Erie on the Park

Chicago, IL

TECHNICAL ASSIGNMENT 3

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Executive Summary

It is the intent of this report to analyze the lateral force resisting system implemented when designing 500 W. Erie St. Chicago, Illinois, under wind and seismic loading.

Lateral System Description

Erie on the Park is a 25 story condominium complex on W. Erie St. in Chicago, IL. It uses a number of three story shear walls at the base that transfer to steel braced frames after the third floor. Two of the concrete shear walls are aligned in the East-West (short) direction of the building and three are in the North-South direction. All for these shear walls are at least 26' long, 18" thick and made of concrete with a 28 day compressive strength of 8000 psi. The steel braced frames continue through the remaining 22 stories are aligned two in each direction symmetric about the center of gravity of the floor slabs. Two of these braced frames can be seen in the building's East and West façades. All the braced frames are designed using large three story chevron braces that transfer the shear load down over a wider area and thus are stiffer than braces that only span column bay. Having the braces extend over three stories also allows for a greater flexibility of the floor layouts because the braces are not crossing between the floors in the same place on every floor.

Structural Design Code

Chicago Building Code
ASCE-7 2002
IBC 2000

Conclusions

During the check of the lateral systems of 'Erie on the Park' I relied heavily on ASCE-7-02 for determining the forces attributed to the wind and seismic loading cases as well as for how to distribute the loads that I found to the building elements. This caused some minor discrepancies with the design that was used for the building since it was designed under the Chicago Building Code which outlines different ways to account for the wind and seismic loads.

Table of Contents

Introduction

'Erie on the Park' is a 25 story, 292' condominium complex located in the River North section of Chicago, IL. The foundation system of this building is drilled caissons that were drilled to a depth of 85' below grade to rest on soil with an allowable bearing strength of 30 KSF. These caissons are tied together at the top with grade beams that are up to 100" deep. The gravity force resisting systems of this building change as you go higher in the building. The ground, 2^{nd}, and 3^{rd} levels are two-way flat-plate concrete slabs resting on cast-in-place concrete columns and walls. Above the third floor the gravity system transitions to steel. The 4^{th}, 5^{th}, and 6^{th} levels are partially composite slab on metal deck and steel beams. The 7^{th} through 25^{th} floors are constructed of concrete slabs on metal deck resting on 14" open-web metal joists. The floor systems of the 4^{th} through 25^{th} floors transfer their loads to steel columns which transfer the load vertically through the building.

For the purpose of this report I used ASCE-7-02 to determine the wind and seismic loading on the lateral system of 'Erie on the Park' even though under the Chicago Building Code (CBC) it says to neglect the effect of seismic loading when designing the building. When modeling this building in ETABS, I used ASCE-7-98 as well as a user defined loading case for determining the wind loading and IBC 2000 for determining the seismic loading conditions.

Lateral System

The lateral system of 'Erie on the Park' is comprised of two different systems, concrete shear walls that go from the ground to the third floor and steel braced frames that run from the third floor to the roof. The shear walls between the ground level and the third level are constructed of cast-in-place concrete shear walls with a 28 day compressive strength of $f'_c = 8000$ psi. There are two 27', 18" thick shear walls running in the E-W direction. There are three running in the N-S direction with lengths of 26', 29'-4", and 52' which are also 18" thick. There are concrete columns that incorporated into the shear walls that take the axial loads from the steel columns above. This allows the walls to resist just the shear loads transferred down from steel brace frames on the upper floors.

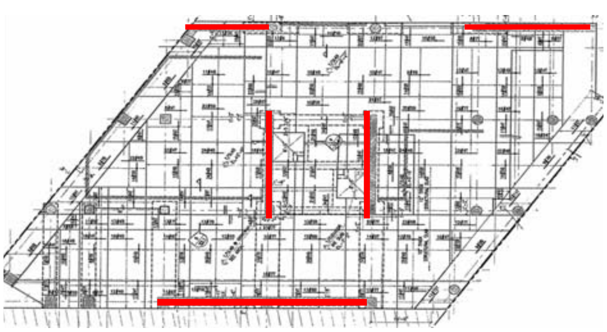


Figure 1: Shear wall location

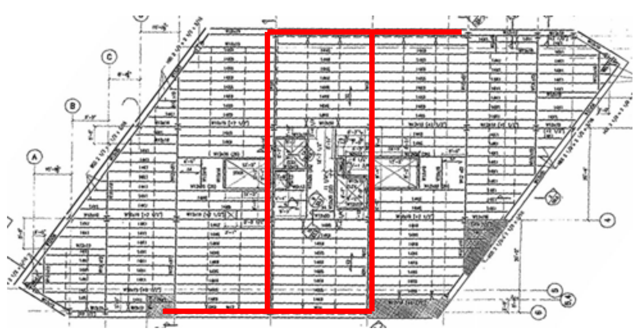


Figure 2: Brace frame location

There are four braced frames used on the upper stories to resist the lateral loads, caused by wind and seismic forces. Two of them run in the north-south direction across the entire width of the building and two of them run east-west and can be seen in the design of the building's façade. The braces were designed as large three story chevrons that would distribute the shear load down from one floor to the next. The diagonal braces are W8 and W10 shapes, the horizontal beams are W12 shapes and the columns are W14 shapes that carry both gravity loads and the axial forces of the braces.

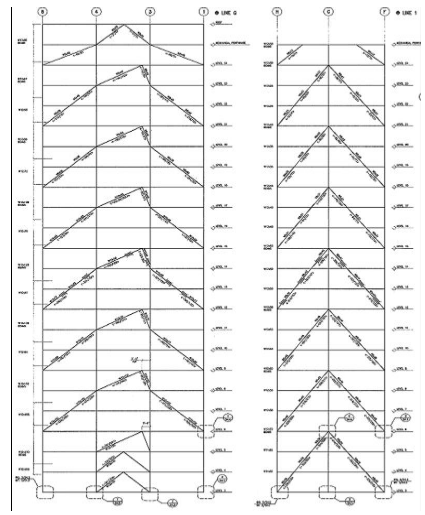


Figure 3: Steel brace design

Lateral Loads

Wind

ASCE-7-02 Ch 6

When considering the wind loads that act on 'Erie on the Park' I found that the wind pressures are distributed along the height of the building as seen in Table 1 below. I assumed that the building was in an exposure category B region. A determination of the building's rigidity as per ASCE-7-02 section 9.5.5 showed that this building is flexible and has a fundamental period 1.41 Hz. The wind pressures were taken and applied to the floor diaphragms based on loading cases 1 and 3 from ASCE-7-02 Figure 6-9 and it was found that a wind in the East-West direction controls the design of this structure if you neglect torsion due to an eccentric load or the abnormal shape of the building. The load distribution to each of the floor diaphragms is tabulated in Appendix A1.

	Wind Ward		Leeward		Total	
	N-S	E-W	N-S	E-W	N-S	E-W
0-15	7.90	7.90	-11.60	-8.63	19.50	16.54
20	8.60	8.60	-11.60	-8.63	20.19	17.23
25	9.15	9.15	-11.60	-8.63	20.75	17.79
30	9.71	9.71	-11.60	-8.63	21.30	18.34
40	10.54	10.54	-11.60	-8.63	22.14	19.17
50	11.23	11.23	-11.60	-8.63	22.83	19.87
60	11.79	11.79	-11.60	-8.63	23.38	20.42
70	12.34	12.34	-11.60	-8.63	23.94	20.98
80	12.90	12.90	-11.60	-8.63	24.49	21.53
90	13.31	13.31	-11.60	-8.63	24.91	21.95
100	13.73	13.73	-11.60	-8.63	25.33	22.36
120	14.42	14.42	-11.60	-8.63	26.02	23.06
140	15.12	15.12	-11.60	-8.63	26.71	23.75
160	15.67	15.67	-11.60	-8.63	27.27	24.30
180	16.22	16.22	-11.60	-8.63	27.82	24.86
200	16.64	16.64	-11.60	-8.63	28.24	25.27
250	17.75	17.75	-11.60	-8.63	29.35	26.38
300	18.72	18.72	-11.60	-8.63	30.32	27.35
350	19.55	19.55	-11.60	-8.63	31.15	28.19
400	20.38	20.38	-11.60	-8.63	31.98	29.02
450	21.08	21.08	-11.60	-8.63	32.68	29.71
500	21.63	21.63	-11.60	-8.63	33.23	30.27
292	18.56	18.56	-11.60	-8.63	30.15	27.19

Table 1: Wind Pressure Distribution

Base Shear:

Case 1:

N-S Direction: 589.5 kips

E-W Direction: 859.4 kips

Case 3: 781.6 kips

Overturning Moment:

Case 1:

N-S Direction 95071 ft-kip

E-W Direction 139429 ft-kip

Case 3: 126596 ft-kip

Seismic Loads ASCE-7 Ch 9

In finding the seismic loads for this building it was determined that the building was in a site class D, which led to the building being in seismic design category B. Since the building was in design category B the equivalent lateral force analysis was used to determine the shear distribution per floor found in the table below. Due to the fact that this building's structural system is not detailed for any seismic resistance, a response modifier (R) of 3 was used when calculating the shear distribution. The rest of the design parameters for seismic loading can be found in Appendix A2.

Floor	Height	Area	Weight	C_{vx}	Shear Distribution	Overturning Moment	Cumulative Shear	Cumulative Moment
	(ft)	(ft^2)	(kip)		(kip)	(ft-kip)	(kip)	(ft-kip)
Roof	291.5	3300	264.0	0.0134	8.73	2545.3	8.73	2545.31
25	274.58	5400	459.0	0.0233	15.18	4168.5	23.91	6713.86
24	263.58	7500	637.5	0.0324	21.09	5557.7	45.00	12271.57
23	252.92	7500	637.5	0.0324	21.09	5332.8	66.08	17604.37
22	242.25	7500	637.5	0.0324	21.09	5107.9	87.17	22712.26
21	231.58	8300	705.5	0.0359	23.33	5403.8	110.50	28116.10
20	220.92	8300	705.5	0.0359	23.33	5154.9	133.84	33271.04
19	210.25	8300	705.5	0.0359	23.33	4906.0	157.17	38177.07
18	199.58	9100	773.5	0.0393	25.58	5106.0	182.76	43283.09
17	188.92	9100	773.5	0.0393	25.58	4833.1	208.34	48116.22
16	178.25	9100	773.5	0.0393	25.58	4560.2	233.92	52676.46
15	167.58	9100	773.5	0.0393	25.58	4287.4	259.51	56963.81
14	156.92	9100	773.5	0.0393	25.58	4014.5	285.09	60978.28
13	145.25	9100	773.5	0.0393	25.58	3716.0	310.67	64694.26
12	135.58	9100	773.5	0.0393	25.58	3468.7	336.26	68162.95
11	124.92	9100	773.5	0.0393	25.58	3195.8	361.84	71358.74
10	114.25	9100	773.5	0.0393	25.58	2922.9	387.42	74281.64
9	103.58	9100	773.5	0.0393	25.58	2650.0	413.01	76931.65
8	92.92	9100	773.5	0.0393	25.58	2377.1	438.59	79308.78
7	82.25	9100	773.5	0.0393	25.58	2104.2	464.17	81413.01
6	71.58	9100	910.0	0.0462	30.10	2154.5	494.27	83567.54
5	60.00	9100	910.0	0.0462	30.10	1805.9	524.37	85373.42
4	50.00	9100	910.0	0.0462	30.10	1504.9	554.47	86878.33
3	40.00	9100	1456.0	0.0740	48.16	1926.3	602.62	88804.61
2	29.75	9100	1456.0	0.0740	48.16	1432.7	650.78	90237.28
1 - Ground	0.00	9100	1592.5	0	0.00	0.0	650.78	90237.28
		219900	19676.0		650.78	90237.3		

Table 2: Seismic Shear Distribution

Base Shear: 650.8 kip

Overturning Moment: 90237 ft-kip

Load Combinations ASCE-7 Ch 2

1) 1.4(D + F)
2) 1.2(D + F + T) +1.6(L + H) + 0.5(L_r or S or R)
3) 1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4) **1.2D + 1.6W + L + 0.5(L_r or S or R)**
5) **1.2D + 1.0E + L + 0.2S**
6) 0.9D + 1.6W + 1.6H
7) 0.9D + 1.0E + 1.6H

These load combinations all have to be taken into consideration when determining the critical load on any member. Typically when designing a member for gravity loading you would use load case 2 which usually simplifies to 1.2D + 1.6L, but when designing member of a braced frame, for instance, the critical load case would probably be either case 4 or 5 depending on whether seismic or wind controlled the design of the building.

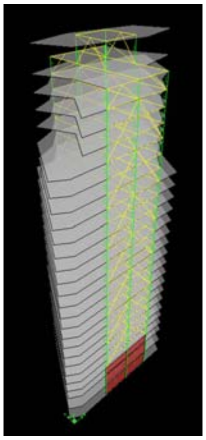
Load Distribution

Realizing that the critical load is the wind in the East-West direction, I found the center of rigidity, based on wall stiffnesses, of the two walls running in the East-West direction. I calculated it for these two walls because they are the two that would resist the direct shear caused by the wind loading. It was found that the two shear walls as well as the two brace frames have the same stiffnesses and are symmetric about the center of mass of the floor slab and thus equally resist any direct shear applied in the East-West direction.

In determining the forces on the shear walls or on the steel members of the braces I just considered the effects of direct shear. I neglected torsion in lieu of a more thorough wind analysis because of the abnormal shape of this building.

ETABS Analysis

In modeling "Erie on the Park' I only modeled the floor slabs and the lateral systems so that the model was less complex and I would be able to better analyze the wind and seismic effects on the lateral systems. I used ASCE-7-98 when modeling the wind load distribution on the building and noticed that the forces the program was using were almost twice what I calculated for this building. For the modeling the seismic loading case I used the IBC 2000 which references the ASCE-7-98 for equivalent lateral load analysis. The values the program produced were much closer to the values I calculated the only difference was a slight variation in the vertical distribution of the loads which led to a slight increase of the overturning moment.



Seismic

	ETABS	Hand Calcs
Building Weight	19591.8	19676
Base Shear	529.49	650.78
Overturn	103505.7	90273.3
Story Distribution		
Roof	19.55	8.73
25	41.01	15.18
24	39.33	21.09
23	36.83	21.09
22	34.14	21.09
21	36.12	23.33
20	33.46	23.33
19	30.68	23.33
18	30.99	25.58
17	28.39	25.58
16	25.72	25.58
15	23.63	25.58
14	21.28	25.58
13	18.87	25.58
12	16.97	25.58
11	14.88	25.58
10	12.79	25.58
9	11.13	25.58
8	9.35	25.58
7	7.60	25.58
6	10.26	30.10
5	7.73	30.10
4	5.69	30.10
3	7.10	48.16
2	5.05	48.16
Ground	0.00	0.00

Wind

	ETABS	Hand Calcs
Base Shear	894.5	894.5
Overturn	139429	139429
Story Distribution		
Roof	29.9	29.9
25	49.34	49.34
24	38.29	38.29
23	37.33	37.33
22	36.59	36.59
21	36.59	36.59
20	36.59	36.59
19	36.59	36.59
18	35.82	35.82
17	35.05	35.05
16	34.66	34.66
15	34.47	34.47
14	35.28	35.28
13	33.7	33.7
12	31.39	31.39
11	32.93	32.93
10	31.97	31.97
9	31.65	31.65
8	30.72	30.72
7	30.14	30.14
6	30.74	30.74
5	29.04	29.04
4	26.19	26.19
3	25.69	25.69
2	48.77	48.77
Ground	35.1	35.1

It is because of the similarities of forces for both the wind and the seismic cases that I believe the story drifts that ETABS calculates are justifiable, but I tend to believe these values are small and would require further investigation into their validity through elastic modeling or virtual work calculations. The values are tabulated in appendix A5

The model was input into ETABS with all the beam, brace and column sizes defined as what the Engineer of Record designed and then asked to do a design check to see if any of the members failed under any of the loading cases. The result of this is that a couple of the bracing members have failed but the wind loads used by ASCE-7 are different than those used by the CBC which would account for the discrepancy.

Spot-Checks

Shear

In checking the components in the building that resist shear, I looked at the shear strength of one of the shear walls between the ground floor and the second floor as well as a brace, chosen at random, between the 15th and 16th floors. I choose these components because they are both aligned in the East-West direction, which is the critical direction under lateral loading.

Overturning

When checking overturning I looked one of the shear walls at the ground level oriented in the East-West direction. It was found that the gravity loads on the wall did not overcome the overturning couple. When I did this calculation I did not consider the weight of the foundation or the fact that there are 90” deep grade beams that tie the caissons below these walls to neighboring caissons thus providing significantly more resistance to uplift.

Conclusion

The loads that were determined by hand calculations from ASCE-7-02 and those that ETABS used to analyze the computer model of ‘Erie on the Park’ were similar and thus produced results that I believe are accurate. The couple members that failed the unity check probably did so because ASCE-7 and the CBC distribute lateral loads different from one another. The results that ETABS was giving for story drift are in the L/2000 range, thus I tend not to trust these results. The results for displacement of the center of gravity of each of the floor diaphragms seems more reasonable with an overall displacement of the roof diaphragm of 4” which leads to a displacement in the L/850 range. These deflections are well below the recommended lateral displacement limits of ASCE-7 which say that drift limits should be on the order of L/600 (Section CB.1.2) or less if the building has brittle cladding such as glass or brick.

Appendix

A1: Wind Load Analysis
A2: Seismic Load Analysis
A3: Center of Rigidity
A4: Spot Checks
- Shear Wall Check
- Brace Check
- Overturning Check
- Column Check

A5: ETABS Results

A1: Wind Loads

Wind Load Analysis

Based on ASCE-7-02: Analytical Procedure

Building Properties:

			Code Referance
Building Width	B (ft)	79.33	
Building Length	L (ft)	130	
Building Height	h (ft)	292	
Topographic Factor	K_{zt}	1	Figure 6.4
Directionality Factor	K_d	0.85	Table 6.4
Basic Wind Speed	V (mph)	90	
Occupancy Category		III	Table 1.1
Hurricane Region		No	
Importance Factor	I_w	1.15	Table 6.1
Exposure Category		B	Section 6.5.6.3
	α	7.0	Table 6.2
	z_g	1200	Table 6.2
	z_{min}	175	Table 6.2
	c	0.300	Table 6.2
	ϵ	0.333	Table 6.2
	l	320	Table 6.2
	$\bar{b}$	0.450	Table 6.2
	$\bar{\alpha}$	0.250	Table 6.2
	$\underline{a}$	0.143	Table 6.2
	$\underline{b}$	0.840	Table 6.2

Period Parameters

Structure Type		Other	Table 9.5.5.3
	C_t	0.020	Table 9.5.5.3.2
	x	0.750	Table 9.5.5.3.2
Fundamental Period	T	1.4109	
Fundamental Frequency	f	0.7087	
Building Rigidity		Flexible	

Gust Effect Factor Section 6.5.8

Rigid — Section 6.5.8.1

$g_Q = g_v$	3.4
ž	175
$I_ž$	0.2272
$L_ž$	557.9243
Q	0.8200
G_f	0.8500

Flexible — Section 6.5.8.2

g_R	4.11
R_n	0.055
N_1	4.39
h_h	10.54
h_B	0.036
h_L	15.74
R_h	0.090
R_B	0.976
R_L	0.062
$V_ž$	90.13
b	0.05
R	0.23
G_f	0.8552

Topographic Effects Section 6.5.7

Pressure Coefficients Section 6.5.11

Internal

Enc. Type	Enclosed		Figure 6.5
Internal (GC_{pi})	0.18	+/-	

Windward

Cp	0.8	Figure 6.6

Leeward

Direction	Ratio	Cp	
N-S	0.610	-0.50	Figure 6.6
E-W	1.639	-0.37	Figure 6.6

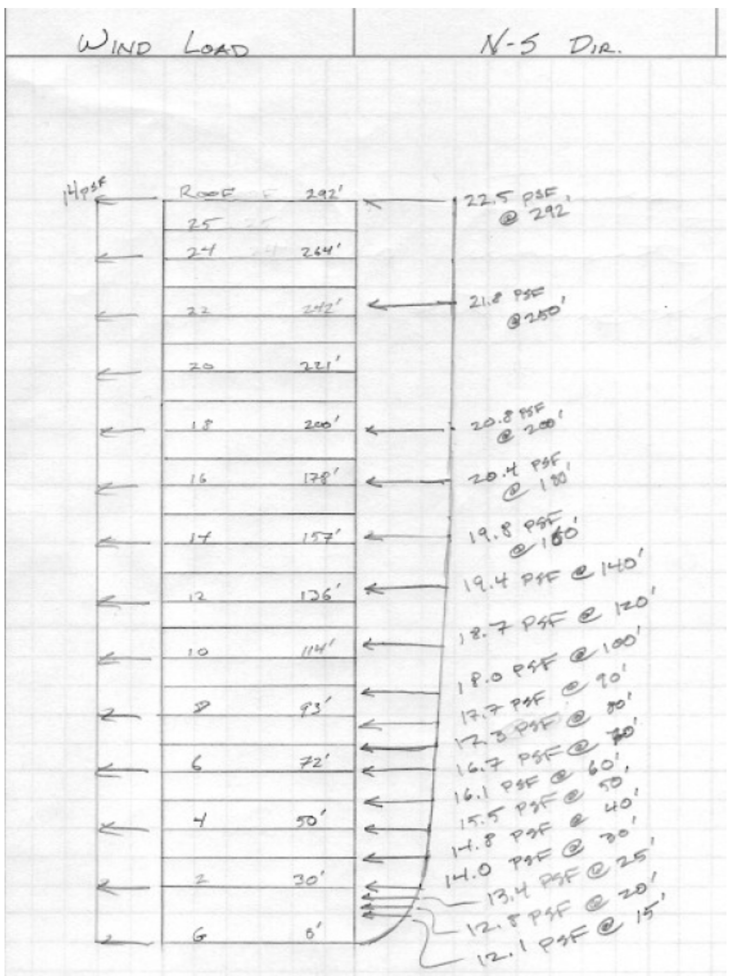
Summary

		Rigid	Flexible
Windward (P_z)	N-S	0.680	0.684
	E-W	0.680	0.684
Leeward (P_h)	N-S	-0.425	-0.428
	E-W	-0.316	-0.318

Wind Pressure

Z(ft)	K_z	q_z
0-15	0.57	11.55
20	0.62	12.57
25	0.66	13.38
30	0.70	14.19
40	0.76	15.40
50	0.81	16.42
60	0.85	17.23
70	0.89	18.04
80	0.93	18.85
90	0.96	19.46
100	0.99	20.07
120	1.04	21.08
140	1.09	22.09
160	1.13	22.90
180	1.17	23.72
200	1.20	24.32
250	1.28	25.94
300	1.35	27.36
350	1.41	28.58
400	1.47	29.80
450	1.52	30.81
500	1.56	31.62
292	1.3381	27.12

	Wind Ward		Leeward		Total	
	N-S	E-W	N-S	E-W	N-S	E-W
0-15	7.90	7.90	-11.60	-8.63	19.50	16.54
20	8.60	8.60	-11.60	-8.63	20.19	17.23
25	9.15	9.15	-11.60	-8.63	20.75	17.79
30	9.71	9.71	-11.60	-8.63	21.30	18.34
40	10.54	10.54	-11.60	-8.63	22.14	19.17
50	11.23	11.23	-11.60	-8.63	22.83	19.87
60	11.79	11.79	-11.60	-8.63	23.38	20.42
70	12.34	12.34	-11.60	-8.63	23.94	20.98
80	12.90	12.90	-11.60	-8.63	24.49	21.53
90	13.31	13.31	-11.60	-8.63	24.91	21.95
100	13.73	13.73	-11.60	-8.63	25.33	22.36
120	14.42	14.42	-11.60	-8.63	26.02	23.06
140	15.12	15.12	-11.60	-8.63	26.71	23.75
160	15.67	15.67	-11.60	-8.63	27.27	24.30
180	16.22	16.22	-11.60	-8.63	27.82	24.86
200	16.64	16.64	-11.60	-8.63	28.24	25.27
250	17.75	17.75	-11.60	-8.63	29.35	26.38
300	18.72	18.72	-11.60	-8.63	30.32	27.35
350	19.55	19.55	-11.60	-8.63	31.15	28.19
400	20.38	20.38	-11.60	-8.63	31.98	29.02
450	21.08	21.08	-11.60	-8.63	32.68	29.71
500	21.63	21.63	-11.60	-8.63	33.23	30.27
292	18.56	18.56	-11.60	-8.63	30.15	27.19



Case 1

			Pressure per Floor											
			Wind Ward		Leeward		Total		Story Force		Cumulative Shear		Overturning Moment	
Floor	Height	Trib Height	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
	(ft)	(ft)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(PSF)	(kip)	(kip)	(kip)	(kip)	(ft-kip)	(ft-kip)
1 - Ground	0.00	0	0	0	0	0	0	0	0	0	589.47	859.42	95071	139429
2	29.75	20.00	10.12	10.12	-11.60	-8.63	21.72	18.76	34.46	48.77	589.47	859.42	1025	1451
3	40.00	10.13	10.89	10.89	-11.60	-8.63	22.48	19.52	18.06	25.69	555.00	810.65	722	1028
4	50.00	10.00	11.51	11.51	-11.60	-8.63	23.11	20.14	18.33	26.19	536.95	784.96	917	1309
5	60.00	10.79	12.06	12.06	-11.60	-8.63	23.66	20.70	20.26	29.04	518.61	758.77	1215	1742
6	71.58	11.13	12.62	12.62	-11.60	-8.63	24.22	21.25	21.37	30.74	498.36	729.73	1530	2200
7	82.25	10.67	13.10	13.10	-11.60	-8.63	24.70	21.74	20.90	30.14	476.99	698.99	1719	2479
8	92.92	10.67	13.52	13.52	-11.60	-8.63	25.12	22.15	21.25	30.72	456.08	668.85	1975	2855
9	103.58	10.67	14.19	14.19	-11.60	-8.63	25.79	22.82	21.82	31.65	434.83	638.13	2260	3278
10	114.25	10.67	14.42	14.42	-11.60	-8.63	26.02	23.06	22.02	31.97	413.01	606.48	2515	3653
11	124.92	10.67	15.12	15.12	-11.60	-8.63	26.71	23.75	22.60	32.93	390.99	574.51	2824	4114
12	135.58	10.17	15.12	15.12	-11.60	-8.63	26.71	23.75	21.54	31.39	368.39	541.57	2921	4256
13	145.25	10.67	15.67	15.67	-11.60	-8.63	27.27	24.30	23.07	33.70	346.84	510.19	3351	4895
14	156.92	11.17	15.67	15.67	-11.60	-8.63	27.27	24.30	24.15	35.28	323.77	476.48	3790	5536
15	167.58	10.67	16.22	16.22	-11.60	-8.63	27.82	24.86	23.54	34.47	299.62	441.20	3945	5777
16	178.25	10.67	16.36	16.36	-11.60	-8.63	27.96	25.00	23.66	34.66	276.07	406.73	4217	6179
17	188.92	10.67	16.64	16.64	-11.60	-8.63	28.24	25.27	23.89	35.05	252.41	372.07	4514	6621
18	199.58	10.67	17.20	17.20	-11.60	-8.63	28.79	25.83	24.36	35.82	228.52	337.02	4863	7148
19	210.25	10.67	17.75	17.75	-11.60	-8.63	29.35	26.38	24.83	36.59	204.16	301.20	5221	7692
20	220.92	10.67	17.75	17.75	-11.60	-8.63	29.35	26.38	24.83	36.59	179.32	264.62	5486	8082
21	231.58	10.67	17.75	17.75	-11.60	-8.63	29.35	26.38	24.83	36.59	154.49	228.03	5751	8473
22	242.25	10.67	17.75	17.75	-11.60	-8.63	29.35	26.38	24.83	36.59	129.66	191.45	6016	8863
23	252.92	10.67	18.29	18.29	-11.60	-8.63	29.88	26.92	25.29	37.33	104.82	154.86	6396	9442
24	263.58	10.83	18.56	18.56	-11.60	-8.63	30.15	27.19	25.91	38.29	79.53	117.53	6830	10093
25	274.58	13.96	18.56	18.56	-11.60	-8.63	30.15	27.19	33.39	49.34	53.62	79.24	9168	13547
Roof	291.5	8.46	18.56	18.56	-11.60	-8.63	30.15	27.19	20.23	29.90	20.23	29.90	5898	8715

Case 3

			Story Force									
			NW-SE Direction			NE-SW Direction			Cumulative Shear		Overturning Moment	
Floor	Height	Trib Height	N-S	E-W	Total	N-S	E-W	Total	N-S	E-W	N-S	E-W
	(ft)	(ft)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(ft-kip)	(ft-kip)
1 - Ground	0.00	0	0.00	0.00	0.00	0.00	0.00	0.00	781.62	781.62	126569	126569
2	29.75	20.00	25.85	36.58	44.79	25.85	36.58	44.79	781.62	781.62	1332	1332
3	40.00	10.13	13.54	19.27	23.55	13.54	19.27	23.55	736.84	736.84	942	942
4	50.00	10.00	13.75	19.64	23.97	13.75	19.64	23.97	713.28	713.28	1199	1199
5	60.00	10.79	15.19	21.78	26.55	15.19	21.78	26.55	689.31	689.31	1593	1593
6	71.58	11.13	16.03	23.05	28.08	16.03	23.05	28.08	662.76	662.76	2010	2010
7	82.25	10.67	15.68	22.61	27.51	15.68	22.61	27.51	634.68	634.68	2263	2263
8	92.92	10.67	15.94	23.04	28.02	15.94	23.04	28.02	607.17	607.17	2603	2603
9	103.58	10.67	16.37	23.74	28.83	16.37	23.74	28.83	579.15	579.15	2987	2987
10	114.25	10.67	16.51	23.98	29.11	16.51	23.98	29.11	550.32	550.32	3326	3326
11	124.92	10.67	16.95	24.70	29.96	16.95	24.70	29.96	521.20	521.20	3742	3742
12	135.58	10.17	16.16	23.54	28.55	16.16	23.54	28.55	491.24	491.24	3871	3871
13	145.25	10.67	17.30	25.28	30.63	17.30	25.28	30.63	462.69	462.69	4449	4449
14	156.92	11.17	18.12	26.46	32.07	18.12	26.46	32.07	432.06	432.06	5032	5032
15	167.58	10.67	17.66	25.85	31.31	17.66	25.85	31.31	399.99	399.99	5247	5247
16	178.25	10.67	17.74	26.00	31.48	17.74	26.00	31.48	368.68	368.68	5611	5611
17	188.92	10.67	17.92	26.29	31.81	17.92	26.29	31.81	337.21	337.21	6010	6010
18	199.58	10.67	18.27	26.86	32.49	18.27	26.86	32.49	305.39	305.39	6484	6484
19	210.25	10.67	18.62	27.44	33.16	18.62	27.44	33.16	272.90	272.90	6973	6973
20	220.92	10.67	18.62	27.44	33.16	18.62	27.44	33.16	239.74	239.74	7326	7326
21	231.58	10.67	18.62	27.44	33.16	18.62	27.44	33.16	206.58	206.58	7680	7680
22	242.25	10.67	18.62	27.44	33.16	18.62	27.44	33.16	173.41	173.41	8034	8034
23	252.92	10.67	18.97	28.00	33.82	18.97	28.00	33.82	140.25	140.25	8553	8553
24	263.58	10.83	19.44	28.72	34.68	19.44	28.72	34.68	106.43	106.43	9140	9140
25	274.58	13.96	25.04	37.00	44.68	25.04	37.00	44.68	71.76	71.76	12269	12269
Roof	291.5	8.46	15.17	22.42	27.08	15.17	22.42	27.08	27.08	27.08	7892	7892

A2: Seismic Loads

Seismic Load Analysis

Based on ASCE-7-02

Building Information			Code Reference
Location		Chicago, IL	
Number of floors:	N	25	
Building Height:	h_n	291.5	
Inter-story height	h_s	10	
Occupancy Category		III	Table 1-1
Seismic Use Group	I	II	Table 9.1.3
Importance Factor		1.25	Table 9.1.4
Site Class		D	Table 9.4.1.2
	S_s	0.17	
	S_1	0.07	
	F_a	1.6	Table 9.4.1.2a
	F_v	2.4	Table 9.4.1.2b
	S_{MS}	0.272	
	S_{M1}	0.168	
	S_{DS}	0.181	
	S_{D1}	0.112	

Design Category:

Based on S_{DS}	B	Table 9.4.2.1a
Based on S_{D1}	B	Table 9.4.2.1b
Applicable Design Category:	**B**	
	Use Equivalent Lateral Force Analysis	

Design Parameters:			
Response Modifier	R	3	Table 9.5.2.2
Over-Strength Factor	Ω_0	3	Table 9.5.2.2
Deflection Amplifier	C_d	3	Table 9.5.2.2
	ρ	1.0	
Structure Type		Other	Table 9.5.5.3
	C_t	0.020	Table 9.5.5.3.2
	x	0.750	Table 9.5.5.3.2
Natural Period	T	1.4109	
	T_0	0.124	
	T_s	0.618	

Weights:

Dead Loads:

	Slab/Deck	Beams/joists	Superimposed	Total
Roof	50	10	20	80
7-25	40	15	15	70
4-6	80	10	10	100
2-3	150	N/A	10	160
Ground	150	N/A	25	175

Live Loads:

Grond Floor	100	
Parking	50	
Residential Units	40	Dwelling
	15	Partition
Corridors	40	
Balconies	40	
Roof	25 + Drift	

Equivalent Lateral Force Section 9.5.5

S_{DS}	0.181	I	1.25
S_{D1}	0.112	T	1.411
R	3		
k	1.27	C_S	0.0330748

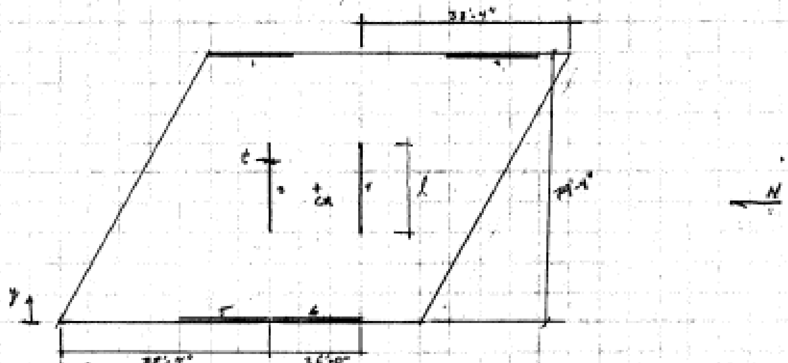
Floor	Height	Area	Weight	C_{vx}	Shear Distribution	Overturning Moment	Cumulative Shear	Cumulative Moment
	(ft)	(ft^2)	(kip)		(kip)	(ft-kip)	(kip)	(ft-kip)
Roof	291.5	3300	264.0	0.0134	8.73	2545.3	8.73	2545.31
25	274.58	5400	459.0	0.0233	15.18	4168.5	23.91	6713.86
24	263.58	7500	637.5	0.0324	21.09	5557.7	45.00	12271.57
23	252.92	7500	637.5	0.0324	21.09	5332.8	66.08	17604.37
22	242.25	7500	637.5	0.0324	21.09	5107.9	87.17	22712.26
21	231.58	8300	705.5	0.0359	23.33	5403.8	110.50	28116.10
20	220.92	8300	705.5	0.0359	23.33	5154.9	133.84	33271.04
19	210.25	8300	705.5	0.0359	23.33	4906.0	157.17	38177.07
18	199.58	9100	773.5	0.0393	25.58	5106.0	182.76	43283.09
17	188.92	9100	773.5	0.0393	25.58	4833.1	208.34	48116.22
16	178.25	9100	773.5	0.0393	25.58	4560.2	233.92	52676.46
15	167.58	9100	773.5	0.0393	25.58	4287.4	259.51	56963.81
14	156.92	9100	773.5	0.0393	25.58	4014.5	285.09	60978.28
13	145.25	9100	773.5	0.0393	25.58	3716.0	310.67	64694.26
12	135.58	9100	773.5	0.0393	25.58	3468.7	336.26	68162.95
11	124.92	9100	773.5	0.0393	25.58	3195.8	361.84	71358.74
10	114.25	9100	773.5	0.0393	25.58	2922.9	387.42	74281.64
9	103.58	9100	773.5	0.0393	25.58	2650.0	413.01	76931.65
8	92.92	9100	773.5	0.0393	25.58	2377.1	438.59	79308.78
7	82.25	9100	773.5	0.0393	25.58	2104.2	464.17	81413.01
6	71.58	9100	910.0	0.0462	30.10	2154.5	494.27	83567.54
5	60.00	9100	910.0	0.0462	30.10	1805.9	524.37	85373.42
4	50.00	9100	910.0	0.0462	30.10	1504.9	554.47	86878.33
3	40.00	9100	1456.0	0.0740	48.16	1926.3	602.62	88804.61
2	29.75	9100	1456.0	0.0740	48.16	1432.7	650.78	90237.28
1 - Ground	0.00	9100	1592.5	0	0.00	0.0	650.78	90237.28
		219900	19676.0		650.78	90237.3		

Seismic Base S V 650.78 kips

A3: Center of Rigidity

CENTER OF RIGIDITY | E-W DIR.

SHEAR WALLS

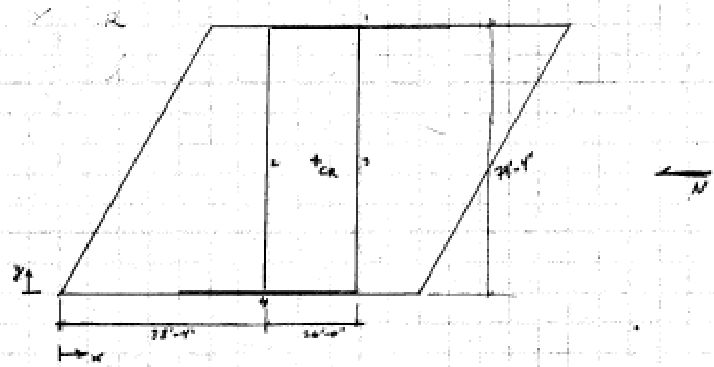


X - DIR

$l_3 = l_4$; $t_3 = t_4$; $h_3 = h_4$ ∴ $k_3 = k_4$

$$\bar{x} = \frac{k_3(38.33) + k_4(64.33)}{k_3 + k_4} = 57.33'$$

BRACE FRAMES



X - DIR

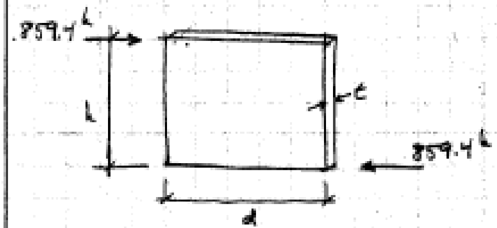
$l_2 = l_3$; $h_2 = h_3$; IDENTICAL BRACING SCHEMES ∴ $k_2 = k_3$

$$\bar{x} = \frac{k_2(38.33) + k_3(64.33)}{k_2 + k_3} = 57.33'$$

A4: Spot Checks

SHEAR CHECK

WALL C & D @ FLOOR 1



$t = 18"$
$d = 312"$
$h = 357"$
$f'_c = 8000$ psi

$V_u = 859.4^k$
$\phi = 0.75$

BOTH WALLS IN E-W DIRECTION HAVE THE SAME STIFFNESS ∴ ½ LOAD TO EACH WALL

$V_u = 429.7^k \qquad V_n = \frac{V_u}{\phi} = 572.9^k$

WALL ONLY TAKES SHEAR & FLEXURE

$V_c = 2\sqrt{f'_c}\, t d = 1004.6^k \qquad \frac{\phi V_c}{2} = 376.7 < V_u$

REINFORCEMENT REQD.!

USE SPACING OF 12"

$A_v = 0.75\sqrt{f'_c}\,\frac{ts}{f_y} = 0.24\ in^2$

$A_v \geq 50\, ts/f_y = 0.18\ in^2$

USE #5 BARS EACH FACE

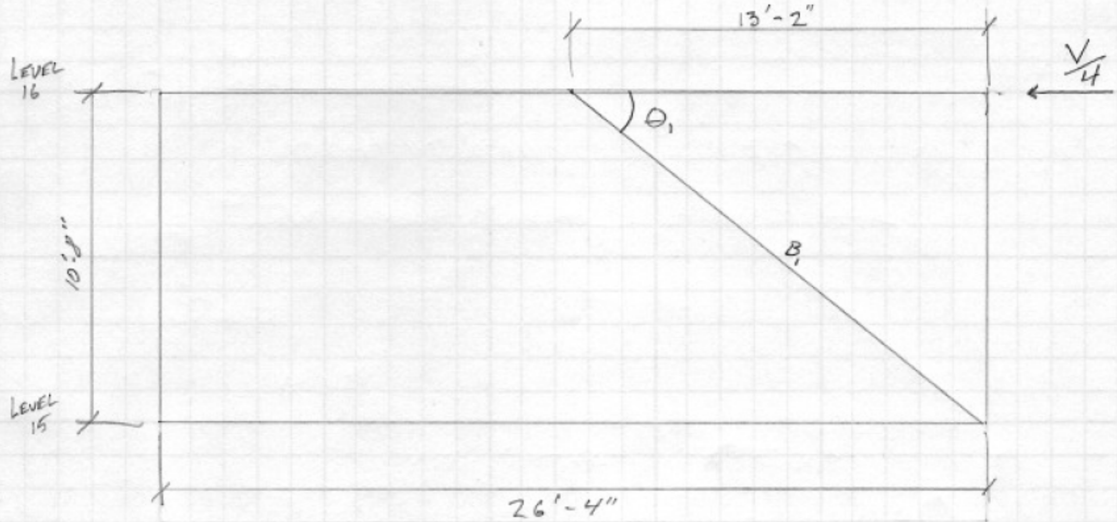
$A_b = 0.62\ in^2 > 0.24\ in^2$ OK

$V_s = \frac{A_s f_y d}{s} = 967.2^k$

$\phi V_n = 0.75(1004.6 + 967.2) = 1478.9^k > 429.7^k$

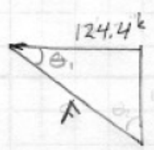
WALL OK IN SHEAR!

LAT. BRACE



BRACE IS 1 OF 4 IDENTICAL ONES BETWEEN LEVELS 15 & 16 IN THE E-W DIRECTION

$V = 497.5^k \qquad \frac{V}{4} = 124.4^k$



$$\theta_1 = \tan^{-1}\left(\frac{10'-8"}{13'-2"}\right) = 39°$$

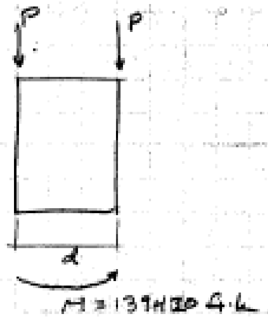
$$L_{B_1} = \frac{13'2"}{\sin\theta_1} = 20'-11"$$

$$F = \frac{124.4}{\sin\theta_1} = 197.6^k = P_u$$

TENSION W8×15
COMPRESSION W10×49 ← USE

DESIGN ENGINEER USED W10×68

	OVERTURN CHECK	WALL C OR D

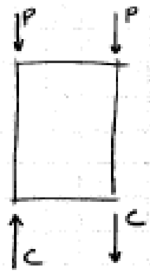


$$d = 26'$$

$$P = 2190^k$$

BOTH WALLS IN E-W DIRECTION HAVE THE SAME STIFFNESS ∴ ½ LOAD TO EACH

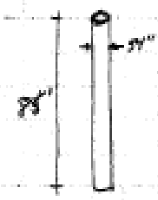
$$M_u = 69715 \text{ ft·k}$$



RESOLVE TO COUPLE

$$C = \frac{M_u}{d} = 2681^k > 2190^k$$

UPLIFT WOULD OCCURE, BUT CALCULATION FOR P DID NOT INCLUDE WEIGHT OF FOUNDATION



$$\omega = \left(\tfrac{54}{12}\left(\tfrac{1}{2}\right)\right)^2 \pi (85)(150) = 203^k$$

← WEIGHT OF CAISSON BELOW CORNER OF WALL

THIS ADDITIONAL WEIGHT STILL NOT ENOUGH TO RESIST OVERTURNING MOMENT

<u>Column Check</u>

Roof Live Load:

DL	80
L_o	50
N-S Span	26
E-W Span	26.33
A_T	684.58
R_1	1
R_2	1

Live Load Reduction

DL	70
L_o	40
N-S Span	26
E-W Span	26.33
A_T	684.58
K_{LL}	4

Column Below:	L_o (PSF)	Dead (PSF)	A_T (ft2)	Reduction	Live (PSF)	Live Load (kip)	Dead Load (kip)	Total Load (kip)
Roof	50	80	684.58	1.000	50	34.2	54.8	120.5
Floor 25	40	70	684.58	0.537	21.47	14.7	102.7	146.7
24	40	70	1369.16	0.453	18.11	24.8	150.6	220.4
23	40	70	2053.74	0.415	16.62	34.1	198.5	292.8
22	40	70	2738.32	0.393	16.00	43.8	246.4	365.8
21	40	70	3422.9	0.378	16.00	54.8	294.4	440.9
20	40	70	4107.48	0.367	16.00	65.7	342.3	515.9
19	40	70	4792.06	0.358	16.00	76.7	390.2	590.9
18	40	70	5476.64	0.351	16.00	87.6	438.1	666.0
17	40	70	6161.22	0.346	16.00	98.6	486.1	741.0
16	40	70	6845.8	0.341	16.00	109.5	534.0	816.0
15	40	70	7530.38	0.336	16.00	120.5	581.9	891.0
14	40	70	8214.96	0.333	16.00	131.4	629.8	966.1
13	40	70	8899.54	0.330	16.00	142.4	677.7	1041.1
12	40	70	9584.12	0.327	16.00	153.3	725.7	1116.1
11	40	70	10268.7	0.324	16.00	164.3	773.6	1191.2
10	40	70	10953.28	0.322	16.00	175.3	821.5	1266.2
9	40	70	11637.86	0.320	16.00	186.2	869.4	1341.2
8	40	70	12322.44	0.318	16.00	197.2	917.3	1416.3
7	40	70	13007.02	0.316	16.00	208.1	965.3	1491.3
6	50	100	13691.6	0.314	20.00	273.8	1033.7	1678.6
5	50	100	14376.18	0.313	20.00	287.5	1102.2	1782.6
4	50	100	15060.76	0.311	20.00	301.2	1170.6	1886.7
3	50	160	15745.34	0.310	20.00	314.9	1280.2	2040.0
2	50	160	16429.92	0.309	20.00	328.6	1389.7	2193.4

Table A4.1 Column Axial Loads

COLUMN CHECK

- CHECK FOR GRAVITY LOADS ONLY
- NEGLECT MOMENT FROM LL. PATTERN LOADING

COLUMN BELOW LEVEL 6:

AXIAL LOAD: $P_u = 1680^k$
UNBRACED LENGTH $L_b = 11.6'$

W14 × 159

DESIGN ENGINEER USED W14 × 193

COLUMN BELOW LEVEL 12:

AXIAL LOAD: $P_u = 1116^k$
$L_b = 11.6'$

W14 × 109

DESIGN ENGINEER USED W14 × 145

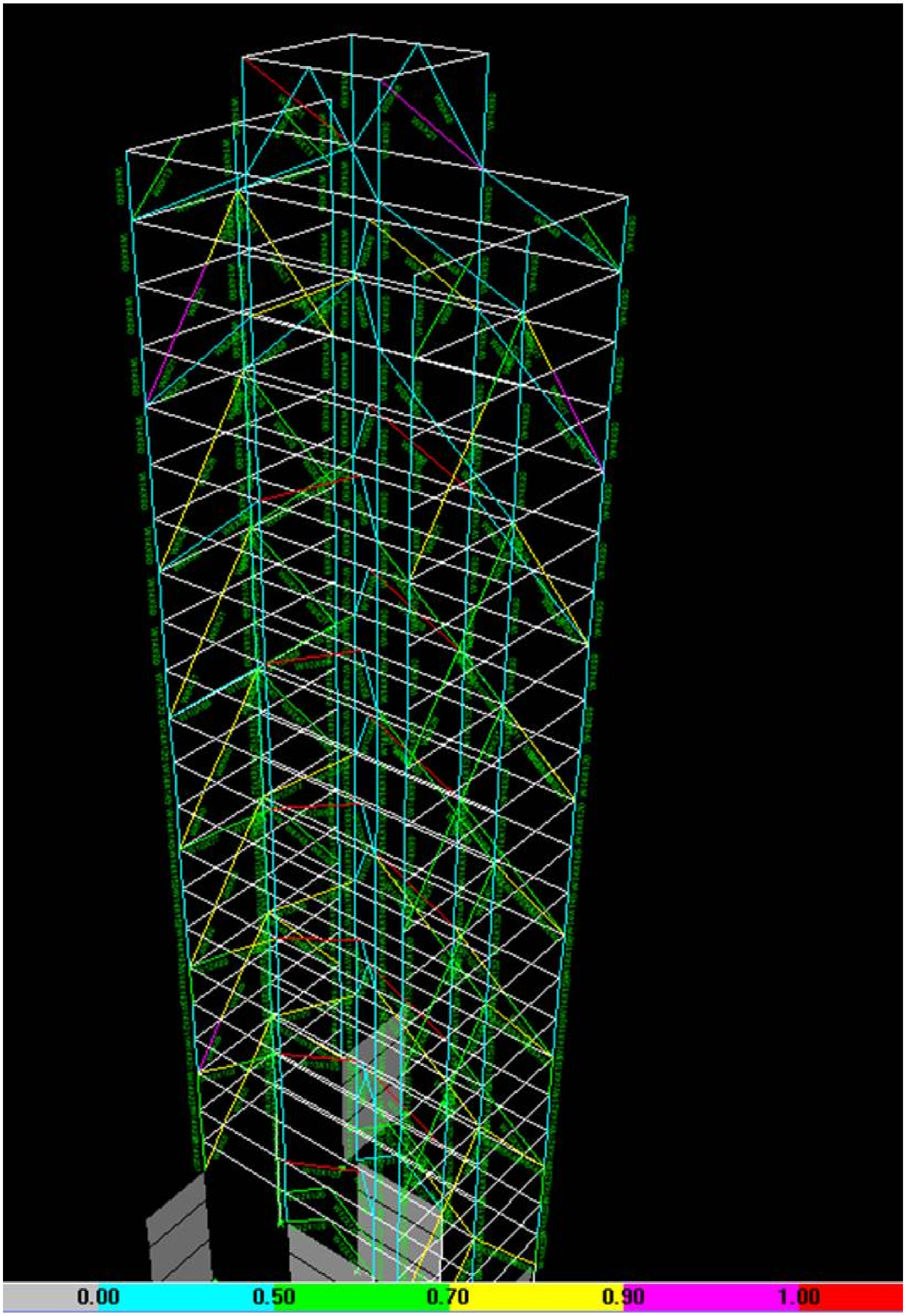
COLUMN BELOW LEVEL 18:

$P_u = 665.9^k$
$L_b = 11.6'$

W14 × 82

DESIGN ENGINEER USED W14 × 99

A5: ETABS Results



Unity Check of Steel Braces

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
STORY3	A	WINDY1	Top	-975.93	-530.43	-7.54	-1619.938	1241.807	120915.3
STORY3	A	WINDY1	Bottom	-975.93	-530.43	-7.54	-1619.938	314.916	55672.91
STORY2	A	WINDY1	Top	-975.93	-161.26	4.1	173.146	314.916	55672.91
STORY2	A	WINDY1	Bottom	-975.93	-161.26	4.1	173.146	1023.698	27774.93
STORY1	A	WINDY1	Top	-975.93	-26.59	-5.56	-49.076	1023.698	27774.93
STORY1	A	WINDY1	Bottom	-975.93	-26.59	-5.56	-49.076	0	22881.92
STORY3	B	WINDY1	Top	-186.12	487.78	-7.43	1871.074	1202.698	-31715.27
STORY3	B	WINDY1	Bottom	-186.12	487.78	-7.43	1871.074	288.589	28282.19
STORY2	B	WINDY1	Top	-186.12	138.71	5.19	-385.869	288.589	28282.19
STORY2	B	WINDY1	Bottom	-186.12	138.71	5.19	-385.869	1186.03	52278.84
STORY1	B	WINDY1	Top	-186.12	10.34	-6.45	42.842	1186.03	52278.84
STORY1	B	WINDY1	Bottom	-186.12	10.34	-6.45	42.842	0	54180.55
STORY3	C	WINDY1	Top	-472.09	692.13	-1.44	-180.185	38.376	124747.1
STORY3	C	WINDY1	Bottom	-472.09	692.13	-1.44	-180.185	-138.186	209879.4
STORY2	C	WINDY1	Top	-640.74	650.03	0.25	-60.626	-137.328	184020.6
STORY2	C	WINDY1	Bottom	-640.74	650.03	0.25	-60.626	-93.759	296476.4
STORY1	C	WINDY1	Top	-849.29	702.76	0.52	-13.82	-94.376	264154.8
STORY1	C	WINDY1	Bottom	-849.29	702.76	0.52	-13.82	0.41	393462.3
STORY3	D	WINDY1	Top	-232.09	592.2	-1.44	-180.185	38.376	124556
STORY3	D	WINDY1	Bottom	-232.09	592.2	-1.44	-180.185	-138.186	197396.4
STORY2	D	WINDY1	Top	-386.66	625.11	0.25	-60.626	-137.328	173680.1
STORY2	D	WINDY1	Bottom	-386.66	625.11	0.25	-60.626	-93.759	281824.9
STORY1	D	WINDY1	Top	-591.54	704.47	0.52	-13.82	-94.376	250071.2
STORY1	D	WINDY1	Bottom	-591.54	704.47	0.52	-13.82	0.41	379694.2
STORY3	E	WINDY1	Top	5.84	19.94	5.31	-117.339	-0.269	-910.636
STORY3	E	WINDY1	Bottom	5.84	19.94	5.31	-117.339	652.805	1542.36
STORY2	E	WINDY1	Top	16.62	11.54	2.28	-57.508	652.155	-139.136
STORY2	E	WINDY1	Bottom	16.62	11.54	2.28	-57.508	1046.673	1856.859
STORY1	E	WINDY1	Top	25.47	8.88	-5.69	-11.107	1047.147	475.754
STORY1	E	WINDY1	Bottom	25.47	8.88	-5.69	-11.107	-0.445	2109.756
STORY3	F	WINDY1	Top	-5.84	19.94	4.74	-117.339	0.269	-910.636
STORY3	F	WINDY1	Bottom	-5.84	19.94	4.74	-117.339	583.101	1542.36
STORY2	F	WINDY1	Top	-16.62	11.54	2.38	-57.508	583.752	-139.136
STORY2	F	WINDY1	Bottom	-16.62	11.54	2.38	-57.508	995.502	1856.859
STORY1	F	WINDY1	Top	-25.47	8.88	-5.41	-11.107	995.028	475.754
STORY1	F	WINDY1	Bottom	-25.47	8.88	-5.41	-11.107	0.445	2109.756

Forces Acting on Shear Walls

Story	Diaphragm	Load	UX	UY	UZ	RX	RY	RZ	Point	X	Y	Z
STORY26	D1	WINDYY	-1.1079	3.1956	0	0	0	0.00042	2439	928.017	476.5	3495.5
STORY25	D1	WINDYY	-1.0257	3.1273	0	0	0	0.00044	2440	928.709	476.5	3295
STORY24	D1	WINDYY	-0.9672	3.0233	0	0	0	0.00044	2441	1011.899	502.816	3163
STORY23	D1	WINDYY	-0.8969	2.9399	0	0	0	0.00045	2442	1012.156	502.897	3035
STORY22	D1	WINDYY	-0.8251	2.8131	0	0	0	0.00044	2443	1012.687	503.064	2907
STORY21	D1	WINDYY	-0.7424	2.5981	0	0	0	0.00036	2444	928.5	476.5	2779
STORY20	D1	WINDYY	-0.6726	2.4882	0	0	0	0.00032	2445	928.5	476.5	2651
STORY19	D1	WINDYY	-0.6045	2.3509	0	0	0	0.0003	2446	928.5	476.5	2523
STORY18	D1	WINDYY	-0.5382	2.1633	0	0	0	0.00027	2447	928.5	476.5	2395
STORY17	D1	WINDYY	-0.474	2.0361	0	0	0	0.00024	2448	928.5	476.5	2267
STORY16	D1	WINDYY	-0.4126	1.8994	0	0	0	0.00021	2449	928.5	476.5	2139
STORY15	D1	WINDYY	-0.3543	1.7189	0	0	0	0.00019	2450	928.5	476.5	2011
STORY14	D1	WINDYY	-0.2987	1.566	0	0	0	0.00016	2451	928.5	476.5	1883
STORY13	D1	WINDYY	-0.2471	1.4311	0	0	0	0.00014	2452	928.5	476.5	1755
STORY12	D1	WINDYY	-0.1996	1.2647	0	0	0	0.00012	2453	928.5	476.5	1627
STORY11	D1	WINDYY	-0.1557	1.1023	0	0	0	0.0001	2454	928.5	476.5	1499
STORY10	D1	WINDYY	-0.1169	0.9831	0	0	0	0.00008	2455	928.5	476.5	1371
STORY9	D1	WINDYY	-0.0832	0.8434	0	0	0	0.00006	2456	928.5	476.5	1243
STORY8	D1	WINDYY	-0.0542	0.6895	0	0	0	0.00005	2457	928.5	476.5	1115
STORY7	D1	WINDYY	-0.031	0.6041	0	0	0	0.00003	2458	928.5	476.5	987
STORY6	D1	WINDYY	-0.013	0.5009	0	0	0	0.00001	2459	928.5	476.5	859
STORY5	D1	WINDYY	0.0025	0.3285	0	0	0	0	2460	928.5	476.5	720
STORY4	D1	WINDYY	0.0093	0.2077	0	0	0	-0.00001	2461	928.5	476.5	600
STORY3	D1	WINDYY	0.0084	0.103	0	0	0	-0.00002	2462	929.952	471.749	480
STORY2	D1	WINDYY	0.0044	0.0634	0	0	0	-0.00001	2463	931.546	466.534	357
STORY1	D1	WINDYY	0.0011	0.0214	0	0	0	0	2464	935.213	454.534	184
BASE	D1	WINDYY	0	0	0	0	0	0	2465	933.1	461.447	0

Displacement of Diaphragm Centers of Mass

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
STORY26	Max Drift X	WINDYY	32	1084.5	212	3495.5	0.000432	
STORY26	Max Drift Y	WINDYY	29	336.625	634.5	3495.5		0.000391
STORY25	Max Drift X	WINDYY	10	460.5	0	3295	0.000538	
STORY25	Max Drift Y	WINDYY	29	336.625	634.5	3295		0.001074
STORY24	Max Drift X	WINDYY	34	236.375	0	3163	0.000603	
STORY24	Max Drift Y	WINDYY	34	236.375	0	3163		0.000735
STORY23	Max Drift X	WINDYY	15	1736.625	953	3035	0.000614	
STORY23	Max Drift Y	WINDYY	14	1857	876.5	3035		0.001091
STORY22	Max Drift X	WINDYY	15	1736.625	953	2907	0.000835	
STORY22	Max Drift Y	WINDYY	14	1857	876.5	2907		0.001937
STORY21	Max Drift X	WINDYY	15	1736.625	953	2779	0.000696	
STORY21	Max Drift Y	WINDYY	14	1857	876.5	2779		0.00115
STORY20	Max Drift X	WINDYY	15	1736.625	953	2651	0.00063	
STORY20	Max Drift Y	WINDYY	14	1857	876.5	2651		0.001265
STORY19	Max Drift X	WINDYY	15	1736.625	953	2523	0.000619	
STORY19	Max Drift Y	WINDYY	14	1857	876.5	2523		0.001663
STORY18	Max Drift X	WINDYY	16	524.375	953	2395	0.000612	
STORY18	Max Drift Y	WINDYY	14	1857	876.5	2395		0.001209
STORY17	Max Drift X	WINDYY	16	524.375	953	2267	0.000578	
STORY17	Max Drift Y	WINDYY	14	1857	876.5	2267		0.00126
STORY16	Max Drift X	WINDYY	16	524.375	953	2139	0.00055	
STORY16	Max Drift Y	WINDYY	14	1857	876.5	2139		0.001594
STORY15	Max Drift X	WINDYY	16	524.375	953	2011	0.000529	
STORY15	Max Drift Y	WINDYY	14	1857	876.5	2011		0.00138
STORY14	Max Drift X	WINDYY	16	524.375	953	1883	0.000486	
STORY14	Max Drift Y	WINDYY	14	1857	876.5	1883		0.001215
STORY13	Max Drift X	WINDYY	16	524.375	953	1755	0.000447	
STORY13	Max Drift Y	WINDYY	14	1857	876.5	1755		0.001447
STORY12	Max Drift X	WINDYY	16	524.375	953	1627	0.000417	
STORY12	Max Drift Y	WINDYY	14	1857	876.5	1627		0.001414
STORY11	Max Drift X	WINDYY	16	524.375	953	1499	0.000375	
STORY11	Max Drift Y	WINDYY	14	1857	876.5	1499		0.00107
STORY10	Max Drift X	WINDYY	16	524.375	953	1371	0.000325	
STORY10	Max Drift Y	WINDYY	14	1857	876.5	1371		0.001213
STORY9	Max Drift X	WINDYY	16	524.375	953	1243	0.000279	
STORY9	Max Drift Y	WINDYY	14	1857	876.5	1243		0.001305
STORY8	Max Drift X	WINDYY	16	524.375	953	1115	0.000249	
STORY8	Max Drift Y	WINDYY	14	1857	876.5	1115		0.000799
STORY7	Max Drift X	WINDYY	16	524.375	953	987	0.000202	
STORY7	Max Drift Y	WINDYY	14	1857	876.5	987		0.000924
STORY6	Max Drift X	WINDYY	16	524.375	953	859	0.000147	
STORY6	Max Drift Y	WINDYY	14	1857	876.5	859		0.001311
STORY5	Max Drift X	WINDYY	16	524.375	953	720	0.000104	
STORY5	Max Drift Y	WINDYY	14	1857	876.5	720		0.001098
STORY4	Max Drift X	WINDYY	13	1332.625	0	600	0.000039	
STORY4	Max Drift Y	WINDYY	14	1857	876.5	600		0.000934
STORY3	Max Drift X	WINDYY	16	524.375	953	480	0.000063	
STORY3	Max Drift Y	WINDYY	11	0	76.5	480		0.000381
STORY2	Max Drift X	WINDYY	16	524.375	953	357	0.000037	
STORY2	Max Drift Y	WINDYY	11	0	76.5	357		0.000277
STORY1	Max Drift X	WINDYY	16	524.375	953	184	0.000012	
STORY1	Max Drift Y	WINDYY	11	0	76.5	184		0.000127

Story Drift (in)