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Erie Convention Center and Sheraton Hotel Erie, Pennsylvania

Technical Report #1: Structural Concepts/ Structural Existing Conditions Report
Submittal Date: 5 October 2005

Executive Summary

The Erie Convention Center and Sheraton Hotel sits on the waterfront of the Presque Isle Bay on West Dobbins Landing, in Erie, Pennsylvania. This site provides a great opportunity to enjoy all that the bay and the surrounding area has to offer, as well as a place for conferences and receptions. The proposed hotel is an eleven story, 132,000 sq.ft., steel structure with an attached parking garage and a pedestrian walkway from the fifth floor to the Bayfront Convention Center.

This report is a full description of the structural system and calculations of all of the loads that affect the design of the structure, including gravity, wind, and seismic loads. A complete list of codes used to obtain these values is also given. In addition, spot checks for gravity and lateral members are completed and compared with the sizes provided by the engineers. Typical frames and bays are drawn with given sizes for ease in understanding the explanation of framing members.

Through my analysis and calculations, I found that the Erie Convention Center and Sheraton Hotel is a steel structure with pre-cast concrete plank floors designed using IBC 2003. Cross and knee braced frames are used in the North/South direction, and moment frames in the East/West direction, both for resistance to lateral loads. The foundation consists of caissons drilled approximately twenty feet into the ground to ensure that they are enclosed in at least three feet of bedrock. Grade beams span each of these caissons, and are poured monolithically with column piers. Through my own beam and column design, I found similar beam sizes to those designed by the engineer. Any discrepancies can possibly be accounted for by the difference in the ASD and the LRFD method of solving, as well as the fact that I took a simplified approach to design, not taking into account all of the surrounding factors.

Structural System:

Framing-

The Erie Convention Center and Sheraton Hotel is an eleven story building with a five story attached parking garage, and a pedestrian crosswalk to the Bayfront Convention Center. The hotel is comprised of a steel structure with a pre-cast concrete plank floor system.

The layout of the structural system for the hotel is comprised of interior bays 25 ft wide, with the exterior bays that are 25'-4" wide. In the North/South (N/S) direction, the bays span 19'-0", 15'-0", and 23'-0", South to North, respectively (Figure 1.1). In the main building tower, typical girders range from W14x30 to W24x55. These girders span in the N/S direction and support the floor loads from the pre-cast concrete plank floor, which spans in the East-West (E/W) direction. Smaller W10x22 shapes frame into the columns in the E/W direction, in the interior bays. Even though they are not carrying any direct floor load, they are needed for column stability. The North and South walls are moment frames, and therefore are constructed with much larger W18's. Elevator openings and mechanical spaces are supported with intermediately sized beams. The roof structural system includes sloping W14, W16, and W18 girders with beams ranging from W12x14's to W12x19's. The two story porte cochere on the North side of the hotel is comprised of beams ranging from W8x24 to W12x40, and girders that span 35'-7", sized at W30x99 and W36x135. Its roof is constructed with W33x18's and trusses. The trusses are 3'-6" deep, and have W12x30 top and bottom chords with HSS 6x4x3/8 diagonals. The columns range in sizes from W14x43 to W14x426, with smaller W8's and W10's supporting the porte cochere and other exterior canopies (Figure 1.2).

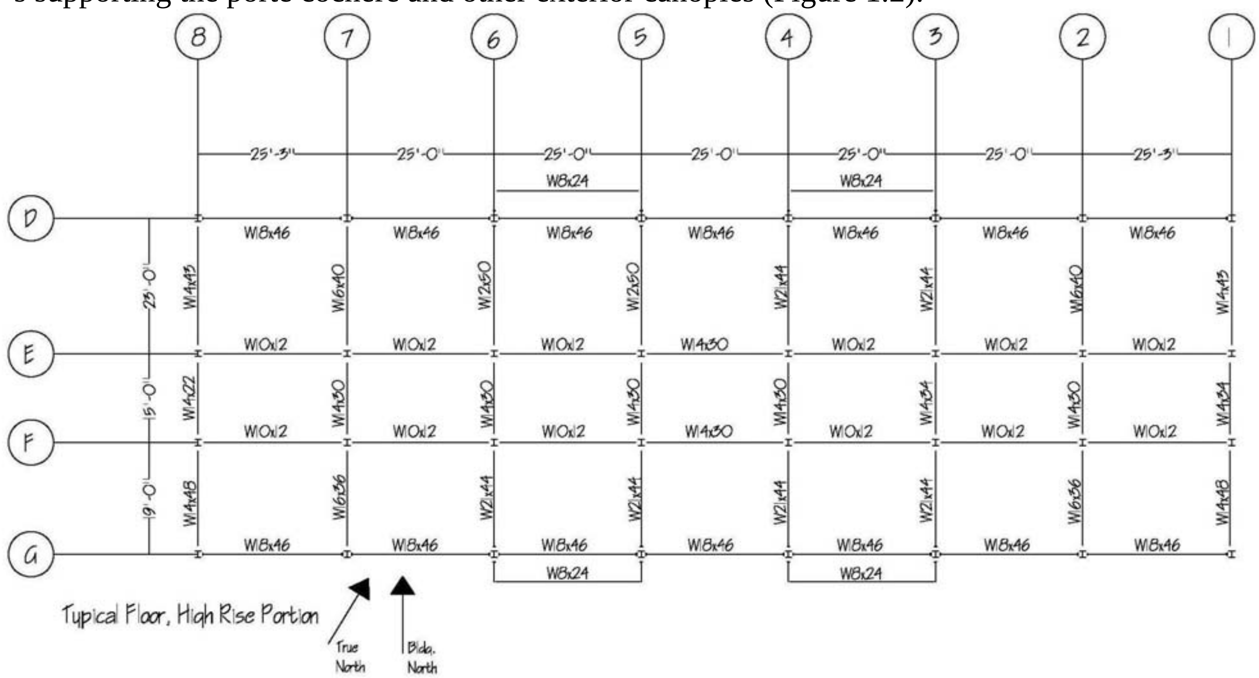


Figure 1.1

This is a typical floor of the high rise portion (floors 3-11) of the Erie Hotel and Convention Center. The beams on the interior of bays supporting mechanical loads have been removed for simplicity. The first and second stories have a porte cochere extending off of the North side of the building. There are also two elevator shafts extending nine feet North, between column lines 8 and 6 for the first through 5th floors, accessing the pedestrian walkway.

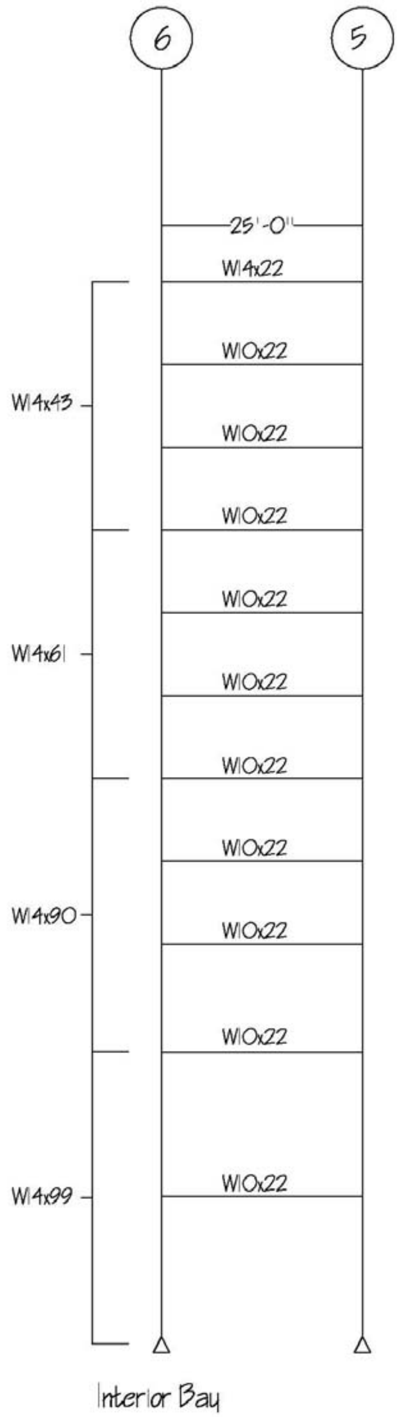


Figure 1.2
The column and beam layout between floors is as shown for a typical interior bay.

Lateral Bracing-

The lateral bracing in the E/W direction is comprised of A500 structural steel tubing, in a combination of cross and knee braces. Cross braces run the full height in the N/S direction on the outer two bays with the exception of the top two floors in the exterior bays to the rear of the building, which use knee braces to allow for windows (Figure 1.3). The middle interior bay in the East Elevation uses knee braces from the ground to roof to allow for a glass curtain wall strip. In the West Elevation, however, knee braces are present from the fourth through eleventh floors to allow for this glazing, but below, cross-braces are used because of the adjoining parking garage. This difference in the two elevations causes the member sizes in each direction to vary slightly. In the East Elevation the exterior bays' cross-bracing members range from HSS 14x6x5/8 on the first floor to HSS 6x6x3/8 on the eleventh floor, with HSS 6x6x3/8 knee braces as well in the interior bay. Alternatively, in the West direction, the cross-bracing members range from HSS 14x6x5/8, however, on the bottom three floors, some of the members are smaller than in the East Elevation due to the cross bracing instead of knee bracing in the center bay. Moment connections are used for lateral support in the north-south direction.

The elevator shaft discussed in the commentary for Figure 1.1, is also braced by cross-braces in both the N/S and E/W directions.

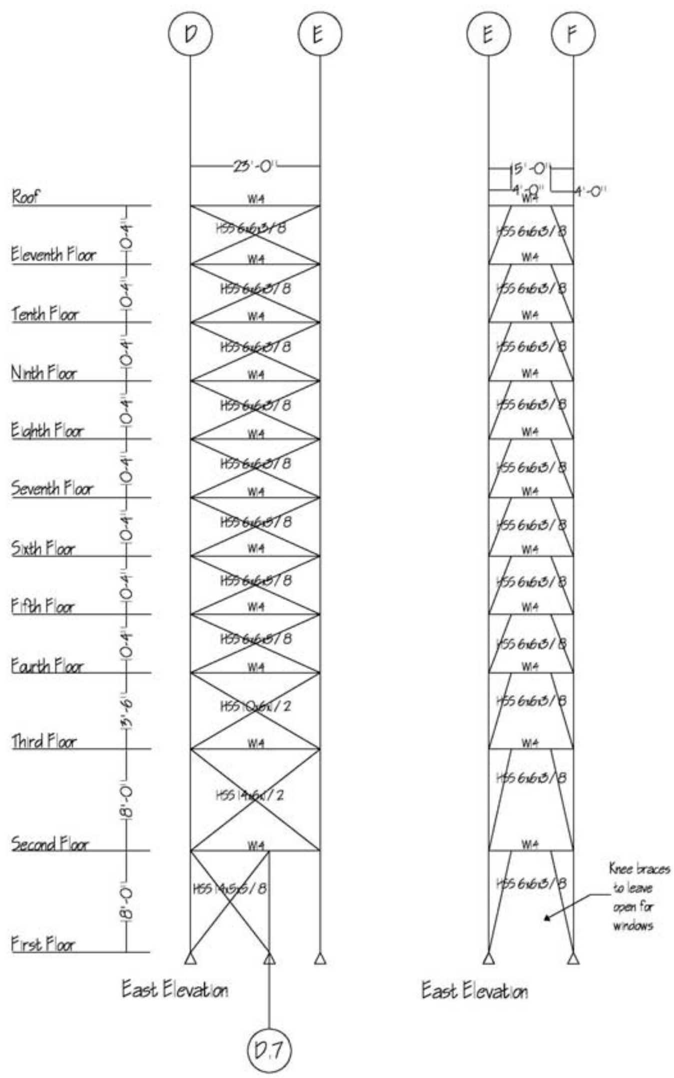


Figure 1.3 (a)
The exterior (left) and interior (right) lateral resistance framing systems allow for window and door placement free from obstruction.

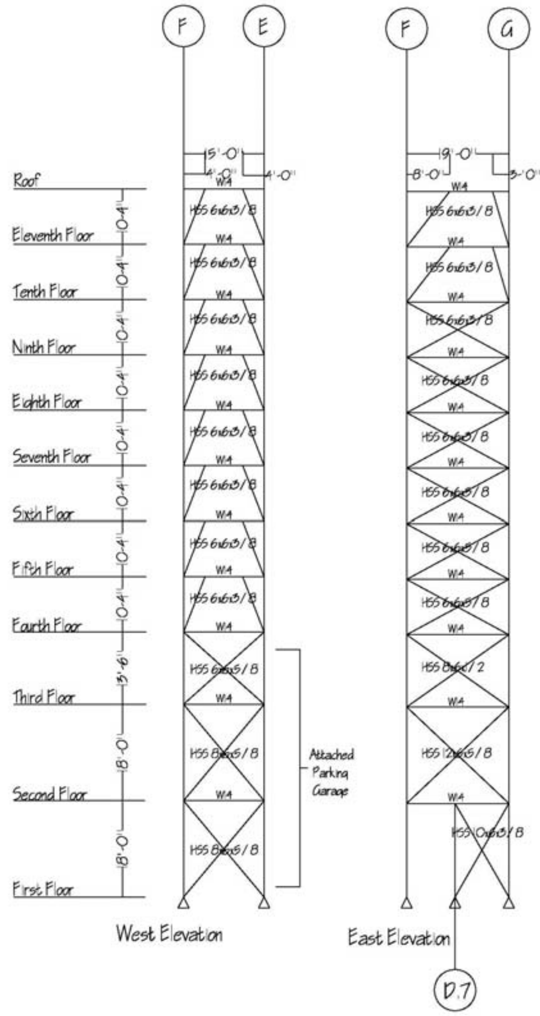


Figure 1.3 (b)
The West Elevation uses cross bracing for structural support, but is not concerned with the allowing the placement of windows because of the adjacent attached parking garage on the first, second, and third floors. The East Elevation is a continuation of the frame in Figure 1.3 (a).

Foundation-

Due to the unstable soil on site, caissons need to be drilled down into at least three feet of bedrock to prevent uplift. These caissons have a minimum required diameter of 24" but range up to as much as 60" in diameter, and are approximately 20 feet deep. By drilling three feet into the bedrock, the net allowable end bearing pressure is 40ksf as compared to the 30ksf maximum end bearing pressure, which could be used in design if the caissons were only bearing directly on the bedrock. In addition, shaft resistance can be added to the caisson capacity using 3.0ksf allowable side friction applied to the socket surface area in the bedrock. Grade beams that vary from 18 to 20 inches in width and depth span the caissons, and are cast monolithically with concrete piers, which vary between 18"x18" and 24"x24". Reinforcing bars and stirrups range from #6 to #8 for the grade beams, and between 4, 6, and 8 #9 bars are used in the piers. The structural concrete slab for the first floor is 8" thick and is reinforced with #6 bars spaced at 10" on center on the top and bottom in the N/S direction, with #4 bars spaced at 12" on center, top and bottom in the E/W direction.

Floors-

The floor system of the Erie Convention Center and Sheraton Hotel is comprised of 8" thick pre-cast concrete planks. These planks run in the E/W direction, spanning 25' for the interior bays or 25'-3" for the exterior bays. The strength of the concrete planks is 5000 psi.

Roof-

Continuous 3-span, 1 ½”, 20 gage, galvanized wide rib steel roof deck with 5” insulation, and a fully adhered EPDM rubber roof membrane are the components of the roofing system.

Exterior Walls-

The exterior wall system is a combination of pre-cast concrete, copper cladding (architectural details), brick, and EIFS (Exterior Insulation and Finish System). The 6” pre-cast concrete and 4” face brick both are backed by an air space, 1” rigid insulation, 8” structural metal studs with 8” batt insulation, and 5/8” gypsum wall board. The 2” EIFS with drainage is backed with 5/8” glassmat gypsum board and 6” structural metal studs with 6” batt insulation.

Material Strengths-

Framing Members (W-shapes): A992 Grade 50
Bracing Members (HSS-shapes): A 500 Grade 50
Rebar: ASTM 615 Grade 60
Concrete:
  Caissons, Grade Beams, and Piers: 3000 psi
  Structural Slab: 4000 psi
  Pre-Cast Concrete Plank: 5000 psi

Building Codes:

IBC 2003
ASCE 7-98 (I will be using ASCE 7-02 for my own wind and seismic calculations)
ACI 318-02
AISC ASD 9th ed. (I will be using LRFD, 3rd ed. for my own analysis.)

Loads:

Live Loads: (Given by Engineer)

Public Rooms and Corridors = 100 psf
(IBC 2003: Table 1607.1 Residential-Hotels and multi family dwellings-public rooms and corridors serving them = 100 psf)

Private Rooms and Corridors = 40 psf
(IBC 2003: Table 1607.1 Residential-Hotels and multi family dwellings-private rooms and corridors serving them = 40 psf)

Mechanical Spaces = 150 psf
(not given in IBC 2003-common assumption)

Stairs = 100 psf
(IBC 2003: Table 1607.1 Stairs and exits = 100 psf)

Ground Snow Load (p_g) = 30 psf
(IBC 2003: Figure 1608.2)
C_e=0.8 (fully exposed, exposure category D)
C_t=1.0
I=1.1 (Building category III)
p_f=0.7 C_e C_t I p_g= 16.8 psf

Dead Loads: (Assumed)

8” concrete plank	= 50 psf
Framing Members	= 10 psf
MEP	= 10 psf
Carpet	= 1 psf
Ceiling Finishing	= 1 psf
Partitions	= 20 psf
Total	92 psf

Wind Loads:

Wind loads were calculated in both the N/S, and E/W directions. Some important assumptions that were made include:

- Building is a rectangular box
  - The 2 story porte cochere was ignored, as well as the five story elevator shaft on the North side
- Building Dimensions used were 177.67ft (E/W), 66.34ft (N/S), and 155.167ft height
  - The height includes the parapet that extends above the roof slab

The lateral resisting systems are braced frames when the wind is blowing in the N/S direction, and moment frames in the E/W direction. (See Appendix A.1for calculations and factors)

North/South Direction

Building Information		
Basic Wind Speed (mph)	V	90
Wind Importance Factor	I_w	1.0
Exposure Category	-	D
Enclosure Classification	-	Enclosed
Building Category	-	III
Importance Factor	I	1.15
Internal Pressure Coefficient	GC_{pi}	0.18
Wind Design Pressure (psf)	$P_{windward}$	21.965
Wind Design Pressure (psf)	$P_{leeward}$	-13.728

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

East/West Direction

Building Information		
Basic Wind Speed (mph)	V	90
Wind Importance Factor	I_w	1.0
Exposure Category	-	D
Enclosure Classification	-	Enclosed
Building Category	-	III
Importance Factor	I	1.15
Internal Pressure Coefficient	GC_{pi}	0.18
Wind Design Pressure (psf)	$P_{windward}$	22.572
Wind Design Pressure (psf)	$P_{leeward}$	-7.508

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

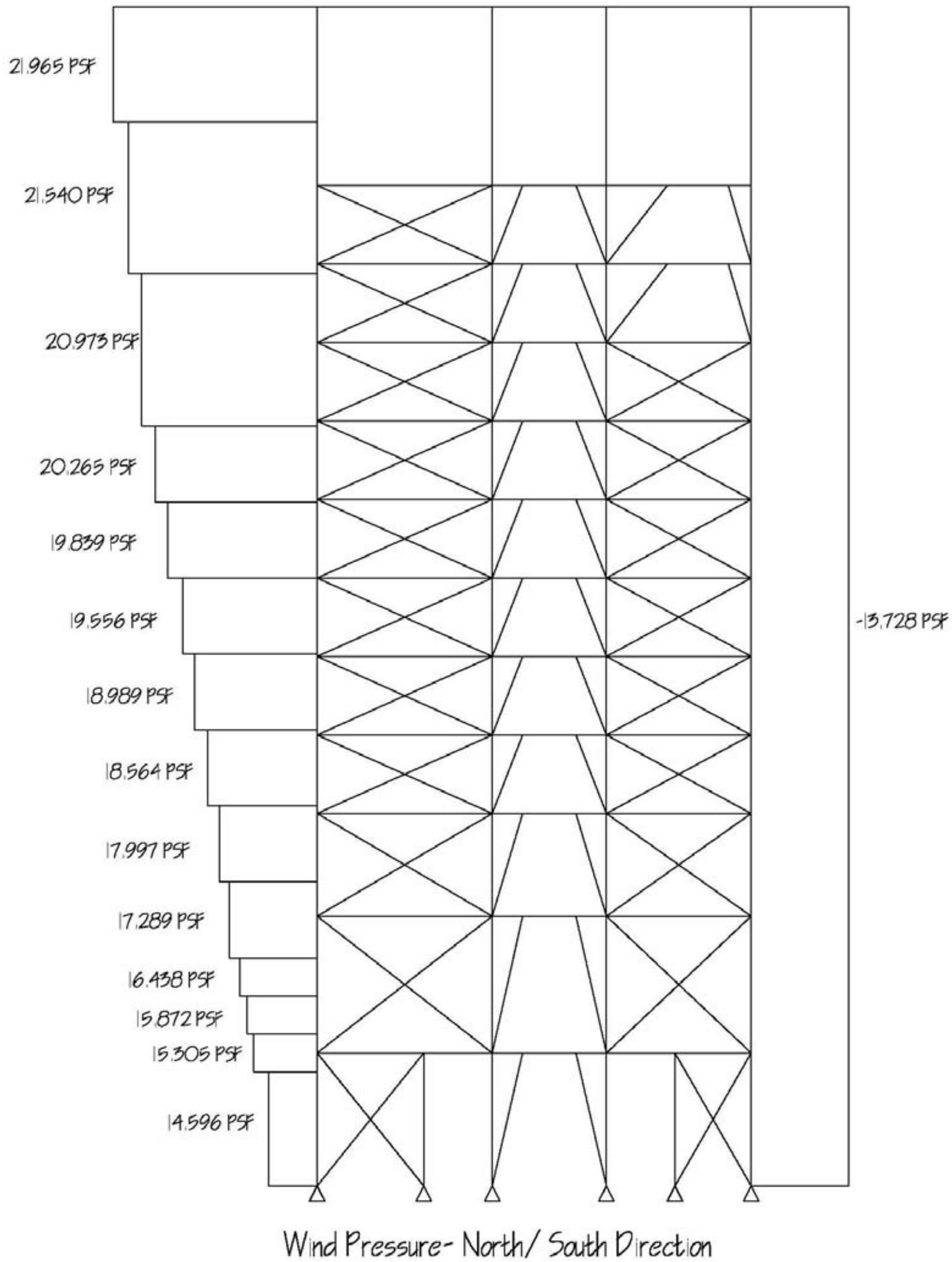


Figure 1.4
Wind blowing in the N/S direction is opposed by braced frames. Shown are the wind pressures acting on the exterior face of the building. The windward pressures increase from 14.596psf from zero feet to fifteen feet to 21.965psf at the top of the parapet. The leeward pressure is a constant -13.728psf.

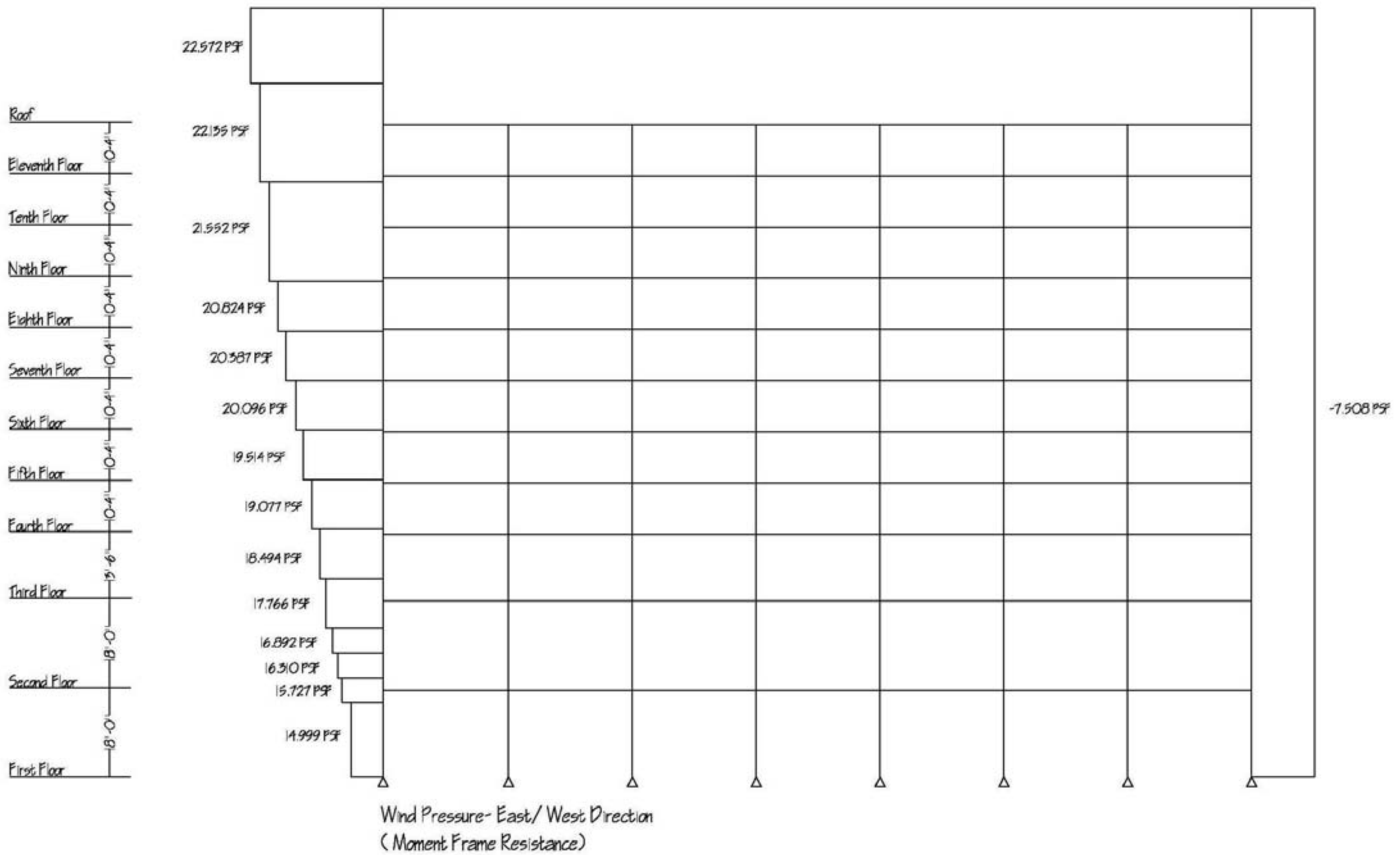


Figure 1.5
Wind blowing in the E/W direction is opposed by moment frames on both the North and South exterior walls. The windward pressures, shown in the figure above, range from 14.999psf to 22.572psf, and there is a leeward pressure of -7.508psf.

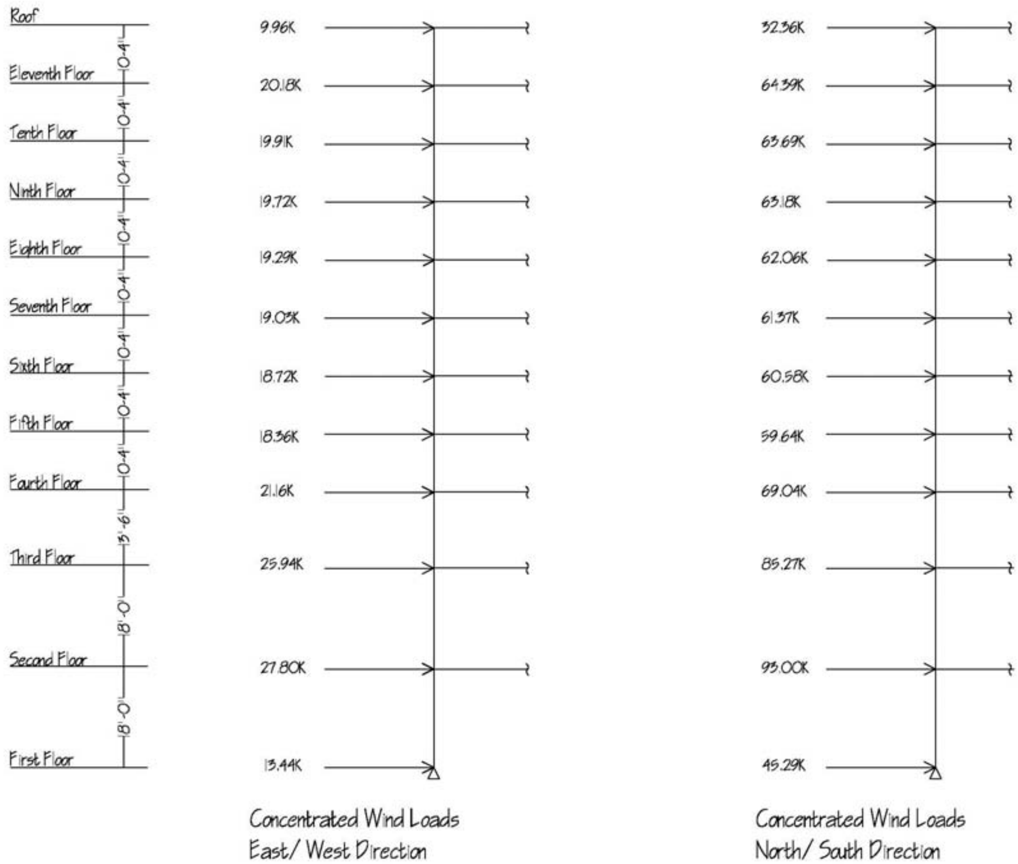


Figure 1.6
Shown above are the concentrated loads on each floor derived from the wind pressures in the E/W and N/W direction. Notice that the point loads in the N/S direction are much larger than in the E/W direction because the length of the wall that the wind pressure is acting on is over twice as long as in the E/W direction. For this calculation I only used the pressures up to the height of the roof, and did not include the parapet. Calculations can be viewed in Appendix A.3.

Seismic Loads:

For the calculation of seismic loads, I again assumed that my building acts as a box. For the ease of the calculations, I took the height to the roof and divided by the number of stories (11), to even out the differences in inner-story heights. See Appendix A.2 for additional factors and calculations.

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	54.726		7259.937
11	1159.525	120.6	700,904	0.189	112.086	55	13517.534
10	1159.525	108.54	608,851	0.165	97.365	167	10567.995
9	1159.525	96.48	520,181	0.141	83.185	264	8025.704
8	1159.525	84.42	435,169	0.118	69.590	347	5874.819
7	1159.525	72.36	354,156	0.096	56.635	417	4098.117
6	1159.525	60.3	277,576	0.075	44.389	474	2676.643
5	1159.525	48.24	206,005	0.056	32.943	518	1589.191
4	1159.525	36.18	140,255	0.038	22.429	551	811.479
3	1159.525	24.12	81,583	0.022	13.046	573	314.680
2	1159.525	12.06	32,309	0.009	5.167	586	62.311
1	1159.525					592	
	Σ=		Σ=	Σ=	Σ=		Σ=
	13253.21		3,699,204	1.000	592		54798.41

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	74.503		9883.530
11	1159.525	120.6	700,904	0.189	94.054	75	11342.889
10	1159.525	108.54	608,851	0.165	81.701	169	8867.860
9	1159.525	96.48	520,181	0.141	69.803	250	6734.562
8	1159.525	84.42	435,169	0.118	58.395	320	4929.703
7	1159.525	72.36	354,156	0.096	47.524	378	3438.829
6	1159.525	60.3	277,576	0.075	37.248	426	2246.035
5	1159.525	48.24	206,005	0.056	27.644	463	1333.528
4	1159.525	36.18	140,255	0.038	18.821	491	680.932
3	1159.525	24.12	81,583	0.022	10.948	510	264.055
2	1159.525	12.06	32,309	0.009	4.336	521	52.286
1	1159.525					525	
	Σ=		Σ=	Σ=	Σ=		Σ=
	13253.21		3,949,806	1.058	525		49774.21

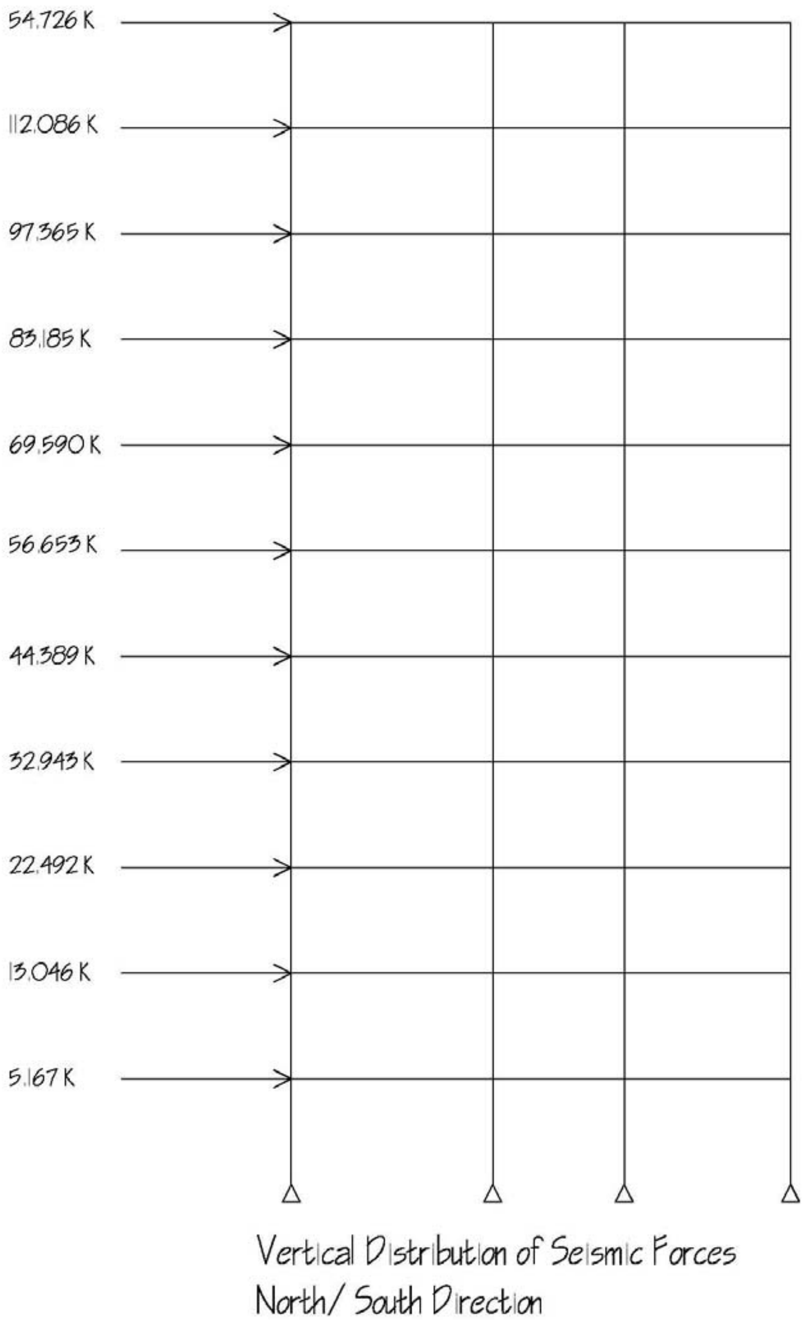


Figure 1.7 (a)
Resisted by Cross-Bracing

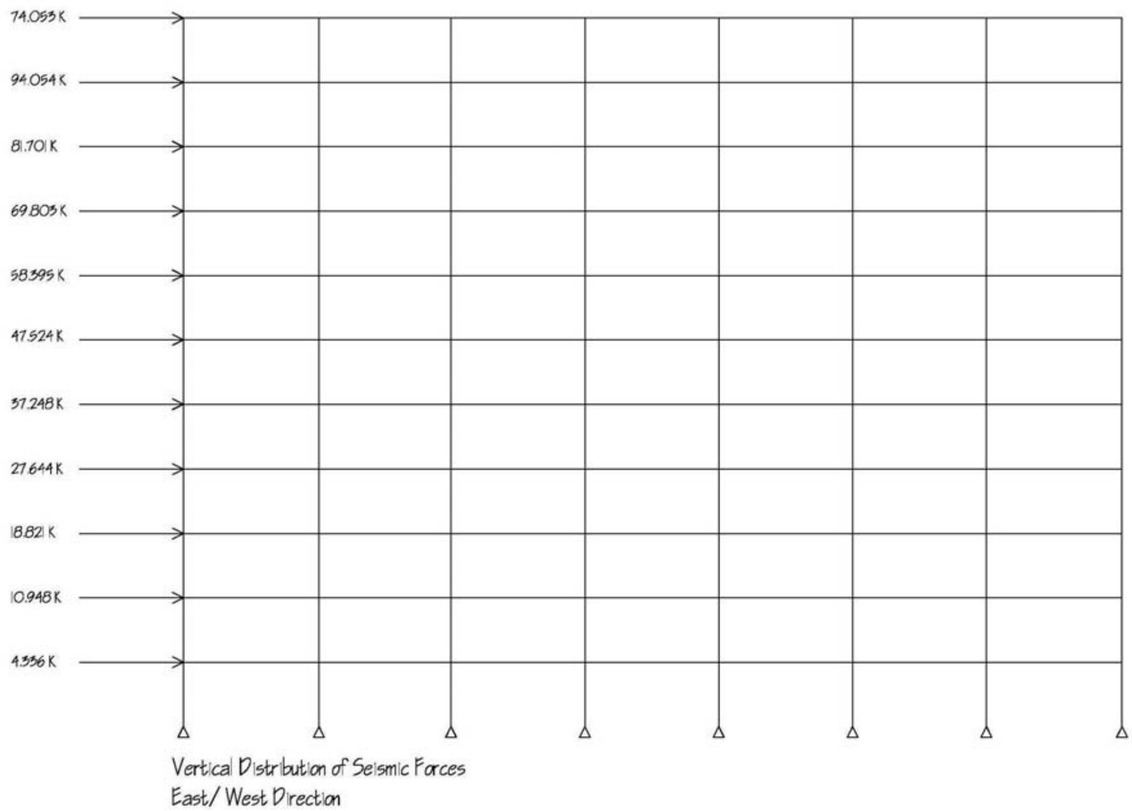


Figure 1.7 (b)
The story forces derived from the seismic forces on the high-rise portion, beginning at the second floor, are shown in the figures (a) and (b) above.

The E/W direction is resisted by moment frames.

Gravity Spot Checks:

To ensure that my assumptions for gravity live and dead loads were correct, I performed two spot checks, one beam, and one column. For the beam check, I analyzed a typical bay (with no additional mechanical weights) on a typical floor. The beam designed by the engineer is a W14x30. It carries the entire floor load due to the fact that the plank spans perpendicular to its spanning direction. After computing the live load reduction factor, and performing a simple beam design, I found the size needed to be a W14x22.

I also checked a column that this beam frames into. I chose to analyze the seventh floor, taking into account the axial loads from the columns above. While, I designed a W14x74 needed, the given design is a W14x61.

These beam and column sizes could differ from the designs given by the engineer, for several reasons. First, I used the LRFD method of design, while ASD was used by the engineer. Also, it is common practice for beams to be sized similarly to other beams in the area. Because of the variation in span lengths, it is possible that surrounding beams needed to be sized as a W14x30, and therefore this particular beam size was increased to a W14x30 as well. Similarly, surrounding columns have different areas of influence. Another possible difference could be in the calculations and assumptions of dead and live load.

Please see Appendix A.4 for complete calculations.

Lateral Element Check:

In the East/West direction, lateral forces are opposed by moment frames. To analyze this resistance, I performed a portal analysis on the tenth floor to find the internal forces acting on the members at a single story level. The seismic forces controlled, and therefore were used to carry out this procedure. By combining the lateral moment and axial load, I designed the column to be a W14x61. The given column design is a W14x43. This difference is a discrepancy of loads and methods used. Please see the Appendix A.5 for full calculations and results.

In addition, I will be preparing a computer model of the cross-bracing system to analyze the members for future use.

Looking Forward:

In addition to the topics previously discussed, there are several factors that I assumed temporarily negligible for simplicity. In addition to the affects and design of the porte cochere, there is a pedestrian walkway that extends out of the fifth floor of the building. Even though I ignored these factors, they will have an impact on lateral and gravity loads.

Appendix A.1

N/S Main Wind Force Resisting System per ASCE7-02
Assumptions: Rectangular Box
Ignore porte cochere and elevator shafts
***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=0.00256$k_h k_{zt} k_d (V^2)$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	$P_1=q_h G_f C_p$	-13.728
Final Pressure (psf)	$P=q_z G_f C_p - q_h G_f C_p$			

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

E/W Main Wind Force Resisting System per ASCE7-02
Assumptions: Rectangular Box
Ignore porte cochere and elevator shafts
***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=0.00256$k_h k_{zt} k_d (V^2)$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	$P_1=q_h G_f C_p$	-7.508
Final Pressure (psf)	$P=q_z G_f C_p - q_h G_f C_p$			

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

Appendix A.2

Seismic Loads per ASCE-02				
Assumptions: Total height to roof slab divided by the number of stories= Inter-story height				
Structure not specifically detailed for seismic resistance				
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			
Response Modification Factor	R_{N-S}	T-9.5.2.2	3
Seismic Response Coefficient	$C_{s, N-S}$	$S_{DS}/(R/I)$	0.090
	$C_{T, N-S}$	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	$C_{S\ max, N-S}$	$S_{D1}/(T*(R_{N-S}/I))$	0.049
Minimum of	$C_{S\ min}$	$0.044*S_{ds}*I$	0.012
Therefore	$C_{s, N-S}$		**0.049**
E-W Direction			
Response Modification Factor	R_{E-W}	T-9.5.2.2	3
Seismic Response Coefficient	$C_{s, E-W}$	$S_{DS}/(R/I)$	0.090
	$C_{T, E-W}$	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	$C_{S\ max, E-W}$	$S_{D1}/(T*(R_{N-S}/I))$	0.041
Minimum of	$C_{S\ min}$	$0.044*S_{ds}*I$	0.012
Therefore	$C_{s, E-W}$		**0.041**

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		50	psf
	Framing Members		10	psf
	MEP		10	psf
	Carpet		1	psf
	Ceiling Finishing		1	psf
	Partitions		20	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< 30psf			

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				
	V_{N-S}	**591.5615**	**kips**	$V=C_s$*W
	V_{E-W}	**496.3935**	**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	54.726		7259.937
11	1159.525	120.6	700,904	0.189	112.086	55	13517.534
10	1159.525	108.54	608,851	0.165	97.365	167	10567.995
9	1159.525	96.48	520,181	0.141	83.185	264	8025.704
8	1159.525	84.42	435,169	0.118	69.590	347	5874.819
7	1159.525	72.36	354,156	0.096	56.635	417	4098.117
6	1159.525	60.3	277,576	0.075	44.389	474	2676.643
5	1159.525	48.24	206,005	0.056	32.943	518	1589.191
4	1159.525	36.18	140,255	0.038	22.429	551	811.479
3	1159.525	24.12	81,583	0.022	13.046	573	314.680
2	1159.525	12.06	32,309	0.009	5.167	586	62.311
1	1159.525					592	
	Σ=		Σ=	Σ=	Σ=		Σ=
	13253.21		3,699,204	1.000	592		54798.41

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	74.503		9883.530
11	1159.525	120.6	700,904	0.189	94.054	75	11342.889
10	1159.525	108.54	608,851	0.165	81.701	169	8867.860
9	1159.525	96.48	520,181	0.141	69.803	250	6734.562
8	1159.525	84.42	435,169	0.118	58.395	320	4929.703
7	1159.525	72.36	354,156	0.096	47.524	378	3438.829
6	1159.525	60.3	277,576	0.075	37.248	426	2246.035
5	1159.525	48.24	206,005	0.056	27.644	463	1333.528
4	1159.525	36.18	140,255	0.038	18.821	491	680.932
3	1159.525	24.12	81,583	0.022	10.948	510	264.055
2	1159.525	12.06	32,309	0.009	4.336	521	52.286
1	1159.525					525	
	Σ=		Σ=	Σ=	Σ=		Σ=
	13253.21		3,949,806	1.058	525		49774.21

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

Appendix A.3

Wind Forces at Story Levels (Concentrated)

N-S Direction:

$F_1 = (28.324\text{ psf})(9')(177.67') = 45.29^k$

$F_2 = [(28.324\text{ psf})(6') + (29.033\text{ psf})(5') + (29.6\text{ psf})(5') + (30.167\text{ psf})(2')](177.67')$
$= 93.00^k$

$F_3 = [(30.167\text{ psf})(3') + (31.017\text{ psf})(10') + (31.725\text{ psf})(2.5')](177.67')$
$= 85.27^k$

$F_4 = [31.725\text{ psf}(7.5') + (32.292\text{ psf})(4.665')](177.67')$
$= 69.04^k$

$F_5 = [(32.292\text{ psf})(5.335') + (32.717\text{ psf})(4.995')](177.67')$
$= 59.64^k$

$F_6 = [(32.717\text{ psf})(5.005') + (33.284\text{ psf})(5.325')](177.67')$
$= 60.583^k$

$F_7 = [(33.284\text{ psf})(4.675') + (33.568\text{ psf})(5.655')](177.67')$
$= 61.37^k$

$F_8 = [(33.568\text{ psf})(4.345') + (33.993\text{ psf})(5.985')](177.67')$
$= 62.06^k$

$F_9 = [(33.993\text{ psf})(4.015') + (34.701\text{ psf})(6.315')](177.67')$
$= 63.18^k$

$F_{10} = [(34.701\text{ psf})(10.33')](177.67')$
$= 63.69^k$

$F_{11} = [(34.701\text{ psf})(3.355') + (35.268\text{ psf})(6.975')](177.67')$
$= 64.39^k$

$F_{12} = [(35.268\text{ psf})(5.165')](177.67')$
$= 32.36^k$

Wind Forces at Story Levels (Concentrated)

E-W Direction:

$$F_1 = (22.507\,psf)(9')(66.34') = 13.44^k$$

$$F_2 = [(22.507\,psf)(6') + (23.235\,psf)(5') + (23.818\,psf)(5') + (24.4\,psf)(2')](66.34') = 27.80^k$$

$$F_3 = [(24.4\,psf)(3') + (25.294\,psf)(10') + (26.002\,psf)(2.5')](66.34') = 25.94^k$$

$$F_4 = [(26.002\,psf)(7.5') + (26.584\,psf)(4.665')](66.34') = 21.16^k$$

$$F_5 = [(26.584\,psf)(5.335') + (27.021\,psf)(4.995')](66.34') = 18.36^k$$

$$F_6 = [(27.021\,psf)(5.005') + (27.604\,psf)(5.325')](66.34') = 18.72^k$$

$$F_7 = [(27.604\,psf)(4.675') + (27.895\,psf)(5.655')](66.34') = 19.03^k$$

$$F_8 = [(27.895\,psf)(4.345') + (28.332\,psf)(5.985')](66.34') = 19.29^k$$

$$F_9 = [(28.332\,psf)(4.015') + (29.060\,psf)(6.315')](66.34') = 19.72^k$$

$$F_{10} = [(29.06\,psf)(10.33\,psf)](66.34') = 19.91^k$$

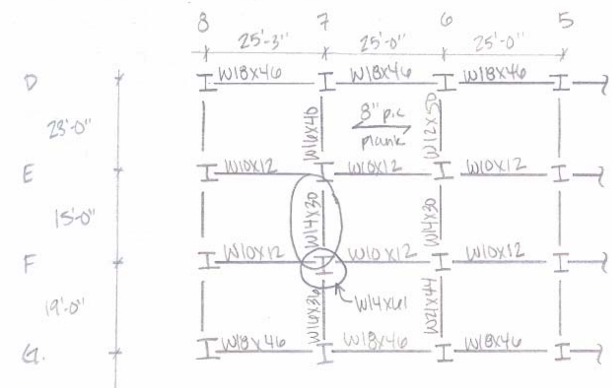
$$F_{11} = [(29.06\,psf)(3.355') + (29.643\,psf)(6.975')](66.34') = 20.18^k$$

$$F_R = (29.06\,psf)(5.165')(66.34') = 9.96^k$$

SPOT CHECKS

Appendix A.4

PORTION OF TYP. FLOOR:



BEAM E-F ℄7:

(ASSUME TYP. FLR. W/ GUEST ROOM LOADING)

LOADS: DL = 92 psf.
LL = 40 psf.

$$A_T = \frac{(25.25' + 25.0')}{2}(15') = 376.875\,ft^2$$

$$T_W = \frac{25.25' + 25.0'}{2} = 25.125\,ft.$$

$K_{LL} = 2$ (TABLE 4-2, ASCE 7-02)

LL REDUCTION:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

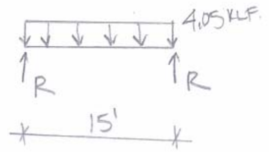
$$L = L_o\left(0.25 + \frac{15}{\sqrt{2(376.875)}}\right)$$

$L = 0.8L_o > 0.4 \therefore$ USE $0.8L_o$.

$L = 0.8(40psf) = 31.85psf$

$$(w_u)_{L+D} = 1.2(92psf) + (1.6)(31.85psf)$$
$$= 110.4\,psf + 50.96\,psf.$$
$$w_u = 161.36\,psf.$$

$$W_u = T_W\,w_u = 25.125'(161.36psf)$$
$$W_u = 4.05\,KLF$$



$R = \frac{4.05\text{KLF}\,(15')}{2} = 30.375^K$

$M_{max} = \frac{WL^2}{8} = \frac{(4.05\text{KLF})(15')^2}{8} = 113.906'^K$

Try W14X22 $\qquad \phi M_p = 123'^K > 113.906'^K$

W14x30 used.

COLUMN F-7, 7th floor (5 floors above)

$A_T = \frac{(25.25' + 25')(15' + 19')}{2}(5) = 4271.25 ft^2$

$K_{LL} = 4$ (INTERIOR COLUMN)

$A_I = K_{LL} A_T = 4(4271.25 ft^2)$
$A_I = 17085.0 ft^2$

LL REDUCTION:

$L = L_o\left(0.25 + \frac{15}{\sqrt{17085}}\right) = 0.36 L_o < 0.4 L_o \quad \therefore$ use $0.4 L_o$.

$L = 0.4(44 psf) = 17.6 psf.$

- $P = 1.4D = 1.4(92 psf) = 128.8 psf$
- $P = 1.2D + 1.6L + 0.5R = 1.2(92 psf) + 1.6(17.6 psf) + 0.5(16.8 psf) = 146.96 psf$ ← * cont.

$P_u = P \cdot A_T = (146.96 psf)(4271.25 ft^2) = 627.7^K$

Guess W14 shape.

$\alpha = 24/d = 24/14 = 1.71$

$M_u = \frac{WL^2}{16} = \frac{4.05(15')^2}{16} = 56.95'^K$

$P_{eff} = P_u + \alpha M_u = 627.7^K + 1.71(56.95'^K) = 725.08^K$

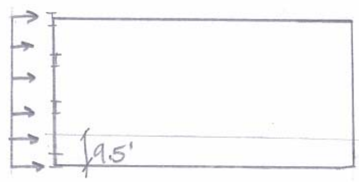
ASSUME $KL = 10.33' \rightarrow 11'$

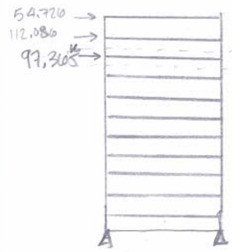
$\therefore$ USE W14x74 $\qquad \phi P_n = 753^K$

Appendix A.5

Portal Method

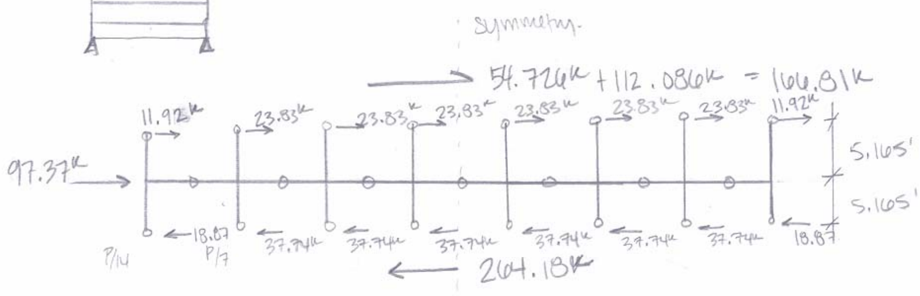
Seismic Controls.



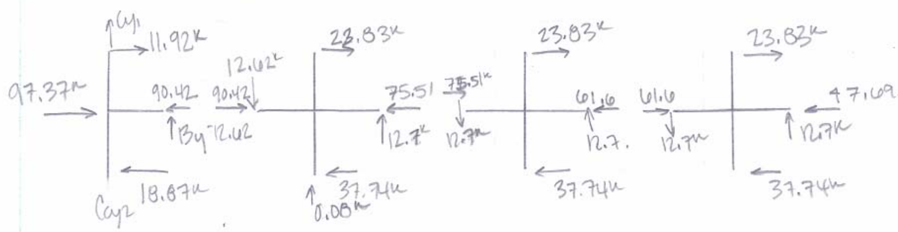


10th floor check.

$F = 97.365^k$



$\Sigma M = 54.726^k (20.66') + 112.086^k (10.33')$
$= 2288.49^k$

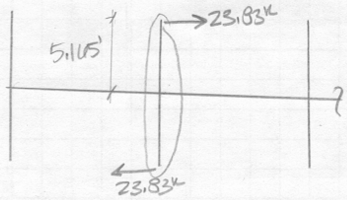


① $\Sigma M_{joint} = B_y(12.6') - 11.92^k(5.165^k) - 18.87^k(5.165^k) = 0$
$B_y = 12.62^K$

② $\Sigma M_{joint} = B_y(12.5') - 23.83^k(5.165') - 37.74^k(5.165') + 12.62(12.6') = 0$
$B_y = 12.7^k$

③ $\Sigma M_{joint}: B_y(12.5') - 23.83^k(5.165') + 12.7^k(12.5') - 37.74^k(5.165') = 0$
$B_y = 12.7^k$

Check Moments for Moment capacity of column:



$M = 23.83^k(5.165') = 123.08'^k$

Gravity Column Check:

$A_T = \frac{(25.25' + 25')(19')}{2}(2) = 954.75 ft^2$

$K_{LL\,(ext.col.)} = 4$

$A_I = A_T K_{LL} = 954.75 ft^2 (4)$

$A_I = 3819 ft^2$

LL Reduction

$L = L_o\left(0.25 + \frac{15}{\sqrt{3819 ft^2}}\right)$

$= 0.49 L_o > 0.4 \therefore$ use $0.49 L_o$.

$L = 0.49(44 psf) = 21.56 psf.$

• $P = 1.4D = 1.4(92 psf) = 128.8 psf.$

• $P = 1.2D + 1.6L + 0.5S = 1.2(92 psf) + 1.6(21.56 psf) + 0.5(16.8 psf.)$

$= 153.3 psf.$ ← controls. ★

$P_u = PA_T = (153.3 psf)(954.75 ft^2)$

$= 146.36^k$

Guess W14, d=14

$\alpha = 24/d = 12/14 = 1.71$

$M_u = \frac{WL^2}{16} = \frac{4.05 klf.(19')^2}{16} = 91.38'^k$

$M_{(portal\,method)} = 91.38'^k + 123.08'^k = 214.46'^k$

$P_{eff} = P_u + \alpha M_u = 146.36^k + 1.71(214.46'^k) = 513.09^k$

Assume $KL = 10.33' \rightarrow 11'$

$\therefore$ W14x61 $\phi P_n = 615^k > 513^k$

W14x43 used.