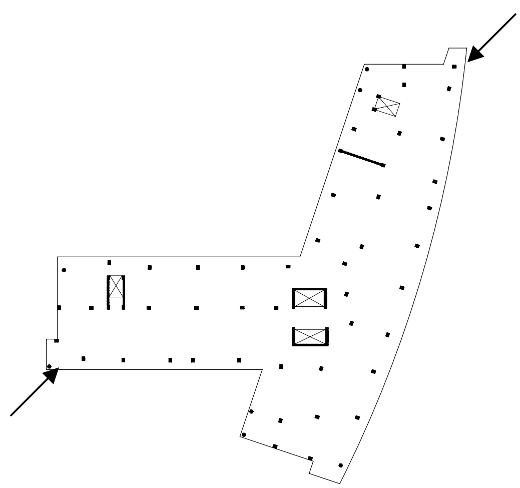
The shear walls were modeled in ETABs and subjected to the applied lateral load cases developed in the previous section. Seismic loads controlled the design and were evaluated in each direction at an accidental eccentricity of 5%. I reduced out of plane stiffness to simulate the shear walls as primarily in-plane resisting elements. The flat plate was modeled as a rigid diaphragm without vertical load transfer to appropriately apply the seismic forces calculated with the equivalent lateral force procedure. The model was then analyzed in iterations by extruding the central shear walls on each run and checking the model against the deflection and story drift design limitations.





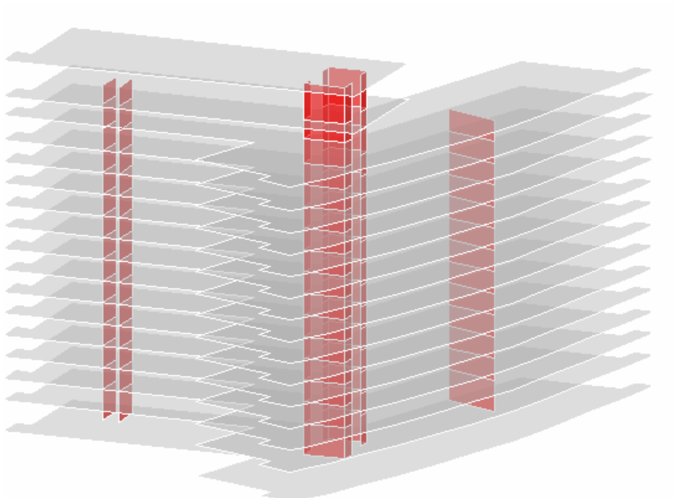


The shear wall systems were first checked for overall displacement of the building at the center of mass of each diaphragm. The irregularity of the building shape created maximum displacements at the extents of the tower wings. These displacements and story drifts would control the design of the shear walls. The control points are depicted in the adjacent figure.

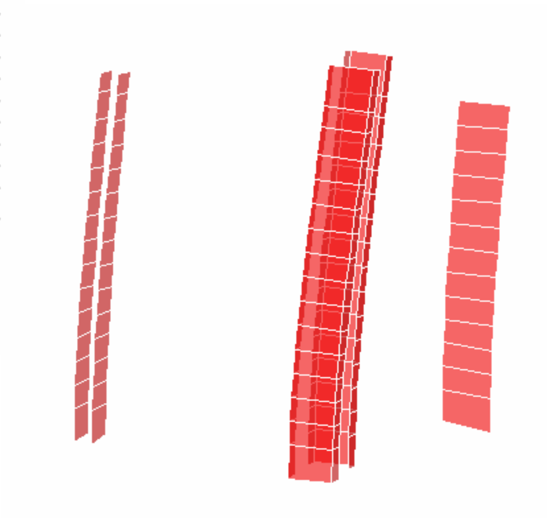


Maximum Displacement Points

The final applicable shear wall design with central shear walls extended to the roof level failed to meet the displacement limitation. The displacement at the diaphragm center of mass reached the design limit at level 6. The total center or mass displacement at the roof level was a total of 6", well over the design limit of 3.33". Below are the maximum shear wall design and deflected shape from the seismic loading condition resolved from the forces calculated with the equivalent lateral force procedure.



Shear Walls at Roof Level

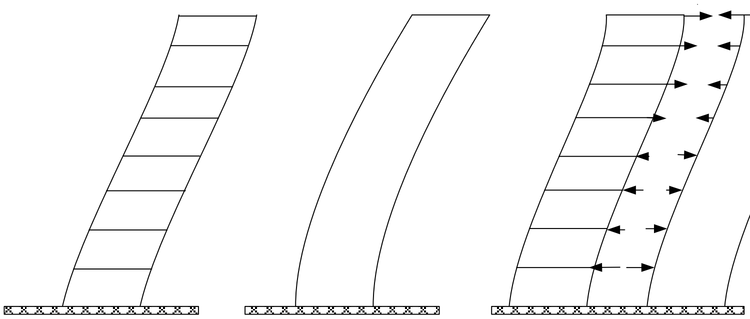


Deflected Shape

The alternative for designing an effective shear wall system was adjusting the wall sizes to increase their stiffness until full lateral resistance is achieved. The new design thicknesses would need to cut the current displacements in half and would jeopardize the architectural integrity of the spaces around the walls. Another alternative was to keep the shear wall design and incorporate the designed flat plate system into the lateral design. This would limit added material costs for larger shear walls and the slab frames would contribute effectively in limiting the displacements at the building corners.

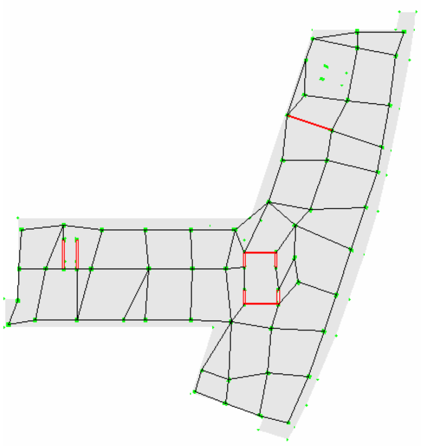
Slab Frame / Shear Wall Analysis

The design of the integrated system would adhere to the criteria and limitations used for the shear wall design. The design advantage of an integrated lateral system composed of shear walls and moment frames lies within the interaction between each system in deflection. The slab frame deflects in shear and tall shear walls deflect predominantly in flexure. A combination system produces opposing internal forces which increase overall stiffness within the system. The resulting deflection of the integrated system is less than individual deflections of each system acting alone. The diagram below depicts the interaction of a moment frame and shear wall system.

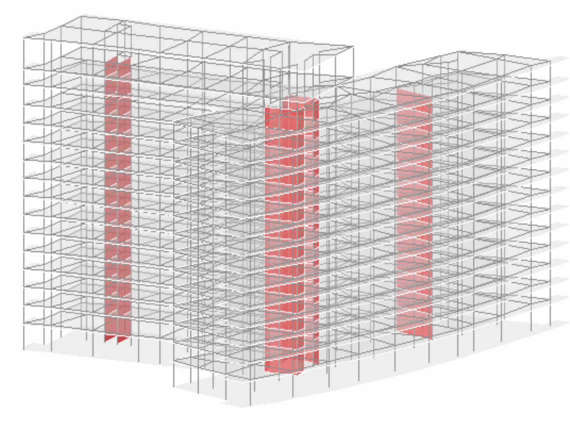


Moment Frame *Shear Wall* *Integrated System*

The slab frame system was simulated in the shear wall model as beams with the same structural depth of the flat plate spanned at panel support lines shown to in the figure to the right. The beams were sized to the average effective column width under the assumption that the concrete within this region would effectively contribute to the resistance of shear transfer by lateral forces. The slab beams and columns were assigned design stiffness modifiers for cracking in accordance with frame analysis provisions of ACI 318-05-05 (10.11).



Simulated Slab Frame System



Shear Wall / Slab Frame System

I followed the same design procedure with the ETABs model that I used for the shear wall design. The seismic load cases still controlled the displacements and drifts of the integrated system. The influence of the slab frame distributed lateral forces enough to reduce the shear wall design to the 14th level. Displacement was met at the H/600 limit at the critical points and story drifts were well under the allowable limit. .

Summary

The lateral analysis of the proposed shear wall system needed to be investigated for the induced lateral loads from the flat plate and column redesigns. Added structural weight to the overall building resulted in seismic loads controlling the lateral design. The structural model of the system incorporated wind load design cases to check the assumption of seismic control.

The proposed shear wall design was unable to resist the seismic loads alone. Central walls were extended through the building to the roof level with a displacement of 6", well over the displacement limit of H/600. The flat plate was integrated into the lateral system design to increase the overall stiffness of the structure. Utilizing the flat plate decreased the displacement at critical points on the building corners. The new design of the lateral system would have the central shear walls extended to the 14th level with the others to their respective limits at the 14th and 15th levels. Displacement was reduced to 3" at the roof level meeting the H/600 limit with a story height of 167' measured from the 1st level.