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Structural Option
Eight Tower Bridge
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November 7th, 2005

Structural Technical Report #1
Structural Concepts/Existing Conditions Report

Executive Summary

The following report is an in-depth summary and preliminary analysis of the structural system for Eight Tower Bridge, a 16-story steel high-rise office building located in Conshohocken, Pennsylvania. Completed in April of 2002, Eight Tower Bridge sits on the shore of the Schuylkill River, next to the Fayette Street Bridge, leading to both interstates I-476 and I-76. This prime location for a multi-tenant office building is less than 15 minutes outside of Centre City Philadelphia. The building was designed by the high profile architecture firm of Skidmore, Owings and Merrill, who have been responsible for such structures as the Sears Tower in Chicago, and are currently designing the new Freedom Tower in New York City. Eight Tower Bridge is the most recent office building to be constructed in the Conshohocken area by the real estate development company Oliver Tyrone Pulver Corporation. The company has built nearly $400 million worth of new office, commercial and retail space in the area over the past 10 years, adding nearly 1.2 million square feet of rentable space. The 315,000 square foot Eight Tower Bridge was the largest single function structure of the Tower Bridge buildings to be constructed, falling second in overall size to the mixed-use One Tower Bridge.

The scope of this report is limited to construction documents issued for construction on March 25th, 2001 and in some cases, revision bulletins one through seven. This report is intended to provide an overview of the existing structural system of the building, including information relative to design concepts and required loadings, as well as design assumptions. This report includes a summary of the building's structural components including the general floor framing, structural slabs, lateral load resisting system, foundation system, and bracing system. Spot checks have been completed for a typical floor beam, column and lateral braced frame. Additionally, both wind and seismic load analysis have been conducted on the structure to further analyze the effectiveness of the lateral reinforcing system. Copies of these calculations can be found in Appendices A and C, while sections, plan drawings and framing details can be found in Appendix B and within the body of this report. All loads for analysis have been developed through use of ASCE7-98, BOCA National Building Code and through use of construction documents.

Code and Code Requirements

Both the gravity and lateral structural systems of Eight Tower Bridge were designed in accordance with requirements set forth by the BOCA National Building Code, 1996 edition. Structural steel members were designed using AISC "Load and Resistance Factor Design Specification for Structural Steel." For the development of lateral load analysis for this report, load development procedures were taken from ASCE7-98, chapters 6 and 9. Material properties have been specified within the construction documents and adhere to ASTM and AISC standards for steel, and with ACI 318 standards for concrete construction.

Gravity and Lateral Loads

As mentioned in "Code and Code Requirements" above, the gravity and lateral loads for this report were developed using methods and standards set forth by ASCE7-98. Additional loading cases and requirements were obtained from the structural documents. A combination of loads from both sources provided the necessary loadings for the structural system design. The following loads were obtained from ASCE7-98 and are the primary loads used within the scope of this report:

Live Loads (psf):

- Typical Offices: 50
- Partitions: 15
- Lobbies/Corridors (1st floor): 100
- Lobbies/Corridors (above 1st floor): 80
- Stairs: 100
- Roof: 20
- Mechanical Rooms: 125

Dead Loads (psf):

- Superimposed Dead Load: 20
- Ceiling, Mechanical, Electrical and Plumbing: 5
- Carpet/Misc: 5
- External wall load: 150

Additional load cases have been specified in the structural documents, but are not used within the scope of this report. They are as follows:

Terrace at Level 15
-Superimposed Dead Load: 75
-Live Load: 50

Roof/Mechanical Penthouse Level-

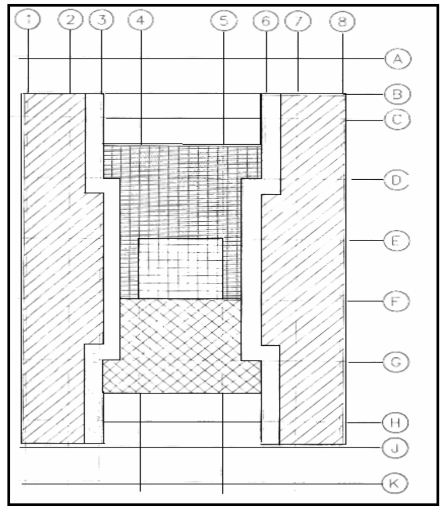
- Roof
  -Live Load: 30
  -Superimposed Dead Load: 12

- Mechanical Rooms
    - -Live Load: 300
    - -CMEP Dead Load: 8

Elevator Machine Room
    - -Live Load: 150
    - -Dead Load: 8
- Cooling Tower
    - -Live Load: 150
    - -Dead Load: 62
    - -CMEP Dead Load: 8
- Roof Drift Snow
    - -Dead Load: 62
    - -Superimposed Dead Load: 12

- Roof Load
-Mechanical Load
-Elevator Room Load
-Cooling Towers
-Snow Drift

Figure 1.1- Shows the load distribution over the penthouse level of the structure.



The additional loadings listed above under apply strictly to the areas specified. Areas to which to loads apply are hatched on the penthouse plan layout in figure 1.1 above.

Lateral Loads:

Wind and seismic lateral loads for the structure were derived from the methods set forth in ASCE7-02, chapters 6 and 9. A copy of ASCE7-98 was not obtained. A table summarizing the results of the both wind and seismic analysis in the E-W direction can be found below. For a complete table of all factors, assumptions and derivations, as well as the results of N-S seismic analysis, please refer to Appendix A.

Location:		Conshohocken, PA			
				Penthouse	
Dimension	N-S	118 ft	N-S	114 ft	
	E-W	196 ft	E-W	44 ft	
	Total Height, h	214 ft	Height	22 ft	
	Inter-story Height, h_s	12.08 ft			
Velocity Pressı	K_{zt}	1.0			(Table 6-4) Assume area is flat
	K_d	0.85			(Table 6-4)
	V	90 mph			(Figure 6-1)
	Group	II			Office Building
	Importance Factor, I	1.0			(Table 6-1)
	Exposure	B			Assume area is flat
	$q_z/K_z = 0.00256K_{zt}K_dV^2I =$	17.6256			

Table 1.1- Results of a wind load analysis

ASCE7-02 Chapter 6: Wind Analysis
Method 2- Analytical Procedure

Story Forces		Storey Shear		Moment	
N-S	E-W	N-S	E-W	N-S	E-W
(kips)	(kips)	(kips)	(kips)	(ft-kips)	(ft-kips)
11.34	25.92			2,426.93	5,545.55
30.18	27.14	11.34	25.92	5,799.92	5,216.01
32.53	47.50	41.52	53.06	5,859.47	8,556.52
32.13	46.84	74.05	100.56	5,398.38	7,871.23
31.72	46.18	106.18	147.40	4,946.93	7,201.70
31.31	45.51	137.90	193.58	4,505.30	6,548.23
30.84	44.73	169.21	239.09	4,064.39	5,895.95
30.33	43.90	200.05	283.83	3,631.06	5,256.08
29.82	43.07	230.38	327.73	3,210.00	4,636.27
29.28	42.18	260.20	370.80	2,797.65	4,030.71
28.67	41.18	289.47	412.98	2,393.11	3,437.93
27.91	39.95	318.14	454.16	1,992.96	2,852.34
27.10	38.62	346.05	494.11	1,607.57	2,290.89
26.24	37.21	373.15	532.73	1,239.62	1,757.97
25.14	35.42	399.39	569.95	884.05	1,245.31
23.69	33.04	424.54	605.36	546.80	762.63
22.44	30.99	448.23	638.41	246.80	340.88
	Total Shear	470.67	669.40		
			Total Moment	51,550.96	73,446.20

(Not to Scale)

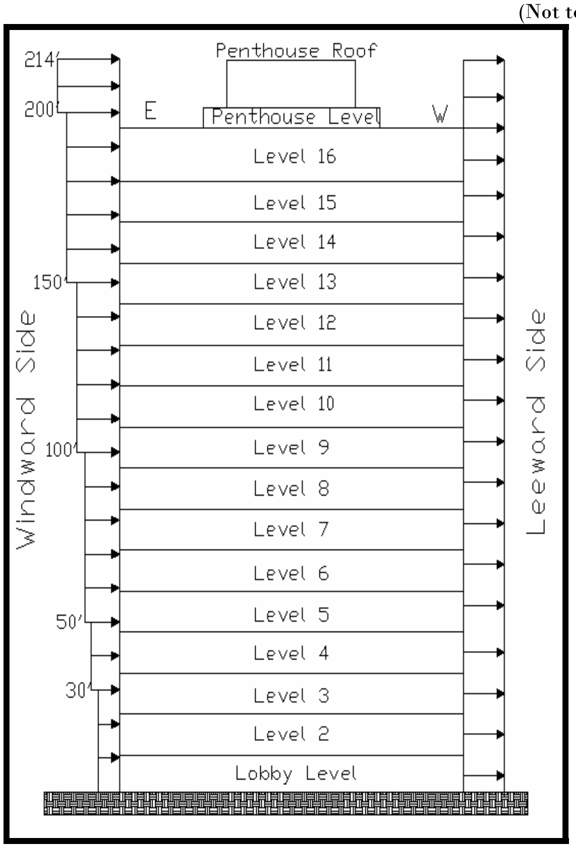


Figure 1.2-Vertical profile of wind loads

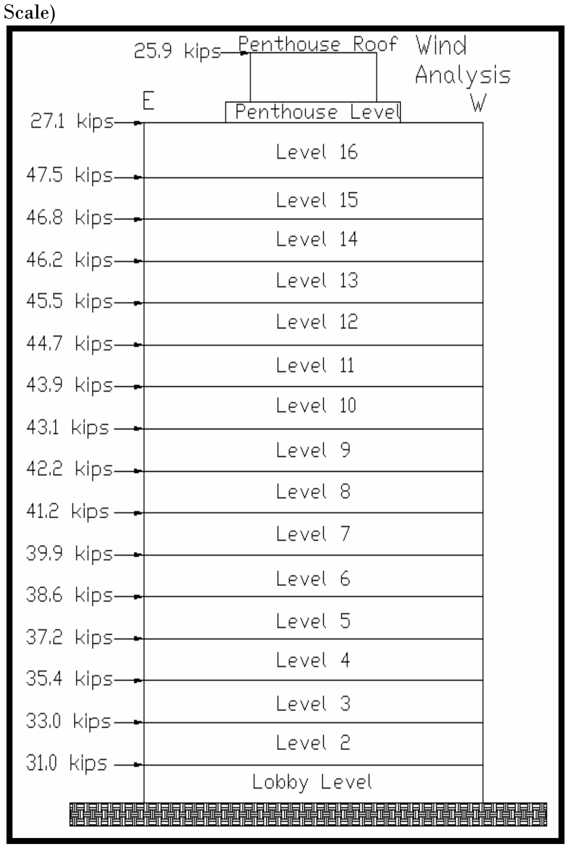


Figure 1.3- Wind loads resolved to floors (E-W)

(Not to Scale)

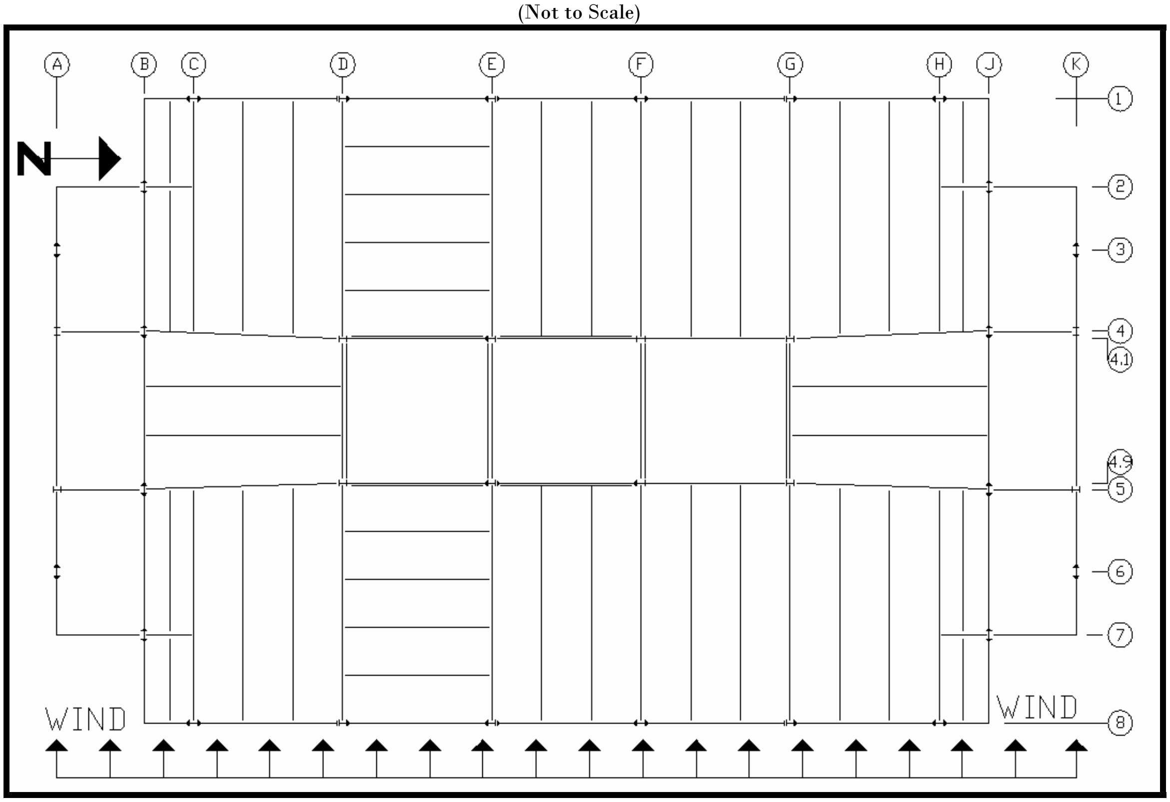


Figure 1.4- Wind force on E-W side of structure

Discussion of Wind Analysis

As mentioned above, the wind analysis of Eight Tower Bridge was conducted in accordance with the provisions set forth in ASCE7-02, Method 2. Several assumptions and interpolations were made during the wind analysis. To obtain the topographic factor (K_{zt}) and exposure category, the area surrounding the structure was assumed to be flat. Eight Tower Bridge was determined to be a use group of II (office building), and the lateral load resisting system was classified as "other structural system" in table 9.5.5.3.2.

To resolve the total wind pressures to forces on the 16th floor and mechanical penthouse roof, half of the 16th story height and half of the mechanical penthouse height were multiplied by the corresponding reduction in area of the mechanical penthouse in both directions. The penthouse roof forces were obtained by multiplying half the penthouse story height by its corresponding width, and multiplying by the directional wind pressure. This procedure resulted in an adjusted and more accurate wind forces for these levels. Figures 1.2 through 1.4 above further illustrate the results of the wind analysis.

Seismic Loads (ASCE7-02)

Table 2 : Vertical Distribution of Seismic Forces (E-W)

Seismic Base Shear, $V_{EW} = C_{s,EW}W =$ **1013** kips

Exponent $k_{EW} = 1 + (T_{EW} - 0.5)/(2.5 - 0.5) =$ 1.27

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	3100	192	2,443,848	0.235	238.6		45,853
16	1545	180	1,121,544	0.108	109.5	238.6	19,720
15	1545	168	1,026,681	0.099	100.2	348.1	16,842
14	1545	156	933,987	0.090	91.2	448.3	14,220
13	1545	144	843,202	0.081	82.3	539.5	11,843
12	1545	132	754,442	0.073	73.6	621.8	9,707
11	1545	120	667,841	0.064	65.2	695.4	7,805
10	1545	108	583,557	0.056	57.0	760.6	6,132
9	1545	96	501,778	0.048	49.0	817.6	4,681
8	1545	83	422,733	0.041	41.3	866.6	3,445
7	1545	71	346,708	0.033	33.8	907.8	2,417
6	1545	59	274,071	0.026	26.8	941.7	1,587
5	1545	47	205,321	0.020	20.0	968.4	947
4	1545	35	141,173	0.014	13.8	988.5	485
3	1545	23	82,771	0.008	8.1	1002.2	186
2	1545	11	32,334	0.003	3.2	1010.3	35
BASE						1013.5	
	Σ =		Σ =	Σ =	Σ =		Σ =
	26268		10381990	1.000	1013.5		145903

Table 1.3-E-W distributions of seismic forces

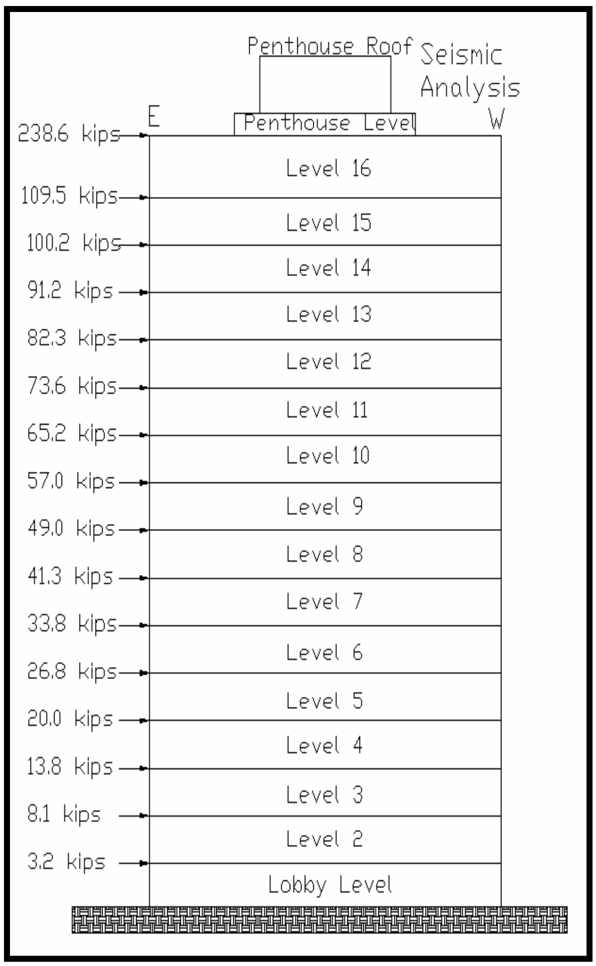


Figure 1.5-Seismic loads resolved to floors (E-W)

Discussion of Seismic Analysis

The seismic analysis of Eight Tower Bridge also required several making several assumptions. Due to the combination of both braced frames and moment resisting frames, a C_t and an x value of 0.02 and 0.75 respectively from table 9.5.5.3.2. These values determine that the approximate period in both directions was sufficient to classify the building as a rigid structure.

In order to simplify the seismic analysis, the structure was analyzed as a 16-story structure, with a height totaling only 193 feet and neglecting the 22 foot mechanical penthouse. Instead, the mechanical penthouse was calculated as a dead load on the 16th story of the building added to the total floor weight for that story. The analysis was then conducted as normal. Table 1.3 above displays the results of the seismic analysis. Figure 1.5 illustrates the resolution of each of the seismic forces to each floor level. A complete table of factors used during the seismic analysis, as well as the tabulation of building loads and weights can be found in Appendix A.

Description of Structural System

Eight Tower Bridge is a steel framed high-rise office tower. The structural system of supports 16 stories stretching 192' into the air. The superstructure also supports a mechanical penthouse level that rises 22' above the lower roof, topping the building out at 214'. The mechanical penthouse protects two massive cooling towers, a fan room, and an elevator machine room that controls the six general access elevators. The structural framing of Eight Tower Bridge provides strong lateral support, as well as opening the floor plan of the building in order to maximize rentable space to nearly 19,800 square feet per floor. In addition to mechanical roof loads, gravity floor loads, and lateral forces, the perimeter of the building must support a façade of pre-cast concrete panels and glazed windows.

Foundation

The building foundation system of Eight Tower Bridge consists of reinforced normal weight concrete pile caps ranging from 36" to 54" in depth. The pile caps range in dimension from approximately 7'x7' to 11'x10'. These pile caps are supported by four to eight 16" diameter auger–cast piles driven to an average bearing depth of thirteen feet below grade. The piles are made of normal weight concrete with a compressive strength of 4,000psi, and have been designed to a capacity of 100 tons.

The core of the building is supported by a 4'3" reinforced concrete mat foundation, supported by additional auger-cast piles. The entire building is supported by a total of 328 piles. Reinforced concrete grade beams connect all of the pile caps, as well as the interior core mat foundation.

Slab at the lobby level consists of a 5" concrete slab-on-grade reinforced with one layer of 4x4 welded wire fabric. The slab sits over a loose granular fill, which sits over compacted sub-grade soil. The inner core slab-on-grade is similar, but is cast 8” thick and has two layers of welded wire fabric as reinforcement. The lobby level also functions as a parking garage, eliminating the space for HVAC equipment underneath the building, thus forcing placement on the roof. The mechanical equipment loading creates an additional dead load on the structure, as well as adding to the complexity of wind and seismic calculations.

Superstructure Frame

Eight Tower Bridge is a steel framed structure. The framing in this system is fairly straight forward in design. The simple design has allowed for 13 of the 16 stories to be designed with a typical framing plan. Beam sizes for this system are most commonly W 18x40 and typically spanning 44’4” and spaced at 9’4”. Variations in this framing system occur at the extreme north and south end of the building, as well as in the buildings core due to mechanical system loads, and the insertion of six elevator towers through the height of the building. Exterior girders have been sized to W21x44 with spans ranging from 28’ to 12’. Interior girders are primarily sized as W18 shapes with weights ranging from 26 to 86 pounds per foot. Interior beam-to-column and beam-to-girder connections are typically simple shear connections. Beam-to-column connections in the moment resisting frames are fully welded moment connections, or as an alternate, have bolted end-plate moment resisting connections. All structural steel beams spanning over 35’ are designed with an upward camber and have been specified to ASTM A992 grade 50 steel. Figure 1.6 below shows a typical floor framing plan.

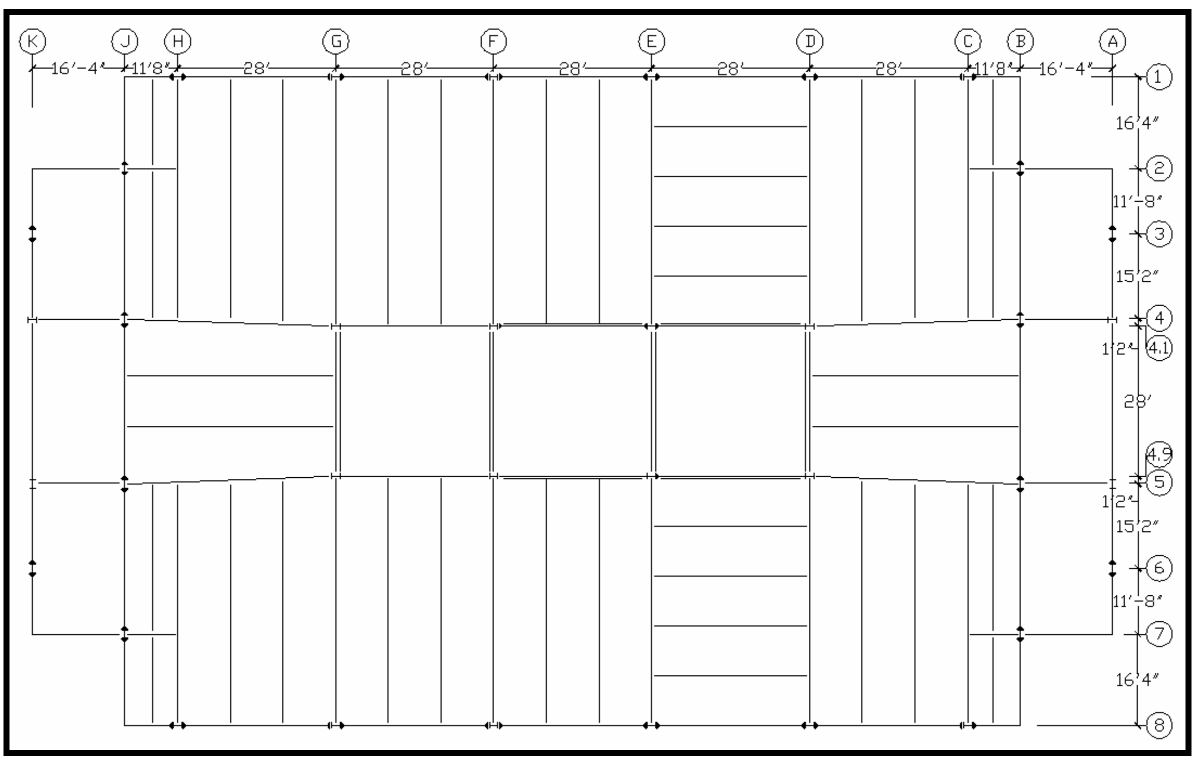


Figure 1.6- **The typical framing plan found at levels 3 to 15**

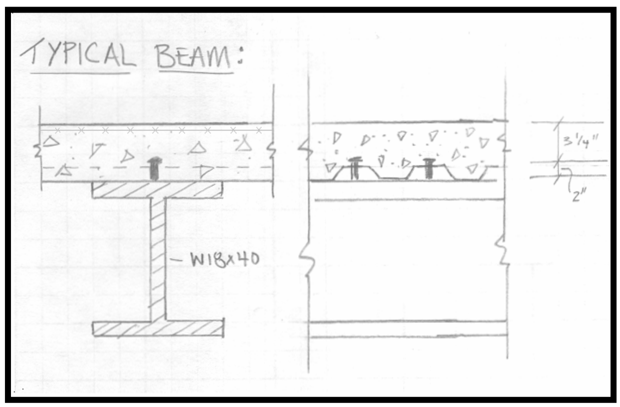
Lateral System

The lateral system of Eight Tower Bridge is actually two separate concentric frame systems. The inner framing structure is an 18-story core tower comprised of a combination of moment and braced frames. The braced frames span 28' along column lines D, E, F and G in the east-west dimension of the building. Additional braced frames span 56' along column lines 4.1 and 4.9 in the north-south direction between column lines D and F. The braced frames can be seen in the typical framing plan in Figure 1.6 above. The typical frame is made of W 14x90 through W 14x550 columns, W18x50 beam members, and braced diagonally with two 8 x 6 x ¾ welded angles of A36 Steel. The core tower supports the elevator machine room, as well as the mechanical fan room.

The outer frame is comprised of structural steel moment resisting frames located around the building perimeter. All structural steel is specified as ASTM A992 grade. These moment connections have been designed with single shear plate slip-critical connections in order for the beam to resist lateral and gravity loads and develop the total designed beam end reaction.

Structural Slab

Figure 1.7- Typical beam section and elevation



Eight Tower Bridge employs the use of reinforced concrete slab poured over metal deck for the flooring system. The typical floor slab is 5-1/4" thick with 3-1/4" light-weight concrete poured over 2" non-cellular metal deck. The system uses 6x6-W1.4xW1.4 welded-wire mesh and shear studs spaced along the span of the beam to develop a full composite structural slab. Designing the slab to act with full composite strength allows the W-shape floor beams to develop a larger moment capacity, thus capable of spanning longer distances. The ability of a beam to span a longer distance results in further column spacing, allowing for flexibility in the floor plan which is a desirable trait in office building design. The concrete slab described above is shown in Figure 1.7 and is used as primary flooring system in all office spaces.

Special Design Cases and Concerns

Additional slab systems were designed for the mechanical penthouse and mechanical fan room. The mechanical fan room located on each level of Eight Tower Bridge requires and 8" thick normal weight concrete slab with 2" deep metal deck. Reinforcing for this slab is specified as #5 bars spaced at 12" for both top and bottom reinforcing. The slab also acts in composite with W-shape floor beams, and also required shoring during construction.

A similar slab system was used for the mechanical penthouse. Differences include the increase in slab depth to 9" total, and reinforcing of #4 bars spaced at 12". The thickened slab and increase in reinforcing is required to support the large cooling towers which are treated as live loads. Additional thickened slabs at the penthouse level occur in the mechanical fan room and elevator machine room, where shoring was required during construction due to the increase in service loads.

Eight Tower Bridge also incorporates a window washing system onto the roof of the structure. Strategically placed davit pedestals comprised of 18"x1"x1' plates with ½" vertical stiffener plates have been attached at various locations on the rooftop. These davit pedestals will be used to lower window washing equipment along the side of the building, which will affect the loading on the structural members to which they are attached. The exact location and effects of this system on the structure was coordinated with a special consultant.

Structural Member Check

Typical Beam Check

A check of a floor beam found within a typical bay was conducted. The beam was arbitrarily chosen from a typical bay on the sixth level in between column lines 4.9 and 8, and G and F. The loads previously listed in this report were used to develop the member loading. These loads were factored using the equation 1.6L+1.2D. Live load reductions were applied where necessary. Although several different members could have been selected, a W18x40 beam was chosen in order to verify that the member does work for this framing system. The member was also checked for defection and results indicate that each of these typical beams must be fabricated with an upward camber. Detailed calculations can be found in Appendix C.

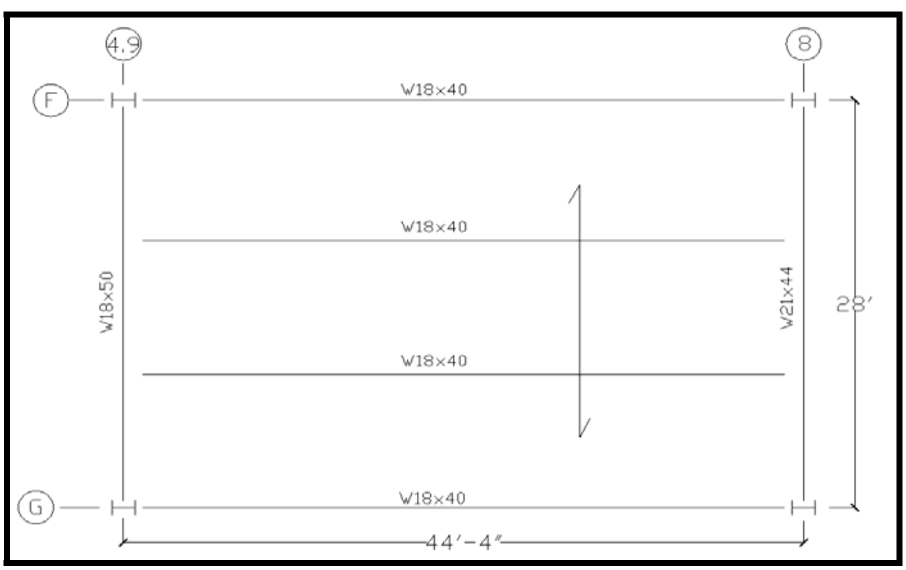
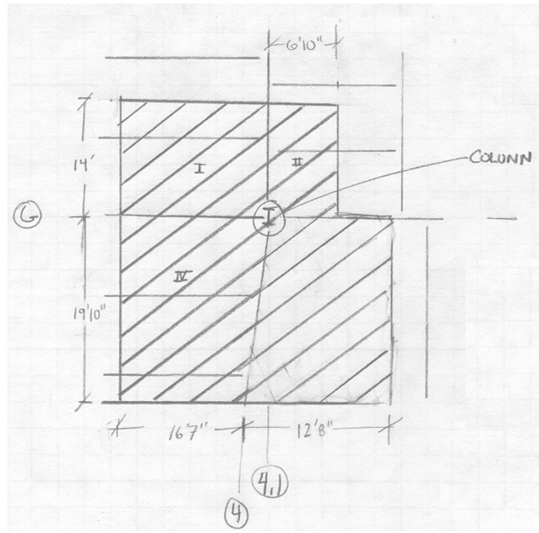


Figure 1.8- A typical bay from the 6th floor.

Typical Column Check

Figure 1.9- Layout of column analyzed



A column check was performed on level 6 of the structure at column mark G-4.1. Columns of eight Tower Bridge are typically two stories high with an average height of 24'2". However, the maximum un-braced length is only 12'1" due to the flooring systems acting as column bracing. A maximum live load reduction of 0.4 was used in development of column axial forces and moment. The column being analyzed supports some of the large mechanical dead loads on the roof, resulting in a column size of W14x311. This column is also part of the laterally braced frame that runs along column line G. However, these two factors were not taken into account in the spot check, resulting in a much smaller W 14x 82 column being selected as the initial size. Additionally, the area contributing to the column compression was not symmetrical. The dimensions of the tributary area for the column check are shown above in figure 1.9. Detailed calculations for the column check can be found in Appendix C.

Lateral Bracing Check

The lateral braced frame along column line F between column lines 4.1 and 4.9 was checked for ability to withstand the seismic and wind loads developed in the previous sections of this report. The frame was analyzed at level 6, where it was found that a total combined wind and seismic shear force of 1319kips was acting on the story. The frame being analyzed is just one of four like frames resisting shear and moment in the same direction. A reasonable assumption was determined that a quarter of that story shear was being resisted by the two A36 steel 8x6x3/4 angle members welded back to back. A detailed calculation is included in Appendix C. The frame is shown below in Figure 1.10.

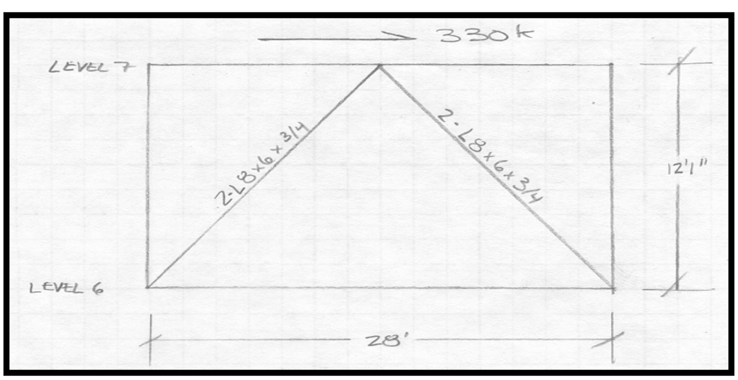


Figure 1.10-Analyzed braced frame on level 6

Appendix A

Table A.1- Basic wind analysis factors and pressure distribution

Location: **Conshohocken, PA**

Dimension				**Penthouse**	
N-S	**196** ft		N-S	**114** ft	
E-W	**118** ft		E-W	**44** ft	
Total Height, h	**214** ft		Height	**22** ft	
Inter-story Height, h_s	**12.08** ft				

Velocity Pressure		
K_{zt}	**1.0**	(Table 6-4) Assumed area is flat
K_d	**0.85**	(Table 6-4)
V	**90** mph	(Figure 6-1)
Group	**II**	Office Building
Importance Factor, I	**1.0**	(Table 6-1)
Exposure	**B**	Assumed area is flat
$q_z/K_z = 0.00256K_zK_dV^2I$ =	17.6256	

	N-S	E-W
Gust Factor Effect, G	**0.832**	**0.821**

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External Pressure Coefficients, C_p		Resultant pressure N-S	Resultant pressure E-W
	Windward	**0.8**	**0.8**
	Leeward	**-0.5**	**-0.36**

Story No.	z (ft)	K_z	q_z (lb/ft²)	q_h (lb/ft²)	$q_zC_pG - q_hC_pG$ (lb/ft²) N-S	$q_zC_pG - q_hC_pG$ (lb/ft²) E-W
Penthouse Roof	214	1.222	21.544	21.544	23.313	20.595
Penthouse Level	192	1.188	20.945	21.544	22.914	20.201
16	180	1.170	20.625	21.544	22.701	19.991
15	168	1.146	20.200	21.544	22.418	19.712
14	156	1.122	19.775	21.544	22.135	19.432
13	144	1.098	19.349	21.544	21.851	19.152
12	132	1.070	18.851	21.544	21.520	18.825
11	120	1.039	18.318	21.544	21.165	18.475
10	108	1.009	17.786	21.544	20.811	18.125
9	95.6	0.977	17.215	21.544	20.430	17.750
8	83.5	0.940	16.576	21.544	20.005	17.330
7	71.4	0.896	15.785	21.544	19.478	16.811
6	59.3	0.847	14.934	21.544	18.911	16.252
5	47.2	0.796	14.034	21.544	18.312	15.660
4	35.2	0.731	12.884	21.544	17.546	14.905
3	23.1	0.645	11.362	21.544	16.533	13.905
2	11	0.570	10.047	21.544	15.657	13.041
Base	0	0.570	10.047	21.544	15.657	13.041

Table A.2- Wind analysis results and force distribution, shear and moment

Location:		**Conshohocken, PA**				
					Penthouse	
Dimension	N-S	**118**	ft	N-S	**114**	ft
	E-W	**196**	ft	E-W	**44**	ft
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Velocity Pressu	K_{zt}	**1.0**		(Table 6-4) Assume area is flat
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	V	**90**	mph	(Figure 6-1)
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	Importance Factor, I	**1.0**		(Table 6-1)
	Exposure	**B**		Assume area is flat
	$q_z/K_z = 0.00256K_zK_dV^2I$ =	17.6256		

ASCE7-02 Chapter 6: Wind Analysis

Method 2- Analytical Procedure

Story Forces		Storey Shear		Moment	
N-S	**E-W**	**N-S**	**E-W**	**N-S**	**E-W**
(kips)	(kips)	(kips)	(kips)	(ft-kips)	(ft-kips)
11.28	25.83			2,414.13	5,525.38
30.02	27.04	11.28	25.83	5,769.32	5,197.03
32.36	47.33	41.30	52.87	5,828.56	8,525.39
31.96	46.67	73.66	100.20	5,369.90	7,842.59
31.55	46.01	105.62	146.87	4,920.83	7,175.50
31.15	45.35	137.17	192.88	4,481.53	6,524.41
30.67	44.57	168.32	238.22	4,042.95	5,874.50
30.17	43.74	198.99	282.79	3,611.91	5,236.95
29.66	42.92	229.16	326.54	3,193.07	4,619.41
29.12	42.03	258.82	369.45	2,782.89	4,016.05
28.52	41.03	287.95	411.48	2,380.48	3,425.42
27.77	39.80	316.46	452.51	1,982.45	2,841.97
26.96	38.48	344.23	492.32	1,599.09	2,282.55
26.10	37.08	371.18	530.79	1,233.08	1,751.57
25.01	35.29	397.29	567.87	879.38	1,240.78
23.57	32.92	422.30	603.16	543.92	759.86
22.32	30.88	445.86	636.08	245.50	339.64
	Total Shear	**468.18**	**666.96**		
			Total Moment	**51,278.99**	**73,179.01**

Table A.3- N-S Wind Gust Factor Effect

Gust Factor Effect Calculation

1. N-S Direction

$B =$	118 ft	$L/B = 1.661$
$L =$	196 ft	
$h =$	214 ft	
$C_t =$	**0.02**	(Table 9.5.5.3.2) Steel Moment Frames/Braced Frame Core
$x =$	**0.75**	
Estimated Frequency		$f = 1/C_t h^x = 0.894$
From Table 6-2	**Exposure B, Case 2**	
z_{min}	**30**	
c	**0.30**	
l	**320**	
$\bar{\epsilon}$	**0.33**	
$\bar{b}$	**0.45**	
$\bar{\alpha}$	**0.25**	
g_Q	**3.40**	(Given)
g_v	**3.40**	(Given)
β	**0.05**	
$0.6h =$	128.40	$> z_{min}$ (OK!)
$z =$	128.40	
$L_z = l(z/33)^{\bar{\epsilon}} =$		503.3
$I_z = c(33/z)^{1/6} =$		0.239
$Q = (1+0.63[(B+h)/L_z]^{0.63})^{-0.5} =$		0.821
$V_z = \bar{b}(z/33)^{\bar{\alpha}} V(88/60) =$		83.426
$n_1 = f =$		0.894
$\eta_h = 4.6 n_1 h/V_z =$		10.545
$\eta_B = 4.6 n_1 B/V_z =$		5.814
$\eta_L = 15.4 n_1 L/V_z =$		32.332
$R_h = 1/\eta_h - (1-e^{-2\eta_h})/2\eta_h^2 =$		0.090
$R_B = 1/\eta_B - (1-e^{-2\eta_B})/2\eta_B^2 =$		0.157
$R_L = 1/\eta_L - (1-e^{-2\eta_L})/2\eta_L^2 =$		0.030
$N_1 = \eta_1 L_z/V_z =$		5.391
$R_n = 7.47N_1/(1+10.3N_1)^{5/3} =$		0.048
$R = [(R_n R_h R_B(0.53+0.47R_L)/\beta]^{0.5}$		0.086
		4.163
$g_R R =$		0.360
		0.832

Table A.4- E-W Wind Gust Factor Effect

Gust Factor Effect Calculation

2. E-W Direction

$B =$	196 ft	$L/B = 0.602$
$L =$	118 ft	
$h =$	214 ft	
$C_t =$	**0.02**	(Table 9.5.5.3.2) Steel Moment Frames/Braced Frame Core
$x =$	**0.75**	
Estimated Frequency		$f = 1/C_t h^x = 0.894$
From Table 6-2	**Exposure B, Case 2**	
z_{min}	**30**	
c	**0.30**	
l	**320**	
$\bar{\epsilon}$	**0.33**	
$\bar{b}$	**0.45**	
$\bar{\alpha}$	**0.25**	
g_Q	**3.40**	(Given)
g_v	**3.40**	(Given)
β	**0.05**	
$0.6h =$	128.40	$> z_{min}$ (OK!)
$z =$	128.40	
$L_z = l(z/33)^{\bar{\epsilon}} =$		503.3
$I_z = c(33/z)^{1/6} =$		0.239
$Q = (1+0.63[(B+h)/L_z]^{0.63})^{-0.5} =$		0.802
$V_z = \bar{b}(z/33)^{\bar{\alpha}} V(88/60) =$		83.426
$n_1 = f =$		0.894
$\eta_h = 4.6 n_1 h/V_z =$		10.545
$\eta_B = 4.6 n_1 B/V_z =$		9.658
$\eta_L = 15.4 n_1 L/V_z =$		19.465
$R_h = 1/\eta_h - (1-e^{-2\eta_h})/2\eta_h^2 =$		0.090
$R_B = 1/\eta_B - (1-e^{-2\eta_B})/2\eta_B^2 =$		0.098
$R_L = 1/\eta_L - (1-e^{-2\eta_L})/2\eta_L^2 =$		0.050
$N_1 = \eta_1 L_z/V_z =$		5.391
$R_n = 7.47N_1/(1+10.3N_1)^{5/3} =$		0.048
$R = [(R_n R_h R_B(0.53+0.47R_L)/\beta]^{0.5} =$		0.069
$g_R = (2\ln(3600n_1))^{0.5} + 0.577/(2\ln(3600n_1))^{0.5} =$		4.163
$g_R R =$		0.287
$[1+1.7I_z((g_Q Q)^2+(g_R R)^2)^{0.5}]/(1+1.7g_v I_z) =$		**0.821**

<u>Table A.5-</u> Seismic analysis factor calculations

ASCE7-02 Chapter 9- Seismic Analysis

Reference					
	Building Location :			Conshohocken, Pennsylvania	
	Number of Stories :	N		16	
	Inter-story Height	h_s		12.08	
	Building Height :	h_n		193	ft
Table 1.1	Seismic Use Group :	I		I	
Table 9.1.4	Occupany Importance Factor :			1.00	
	Site Classification :			D	
Figure 9.4.1.1a	0.2s Acceleration :	S_S		0.31	g-s
Figure 9.4.1.1b	1s Acceleration :	S_1		0.08	g-s
Table 9.4.1.2.4a	Site Class Factor :	F_a		1.55	
Table 9.4.1.2.4b	Site Class Factor :	F_v		2.40	
	Adjusted Accelerations :	S_{MS}	$= F_a S_S$	0.481	g-s
		S_{M1}	$= F_v S_1$	0.180	g-s
	Design Spectral Response Accelerati	S_{DS}	$= (2/3)S_{MS}$	0.320	g-s
		S_{D1}	$= (2/3)S_{M1}$	0.120	g-s
Table 9.4.2.1a	Seismic Design Category :			B	
Table 9.4.2.1b	Both design category B				
	<u>N-S Direction</u>				
Table 9.5.2.2	Response Modification Factor :	R_{N-S}		3	
	Seismic Response Coefficient :	$C_{s, N-S}$	$= S_{DS}/(R_{N-S}/I)$	**0.107**	
Table 9.5.5.3.2		$C_{T, N-S}$		0.028	
Table 9.5.5.3.2	(moment frames only)	x		0.80	
	Approximate Period of Structure :	T_{N-S}	$= C_{T, N-S} h_n^x$	1.89	
	Seismic Response Coefficient need				
	not be greater than	$C_{s\,max, N-S}$	$S_{D1}/T(R_{N-S}/I)$	**0.021**	
	and	$C_{s\,min}$	$= 0.044 I S_{DS}$	**0.0141**	
	Seismic Response Coefficient ($C_{s, N-S}$)			**0.021**	
	<u>E-W Direction</u>				
Table 9.5.2.2	Response Modification Factor :	R_{N-S}		3	
	Seismic Response Coefficient :	$C_{s, EW}$	$= S_{DS}/(R_{E-W}/I)$	**0.107**	
Table 9.5.5.3.2		$C_{T, EW}$		0.02	
Table 9.5.5.3.2	(moment and braced frame)	x		0.75	
	Approximate Period of Structure :	T_{N-S}	$= C_{T, E-W} h_n^x$	1.04	
	Seismic Response Coefficient need				
	not be greater than	$C_{s\,max, N-S}$	$S_{D1}/T(R_{E-W}/I)$	**0.039**	
	and	$C_{s\,min}$	$= 0.044 I S_{DS}$	**0.0141**	
	Seismic Response Coefficient ($C_{s, E-W}$)			**0.039**	

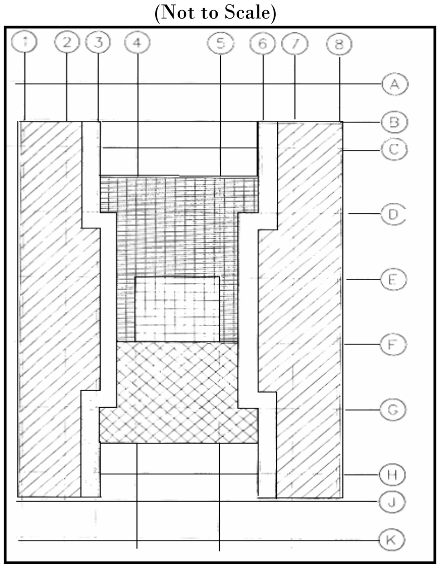
<u>Table A.6-</u> Building Weight Calculations

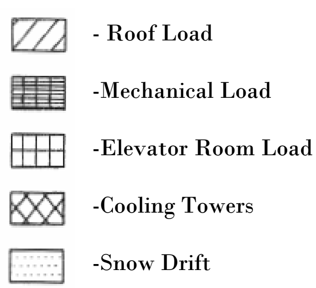
ASCE7-02 Chapter 9- Seismic Analysis

Load Summary for Building Weight

i. Roof :				
Dead				
Membrane	1.0	psf		
Rigid Insulation	2.0	psf		
Metal Roof Deck	2.0	psf		
Roof Framing	12.0	psf		
M&E Services	5.0	psf		
TOTAL q_{roof} :	**22**	psf of roof area		
ii. All other Floors :				
Dead				
Flooring	1.0	psf		
Concrete Slab	2.0	psf		
3" Metal Deck	2.0	psf		
Structural Steel	5.0	psf		
0.5" Drywall Ceiling	5.0	psf		
M&E Services	5.0	psf		
Live				
Moveable Partition	20.0	psf		
Office Live Load	30.0			
TOTAL q_{floor} :	**70.0**	psf of floor area		
iii. Perimeter Wall:				
Dead				
Exterior Wall, q_{wall} :	**180.0**	psf		
iv. Weight of Roof	(PSF)	Area		
Factored Roof Load	68		7500 ft^2	510000
Mechanical Load	330		2500 ft^2	825000
Elevator	250		790 ft^2	197500
Cooling Tower	350		2050 ft^2	717500
Snow	120		4000 ft^2	480000
Additional Dead Load	22		16840 ft^2	370480
			Total Roof Load (kips)	3100
Building Dimensions:				
Length	196	ft		
Width	118	ft		
Gross Floor Area	23128	ft^2		
Area of Voids	1065	ft^2		
Total Floor Area	22063	ft^2		
Weight of each floor	1545	kips		
$w_{floors} = (N-1)w_{per\ floor}$ =	23168	kips		
Total Building Weight	26268	kips		

(Not to Scale)





<u>Figure A.1</u>- Shows the load distribution over the penthouse level of the structure. This distribution was used to determine the weight of the roof for the seismic analysis.

Appendix B

(Drawings not to scale)

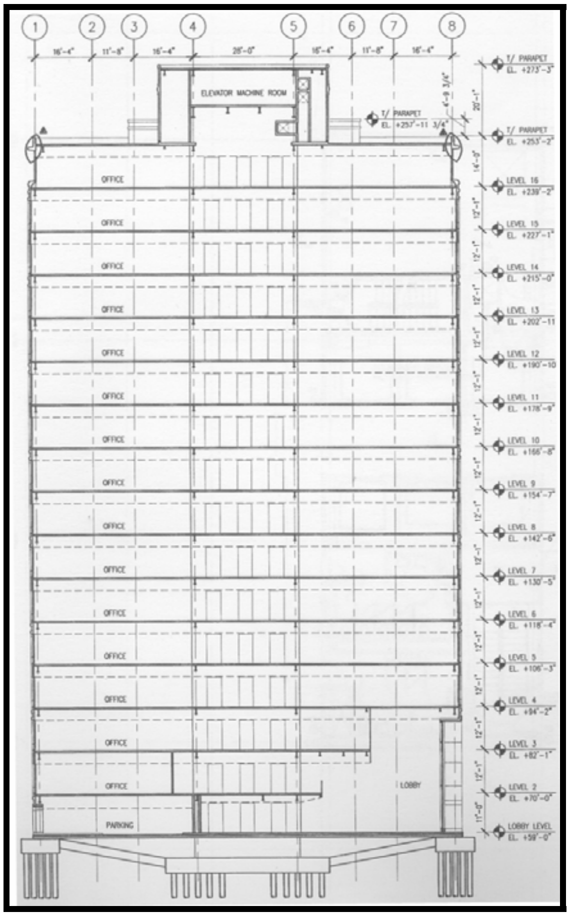


Figure B.1- East/West Section

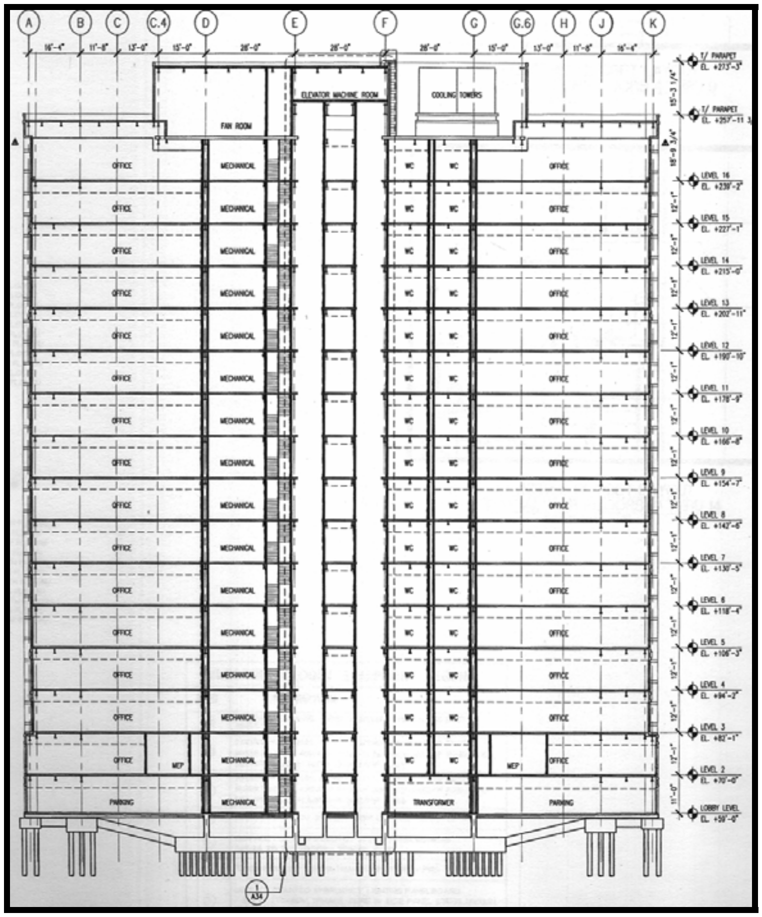


Figure B.2- North/South Section

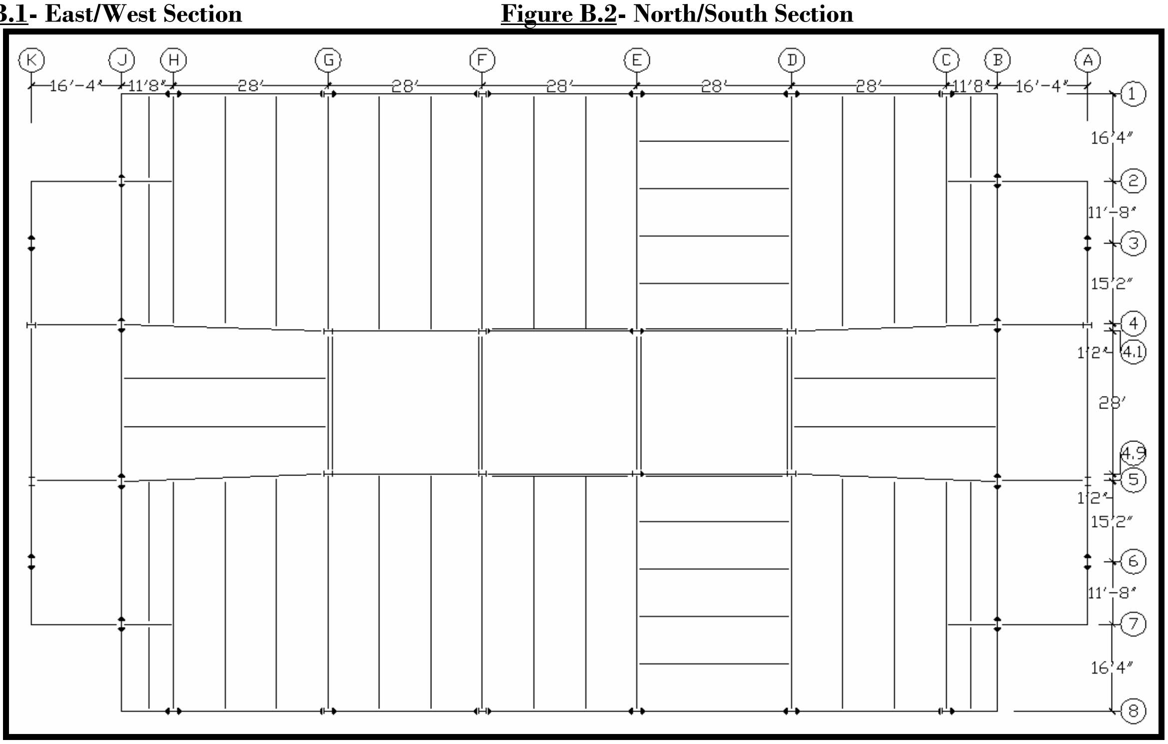
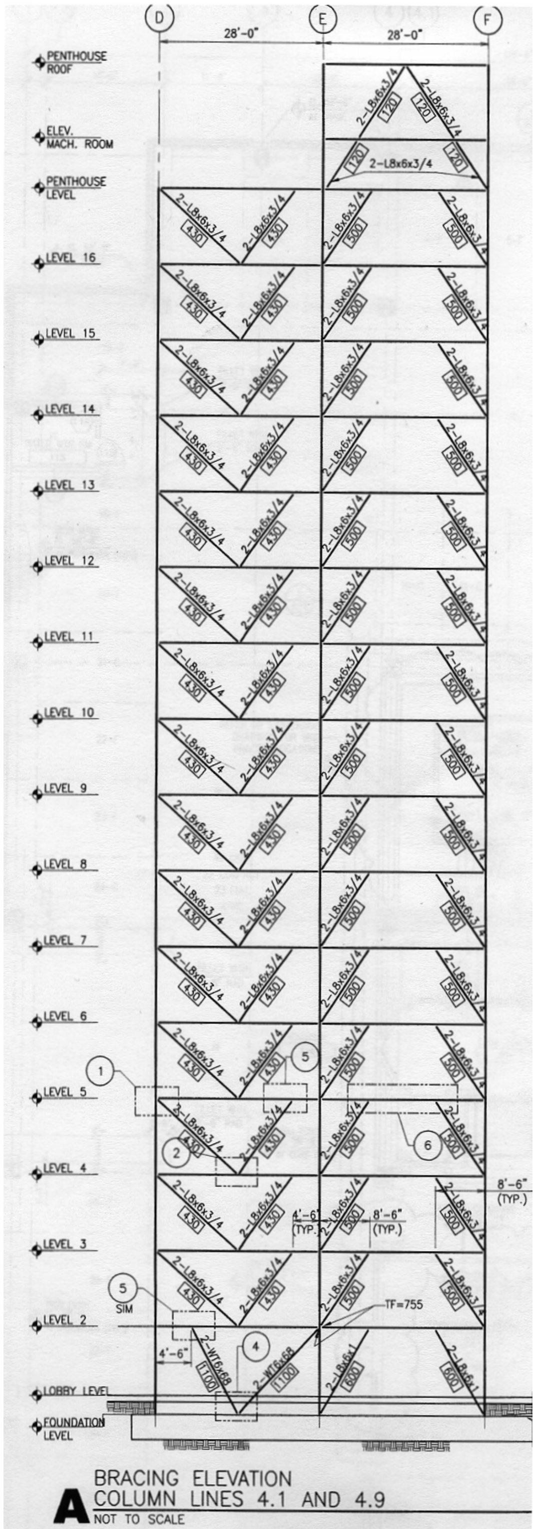
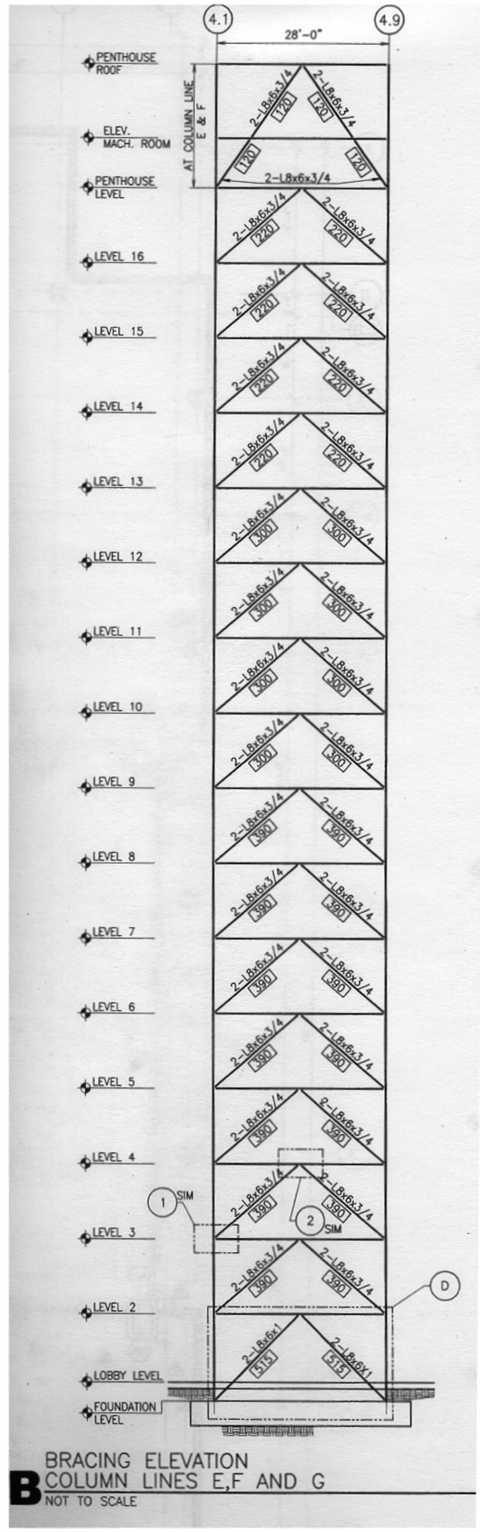


Figure B.3- Typical Framing Plan (Floors 3-15)

<u>Figures B.4 and B.5</u>- Typical Braced Frame elevations





Appendix C

Beam Check

CHRIS McCUNE | BEAM CHECK | EIGHT TOWER BRIDGE | 1/3

BEAM CHECK:

LIVE LOAD: 50 psf OFFICE
15 psf PARTITIONS
TOTAL 65 psf

DEAD LOAD: 20 psf SUPERIMPOSED
5 psf CEILING, MECH, ELEC, PLUMB
5 psf CARPET / MISC.
TOTAL 30 psf

CONC. SLAB: 115 pcf $(3\text{''}/4'' + 2''/2)$ = 40.7 psf

LIVE LOAD REDUCTION: (LEVEL 6)

$L = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$ $\quad A_I = 2A_{TRIB}$

$L = L_o\left(.25 + \frac{15}{\sqrt{827}}\right)$ $\quad A_I = 2(9.33')(44.33')$

$L = .35 L_o \not< .6 L_o$ $\quad A_I = 827\ ft^2$

∴ OK TO USE REDUCTION OF 0.6 L.

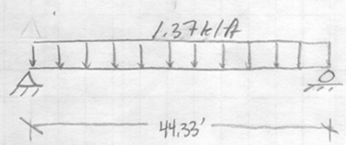
TOTAL FACTORED LOAD:

$W_u = 1.6L + 1.2D$

$W_u = 1.6(.6(65 psf)) + 1.2(30 psf + 40.7 psf) \cdot 9.33'/1000$

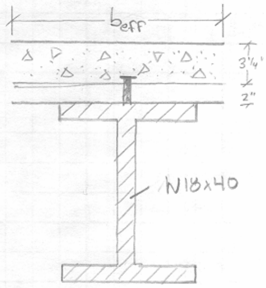
$W_u = 1.37\ k/ft$

SIMPLE BEAM:



Chris McCune | Beam Check | Eight Tower Bridge | 2/3

BEAM CHECK (CONT.)



$W_u = 1.18$ k/ft

ASTM 992
$f'_c = 4000$ psi
$F_y = 50$ ksi

$$M_u = \frac{wl^2}{8}$$

$$M_u = \frac{(1.37\ k/ft)(44.33^2)}{8}$$

$$M_u = 337.4\ ft.k$$

$$b_{eff} = \begin{cases} 1/4(44.33 \cdot 12) = 133 in \\ 9.33 \cdot 12 = 112 in \end{cases}$$

$b_{eff} = 112$ in

ASSUME: $a = 1"$

$Y_2 = 5.25" - 1"/2 = 4.75"$

TABLE 5-14 W18x40 $\phi_b M_p = 294$ ft.k < 337.4 ft.k

$\therefore$ CHECK COMPOSITE ACTION

$Y_2 = 4.75"$

TABLE 5-14 (p 5-143) LRFD
- IN W18x40, PNA CAN BE ANYWHERE IN SECTION AND STILL MEET $M_u < \phi_b M_n$
→ TRY LOCATION 4

Y_2	$\phi_b M_n$
4.5"	483
5"	496

$$\phi_b M_n = 483 + \frac{496 - 483}{2} = 489.5\ ft.k$$

@ LOCATION 4 $\Sigma Q_n = 353$ k

CHECK $a = 1"$

$$C_c = .85 f'_c b a = \Sigma Q_n$$

$$a = \frac{\Sigma Q_n}{.85 f'_c b} = \frac{353^k}{.85(4ksi)(112in)} = 0.93 in < 1"\ \therefore OK$$

$$Y_2 = 5\tfrac{1}{4}" - \frac{.93"}{2} = 4.78 in \rightarrow Y_2 = 5 in$$

TABLE 5-14 @ LOCATION 4, $Y_2 = 5$ in $M_n = 496$'k > 337.45'k

$\therefore$ OK

BEAM CHECK (CONT):

$\phi_b M_n = 496'k \qquad \Sigma Q_n = 353\,k$

SHEAR STUD CAPACITY: 21^k/STUD

OF STUDS $= \frac{353k}{21^k} = 16.8$ OR 17 STUDS

DEFLECTION CHECK:

$\Delta = \frac{5\,w\ell^4}{384\,EI}$ $\qquad 1.37\,k/ft \cdot \frac{1ft}{12in} = .098\,k/in$

$\Delta = \frac{5(.114\,k/in)(532\,in)^4}{384\,(29000)\,(612\,in^4)} = 5.75\,in$

$\ell/360 = 1.5''$ $\therefore$ MUST CAMBER!

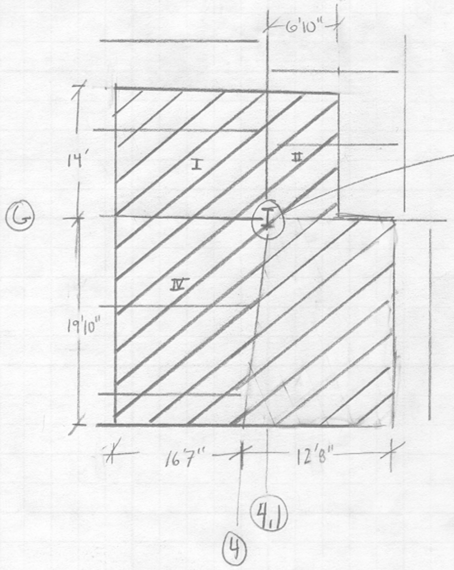
COMMENTS:

Column Check

CHRIS McCUNE | COLUMN CHECK | EIGHT TOWER BRIDGE | 1/2

COLUMN CHECK FOR COLUMN MARK G-4.1, LEVEL 6/7

↳ TWO STORY COLUMN



$A_{TRIB} = (14')(16.6') + (14')(6.84')$

$+ (19.8')(16.6') + (19.8')(12.66')$

$A_{TRIB} = 907\ ft^2$

9 STORIES ABOVE COLUMN

$A_{TRIB} = 9(907\ ft^2) = 8163\ ft^2$

$A_I = 4A_{TRIB} = 4(8163\ ft^2) = 32{,}652\ ft^2$

LL REDUCTION:

$L = L_o\left(.25 + \frac{15}{\sqrt{32{,}652}}\right) = .33 < .4$

→ .4 CONTROLS DESIGN

LOADS:

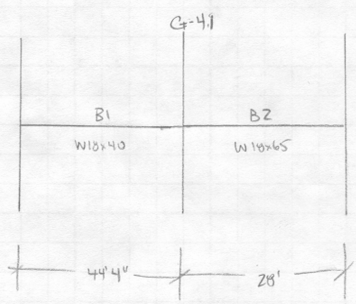
LIVE LOAD: 50 PSF (OFFICE)
15 PSF (PARTITIONS)
65 PSF

DEAD: 20 PSF (SDL)
5 PSF CMEP
5 PSF MISC. CARPET
30 PSF

$1.6(.4(65\ PSF)) + 1.2(30\ PSF) = 77.6\ PSF$

$P_{FLOOR} = (77.6\ PSF)(8163\ FT^2) = 633.4\ KIPS$

MOMENT CALCULATION:

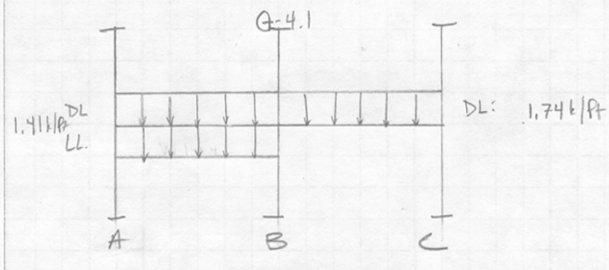


BEAM 1 LOAD: $(1.37\ k/ft) + (.04\ k/ft) = 1.41\ k/ft$

BEAM 2 LOAD: DEAD LOAD ONLY

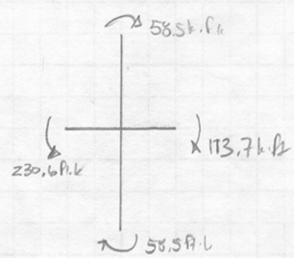
$1.2(30\ psf + 40.7\ psf) \cdot 19.83'/_{1000} + .005\ k/ft$

$\omega_D = 1.74\ k/ft$



$FEM_{BA} = \frac{(1.41k/ft)(44.3^2)}{12}$ $\qquad$ $FEM_{BC} = \frac{(1.74k/ft)(28'^2)}{12}$

$FEM_{BA} = 230.6$ ft.k $\qquad$ $FEM_{BC} = 113.7$ k.ft



$M_u = \frac{\Delta FEM}{2} = \frac{230.6 - 113.7}{2}$

$M_u = 58.5$ k.ft

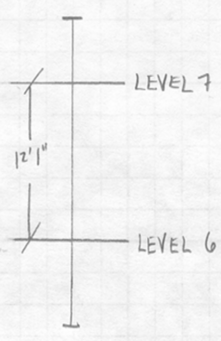
DESIGN COLUMN G-4.1

$P_u = 633.4$ kips $\qquad$ $M_u = 58.5$ k.ft

$P_{EFF} = P_u + \alpha M_u$ $\qquad$ $\alpha = \frac{24}{d} = \frac{24}{14} = 1.71$

$P_{EFF} = 633.4k + 1.71(58.5)$

$P_{EFF} = 733.4$ k



USE $KL = 12'$

LRFD P 4-23 $\qquad$ W14X82 $\quad \phi_c P_n = 797^k > 733.4^k$

ACTUAL COLUMN SIZE - W14x311 $\phi_c P_n = 3560^k$

CONCLUSION: THE LARGE MECHANICAL PENTHOUSE LOADS WERE EXCLUDED FROM THIS CALCULATION, RESULTING IN A MUCH SMALLER COLUMN SIZE

Lateral Frame Check

CHRIS McCUNE | LAT. SYSTEM CHECK | EIGHT TOWER BRIDGE | 1/1

LATERAL SYSTEM CHECK:

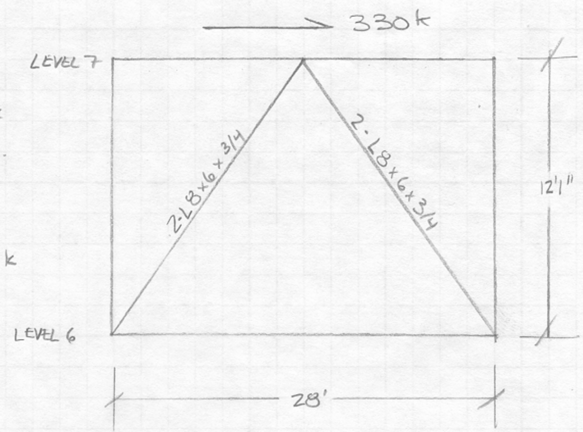
CHECK OF BRACED FRAME ALONG COLUMN LINE (F) BETWEEN COLUMN LINES (4.1) AND (4.9) ON LEVEL 6.

STORY WIND SHEAR @ 7: 411.28 k

STORY SEISMIC SHEAR @ 7: 907.8 k

TOTAL SHEAR: 1319^k

ASTM 36 STEEL
F_y = 36 ksi



THERE ARE 4 BRACED FRAMES RESISTING SHEAR IN E-W DIRECTION. EACH FRAME IS ASSUMED TO TAKE 1/4 OF THE LOAD.

$1319^k / 4 = 330^k$

$A_{g_{REQD}} = \dfrac{330^k}{36\ ksi} = 9.16\ in^2$

TABLE 1-7, p 1-34 LRFD.

A_g OF 2L-8x6x3/4 = 2(9.99 in)

A_g = 19.98 in^2 > 9.16 in^2