Technical Report Three

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AE 481W-Senior Thesis
The Pennsylvania State University

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Executive Summary

Building Description:

The Washingtonian Center is an eight story office building located in Gaithersburg Maryland. The building is approximately 106.5 feet to the roof level, while the top of the mechanical penthouse rises to 120'. The structure of the building is a steel frame. The gravity framing consists of composite steel deck and beams with a lightweight concrete topping while the lateral forces are resisted by two concentrically braced frames located around the core of the building.

Report Summary:

The purpose of this report was to verify that the lateral system in the Washingtonian Center is adequate from both a strength and serviceability stand point. This analysis was aided greatly by the use of ETABs to develop and to verify the loading on the building as well as the drift and member forces in the braced frames. Two separate ETABs models were developed, one with the serviceability load combinations and one with the strength combinations. The service model was used to check the drift of the structure while the strength model was used to find the forces in the braced frame resulting from the strength load combinations. In both the models the load parameters were entered into the program and it was allowed to generate the lateral forces itself. These forces were then checked by the lateral forces that were calculated by hand using the procedures of ASCE 7-05. The drifts obtained from the service model were then checked using doing a simplified drift check by hand using the loads that had been calculated manually. The strength of the frames were verified by taking the member forces from the ETABs analysis and using the AISC Steel Construction Manual to verify that the members were capable of taking the stresses.

Introduction

Building Description:

The Washingtonian Center is planned to be constructed on a previously undeveloped site just off of interstate 270 in Gaithersburg Maryland. Planned occupancy for the building is office space. To reflect this use the building is being designed as an envelope and core structure to allow for maximum flexibility of the tenant space. The building design consists of eight floors with a mechanical penthouse on the roof. The building rises 106.5 feet to the roof top, with the penthouse roof extending to 120 feet.

The general 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 2^{nd} – 8^{th} 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.

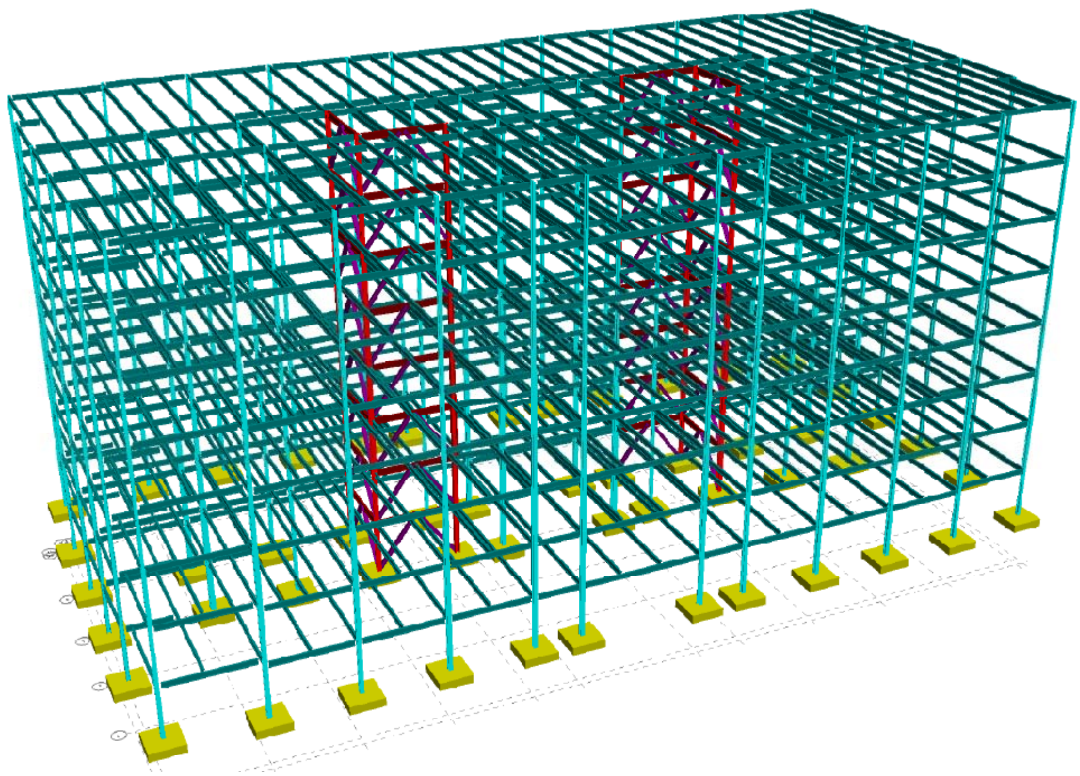
The steel deck floor system is supported on W21x44 beams spaced every 10’ and spanning a distance of 45’ on the exterior bays. The interior bays are supported by W14x22 spaced every 10’ and spanning a distance of 20’. The girders supporting these beams are typically W14x22 spanning 20’.

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”.

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 span in both directions for a distance of 20’. 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.

Current Framing Layout

Below is representation of the steel framing layout that was described in the preceding section. Note that the gravity framing is represented by the teal colored members while the lateral frame is illustrated with the red and purple member.



Report Topics:

- Load Calculations and Load Cases
- Load Distribution Discussion
- Building Lateral Analysis
    - Drift
    - Strength
- Lateral Frame Hand Checks
    - Drift
    - Strength
- Conclusions

Loads and Load Cases

Introduction:

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 designer for 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 and have been used for the purposed of this report as well. The lateral loads were calculated by hand 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

Lateral Loads Discussion:

The wind and seismic loads presented here were calculated with the aid of spread sheets, based on ASCE 7-05. The results show that the base shear and over-turning moment in the two directions of the building are controlled by different factors. Wind pressures on the long side of the building resulted in a total base shear of 415 kips, however wind along the short length of the building resulted in just 181 kips. The seismic forces induce a base shear of 355 kips that applies regardless of building orientation. This means that the lateral system will need to consider both wind and seismic forces in the design. Each member of the braced frame will need to provide adequate strength to resist the highest load that could be applied to it. This doesn’t mean that the framing in the Y direction will be checked for wind and the framing in the X will be checked for seismic because the frames in the two directions are connected, therefore the highest forces applied to a member may not come from the lateral loads in the direction of the members span.

Wind Loads:

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

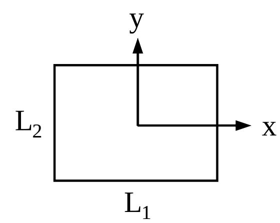
For Buildings of All Heights

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	L/B	Leeward	External wall pressure			w/ pos. internal pressure			w/ neg. internal pressure			Total	Story
z (ft)		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)
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)
<u>Output</u>		
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	
<u>Base Shear</u>		***<u>Over-Turning Moment</u>***
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

Load Distribution

Introduction

For the purposes of this report, it was assumed that the composite steel floor system would act as a rigid diaphragm, and would perfectly distribute lateral forces to the braced frames based on the relative stiffness of each frame compared to the total stiffness of the entire lateral system. The Washingtonian Center is perfectly symmetrical about both axis's of the building in both geometry and in stiffness because the frames are at the same location each way and are composed of exactly the same members. This means that in this report, it was assumed that exactly one half of the lateral forces were distributed into each of the two frames in each direction. These assumptions are justified because the composite floor is generally stiff enough for the rigid diaphragm model to provide results that are close to the actual force distribution. Additionally there are no significant obstructions such as slab openings near the braced frames to cause a different distribution of forces.

Presented here is a comparison of the wind and seismic load distributions that were calculated by hand and taken from ETABs. The wind and seismic parameters were entered into ETABs and program was permitted to calculate the loads and the distribution.

Lateral Loads in the X Direction

Below is a chart comparing the wind and seismic loads taken from Etabs to the loads that were calculated by hand. It is clear that the winds loads are nearly identical, with the two total shears being within 4% of each other. This verifies the wind loads are have been correctly calculated. The Seismic forces differ by a much more, nearly 20%. This could be attributed to the fact that the building was modeled with only the floor slab and lateral frame in Etabs. This neglected the weight of the façade, and interior framing that was accounted for in the hand calculations. Clearly this would have a significant impact on the calculation of the loads and would need to be accounted for in a more accurate model if Etabs was going to be used for the design of the building. What is evident from this comparison is that seismic forces do control in the X direction of the building.

Story Shear Comparison in the X Direction				
	Wind		Seismic	
	Shear (k)		Shear (k)	
Story	Etabs	Hand Calculations	Etabs	Hand Calculations
Roof	14.22	18	21.34	65.41
8	28	25	41.99	78.99
7	27.26	23	40.89	63.95
6	26.44	23	39.66	49.82
5	25.47	22	38.21	36.69
4	24.33	20	36.49	24.75
3	22.84	19	34.26	14.2
2	21.35	17	32.03	5.49
Total:	189.91	182	284.87	354.81

Lateral Loads in the Y Direction

For the lateral force in this direction much of the same discussion above applies here as well. Again the wind forces are quite close and provide reasonable answer while the seismic forces are off significantly (differing by 8.5% and 20% respectively). In this case wind clearly controls the design with significantly higher forces.

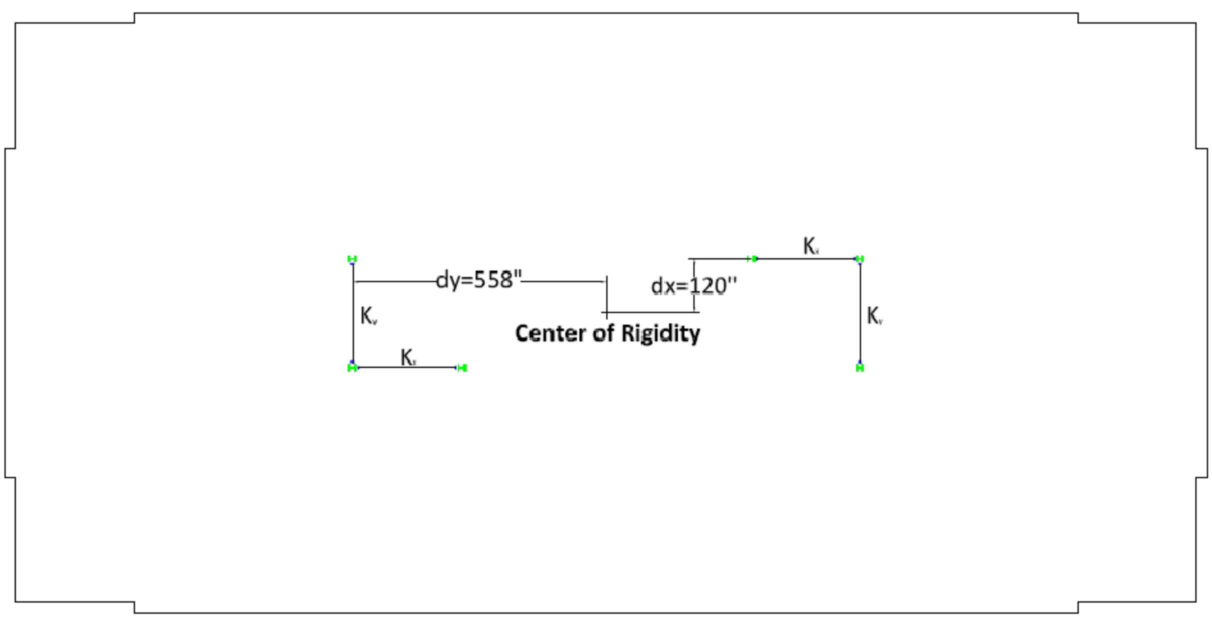
Story Shear Comparison in the Y Direction				
	Wind		Seismic	
	Shear (k)		Shear (k)	
Story	Etabs	Hand Calculations	Etabs	Hand Calculations
Roof	28.45	42	21.34	65.41
8	55.99	58	41.99	78.99
7	54.52	53	40.89	63.95
6	52.87	53	39.66	49.82
5	50.96	51	38.21	36.69
4	48.65	47	36.49	24.75
3	45.68	46	34.26	14.2
2	42.71	42	32.03	5.49
Total:	379.83	415	284.87	354.81

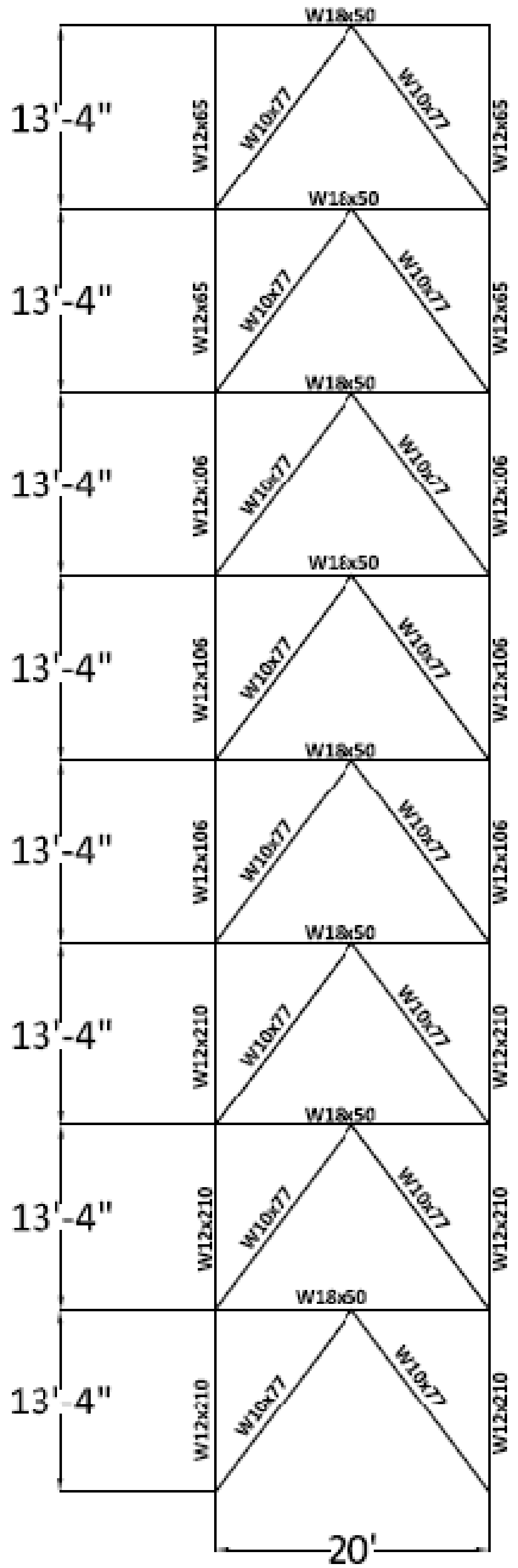
Analysis

Introduction

This portion of the report will compare drifts that were calculated by the Etabs serviceability model, to drift estimations done by a simplified hand procedure. This was complicated by the fact the Washingtonian Center is very susceptible to torsional drift. The two braced frames in the building are both located near the center of the building around the central elevator core. This limits the lateral rigidity of the building around the perimeter and makes it twist under lateral loads applied at an eccentricity. The Etabs analysis confirmed that the buildings first modal displacement is a torsional case.

Shown below is depiction of the floor slab and braced frame layout in the building. Note that the center of rigidity is assumed to be at the geometric center of the slab because slab penetrations have been ignored for the purposed of this report. Additionally shown is the lateral stiffness for the floor in each direction and the distance from each frame to the center of rigidity. These values are needed in the calculation of the floors torsional rigidity.





Depicted to the left is an elevation of one of the braced frames. Each of the four frames are identical, and the geometric layout of the frames is needed to find the lateral stiffness estimate at each floor. It should be noted here that in the calculations below, the two braces on each floor are assumed to have the same geometry. This isn't entirely true, however that assumption doesn't create any inaccuracies in the axial stiffness calculation because one brace would be in tension and one would be in compression, however they contribute an equal amount of axial stiffness, therefore it doesn't matter if the stiffness is compressive or tensile and the assumption that they both act in tension at an angle of 53 degrees gives an accurate lateral stiffness.

Drift Estimation

To estimate the drift, it was assumed that only the diagonal braces in the frame provide any lateral stiffness. The contribution of the columns and beams in the frame were ignored. This should result in a drift that is higher than what Etabs calculates. First the axial stiffness of the braces were calculated and then converted from local coordinates to global coordinates considering only the portion of the axial stiffness the correlated to the X or Y directions of the building respectively (the vertical component of the stiffness was ignored because the forces are acting laterally). Next the lateral stiffness of each brace at each floor level was converted to an equivalent torsional rigidity of the floor diaphragm. The moment applied to each floor from the wind loading (using the hand calculated loads) was then calculated using three quarters of the total wind force applied at a 15% eccentricity. Using the property related rotational stiffness, the applied moment, and the resulting rotation, the angle of rotation of each diaphragm was calculated. These rotations were summed over all the floors to obtain the total rotation of the upper diaphragm. Finally the drift was calculated using the length from the center of rigidity to the corner of the diaphragm as the arm over which the rotation occurred. Using small angle principles the drift was found.

Procedure

Step 1: Estimate the lateral stiffness of a single frame at each floor. In this case, each floor will have the same lateral stiffness because only the braces are being considered and all floor of the frame have braces of W10x77. There are two braces per floor with an area of 22.6 square inches, and a length of 200 inches. Each member provides axial stiffness at an angle of 53.13 degrees with respect to the horizontal.

$$K = n\frac{\sum \mathrm{AE}}{L}\cos^2\alpha = 2\,\frac{[(22.6)(29000ksi)]}{200}\cos^2(53.13) = 2359.45\,\frac{K}{in}$$

Step 2: On each floor, the lateral stiffness of the four frames work together to provide torsional stiffness against moments applied to the diaphragm from eccentric loads. The lateral stiffness, needs to be converted to torsional stiffness for the floor diaphragm.

$$J = \sum K_i d^2 = 2K_x d_x^2 + 2K_y d_y^2 = 2(2359.45)(120^2) + 2(2359.45)(558^2) = 76{,}862{,}369.8 \text{ k-in}$$

Step 3: The moments applied to each floor by the eccentric wind load needs to be calculated. This step is shown in the table below. Additionally shown on the table is the rotation of each floor from the moment applied to it. This rotation is calculated by dividing moment by the torsional stiffness of the floor. Finally the deflection on the top floor is calculated by summing the rotations of each floor below it and multiplying by the distance to the corner of the floor slab.

$$M = J\Phi$$

$$\Delta = \Phi\ell$$

Hand Calculated Wind Drift							
Story	Force (K)	0.75*Force	Width (ft)	Eccentricity (ft)	Moment (k-in)	Torsional Rigidity (k-in)	Rotation (rad)
Roof	42	31.5	220	33	12474	768,623,869.80	0.000162
8th	58	43.5	220	33	17226	768,623,869.80	0.000224
7th	53	39.75	220	33	15741	768,623,869.80	0.000205
6th	53	39.75	220	33	15741	768,623,869.80	0.000205
5th	51	38.25	220	33	15147	768,623,869.80	0.000197
4th	47	35.25	220	33	13959	768,623,869.80	0.000182
3rd	46	34.5	220	33	13662	768,623,869.80	0.000178
2nd	42	31.5	220	33	12474	492,663,600.00	0.000253
						Total Rotation=	0.001606
						Largest Drift=	3.31739

Etabs Drift

The story drifts taken from the Etabs Service model are presented below along with the allowable drifts. 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. It can clearly be seen that the current lateral frame provides adequate strength to limit the buildings movement under lateral loading to acceptable limits.

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

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

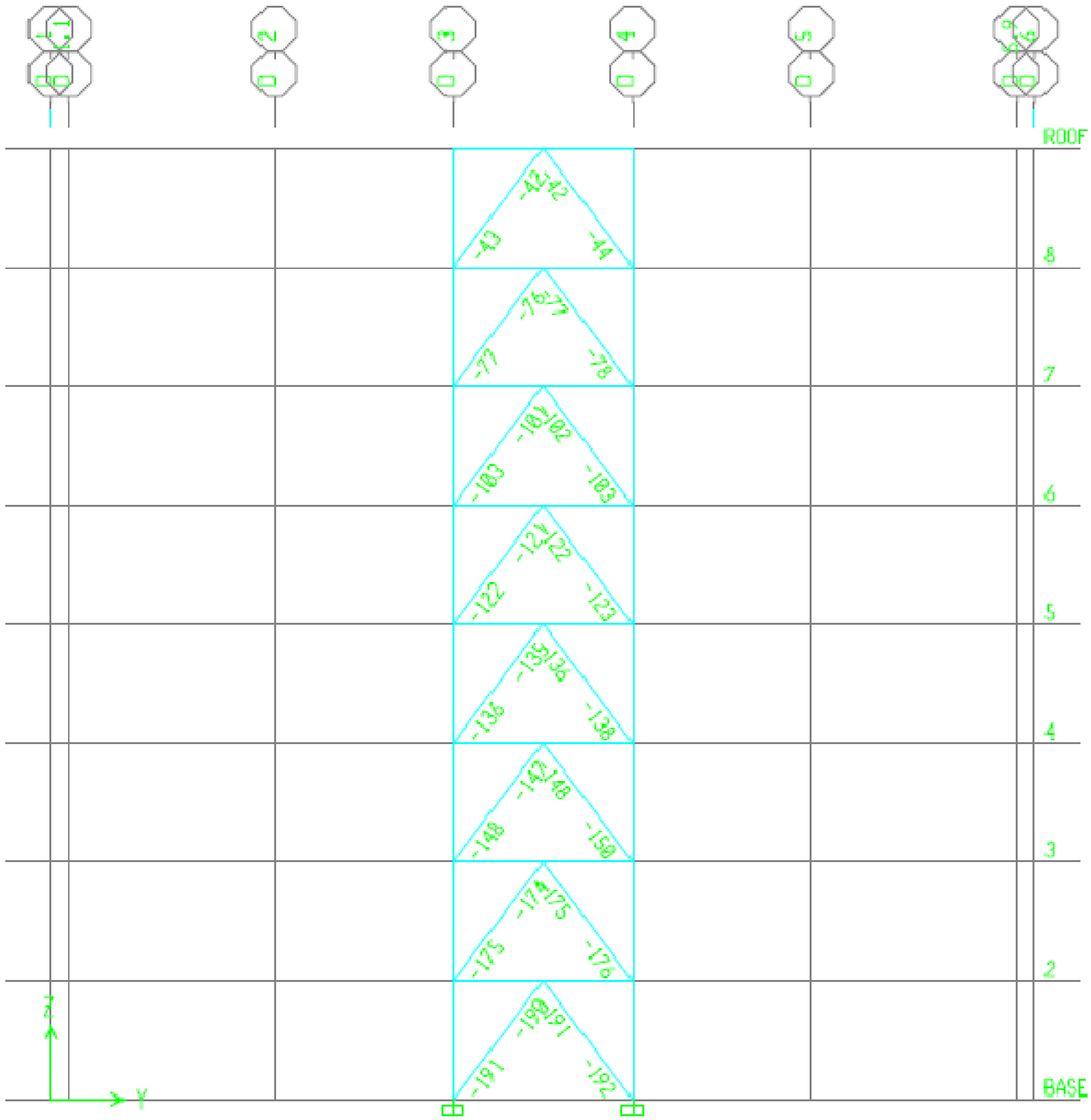
Member Checks

Introduction

To check the strength of the frame members, the forces were taken from the Etabs model that used the strength load combinations list above. Each member's strength was checked against the highest loads sustained in that type of member from all possible load combinations and load directions. For example, the braces are all W10x77's, therefore the highest forces seen by any of these members is what every member was checked against. The checks were done using the ASIC Steel Construction Manual, 13^{th} Edition, using load factored resistance design. Additionally this report assumes that the lateral frames in the Washingtonian Center resister forces from lateral loads only. This doesn't entirely describe the behavior of the building because the breams in the frame also have part of the floor framing into them and therefore take some gravity loading as well. The building was modeled in Etabs with only the lateral frame and a diaphragm extending to the extents of the building that had no properties. A seismic mass was assigned to this diaphragm to estimate the floor weights when calculating seismic loads. This means that the in frame as modeled truly did take only later forces.

Braces Check

The braces are connected at each end with a shear connection. This affectively makes them axial members only, capable of providing lateral resistance in compression or tension, depending on the direction of the loading. As expected the braces spanning the Y direction of the building were exposed to the highest axial forces. Clearly the braces located at the bottom floor should have the highest forces in them, and this was the case. The highest axial load in the braces was 192 kips, therefore the W10x77 member's capacity was checked for 192 kips in axial tension, and compression because of the possibility of load reversal. The un-braced length used for the compression case is the entire length of the member, in this case 16'-8". For the axial tension case, failure modes of both yielding and rupture were considered.



The figure shown above depicts the lateral frame located at gridline D. This frame is shown because it has the highest forces in it. It should be noted that the frame located at gridline I also experiences identical loads.

Table 5-1 (continued)
Available Strength in Axial Tension
W Shapes

W12–W10

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W12×58	17.0	12.8	509	765	416	624
×53	15.6	11.7	467	702	380	570
W12×50	14.6	10.9	437	657	354	531
×45	13.1	9.83	392	590	319	479
×40	11.7	8.78	350	527	285	428
W12×35	10.3	7.73	308	463	251	377
×30	8.79	6.59	263	396	214	321
×26	7.65	5.74	229	344	187	280
W12×22	6.48	4.86	194	292	158	237
×19	5.57	4.18	167	251	136	204
×16	4.71	3.53	141	212	115	172
×14	4.16	3.12	125	187	101	152
W10×112	32.9	24.7	985	1480	803	1200
×100	29.4	22.0	880	1320	715	1070
×88	25.9	19.4	775	1170	631	946
×77	22.6	17.0	677	1020	553	829
×68	20.0	15.0	599	900	488	731

This figure was taken from the AISC Steel Construction manual. It is a table complied to show the axial tensile strengths of various shapes. The highlighted values on the table show capacities of a W10x77 for failures in both yielding of the material, and material rupture.

Yielding Check:

$$\Phi P_n = 1020\ kips\ \geq\ P_U = 192\ kips$$

Rupture Check:

$$\Phi P_n = 829\ kips\ \geq\ P_U = 192\ kips$$

Table 4–1 (continued)

Available Strength in Axial Compression, kips

W Shapes

F_y = 50 ksi

W10

Shape	W10×											
Wt/ft	112		100		88		77		68		60	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	986	1480	880	1320	775	1170	678	1020	598	899	528	794
6	936	1410	834	1250	734	1100	641	963	565	850	499	750
7	918	1380	818	1230	720	1080	628	944	554	833	489	734
8	898	1350	800	1200	703	1060	614	922	541	813	477	717
9	876	1320	780	1170	685	1030	598	898	527	792	464	698
10	852	1280	758	1140	666	1000	580	872	511	768	450	677
11	826	1240	734	1100	645	969	561	844	495	744	436	655
12	799	1200	709	1070	623	936	542	814	477	717	420	631
13	770	1160	683	1030	600	901	521	783	459	690	404	606
14	740	1110	656	986	575	865	500	751	440	661	387	581
15	709	1070	628	944	551	827	477	718	420	632	369	555
16	678	1020	600	901	525	789	455	684	400	602	351	528
17	646	970	571	858	499	751	432	650	380	571	333	501
18	613	922	542	814	474	712	409	615	360	541	315	474

...spect to least radius of gyration r_y

This figure was taken from the AISC Steel Construction manual. It is a table complied to show the axial compressive strengths of various shapes. The highlighted values on the table show capacities of a W10x77.

Check:

$$\Phi P_n = 650\ kips\ \geq\ P_U = 192\ kips$$

Column Check

For the purposes of this report, it is assumed that the columns act as axial members only. This isn't entirely representative of how the frames truly function but it gives a close approximation. For the columns to truly be only axial members, they would have to be splice at each floor level. In this case the columns are spliced at the fourth and seventh levels only. This means that bending moments will be introduced into the columns, however these will be ignored. The column forces presented in this section represent the highest loads applied to each column section, anywhere in the four braced frames. The loads presented show the highest compressive forces experienced by the columns along with the highest tensile force. The checks included account for both loading situations. The un-braced length used for the compression check was taken as the floor to floor heights, which is 13'-4".

Frame Column Forces		
	$P_{compressive}$	$P_{tensile}$
W12x210	839	-895
W12x106	275	-303
W12x65	30	-37

W12x210 Column in Tension

Yielding Check:

$$\Phi P_n = 2780\ kips \geq P_U = 895\ kips$$

Rupture Check:

$$\Phi P_n = 2260 kips \geq P_U = 839 kips$$

W12x106 Column in Tension

Yielding Check:

$$\Phi P_n = 1400\ kips \geq P_U = 303\ kips$$

Rupture Check:

$$\Phi P_n = 1140\ kips \geq P_U = 303\ kips$$

W12x65 Column in Tension

Yielding Check:

$$\Phi P_n = 860\ kips \ \geq \ P_U = 37 kips$$

Rupture Check:

$$\Phi P_n = 697 kips \ \geq \ P_U = 37\ kips$$

The values used above came from the AISC Manual of Steel Construction table reproduced below.

F_y = 50 ksi
F_u = 65 ksi

Table 5–1 (continued)
Available Strength in Axial Tension
W Shapes

W14–W12

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W14×132	38.8	29.1	1160	1750	946	1420
×120	35.3	26.5	1060	1590	861	1290
×109	32.0	24.0	958	1440	780	1170
×99	29.1	21.8	871	1310	709	1060
×90	26.5	19.9	793	1190	647	970
W14×82	24.0	18.0	719	1080	585	878
×74	21.8	16.4	653	981	533	800
×68	20.0	15.0	599	900	488	731
×61	17.9	13.4	536	805	436	653
W14×53	15.6	11.7	467	702	380	570
×48	14.1	10.6	422	634	345	517
×43	12.6	9.45	377	567	307	461
W14×38	11.2	8.40	335	504	273	410
×34	10.0	7.50	299	450	244	366
×30	8.85	6.64	265	398	216	324
W14×26	7.69	5.77	230	346	188	281
×22	6.49	4.87	194	292	158	237
W12×336	98.8	74.1	2960	4450	2410	3610
×305h	89.6	67.2	2680	4030	2180	3280
×279h	81.9	61.4	2450	3690	2000	2990
×252h	74.0	55.5	2220	3330	1800	2710
×230h	67.7	50.8	2030	[illegible]	1650	[illegible]
×210h	61.8	46.3	1850	2780	1500	2260
×190	55.8	41.9	1670	2510	1360	2040
×170	50.0	37.5	1500	2250	1220	1830
×152	44.7	33.5	1340	2010	1090	1630
×136	39.9	29.9	1190	1800	972	1460
×120	35.3	26.5	1060	1590	861	[illegible]
×106	31.2	23.4	934	1400	761	1140
×96	28.2	21.2	844	1270	689	1030
×87	25.6	19.2	766	1150	624	936
×79	23.2	17.4	695	1040	566	848
×72	21.1	15.8	632	949	514	770
×65	19.1	14.3	572	860	465	697

W12x210 Column in Compression

Check:

$$\Phi P_n = 2290\ kips \ \geq \ P_U = 839 kips$$

W12x106 Column in Compression

Check:

$$\Phi P_n = 1130\ kips \ \geq \ P_U = 275\ kips$$

W12x65 Column in Compression

Check:

$$\Phi P_n = 685\ kips \ \geq \ P_U = 30\ kips$$

F_y = 50 ksi

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W12

Shape		W12×											
Wt/ft		336[h]		305[h]		279[h]		252[h]		230[h]		210	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
east radius of gyration r_y	0	2960	4440	2680	4030	2450	3690	2220	3330	2030	3050	1850	2780
	6	2870	4310	2600	3910	2370	3570	2140	3220	1960	2940	1790	2680
	7	2830	4260	2570	3860	2340	3520	2120	3180	1930	2910	1760	2650
	8	2800	4200	2530	3810	2310	3470	2090	3140	1910	2870	1740	2610
	9	2760	4140	2500	3750	2280	3420	2050	3090	1880	2820	1710	2570
	10	2710	4070	2450	3690	2240	3360	2020	3030	1840	2770	1680	2520
	11	2660	4000	2410	3620	2190	3300	1980	2970	1800	2710	1640	2470
	12	2610	3920	2360	3540	2150	3230	1940	2910	1770	2650	1610	2410
	13	2550	3830	2310	3460	2100	3150	1890	2840	1720	2590	1570	2360
	14	2490	3740	2250	3380	2050	3080	1840	2770	1680	2520	1530	2290
	15	2430	3650	2190	3290	1990	3000	1790	2690	1630	2450	1480	2230

W12

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

Shape		W12×											
Wt/ft		190		170		152		136		120		106	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
…o least radius of gyration r_y	0	1670	2510	1500	2250	1340	2010	1200	1800	1060	1590	933	1400
	6	1610	2420	1440	2170	1290	1940	1150	1730	1020	1530	897	1350
	7	1590	2390	1420	2140	1270	1910	1140	1710	1000	1510	884	1330
	8	1570	2360	1400	2110	1250	1080	1120	1680	986	1480	870	1310
	9	1540	2320	1380	2070	1230	1850	1100	1650	968	1460	854	1280
	10	1510	2270	1350	2030	1210	1820	1080	1620	949	1430	837	1260
	11	1480	2230	1320	1990	1180	1780	1050	1580	927	1390	818	1230
	12	1450	2180	1290	1940	1150	1730	1030	1540	905	1360	798	1200
	13	1410	2120	1260	1900	1120	1690	1000	1500	881	1320	776	[illegible]
	14	1380	2070	1230	1840	1090	1640	972	1460	856	1290	754	1130
	15	1340	2010	1190	1790	1060	1600	943	1420	829	1250	730	1100

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

W12

Shape		W12×									
Wt/ft		96		87		79		72		65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
…least radius of gyration r_y	0	844	1270	766	1150	694	1040	633	951	571	859
	6	811	1220	735	1110	667	1000	607	913	548	824
	7	800	1200	725	1090	657	987	598	899	540	811
	8	787	1180	713	1070	646	971	588	884	531	798
	9	772	1160	699	1050	634	952	577	867	520	782
	10	756	1140	685	1030	620	932	565	849	509	765
	11	739	1110	669	1010	606	910	551	828	497	747
	12	720	1080	652	960	590	887	537	807	484	727
	13	701	1050	634	953	573	862	522	784	470	706
	14	680	1020	615	924	556	836	506	761	456	685
	15	659	990	595	865	538	809	490	736	441	[illegible]

Conclusions

After conducting the analysis of the current lateral force resisting systems, a few things become obvious. The first and most important conclusion is that the current system is more than adequate to control the drift of the building and provide the required strength to resist the lateral forces on the building. The maximum drift occurring on the building was limited to 2.35” which is well within the h/400 limit of 3.2”. The actual drift was limited to about h/540. Additionally the seismic story drifts were all well within the code specified limits. From a strength stand point, even the members that were taking the highest loads in the frame weren’t stressed near their actual capacities. The logical conclusion that can be drawn here is that the frames were designed, not for strength but to provide adequate drift control without resorting to additional frames, or frames closer to the outsides of the building. This always for the expansive glass façade