Christopher McCune
Structural Option
Eight Tower Bridge
Faculty Advisor: Dr. Hanagan
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Structural Technical Report #3
Lateral System Analysis

Executive Summary

This report is an in depth analysis of the lateral load resisting system of Eight Tower Bridge. The first section of the report is a brief introduction and overview of the building superstructure. The next section is an in-depth look at the lateral system of Eight Tower Bridge. This section includes a detailed description of the type of system used, typical framing members, and some of the benefits of the system. The report goes on to review the seismic and wind analysis conducted for Technical Assignment 1, "Structural Concepts/Existing Conditions Report". This section describes changes that were made to the original analysis, as well as compares the two separate loading cases. Methods used to conduct both of the analysis are again derived from ASCE7-02. The final section includes a discussion of the lateral load distribution within Eight Tower Bridge, and also describes the analysis of the system conducted in the structural modeling program, ETABS. The computer model was used to analyze the structural behavior of the building under wind and seismic loads. The computer model was helpful in obtaining shear, moment, and drift data for Eight Tower Bridge, as well as being able to view any deformed shapes through animation of the model.

The wind and seismic load analysis from Technical Assignment 1 were reviewed and error corrected in order to provide a more accurate results. The wind analysis was conducted again through ASCE7-02, Chapter 6, Method 2. A change in C_p factors resulted in a more accurate wind analysis in regards to the long and short sides of the building, as the short side was determined to have stronger wind forces. The seismic analysis was reviewed and a more accurate assumption of the total building weight was determined, as well as the removal of live loads from the seismic calculations. The removal of the live loads was accidentally overlooked in the first technical report.

Finally, an 3D frame model of Eight Tower Bridge was constructed in the computer modeling software package, ETABS. Both seismic and wind load cases were analyzed through this program. The lateral load resisting system was checked for strength as well as drift. The model was constructed using the structural documents provided from Skidmore, Owings and Merril, LLP. Certain assumptions were in creation of the model to focus on key members of the lateral system.

Introduction

General Overview

Eight Tower Bridge

Eight Tower Bridge is a 16 story high-rise office tower located outside of Philadelphia in Conshohocken, Pennsylvania. The office tower provides nearly 315,000 total square feet of office space on levels 2 through 16, while the ground level houses the entrance lobby, parking for nearly 50 vehicles, and a small space for a retail tenant. In addition to the 16 story office tower, a mechanical penthouse has been placed on the building roof, housing two large cooling towers, a mechanical room, and an elevator machine room.

The superstructure system for Eight Tower Bridge is comprised of entirely steel members. Typical bays are 28'x44'4" with wide flange beams spanning in the long direction. These members support a 3-1/4" lightweight concrete slab poured over 2" composite deck. The slab obtains composite action through the use of 4"-3/4" diameter shear studs, spaced evenly along the length of each member.

The lateral reinforcing system consists of both moment resisting frames around the building perimeter, and laterally braced frames in combination with moment resisting connections at the building core. The lateral system will be the main focus of this report.

Lateral Reinforcing System

The lateral system of Eight Tower Bridge is actually two separate concentric steel frame systems. The inner framing structure is an 18-story core tower comprised of braced frames in combination with moment resisting connections at various points within the frame. There are six frames located at the core the building. The four frames located along column lines D, E, F and G help resist the wind and seismic lateral forces in the East-West direction and are 28 feet in length, or the length of a typical bay. The remaining two frames are located along column lines 4.1 and 4.9 between lines D and F, and help resist the lateral loads created in the North-South Direction. These frames are twice the length of the frames mentioned above, spanning two typical bay lengths. Each frame is connected to the building foundation through ¾" steel anchor rods extending 2'6" into 4' pilecaps. Frame columns are encased in concrete at the structure base to further resist the overturning moment created by lateral loads.

The East-West lateral force resisting frames are comprised of columns ranging in size from W14x550 at the frame base to W 14x90 at the frame top. Horizontal members are W18x65 and W18x50 beams with typical pin connections at either end. The diagonal bracing for these frames consists of two 8x6x3/4" double angles pin connected to a ¾" gusset plate at each corner with slip-critical type connections arranged in a "V" frame formation. The double angle braces are oriented with the long legs back to back.

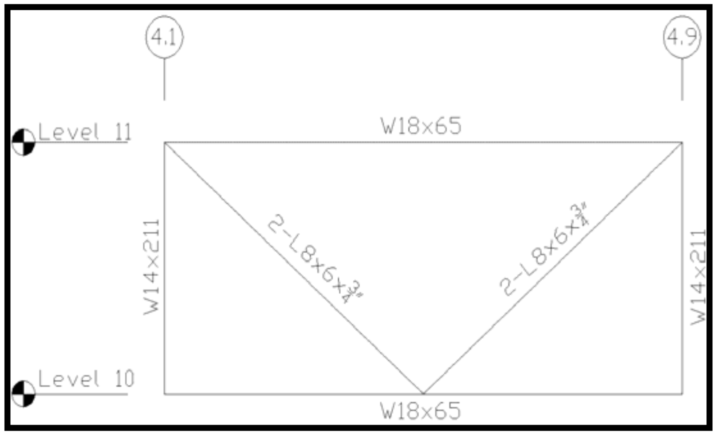


Figure 1.1- Typical braced frame along column line D

The double angle bracing members described above are typical diagonal bracing members throughout the frame with the exception of the bottom level. The bracing at the bottom of the frames along column lines D, E, F and G consists of two L-8x6x1" angle brackets; a ¼" increase in thickness over the typical frames diagonals, most likely to resist the total base shear and overturning moment. The frames along column lines 4.1 and 4.9 between D and E actually use two WT sections as the diagonal bracing rather than back to back angle shapes.

The frames in the North-South direction span the length of two bays and consist of similar column members, but W14x132 and W18x50 beam members with moment connections at either end. The reason for the variation in member size is due to the bays that frame into them. The W14x132 beam that spans each of these frames between column lines D and E does not carry any loads from the adjacent bay, as the floor beams are oriented perpendicular to those of a typical bay. Diagonal bracing of these frames consists of two L6x8x3/4" double angles in two different arrangements. The portion of the frames between column lines D and E have the diagonal braces arranged in a "V" frame formation, but with eccentrically placed bracing. The diagonal bracing on the frame between column lines E and F are also placed eccentrically and can be seen in Figure 1.2 below.

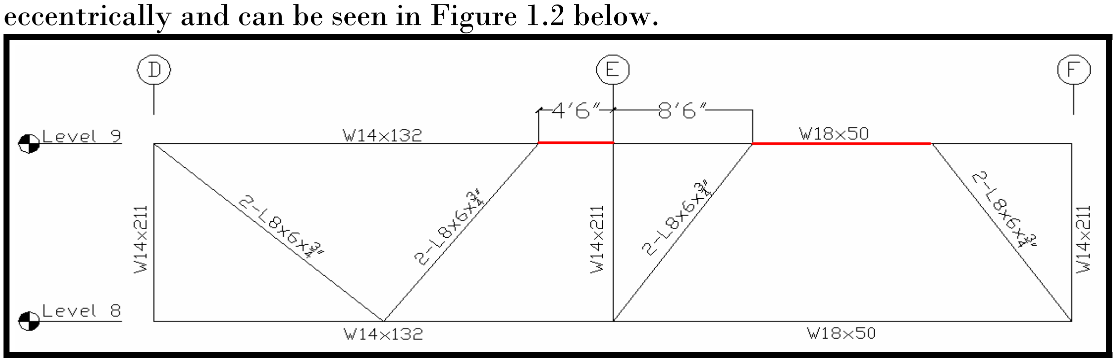


Figure 1.2- Typical braced frame along column lines 4.1 and 4.9

Eccentric diagonal bracing of these frames combines the strength of both moment frames and braced frames. They provide the strength and stiffness of a braced frame, and the open space between joints (highlighted in red above) allow for energy absorption under cyclical loading, which is a benefit when sustaining seismic loads. However, perhaps the most practical use for eccentrically braced frames in the lateral system of Eight Tower Bridge is to allow for doorway openings to the stair tower found between column lines 4.1 and 4.9, and D and E, and also to provide an opening to the elevator lobby found on each level between column lines E and F. Refer to the typical floor plan found in Appendix A.

The exterior framing structure is made up of entirely moment resisting frames. The exterior frames of the building are comprised of columns ranging in size from W14x176 to W14x90, with a majority of them being sized at W14x61. The horizontal frame members are W21x44 beams with moment resisting connections at either end. The moment resisting frames retain good ductile quality and are more flexible than the braced frames found at the building interior, which is important to consider when dealing with the ability of the building envelope to flex when sustaining lateral loads. However, moment frames may allow too much deflection, often resulting in non-structural damage such as window cracking and cracking of pre-cast panels and in worst cases, structural failure. The diagonal braced frames at the interior resist the excessive deformations and racking stresses that can be found in moment frames.

The analysis conducted in this report has been focused to the braced frames found at the core of Eight Tower Bridge. It has been assumed that the braced frame core of the structure resists most of the lateral loads.

Lateral Load Development

Wind Loading

A wind analysis of Eight Tower Bridge was previously conducted in Technical Assignment 1, "Structural Concepts/Existing Conditions Report". Wind loads were developed 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, due to it's combination of both moment and laterally braced frames.

The analysis conducted in Technical Assignment 1 was found to have multiple errors, which have been corrected for the analysis in this report. The leeward external pressure coefficients (C_p) for the N-S direction and E-W direction were accidentally switched. The calculation for each C_p is as follows:

N-S: L/B= 196/118= 1.66 C_p = -0.368 [ASCE07-02 Figure 6-6]
E-W: L/B=118/196= .602 C_p = -0.50 [ASCE07-02 Figure 6-6]

The following adjustments resulted in the shear resultant forces displayed in Table 1.1 below:

ASCE7-02 Chapter 6: Wind Analysis Method 2- Analytical Procedure					
Story No.	z (ft)	Story Forces		Story Shear	
		N-S (kips)	E-W (kips)	N-S (kips)	E-W (kips)
Penthouse Roof	214	10.14	28.84		
Penthouse Level	192	26.92	30.26	10.14	28.84
16	180	28.99	53.03	37.06	59.10
15	168	28.58	52.37	66.05	112.13
14	156	28.18	51.71	94.63	164.50
13	144	27.77	51.04	122.81	216.21
12	132	27.30	50.27	150.58	267.25
11	120	26.80	49.44	177.88	317.52
10	108	25.75	48.61	204.68	366.96
9	96	25.14	47.72	230.43	415.57
8	83	24.39	46.73	255.57	463.29
7	71	23.58	45.50	279.96	510.02
6	59	22.73	44.18	303.54	555.52
5	47	21.64	42.78	326.27	599.70
4	35	20.19	40.99	347.91	642.48
3	23	18.94	38.62	368.10	683.47
2	11	18.94	36.57	387.04	722.09
Base	0		Total Shear	405.98	758.66

Table1.1- Story shears from wind analysis

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.

Seismic Loading

A seismic analysis of Eight Tower Bridge was also conducted in Technical Report 1. Loads were developed using methods set forth in ASCE07-02, Chapter 9. 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 were obtained from table 9.5.5.3.2 in ASCE07-02. These values determine that the approximate period in both directions was sufficient to classify the building as a rigid structure.

The seismic analysis conducted in Technical Assignment 1 was reviewed and found to have errors that yielded a larger base shear, and consequently larger story shears. Error was involved when interpreting the site class factors F_a and F_v, resulting in a N-S base shear of 556 kips and a E-W base shear of 1013 kips. After determining the value of these site class factors to be 1.0 rather than values nearly twice that, the resulting base shears came out to be 179.9 kips and 318 kips in the N-S and E-W directions respectively. A summary of the Seismic analysis is displayed in Table 1.2 below:

ASCE7-02 Chapter 9- Seismic Analysis					
Level, x	$h_{x\ (ft)}$	North-South Forces		East-West Forces	
		F_x	V_x	F_x	V_x
		(kips)	(kips)	(kips)	(kips)
Roof	192	56.5		58.7	
16	180	25.2	56.5	36.6	58.7
15	168	17.3	81.7	33.5	95.4
14	156	15.3	99.0	30.5	128.9
13	144	13.3	114.2	27.5	159.4
12	132	11.5	127.5	24.6	186.9
11	120	9.7	139.0	21.8	211.6
10	108	8.1	148.7	19.1	233.4
9	96	6.7	156.9	16.4	252.4
8	83	5.3	163.5	13.8	268.8
7	71	4.1	168.8	11.3	282.6
6	59	3.0	172.9	9.0	294.0
5	47	2.0	175.9	6.7	302.9
4	35	1.2	177.9	4.6	309.6
3	23	0.6	179.1	2.7	314.2
2	11	0.2	179.7	1.1	316.9
BASE			**179.9**		**318.0**

Table 1.2- Story shear and resultant base shear from Seismic Analysis

Additional flaws in the original analysis also included a miscalculation of the building total weight. The building façade comprised of precast concrete panels and glass windows was estimated to be 180psf, but was not multiplied by the story height. The weight of the concrete slab was also miscalculated. Finally, a better estimate of steel member weight was determined to be 10psf per floor, and an additional 114 kips was added to the total building weight to allow for concrete reinforcement weight, MEP slabs and the weight of additional framing members and connections.

The penthouse was again was not assessed as a separate floor that would take a seismic shear force in the analysis. Instead, the dead weight of the mechanical penthouse was calculated and added to the weight of the roof. This additional weight contributes to the rather large shear force found at the roof the structure in the seismic analysis, and added to the “whiplash” effect which ultimately affects the size of the shear forces near the top of the structure.

Comparison

Both the corrected seismic and wind load analysis have been combined and compared in Table 1.3 below.

Comparison of Wind vs. Seismic Story Shear for Eight Tower Bridge in kips					
Level, x	$h_{x\ (ft)}$	Story Shear N-S		Story Shear E-W	
		Wind	Seismic	Wind	Seismic
Roof	192				
16	180	37.1	56.5	59.1	58.7
15	168	66.1	81.7	112.1	95.3
14	156	94.6	99.0	164.5	128.8
13	144	122.8	114.3	216.2	159.3
12	132	150.6	127.6	267.3	186.8
11	120	177.9	139.1	317.5	211.4
10	108	204.7	148.8	367.0	233.2
9	96	230.4	156.9	415.6	252.3
8	83	255.6	163.6	463.3	268.7
7	71	280.0	168.9	510.0	282.5
6	59	303.5	172.9	555.5	293.8
5	47	326.3	177.0	599.7	302.8
4	35	347.9	180.0	642.5	309.5
3	23	368.1	181.2	683.5	314.1
2	11	387.0	181.8	722.1	316.8
BASE		406.0	182.0	758.7	317.9

= Controls for design

Table 1.3 Comparison of Seismic and Wind Loads

The controlling lateral force in for the majority of levels in both directions are the wind forces. This result makes sense, as Conshohocken, Pennsylvania is not a very seismically active location and would be controlled by wind design. Seismic design only controls for the floors 14 through 16 in the North-South direction, again due to the mechanical penthouse weight atop the structure. A full spreadsheet for both wind and seismic analysis can be found in Appendix B.

Lateral Load Distribution

As previously described, the lateral system of Eight Tower Bridge employs the use of moment resisting connections around the perimeter of the building, as well as laterally braced frames at the building core. The rectangular footprint of the building is constructed symmetrically around the building core. Due this symmetry, the building's center of mass is located nearly at the geometric center of the structure.

The lateral loads developed on the structure from both wind and seismic forces are transferred from the building perimeter to the building core and braced frames through the composite concrete slab and steel framing members. The gravity members are connected with simple pin connections, allowing for rotation and the transfer of shear through the length of the member. In a very complex analysis, the columns and moment frames around the perimeter of the building would be included in a lateral system analysis. However, this report analyzes the structure neglecting the effects of "leaning columns" and assumes that the braced frames with larger column member and a greater rigidity will draw all of the lateral forces.

The symmetry also allows for the assumption to be made that each braced frame carries an equal distribution of the lateral load in each direction. That is, a quarter of the controlling lateral load in the East-West direction will be distributed to each frame of the four frames spanning that direction, while the lateral load developed in the North-South direction will be divided evenly among the two braced frames in the same direction.

In order to conduct a simplified lateral analysis, it has been assumed that the floor slab of each level is a rigid diaphragm.

Lateral System Analysis

In attempt to perform a more accurate wind and seismic analysis, the computer engineering software program ETABS was used. ETABS is a very powerful 3D-structural modeling program with both design and analysis capabilities. The analysis side of the program was used to model the structure, as the building members have already been designed. The model was created simply to analyze the structure behavior.

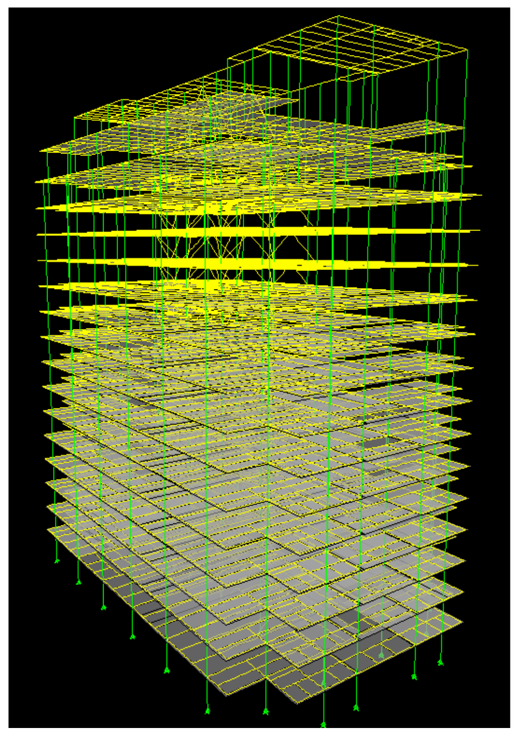


Figure 1.3- 3D frame model in ETABS

Several assumptions were made in order to analyze the structure for wind and seismic loadings only. Gravity loads were not included as part of the static load case in ETABS, although a more accurate model could be developed if these loads had been included. The floor slabs were meshed around floor beams, which allow any loads placed on the slab including self weight, to be transferred to the beams and columns of the structure.

The floor slab for this report will be modeled in the ETABS computer model as a rigid diaphragm only. Modeling these structure components as a rigid diaphragm will only allow the slab to translate in its own plane and rotate about an axis perpendicular to this plane. This allows for the transfer of lateral forces to the building core while the program runs the structure analysis. This will also eliminate the out-of-plane behavior of the slab (i.e. slab bending) in the model. Figure 1.3 below shows the rigid diaphragm assignment to each of the floor slabs. The white lines converge at the structures center of rigidity.

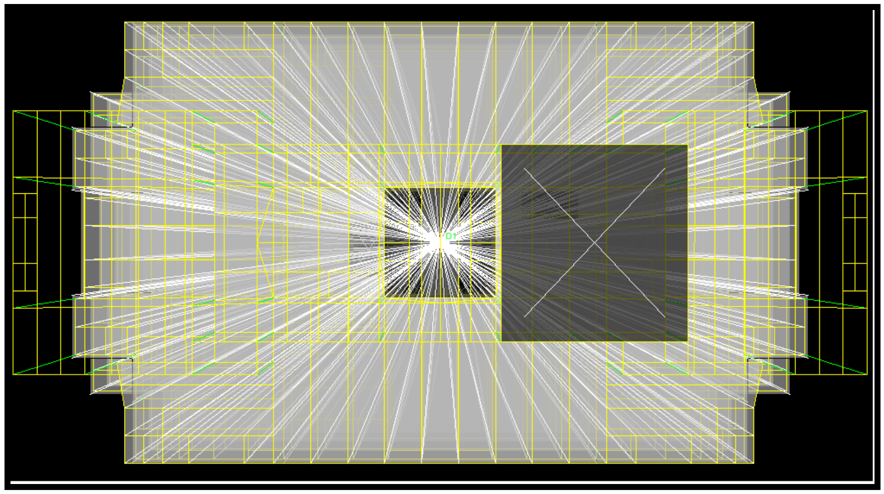


Figure 1.4 ETABS model in 3-D plan view with rigid diaphragms assigned

The building superstructure was modeled as close as possible from the information given in the structural documents. All floor beams and columns were modeled as pin connections, with the exception of the members noted as having moment resisting connections and also at column splices. The connections at the base of the building were assumed to be fixed in order to resist overturning moment. All slab and deck properties were specified in ETABS as noted in the structural drawings. The mechanical penthouse was modeled as a single story rather than a separate penthouse level and recessed mechanical room. It is acknowledged that this may cause a small degree of error on the conservative side when modeling the seismic loads, as the penthouse does not actually have as much surface area as modeled.

<u>Seismic and Wind Analysis in ETABS</u>

After the model was completed, an analysis was performed for both wind and seismic loadings in each direction. From this model analysis, story forces under both shear and seismic loadings were obtained and are displayed below in Tables 1.4 and 1.5 respectively.

ETABS WIND ANALYSIS OUTPUT					
		North-South		East-West	
Level	h (ft)	Story Force (kips)	Story Shear (kips)	Story Force (kips)	Story Shear (kips)
Roof	214				
Penthouse	194	18.0		23.0	
16	180	34.1	18.0	46.6	23.0
15	168	32.2	52.1	47.0	69.7
14	156	31.7	84.2	46.4	116.7
13	144	31.3	116.0	45.6	163.0
12	132	30.9	147.3	44.9	208.7
11	120	30.4	178.1	44.1	253.6
10	108	29.9	208.5	43.2	297.6
9	96	29.3	238.4	42.3	340.9
8	83	28.7	267.6	41.3	383.2
7	71	28.0	296.3	40.1	424.4
6	59	27.2	324.3	38.8	464.5
5	47	26.3	351.5	37.4	503.4
4	35	25.2	377.7	35.5	540.7
3	23	23.8	402.9	33.2	576.3
2	11	21.6	426.7	30.0	609.5
BASE			448.4		639.5

Table 1.4- ETABS output for wind analysis

ETABS SEISMIC ANALYSIS OUTPUT					
		North-South		East-West	
Level	h (ft)	Story Force (kips)	Story Shear (kips)	Story Force (kips)	Story Shear (kips)
Roof	214	3.13		0	
Penthouse	194	156.49	3.1	249.78	0.0
16	180	50.9	156.5	83.87	249.8
15	168	50.5	207.4	85.71	333.7
14	156	46.57	257.9	81.58	419.4
13	144	40.67	304.5	73.72	500.9
12	132	35.08	345.1	66.01	574.7
11	120	29.84	380.2	58.49	640.7
10	108	24.95	410.1	51.17	699.2
9	96	20.43	435.0	44.07	750.3
8	83	16.28	455.4	37.19	794.4
7	71	12.52	471.7	30.56	831.6
6	59	9.16	484.2	24.2	862.2
5	47	6.25	493.4	18.18	886.4
4	35	3.8	499.6	12.53	904.5
3	23	1.73	503.4	6.84	917.1
2	11	0.47	505.2	2.56	923.9
BASE			505.6		926.5

Table 1.5- ETABS output for seismic analysis

The results of the ETABS analysis for wind loading on the structure were fairly agreeable with the results of the wind analysis performed through ASCE7-02. Each of the story shear forces generated from ETABS differs from the story shear forces derived from ASCE7-02 between 5% and 9% with a total base shear difference of 9.4% in the North-South direction and 15.7% in the East-West direction. While it has been previously mentioned that there were inconsistencies between the model created in ETABS and the simplified model used in ASCE7-02, the discrepancies in base shear and stories shears must be examined further. The 3D model in ETABS will be refined in regards to geometry and behavior for the proposal and final thesis work.

The seismic loads generated through ETABS were found to be nearly three times as great as the loads developed through ASCE7-02. This can most likely be attributed to a difference in total building weight calculated. The total building weight dead weight computed for the initial seismic calculations, was 19,717 kips. This weight is a summation of dead weight only. A list of load allowances and the dead weight calculation can be found in Appendix B. The building weight generated by ETABS was determined to be 53,802 kips, near two and a half times the weight found by hand. While it is agreed that computer software is more accurate in calculating the self weight of members due an extensive material properties database, the difference in weights is still too great. The model will be reviewed for the proposal and for future thesis work in hopes of generating more agreeable answers.

Bracing Check

A member check was conducted for the lateral resisting frame along column line D in the East-West direction and the frame along column line 4.1 in the North-South direction. Due to the symmetry of the building, it was assumed that each of the frames would resist an equal share of the story shear at each particular level and for the building as a whole. The frames found along column lines E, F and G are similar to the frame along column line D, so it is implied that these frames will resist the same magnitude of lateral forces in the East-West direction. The same assumption was made for the frame along column line 4.9 with respect to the frame along column 4.1.

The forces used for the bracing check were taken from the wind load analysis developed through both ASCE7-02 and the ETABS model. The bracing was found to be adequate for all frames in both directions with outputs from both methods. The diagonal bracing was analyzed as if the horizontal beams in the brace were to act at zero force members and take no load in the idealized truss model. Hand calculations for this member check can be found in Appendix C.

A full spread sheet for the axially forces developed on each diagonal brace was generated using output from the ETABS model and can be referenced in Appendix C.

<u>Conclusions</u>

In conclusion, it was found that the braced frames located at the core the building were adequately designed to withstand the wind and seismic forces generated through both the computer model and by hand using ASCE7-02. In actuality, this framing system is just a piece of the total framing system, as there are moment resisting connections around the perimeter of the building. These moment frames will resist some of the lateral loads developed. However, the braced frames will take a majority of this load.

Additionally, it has been determined that the model created through ETABS was not completely accurate, as is indicated by the rather large seismic loads and the difference in wind loads. This model will be further examined in order to refine and improve it, in hopes to create a model that will accurately predict the behavior of Eight Tower Bridge under multiple loading conditions, both lateral and gravity.

Appendix A

Typical Floor Framing Plan,
Lateral Braced Frame Drawings

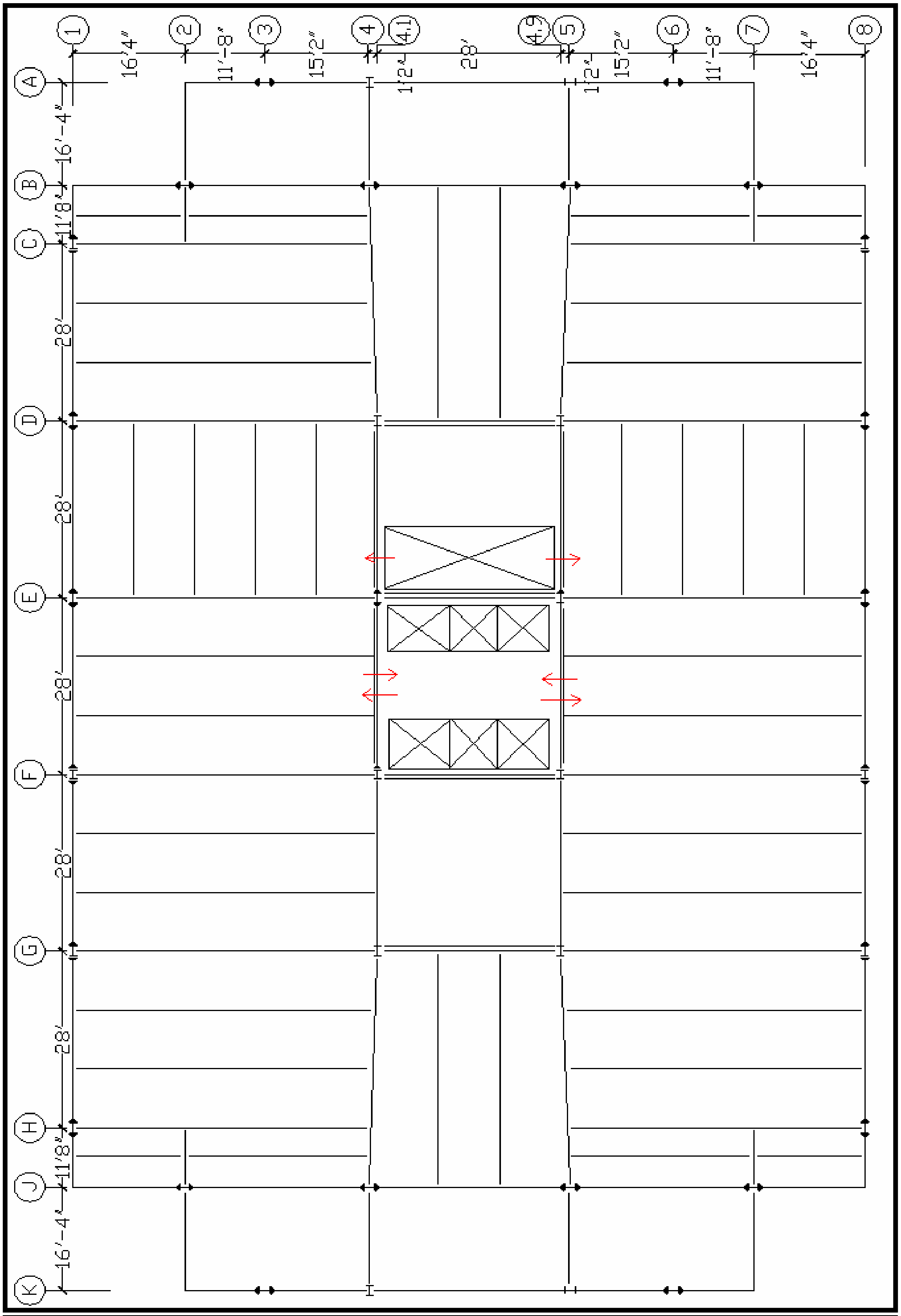


Figure A.1- Typical Framing Plan (floors 3-15) with stair and elevator entrances noted

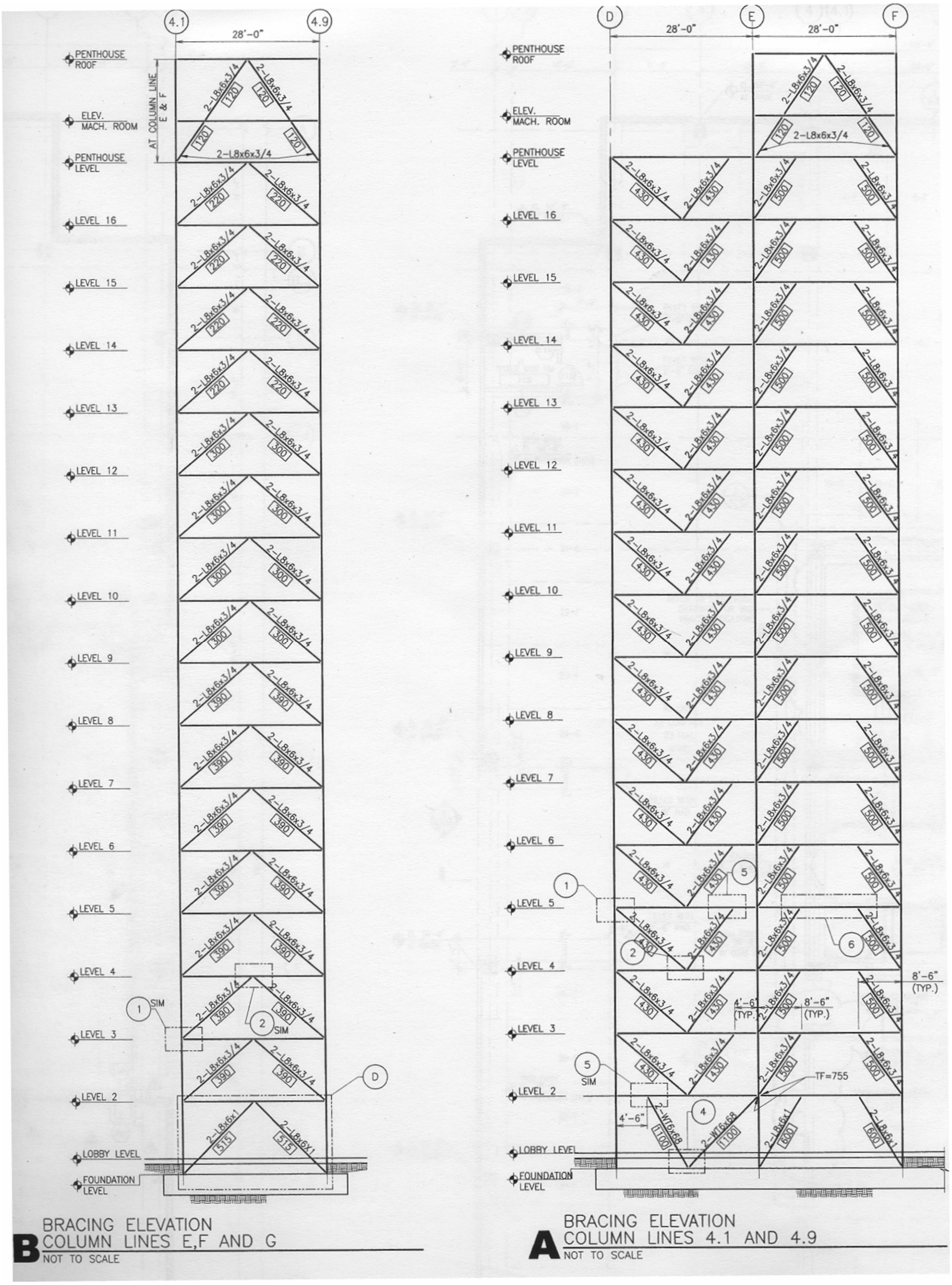


Figure A.2- Laterally braced frames found at building core. The brace in elevation B can be found along column line D, in addition to column lines E, F and G.

Appendix B

Wind Analysis Spreadsheets,
Seismic Analysis Spreadsheets

Location:	**Conshohocken, PA**				
				Penthouse	
Dimension	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
	$q_z/K_z = 0.00256K_{zt}K_dV^2I =$	**17.6256**	

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

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

					Resultant pressure N-S	Resultant pressure E-W
External Pressure Coefficients, C_p				Windward	**0.8**	**0.8**
				Leeward	**-0.368**	**-0.50**
Story No.	z (ft)	K_z	q_z (lb/ft²)	q_h (lb/ft²)	$q_zC_pG - q_hC_pG$ (lb/ft²)	$q_zC_pG - q_hC_pG$ (lb/ft²)
Penthouse Roof	214	1.222	21.544	21.544	20.946	23.001
Penthouse Level	192	1.188	20.945	21.544	20.547	22.607
16	180	1.170	20.625	21.544	20.334	22.397
15	168	1.146	20.200	21.544	20.051	22.118
14	156	1.122	19.775	21.544	19.768	21.838
13	144	1.098	19.349	21.544	19.484	21.558
12	132	1.070	18.851	21.544	19.152	21.231
11	120	1.039	18.318	21.544	18.798	20.881
10	108	1.009	17.786	21.544	18.443	20.532
9	95.6	0.977	17.215	21.544	18.063	20.156
8	83.5	0.940	16.576	21.544	17.638	19.737
7	71.4	0.896	15.785	21.544	17.111	19.217
6	59.3	0.847	14.934	21.544	16.544	18.658
5	47.2	0.796	14.034	21.544	15.945	18.066
4	35.2	0.731	12.884	21.544	15.179	17.311
3	23.1	0.645	11.362	21.544	14.166	16.311
2	11	0.570	10.047	21.544	13.290	15.447
Base	0	0.570	10.047	21.544	13.290	15.447

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 Pressu	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**			

ASCE7-02 Chapter 6: Wind Analysis

Method 2- Analytical Procedure

Story Forces		Story Shear		Moment	
N-S	**E-W**	**N-S**	**E-W**	**N-S**	**E-W**
(kips)	(kips)	(kips)	(kips)	(ft-kips)	(ft-kips)
10.14	28.84			2,169.00	6,170.96
26.92	30.26	10.14	28.84	5,173.31	5,816.09
28.99	53.03	37.05	59.10	5,220.78	9,551.57
28.58	52.37	66.04	112.13	4,802.89	8,799.95
28.18	51.71	94.62	164.50	4,394.58	8,064.03
27.77	51.04	122.80	216.21	3,996.04	7,344.12
27.30	50.27	150.57	267.25	3,598.22	6,625.40
26.80	49.44	177.87	317.52	3,207.94	5,919.02
26.29	48.61	204.67	366.96	2,829.86	5,232.65
25.75	47.72	230.96	415.57	2,460.44	4,560.47
25.14	46.73	256.71	463.30	2,098.80	3,901.02
24.39	45.50	281.85	510.03	1,741.53	3,248.75
23.58	44.18	306.24	555.53	1,398.93	2,620.51
22.73	42.78	329.82	599.70	1,073.67	2,020.71
21.64	40.99	352.55	642.48	760.74	1,441.09
20.19	38.62	374.19	683.46	466.04	891.35
18.94	36.57	394.38	722.08	208.38	402.31
	Total Shear	413.32	758.66		
			Total Moment	45,601.14	82,610.00

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		1.00
Table 9.4.1.2.4b	Site Class Factor :	F_v		1.00
	Adjusted Accelerations :	S_{MS}	$= F_a S_S$	0.310 g-s
		S_{M1}	$= F_v S_1$	0.075 g-s
	Design Spectral Response Accelerations	S_{DS}	$= (2/3)S_{MS}$	0.207 g-s
		S_{D1}	$= (2/3)S_{M1}$	0.050 g-s
Table 9.4.2.1a	Seismic Design Category :			B
Table 9.4.2.1b	Both design category B			
	N-S Direction			
Table 9.5.2.2	Response Modification Factor :	$R_{N\text{-}S}$		3
	Seismic Response Coefficient :	$C_{s,\,N\text{-}S}$	$= S_{DS}/(R_{N\text{-}S}/I)$	**0.069**
Table 9.5.5.3.2		$C_{T,\,N\text{-}S}$		0.028
Table 9.5.5.3.2	(moment frames only)	x		0.80
	Approximate Period of Structure :	$T_{N\text{-}S}$	$= C_{T,\,N\text{-}S} h_n^x$	**1.89**
	Seismic Response Coefficient need			
	not be greater than	$C_{S\,max,}$	$S_{D1}/T(R_{N\text{-}S}/I)$	**0.009**
	and	$C_{S\,min}$	$= 0.044 I S_{DS}$	**0.0091**
	Seismic Response Coefficient ($C_{s,\,N\text{-}S}$)			**0.009**
	E-W Direction			
Table 9.5.2.2	Response Modification Factor :	$R_{N\text{-}S}$		3
	Seismic Response Coefficient :	$C_{s,\,E\text{-}W}$	$= S_{DS}/(R_{E\text{-}W}/I)$	**0.069**
Table 9.5.5.3.2		$C_{T,\,E\text{-}W}$		0.02
Table 9.5.5.3.2	(moment and braced frame)	x		0.75
	Approximate Period of Structure :	$T_{N\text{-}S}$	$= C_{T,\,E\text{-}W} h_n^x$	**1.04**
	Seismic Response Coefficient need			
	not be greater than	$C_{S\,max,}$	$S_{D1}/T(R_{E\text{-}W}/I)$	**0.016**
	and	$C_{S\,min}$	$= 0.044 I S_{DS}$	**0.0091**
	Seismic Response Coefficient ($C_{s,\,E\text{-}W}$)			**0.016**

ASCE7-02 Chapter 9- Seismic Analysis

Load Summary for Building Weight

i. Roof :

Dead			
Membrane	1.0	psf	1
Rigid Insulation	2.0	psf	2
Metal Roof Deck	2.0	psf	2
Roof Framing	12.0	psf	12
M&E Services	5.0	psf	5
TOTAL q_{roof} :	**22**	psf of roof area	

ii. All other Floors :

Dead			
Flooring	1.0	psf	1
Concrete Slab	31.0	psf	2
3" Metal Deck	2.0	psf	2
Structural Steel	10.0	psf	5
0.5" Drywall Ceiling	5.0	psf	5
M&E Services	5.0	psf	5
Live			
Moveable Partition	0.0	psf	20
Office Live Load	0.0		30
TOTAL q_{floor} :	**54.0**	psf of floor area	

iii. Perimeter Wall:

Dead			
Exterior Wall, q_{wall} :	**180.0**	psf	180

iv. Weight of Roof

	(PSF)	Area	
Roof Load	12	7500 ft^2	90000
Mechanical Load	200	2500 ft^2	500000
Elevator	200	790 ft^2	158000
Cooling Tower	300	2050 ft^2	615000
Snow	0	4000 ft^2	0
Additional Dead Load	20	16840 ft^2	336800
		Total Roof Load (kips)	1700

v. Allowances

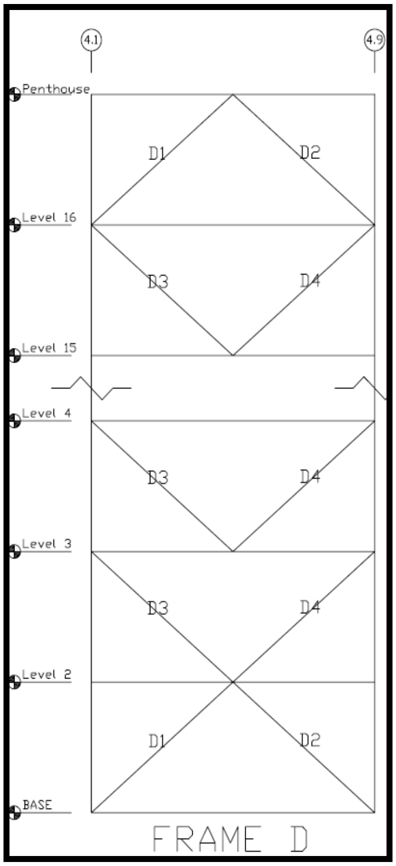
Misc. Steel	10	kips
Rebar	4	kips
Welded Wire Fabric	2	kips
MEP Equipment pads (25CY 4000psi conc.)	97.8	kips
Total	**113.8**	

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	1194	kips
$w_{floors} = (N-1)w_{per\ floor}$ =	17903	kips
Total Building Weight	19717	kips

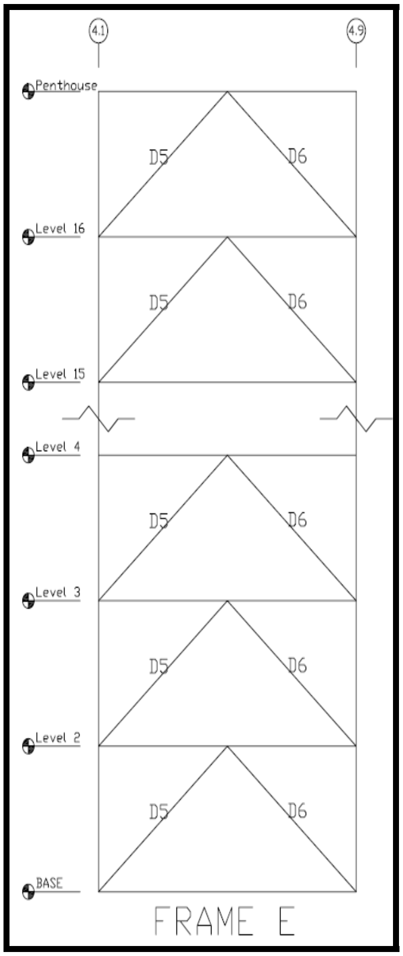
Appendix C

ETABS Output



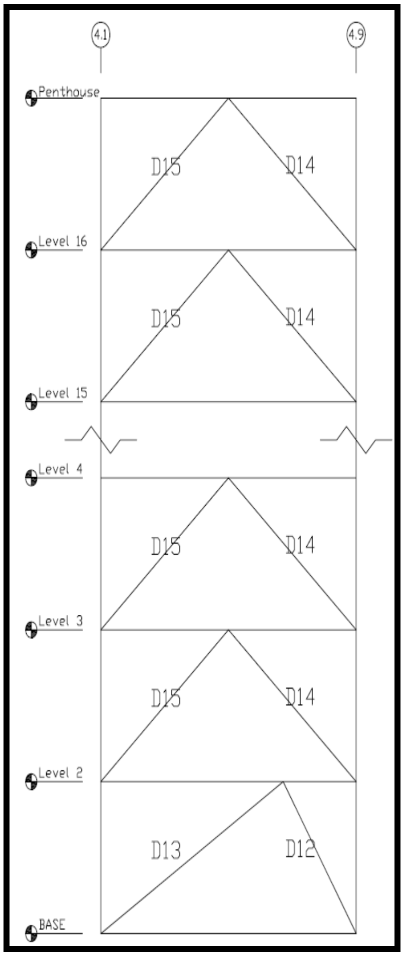
Frame D

STORY	BRACE	AXIAL FORCE, P (kips)			
		Wind-X	Wind-Y	Seismic-X	Seismic-Y
ROOF	D1	0	0	0	0
	D2	0	0	0	0
PENTHOUSE	D3	-15.47	-3.99	-31.77	-43.54
	D4	-15.4	3.86	-31.59	43.31
LEVEL 16	D3	-13.31	-6.17	-23.23	-40.12
	D4	-12.96	6.07	-22.25	39.96
LEVEL 15	D3	-10.93	-14.28	-16.17	-52.83
	D4	-10.27	14.2	-13.46	52.7
LEVEL 14	D3	-8.64	-20.76	-9.11	-61.54
	D4	-7.75	20.68	-5.52	61.44
LEVEL 13	D3	-4.59	-28.05	-1.7	-73.36
	D4	-3.2	28	3.26	73.32
LEVEL 12	D3	-1.61	-35.81	6.39	-85.56
	D4	0.05	33.22	12.3	76.11
LEVEL 11	D3	1.44	-42.03	10.63	-92.2
	D4	3.71	41.99	16.39	92.16
LEVEL 10	D3	5.19	-48.27	17.64	-97.93
	D4	7.77	48.23	24.14	97.86
LEVEL 9	D3	7.4	-55.29	19.91	-105.92
	D4	10.58	55.26	27.37	105.85
LEVEL 8	D3	11.48	-61.16	26.53	-109.58
	D4	15.02	61.12	34.47	109.5
LEVEL 7	D3	11.44	-68.79	23.99	-117.47
	D4	15.8	68.77	33.04	117.41
LEVEL 6	D3	15.29	-73.31	29.5	-116.93
	D4	20.14	73.3	39.01	116.87
LEVEL 5	D3	16.22	-80.64	29.38	-122.83
	D4	22.05	80.63	39.96	122.78
LEVEL 4	D3	18.18	-85.57	31.69	-123.56
	D4	29.6	85.6	49.39	123.56
LEVEL 3	D3	22.62	-104.67	36.74	-144.64
	D4	37.1	104.71	57.89	144.63
LEVEL 2	D1	34.61	115.5	53.42	160.83
	D2	28.21	-115.5	160.67	-160.67



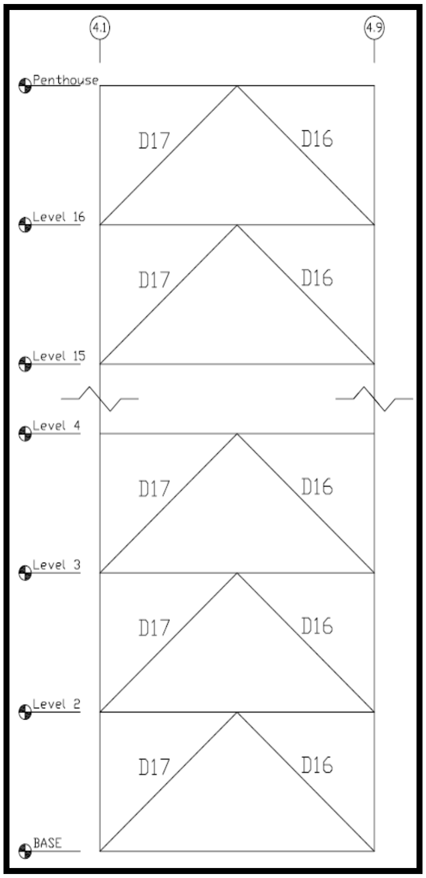
Frame E

STORY	BRACE	AXIAL FORCE, P (kips) Wind-X	Wind-Y	Seismic-X	Seismic-Y
ROOF	D5	0	0	0	0
	D6	0	0	0	0
PENTHOUSE	D5	22.45	4.67	41.61	47.8
	D6	23.04	-4.65	43.1	-47.93
LEVEL 16	D5	0.15	5.59	0.08	37.91
	D6	0.81	-5.59	1.75	-37.92
LEVEL 15	D5	-0.01	15.29	-0.2	59.32
	D6	0.74	-15.29	1.5	-59.32
LEVEL 14	D5	-0.11	22.91	-0.46	74.49
	D6	0.74	-22.92	1.49	-74.5
LEVEL 13	D5	-0.19	30.15	-0.65	86.66
	D6	0.58	-30.16	1.21	-86.66
LEVEL 12	D5	-0.24	37.76	-0.77	99.28
	D6	0.52	-37.76	1.07	-99.29
LEVEL 11	D5	-0.27	44.09	-0.5	106.89
	D6	0.38	-44.09	0.53	-106.9
LEVEL 10	D5	-0.28	51.39	-0.51	116.68
	D6	0.26	-51.39	0.29	-116.68
LEVEL 9	D5	-0.23	57.49	-0.4	122.63
	D6	0.08	-57.49	0.01	-122.63
LEVEL 8	D5	-0.19	64.51	-0.3	130.25
	D6	-0.11	-64.51	-0.32	-130.25
LEVEL 7	D5	-0.05	69.84	-0.04	133.18
	D6	-0.33	-69.84	-0.67	-133.18
LEVEL 6	D5	0.06	76.5	0.18	138.9
	D6	-0.59	-76.5	-1.1	-138.9
LEVEL 5	D5	0.29	80.16	0.57	137.65
	D6	-0.82	-80.16	-1.41	-137.66
LEVEL 4	D5	1.12	84.38	1.73	137.73
	D6	-1.79	-84.38	-2.74	-137.74
LEVEL 3	D5	1.03	79.73	1.51	124
	D6	-1.64	-79.74	-2.41	-124
LEVEL 2	D5	0.42	80.09	0.76	114.61
	D6	-1.18	-80.09	-1.83	-114.62



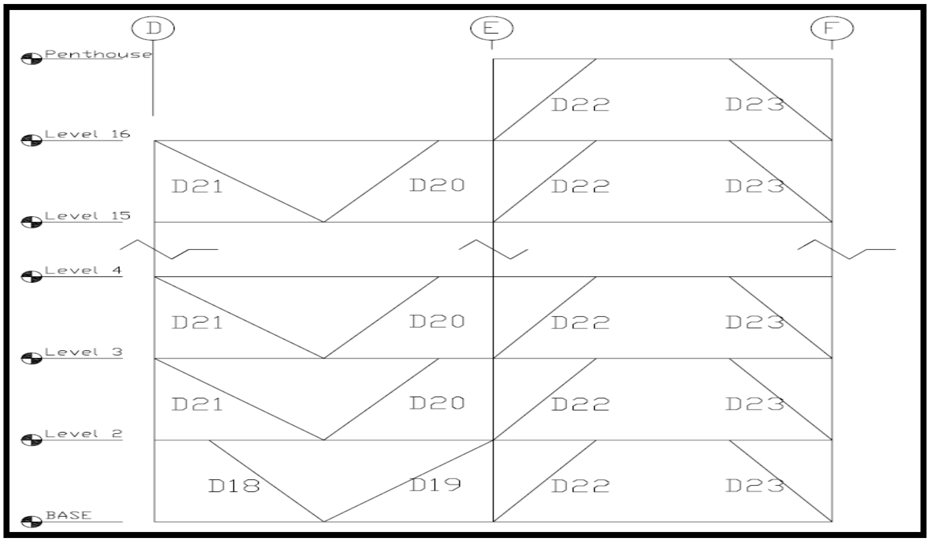
Frame F

STORY	BRACE	AXIAL FORCE, P (kips)			
		Wind-X	Wind-Y	Seismic-X	Seismic-Y
ROOF	D15	0	0	0	0
	D14	0	0	0	0
PENTHOUSE	D15	-5.85	3.64	-7.48	41.75
	D14	-6.12	-3.84	-8.16	-42.2
LEVEL 16	D15	-8.79	2.24	-18.01	28.99
	D14	-9.1	-2.5	-18.83	-29.63
LEVEL 15	D15	-6.87	14.49	-14.21	56.04
	D14	-7.19	-14.74	-14.82	-56.68
LEVEL 14	D15	-6.24	22.27	-13.08	70.34
	D14	-6.55	-22.57	-13.67	-71.06
LEVEL 13	D15	-4.46	29.73	-9.57	82.19
	D14	-4.62	-30.03	-9.84	-82.93
LEVEL 12	D15	-4.05	37.63	-8.95	94.93
	D14	-4.08	-37.99	-8.96	-95.78
LEVEL 11	D15	-3.14	44.53	-6.99	103.55
	D14	-2.9	-44.9	-6.96	-104.34
LEVEL 10	D15	-2.82	52.47	-6.6	114.52
	D14	-2.33	-52.9	-6.14	-115.37
LEVEL 9	D15	-2.22	59.54	-5.45	122.31
	D14	-1.35	-60.01	-4.36	-123.14
LEVEL 8	D15	-1.96	67.81	-5.19	132.44
	D14	-0.72	-68.4	-3.51	-133.49
LEVEL 7	D15	-1.57	74.68	-4.36	138.3
	D14	0.23	-75.4	-1.83	-139.55
LEVEL 6	D15	-1.3	83.41	-4.13	148.02
	D14	0.96	-84.31	-0.92	-149.52
LEVEL 5	D15	-0.98	89.01	-3.27	150.07
	D14	1.79	-90.19	0.64	-152
LEVEL 4	D15	-1.29	96.43	-3.84	155.65
	D14	3.27	-97.8	2.5	-157.71
LEVEL 3	D15	-1.25	97.53	-3.58	151.24
	D14	3.89	-99.07	3.54	-153.36
LEVEL 2	D13	-0.08	91.68	-1.88	130.3
	D12	2.87	129.57	0.93	-204.31



Frame G

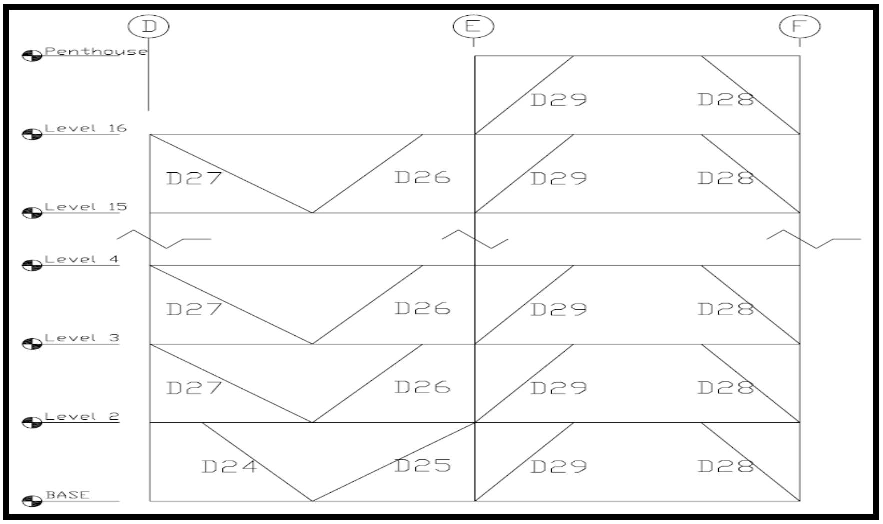
STORY	BRACE	AXIAL FORCE, P (kips)			
		Wind-X	Wind-Y	Seismic-X	Seismic-Y
ROOF	D17	0	0	0	0
	D16	0	0	0	0
PENTHOUSE	D17	-2.94	3.97	-8.55	43.2
	D16	-3.19	-3.81	-9.17	-42.79
LEVEL 16	D17	-3.08	0.76	-8.79	25.82
	D16	-3.05	-0.67	-8.71	-25.56
LEVEL 15	D17	-3.22	15.01	-9.49	59.89
	D16	-2.91	-14.85	-7.77	-59.44
LEVEL 14	D17	-3.87	23.15	-11.2	75.61
	D16	-3.42	-22.98	-8.82	-75.14
LEVEL 13	D17	-3.95	30.54	-11.06	87.93
	D16	-3.05	-30.42	-7.44	-87.57
LEVEL 12	D17	-4.77	38.28	-12.91	100.55
	D16	-3.71	-38.17	-8.66	-100.21
LEVEL 11	D17	-5.1	45.23	-13.12	109.81
	D16	-3.58	-45.17	-8.01	-109.63
LEVEL 10	D17	-6.02	53.21	-14.89	121.16
	D16	-4.33	-53.2	-9.32	-121.1
LEVEL 9	D17	-5.9	59.59	-13.93	127.18
	D16	-3.72	-59.72	-7.58	-127.49
LEVEL 8	D17	-6.8	67	-15.4	135.24
	D16	-4.43	-67.09	-8.76	-135.39
LEVEL 7	D17	-7.07	73.22	-15.21	139.69
	D16	-4.02	-73.29	-7.64	-139.78
LEVEL 6	D17	-8.05	80.3	-16.57	145.56
	D16	-4.68	-80.34	-8.68	-145.55
LEVEL 5	D17	-8.46	86.74	-16.42	149.72
	D16	-4.14	-86.69	-7.47	-149.5
LEVEL 4	D17	-12.24	95.52	-21.28	157.93
	D16	-2.13	-95.68	-4.9	-158.22
LEVEL 3	D17	-11.65	98.55	-18.84	155.03
	D16	0.11	-99.04	-1.17	-156.05
LEVEL 2	D17	-10.09	111.84	-17.54	169.05
	D16	-4.17	-112.08	-6.57	-169.57



Frame 4.1

STORY	BRACE	AXIAL FORCE, P (kips)			
		Wind-X	Wind-Y	Seismic-X	Seismic-Y
ROOF	D31	0	0	0.6	0
	D32	0	0	-0.6	0
PENTHOUSE	D21	14.01	-0.76	-5.14	1.93
	D20	-6.66	-1.3	30.16	3.16
	D22	44.73	0.36	109.8	4.16
	D23	-27.63	-1.97	-79.3	-8.23
LEVEL 16	D21	-2.04	0.72	-30.8	9.12
	D20	13.03	-0.94	56.03	4.76
	D22	13.1	0.21	31.91	3
	D23	-12.43	-0.19	-31.22	0.36
LEVEL 15	D21	-7.94	1.42	-39.59	11.89
	D20	19.89	0.03	64.64	9.12
	D22	14.81	0.56	32.97	4.23
	D23	-15.88	-0.25	-37.17	0.81
LEVEL 14	D21	-14.9	3.56	-50.61	18.71
	D20	27.12	1.55	74.08	14.6
	D22	17.08	1.14	35.51	6.15
	D23	-19.53	0.21	-42.91	2.72
LEVEL 13	D21	-20.71	5.34	-57.4	21.28
	D20	33.54	2.57	80.66	16.44
	D22	17.32	1.5	34.15	6.66
	D23	-21.95	0.7	-45.66	3.77
LEVEL 12	D21	-28.39	9.65	-68.12	33.27
	D20	41.24	5.29	88.24	24.33
	D22	18.4	2.31	34.13	8.75
	D23	-24.39	1.52	-48.3	6.11
LEVEL 11	D21	-34.05	10.95	-71.76	33.25
	D20	47.96	6.36	93.76	25.53
	D22	18.29	2.8	32.24	9.57
	D23	-25.81	2	-48.65	6.84
LEVEL 10	D21	-42.76	15.08	-81.06	42.23
	D20	56.2	9.01	100.91	31.55
	D22	18.41	3.65	30.75	11.35
	D23	-27.18	3.03	-48.99	9.31
LEVEL 9	D21	-47.84	16.69	-83.27	43.32
	D20	62.75	10.17	104.21	32.66
	D22	17.58	3.99	27.98	11.53
	D23	-27.46	3.45	-47.23	9.74

STORY	BRACE	AXIAL FORCE, P (kips)			
		Wind-X	Wind-Y	Seismic-X	Seismic-Y
LEVEL 8	D21	-57.29	21.13	-91.89	51.92
	D20	71.39	13.02	110.43	38.15
	D22	16.84	4.92	25.33	13.28
	D23	-27.79	4.57	-45.7	12.12
LEVEL 7	D21	-63.07	20.4	-94.26	47.03
	D20	78	12.6	112.75	34.97
	D22	15.48	4.95	22.11	12.64
	D23	-27.05	4.83	-42.38	12.04
LEVEL 6	D21	-73.72	24.78	-102.65	54.85
	D20	86.02	14.63	116.95	37.76
	D22	13.98	5.78	18.69	13.99
	D23	-26.47	6.11	-39.56	14.54
LEVEL 5	D21	-78.28	26.1	-101.9	55.23
	D20	89.95	14.6	115.6	35.55
	D22	12.21	5.14	15.44	11.89
	D23	-24.63	5.72	-34.89	12.95
LEVEL 4	D21	-99.92	29.81	-122.32	61.22
	D20	92.63	18.21	114.05	41.14
	D22	10.07	5.95	11.58	13.1
	D23	-23.35	6.87	-31.35	15.02
LEVEL 3	D21	-143.94	24.26	-166.93	48.26
	D20	63.47	17.87	75.8	37.63
	D22	7.15	5.72	7.33	12.03
	D23	-20.12	6.35	-25.47	13.43
LEVEL 2	D18	-142.99	35.83	-160.01	71.29
	D19	13.1	0.21	31.91	3
	D22	2.32	5.22	1.19	10.71
	D23	-17.24	7.33	-20.71	14.75

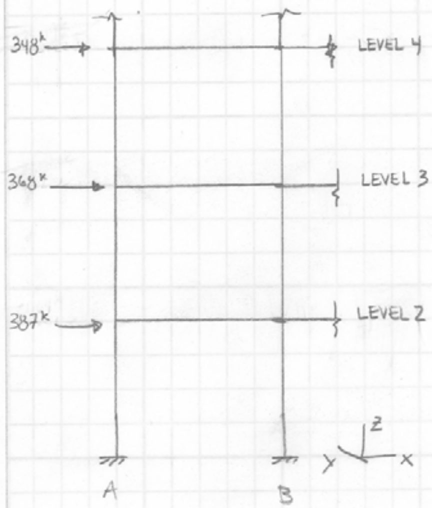


Frame 4.9

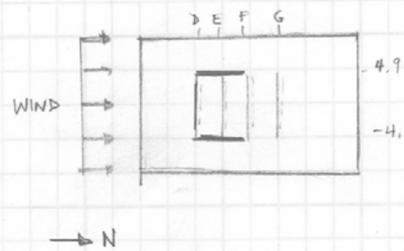
STORY	BRACE	AXIAL FORCE, P (kips)			
		Wind-X	Wind-Y	Seismic-X	Seismic-Y
ROOF	D33	0	0	0.6	0
	D34	0	0	-0.6	0
PENTHOUSE	D27	14.16	0.8	-4.78	-1.97
	D26	-5.72	1.26	32.46	-3.13
	D29	45.61	-0.48	112.04	-4.81
	D28	-28.61	1.69	-81.76	7.7
LEVEL 16	D27	-2.1	-0.81	-31.02	-9.43
	D26	13.83	0.98	57.98	-4.65
	D29	13.38	-0.19	32.61	-3.04
	D28	-12.7	0.13	-31.92	-0.42
LEVEL 15	D27	-8.31	-1.48	-40.96	-12.1
	D26	20.87	-0.01	67.07	-9.09
	D29	15.1	-0.55	33.73	-4.3
	D28	-16.21	0.21	-38.06	-0.79
LEVEL 14	D27	-15.53	-3.64	-52.81	-18.99
	D26	28.29	-1.54	76.84	-14.6
	D29	17.41	-1.12	36.35	-6.22
	D28	-19.91	-0.24	-43.93	-2.65
LEVEL 13	D27	-21.8	-5.46	-60.7	-21.7
	D26	34.88	-2.59	83.64	-16.58
	D29	17.66	-1.48	34.99	-6.74
	D28	-22.39	-0.7	-46.77	-3.62
LEVEL 12	D27	-29.88	-10.08	-69.97	-34.81
	D26	42.65	-5.45	92.76	-25.02
	D29	18.75	-2.29	35.03	-8.85
	D28	-24.88	-1.52	-49.5	-5.93
LEVEL 11	D27	-35.78	-10.83	-76.76	-32.73
	D26	49.53	-6.61	97.93	-26.59
	D29	18.64	-2.66	33.14	-9.24
	D28	-26.36	-2.13	-49.92	-7.12
LEVEL 10	D27	-44.75	-15.1	-86.5	-42.2
	D26	57.87	-9.04	104.98	-31.85
	D29	18.74	-3.56	31.56	-11.22
	D28	-27.76	-3.14	-50.28	-9.53
LEVEL 9	D27	-50.18	-16.73	-89.19	-43.27
	D26	64.51	-10.15	108.25	-32.86
	D29	17.9	-3.89	28.72	-11.4
	D28	-28.07	-3.54	-48.54	-9.88

STORY	BRACE	AXIAL FORCE, P (kips)			
		Wind-X	Wind-Y	Seismic-X	Seismic-Y
LEVEL 8	D27	-59.86	-21.2	-98.08	-51.91
	D26	73.15	-12.96	114.25	-38.28
	D29	17.12	-4.82	25.95	-13.15
	D28	-28.41	-4.67	-46.95	-12.25
LEVEL 7	D27	-65.96	-20.5	-100.71	-47.03
	D26	79.82	-12.48	116.49	-35.02
	D29	15.73	-4.84	22.64	-12.5
	D28	-27.72	-4.92	-43.63	-12.16
LEVEL 6	D27	-76.82	-24.92	-109.24	-54.93
	D26	87.78	-14.43	120.35	-37.67
	D29	14.16	-5.67	19.06	-13.84
	D28	-27.13	-6.21	-40.71	-14.67
LEVEL 5	D27	-81.66	-26.25	-108.6	-55.3
	D26	91.72	-14.29	118.79	-35.27
	D29	12.37	-5.02	15.75	-11.72
	D28	-25.39	-5.83	-36.11	-13.09
LEVEL 4	D27	-105.28	-30.68	-131.41	-62.42
	D26	94.95	-17.57	117.66	-40.32
	D29	10.3	-5.78	11.9	-12.84
	D28	-24.36	-7.04	-32.8	-15.24
LEVEL 3	D27	-149.47	-24.68	-175.63	-48.74
	D26	65.39	-17.7	78.55	-37.48
	D29	7.26	-5.63	7.44	-11.89
	D28	-21.2	-6.41	-26.88	-13.48
LEVEL 2	D24	-146.03	-36.28	-165.51	-71.86
	D25	141.75	-23.99	163.41	-50.81
	D29	2.16	-5.34	0.9	-10.93
	D28	-18.02	-6.31	-21.61	-12.95

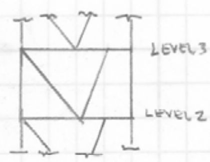
CHRIS MCUNE | BRACING CHECK | EIGHT TOWER BRIDGE



WIND X DIRECTION



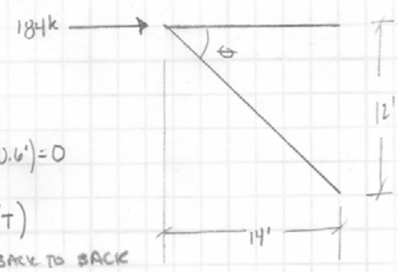
THE LATERAL FORCE ON EACH FRAME IS ASSUMED TO BE HALF OF THE SHEAR STORY FORCE



@ LEVEL 3 : $V/2 = V_F$

C.L. 4.1

$368^k/2 = 184^k$



$\theta = TAN^{-1}\left(\frac{12}{14}\right)$

$\theta = 40.6°$

$184^k - F\cos(40.6°) = 0$

$F = 242.3^k$ (T)

2-L 8x6x3/4" ANGLES, LONG LEGS BACK TO BACK

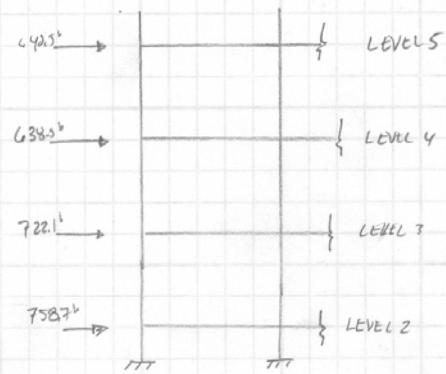
EFFECTIVE LENGTH : 18.4' →

TABLE 4-10, P 4-82 IN LRFD

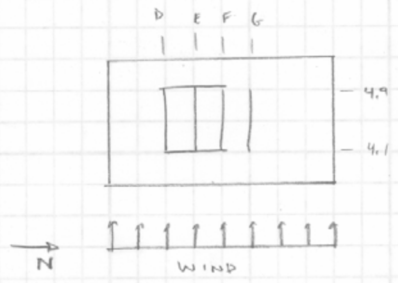
l_{eff}	ϕP_n
18'	416^k
18.4'	→ $408.8^k > 242.3^k$ ∴ OK
20'	380^k

CHECK WITH OUTPUT FROM ETABS

$V = 402.9^k/2 = 201.5^k$ $\quad F = 265.3^k$; $408.8^k > 265.3^k$ ∴ OK

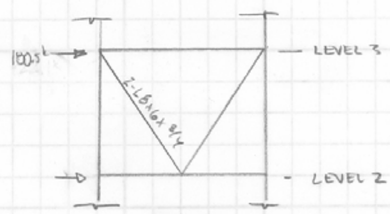


WIND - Y DIRECTION



THE LATERAL FORCE ON EACH FRAME IS ASSUMED TO BE 1/4 ON THE TOTAL STORY FORCE IN THE E-W DIRECTION.

@ LEVEL 3



$V/4 = 722.1^k/4$

$V = 180.5^k$

SINCE THE GEOMETRY OF THE FRAME ANALYZED ALONG COLUMN LINE 4.1 AND THIS FRAME ARE THE SAME, AND MEMBER SIZES ARE ALSO THE SAME, IT IS ASSUMED THAT THE FRAME WILL WITHSTAND THE 180.5k FORCE BECAUSE THE BRACE WAS ADEQUATE FOR A 242.3k FORCE

$408.8^k > 180.5^k \quad \therefore$ ok!

CHECK W/ ETABS OUTPUT

$V = 576.3^k/4 = 144^k \qquad F\cos(40.6) = 144^k$

$F = 189.7^k < 408.8^k$