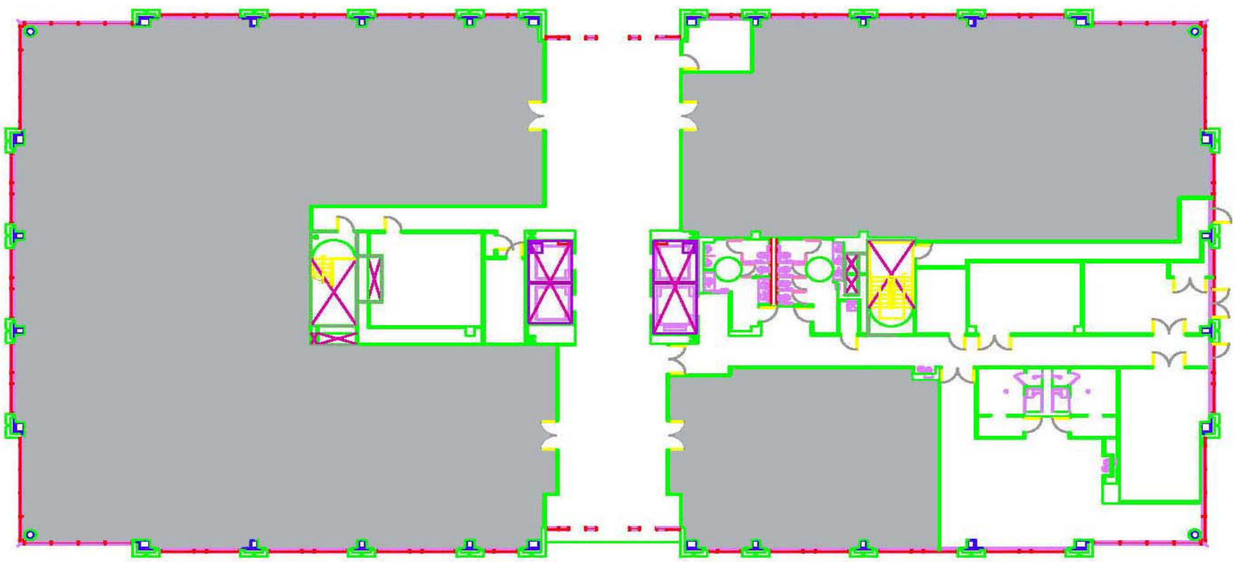
Structural Depth

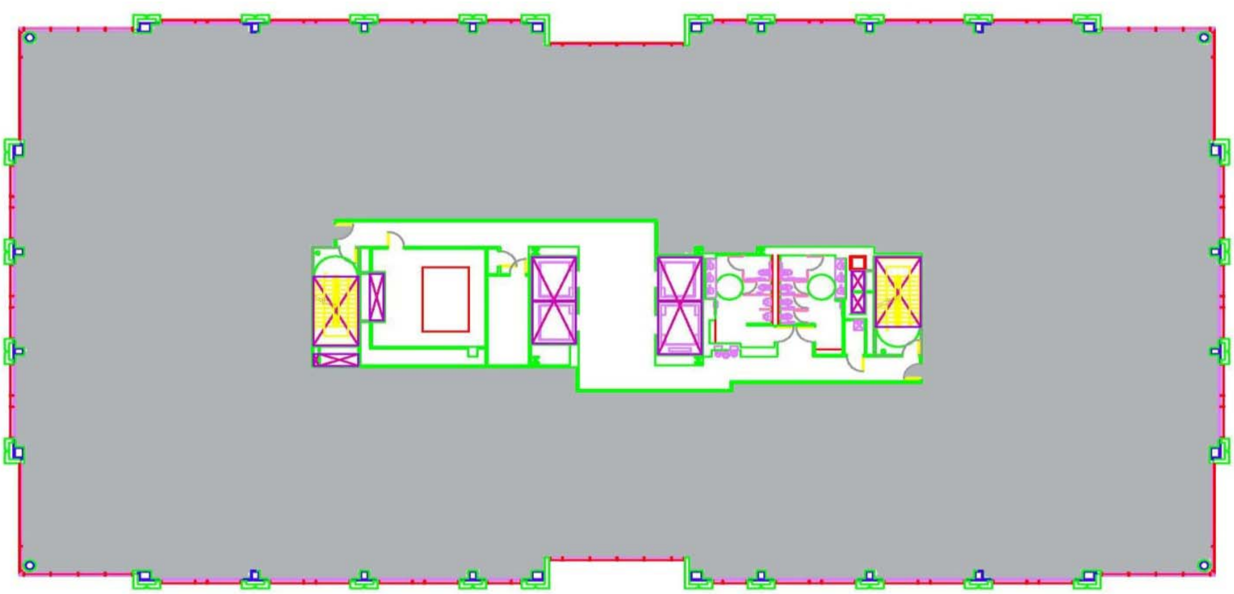
Existing Structural Systems

Building Floor Plans and Framing

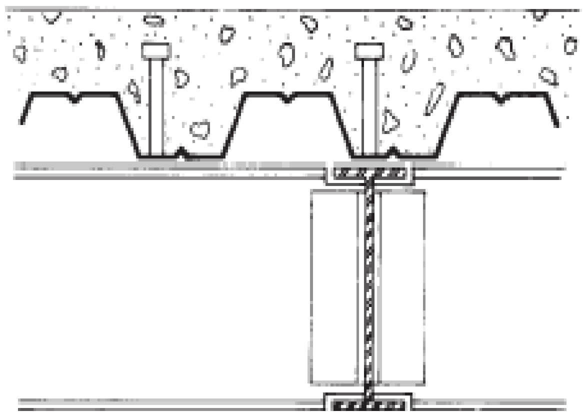
The Washingtonian Center utilizes a core and envelope design to allow the individual tenants to finish their spaces as they see fit. To allow for minimum intrusions by the structural system, the beams over the tenant spaces span 45'. The figures below show the architectural layouts of the core and essential spaces. It should be noted that the leasable spaces are shaded in grey.



First Floor Architectural Plan

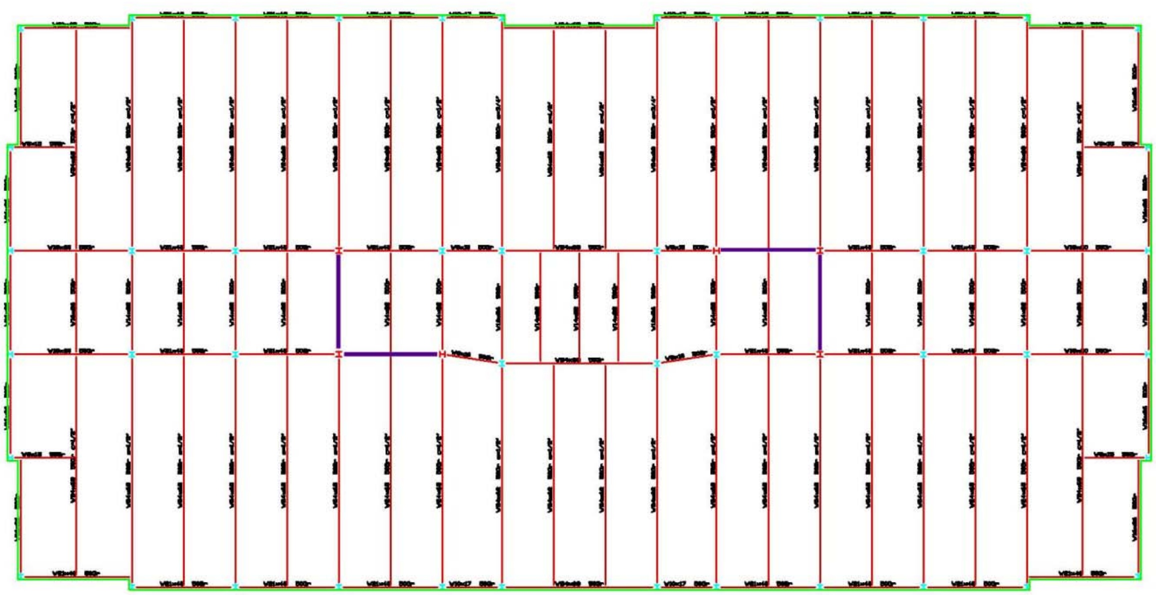


Typical Floor Architectural Plan



The composite floor system used is 3" 20 gage composite steel floor deck with 3.25" inch topping of light weight concrete with a compressive strength of 4000 psi. The floor is reinforced with 6" x 6"-W2.1 x W2.1 welded wire fabric placed 1" below the top of the concrete. This system is utilized for the 2nd – 8th floors. The ground floor is a slab on grade that is 5" thick and reinforced with 6" x 6"-W2.1 x W2.1 welded wire fabric. The slab on grade is poured on a 6" granular base.

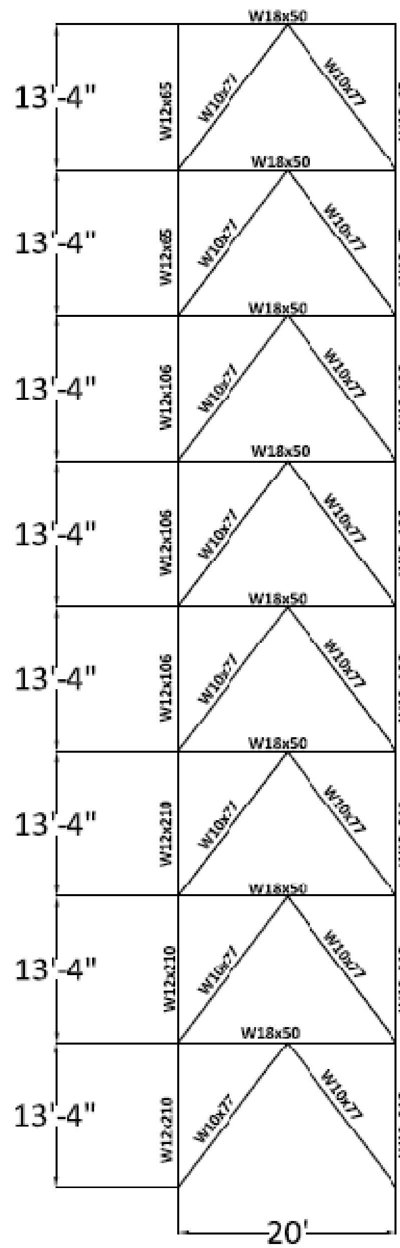
The steel deck is supported on W21x44 beams spaced every 10' and spanning a distance of 45' on the exterior bays. The interior bays are supported by W14x22 spaced every 10' and spanning a distance of 20'. The girders supporting these beams are typically W14x22 spanning 20'.



Typical Floor Architectural Plan

Lateral System

The lateral force resisting system implemented in the Washingtonian Center is a series of concentrically braced chevron frames around the elevator cores near the center of the building (The frames are shown in the figure above as purple members). The frames span in both directions for a distance of 20'. All four of the lateral frames are identical. The columns in the frames are spliced at the fourth and seventh levels and are W12x210 at the bottom, W12x106 at the middle levels and 12x65 at the upper floors. The beams in the frame are W18x50 and the chevron braces are W10x77.



Brace Frame Elevation

Columns

The columns in the building are spliced at the fourth floor and the seventh. All gravity columns in the building are either a W10 or W12 with sizes below the first splice point ranging from W10x49 to W12x96. Above the first splice location (floors 4,5 and 6) the columns range in size from W10x39 to W12x65. On the upper levels (floors 7, 8, the roof and mechanical penthouse) the columns range in size from W10x33 to W12x53. The un-braced length of the columns is the floor to floor height of 13'-4".

Building Codes use in the Design

The Washingtonian Center was designed under the provisions of the 2003 International Building Code. The design loads were determined from the referenced ASCE 7-02 edition. The steel gravity and lateral framing were designed using AISC, LRFD Third Edition. The concrete footings were designed using the provisions of ACI 318-02.

Loads

The loads presented here are based on values and procedures from the International building code 2003 and ASCE 7-05. It should be noted that while these codes give the minimum required loads for design, the designers of the Washingtonian Center used larger loads in some cases at their professional discretion. The live and dead loads presented below are the loads used by the design professionals. The lateral loads were calculated by with the aid of spreadsheets based on the requirements of the above mentioned codes.

Dead Loads:

Metal Deck and Concrete Topping for Strength	65psf
Floor mass for Seismic Design	85psf
Partition Allowance	25psf
Sprinkler Allowance	5psf

Live Loads:

Stairs and Exits	100psf
Elevator Machine Room	100psf
Offices	100psf
Public Spaces	100psf
Mechanical/Electrical Rooms	150psf
Roof	20psf

Wind Loads:

Main Wind-Force Resisting System Wall Pressures, ASCE 7-05

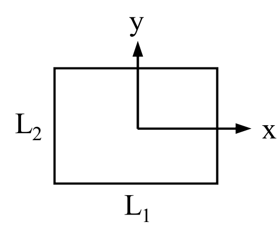
For Buildings of All Heights

General parameters:

Classification Category (I, II, III, IV):	**II**	
Basic Wind Speed, V:	**90**	mph
Hurricane Region (Y or N)?	**N**	
Importance factor, I:	1.00	
Mean recurrence interval:	**50**	year
MRI factor:	1.00	
Adjusted Wind Speed, V:	90	mph
Exposure Category (A, B, C, D):	**B**	
α:	7.00	
z_g:	1200	
Topographic factor, K_{zt}:	**1.00**	
Wind directionality factor, K_d:	**0.85**	
Gust Factor, G (x-dir wind):	**0.86**	
Gust Factor, G (y-dir wind):	**0.83**	
Internal pressure coefficient, $+GC_{pi}$:	**0.18**	
Internal pressure coefficient, $-GC_{pi}$:	**-0.18**	
Windward pressure coefficient, C_p:	0.80	
Side pressure coefficient, C_p:	-0.70	

Building properties:

Mean Roof Height, h:	**120**	ft
Typical length in x-direction, L_1:	**220**	ft
Typical length in y-direction, L_2:	**110**	ft



Recommended Gust Effect Factors:

Damping ratio, β:	**0.015**
Gust Factor, G (x-dir wind):	0.855
Gust Factor, G (y-dir wind):	0.827

Calculated values:

Velocity pressure coeff. at h, K_h:	1.04	
Velocity pressure at h, q_h:	18.3	psf

Base moments and shears:

Distance from ground level to bottom of pilecap:	**2**	ft
Base shear due to x-direction wind:	181	k
Moment at ground due to x-direction wind:	11,018	k-ft
Moment at bottom of pilecap due to x-direction wind:	11,379	k-ft
Base shear due to y-direction wind:	415	k
Moment at ground due to y-direction wind:	24,324	k-ft
Moment at bottom of pilecap due to y-direction wind:	25,154	k-ft

Notes:

1. Positive and negative pressures signify pressures acting toward and away from the surfaces, respectively.
2. Refer to Figure 6-6, ASCE 7-05 for wind pressure diagrams.

Main Wind-Force Resisting System Wall Pressures, ASCE 7-05

Story Elevations and Widths:

Story	z (ft)	L_1 (ft)	L_2 (ft)	z (ft)	K_z	q_z (psf)	L_1 (ft)	L_2 (ft)
Pent.	**125**	**60**	**110**	**125**	1.05	18.6	60	25
Roof	**108**	**220**	**110**	**108**	1.01	17.8	220	110
8	**93**	**220**	**110**	**93**	0.97	17.1	220	110
7	**80**	**220**	**110**	**80**	0.93	16.3	220	110
6	**67**	**220**	**110**	**67**	0.88	15.5	220	110
5	**53**	**220**	**110**	**53**	0.82	14.5	220	110
4	**40**	**220**	**110**	**40**	0.76	13.4	220	110
3	**27**	**220**	**110**	**27**	0.68	12.0	220	110
2	**13**	**220**	**110**	**13**	0.57	10.1	220	110
1	**0**	**220**	**110**	**0**	0.57	10.1	220	110

Wind Pressures and Story Forces:

X-DIRECTION WIND

z (ft)	L/B	Leeward C_p	External wall pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	w/ pos. internal pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	w/ neg. internal pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	Total pressure (psf)	Story Force (k)
125	2.40	-0.280	12.7	-4.4	-11.0	9.4	-7.7	-14.3	16.0	-1.1	-7.7	**17.1**	**6**
108	2.00	-0.300	12.2	-4.7	-11.0	8.9	-8.0	-14.3	15.5	-1.4	-7.7	**16.9**	**18**
93	2.00	-0.300	11.7	-4.7	-11.0	8.4	-8.0	-14.3	15.0	-1.4	-7.7	**16.4**	**25**
80	2.00	-0.300	11.2	-4.7	-11.0	7.9	-8.0	-14.3	14.5	-1.4	-7.7	**15.9**	**23**
67	2.00	-0.300	10.6	-4.7	-11.0	7.3	-8.0	-14.3	13.9	-1.4	-7.7	**15.3**	**23**
53	2.00	-0.300	9.9	-4.7	-11.0	6.6	-8.0	-14.3	13.2	-1.4	-7.7	**14.6**	**22**
40	2.00	-0.300	9.2	-4.7	-11.0	5.9	-8.0	-14.3	12.5	-1.4	-7.7	**13.9**	**20**
27	2.00	-0.300	8.2	-4.7	-11.0	4.9	-8.0	-14.3	11.5	-1.4	-7.7	**12.9**	**19**
13	2.00	-0.300	6.9	-4.7	-11.0	3.6	-8.0	-14.3	10.2	-1.4	-7.7	**11.6**	**17**
0	2.00	-0.300	6.9	-4.7	-11.0	3.6	-8.0	-14.3	10.2	-1.4	-7.7	**11.6**	**9**

Main Wind-Force Resisting System Wall Pressures, ASCE 7-05

For Buildings of All Heights

Wind Pressures and Story Forces:

Y-DIRECTION WIND

z (ft)	L/B	Leeward C_p	External wall pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	w/ pos. internal pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	w/ neg. internal pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	Total pressure (psf)	Story Force (k)
125	0.42	-0.500	12.3	-7.6	-10.6	9.0	-10.9	-13.9	15.6	-4.3	-7.3	**19.9**	**2**
108	0.50	-0.500	11.8	-7.6	-10.6	8.5	-10.9	-13.9	15.1	-4.3	-7.3	**19.4**	**42**
93	0.50	-0.500	11.3	-7.6	-10.6	8.0	-10.9	-13.9	14.6	-4.3	-7.3	**18.9**	**58**
80	0.50	-0.500	10.8	-7.6	-10.6	7.5	-10.9	-13.9	14.1	-4.3	-7.3	**18.4**	**53**
67	0.50	-0.500	10.3	-7.6	-10.6	7.0	-10.9	-13.9	13.6	-4.3	-7.3	**17.9**	**53**
53	0.50	-0.500	9.6	-7.6	-10.6	6.3	-10.9	-13.9	12.9	-4.3	-7.3	**17.2**	**51**
40	0.50	-0.500	8.9	-7.6	-10.6	5.6	-10.9	-13.9	12.2	-4.3	-7.3	**16.5**	**47**
27	0.50	-0.500	7.9	-7.6	-10.6	4.6	-10.9	-13.9	11.2	-4.3	-7.3	**15.5**	**46**
13	0.50	-0.500	6.7	-7.6	-10.6	3.4	-10.9	-13.9	10.0	-4.3	-7.3	**14.3**	**42**
0	0.50	-0.500	6.7	-7.6	-10.6	3.4	-10.9	-13.9	10.0	-4.3	-7.3	**14.3**	**21**

Wind Pressures and Story Forces: Summary

z (ft)	K_z	q_z (psf)	L_1 (ft)	L_2 (ft)	X-DIRECTION WIND Leeward C_p	Total pressure (psf)	Story Force (k)	Y-DIRECTION WIND Leeward C_p	Total pressure (psf)	Story Force (k)
125	1.05	18.6	60	25	-0.300	**17.1**	**6**	-0.500	**19.9**	**2**
108	1.01	17.8	220	110	-0.300	**16.9**	**18**	-0.500	**19.4**	**42**
93	0.97	17.1	220	110	-0.300	**16.4**	**25**	-0.500	**18.9**	**58**
80	0.93	16.3	220	110	-0.300	**15.9**	**23**	-0.500	**18.4**	**53**
67	0.88	15.5	220	110	-0.300	**15.3**	**23**	-0.500	**17.9**	**53**
53	0.82	14.5	220	110	-0.300	**14.6**	**22**	-0.500	**17.2**	**51**
40	0.76	13.4	220	110	-0.300	**13.9**	**20**	-0.500	**16.5**	**47**
27	0.68	12.0	220	110	-0.300	**12.9**	**19**	-0.500	**15.5**	**46**
13	0.57	10.1	220	110	-0.300	**11.6**	**17**	-0.500	**14.3**	**42**
0	0.57	10.1	220	110	-0.300	**11.6**	**9**	-0.500	**14.3**	**21**

Gust Effect Factor, Gf (ASCE 7-05, Section 6.5.8.2)

General parameters:

V =	90	mph
Exp. Cat. =	B	
h =	120	ft
L_1 =	220	ft
L_2 =	110	ft
T =	1.23	
n_1 = 1/T =	0.81	Hz
β =	0.015	

Results:

Flexible building:

G_f, x-dir =	**0.855**
G_f, y-dir =	**0.827**

Building Parameters

mean α =	0.25	
mean b =	0.45	
c =	0.30	
l =	320	ft
mean ε =	0.33	
z_{min} =	30	ft
mean z =	72	ft
I_z =	0.263	
L_z =	415	
g_Q =	3.4	
g_v =	3.4	

Wind blowing in x-direction:

L = L1 =	220	ft
B = L2 =	110	ft
Q =	0.835	

Wind blowing in y-direction:

L = L2 =	110	ft
B = L1 =	220	ft
Q =	0.802	

Flexible Buildings (Buildings with fundamental frequency less than 1.0 Hz):

g_R =	4.14
mean V_z =	72.2
N_1 =	4.7
R_n =	0.053

Wind blowing in x-direction:

L = L1 =	220	ft
B = L2 =	110	ft
η_h =	6.20	
R_h =	0.148	
η_B =	5.68	
R_B =	0.160	
η_L =	38.06	
R_L =	0.026	
R =	0.214	
G_f =	**0.855**	

Wind blowing in y-direction:

L = L2 =	110	ft
B = L1 =	220	ft
η_h =	6.20	
R_h =	0.148	
η_B =	11.37	
R_B =	0.084	
η_L =	19.03	
R_L =	0.051	
R =	0.156	
G_f =	**0.827**	

Seismic Loads:

Equivalent Lateral Force Procedure, ASCE 7-05			
Input for General Analysis			
W=	22700 kips		
S_s =	0.157	(http://earthquake.usgs.gov/research/hazmaps/design/- for a short period)	
S_1 =	0.051	(http://earthquake.usgs.gov/research/hazmaps/design/- for a period of 1 sec.)	
Soil =	C	(Geotech Report)	
F_a =	1.2	(Table 11.4-1)	
F_v =	1.7	(Table 11.4-2)	
S_{MS} =	0.188	(Eq. 11.4-1)	
S_{M1} =	0.087	(Eq. 11.4-2)	
S_{DS} =	0.126	(Eq. 11.4-3)	
S_{D1} =	0.058	(Eq. 11.4-4)	
R =	3	(Table 12.2-1)	
I =	1	(Table 11.5-1)	
C_T =	0.02	(Table 12.8-2)	
h_n =	120		
x=	0.75	(Table 12.8-2)	
Cu=	1.7	(Table 12.8-1)	
TL=	8	(Section 11.4.4)	
Output			
Ta =	0.725	(Approximate Period)	
T=	1.233	(Period)	
C_s =	0.042	(Eq. 12.8-2)	
C_s =	0.016	(Eq. 12.8-3)	**<controls**
C_s =	0.101	(Eq. 12.8-4)	
Use C_s =	0.016		
Base Shear		***Over-Turning Moment***	
V=	354.8 kips	**M=** 27,883 k-ft	

Equivalent Lateral Force Procedure, ASCE 7-05

Vertical Distribution of Base Shear

k= 1.37 (Section 12.8.3)

Level	h_x (ft)	W_x (kips)	$W_x h_x^k$	C_{vx}	F_x (kips)
Pent.	120	400	282188	0.04	15.51
Roof	106	2000	1190417	0.18	65.41
8th	92.75	2900	1437534	0.22	78.99
7th	79.5	2900	1163861	0.18	63.95
6th	66.25	2900	906615	0.14	49.82
5th	53	2900	667815	0.10	36.69
4th	39.75	2900	450288	0.07	24.74
3rd	26.5	2900	258372	0.04	14.20
2nd	13.25	2900	99962	0.02	5.49
Sum		22700	6457052	1	354.8

Load Cases:

Serviceability Combinations

1. D + F
2. D + H + F + L + T
3. D + H + F + (L_r or S or R)
4. D + H + F + 0.75(L + T) + 0.75(L_r or S or R)
5. D+ H + F + (W or 0.7E)
6. D + H + F + 0.75(W or 0.7E) + 0.75L + 0.75(L_r or S or R)
7. 0.6D + W + H
8. 0.6D + 0.7E + H

Strength Combinations

1. 1.4 (D+ F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4. 1.2D + 1.6W + L +0.5(L_r or S or R)
5. 1.2D + 1.0E + L +0.2S
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

Analysis of the Existing Structure

An Etabs serviceability model of the original structure was constructed to evaluate the ability of the lateral frame to limit building sway to code imposed limits. The allowable drifts for the wind were calculated based on the common practice of L/400. For the seismic drifts, the allowable drifts were based on the limits imposed by ASEC 7-05. The limitations were given as two percent of the story height. The seismic drifts were multiplied by an amplification factor of three required by code for steel structures not specifically detailed for seismic resistance. The importance factor for the building is one so there was no need to divide the amplification factor by it. The numbers presented below represent the worst case deflections from the serviceability load combinations. These combinations were applied with the wind loads applied in all four load cases required by ASCE 7-05, with the controlling case being the torsional loads applied in the Y direction. The seismic loads were also applied with the service load combinations, with the controlling case being in the Y direction. This analysis shows that the current lateral design of the building meets the serviceability requirements.

Wind Drifts

Drift Due to Wind			
Story	Height (ft)	Drift (in)	Allowable Drift (in)
ROOF	106.4	2.3492	3.192
8	93.1	2.0233	2.793
7	79.8	1.6790	2.394
6	66.5	1.3259	1.995
5	53.2	0.9828	1.596
4	39.9	0.6755	1.197
3	26.6	0.3945	0.798
2	13.3	0.1599	0.399

Seismic Drifts

Drift Due to Seismic Forces					
Story	**Height (ft)**	**Total Drift (in)**	**Story Drift (in)**	**Story Drift With Amplification Factor (in)**	**Allowable Drift (in)**
ROOF	106.4	1.6623	0.2828	0.84830	3.192
8	93.1	1.3795	0.2863	0.85885	3.192
7	79.8	1.0933	0.2787	0.83602	3.192
6	66.5	0.8146	0.2525	0.75745	3.192
5	53.2	0.5621	0.2052	0.61551	3.192
4	39.9	0.3569	0.1702	0.51074	3.192
3	26.6	0.1867	0.1232	0.36967	3.192
2	13.3	0.0635	0.0635	0.19041	3.192

Justification for a Redesign

From Steel to Concrete

The Washingtonian Center was designed around the requirement that it provide maximum flexibility to the leasable space to accompany any layout a tenant might desire. With this goal and the architectural layout with forty five foot outer bays, a composite steel structure was the simplest choice.

It was observed that this may not have been the most economical choice. While tenant spaces without columns certainly help to make the space more desirable and marketable, it requires a very thick floor system to achieve. In this case, the steel beams with the composite decking on top combined for a structural depth of 28”. After leaving another 2’ for a drop ceiling to house the mechanical and electrical systems, and a 9’ floor to ceiling height in the space, the total story height came to 13’-4”. With the building having a height restriction of 125’ tall by the Gaithersburg zoning regulations, this allowed for only eight stories and a mechanical penthouse on the roof. If a thinner floor system was selected, it could be possible to save enough space over the eight stories of the building to have enough room to add an addition floor to the building. This would increase the leasable space of the building by 12.5% and therefore allow for additional revenue for the building owner. A very efficient floor system design in terms of slab thickness would be to change it to a two-way post-tensioned flat plate. This system would save the depth to add the additional floor; however it would require a row of columns to be placed at the center of each of the outer two bays of the building. This would split the tenant’s space down the middle, and create approximately 20’ by 20’ square bays. A change to a concrete floor system may also save money in the actual cost of the structure.

From Braced Frames to Shear Walls

With the structural system being changed from steel to concrete, braced frames are no longer a reasonable choice for the lateral force resisting system. Additionally, the new shear walls should provide more structural rigidity and therefore limit the drift of the structure. In an area of low seismic activity such as Gaithersburg Maryland, this can be considered a benefit because seismic force dissipation will not be a problem.

Impact of the Redesign

To complete a thorough investigation, the effects that the changes to the structure will have on the rest of the building need to be considered. The new concrete structural will inevitably weight more than its steel counterpart. This has two effects. It will increase the seismic loads that structure will be exposed to, and it will also require a design of the foundations to handle the additional loading.

Proposed Structural System

Loads Used in Redesign

Introduction

The intent behind the structural redesign was to create a possible alternative to the steel system for comparison purposes. With this goal in mind, it only made sense to keep the loading on the structure the same as what the original designers used, with the obvious exception being self weight. Additionally the lateral forces required recalculating because of the elevation changes made to the story heights, and the change of the seismic parameters to account for the new concrete system and the added seismic mass. These lateral loads were recalculated using the procedures detailed in ASCE 7-05. The seismic forces were determined using the equivalent lateral force procedure (section 12.8). The updated loads that were used for design purposes are presented in this section.

Dead Loads

Post-Tensioned Concrete Flat Plate	100psf
Floor mass for Seismic Design	100psf
Partition Allowance	25psf
Sprinkler Allowance	5psf

Live Loads

Stairs and Exits	100psf
Elevator Machine Room	100psf
Offices	100psf
Public Spaces	100psf
Mechanical/Electrical Rooms	150psf
Roof	20psf

Wind Loads

Main Wind-Force Resisting System Wall Pressures, ASCE 7-05

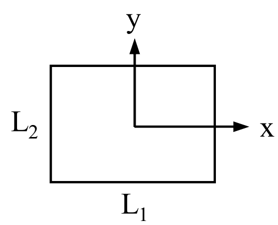
For Buildings of All Heights

General parameters:

Classification Category (I, II, III, IV):	**II**	
Basic Wind Speed, V:	**90**	mph
Hurricane Region (Y or N)?	**N**	
Importance factor, I:	1.00	
Mean recurrence interval:	**50**	year
MRI factor:	1.00	
Adjusted Wind Speed, V:	90	mph
Exposure Category (A, B, C, D):	**B**	
α:	7.00	
z_g:	1200	
Topographic factor, K_{zt}:	**1.00**	
Wind directionality factor, K_d:	**0.85**	
Gust Factor, G (x-dir wind):	**0.86**	
Gust Factor, G (y-dir wind):	**0.83**	
Internal pressure coefficient, $+GC_{pi}$:	**0.18**	
Internal pressure coefficient, $-GC_{pi}$:	**-0.18**	
Windward pressure coefficient, C_p:	0.80	
Side pressure coefficient, C_p:	-0.70	

Building properties:

Mean Roof Height, h:	**118**	ft
Typical length in x-direction, L_1:	**220**	ft
Typical length in y-direction, L_2:	**110**	ft



Recommended Gust Effect Factors:

Damping ratio, β:	**0.020**
Gust Factor, G (x-dir wind):	0.863
Gust Factor, G (y-dir wind):	0.831

Calculated values:

Velocity pressure coeff. at h, K_h:	1.04	
Velocity pressure at h, q_h:	18.3	psf

Base moments and shears:

Distance from ground level to bottom of pilecap:	**2**	ft
Base shear due to x-direction wind:	174	k
Moment at ground due to x-direction wind:	10,277	k-ft
Moment at bottom of pilecap due to x-direction wind:	10,625	k-ft
Base shear due to y-direction wind:	398	k
Moment at ground due to y-direction wind:	22,516	k-ft
Moment at bottom of pilecap due to y-direction wind:	23,311	k-ft

Notes:

1. Positive and negative pressures signify pressures acting toward and away from the surfaces, respectively.
2. Refer to Figure 6-6, ASCE 7-05 for wind pressure diagrams.

Main Wind-Force Resisting System Wall Pressures, ASCE 7-05

Story Elevations and Widths:

z (ft)	L_1 (ft)	L_2 (ft)	z (ft)	K_z	q_z (psf)	L_1 (ft)	L_2 (ft)
118	60	30	118	1.04	18.3	60	30
108	60	30	108	1.01	17.8	60	30
105	220	110	105	1.00	17.6	220	110
93	220	110	93	0.97	17.1	220	110
81	220	110	81	0.93	16.4	220	110
70	220	110	70	0.89	15.7	220	110
58	220	110	58	0.85	14.9	220	110
47	220	110	47	0.79	14.0	220	110
35	220	110	35	0.73	12.9	220	110
23	220	110	23	0.65	11.5	220	110
11	220	110	11	0.57	10.1	220	110
0	220	110	0	0.57	10.1	220	110

Wind Pressures and Story Forces:

X-DIRECTION WIND

		Leeward	External wall pressure			w/ pos. internal pressure			w/ neg. internal pressure			Total	Story
z (ft)	L/B	C_p	p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	pressure (psf)	Force (k)
118	2.00	-0.300	12.5	-4.7	-10.9	9.2	-8.0	-14.2	15.8	-1.4	-7.6	**17.2**	**6**
105	2.00	-0.300	12.1	-4.7	-10.9	8.8	-8.0	-14.2	15.4	-1.4	-7.6	**16.8**	**25**
81	2.00	-0.300	11.2	-4.7	-10.9	7.9	-8.0	-14.2	14.5	-1.4	-7.6	**15.9**	**31**
70	2.00	-0.300	10.8	-4.7	-10.9	7.5	-8.0	-14.2	14.0	-1.4	-7.6	**15.4**	**20**
58	2.00	-0.300	10.2	-4.7	-10.9	6.9	-8.0	-14.2	13.5	-1.4	-7.6	**14.9**	**19**
47	2.00	-0.300	9.6	-4.7	-10.9	6.3	-8.0	-14.2	12.9	-1.4	-7.6	**14.3**	**18**
35	2.00	-0.300	8.8	-4.7	-10.9	5.5	-8.0	-14.2	12.1	-1.4	-7.6	**13.5**	**17**
23	2.00	-0.300	7.9	-4.7	-10.9	4.6	-8.0	-14.2	11.1	-1.4	-7.6	**12.5**	**16**
11	2.00	-0.300	6.9	-4.7	-10.9	3.6	-8.0	-14.2	10.2	-1.4	-7.6	**11.6**	**15**
0	2.00	-0.300	6.9	-4.7	-10.9	3.6	-8.0	-14.2	10.2	-1.4	-7.6	**11.6**	**7**

Main Wind-Force Resisting System Wall Pressures, ASCE 7-05

For Buildings of All Heights

Wind Pressures and Story Forces:

Y-DIRECTION WIND

z (ft)	L/B	Leeward C_p	External wall pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	w/ pos. internal pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	w/ neg. internal pressure p_{ww} (psf)	p_{lw} (psf)	p_{side} (psf)	Total pressure (psf)	Story Force (k)
118	0.50	-0.500	12.1	-7.6	-10.6	8.8	-10.8	-13.9	15.4	-4.3	-7.3	**19.6**	**2**
105	0.50	-0.500	11.7	-7.6	-10.6	8.4	-10.8	-13.9	15.0	-4.3	-7.3	**19.2**	**57**
81	0.50	-0.500	10.9	-7.6	-10.6	7.6	-10.8	-13.9	14.2	-4.3	-7.3	**18.4**	**71**
70	0.50	-0.500	10.4	-7.6	-10.6	7.1	-10.8	-13.9	13.7	-4.3	-7.3	**18.0**	**46**
58	0.50	-0.500	9.9	-7.6	-10.6	6.6	-10.8	-13.9	13.2	-4.3	-7.3	**17.4**	**45**
47	0.50	-0.500	9.3	-7.6	-10.6	6.0	-10.8	-13.9	12.5	-4.3	-7.3	**16.8**	**43**
35	0.50	-0.500	8.5	-7.6	-10.6	5.2	-10.8	-13.9	11.8	-4.3	-7.3	**16.1**	**41**
23	0.50	-0.500	7.6	-7.6	-10.6	4.3	-10.8	-13.9	10.9	-4.3	-7.3	**15.2**	**39**
11	0.50	-0.500	6.7	-7.6	-10.6	3.4	-10.8	-13.9	10.0	-4.3	-7.3	**14.3**	**37**
0	0.50	-0.500	6.7	-7.6	-10.6	3.4	-10.8	-13.9	10.0	-4.3	-7.3	**14.3**	**18**

Wind Pressures and Story Forces: Summary

z (ft)	K_z	q_z (psf)	L_1 (ft)	L_2 (ft)	X-DIRECTION WIND Leeward C_p	X-DIRECTION WIND Total pressure (psf)	X-DIRECTION WIND Story Force (k)	Y-DIRECTION WIND Leeward C_p	Y-DIRECTION WIND Total pressure (psf)	Y-DIRECTION WIND Story Force (k)
118	1.04	18.3	60	30	-0.300	**17.2**	**6**	-0.500	**19.6**	**2**
105	1.00	17.6	220	110	-0.300	**16.8**	**25**	-0.500	**19.2**	**57**
81	0.93	16.4	220	110	-0.300	**15.9**	**31**	-0.500	**18.4**	**71**
70	0.89	15.7	220	110	-0.300	**15.4**	**20**	-0.500	**18.0**	**46**
58	0.85	14.9	220	110	-0.300	**14.9**	**19**	-0.500	**17.4**	**45**
47	0.79	14.0	220	110	-0.300	**14.3**	**18**	-0.500	**16.8**	**43**
35	0.73	12.9	220	110	-0.300	**13.5**	**17**	-0.500	**16.1**	**41**
23	0.65	11.5	220	110	-0.300	**12.5**	**16**	-0.500	**15.2**	**39**
11	0.57	10.1	220	110	-0.300	**11.6**	**15**	-0.500	**14.3**	**37**
0	0.57	10.1	220	110	-0.300	**11.6**	**7**	-0.500	**14.3**	**18**

Seismic Forces

The seismic loads were computed using the Equivalent Lateral Force Procedure. This was deemed an acceptable application of the procedure based the requirements of section 12.6. The table following this discussion show all of the values used along with the specific code references from which they were found. The S_s and S_1 values for the site in Gaithersburg were determined using the U.S.G.S. website and their seismic hazard maps that use the latitude and longitude coordinates to determine the results. The period of the building was determined from the elastic analysis preformed by Etabs. The upper limit on the period that is allowed to be used for the determination of Cs, was found to be less than the fundamental period from the Etabs analysis. Therefore the period used to determine Cs was the limit imposed by section 12.8.2. The seismic forces were distributed vertically following section 12.8.3. The numbers can be found on the table on the next page. K was determined through interpolation based on the period of the building.

The additional seismic requirements from equivalent lateral force procedure will be addressed here. Some of these requirements did not apply to the Washingtonian Center, while others were fulfilled without any additional hand calculations. Section 12.8.4.1 requires that the inherent torsion of the building resulting from the center of mass and the center of rigidity not occurring at the same location be included in the analysis. For the Washingtonian Center, this wasn't a large concern in the design of the shear walls because the center of rigidity and the center of mass nearly fell on the same point. Etabs calculated both of these points and then included the eccentricity of the loading in the elastic analysis. The accidental torsion requirement of section 12.8.4.2 was satisfied by specifying the torisonal seismic load cases to include the 5% offset. The amplification of the accidental torsional moment was not required based on the Washingtonian Center's seismic design category being A (as determined by the requirements of section 11.6). For a discussion of the story drift determination and the limits imposed by sections 12.8.6.1-2, 12.8.7 and 12.12 please refer the section entitled "Analysis of the Concrete Structure".

Equivalent Lateral Force Procedure, ASCE 7-05

Input for General Analysis

W=	29295 kips	
S_s =	0.157	(http://earthquake.usgs.gov/research/hazmaps/design/)
S_1 =	0.051	(http://earthquake.usgs.gov/research/hazmaps/design/)
F_a =	1.2	(Table 11.4-1)
F_v =	1.7	(Table 11.4-2)
S_{MS} =	0.188	(Eq. 11.4-1)
S_{M1} =	0.087	(Eq. 11.4-2)
S_{DS} =	0.126	(Eq. 11.4-3)
S_{D1} =	0.058	(Eq. 11.4-4)
R =	5	(Table 12.2-1)
I =	1	(Table 11.5-1)
	A	Seismic Design Category (Table 11.6-1)
C_T =	0.02	(Table 12.8-2)
h_n =	117.9	
x=	0.75	(Table 12.8-2)
Cu=	1.7	(Table 12.8-1)
TL=	8	(Section 11.4.4)

Output

Ta =	0.716	(Approximate Period)	
T=	1.217	Upper Limit on Period-Section 12.8.2	**<controls**
T=	1.7744	Fundamental Period From Etabs Analysis	
C_s =	0.025	(Eq. 12.8-2)	
C_s =	0.010	(Eq. 12.8-3)	**<controls**
C_s =	0.062	(Eq. 12.8-4)	
Use C_s =	0.010		

Base Shear

V= 292.95 kips

Over-Turning Moment

M= 23082 k-ft

Equivalent Lateral Force Procedure, ASCE 7-05

Vertical Distribution of Base Shear

k= 1.37 (Section 12.8.3)

Level	h_x (ft)	W_x (kips)	$W_x h_x^k$	C_{vx}	F_x (kips)
Pent. Roof	117.9	400	260742	0.03	10
Pent. Floor	108.4	175	101771	0.01	4
Roof	104.7	3000	1664251	0.21	61
9th	93.1	3215	1520544	0.19	56
8th	81.4	3215	1266964	0.16	47
7th	69.8	3215	1028155	0.13	38
6th	58.2	3215	803210	0.10	30
5th	46.5	3215	592128	0.07	22
4th	34.9	3215	400968	0.05	15
3rd	23.3	3215	231598	0.03	9
2nd	11.6	3215	89794	0.01	3
Sum		29295	7960125	1.00	293

Lateral Load Discussion

When the new lateral loads were calculated, it was expected that the wind loads would be essentially the same and the seismic loads would have increased significantly based on the fact that the weight of the structure increased by roughly 28%. This expectation however didn't take into consideration that the response modification factor changed from 3 in the steel building to 5 in the concrete shear wall building. This effectively dropped the Cs value down to 0.01, from 0.016. This resulted in the design loads for the building to be less for the concrete structure then the steel structure.

Two-Way Post-Tensioned Flat Plate Design

Design Procedure

Ram Concept served as the program that was chosen to do the PT design. It was picked because of its ability to do an entire plate design at once and because the Washingtonian Center is a very simple building without irregularities that would justify an individual frame design program. The slab edge, column locations and shear wall locations were imported from an Auto CAD dxf file. Preliminary calculations were done to estimate the thickness of the plate needed to resist punching shear around the columns, with a thickness of 8” selected. The floor plate was then loaded with the dead and live loads at the appropriate locations. Once the model of the floor was constructed, the design spans were laid out in each direction at locations where the forces and concrete stresses were believed to be critical. These spans corresponded to strips along the column lines and around openings in the slab.

The next step in the design process was to determine which direction the banded tendons would run in, and which direction the distributed tendons were span. The east-west direction of the building was chosen for the banded tendons because minimal breaks for openings along the span. This direction also corresponded to the direction of the girders in the original design. The distributed tendons were then left to span in the north-south direction. The number of banded tendons along each column line was then selected based on the amount needed for a pre-compression of 150 psi, this was based on the requirement in 18.12.4 of a minimum of 125 psi pre-compression in the slab. The pre-compression was increased because if a slab is designed for the code minimum, there will inevitably be sections that will fall below the threshold , due to irregular geometry and losses in the post-tensioned cable. The geometry of the distributed tendons was then decided. The spacing that was used came from several factors. The first being the requirement of 18.12.6 that two tendons must pass over each column in each direction and the second was the same pre-compression requirement as before. Eventually it was decided to use two tendons per group and space them at 48” wherever possible. In an 8” slab this would give a pre-compression value of 140 psi, above the code minimum and within acceptable values. To start, the tendons were given the maximum amount of drape possible based on cover requirements, with lowest points occurring at the center of the span and the high points corresponding to the column locations.

After all of the geometric concerns of the PT were worked out, an iterative design process was used to arrive at the final design of the slab. This first involved analyzing the model and then making changes in the drapes of the tendons at mid-span to achieve a balancing load of approximately 75% of the dead load. It should be noted here that if one or more of the spans had a balance load of significantly less than 75%, the spacing of the tendons would have been changed or strands would have been added in those particular spans. As it turned out, many of the spans balanced very close to the 75% goal at the maximum drape. Changes were made to the drapes of both the banded and distributed tendons until every span met the balance load requirements. Once this phase of the design was complete, the slab was checked for failures along any of the spans. In a few cases the allowable tensile stress for the slab was exceeded at the lower face. This was a result of too little pre-compression to cancel out the tensile stresses. To rectify the situation, additional strands were added to this spans and the dead load was rebalanced by altering the tendon profiles.

Shear checks were done around the columns, walls and any other condition that justified one. The 8" slab was ok at most locations, however around a number of the columns punching shear failures were occurring without any shear reinforcement. At these locations stud-rails were specified to be design to handle the extra shear. Once stud-rails were added there no longer were and shear failures.

In addition to the shear and PT steel, mild steel reinforcement was added to the slab to satisfy code requirements. Minimum top steel as prescribed by 18.9.3.3 was added around columns and disturbed between lines that were 12" (1.5h) outside opposite faces of the columns. This reinforcement was extended one-sixth the clear span from the face of the column per section 18.9.4.2. In tensile regions (the bottom of the slab at mid-span) minimum steel was added to satisfy 18.9.3.2 and extended one-third the clear span to satisfy 18.9.4.2. Once all the steel detailing was finished, the design of the slab was complete.

Material Properties

PT System

½" Un-bonded Seven Wire Strand

$A_{ps} = 0.153\ in^2$

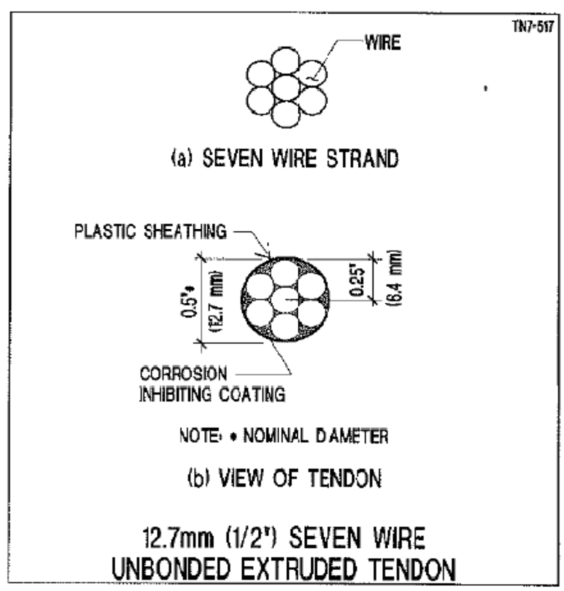
$E_{ps} = 28000$ ksi

$f_{se} = 175$ ksi

$f_{py} = 243$ ksi

$f_{pu} = 270$ ksi

$f_j = 216$ ksi



½" Supplemental Shear Reinforcement

Stud Area = 0.196 in^2

Head Area = 1.96 in^2

Minimum Head Spacing = 0.5 in

F_y = 50 ksi

Minimum Studs Per Rail: 2

Concrete

f'_{ci} = 3000 psi

f'_c = 5000 psi

Density: 150 pcf

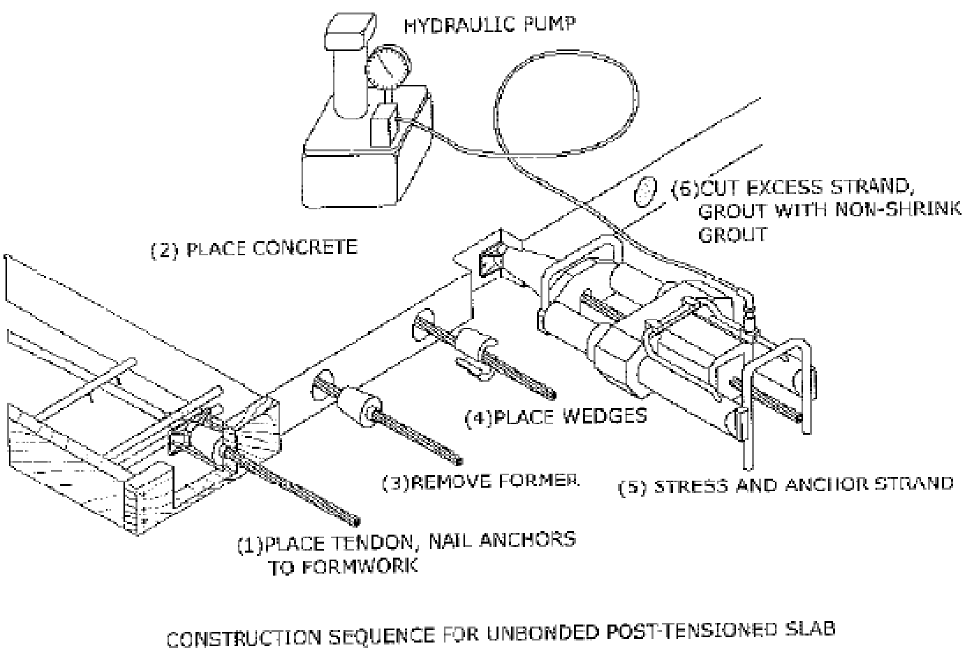
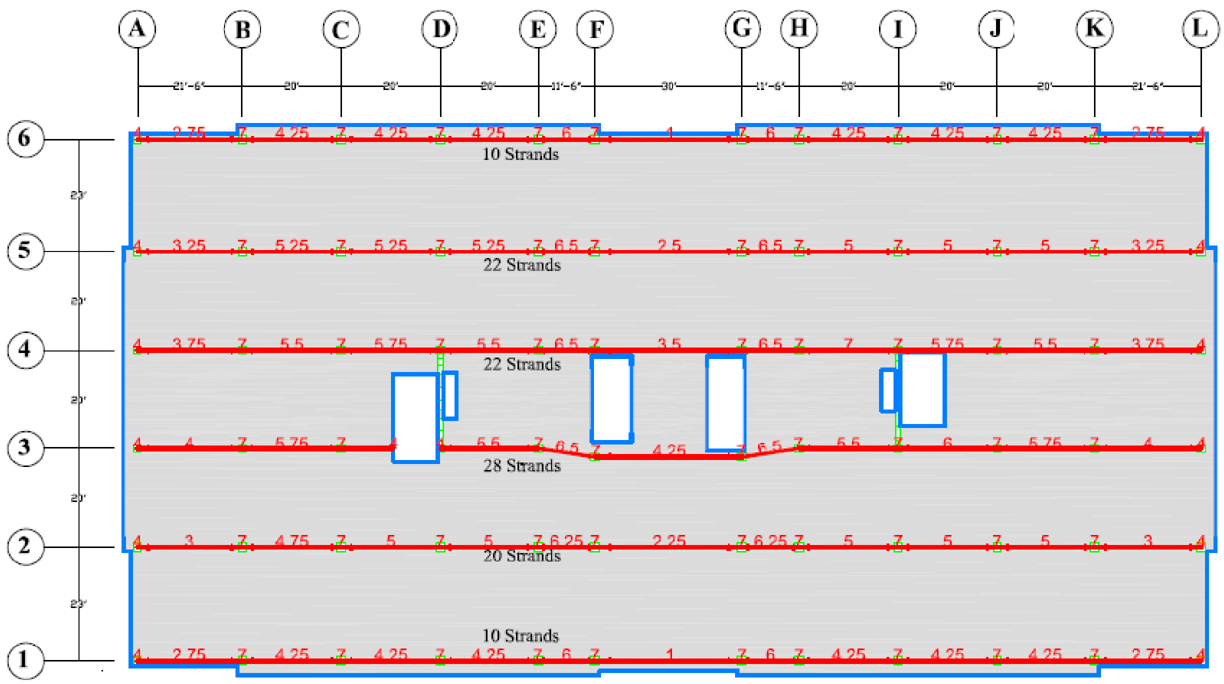
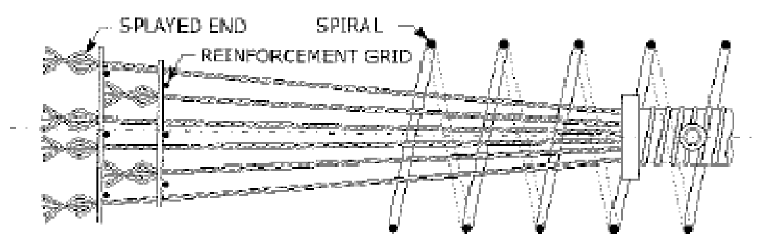


Diagram taken from the Post-Tensioning Manual

Banded Tendon Design

The diagram below depicts the banded tendon design. The elevations of the tendon profile are shown at the points over the column and wall supports as well as at mid-span. It should be noted that the tendons end at a profile of 4” which corresponds to the center of the 8” slab. This is standard practice and ensures that the post-tensioning force can be evenly distributed to the end face of the slab. Additionally the banded tendons will be splayed at the ends to further distribute the force and prevent blow-outs from occurring. Please refer to the detail below for a visual depiction of the splayed ends. The lack of symmetry in the number of strands per band is a result of uneven loading across the spans. Additional strands were added to account for tensile failures in the bottom of the slab.



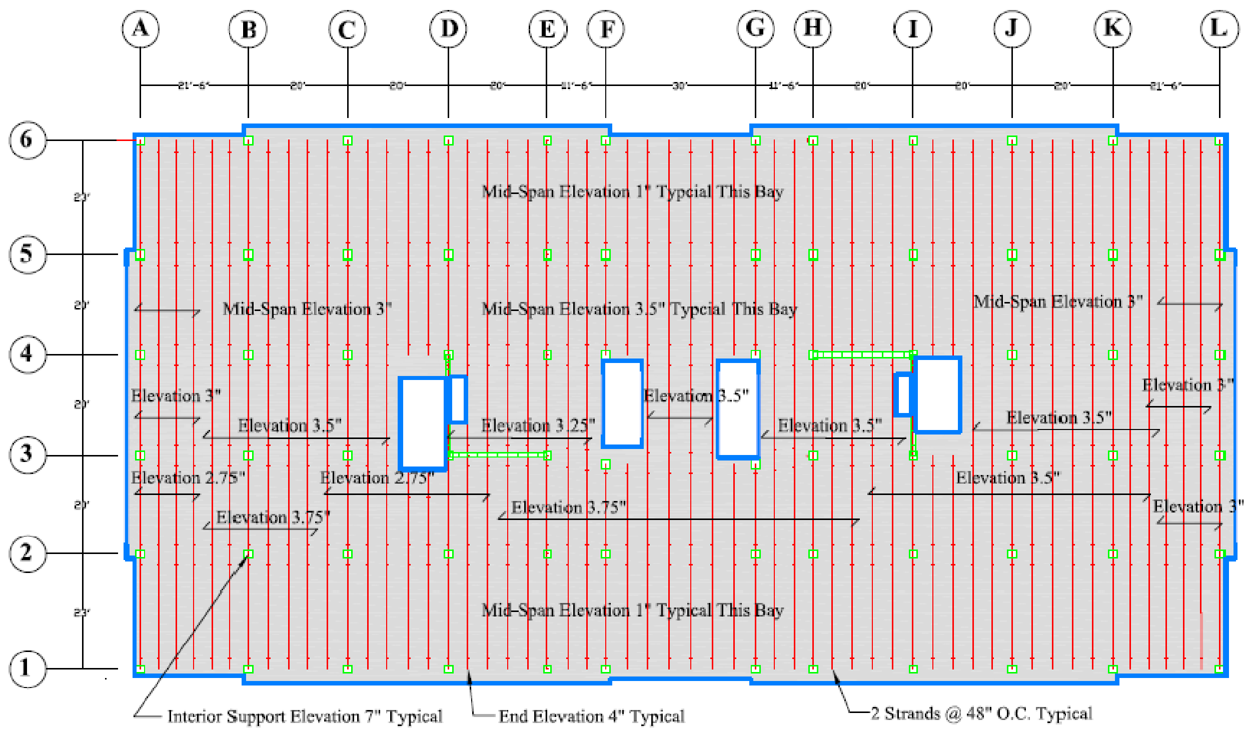


(b) SPLAYED FIXED END ANCHOR

Diagram taken from the Post-Tensioning Manual

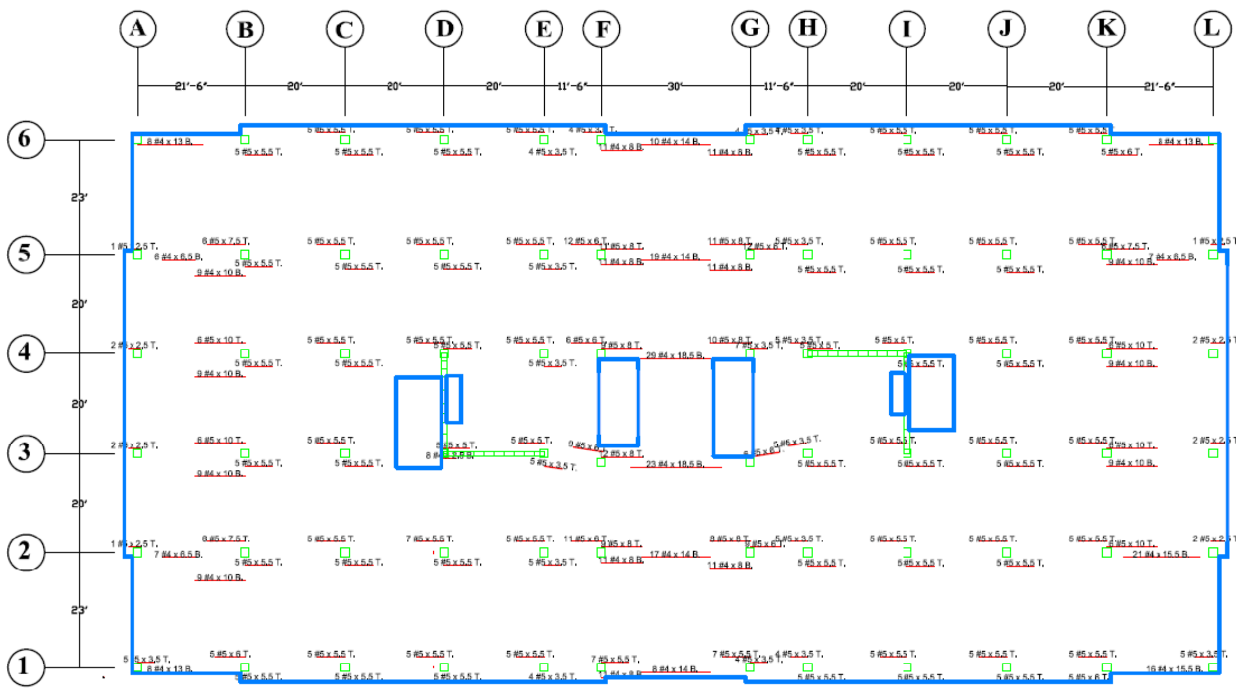
Distributed Tendon Design

The goal in the distributed tendon design was to keep the geometry of the tendons as uniform as possible. This included both the spacing of the tendons and the profiles. All tendon profiles are at 7" over a support, and 4" at the end of the tendon. The mid-span profiles were manipulated to achieved the proper balancing force and to meet the required stress levels in the slab. It should be noted that each tendon shown on the drawing is actually a pair of tendons.

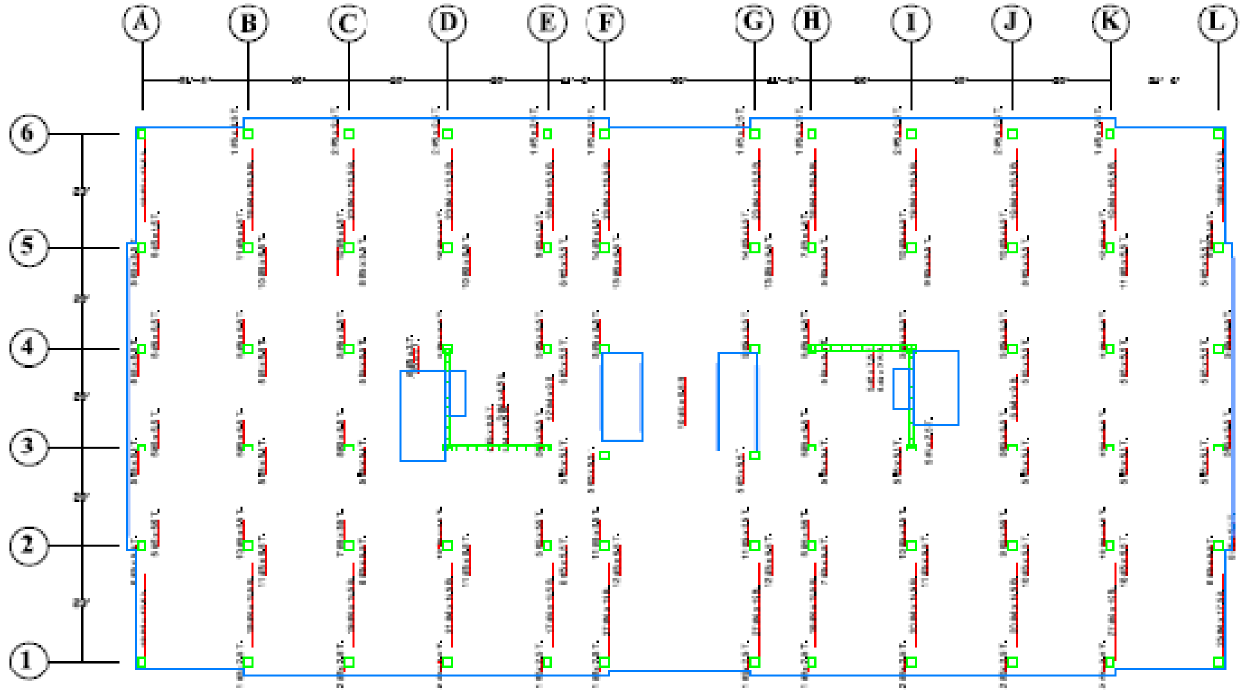


Additional Latitude Reinforcement

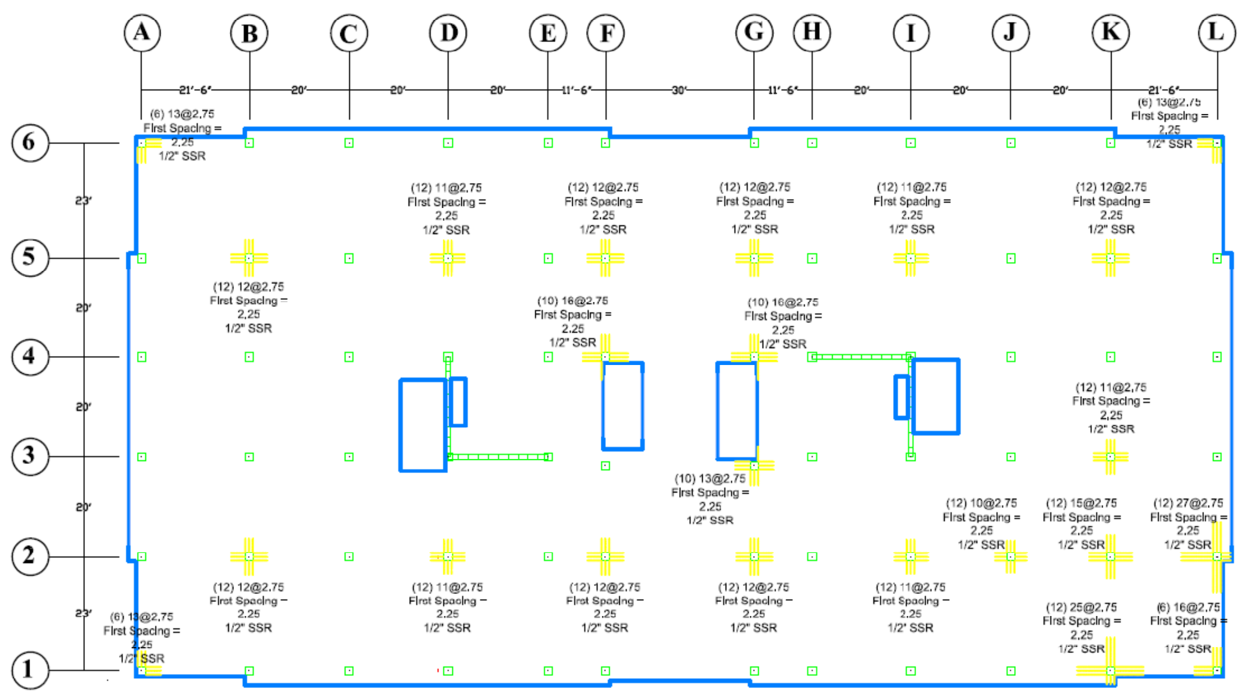
Additional reinforcement required typically was #5 bars on the top and #4 bars on the bottom. The number of bars required changed significantly depending on location and loading. An enlarged version of this diagram can be found in the appendix.



Additional Longitudinal Reinforcement



Supplemental Shear Reinforcement



Concrete Shear Wall Design

Design Procedure

The first step in designing any lateral system is first to select the location to place it. For the Washingtonian Center, the most logical choice was determined to be in the same location that the braced frame occupied in the previous design. This made sense because it would locate the walls around the core of the building and wouldn't interfere with the open floor plan in the tenant space. One thing that needs to be mentioned with this location is that the wall passes between the stairwells and the air shaft that is used to pressurize the stairs in the advent of a fire. This is a simple problem to rectify and only requires an opening being cut into the wall at every level so that the stairwell remains a safety zone in the advent of a fire.

The overall plan in the design of the shear walls was twofold. The first objective was to size the walls to limit the drift of the building under lateral loading to acceptable limits. The second part of the design process was to specify the proper reinforcement in the wall to ensure the walls met their strength requirements. Both of these steps were completed with the use of Etabs to model and analyze the structure. A model was created using the standard material properties for concrete and the steel reinforcement. The columns were done with an assumed initial size, and then were later changed to reflect the actual size used in the design. The post-tensioned floor was modeled as a shell, so that both membrane and plate behavior would be accounted for (the membrane definition allows both in plane and out of plane deformations). The shear walls were modeled using a shell definition for the same reasons as mentioned above.

Once the physical model was constructed the floor areas were manually meshed to ensure proper behavior results for the diaphragm. The meshing was done carefully to ensure all corners of the elements aligned with another to allow for force transfer between the finite elements. The floors were meshed into elements of approximately 2'x2' squares. Careful attention was paid to the meshing to keep all elements as square as possible, with a maximum side to length ratio of 2 to 1. This is in line with common good modeling technique and provides for accurate transfer of forces. Once the floors were meshed, they were loaded with the appropriate live, dead and other loads. The façade of the building was accounted for with line loads on the edge of the slab. Wind and seismic load parameters were assigned, and the program was allowed to calculate the lateral loads on its own. This was deemed reasonable because the building is a simple geometric rectangle without any irregularities that would warrant a manual assignment of the lateral loads. The loads that the program calculated were later checked against the loads found by hand and were found to be within 5%. Rigid diaphragm definitions were assigned to each of the floors, and the roof as well. Each level was given its own diaphragm assignment.

The final step in the modeling process was too define all of the possible load combinations. These were based off of the service load combinations given in ASCE 7-05. It was complicated by the need to reverse all of the lateral loads in every combination to account for the possibility of wind or seismic forces coming from any direction. In all, the total load combinations total more than 100.

After all that was completed the model was analyzed for the first time. Once completed, point displacements on the highest level resulting from wind loading were compared to the industry standard of h/400 for the allowable building drift. It was found that the initial 12” shear walls provided plenty of stiffness and limited the drift to well under acceptable levels. It was decided to change the shear wall size to 10” and the model was analyzed again. After doing the same comparison, the drift levels were still well within acceptable levels but it was decided that anything narrower just isn’t done in common practice for shear wall design. The maximum story drifts from seismic loads were compared to the code specified limits and also were within the acceptable range. The complete analysis details are presented later in the section entitled “Analysis of the Concrete Structure”.

One of the most powerful features of Etabs, is the ability to use section property modifiers for model elements to account for reduced stiffness due to cracking of the concrete. If these modifiers are used correctly the actual behavior of the building can be accurately modeled. In a shear wall building like the Washingtonian Center, it is important to modify the section properties of the shear walls in areas where stresses exceed the limit to be considered un-cracked. When the section properties of the walls are modified, the drift of the building will increase because of the reduced stiffness of the structure. This increase in drift could exceed the allowable drift limits and show the importance of using accurate section property modifiers. The cracking factors are taken from section 10.10.4.1 of the ACI 318-05. For the service model of the Washingtonian Center, the cracking coefficients of the walls and columns were initially set at 1.0. After the shear walls were sized at 10”, the stress levels in the walls and columns were checked using the output from the analysis. Where tensile forces exceeded $7.5\sqrt{f'c}$, the membrane modifiers f_{22} and f_{12} along with the bending modifiers m_{11}, m_{22}, and m_{12} were changed to 0.5. This effectively reduced the section properties of these portions of the wall to half their initial values.

Once the shear walls were sized at 10” and deemed adequate to meet the drift limits, the second part of the design process began. This involved saving a second copy of the model to be used as the strength model. The two models are essentially the same, with the exception of the load combinations used, and the section modifiers. The load combinations were changed to the strength combinations specified in ASCE 7-05. These combinations were applied to account for lateral loading from all directions, for each load case. In all, the load combinations again totaled over 100. The membrane modifiers for the shear walls were started at 0.7, on account of the higher loads the building is subjected to under the strength combinations and the cracking that is expected to occur under these loads.

Pier labels were also assigned to the finite elements of the shear walls so that the program would record and report the forces in these elements. The “simplified tension and compression” assignment was given to the walls to specify to the program to perform this preliminary step in the design of the wall. This means the program will consider the forces in the walls and report the total area of vertical steel that is required to resist the bending moments in the wall. For this assignment the program assumes all steel is placed at the ends of the wall, and act as cords to resist the bending forces. This is the first step in the shear design module of Etabs and will assist in specifying the amount of uniform reinforcing to place in the wall later on. Once this was done the model was analyzed and the results were examined. In most areas, the minimum area of shear reinforcement controlled. Next a more complicated "Uniform Reinforcing" assignment was given to the walls. This also required specifying the size and spacing of the bars to be used. The model was re-analyzed and the results were checked for areas where there were failures. Then the reinforcement specified was rethought and the model was run again to check the new layout. This process continued until there were no failures in the wall. Once the design was at this point, the tensile stresses in the walls were checked at various locations as in the service model to check for cracking. Where the stresses were too high the membrane modifiers were changed to 0.35 and the model was run again, and the shear wall design module was additionally run. The reinforcement was checked again for failures and adjustments were made as required. The complete wall design can be found in the following sections.

Material Properties

Concrete

f'_c = 5000 psi

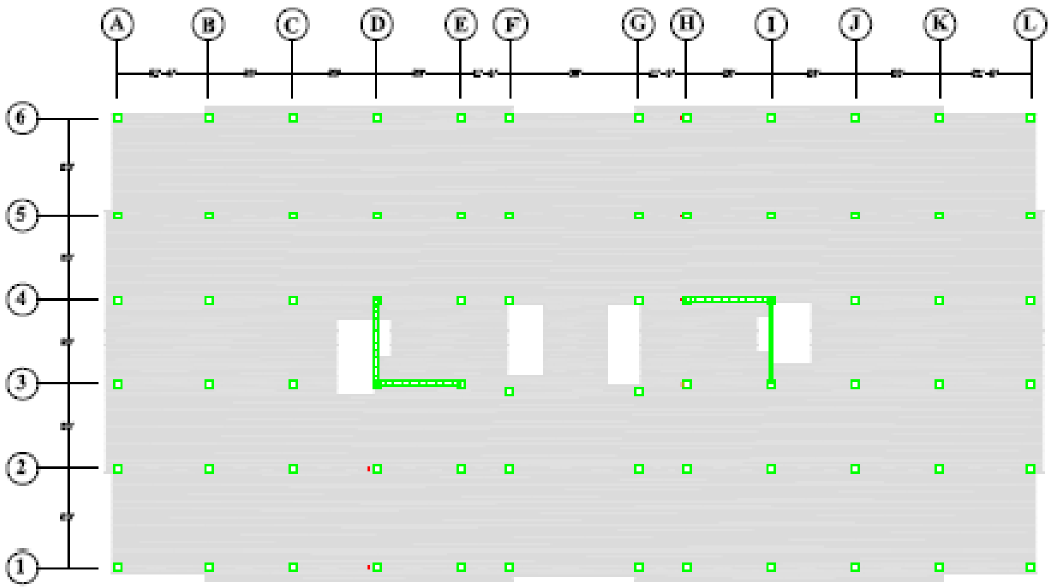
Density: 150 pcf

Reinforcement

Bar Size Designation	Nominal Area	Nominal Weight	Nominal Diameter
#3	0.11	0.376	0.375
#4	0.20	0.668	0.500
#5	0.31	1.043	0.625
#6	0.44	1.502	0.750
#7	0.60	2.044	0.875
#8	0.79	2.670	1.000
#9	1.00	3.400	1.128
#10	1.27	4.303	1.270
#11	1.56	5.313	1.410
#14	2.25	7.65	1.693
#18	4.00	13.60	2.257

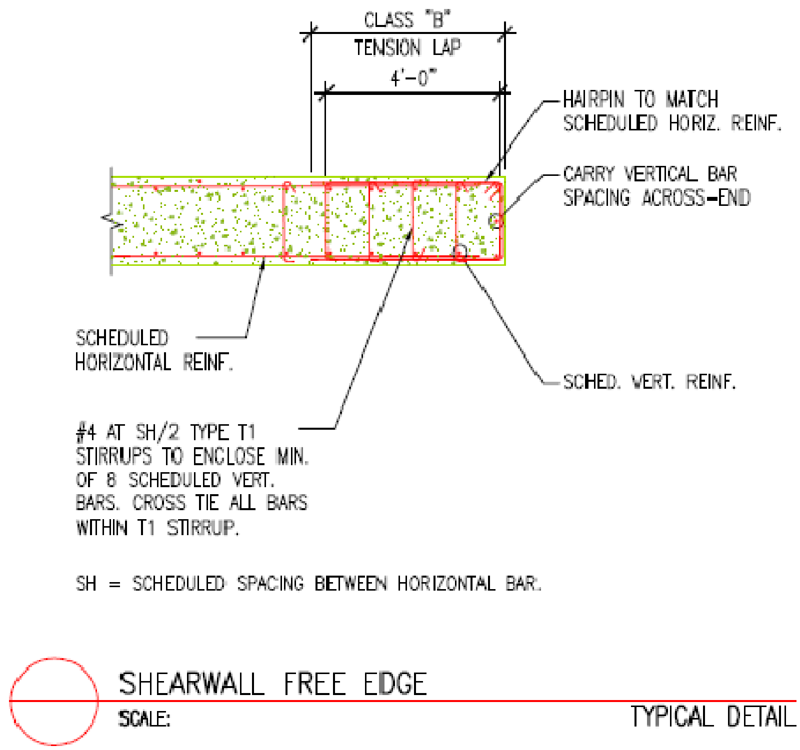
Shear Wall Design

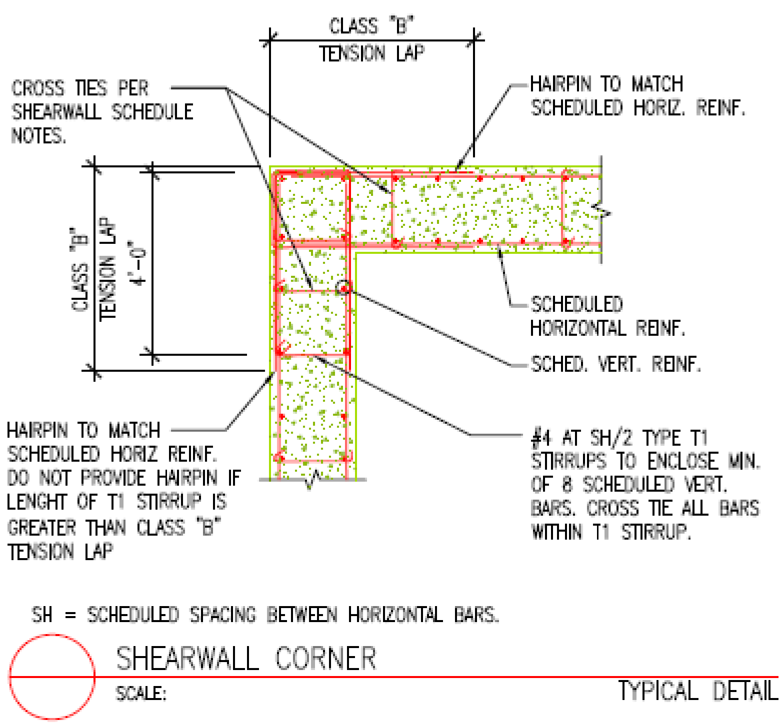
The location of the shear walls is shown in the figure below. These are the same locations that the braced frames occupied in the original design. These particular locations of the walls are fairly close to the center of the building and therefore resulted in the first modal displacement of the building to be torsion. It can be seen that there are two shear walls helping to resist the lateral loads in each direction. Each one of the shear walls is 20' in length and 10" wide. Please refer to the shear wall schedule below for the typical horizontal and vertical reinforcement.



Shear Wall Schedule

Level	Vert. Bars Each Face	Hor. Bars Each Face
Roof	#5 @ 12" o.c.	#4 @ 12" o.c.
9th	#5 @ 12" o.c.	#4 @ 12" o.c.
8th	#5 @ 12" o.c.	#4 @ 12" o.c.
7th	#5 @ 12" o.c.	#4 @ 12" o.c.
6th	#5 @ 12" o.c.	#4 @ 12" o.c.
5th	#7 @ 12" o.c.	#4 @ 12" o.c.
4th	#7 @ 12" o.c.	#4 @ 12" o.c.
3rd	#7 @ 12" o.c.	#4 @ 12" o.c.
2nd	#7 @ 12" o.c.	#4 @ 12" o.c.
Base	#7 @ 12" o.c.	#4 @ 12" o.c.


CLASS "B"
TENSION LAP
4'-0"
HAIRPIN TO MATCH SCHEDULED HORIZ. REINF.
CARRY VERTICAL BAR SPACING ACROSS-END
SCHEDULED HORIZONTAL REINF.
SCHED. VERT. REINF.
#4 AT SH/2 TYPE T1 STIRRUPS TO ENCLOSE MIN. OF 8 SCHEDULED VERT. BARS. CROSS TIE ALL BARS WITHIN T1 STIRRUP.
SH = SCHEDULED SPACING BETWEEN HORIZONTAL BAR.
SHEARWALL FREE EDGE
SCALE:
TYPICAL DETAIL

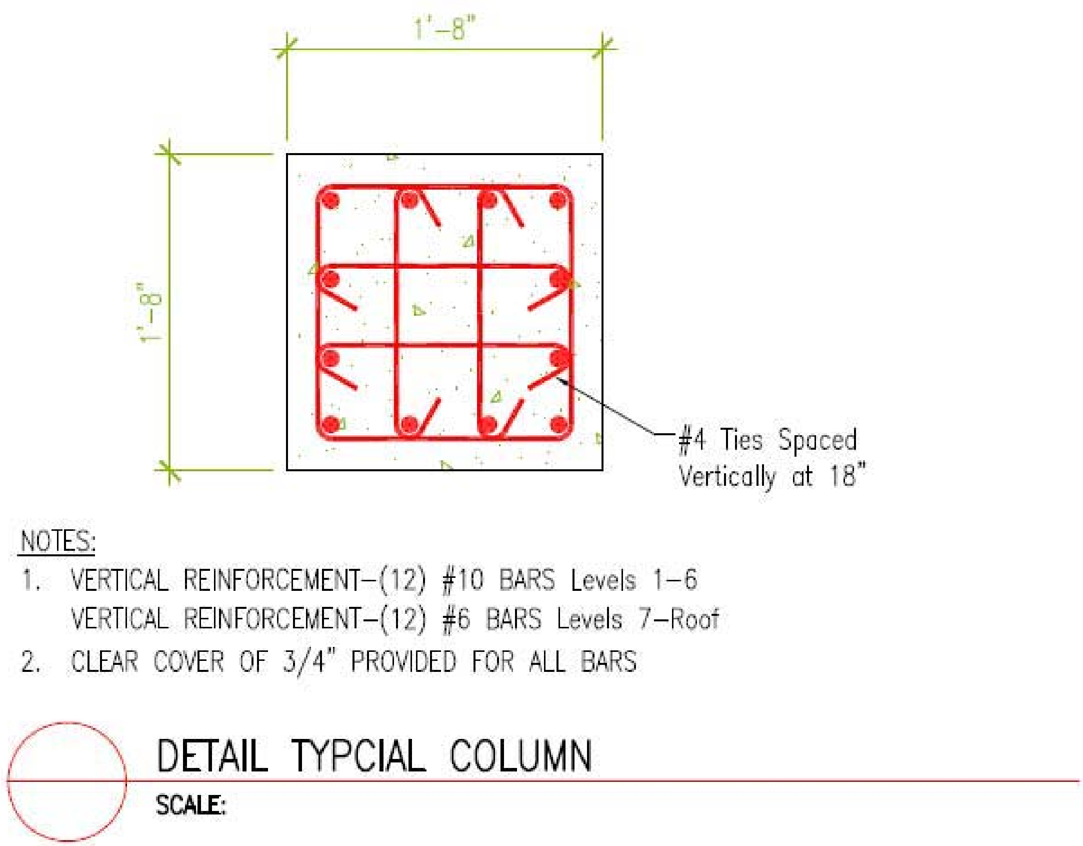

CLASS "B"
TENSION LAP
CROSS TIES PER SHEARWALL SCHEDULE NOTES.
HAIRPIN TO MATCH SCHEDULED HORIZ. REINF.
CLASS "B" TENSION LAP 4'-0"
SCHEDULED HORIZONTAL REINF.
SCHED. VERT. REINF.
HAIRPIN TO MATCH SCHEDULED HORIZ REINF. DO NOT PROVIDE HAIRPIN IF LENGHT OF T1 STIRRUP IS GREATER THAN CLASS "B" TENSION LAP
#4 AT SH/2 TYPE T1 STIRRUPS TO ENCLOSE MIN. OF 8 SCHEDULED VERT. BARS. CROSS TIE ALL BARS WITHIN T1 STIRRUP.
SH = SCHEDULED SPACING BETWEEN HORIZONTAL BARS.
SHEARWALL CORNER
SCALE:
TYPICAL DETAIL

Concrete Column Design

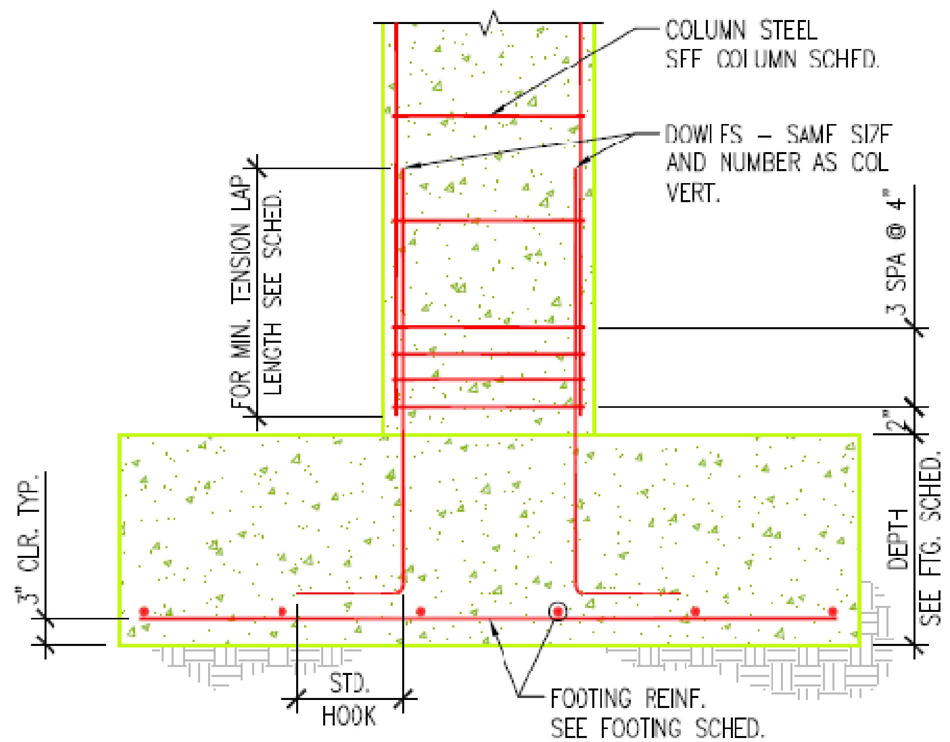
Design Procedure

To simplify the design and to make construction easiest, it was decided that all columns in the building would use the same geometric dimensions. This allows for the same forms to be used to cast all of the columns. Another way to simplify the construction of the columns was to only change the specified reinforcing at one level in the building. This avoids complicated fabrication and installation of many different rebar cages in the building. It was decided that the reinforcement would be changed starting at the 6^{th} floor. The columns for the Washingtonian Center were designed with the aid of PCA Column. To find the design loads, the maximum axial load at the base of the structure in any column was taken from the strength Etabs model. This was the maximum load from all of the considered load cases, but it was found that the 1.2D + 1.6L case controlled with the highest axial loads. This makes sense because the lateral forces on the building were almost entirely being transferred to the shear walls because of their far greater stiffness as compared to the columns. The maximum moment experience by any column on the ground level was also used in the design; however these moments were very small because of the distribution of the lateral forces mentioned above.

Column Design

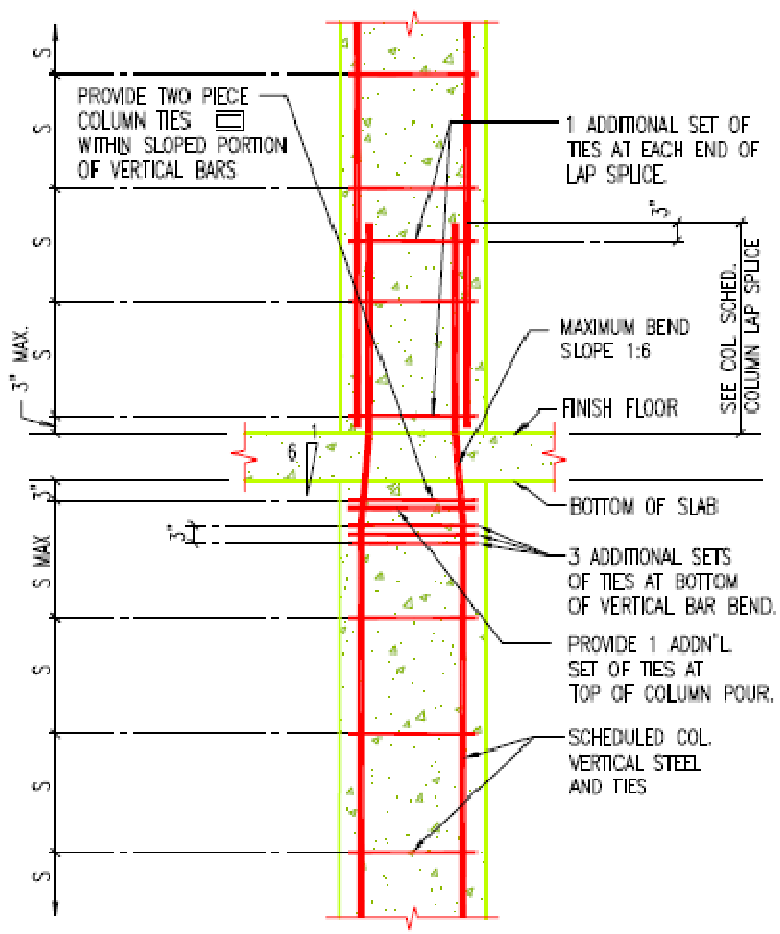


This detail shows a standard gravity column tied into a spread footing. It applies to all columns with a footing designation beginning with an F. Refer to the footing design section of clarification.



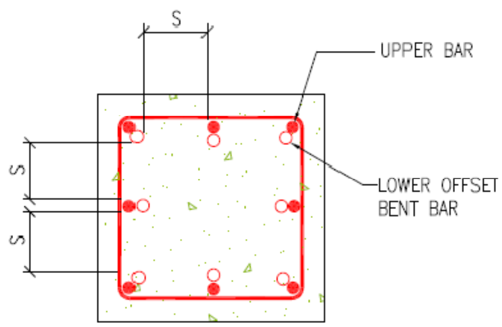
TYPICAL CONCRETE COLUMN FOOTING
SCALE:

Shown below is the standard detail for all column through slab conditions. Note that the vertical steel in the column must be spliced to keep the reinforcement continuous. For lap splice details refer to the diagram on the next page.



NOTE:
1. "S" DENOTES SCHEDULED TIE SPACING. TYP. IS 18"

COLUMN SPLICE AT FLOOR LINE SLAB FL. SYSTEM
SCALE: TYPICAL DETAIL

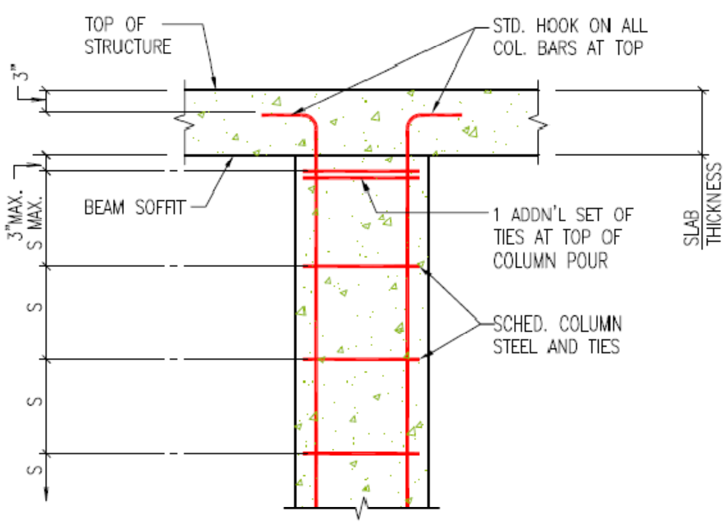


NOTES:

1. LOWER COLUMN VERTICAL BARS SHALL BE OFFSET BENT IN THE SHOP TOWARD THE INTERIOR OF THE COLUMN AS SHOWN.
2. S = CLEAR BAR SPACING TO BE USED FOR DETERMINATION OF TENSION SPLICE LENGTH CATEGORY.
3. WHEN CLEAR BAR SPACING IS NOT THE SAME AT DIFFERENT COLUMN FACES, SMALLER SPACING SHALL BE USED TO DETERMINE THE APPLICABLE SPLICE CATEGORY.
4. REFER TO "TYPICAL DETAIL, CLEAR SPACING OF SPLICED BARS, STAGGERED SPLICES," FOR CRITERIA TO DETERMINE CLEAR SPACING FOR BARS AT STAGGERED SPLICES.

DETAIL NORMAL LAP SPLICE

SCALE: (FOR SQUARE OR RECTANGULAR COLUMNS)



NOTES:

1. "S" DENOTES SCHEDULED TIE SPACING.

12
S9-01

COLUMN AT ROOF OR TOP FLOOR
SLAB FLOOR SYSTEM DETAIL

SCALE: 1" = 1'-0" TYPICAL DETAIL

Spread Footing Designs

Design Procedure

The column and shear wall loads were taken from the Etabs models. The strength model supplied the ultimate loads on the foundations while the service model loads were used to check the serviceability of the foundations. PCA Mats was used to design all of the footings. The required reinforcement was calculated for each finite element that the foundation was broken up into. These areas were based off of the average requirement for that particular finite element. To get the general reinforcement specified in the designs, the average of reinforcement needed across the footing was taken. The columns supported essentially only the gravity loading and therefore didn't present any big challenges in their designs. Top reinforcement wasn't needed for any of the gravity column footings, which was expected because they didn't have any uplift on them. The shear wall foundations had a bit of a problem with uplift and required a large footing that once designed nearly ran into the footings for the gravity columns nearby. To simplify things the surrounding columns and the shear walls were combined into one large mat footing with all of the loads applied at the appropriate points. In addition to this combined footing, the columns spaced along E, F and G, H intersected each other so they were combined into six combined footings of the same size. An allowable bearing pressure of 6 kips per square foot was taken from the geo-tech report of the site.

Spread Footing Designs:

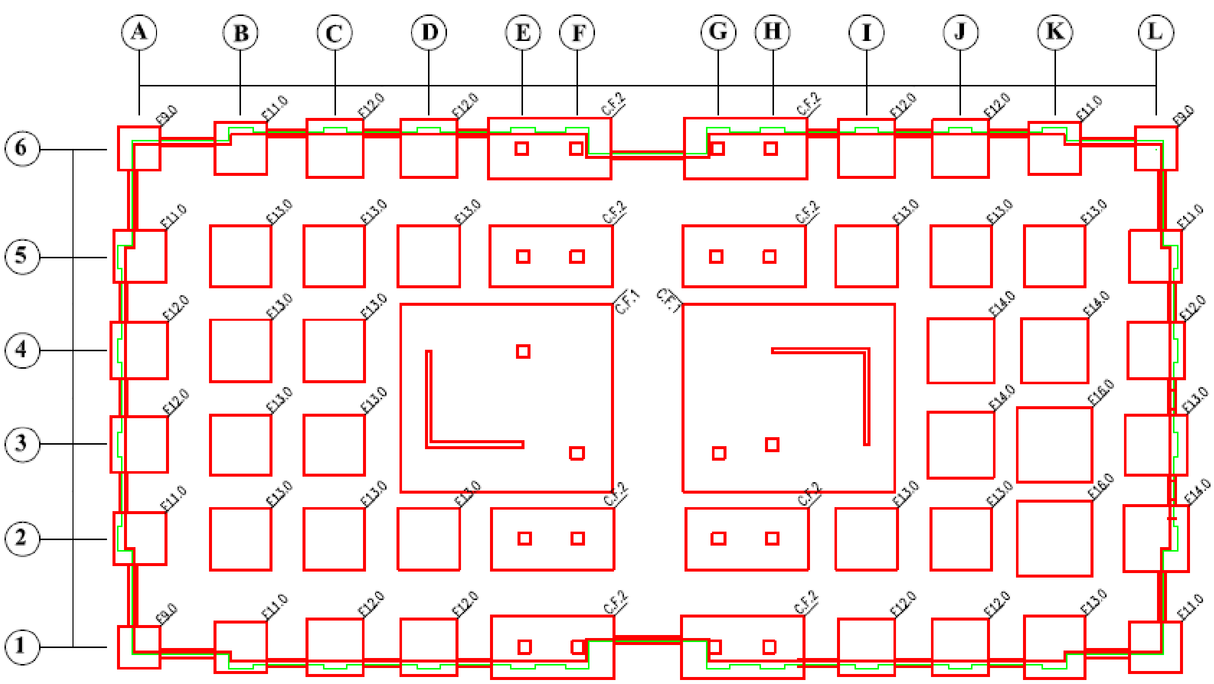


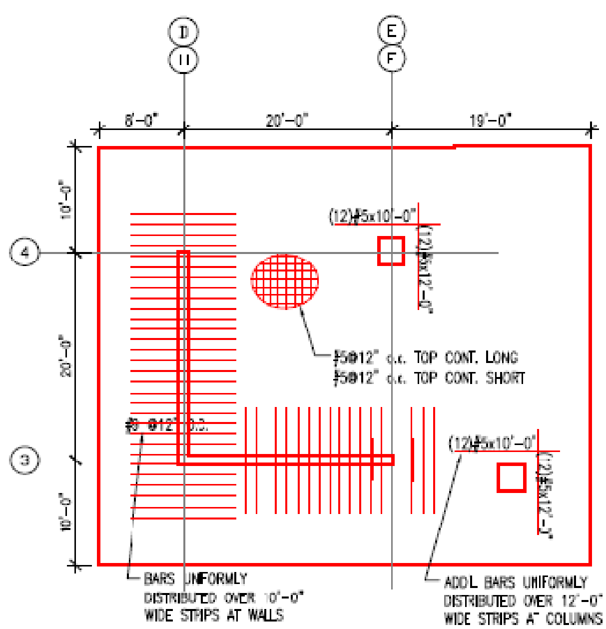
Figure Insert Number-Foundation Flan

In the above figure, the footing designs are specified with identification in the upper right corner of the footing. For the gravity system footings, please refer to the table below for the design details. The combined footings are detailed on the two pages of this report.

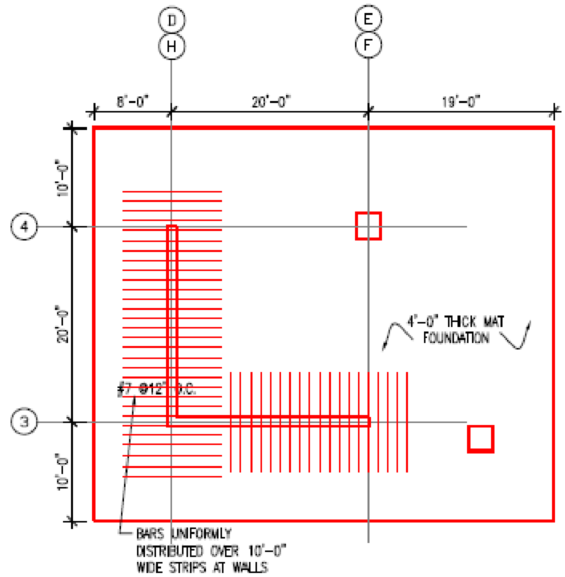
Designation	Size	Depth	Bot. Reinforcing
F 9.0	9' x 9'	24"	11-#6 E.W.
F 10.0	10' x 10'	24"	9-#7 E.W.
F 11.0	11' x 11'	30"	12-#7 E.W.
F 12.0	12' x 12'	30"	13-#7 E.W.
F 13.0	13' x 13'	36"	17-#7 E.W.
F 14.0	14' x 14'	36"	11-#9 E.W.
F 15.0	15' x 15'	42"	23-#7 E.W.
F 16.0	16' x 16'	42"	23-#7 E.W.

Gravity Column Footing Schedule

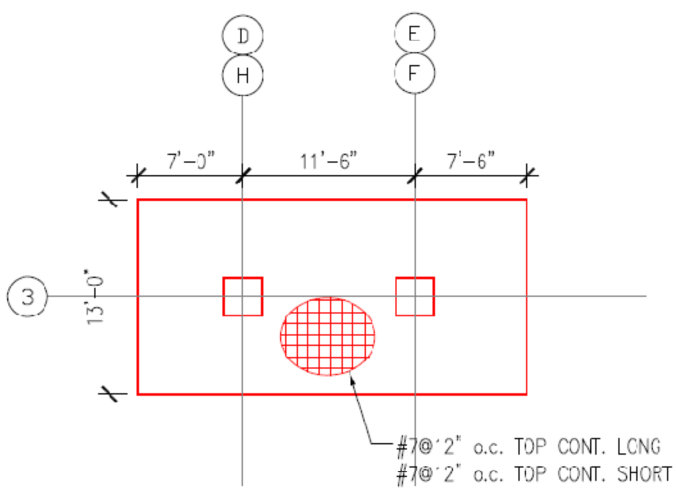
Detailed below is the shear wall combined footing design. The mat footing is detailed in two drawings, one for the bottom steel reinforcement, and one for the top steel reinforcement. The bottom steel plan has a basic mat of #5 bars spaced at 12" in each direction with additional steel added around the columns and under the shear walls. Due to the fact that the shear walls are the only elements with any substantial lift, top reinforcement is only needed under the walls. The column piers are shown to indicate the location of the two columns within the combined footing.



COMBINE FOOTING 1 – MAT FOUNDATION BOTTOM REINFORCEMENT



COMBINED FOOTING 1– MAT FOUNDATION TOP REINFORCEMENT



COMBINE FOOTING 2 – BOTTOM REINFORCEMENT

Analysis of the Concrete Structure

The Etabs serviceability model that was used to size the shear walls (refer to the design portion of the report for the details of the model) provided the results from the elastic analysis that are presented below. It should be noted here that the drifts determined from the Etabs model are based on the fundamental period of the building (which as noted in the seismic load section above, was larger than the upper limit of C_u x T_a specified in 12.8.2). This is acceptable per the allowance of section 12.8.6.2 that states simply that the drifts are allowed to be determined based on the fundamental period of the structure as determined through elastic analysis without the imposed upper limit. Additionally it should be noted that P-delta effects were included in the Etabs analysis, thus making the calculations of section 12.8.7 unnecessary.

To determine the controlling drift case for wind loading, the corner points at the upper levels were selected (these points were expected to produce the largest drift numbers because they are the greatest distance from the center of the rigid diaphragm) and the drift numbers for these points for all wind load cases were imported into an excel sheet that had been set up to search the output, and report the highest drift. Using this method it was found that the wind case based on the ACI serviceability case 5, with the wind loading in the Y direction (which was expected because that corresponds to the long side of the structure and the lateral stiffness is identical in the X and Y directions) produced the highest drift numbers. These numbers were than compared to the limit of L/400 which corresponds to the limit commonly used in practice.

The seismic story drift numbers were also taken from the Etabs model. To determine the controlling seismic drift case the same procedure used in the wind cases was used. This involved exporting the drift points at the center of mass of each diaphragm and importing them into an excel spreadsheet that was set up to search the data and report the highest drifts are each story and the load case that caused these drifts. The drifts at each story were taken at the center of mass as prescribed by section 12.8.6. The controlling drift numbers were then adjusted with the amplification factor and importance factor as specified in section 12.8.6.

The controlling drift cases along with the code allowable limits are shown in the tables found on the following page.

Wind Drifts

Drift Due to Wind			
Story	Height (ft)	Drift (in)	Allowable Drift (in)
PHRoof	117.9	0.8	3.537
PHFloor	108.4	0.76	3.252
ROOF	104.7	0.72	3.141
9	93.1	0.62	2.793
8	81.4	0.53	2.442
7	69.8	0.43	2.094
6	58.2	0.33	1.746
5	46.5	0.24	1.395
4	34.9	0.16	1.047
3	23.3	0.08	0.699
2	11.6	0.03	0.348

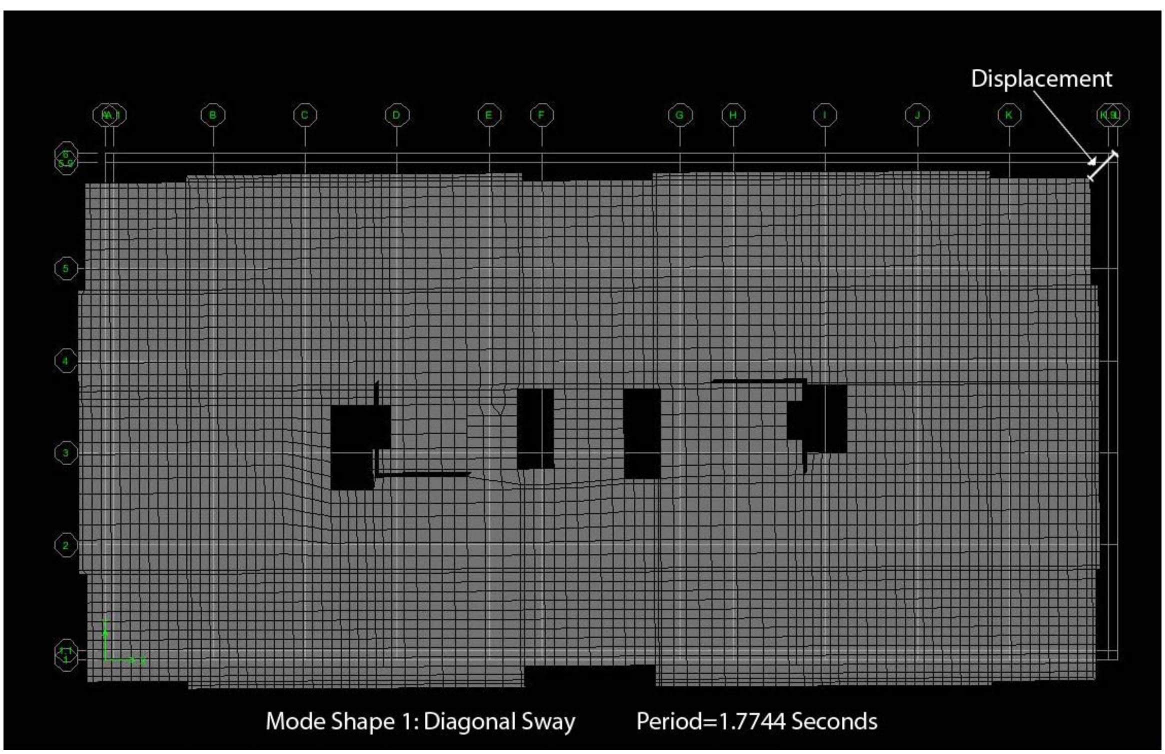
Seismic Drifts

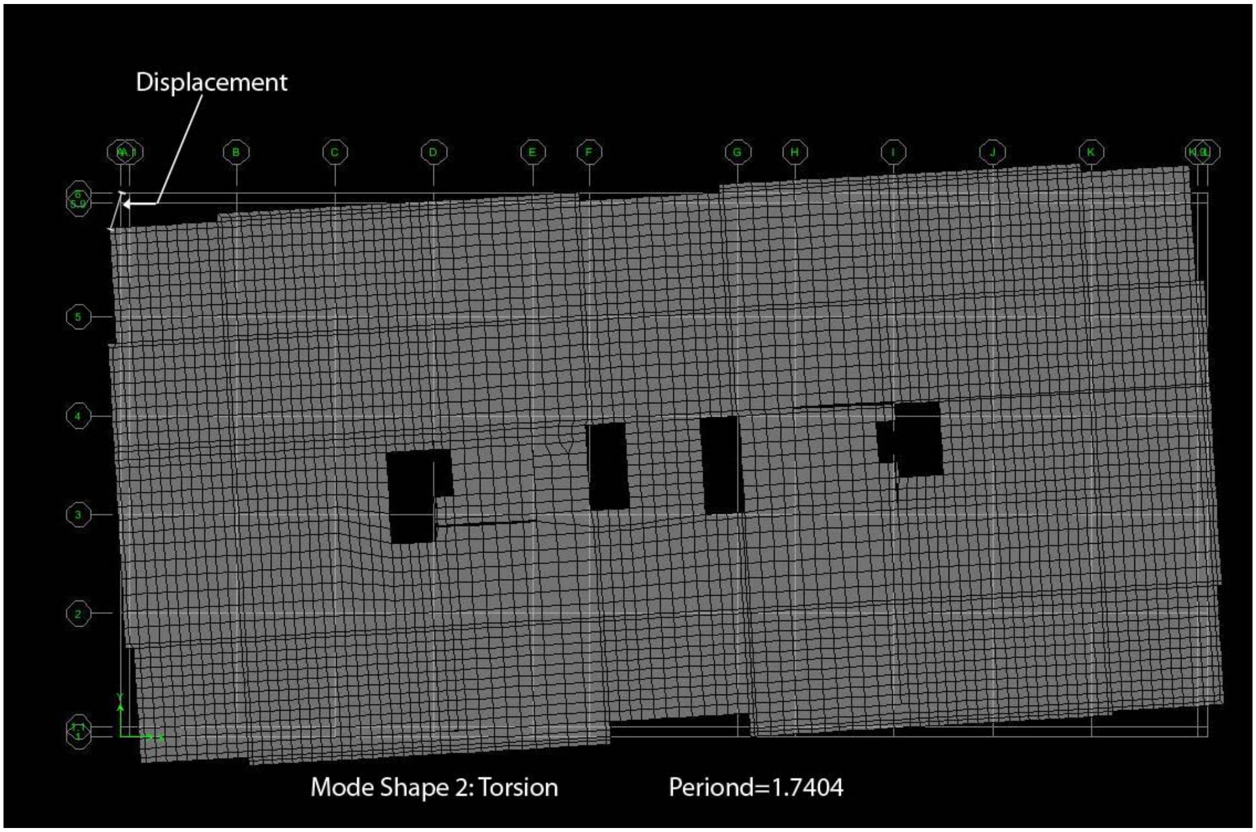
Drift Due to Seismic Forces					
Story	Height (ft)	Total Drift (in)	Amplified Drift (in)	Story Drift (in)	Allowable Drift(in)
PHRoof	117.9	0.53	2.39	0.27	2.28
PHFloor	108.4	0.47	2.12	0.09	0.888
Roof	104.7	0.45	2.03	0.27	2.784
9th	93.1	0.39	1.76	0.32	2.808
8th	81.4	0.32	1.44	0.27	2.784
7th	69.8	0.26	1.17	0.32	2.784
6th	58.2	0.19	0.86	0.23	2.808
5th	46.5	0.14	0.63	0.23	2.784
4th	34.9	0.09	0.41	0.23	2.784
3rd	23.3	0.04	0.18	0.14	2.808
2nd	11.6	0.01	0.05	0.05	2.784

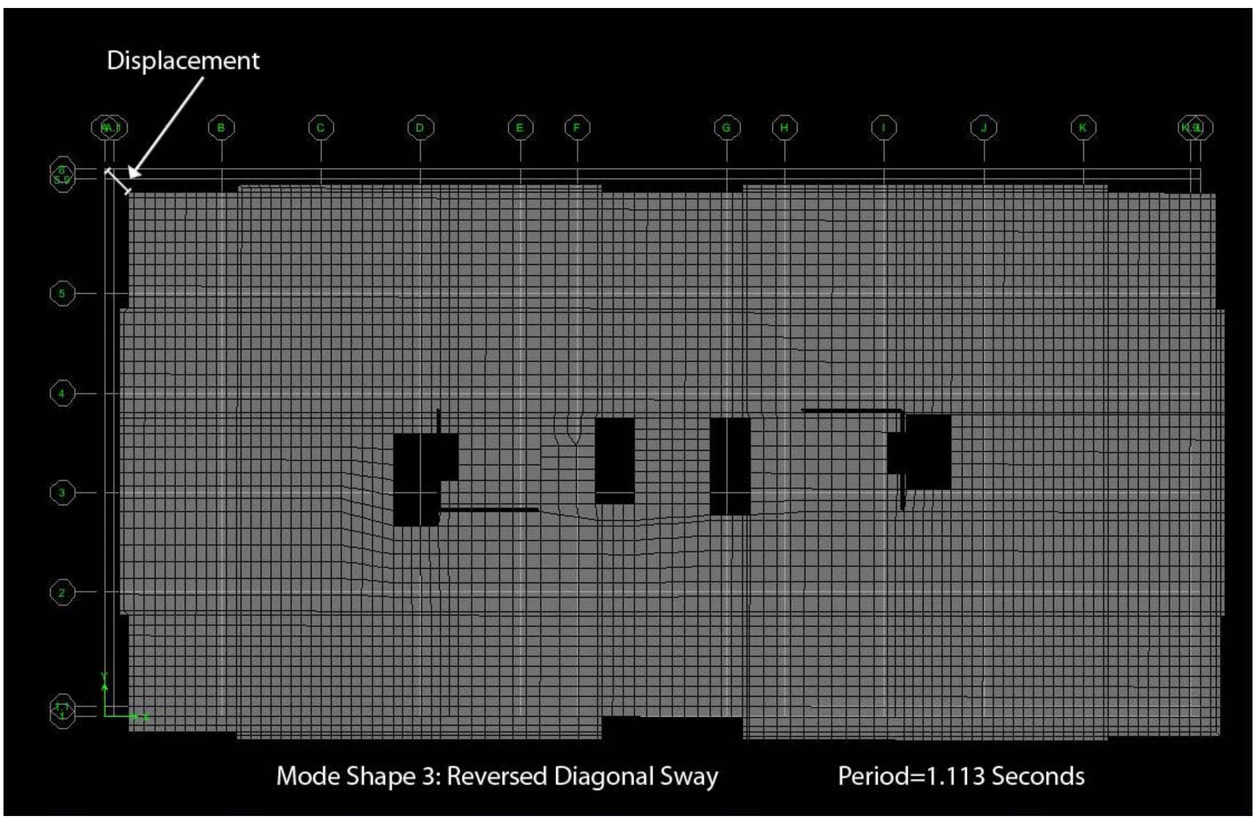
Building Modes and Fundamental Periods

The elastic analysis done with the help of Etabs calculated the first twelve modal responses of the building under lateral loads. Each response is defined by a displacement type and the period of vibration that causes such a displacement. This information is helpful because it can be used to predict how the structure will respond to a particular loading scenario. Additionally the modal responses can help to show inherent weakness in the lateral system's ability to resist a certain type of loading. The response type with the largest period of vibration shows how the building is most likely to deform and the weakest response of the structural system. The first three mode shapes are presented in this section. The discussion is limited to the first three because they give the greatest displacements, and further more the other responses are simply copies of these three with smaller periods and displacements.

The first and second mode shapes occur at almost the exact same period. The first mode, which is characterized by diagonal sway of the building along a line passing through the North-East and South-West corners of the building. The period of vibration that induces this type of reaction is 1.7744 seconds. The second mode shape is a twisting of the building around the center of rigidity. This was expected to be one of the first modes due to the fact that the shear walls are located relatively close to the center of the building, thus providing minimal torsional rigidity. This mode shape occurs at a period of 1.7404 seconds. The third shape and the final one that will be discussed is the reversal of the first mode shape. It is a diagonal sway displacement along a line passing through the North-West and South-East corners of the building. This mode shape occurs at a period of 1.113 seconds. All three of these modal reactions are shown visually below with a displacement profile of the ninth floor diaphragm.




Displacement
Mode Shape 2: Torsion
Periond=1.7404


Displacement
Mode Shape 3: Reversed Diagonal Sway
Period=1.113 Seconds