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Erie Convention Center and Sheraton Hotel   Erie, Pennsylvania

Technical Report #3: Lateral System Analysis and Confirmation Design
Submittal Date: 21 November 2005

Executive Summary

The Erie Convention Center and Sheraton Hotel, located on the Presque Isle Bay in Erie, Pennsylvania, is an eleven story steel frame structure. Technical Report #3 focuses on the lateral system of the building, and its effectiveness to resist lateral wind and seismic loads. The lateral system for this hotel and convention center is located around the perimeter wall. There are braced frames that resist loads in the North/South direction, while the loads in the East/West direction are resisted by moment frames that span in the long direction of the building. The braced frames are comprised of both cross braces, and eccentric knee braces.

This report contains the calculations of the story forces for both wind and seismic loads, as well as the determination of controlling load combinations. A discussion of the transfer of loads, as well as an analysis is also included. Member checks for strength, drift, overturning, and torsional effects are included to compare to the output found by frame models completed using STAAD.

Through analysis, it was found that wind forces controlled in the North/South direction, therefore the load combination 1.2D +1.6W +L +0.5R governs. In the East/West direction, the load combination 1.2D + 1.0E + L + 0.2S controls because of the high seismic forces. The distribution of loads is taken as a simplification due to the symmetry of the building. Therefore, in the East/West direction, each moment frame resists half of the lateral load, while in the North/South direction, each braced frame resists half of the lateral load. From the member checks, it was found that critical members passed for strength, except for a few girders on the second floor, which had stresses that slightly exceeded the 50ksi strength capacity of steel. The over stressing could possibly be caused by incorrect assumptions made about the dead load of the exterior wall, and this impact on a large tributary width to the second floor members. The story drift as well as the overall building drift was found acceptable for the serviceability limitations of H/400 for the wind loads in both directions. However, in the East/West direction where seismic loads control, the drift limitations were exceeded. Overturning due to the lateral forces was determined to not be a problem, and torsional effects were found to be negligible as compared to the shear forces on the building.

Introduction

The proposed Erie Convention Center and Sheraton Hotel is located on West Dobbins Landing, along the Presque Isle Bay waterfront, in Erie, Pennsylvania. The 132,000 square feet Erie Convention Center and Sheraton Hotel rises eleven stories reaching a height of approximately 132 feet. Including mechanical penthouses and parapets, the height rises to a total of 155 feet. The first floor as well as half of the second floor, is dedicated to public spaces for the hotel and convention center, while the remaining nine and a half floors contain 200 private rooms. A pedestrian catwalk will connect the Erie Convention Center and Sheraton Hotel to the proposed Bayfront Convention Center, however this will not be included in the lateral analysis carried out in this report. Similarly, a parking garage is attached to the west wing of the hotel, but also will not be included in the lateral analysis.

The structural system is comprised of steel framing members that support 8-inch hollow core precast concrete planks that span in the East/West direction for the floor system. The design used for the Erie Convention Center and Sheraton Hotel specifies that no topping is needed except for a ¼" polymer modified cement product for leveling in select places. Grout placed in between the sections of the plank will allow the floor system to act as a diaphragm for the lateral analysis. The building is supported by a foundation system composed of caissons, monolithically cast concrete grade beams and piers, and an 8" structural slab on grade. The caissons have a minimum required diameter of 24", are drilled approximately 20 feet deep, and are made of 3000 psf concrete. By drilling three feet into the bedrock, the net allowable end bearing pressure is 40 ksf. In addition, shaft resistance can be added to the caisson capacity using 3.0 ksf allowable side friction applied to the socket surface area in the bedrock.

Laterally, the Erie Convention Center and Sheraton Hotel is restrained by four frames. In the East/West (E/W) direction, it is fully restrained by two moment frames, one located on the northern elevation, and one on the southern elevation (Figure 3.1). The structure is partially restrained by two braced frames in the North/South (N/S) direction (Figure 3.2). The diagonal bracing is comprised of A500, Grade 50 structural steel tubing of varying sizes and is connected to the frames with pin connections. As shown in Figure 3.3, the two exterior bays are restrained with cross bracing while the interior bay is restrained by eccentric knee braces. The framing girders are various W14 shapes ranging from W14×43 to W14×53, while the columns range from W14×90 to W14×370. In the N/S direction, the girders are sizes ranging from W16×40 to W18×55, while the columns vary between W14×74 and W14×342. All girders and columns are composed of A992, Grade 50 steel. The lateral system was chosen based on architectural characteristics and needs as well as the geometry of the building. The knee braces in the center bay in the N/S direction allow for the use of a vertical ribbon window. Diagonal bracing was also chosen to be used in this direction because of the large surface area that the wind acts on. Guest rooms line the exterior in the E/W direction, therefore moment connections were used to maximize the allowable space for windows. Also, the moment connections oppose wind loads acting on a smaller surface area.

This report includes the calculation and results of the wind and seismic story forces acting on the Erie Convention Center and Sheraton Hotel. From these forces, loading combinations were analyzed and evaluated to find the controlling case. Member checks were carried out to check for strength, as well as story drift, overturning moment, and torsional effects.

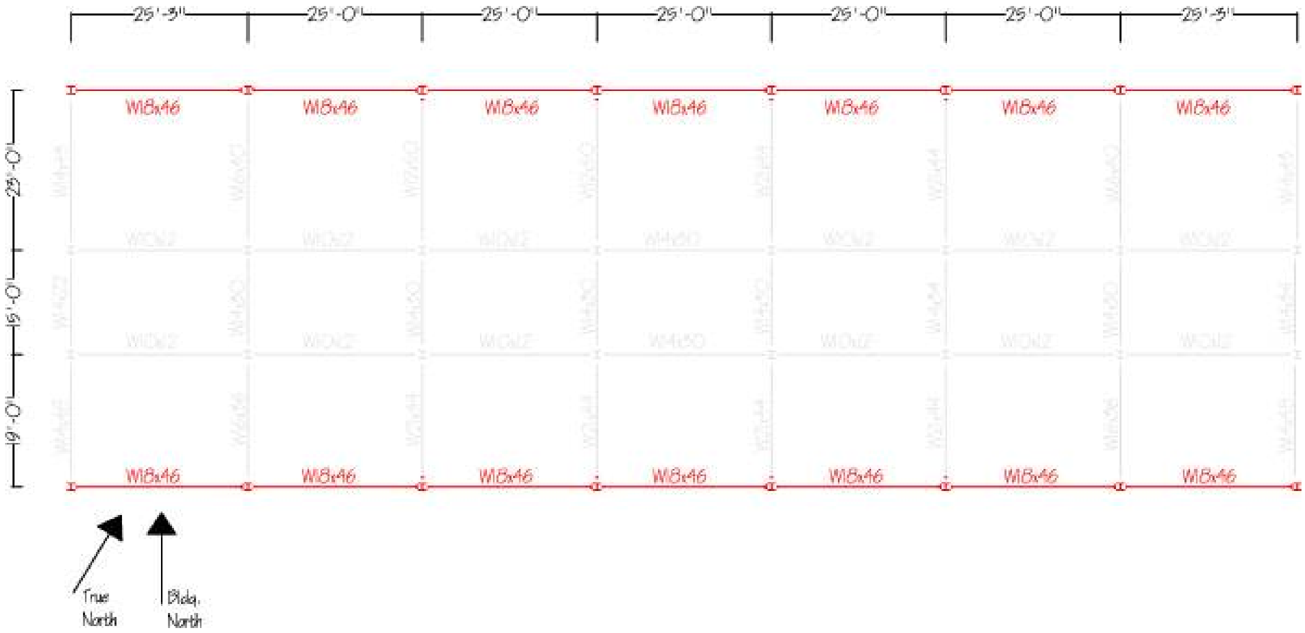


Figure 3.1
The red highlighted portions of the plan view above show the perimeter location of the moment frames.

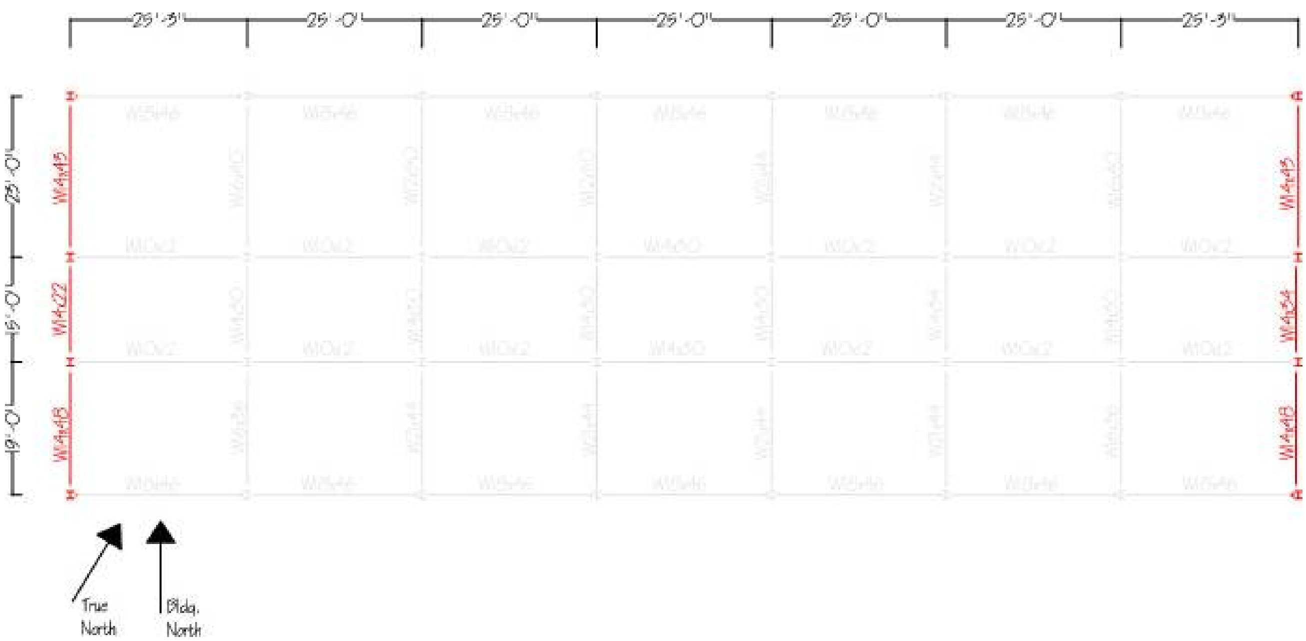
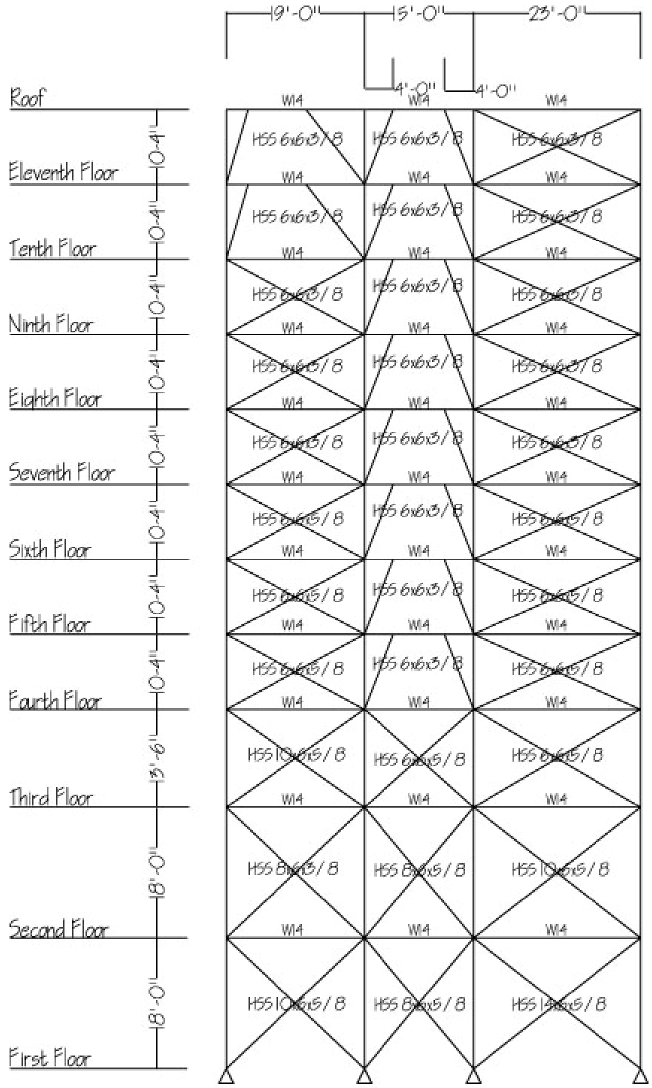


Figure 3.2
The red highlighted frames of the plan view above show the perimeter locations of the braced frames.



West Elevation

Figure 3.3
There are two braced frames that span in the N/S direction on the perimeter of the building. The sizes of the diagonal members are shown here.

Lateral Loads and Load Cases

Wind and seismic story forces were calculated by the procedures outlined in ASCE 7-02. Some assumptions were made to do this analysis. As prescribed by the engineer of record for the design of the Erie Convention Center and Sheraton Hotel, the Building/Occupancy type is to be type III. As seen in Section 1.5.1 of ASCE7-02, the occupancy type could be interpreted as Type II. To be consistent for comparison reasons, and to be more conservative, I chose to use Type III. A second assumption is in reference to the seismic calculations. In the N/S direction, lateral forces are resisted by both concentrically braced and eccentrically braced frames. These two cases would each use a different Response Modification Factor: R=6 for the concentrically braced frames, and R=7 for the eccentrically braced frames. Because of these differences, I used the controlling case of R=7, which results in higher story forces.

Shown below are the results found from the analysis. Full calculations are shown in Appendix 3.1.

Story Forces (kips)				
	E/W		N/S	
Floor	Wind	Seismic	Wind	Seismic
1	11.92	-	45.85	-
2	25.93	3.716	94.84	2.214
3	23.34	9.384	86.70	5.591
4	18.06	16.132	67.02	9.612
5	15.91	23.695	59.00	14.119
6	16.25	31.927	59.82	19.024
7	16.42	40.735	60.98	24.272
8	16.68	50.053	62.26	29.824
9	17.11	59.831	62.90	35.651
10	17.11	70.03	63.42	41.728
11	17.71	80.618	63.93	48.037
Roof	8.86	63.859	31.97	23.454

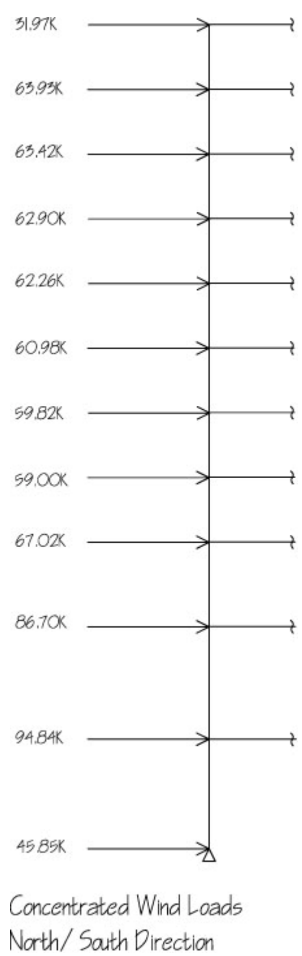
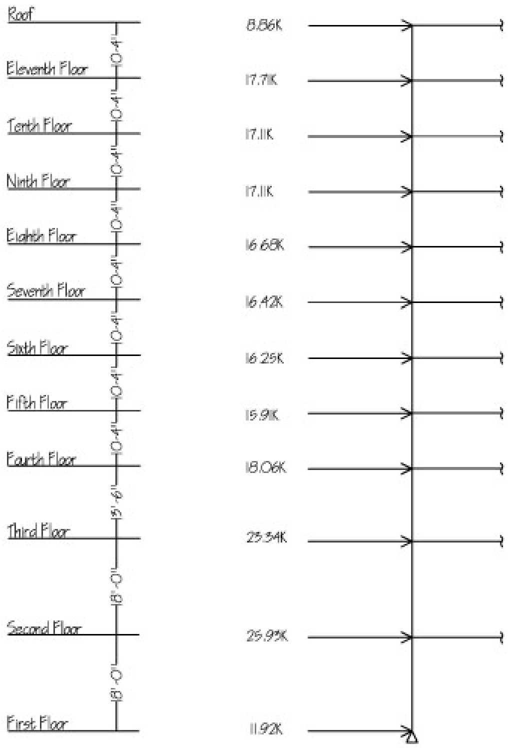


Figure 3.4

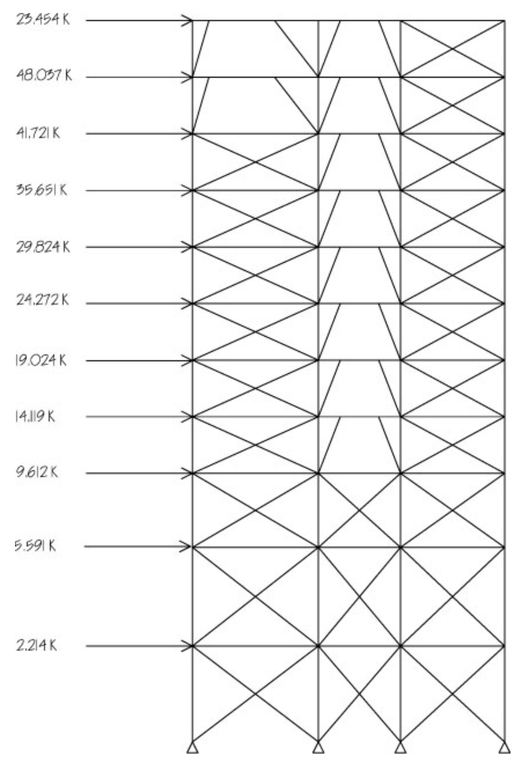
The concentrated wind story loads for the North/South direction are shown in this figure. These wind loads are resisted by two braced frames.



Concentrated Wind Loads
East/ West Direction

Figure 3.5

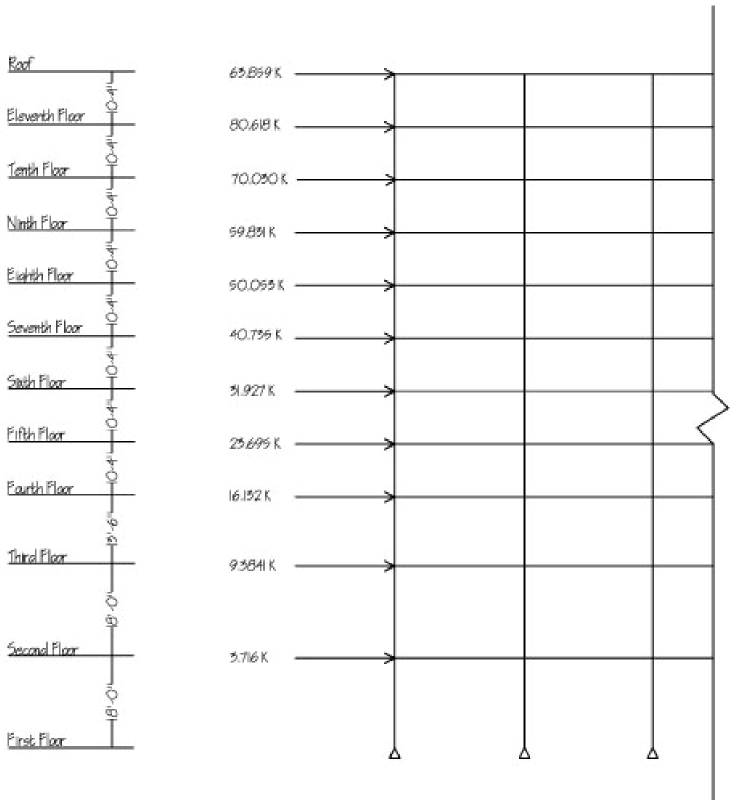
The story concentrated wind loads are shown in this figure. Calculations are available in Appendix 3.1. These loads are resisted equally by two moment frames.



Vertical Distribution of Seismic Forces
North/ South Direction

Figure 3.6

The vertical distribution per floor of seismic forces is shown in this diagram. These forces will be resisted by two braced frames similar to the one shown here.



Vertical Distribution of Seismic Forces
East/ West Direction

Figure 3.7
The seismic forces per floor in the E/W direction are shown here. They will be resisted by two moment frames.

The following load combinations to determine the critical cases were determined from ASCE7-02 (Note: loads not included in analysis were not included in load combinations given):

- 1.4 D
- 1.2D + 1.6L + 0.5R
- 1.2D + 1.6R +L
- 1.2D +1.6W +L +0.5R
- 1.2D + 1.0E + L + 0.2S
- 0.9D + 1.6W
- 0.9D + 1.0E

By using STAAD to analyze the frames, and applying the load combinations, I found that there were two different controlling load combinations. In the N/S direction, the combination 1.2D +1.6W +L +0.5R controlled because of the high wind forces based on the large surface area of the building in that direction. In the E/W direction, the combination 1.2D + 1.0E + L + 0.2 S controls because of the much larger seismic forces.

Distribution of Loads

Due to the symmetry of the Erie Convention center and Sheraton Hotel, a simplified approach to distributing loads was used. To do this, the one story, open entrance canopy was not taken into consideration. In the N/S direction, lateral loads are resisted by two braced frames, each equidistant from the center of rigidity of the building. Since each moment frame and each braced frame are identical, and therefore have the same stiffness, each will carry the same amount of load. Therefore, both wind and seismic loads can be distributed to each of these evenly, effectively placing one half of the load on each frame. Because the precast concrete plank flooring spans in the E/W direction, all dead and live loads from within the building are also distributed to the girders within the frame, which transfer the loads to the columns and then down to the foundation. Two moment frames, also equidistant from the center of rigidity of the building, resist lateral forces in the E/W direction. The same approach for the distribution of loads can be taken in this case. The lateral forces will each be distributed evenly to each moment frame, one half of the load to each. These moment frames are carrying very little additional load other than the exterior building façade because the precast concrete plank floor is carrying the loads to the frames perpendicular to these, as mentioned previously. The computation and description of gravity and lateral loads used on each frame is located in Appendix 3.2.

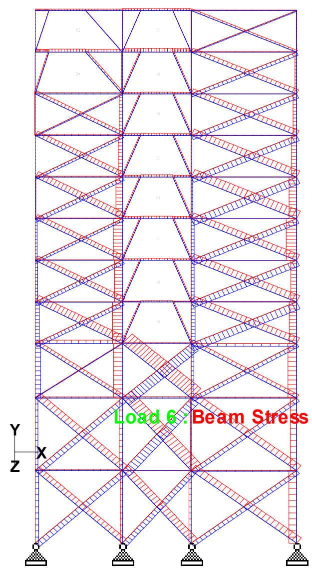
Analysis

To analyze the resistance to lateral loads, and how the members act under these loads, STAAD was used. Three versions of each frame were made to calculate and analyze stiffness, drift, and internal forces, stresses, and moments. The first version of each braced and moment frame did not include any gravity or lateral loads, but was used to determine the stiffness of the frame as a whole. A 1 kip load was applied at roof level, and the deflection under this load was found. For the moment frame, this deflection was determined to be 0.067 inches, and 0.014 inches for the braced frame. Using the equation $k=P/\Delta$, where k is the stiffness, P is the force applied (one kip in this case), and Δ is the deflection, the stiffness of each frame was determined. The stiffness of the moment frame is 14.93 kip/in. and the stiffness of the braced frame is 71.43 kip/in. Since the stiffnesses are the same for each frame in both the E/W and N/S direction, the loads will be carried equally by each frame. These stiffnesses will also be used for the determination of torsional shear effects.

The second version of the STAAD models included the application of all gravity and lateral loads and analyzed all load combination cases. As stated before, for the braced frame, the load combination 1.2D +1.6W +L +0.5R controlled because of the high wind loads. For the moment frame, the seismic loads controlled, thus the combination 1.2D + 1.0E + L + 0.2 S controlled. From this version, the design stresses, forces, ground reactions, and moments were calculated and will be checked and verified later in this report.

The third and final version of the STAAD models was used to determine story drift and overall drift. Since drift is a serviceability requirement, loads were not factored, but combinations of D + L + W + R and D + L + E + S were used. As anticipated, the case including wind controlled for the braced frames, while the case including earthquake loads controlled for the moment frames. A check for code requirements pertaining to drift will be discussed later in this report.

For the braced frame, the distribution of lateral loads through the frame can be seen through the horizontal members in the stress diagram below.

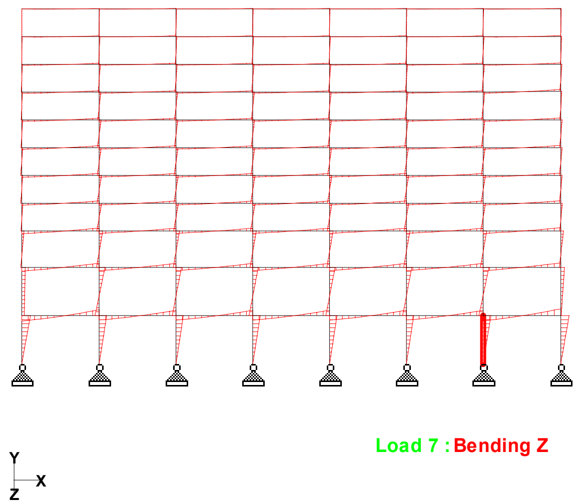


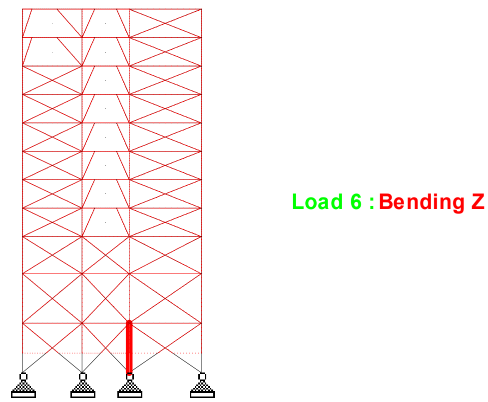
Member Checks

From the STAAD output, I analyzed the members to determine any overstressing. All steel members have a capacity of 50 ksi. For the braced frame, the output shows that the stresses are all well under the capacity, with the maximum stresses being 16 ksi. The moment frame, however, has beams with stresses that are slightly too high on the second story. The highest stress is 58.544 ksi. The overstressing is most likely caused by the assumptions made in determining the weight of the exterior wall. The second floor has the largest tributary width (half of the floor below and half of the floor above) of 18 feet. The columns and beams on the remaining floors are under the 50 ksi capacity.

Further strength tests are checked below for combined loading for columns in the moment frame. Combined loading requires that $P_u/b + M_u/m<1$. The member checked is highlighted in a thick red line on the images below.

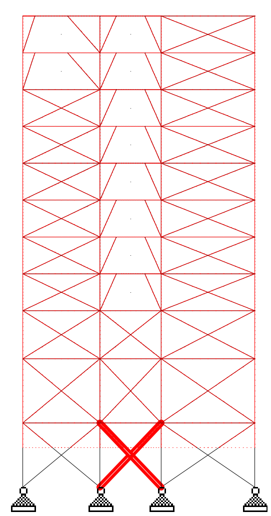
Column	P_u (k)	M_u (ft.-k)	$b\times10^3$ (kips^{-1})	$m\times10^3$ (ft-k)$^{-1}$	$P_u/b + M_u/m$
W14×176 (Moment)	103.149	504.317	0.561	0.758	0.849
Column	**P_u (k)**	**Axial Capacity (k)**			
W14×257 (Braced)	176	2630			





A member check was also done on the diagonal braces on the first floor. The axial load and capacity is shown in the following chart. The members checked are highlighted in the diagram below.

Diagonal Bracing	Axial Load (k)	Axial Capacity (k)
HSS10×6×5/8	150.085	231



Drift

Story drift and building drift were calculated using service loads for each frame. Once again, the seismic equation controlled for the moment frame, while the wind controlled for the braced frame. The drifts shown below were calculated using STAAD.

As shown in the chart below, the story drifts for the braced frame were well under the restriction of H/400. For the total building, the drift is 1.302 inches. The drift limitation for the total building is:

(132 ft.)(12”/1’)/400 = 3.96” > 1.302” OK

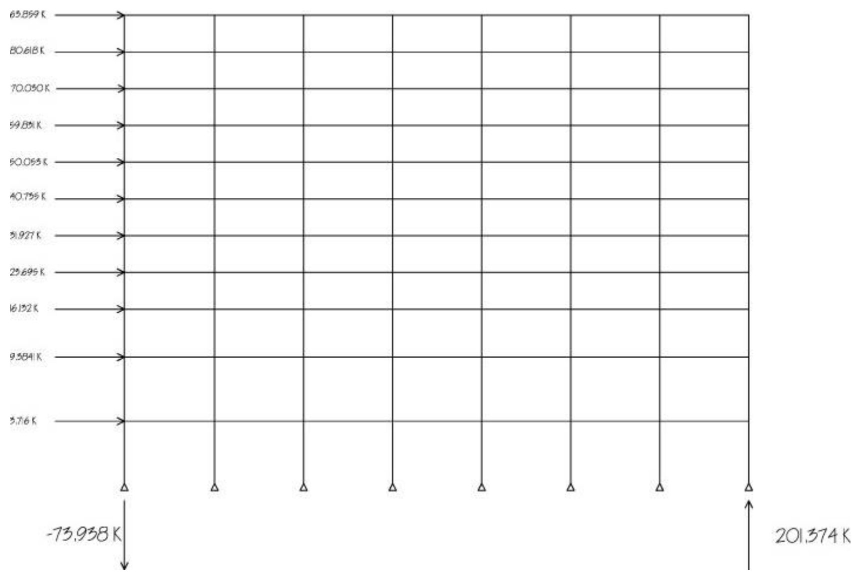
Story Drift (Braced Frame)				
Story	**Drift**	**Story Drift**	**Floor Ht**	**H/400**
1	0	0	0	0
1 to 2	0.069	0.069	18	0.54
2 to 3	0.176	0.107	18	0.54
3 to 4	0.255	0.079	13.5	0.405
4 to 5	0.359	0.104	10.33	0.3099
5 to 6	0.483	0.124	10.33	0.3099
6 to 7	0.615	0.132	10.33	0.3099
7 to 8	0.772	0.157	10.33	0.3099
8 to 9	0.972	0.2	10.33	0.3099
9 to 10	1.082	0.11	10.33	0.3099
10 to 11	1.214	0.132	10.33	0.3099
11 to R	1.302	0.088	10.33	0.3099

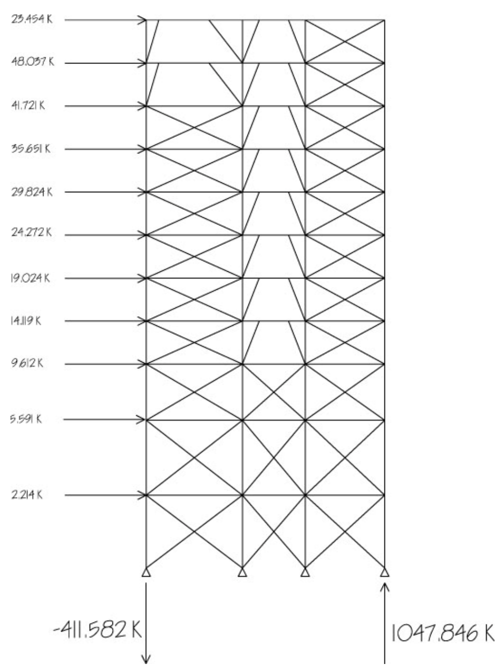
The moment frame, however, exceeds the drift limitations. Further examination found that this did not happen with the case including wind forces, but seismic loads have a very large impact on the drift. There is a possibility that there might be a reduction coefficient for the seismic drift, which would bring the drift much closer to acceptable values. The total building drift is 11.566 inches, which far exceeds the allowable drift of 3.96 inches.

Story Drift (Moment Frame)				
Story	**Drift**	**Story Drift**	**Floor Ht**	**H/400**
1	0	0	0	0
1 to 2	3.384	3.384	18	0.54
2 to 3	5.616	2.232	18	0.54
3 to 4	6.952	1.336	13.5	0.405
4 to 5	7.748	0.796	10.33	0.3099
5 to 6	8.496	0.748	10.33	0.3099
6 to 7	9.252	0.756	10.33	0.3099
7 to 8	9.93	0.678	10.33	0.3099
8 to 9	10.513	0.583	10.33	0.3099
9 to 10	11.081	0.568	10.33	0.3099
10 to 11	11.37	0.289	10.33	0.3099
11 to R	11.566	0.196	10.33	0.3099

Overturning and Impact on Foundations

From the STAAD output, the base reactions can be determined to examine the possibility of overturning. In both cases the right end reaction is greater than the left end reaction. This shows that the frame is being resisted against the lateral forces by the dead load of the building and overturning is prevented. Uplift is also partially resisted by the friction of the caissons against the bedrock. The reactions can be seen in the figures below.





Building Torsion

Due to the symmetry of the Erie Convention Center and Sheraton Hotel and its lateral system, there is no eccentricity when lateral forces are acting on the building. An eccentricity of 5% of the building length is required by code to check for torsional effects on the building due to lateral forces. The torsional effects due to the controlling lateral loads in each direction were found to be negligible in respect to the story forces. Complete calculations can be found in Appendix 3.3.

Conclusions

The lateral system of the Erie Convention Center and Sheraton Hotel has proven to be very effective. The controlling lateral loads were found to be the combination of 1.2D +1.6W +L +0.5R for the N/S direction, resisted by braced frames, and 1.2D + 1.0E + L + 0.2 S for the E/W direction, which is resisted by moment frames. Through an analysis of the braced frames and the moment frames that resist lateral loads in the building, I have found that the members are acceptable for strength, overturning, and torsion. An overall building drift as well as story drifts were found to be acceptable for the serviceability requirement of H/400 for the braced frame wind loads. The moment frames, however, did not pass this requirement for the controlling seismic loads but this may be due to incorrect assumptions for the exterior finish dead load, as well as the possibility of a reduction coefficient for seismic loading. Other girders on the second floor of the moment frame also did not pass the strength capacity of 50ksi for the steel. This could also be because of an incorrect assumption of exterior finish dead loads, and their impact on a very large tributary width.

Appendix 3.1 - Wind and Seismic Calculations

Main Wind Force Resisting System per ASCE7-02
Assumptions:
***FOR ALL "h"

***Calculating Wind in Direction:	**N/S**

Building Name	Erie Convention Center and Sheraton Hotel			
Building Location	Erie, Pennsylvania			
Location Data	**Variable**	**Reference**	**Chart/Fig.**	**Value**
Occupancy Type	-	1.5.1	T1-1	III
Importance Factor	I	6.5.5	T6-1	1.15
Surface Roughness	-	6.5.6.2	-	-
Exposure Factor	-	6.5.6.3	-	D
Enclosure Classification**	-	-		Open
				Partially
			X	Enclosed
Internal Pressure Coefficient	GC_{pi}	-	-	0.18
Topographic	K_{zt}	6.5.7.2	F6-4*	1.00
*$K_{zt}=(1+k_1k_2k_3)^2$				
**Place an "X" in the box indicating Enclosure Classification				

Building Dimensions (ft)	**Variable**	**Reference**	**Source**	**Value**
Height Above Base	h_n	9.5.5.3	Spec	155.167
Height Above Ground	z	6.300	Spec	155.167
Horiz. Length II to Wind Dir.	L	6.300	Spec	66.34
Horiz. Length Perp. to Wind	B	6.300	Spec	177.67
Horizontal Dimension Ratio	L/B	F6-6	Spec	0.37
Mean Roof Height	h	6.200	*	155.167
*Average of roof eave height and height of highest point of roof				

Wind Velocity (mph)	Variable	Reference	Chart/Fig.	Value
Basic Wind Speed	V	6.5.4	F6.1	90
Wind Directionality	k_d	6.5.4.4	T6-4	0.85
3-sec Gust Power Law	α	6.300	T6-2	11.5
Mean Wind Speed Factor: α hat	a	6.5.8.2	T6-2	0.11111111
Wind Coefficient: b hat	b	6.5.8.2	T6-2	0.8
Min Height	z_{min}	6.5.8.2	T6-2	7
Equivalent Height: z hat	z	6.5.8.2	T6-2	93.1002
Mean Hourly Wind Speed	V_z	6.5.8.2	Eq 6-14	118.50
Height atm Boundary	z_g	6.300	T6-2	700
Velocity Pressure Exp.*	k_z	6.5.6.6	T6-3**	1.55
Velocity Pressure Exp.*	k_h	6.5.6.6	T6-3**	1.55
*Calculated for ($15' < z < z_g$), or use Table 6-3				
**k_z and k_h: Use "Kz" Sheet to find value coordinating to largest "z"				

Integral Length Scale	Variable	Reference	Chart/Fig.	Value
Integral Length Scale Factor	l	6.5.8.1	T6-2	650
Integral Length Scale Exp	ε	6.5.8.1	T6-2	0.125
Integral Length Scale, Turb.	L_z	6.5.8.1	Eq 6-7	739.98
Turbulence Intensity Factor	c	6.300	T6-2	0.15
Intensity of Turbulence	I_z	6.5.8.1	Eq 6-5	0.13

Fundamental Period	Variable	Reference	Chart/Fig.	Value
Period Coefficient	C_t	9.5.3.2	T9.5.5.3.2	0.03
Period Exponent	x	9.5.3.2	T9.5.5.3.2	0.75
Approx. Fund. Period	T_a	9.5.3.2	$T_a=C_t(h_n^x)$	1.32
Natural Frequency	n_1	6.5.8.2	$n_1=1/T_a$	0.76
Rigid or Flexible	-	6.5.8.2	$n_1>1$?	Flexible

Resonance	Variable	Reference	Chart/Fig.	Value	η
R_1 Coefficient	R_h	6.5.8.2	Eq 6-13	0.195	4.567
R_1 Coefficient	R_b	6.5.8.2	Eq 6-13	0.173	5.229
R_1 Coefficient	R_l	6.5.8.2	Eq 6-13	0.141	6.537
Reduced Frequency	N_1	6.5.8.2	Eq 6-13	4.735	
Resonance Coefficient	R_n	6.5.8.2	Eq 6-13	0.053	
Damping Ratio	β	6.300	Section 9	0.050	
Resonant Response Factor	R	6.5.8.2	Eq 6-10	0.145	

Gust Effect Factor	Variable	Reference	Chart/Fig.	Value
Gust Coefficient	g_q	6.5.8.2	Eq 6-8	3.4
Gust Coefficient	g_v	6.5.8.2	Eq 6-8	3.4
Gust Coefficient	g_r	6.5.8.2	Eq 6-9	4.12
Background Response	Q	6.5.8.1	Eq 6-6	0.85
Gust Factor	G_f	6.5.8.2	Eq 6-8	0.87

Wind Pressure	Variable	Reference	Chart/Fig.	Value
Velocity Pressure	qz	6.5.10	Eq 6-15	31.418
Velocity Pressure @ h	qh	6.5.12.2	T6-3*	31.418
*$q_h=0.00256k_hk_{zt}k_d(V^2)I$				

Ext. Pressure Coefficient	Variable	Reference	Chart/Fig.	Value
Windward Side	C_p	6.5.11.2	F6-6*	0.8
Leeward Side	C_p	6.5.11.2	F6-6*	-0.5
Sidewall	C_p	6.5.11.2	F6-6*	-0.7
*Formulas must be checked with any new code changes				

Leeward Pressure (psf)	P_1	6.5.12.2	$P_1=q_hG_fC_p$	-13.728
Final Pressure (psf)	$P=q_zG_fC_p-q_hG_fC_p$			

RESULTS						
z(ft)	k_z(T6-3)	q_z	$P_{sidewall}$(psf)	$P_{leeward}$(psf)	$P_{windward}$(psf)	P_{total}(psf)
0-15	1.03	20.878	-12.772	-13.728	14.596	28.324
20	1.08	21.891	-13.392	-13.728	15.305	29.033
25	1.12	22.702	-13.888	-13.728	15.872	29.600
30	1.16	23.513	-14.384	-13.728	16.438	30.167
40	1.22	24.729	-15.128	-13.728	17.289	31.017
50	1.27	25.742	-15.748	-13.728	17.997	31.725
60	1.31	26.553	-16.244	-13.728	18.564	32.292
70	1.34	27.161	-16.616	-13.728	18.989	32.717
80	1.38	27.972	-17.112	-13.728	19.556	33.284
90	1.40	28.377	-17.360	-13.728	19.839	33.568
100	1.43	28.985	-17.732	-13.728	20.265	33.993
120	1.48	29.999	-18.352	-13.728	20.973	34.701
140	1.52	30.810	-18.848	-13.728	21.540	35.268
160	1.55	31.418	-19.220	-13.728	21.965	35.693

Main Wind Force Resisting System per ASCE7-02
Assumptions:
***FOR ALL "h"

***Calculating Wind in Direction:	**E/W**

Building Name	Erie Convention Center and Sheraton Hotel			
Building Location	Erie, Pennsylvania			
Location Data	**Variable**	**Reference**	**Chart/Fig.**	**Value**
Occupancy Type	-	1.5.1	T1-1	III
Importance Factor	I	6.5.5	T6-1	1.15
Surface Roughness	-	6.5.6.2	-	-
Exposure Factor	-	6.5.6.3	-	D
Enclosure Classification**	-	-		Open
				Partially
			X	Enclosed
Internal Pressure Coefficient	GC_{pi}	-	-	0.18
Topographic	K_{zt}	6.5.7.2	F6-4*	1.00
$*K_{zt}=(1+k_1k_2k_3)^2$				
**Place an "X" in the box indicating Enclosure Classification				

Building Dimensions (ft)	Variable	Reference	Source	Value
Height Above Base	h_n	9.5.5.3	Spec	155.167
Height Above Ground	z	6.300	Spec	155.167
Horiz. Length II to Wind Dir.	L	6.300	Spec	177.67
Horiz. Length Perp. to Wind	B	6.300	Spec	66.34
Horizontal Dimension Ratio	L/B	F6-6	Spec	2.68
Mean Roof Height	h	6.200	*	155.167
*Average of roof eave height and height of highest point of roof				

Wind Velocity (mph)	Variable	Reference	Chart/Fig.	Value
Basic Wind Speed	V	6.5.4	F6.1	90
Wind Directionality	k_d	6.5.4.4	T6-4	0.85
3-sec Gust Power Law	α	6.300	T6-2	11.5
Mean Wind Speed Factor: α hat	a	6.5.8.2	T6-2	0.11111111
Wind Coefficient: b hat	b	6.5.8.2	T6-2	0.8
Min Height	z_{min}	6.5.8.2	T6-2	7
Equivalent Height: z hat	z	6.5.8.2	T6-2	93.1002
Mean Hourly Wind Speed	V_z	6.5.8.2	Eq 6-14	118.50
Height atm Boundary	z_g	6.300	T6-2	700
Velocity Pressure Exp.*	k_z	6.5.6.6	T6-3**	1.55
Velocity Pressure Exp.*	k_h	6.5.6.6	T6-3**	1.55
*Calculated for (15'<z<z_g), or use Table 6-3				
**k_z and k_h: Use "Kz" Sheet to find value coordinating to largest "z"				

Integral Length Scale	Variable	Reference	Chart/Fig.	Value
Integral Length Scale Factor	l	6.5.8.1	T6-2	650
Integral Length Scale Exp	ε	6.5.8.1	T6-2	0.125
Integral Length Scale, Turb.	L_z	6.5.8.1	Eq 6-7	739.98
Turbulence Intensity Factor	c	6.300	T6-2	0.15
Intensity of Turbulence	I_z	6.5.8.1	Eq 6-5	0.13

Fundamental Period	Variable	Reference	Chart/Fig.	Value
Period Coefficient	C_t	9.5.3.2	T9.5.5.3.2	0.028
Period Exponent	x	9.5.3.2	T9.5.5.3.2	0.8
Approx. Fund. Period	T_a	9.5.3.2	$T_a=C_t(h_n^x)$	1.58
Natural Frequency	n_1	6.5.8.2	$n_1=1/T_a$	0.63
Rigid or Flexible?	-	6.5.8.2	$n_1>1$?	Flexible

Resonance	Variable	Reference	Chart/Fig.	Value	η
R_1 Coefficient	R_h	6.5.8.2	Eq 6-13	0.228	3.802
R_1 Coefficient	R_b	6.5.8.2	Eq 6-13	0.433	1.626
R_1 Coefficient	R_l	6.5.8.2	Eq 6-13	0.066	14.576
Reduced Frequency	N_1	6.5.8.2	Eq 6-13	3.942	
Resonance Coefficient	R_n	6.5.8.2	Eq 6-13	0.059	
Damping Ratio	β	6.300	Section 9	0.050	
Resonant Response Factor	R	6.5.8.2	Eq 6-10	0.256	

Gust Effect Factor	**Variable**	**Reference**	**Chart/Fig.**	**Value**
Gust Coefficient	g_q	6.5.8.2	Eq 6-8	3.4
Gust Coefficient	g_v	6.5.8.2	Eq 6-8	3.4
Gust Coefficient	g_r	6.5.8.2	Eq 6-9	4.08
Background Response	Q	6.5.8.1	Eq 6-6	0.88
Gust Factor	G_f	6.5.8.2	Eq 6-8	0.90

Wind Pressure	**Variable**	**Reference**	**Chart/Fig.**	**Value**
Velocity Pressure	qz	6.5.10	Eq 6-15	31.418
Velocity Pressure @ h	qh	6.5.12.2	T6-3*	31.418
*$q_h=0.00256k_hk_{zt}k_d(V^2)I$				

Ext. Pressure Coefficient	Variable	Reference	Chart/Fig.	Value
Windward Side	C_p	6.5.11.2	F6-6*	0.8
Leeward Side	C_p	6.5.11.2	F6-6*	-0.266
Sidewall	C_p	6.5.11.2	F6-6*	-0.7
*Formulas must be checked with any new code changes				

Leeward Pressure (psf)	P_1	6.5.12.2	$P_1=q_hG_fC_p$	-7.508
Final Pressure (psf)	$P=q_zG_fC_p-q_hG_fC_p$			

RESULTS						
z(ft)	k_z(T6-3)	q_z	$P_{sidewall}$(psf)	$P_{leeward}$(psf)	$P_{windward}$(psf)	P_{total}(psf)
0-15	1.03	20.878	-13.124	-7.508	14.999	22.507
20	1.08	21.891	-13.761	-7.508	15.727	23.235
25	1.12	22.702	-14.271	-7.508	16.310	23.818
30	1.16	23.513	-14.781	-7.508	16.892	24.400
40	1.22	24.729	-15.545	-7.508	17.766	25.274
50	1.27	25.742	-16.182	-7.508	18.494	26.002
60	1.31	26.553	-16.692	-7.508	19.077	26.584
70	1.34	27.161	-17.074	-7.508	19.514	27.021
80	1.38	27.972	-17.584	-7.508	20.096	27.604
90	1.40	28.377	-17.839	-7.508	20.387	27.895
100	1.43	28.985	-18.221	-7.508	20.824	28.332
120	1.48	29.999	-18.858	-7.508	21.552	29.060
140	1.52	30.810	-19.368	-7.508	22.135	29.643
160	1.55	31.418	-19.750	-7.508	22.572	30.079

Wind Story Forces					
East/West Direction					
Floor	**Pressure**	**Total Pressure**	**Tributary ht.**	**Length**	**Force (K)**
1	15.727	23.235	9	57	11.92
2	17.766	25.274	18	57	25.93
3	18.494	26.002	15.75	57	23.34
4	19.077	26.585	11.915	57	18.06
5	19.514	27.022	10.33	57	15.91
6	20.096	27.604	10.33	57	16.25
7	20.387	27.895	10.33	57	16.42
8	20.824	28.332	10.33	57	16.68
9	21.552	29.06	10.33	57	17.11
10	21.552	29.06	10.33	57	17.11
11	22.572	30.08	10.33	57	17.71
roof	22.572	30.08	5.165	57	8.86
North/South Direction					
1	15.305	29.033	9	175.48	45.85
2	17.289	31.017	18	175.48	94.84
3	17.997	31.725	15.75	175.48	86.70
4	18.654	32.382	11.915	175.48	67.02
5	18.989	32.717	10.33	175.48	59.00
6	19.556	33.284	10.33	175.48	59.82
7	20.265	33.993	10.33	175.48	60.98
8	20.973	34.701	10.33	175.48	62.26
9	20.973	34.701	10.33	175.48	62.90
10	21.54	35.268	10.33	175.48	63.42
11	21.54	35.268	10.33	175.48	63.93
roof	21.54	35.268	5.165	175.48	31.97

Seismic Loads per ASCE7-02				
Assumptions: Total height to roof slab divided by the number of stories= Inter-story height				
Building Name	Erie Convention Center and Sheraton Hotel			
Building Location	Erie, Pennsylvania			
Seismic Design Parameters				
Number of stories	N		11	
Inter-story Height	h_s		12.06	ft
Building Height	h_n		132.66	ft
Seismic Use Group	I	Table 9.1.3	II	
Occupancy Importance Factor		Table 9.1.4	1.25	
Site Classification		(Soil)	E	
0.2 s Acceleration	S_s	Figure 9.4.1.1a	0.13	
1 s Acceleration	S_1	Figure 9.4.1.1b	0.059	
Site Class Factor	F_a	Table 9.4.1.2.4a	2.5	
Site Class Factor	F_v	Table 9.4.1.2.4b	3.5	
Adjusted Accelerations	S_{ms}	S_s*F_a	0.325	g-s
	S_{m1}	S_1*F_v	0.2065	g-s
Design Spectral Response Accelerations	S_{DS}	2/3*S_{ms}	0.217	g-s
	S_{D1}	2/3*S_{m1}	0.138	g-s
Seismic Design Category (Short-Period)		Table 9.4.2.1a	B	
Seismic Design Category (1-Second)		Table 9.4.2.1b	C	

Equivalent Lateral Force Procedure			
Seismic Base Shear Coefficient			
N-S Direction (Concentrically and Eccentrically Braced Frames)			
Response Modification Factor	R_{N-S}	T-9.5.2.2	7
Seismic Response Coefficient	$C_{s,\ N-S}$	$S_{DS}/(R/I)$	0.039
	$C_{T,\ N-S}$	T-9.5.5.3.3	0.03
	x		0.75
Approximate Period	T_{aN-S}	$C_{T,\ N-S}h_n^x$	1.173
Seismic Response Coefficient			
Need not be greater than	$C_{S\ max,\ N-S}$	$S_{D1}/(T^*(R_{N-S}/I))$	0.021
Minimum of	$C_{S\ min}$	$0.044^*S_{ds}^*I$	0.012
Therefore	$C_{s,\ N-S}$		**0.021**
E-W Direction (Ordinary Steel Moment Frames)			
Response Modification Factor	R_{E-W}	T-9.5.2.2	3.5
Seismic Response Coefficient	$C_{s,\ E-W}$	$S_{DS}/(R/I)$	0.077
	$C_{T,\ E-W}$	T-9.5.5.3.2	0.028
	x		0.8
Approximate Period	T_{aE-W}	$C_{T,\ N-S}h_n^x$	1.397
Seismic Response Coefficient			
Need not be greater than	$C_{S\ max,\ E-W}$	$S_{D1}/(T^*(R_{N-S}/I))$	0.035
Minimum of	$C_{S\ min}$	$0.044^*S_{ds}^*I$	0.012
Therefore	$C_{s,\ E-W}$		**0.035**

Loading Characteristics			
Roof			
Dead	(Assumed in Conjunction with ASCE 7-02, Table C3-1)		
	Metal Roof Deck	2.1	psf
	5" Polystyrene Insulation	1	psf
	EPDM Rubber Roof	1	psf
	Roof Framing	20	psf
	MEP	10	psf
	Finishing	5	psf
	Total q_{roof}	**39.1**	**psf of roof area**

All Other Floors				
Dead	(Assumed in Conjunction with ASCE 7-02, Table C3-1)			
	8" Concrete Plank		60	psf
	Framing Members		10	psf
	MEP		10	psf
	Carpet		1	psf
	Ceiling Finishing		1	psf
	Metal Stud Walls w/ 5/8" gwb		10	psf
	Total	q_{floor}	**92**	**psf of floor area**

Perimeter Wall			
Dead	(Assuming 10% brick veneer (48psf),		
40% glazing (8psf), 50% stud walls with EIFS (12psf))			
	q_{wall}	154	psf
Snow Load not included because $p_f < 30$psf			

Building Width		177.67	ft	
Building Length		66.34	ft	
Gross Roof/Floor Area		11786.63	ft^2	
Total Weight of Roof		498	kips	$Area*q_{roof}+2(L+W)*0.5*q_{wall}$
Total Weight per Floor		1160	kips	$Area*q_{floor}+2(L+W)q_{wall}$
Total Weight of Floors		11595	kips	(N-1)*Total Weight per floor
Total Building Weight	W	12094	kips	Weight of roof + weight of floors

Seismic Base Shear V			
$\mathbf{V_{N\text{-}S}}$	**253.5263**	**kips**	$V=C_s*W$
$\mathbf{V_{E\text{-}W}}$	**425.4802**	**kips**	$V=C_s*W$

Vertical Distribution of Seismic Forces

Exponent $k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ 1.336

T<0.5, k=1 ; T>2.5, k=2 ; 0.5<T<2.5, linear interpolation between 1 and 2

Vertical Redistribution of Seismic Forces North-South Direction							
Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(Kips)	(ft)			(kips)	(kips)	(ft-kips)
Roof	498.4347	132.66	342,217	0.093	23.454		3111.402
11	1159.525	120.6	700,904	0.189	48.037	23	5793.229
10	1159.525	108.54	608,851	0.165	41.728	71	4529.141
9	1159.525	96.48	520,181	0.141	35.651	113	3439.588
8	1159.525	84.42	435,169	0.118	29.824	149	2517.780
7	1159.525	72.36	354,156	0.096	24.272	179	1756.336
6	1159.525	60.3	277,576	0.075	19.024	203	1147.133
5	1159.525	48.24	206,005	0.056	14.119	222	681.082
4	1159.525	36.18	140,255	0.038	9.612	236	347.777
3	1159.525	24.12	81,583	0.022	5.591	246	134.863
2	1159.525	12.06	32,309	0.009	2.214	251	26.705
1	1159.525					254	
	Σ=		Σ=	Σ=	Σ=		Σ=
	13253.21		3,699,204	1.000	254		23485.03

Exponent $k_{E\text{-}W} = 1 + (T_{E\text{-}W} - 0.5)/(2.5 - 0.5) =$ 1.449

Vertical Redistribution of Seismic Forces East-West Direction							
Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(Kips)	(ft)			(kips)	(kips)	(ft-kips)
Roof	498.4347	132.66	592,818	0.150	63.859		8471.597
11	1159.525	120.6	700,904	0.189	80.618	64	9722.476
10	1159.525	108.54	608,851	0.165	70.030	144	7601.023
9	1159.525	96.48	520,181	0.141	59.831	215	5772.482
8	1159.525	84.42	435,169	0.118	50.053	274	4225.459
7	1159.525	72.36	354,156	0.096	40.735	324	2947.568
6	1159.525	60.3	277,576	0.075	31.927	365	1925.173
5	1159.525	48.24	206,005	0.056	23.695	397	1143.024
4	1159.525	36.18	140,255	0.038	16.132	421	583.656
3	1159.525	24.12	81,583	0.022	9.384	437	226.333
2	1159.525	12.06	32,309	0.009	3.716	446	44.817
1	1159.525					450	
	Σ=		Σ=	Σ=	Σ=		Σ=
	13253.21		3,949,806	1.058	450		42663.61

where $C_{vx} = w_x h_x^k / \Sigma_{\text{all levels}} (w_x h_x^k)$

$F_x = C_{vx} V$

Appendix 3.2 - Distribution of Loads

The gravity loads used for analysis are as follows:

Dead Loads: (Assumed)

- Framing members = 10 psf
- 8" Hollow core precast concrete plank = 60 psf
  (This is an estimated and slightly conservative weight based on loads given by the engineer and by Nitterhouse Concrete Products)
- Metal Stud Walls with 5/8" gypsum wall board = 10 psf
- MEP = 10 psf
- Carpet = 1 psf
- Ceiling Finishing = 1 psf

Total 92 psf

Live Loads: (IBC 2003)

- Public Rooms and Corridors =100 psf
- Private Rooms and Corridors = 40 psf
- Mechanical Spaces =150 psf
- Stairs =100 psf
- Ground Snow Load = 30 psf
  p_f=16.8 psf

The gravity loads are carried from the precast concrete planks to the interior girders and from the girders to the columns, which carry the loads to the ground. On the exterior bays running N/S, the girders carry half of the gravity load in that bay to the columns, which carry the loads to the ground. Because of the spanning direction of the floor system, the moment frames in the E/W direction carry little gravity loads besides the self weight and the weight of the building exterior. The gravity loads for each frame are shown below:

Braced Frame:

Tributary width = 12.67 ft.

DL (floors 1-11) = 92 psf × 12.67 ft. = 1.167 klf
Roof load = 39 psf × 12.67 ft. = 0.494 klf
LL (floors 1-2) = 100 psf × 12.67 ft. = 1.267 klf
LL (floors 3-11) = 40 psf × 12.67 ft. = 0.507 klf
Snow Load = 16.8 psf × 12.67 ft. = 0.213 klf

The following exterior wall loads apply for both the braced frames and moment frames:

Wall Dead Loads (14 psf)

Floor 2 (18 ft.) = 0.252 klf
Floor 3 (15.75 ft.) = 0.220 klf
Floor 4 (11.92 ft.) = 0.167 klf
Floors 5-11 (10.33 ft.) = 0.145 klf
Roof (5.165 ft.) = 0.072 klf

The following loads are the lateral wind and seismic forces found for each story level. The E/W direction loads resisted by the moment frames, and the N/S direction loads are resisted by braced frames:

Story Forces (kips)				
	E/W		N/S	
Floor	Wind	Seismic	Wind	Seismic
1	11.92	-	45.85	-
2	25.93	3.716	94.84	2.214
3	23.34	9.384	86.70	5.591
4	18.06	16.132	67.02	9.612
5	15.91	23.695	59.00	14.119
6	16.25	31.927	59.82	19.024
7	16.42	40.735	60.98	24.272
8	16.68	50.053	62.26	29.824
9	17.11	59.831	62.90	35.651
10	17.11	70.03	63.42	41.728
11	17.71	80.618	63.93	48.037
roof	8.86	63.859	31.97	23.454

The following loads are the lateral wind and seismic forces found for each story level divided by two to be distributed to each moment and braced frame:

Story Forces/2 (kips)				
	E/W		N/S	
Floor	Wind	Seismic	Wind	Seismic
1	5.96	0.00	22.93	0.00
2	12.97	1.86	47.42	1.11
3	11.67	4.69	43.35	2.80
4	9.03	8.07	33.51	4.81
5	7.96	11.85	29.50	7.06
6	8.13	15.96	29.91	9.51
7	8.21	20.37	30.49	12.14
8	8.34	25.03	31.13	14.91
9	8.56	29.92	31.45	17.83
10	8.56	35.02	31.71	20.86
11	8.86	40.31	31.97	24.02
roof	4.43	31.93	15.98	11.73

Stiffnesses:

Moment Frames (E/W)

P=1.0 kip
Δ=0.067”

k=P/Δ
k= 1.0/.067 = 14.93 k/in.

Braced Frames (N/S)

P=1.0 kip
Δ=0.014

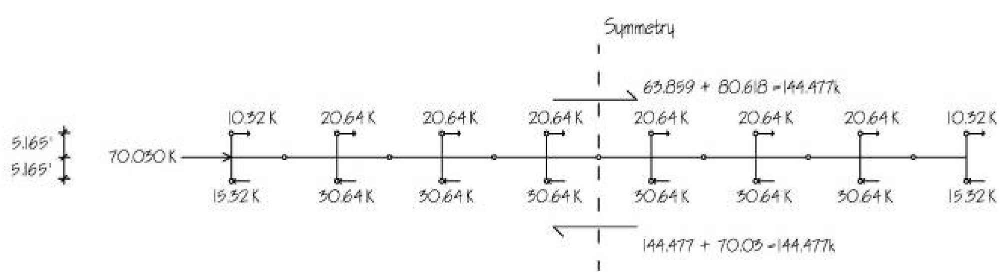
k=P/ Δ
k=1.0/0.014 = 71.43

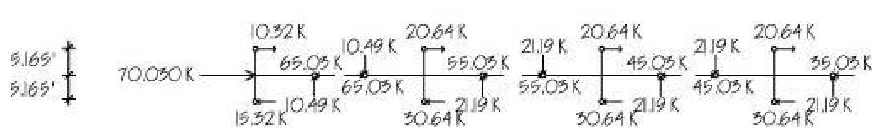
Additional Drift: (Based on STAAD Model)

Moment Frame Wind Deflection - E/W (overall)
Drift = 3.871inches
H/400= 132ft (12”/1’)/ 400 = 3.96” > 3.87” ok

Therefore, the drift passes for the wind forces in the E/W direction, but not for the seismic forces, which control.

Portal Method:





Appendix 3.3 - Torsional Effects

e= 5% of the building length in each direction

$e_{N/S}$ = 0.05 (57ft.) = 2.85 ft.
$e_{E/W}$= 0.05 (175.5 ft.) = 8.775 ft.

Torsional Moment (M_T):

N/S - Braced Frame (Wind loads control)

Braced Frame		M_T=Pe		
Story	P	e	M_T	
1	22.93	2.85	65.34	ft.-k
2	47.42	2.85	135.14	ft.-k
3	43.35	2.85	123.55	ft.-k
4	33.51	2.85	95.50	ft.-k
5	29.50	2.85	84.08	ft.-k
6	29.91	2.85	85.24	ft.-k
7	30.49	2.85	86.89	ft.-k
8	31.13	2.85	88.72	ft.-k
9	31.45	2.85	89.64	ft.-k
10	31.71	2.85	90.37	ft.-k
11	31.97	2.85	91.10	ft.-k
Roof	15.98	2.85	45.55	ft.-k

E/W – Moment Frame (Seismic loads control)

Moment Frame		M_T=Pe		
Story	P	e	M_T	
1	0	8.775	0.00	ft.-k
2	1.86	8.775	16.30	ft.-k
3	4.69	8.775	41.17	ft.-k
4	8.07	8.775	70.78	ft.-k
5	11.85	8.775	103.96	ft.-k
6	15.96	8.775	140.08	ft.-k
7	20.37	8.775	178.72	ft.-k
8	25.03	8.775	219.61	ft.-k
9	29.92	8.775	262.51	ft.-k
10	35.02	8.775	307.26	ft.-k
11	40.31	8.775	353.71	ft.-k
Roof	31.93	8.775	280.18	ft.-k

k = stiffness of frame (found in Appendix 3.2)
x = distance from frame to center of rigidity (1/2 of the building length)

Torsional Shear = M_T*kx/(Σkx^2)				
Frame	**k**	**x**	**kx^2**	**kx/(Σkx^2)**
Moment N	14.93	28.5	12126.89	0.000378
Moment S	14.93	28.5	12126.89	0.000378
Braced W	71.43	87.75	550015.5	0.005575
Braced E	71.43	87.75	550015.5	0.005575
		Σkx^2=	1124285	

$F_{TORSION}$ is negligible compared to seismic shear forces.

Moment Frame- Torsional Shear		
Story	**$F_{TORSION}$**	
1	0.0000	k
2	0.0062	k
3	0.0156	k
4	0.0268	k
5	0.0393	k
6	0.0530	k
7	0.0676	k
8	0.0831	k
9	0.0994	k
10	0.1163	k
11	0.1339	k
Roof	0.1060	k

$F_{TORSION}$ is negligible as compared to wind story forces.

Braced Frame- Torsional Shear		
Story	**$F_{TORSION}$**	
1	0.3643	k
2	0.7534	k
3	0.6888	k
4	0.5324	k
5	0.4687	k
6	0.4752	k
7	0.4844	k
8	0.4946	k
9	0.4997	k
10	0.5038	k
11	0.5079	k
Roof	0.2539	k