33 Harry Agganis Way

Boston, Massachusetts

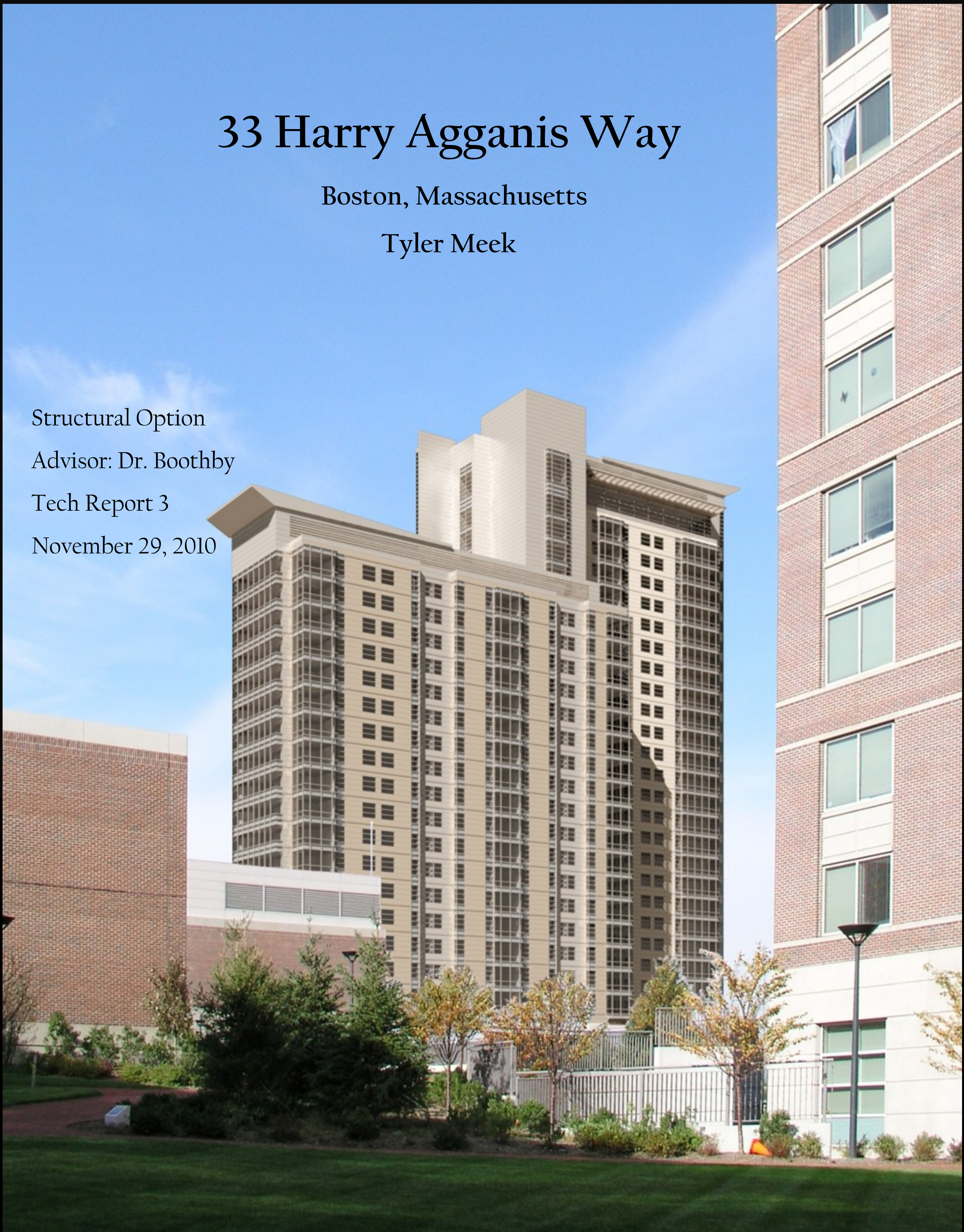
Tyler Meek

Structural Option

Advisor: Dr. Boothby

Tech Report 3

November 29, 2010



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Executive Summary

Res Tower II is a 26 story, 296 ft tall, dormitory located in Boston, Massachusetts. There are three levels of public lobby and presentation space with 23 levels of private study and living spaces. A steel framing system supports the lightweight concrete composite floor system and the lateral loads are resisted by moment connected steel braced frames connected to a mat foundation.

The goal of this technical report entitled, "*Lateral System Analysis and Confirmation Design,"* is to evaluate the existing lateral system of Res Tower II and confirm that it has sufficient strength and meets serviceability requirements. This report also includes the determination of the controlling load case and load combination, and how these loads are distributed throughout the structure. Following the rule that load follows stiffness, the relative stiffness of each braced frame was determined using Etabs. By placing a 1 kip load on each frame, the stiffness can be determined by inverting the frames deflection. Story shears are distributed to each frame in the form of direct shear and torsional shear. These shears were calculated by hand using the relative stiffnesses. Drift values of Res Tower II were limited to H/400, where H is the floor to floor height of each level, and it was determined that the structure meets this serviceability requirement. Overturning moments and uplift forces were calculated and evaluated for their impacts on the mat foundation. Spot checks were completed at critical locations of the structure.

After the lateral system of Res Tower II was fully evaluated, it was found to be adequate in all strength and serviceability requirements.

Introduction

Located on the Boston University Campus, 33 Harry Agganis Way, which will be referred to as Res Tower II, is a 27 story, steel framed dormitory. It is located on the northwest corner of the John Hancock Student Village, bordered by the Charles River and Commonwealth Ave. Because two more dormitories are planned for the JH Student Village and the cost of developing in Boston is so high, the footprint of Res Tower II had to be as small as possible, thus forcing the structure to be tall.

The south tower is 19 stories tall with a fan room and mechanical penthouse on the top level. A student activity space, with large windows and a terracotta surfaced walkout space, occupies the 27th story of the north tower. The roof of the north tower supports a fan room, large air handling units and other large service equipment. Floors 3 through 26, aside from the spaces mentioned above, are all private residential areas with some study rooms and computer labs mixed in. The first two levels of Res Tower II serve as the public and service offices for the rest of the building.

The façade of Res Tower II is a panelized skin comprised of terracotta and a metal panel rainscreen. This façade is a curtain wall system with its self-weight being supported by the floor above it; this can be assumed to be a continuous load due the small spacing of hung supports.

Res Tower II utilizes four main roof systems, all of which include gypsum under-laminate board, a vapor retarder and an adhered roofing membrane; the prior three aspects will be referred to as the typical roof assembly. Where mechanical equipment is being supported the typical roof assembly is placed on concrete deck while on the outer edges of the building, a metal deck is used. On the 26th story, to support the walkout space mentioned above, terracotta pavers on concrete deck are combined with the typical roof assembly to create an attractive and durable roof system.

Structural Systems

Foundation

Haley & Aldrich performed the geotechnical studies for the JH Student Village area and provided the report in which H&A explain site and below-grade conditions along with recommendations for the structure. A net allowable soil bearing pressure of 6 kips per square foot (ksf) was recommended for the design of foundations on the natural, undisturbed glacial deposits below the site. A recommended design groundwater level was also given which is on average 10-12’ below the bottom of the existing foundation.

Res Tower II utilizes a mat foundation system with two main thicknesses, 4’-3”and 3’-9”. Logically, the taller tower is supported using the deeper mat foundation to resist the higher loads transferred by the braced frames. The foundation step occurs between grid lines 9 and 10. The typical reinforcement in the east-west direction is #10’s spaced at 10” on center top and bottom while in the north-south direction, the reinforcement is #9’s spaced at 10” on center top and bottom. Additional reinforcing cages are placed under the braced frame columns with the anchor bolts of these columns being tied to the bottom of the cage to increase the resistance to uplift. A detail of this connection is shown below in figure 1.

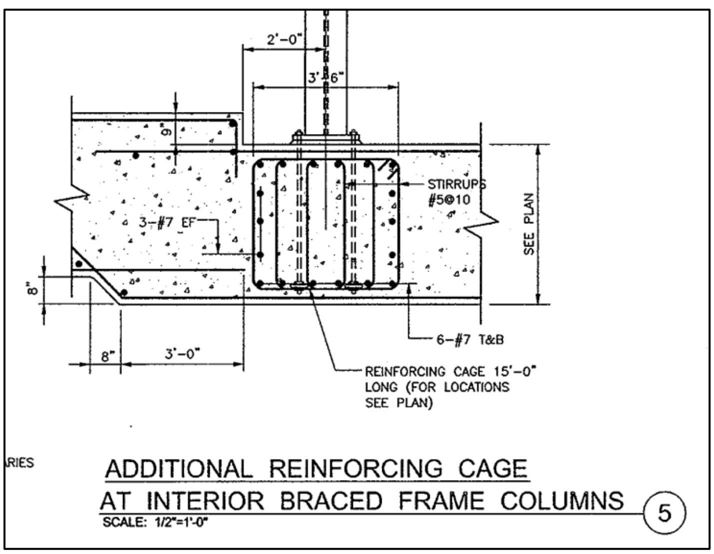

Figure 1: Additional foundation reinforcing

A 9” deep trench runs along the center of each towers foundation, parallel to the length of the building. This trench is filled in with 4000 psi concrete and reinforced with welded wire fabric after the erection of the interior columns in this area. In figure 2 below, the trench is shaded and outlined in red with the lateral force resisting system columns marked in blue.

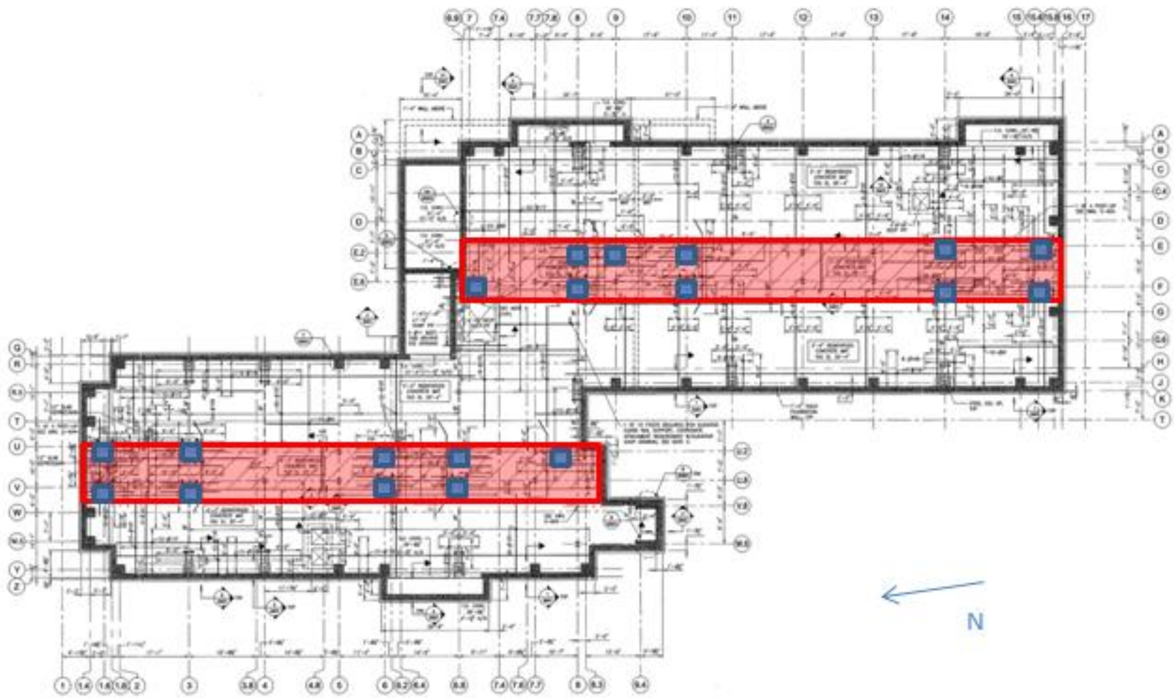

Figure 2: Foundation Trench

Floor Construction

The typical floor construction for Res Tower II is 3" 18 gage galvanized steel deck with 3 ¼" lightweight concrete topping, a total thickness of 6 ¼", and 6 x 6 welded wire fabric reinforcement. This is used everywhere except the loading dock and trash compactor area on the first floor. The floor system for these areas is comprised of 3" 16 gage steel deck with 6" normal weight concrete topping, a total thickness of 9", and epoxy coated reinforcement of #7's spaced at 12" on center in the bottom of the flutes and #5's spaced at 12" on center in the top running each way. All deck acts compositely with beams.

Decking typically spans about 8'-9" supported by beams ranging in size from W14's to W18's. These composite beams span roughly 23 feet to girders or columns. The girders have the same range in sizes as the beam. These spans create a typical bay size of 17-18' x 24-23'. The actual bay sizes vary moderately from typical dimensions. Figure 3 shows a typical floor plan for floors 3-18.

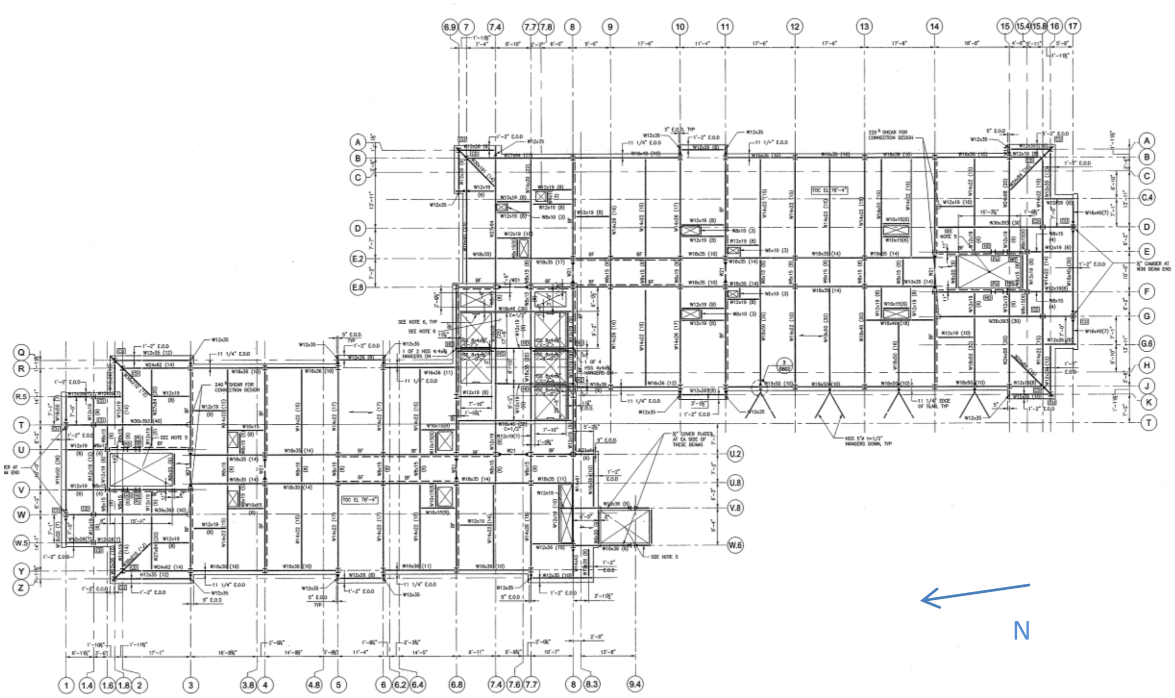

Figure 3: Typical Floor Plan

Lateral System

Steel braced frames are used to resist the lateral loads placed on the structure. At the termination of these columns, extra reinforcement is added to better tie the columns to the foundation and resist overturning forces. All columns in these braced frames are W14's ranging in size from W14x61 near the top of the structure to W14x398 for the bottom columns. The diagonal bracing members are W12's ranging in size from W12x152 to W12x45. This braced frame construction is categorized as a concentrically braced frame in ASCE7-10 for which an R value of 3.25 is prescribed but due to the moment connections an R value of 5 was used by the engineer for design. To allow for corridors to pass through the center of these braced frames, moment connections were made. Figure 4 shows an elevation of a braced frame with the moment connections clearly shown. The braced framed locations are highlighted in figure 5.

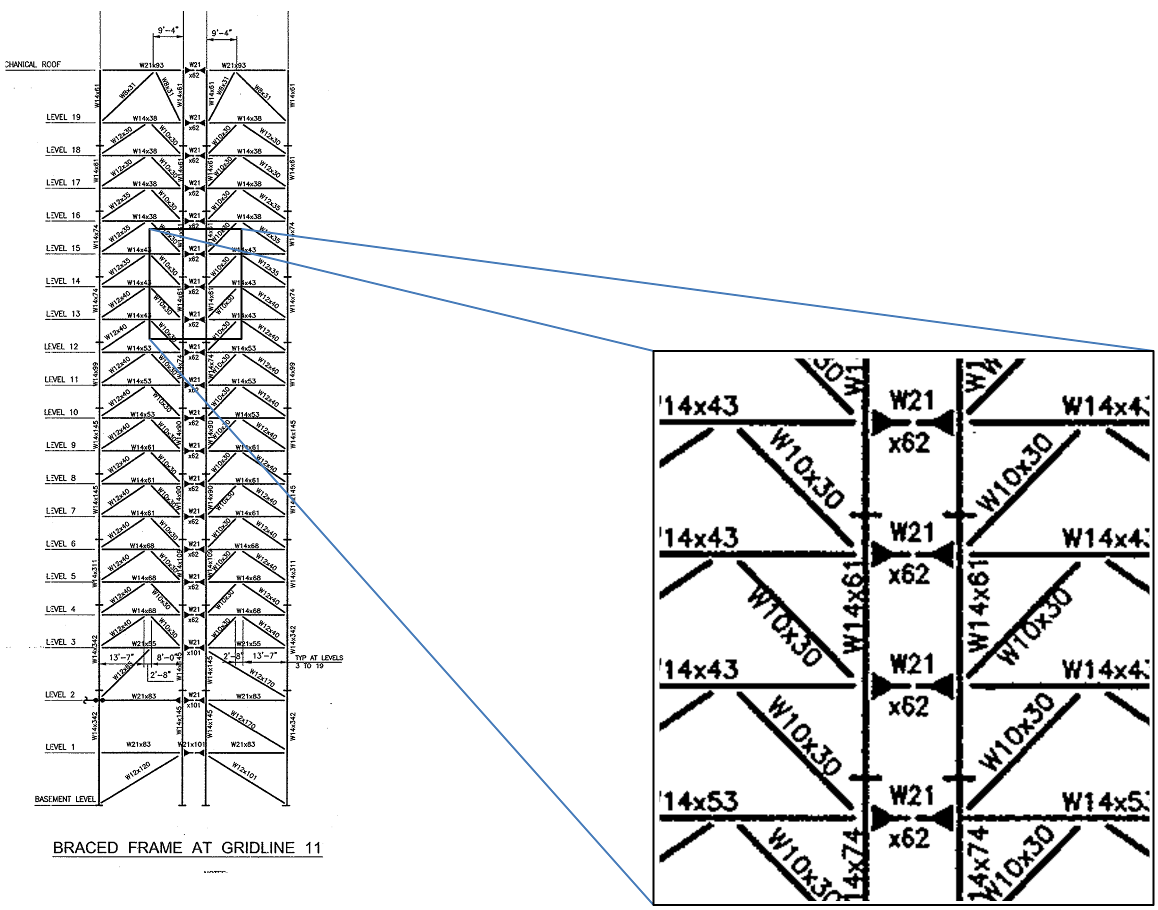

Figure 4: Braced frame elevation with moment connection

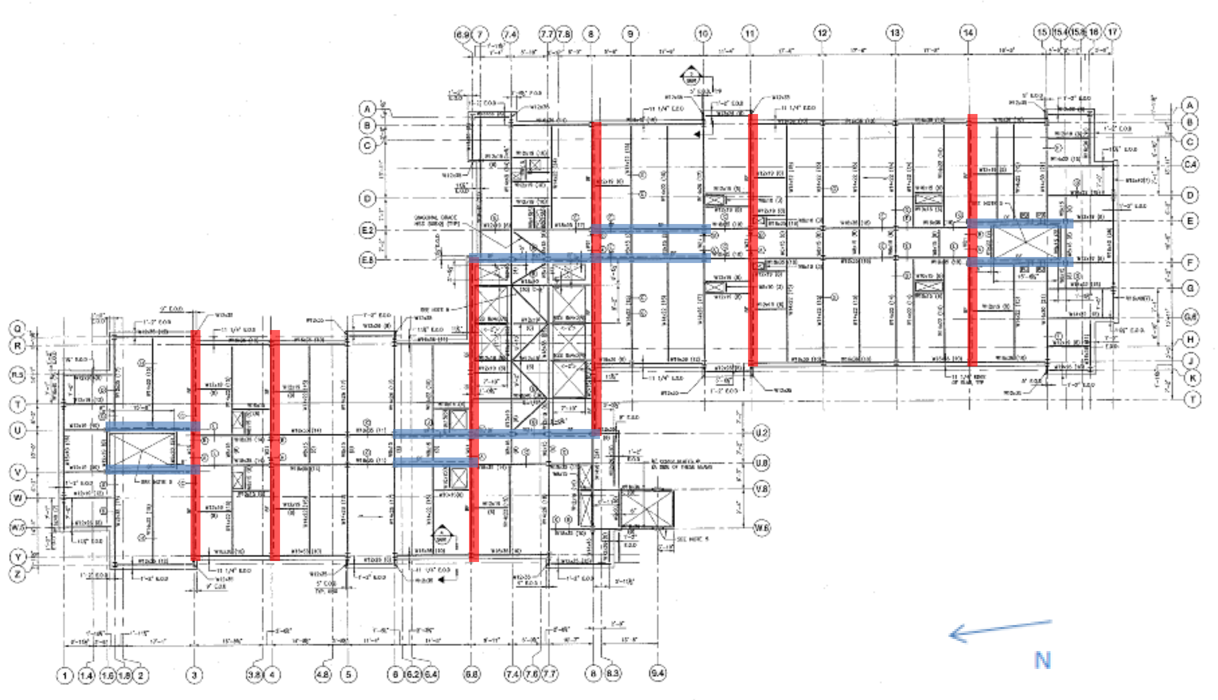

Figure 5: Typical plan with braced frame locations highlighted

Due to the slender shape of the building in the short direction, the braced frames in this direction (highlighted in red) have wider bases than the braced frames in the longer direction (shown in blue). The wider base provides a more effective geometry for transferring lateral loads to the foundation in the form of vertical loads.

Figure 6: Connection construction photo

Some of the braced frames in perpendicular directions utilize the same columns making for very complicated connection details and erection processes. To successfully portray these connections, 3 dimensional models had to be built, presented and provided for the contractors. Because of this, the design phase of the schedule had to be extended and more risk was taken by the structural engineer who designed the connections. A construction photo of these connections is shown in figure 6.

Figure 7 shows the trench mentioned in the foundation section and one of the further issues encountered due to the connections of the braced frames. Where the columns terminate, some of the foundation had to be cut away to allow for the columns to be placed due to the large connections for the diagonal bracing members. A last minute adjustment of this type is both unnecessary and disruptive. This issue also pushed the steel erection schedule and caused delays in the overall construction schedule.

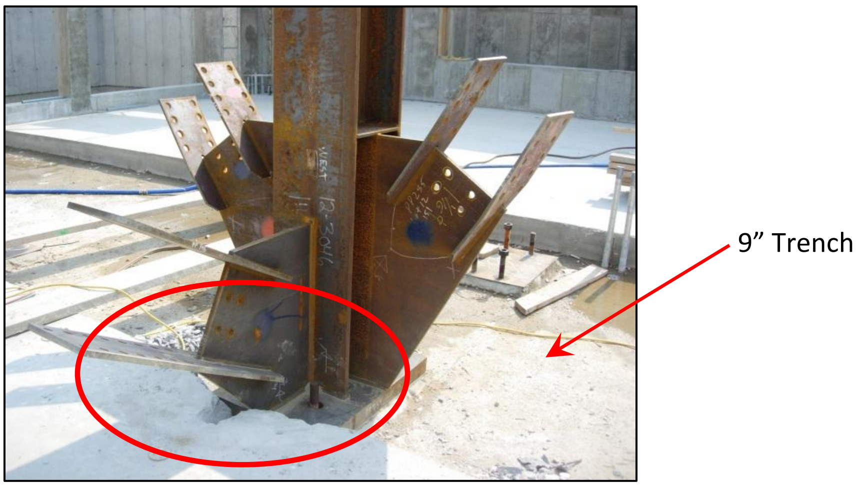

Figure 7: Foundation braced frame connection issues

Design Codes & Standards

Original Design	Thesis Design
Massachusetts Building Code 6th Edition	2009 International Building Code
1993 BOCA National Building Code	American Society of Civil Engineers (ASCE7-10) And ASCE7-05
American Institute of Steel Construction (2005 Manual)	2005 AISC Steel Manual

Table 1: Design codes vs. Thesis codes

Structural Materials

The materials listed in the chart below are specified in the structural drawings via the General Notes page of the structural drawings (S000) or general notes on the individual framing plans.

Material Properties		
Material		Strength
Steel	**Grade**	**fy = ksi**
Structural Shapes	A992	50
Plates	A36	36
Angles	A36	36
Structural Tubes	A500, B	46
Structural Pipes	A53, B or A501	30
Column Base Plates	A572, 50	50
Concrete	**Weight (lb/ft^3)**	**f'_c = psi**
Mat Foundation	145	4000
Slabs (Dock & Trash)	145	4000
Walls	145	4000
Typ. Slabs	115	3000
Reinforcing Steel		**fy = 60 ksi**
Welding Electrodes	E70 XX	**70 ksi**

Table 2: Material properties

Building Loads

In the tables that follow, the dead and live loads that were used by the designers and that were used for this thesis are listed. The dead loads were looked up in literature, assumed or calculated depending on the type of material they consist of; while the live loads were designated as specified by the codes listed in the tables.

Dead Load

Dead Loads	
Material	Load (psf)
Slab	
-Roof Deck	56
-Floor Deck	46
Façade	18
Superimposed	30

Table 3: Dead loads

Live Load

Live Loads		
	Design Load (psf)	Thesis Load (psf)
Occupancy Type	Mass. State Building Code	IBC 2009 & ASCE7-10
Public Area	100	100
Corridor	80	80
Dwelling Unit	40	40
Loading Dock	250	250
Mechanical Penthouse	150	125
Roof	30	20

Table 4: Live loads

Wind Load

In Technical Report 1, "*Existing Conditions and Design Concepts,*" ASCE7-10 was used to determine the wind pressures on Res Tower II. After further discussion and investigation, the decision was made to follow the procedure specified in ASCE7-10 but to replace the basic wind speed value (V=140) with the value (V=110) specified in ASCE7-05. Reasoning behind this substitution was based on the large difference in basic wind speed values from ASCE7-10 and the 1993 BOCA National Building Code which was used in the original design. The same assumptions were made in the process of calculating wind forces as were made in Technical Report 1 but due to a decreased wind speed, the forces were much lower and closer to the original design forces.

Due to a slender floor plan, the structure had to be assumed flexible as opposed to rigid. Because of this assumption, the method of determining a structures approximate natural frequency (ASCE 26.9.2.1) could not be used. The natural frequency was calculated using equations given in the seismic design section (ASCE 12.8.2.1) and by modeling the structure using Etabs. Inverting equation 12.8-7 (ASCE), $T_a = C_t h_n^x$, provided a natural frequency equal to 0.701 Hz. The computer model calculated a natural period of vibration equal to 2.4020 seconds and when inverted, this value provides a natural frequency of 0.416 Hz. ASCE7 specifies that any natural frequency less than 1.0 Hz implies that the structure is flexible; because 0.701 Hz and 0.416 Hz are less than 1.0 Hz, the assumption of a flexible building was correct.

Assumptions were also made to the geometry of the building. A simplified building shape was used to compensate for setbacks and the vertical geometry was broken into two pieces to take advantage of similar floor plans. The lower section of the building was adjusted from the original shape to the red outline shown in figure 8 and the upper section of the building was adjusted to the green outline, also shown in figure 8.

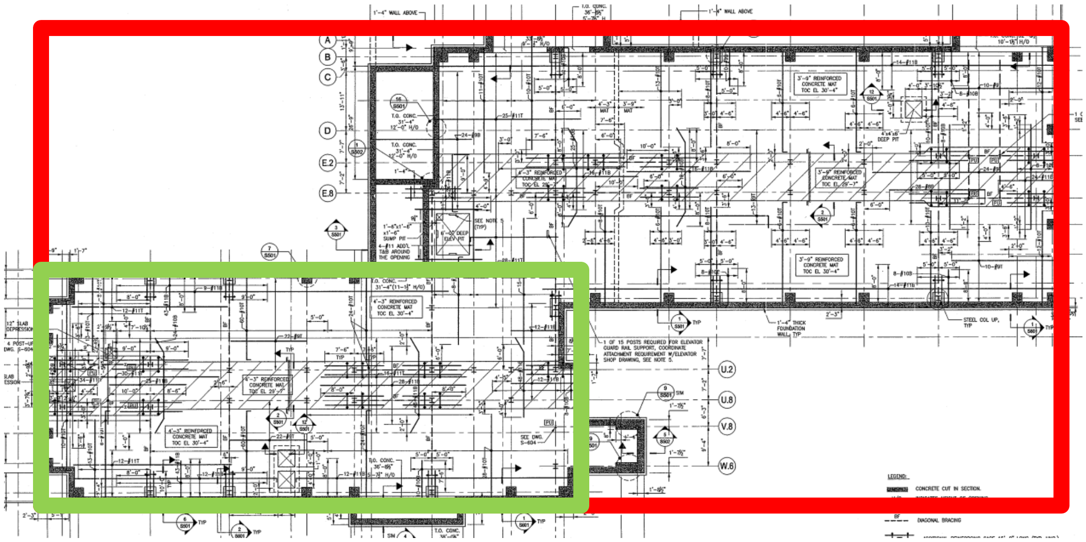

Figure 8: Simplified building plan for wind calculations

Figure 9 shows a rough Google SketchUp model of how the vertical geometries of the building were broken up. Using these two separate pieces allowed for more specific Gust Factors and therefore better approximations of wind force distribution (26.9.5 ASCE).

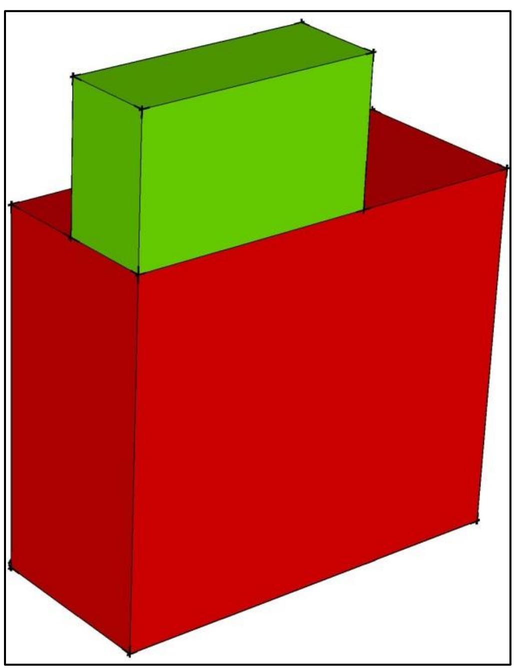

Figure 9: Simplified building geometry

A sample hand calculation of the wind pressures is provided in appendix A. After a firm understanding of the calculations necessary, excel spreadsheets were used to find the pressures in other directions and on the other piece of the building.

Forces caused by Res Tower II's internal pressure were neglected because they have no influence on the main lateral force resisting structure. Internal wind pressure is either all pressure or all suction and therefore create equal and opposite forces that cancel one another in the overall contribution to the lateral wind force. A schematic below provides a visual aid of the internal pressure and how the forces act on the building.

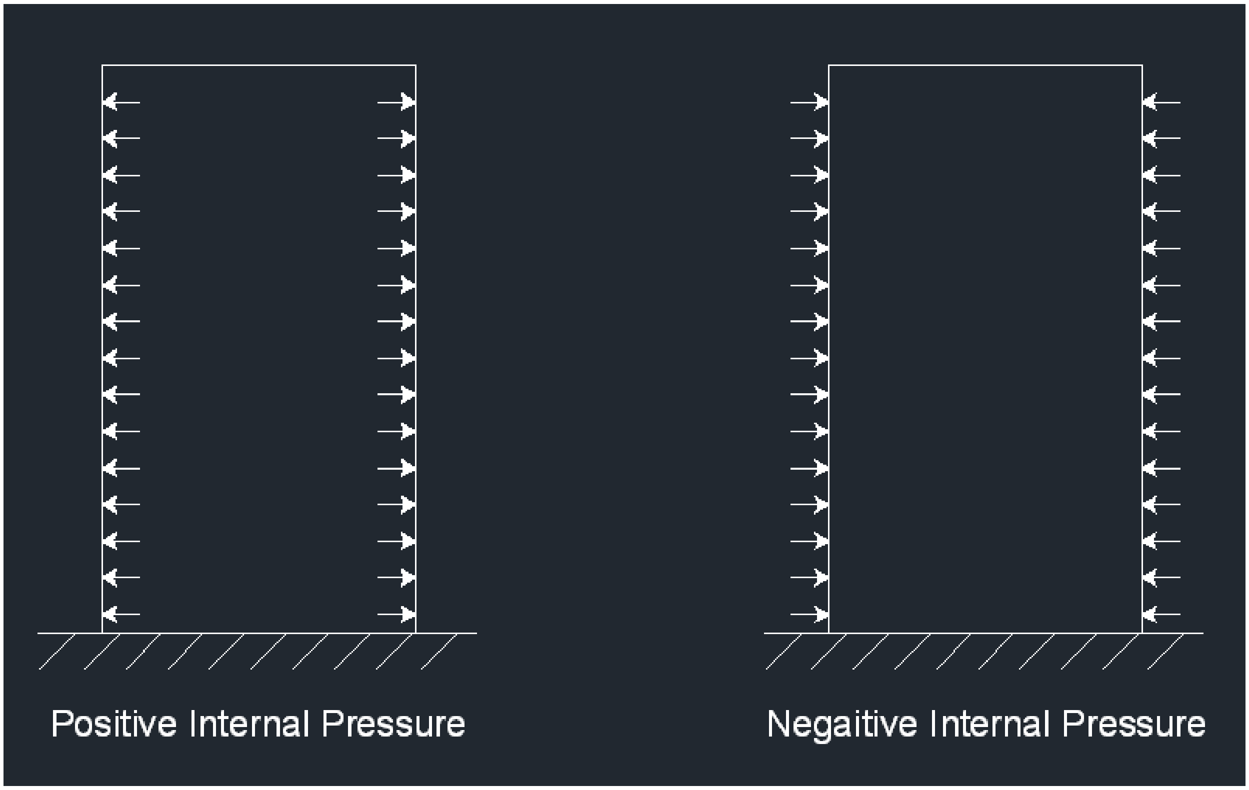

Figure 10: Schematic depiction of internal pressures

The final base shear and overturning moment were calculated using an excel spreadsheet which is shown in the following table. In the image after the table, a schematic depiction shows how the wind pressure is distributed along the height of the building. For wind pressures on the windward and leeward side in both directions, see appendix A.1.

North South (X-Direction, Etabs)			
Floor	Force (k)	Height (ft)	Moment (ft k)
1	24.22	0	0.00
2	48.61	16	777.75
3	40.96	32	1310.80
4	33.74	42	1417.11
5	34.81	52	1810.29
6	35.70	62	2213.64
7	36.51	72	2628.96
8	37.30	82	3058.82
9	37.97	92	3493.29
10	38.47	102	3923.58
11	38.93	112	4360.38
12	39.44	122	4811.43
13	39.94	132	5272.60
14	40.43	142	5741.01
15	40.81	152	6203.76
16	41.13	162	6662.72
17	41.49	172	7136.66
18	41.89	182	7623.41
19	42.23	192	8108.34
20	42.53	202	8591.56
21	52.15	212	11056.31
22	49.96	222	11092.05
23	38.41	232	8910.14
24	38.61	242	9342.61
25	46.58	252	11738.91
26	58.54	266	15572.22
27	58.90	282	16610.42
ROOF	27.76	296	8216.40
BASE	1148.05		177685.17

East West (Y-Direction, Etabs)			
Floor	Force (k)	Height (ft)	Moment (ft k)
1	53.26	0	0.00
2	106.87	16	1709.89
3	90.06	32	2881.80
4	74.18	42	3115.52
5	76.54	52	3979.94
6	78.50	62	4866.69
7	80.27	72	5779.79
8	82.01	82	6724.84
9	83.48	92	7680.03
10	84.57	102	8626.01
11	85.59	112	9586.32
12	86.70	122	10577.95
13	87.82	132	11591.83
14	88.88	142	12621.65
15	89.73	152	13639.00
16	90.42	162	14648.03
17	91.22	172	15689.99
18	92.09	182	16760.11
19	92.84	192	17826.23
20	93.51	202	18888.60
21	103.57	212	21956.09
22	87.73	222	19476.52
23	62.33	232	14461.70
24	62.67	242	15165.60
25	75.63	252	19058.09
26	95.06	266	25284.84
27	95.65	282	26974.69
ROOF	45.09	296	13346.34
BASE	2336.26		342918.09

Table 5: Wind forces

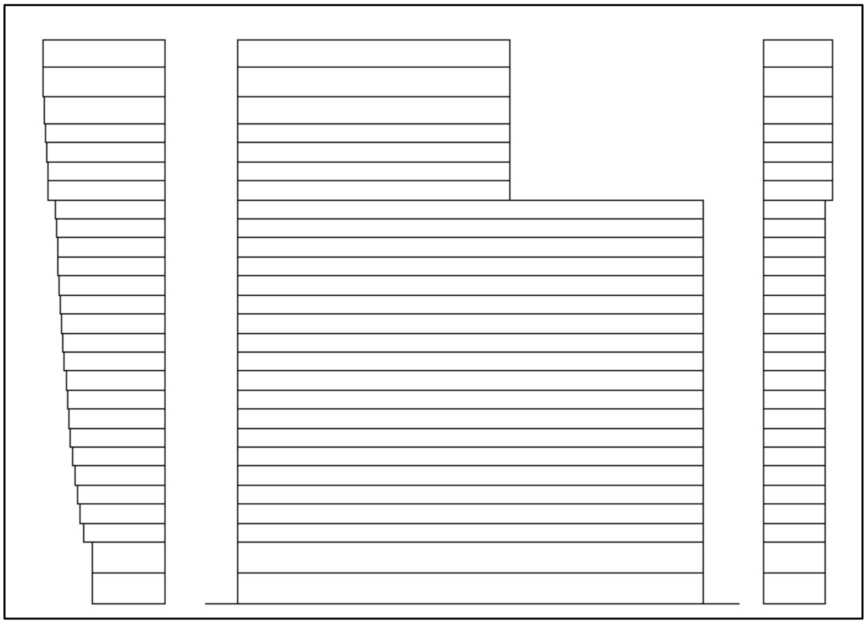

Figure 11: Wind pressure vertical distribution, North-South direction

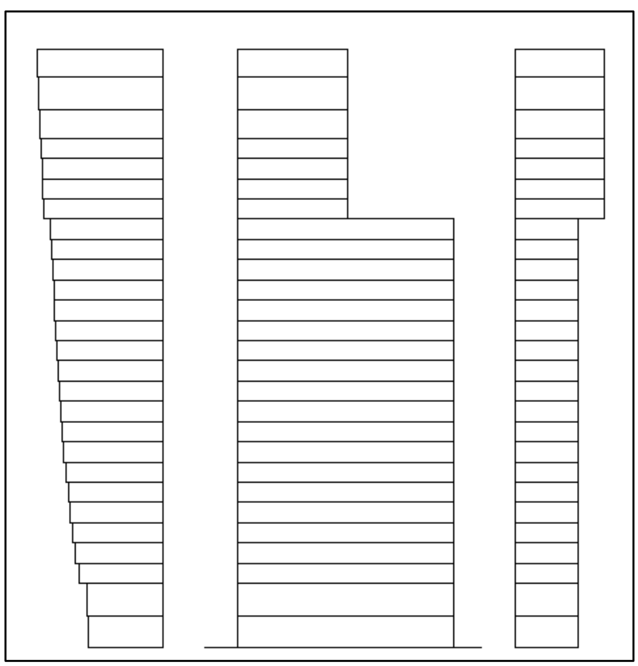

Figure 12: Wind pressure vertical distribution, East-West direction

Seismic Load

Seismic design for Res Tower II was done following the Equivalent Lateral Force Procedure (ASCE7-10, 12.8) and the criteria specified in ASCE7-10 chapters 11 and 12. Due to the geotechnical report being completed relevant to the Massachusetts Building Code, comparisons had to be made between that and ASCE7-10. In the geotechnical report, H&A give the soil a category rating of S3 from the Massachusetts Building Code, which compared relatively close to both site class C and D from ASCE7-10. Taking the more conservative case meant categorizing the soil as class D. The table below gives the values used for determining the base shear in the x and y direction. The base shear is equal in both directions because the period of vibrations found the computer model were each greater than C_uT_a (ASCE 7-10, 12.8.2).

Seismic Design Variables				
S_{DS} =	0.40615		$T_{model\text{-}x}$ =	2.402 s
S_{D1} =	0.2263		$T_{model\text{-}y}$ =	1.960 s
R =	5		C_u =	1.474
I =	1.25		T_a =	0.701
T =	1.033 s		C_s =	0.0548

To proceed with the specified calculations, the total building weight had to be calculated. This was done by counting beams and columns, then multiplying their respective lengths by the unit weight of the particular shape. Using the Vulcraft Metal Decking catalog, weights were found for the specified floor systems. A superimposed dead load of 30 psf was used to account for MEP systems, ceiling systems and fixtures, partitions and the different types of floor finishes including tile, wood and carpet. The façade system was specified to weigh 18 psf with 2 ft thick exterior walls which lead to 36 lbs per linear foot of exterior wall. These weights are shown below in tabulated form.

Material	Weight (k)
Non-lateral Columns	385
Lateral Columns	1250
Concrete: Slab and Deck	18000
Beams	1575
Façade	621
Superimposed	11000
Total Self Weight	32831

Table 6: Tabulation of building self-weight

For the repetitive calculations, an excel spreadsheet (from AE 597A) was used to determine the load on each floor, the base shear and the overturning moment. This table is shown below.

Level	Height (ft)	Weight (k)	$w*h^k$	C_{VX}	Fi (k)	V_i (k)	M (ft-k)
roof	296	730.1	3020771	0.063	113	113	33521.23
27	282	730.1	2813986	0.059	105	219	29749.62
26	266	715.65	2532299	0.053	95	314	25252.65
25	252	715.65	2339647	0.049	88	401	22103.51
24	242	715.65	2205030	0.046	83	484	20005.08
23	232	715.65	2072967	0.043	78	562	18029.79
22	222	715.65	1943517	0.041	73	635	16175.27
21	212	715.65	1816742	0.038	68	703	14439.08
20	202	1424.02	3368200	0.070	126	829	25507.01
19	192	1424.02	3126997	0.065	117	946	22508.11
18	182	1424.02	2891549	0.060	108	1055	19729.33
17	172	1424.02	2662024	0.056	100	1154	17165.28
16	162	1424.02	2438604	0.051	91	1246	14810.40
15	152	1424.02	2221489	0.046	83	1329	12658.96
14	142	1424.02	2010898	0.042	75	1405	10705.05
13	132	1424.02	1807074	0.038	68	1472	8942.53
12	122	1424.02	1610289	0.034	60	1533	7365.02
11	112	1424.02	1420845	0.030	53	1586	5965.89
10	102	1424.02	1239089	0.026	46	1632	4738.20
9	92	1424.02	1065417	0.022	40	1672	3674.67
8	82	1424.02	900290	0.019	34	1706	2767.62
7	72	1424.02	744256	0.016	28	1734	2008.93
6	62	1424.02	597973	0.012	22	1756	1389.90
5	52	1424.02	462261	0.010	17	1774	901.16
4	42	1424.02	338175	0.007	13	1786	532.48
3	32	1424.02	227145	0.005	9	1795	272.50
2	16	1426	82480	0.002	3	1798	49.47
	Σ	32812.46		**Base:**	1798		340969

Table 7: Seismic story forces, base shear and overturning moment

Analysis

Computer Model

A computer model was made using Etabs, a Computer and Structures Inc. modeling and analysis program. This model was used to determine lateral drift of the structure and to confirm the controlling wind load case.

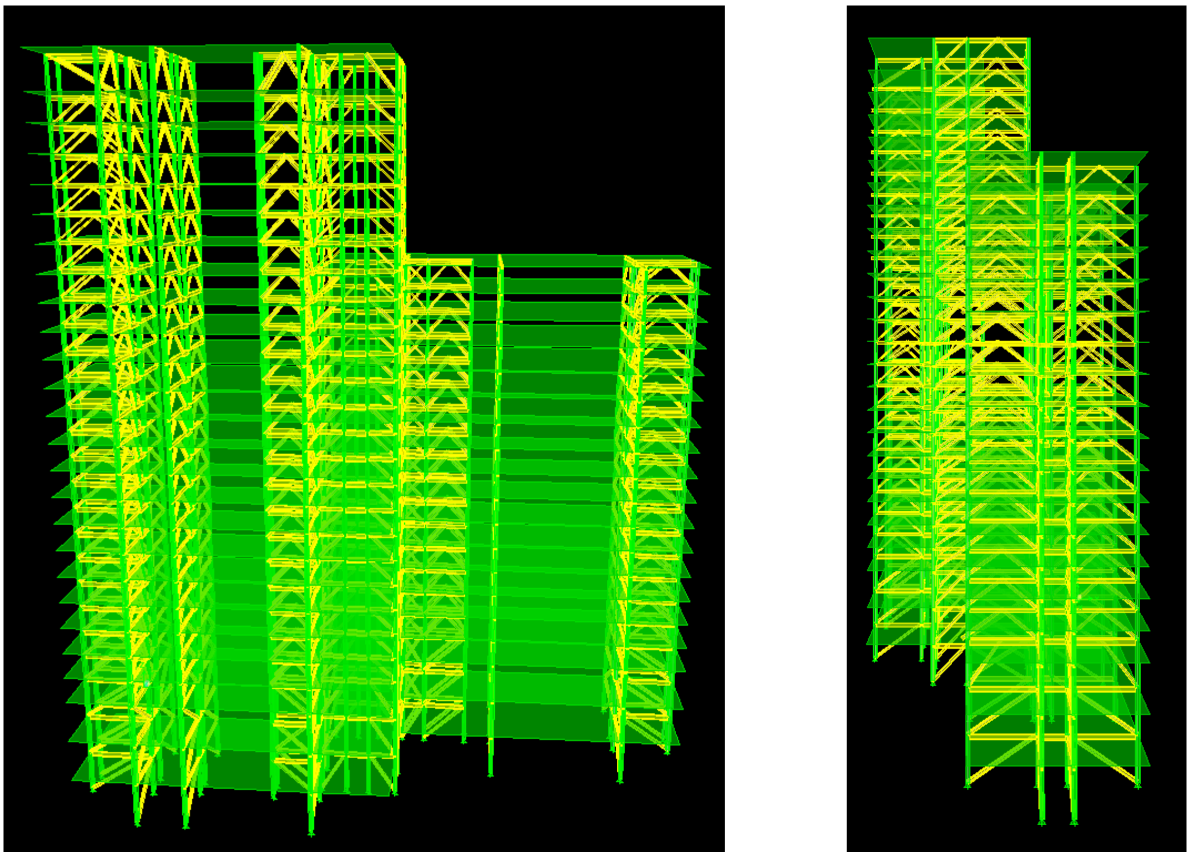

Figure 13: Views of Etabs model

To match what the structural engineer designed, the following considerations were made when developing the model:

- Diagonal and horizontal members of each braced frame were specified to have moment releases to ensure these members only took axial forces.
- Each floor level was modeled as a rigid diaphragm so that all points on that level deflect together.
- Pin base supports were assumed for each column due to the connection at the foundation level.

To simplify the Etabs model, the light moment frames supporting the mechanical penthouses were neglected. This assumption caused all the braced frames to stop at the same levels allowing for a direct comparison of relative stiffness.

Relative Stiffness

To further understand how the structural system of Res Tower II acts under lateral loads, a closer examination had to be made for individual pieces of the system. Relative stiffness (k) values were calculated for each of the braced frames individually. This was done using Etabs by placing a 1 kip load at the 26th floor and measuring the deflection at that level then repeating the process at the 19th floor. By logic and the equation K = P/δ, the frame with the smallest amount of deflection is the stiffest frame. To determine the relative stiffness of each frame the minimum deflection of frames at that level was divided by the deflection of an individual frame, this ratio equals the relative stiffness of that individual frame. Deflections and stiffness values for each frame are shown in table 9 and there corresponding locations are shown in plane in figure 13. A sample calculation is also presented below to help clarify the procedure.

$$ki = \frac{Minimum\ Deflection}{Individual\ Deflection}$$

$$k1 = \frac{0.0223}{0.0326} = 0.6841$$

		Deflection		Relative K		
Force at Story:		26	19	26	19	
FRAME	1	0.0326	0.0139	0.6841	0.6988	Y
	2	0.0332	0.0149	0.6725	0.6517	
	3	0.0223	0.0100	1.0000	0.9697	
	4	0.0277	0.0097	0.8059	1.0000	
	5	--	0.0185	--	0.5269	
	6	--	0.0264	--	0.3680	
	7	0.1225	0.0488	0.1823	0.1995	X
	8	0.1225	0.0488	0.1823	0.1995	
	9	0.0371	0.0166	0.6013	0.5858	
	10	0.3551	0.1291	0.0629	0.0754	
	11	0.0608	0.0191	0.3671	0.5089	
	12	--	0.0645	--	0.1508	
	13	--	0.0888	--	0.1096	
	14	--	0.1064	--	0.0914	

Table 8: Deflections and relative stiffness values

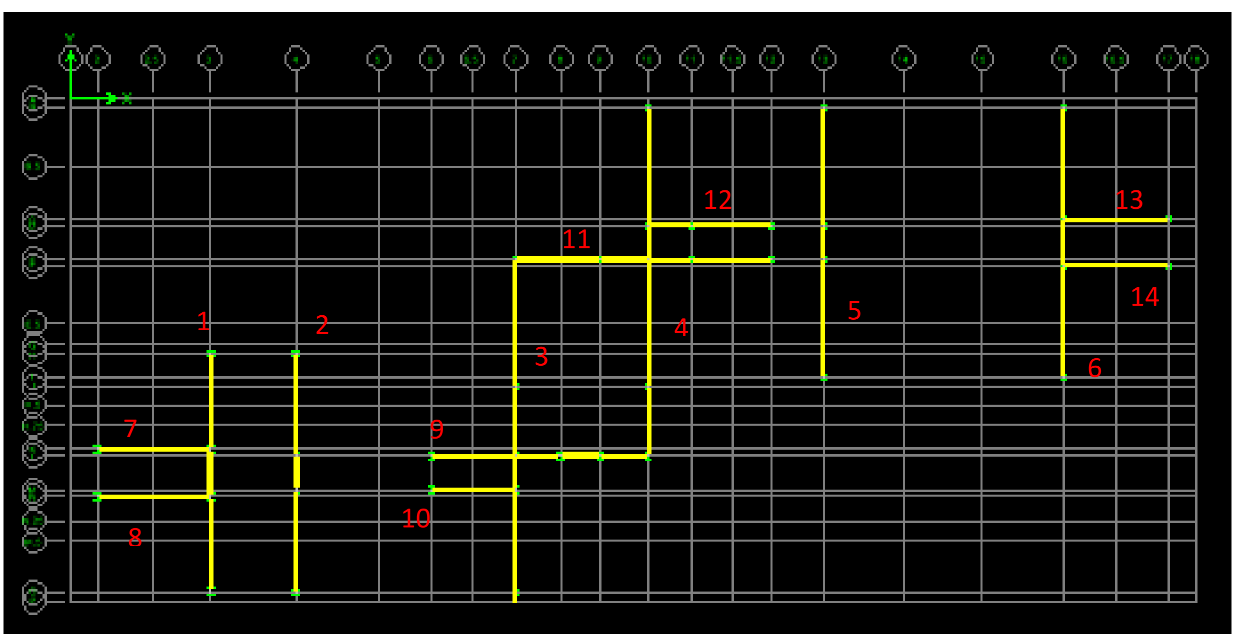

Figure 14: Layout and numbering of braced frames

Once stiffness values were found for each frame, the center of rigidity was calculated for the upper floors and the lower floors. Comparing the center of rigidity to the center of mass for each level led to the need to determine the controlling wind load case.

Load Cases

ASCE7-05 specifies 4 Design Wind Load Cases which are shown below in figure 14. Level 22 was used to determine which of the 4 load cases produced the worst case scenario. The full calculations can be found in appendix E. Force distribution of story shear to braced frames will be discussed in more detail later. Resulting forces for each load case are presented on the following pages.

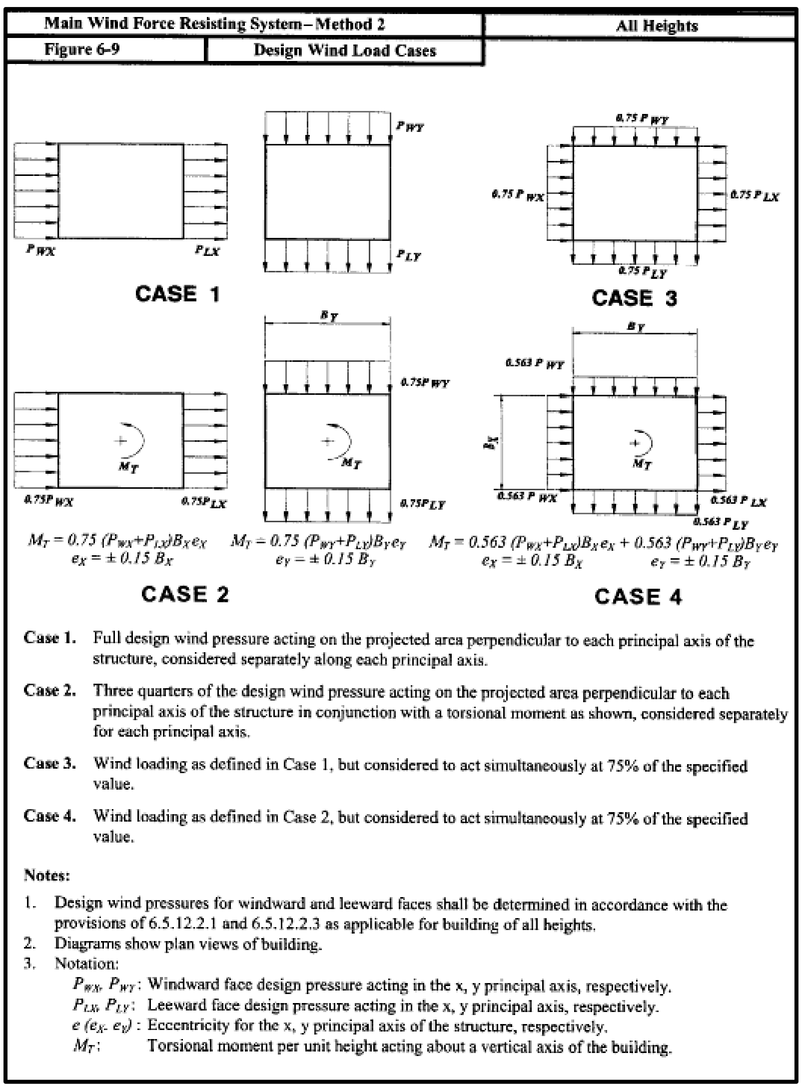

Figure 15: Design Wind Load Cases (ASCE7-05)

CASE 1

TOTAL FORCE: X Direction			TOTAL FORCE: Y Direction		
F_1	0.979464	k	F_1	18.25023	k
F_2	0.207537	k	F_2	13.70815	k
F_3	0.785959	k	F_3	14.25238	k
F_4	1.170624	k	F_4	8.47569	k
F_7	5.514957	k	F_7	0.093821	k
F_8	5.556389	k	F_8	0.325954	k
F_9	18.20978	k	F_9	0.417921	k
F_{10}	1.914848	k	F_{10}	0.101104	k
F_{11}	10.76962	k	F_{11}	1.69454	k

CASE 2

TOTAL FORCE: X Direction			TOTAL FORCE: Y Direction		
F_1	2.613421	k	F_1	18.26351	k
F_2	0.553755	k	F_2	11.25068	k
F_3	2.097106	k	F_3	7.017463	k
F_4	3.123476	k	F_4	0.887873	k
F_7	4.168339	k	F_7	0.148596	k
F_8	4.278887	k	F_8	0.516254	k
F_9	13.80042	k	F_9	0.661914	k
F_{10}	1.47075	k	F_{10}	0.160131	k
F_{11}	7.497066	k	F_{11}	2.683853	k

CASE 3

TOTAL FORCE:		
F_1	14.42227	k
F_2	10.43677	k
F_3	11.27876	k
F_4	7.234736	k
F_7	4.206584	k
F_8	4.411757	k
F_9	13.97078	k
F_{10}	1.511964	k
F_{11}	9.348123	k

CASE 4

TOTAL FORCE:		
F_1	15.67162	k
F_2	8.861197	k
F_3	6.842004	k
F_4	3.011186	k
F_7	3.240579	k
F_8	3.599552	k
F_9	10.85639	k
F_{10}	1.224248	k
F_{11}	7.642477	k

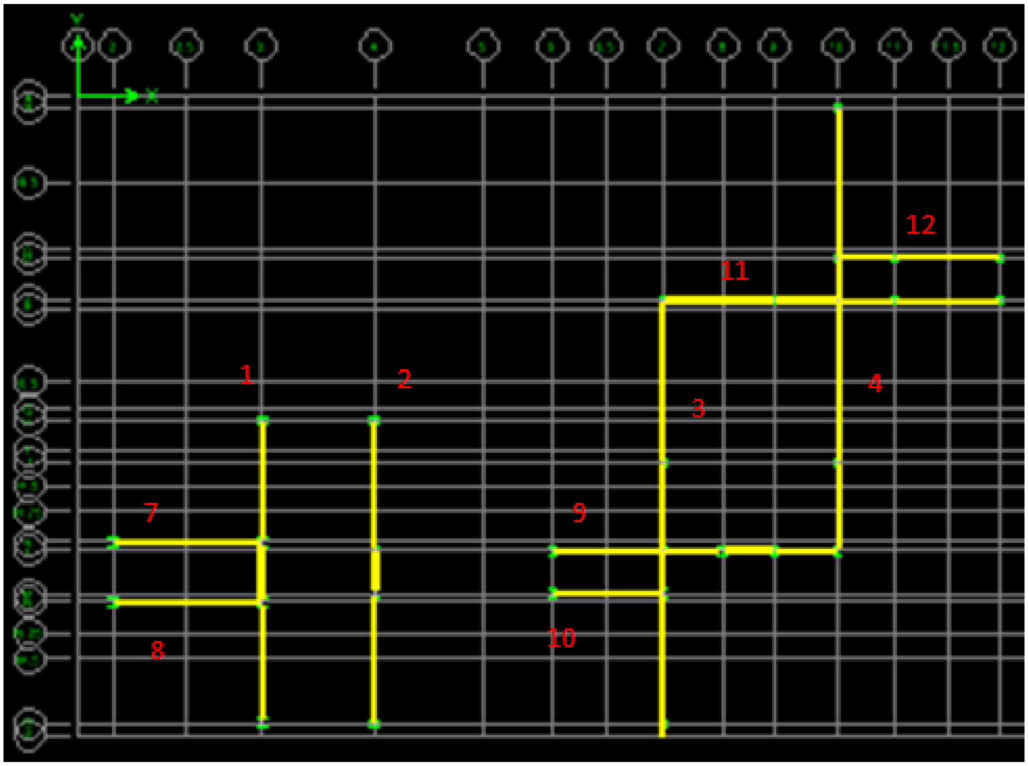

Figure 16: Braced frame numbering and location

Case 3 was chosen as the controlling scenario for multiple reasons. The first and most obvious reason is that case 3 produces the largest forces in the frames on average. Case 1 and Case 2 only consider wind in one direction at a time whereas Case 3 and Case 4 consider wind acting in both directions simultaneously. Wind in both directions is a reasonable assumption due to the orientation of Res Tower II, its surrounding geography and the buildings around it. Drift values are also greatest when wind is considered in both directions, even when only 75% of the forces are applied. Drift values and diagrams are shown below. The diagrams of each wind case can be compared to figure 17 which shows the undeformed braced frame layout.

Maximum Displacement (in)		
	X	Y
Wind - X	4.059	0.626
Wind - Y	0.728	3.646
Combination Wind	2.498	2.265

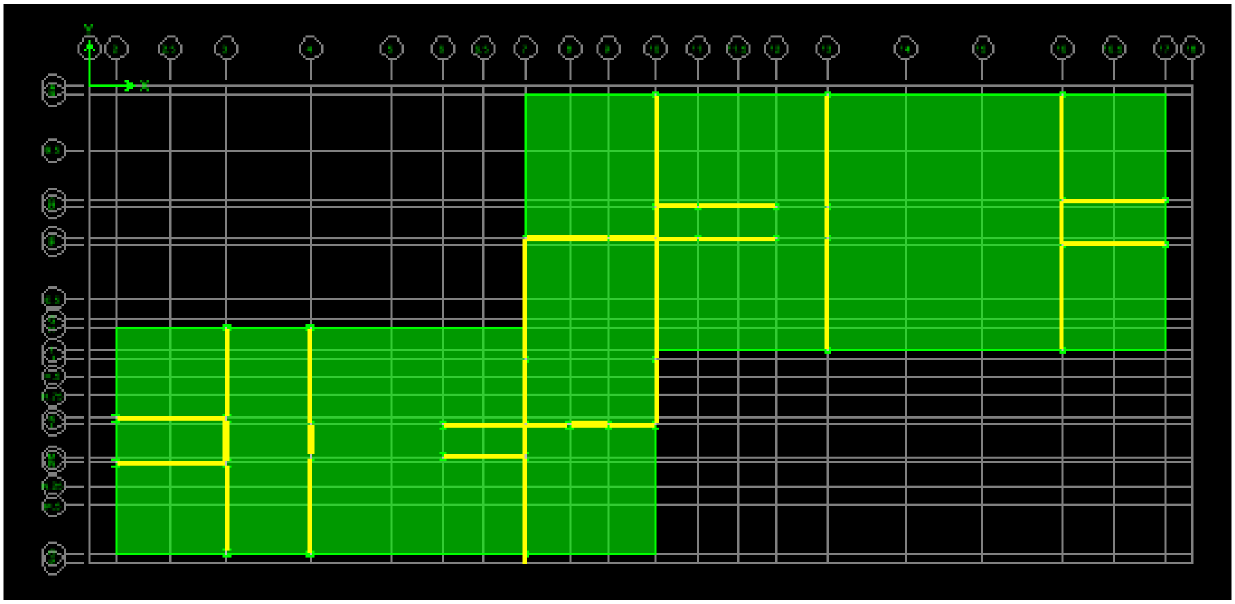

Figure 17: Undeformed braced frame layout

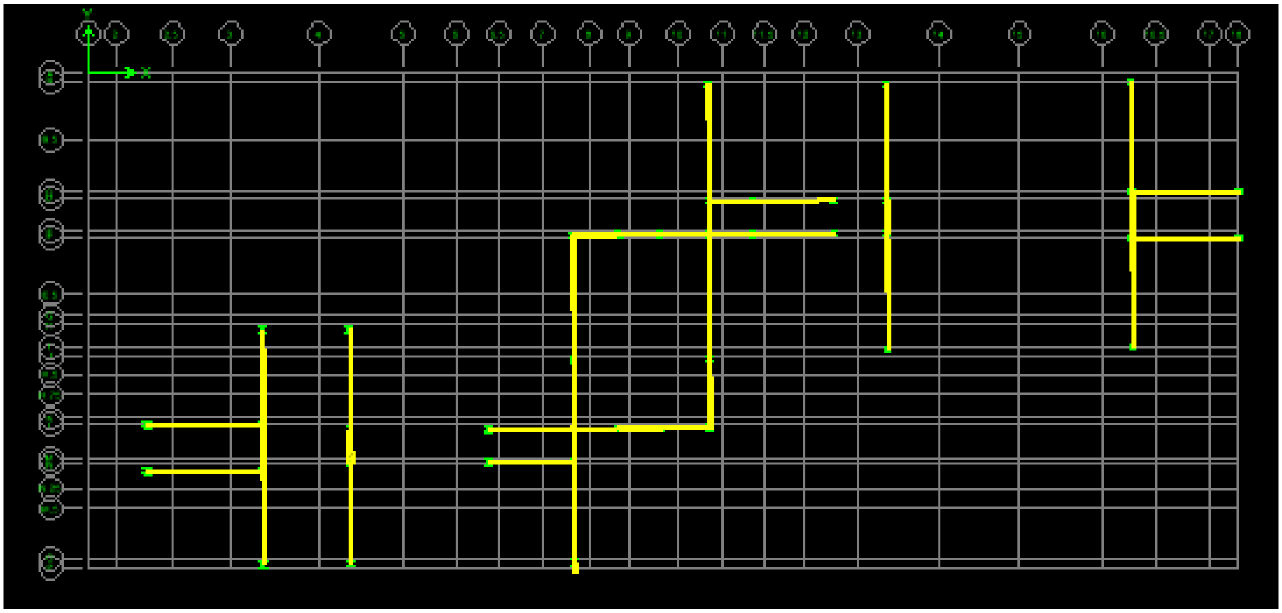

Figure 18: Wind-X deformation

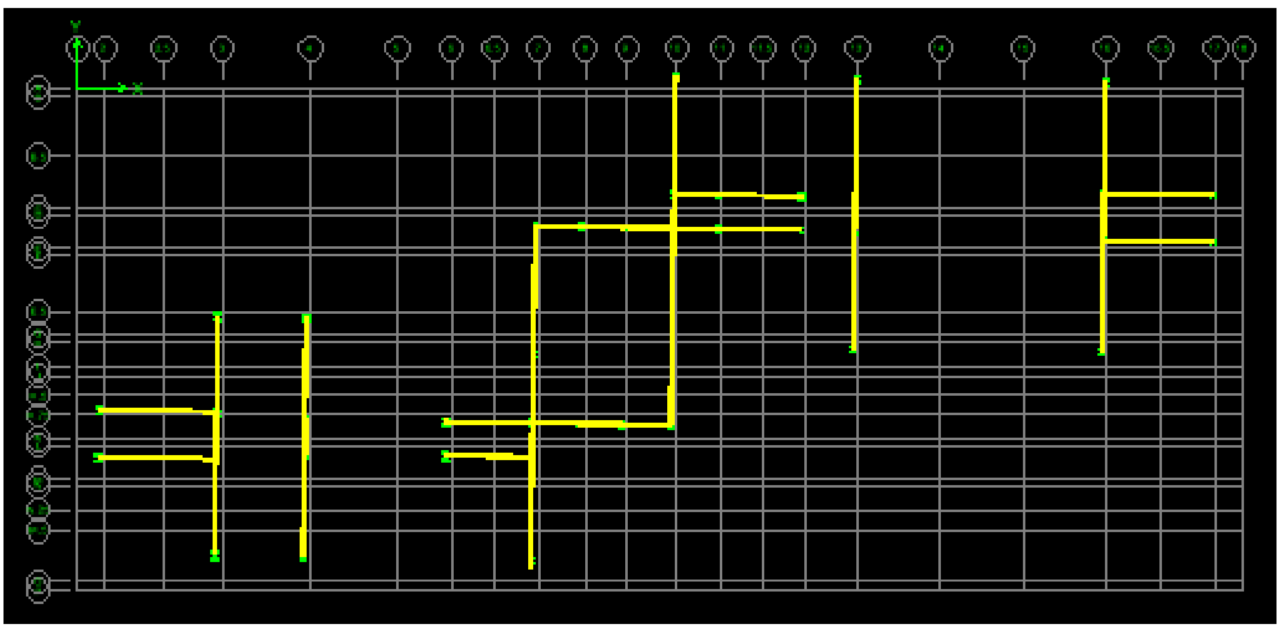

Figure 19: Wind-Y deformation

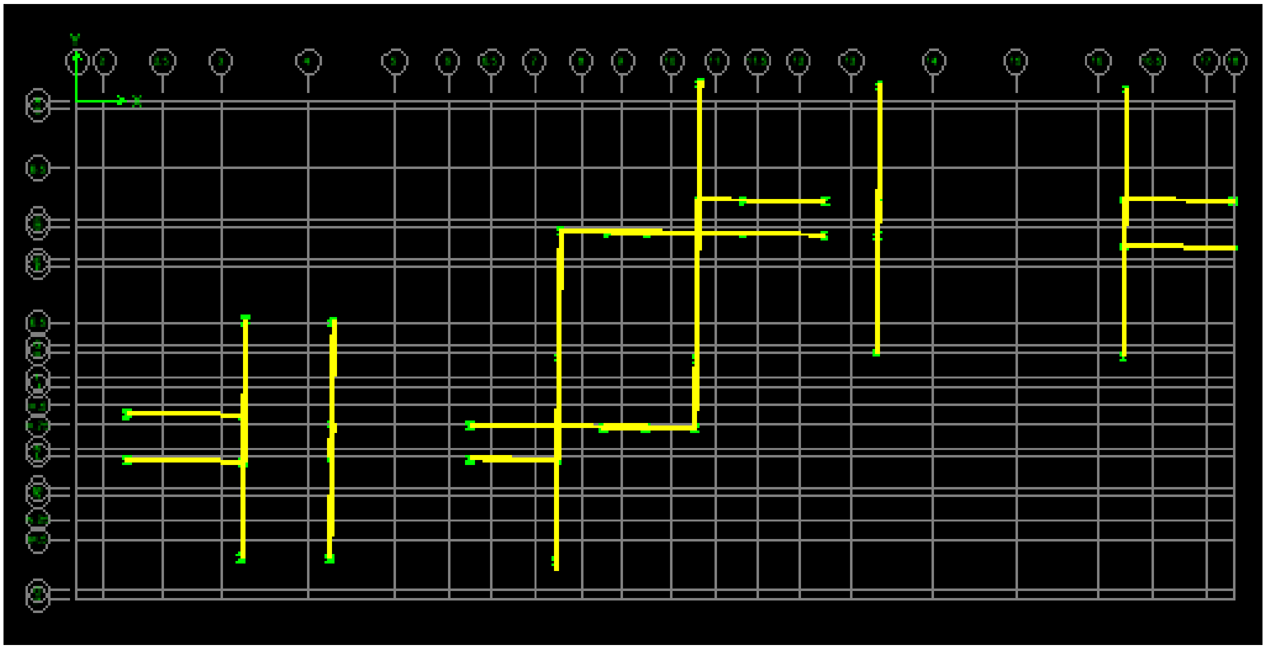

Figure 20: Combined wind deformation

Load Combinations

ASCE7 gives 7 basic load combinations in section 2.3.2:

1. 1.4D
2. 1.2D+1.6L+0.5S
3. 1.2D+1.6S+(L or 0.8W)
4. 1.2D+1.6W+L+0.5S
5. 1.2D+1.0E+L+0.2S
6. 0.9D+1.6W
7. 0.9D+1.0E

Because only lateral forces and floor dead loads were considered for this technical report, the load combinations results in a comparison of 1.0E and 1.6W. When these load factors are applied to the base shears caused by seismic and wind forces, the wind controls in both directions. Only wind forces were modeled and used in load distribution calculations because these forces would control the design of lateral force resisting members. No load factors were applied during the analysis of the lateral structural system.

Load Distribution

Direct Shear

The driving principle of load distribution is load follows stiffness. A stiffer member will take more load than a less stiff member. Table 9 shows the total story shear in the north-south direction at the 22nd floor and the distribution of this story shear to each frame according to its relative stiffness. The stiffest frame, number 9, takes the greatest amount of load, 10.2 kips, and the least stiff frame, number 10, takes the least amount of load, 1.1 kips.

Direct:	F_T =	23.70257	k
	Relative Stiffness	FORCE	
F_7	0.182304	3.095493	k
F_8	0.182304	3.095493	k
F_9	0.601308	10.21011	k
F_{10}	0.062892	1.0679	k
F_{11}	0.367116	6.233573	k

Table 9: Relative stiffness and load distribution at floor 22

Torsional Shear

While direct shear only considers relative stiffness to distribute the loads, torsional shear considers relative stiffness and the distance of each frame to that floor's center of rigidity. The force in each braced frame was calculated using the equation F = k*d*Θ where Θ is the angle of rotation of that level. Table 10 gives the final forces for each frame relevant to relative stiffness and each frame's distance to the center of rigidity. The equations listed below the table show how these values were calculated and define the variables used in each equation. Full calculations of both direct and torsional shear distribution can be found in appendix E.

Torsional:	F_T =	23.70257 k	
	k_i	d_i = distance to Cp (ft)	F_i
F_7	0.182304	4.0417	0.03354
F_8	0.182304	14.0417	0.116524
F_9	0.601308	5.4583	0.149401
F_{10}	0.062892	12.625	0.036143
F_{11}	0.367116	36.25	0.605776

$F_i = k_i * d_i * \Theta$ — k_i = relative stiffness

$\Theta = P*e/\Sigma(K_i * d_i^2)$ — d_i = defined in table

$P = F_T$ — Θ = angle of rotation caused by eccentric load

$e = C_r - C_p$ — e = eccentricity of load

C_r = Center of rigidity

C_p = Center of pressure/mass

Drift

Overall building drift and story drift are serviceability requirements therefore unfactored loads were used to determine the lateral displacements in the Etabs model. An industry standard of limiting overall building drift to H/400, where H is equal to the floor to floor of each story, was used in this analysis. Worst case displacements were used for comparison in the table below and all story drifts, in both the x and y directions, are within the H/400 limit.

Story Displacement/Drift Due to Unfactored Wind Loads						
	Height	Actual Displacement		H/400	Story Drift	
Story	(ft)	X (in)	Y (in)	(in)	X (in)	Y (in)
2	16	0.16	0.20	0.48	0.16	0.20
3	32	0.27	0.33	0.48	0.11	0.13
4	42	0.34	0.41	0.3	0.06	0.09
5	52	0.41	0.51	0.3	0.07	0.09
6	62	0.48	0.60	0.3	0.07	0.10
7	72	0.56	0.71	0.3	0.08	0.11
8	82	0.64	0.82	0.3	0.08	0.11
9	92	0.72	0.94	0.3	0.08	0.12
10	102	0.81	1.06	0.3	0.09	0.12
11	112	0.90	1.18	0.3	0.09	0.13
12	122	0.99	1.31	0.3	0.09	0.13
13	132	1.09	1.44	0.3	0.09	0.13
14	142	1.18	1.58	0.3	0.10	0.14
15	152	1.28	1.72	0.3	0.10	0.14
16	162	1.37	1.86	0.3	0.10	0.14
17	172	1.47	2.00	0.3	0.10	0.14
18	182	1.57	2.15	0.3	0.10	0.15
19	192	1.67	2.29	0.3	0.10	0.15
20	202	1.79	2.45	0.3	0.12	0.16
21	212	1.91	2.61	0.3	0.12	0.16
22	222	2.03	2.77	0.3	0.12	0.16
23	232	2.14	2.92	0.3	0.11	0.15
24	242	2.25	3.07	0.3	0.11	0.15
25	252	2.36	3.21	0.3	0.11	0.15
26	266	2.50	3.41	0.42	0.14	0.20

Table 10: Story Drift

Overturning Analysis and Foundation Considerations

Overturning moments on a building are caused by uplift forces be present at the base of lateral load resisting columns. Each lateral load resisting column has an extra cage of reinforcement in the mat foundation to help resist uplift forces (see Foundation section). These cages are 3’-6” wide by 15’ long. Using these two dimensions and the thickness of the foundation, 3’9”, the weight of the foundation in each columns tributary area can be found. It was calculated that the weight of this area is equal to 29.5 kips and with the additional weight of the columns tributary area this value will be greater than any uplift forces found in the braced frame columns. Tabulated values of uplift forces with their corresponding load cases can be found on the next page with the three greatest uplift forces highlighted in yellow. The columns with the largest uplift forces are the columns located the farthest from the center of the building. Calculations for this section can be found in appendix C.

Point	Load	Uplift (k)
7	WINDY	1215.27
7	COMBINEDWIND	582.18
9	WINDY	279.19
17	WINDY	1045.89
17	COMBINEDWIND	689
18	WINDX	127.22
19	WINDX	3.52
20	WINDY	143.08
20	COMBINEDWIND	104.67
26	WINDY	1012.81
26	COMBINEDWIND	605.09
28	WINDX	25.23
28	WINDY	396.23
28	COMBINEDWIND	316.1
29	WINDX	36.68
29	WINDY	27.88
29	COMBINEDWIND	48.42
30	WINDX	480.32
51	WINDY	1162.87
51	COMBINEDWIND	411.43
52	WINDY	34.62
52	COMBINEDWIND	0.13
61	WINDX	257.44
61	WINDY	52.53
61	COMBINEDWIND	232.49
62	WINDX	165.23
65	WINDX	27.39
66	WINDX	3.24
66	WINDY	79.19
66	COMBINEDWIND	61.83

Point	Load	Uplift (k)
68	WINDY	669.84
68	COMBINEDWIND	483.51
86	WINDX	216.16
88	WINDX	201.04
88	WINDY	416.31
88	COMBINEDWIND	463.03
89	WINDX	277.7
89	WINDY	131.39
89	COMBINEDWIND	306.82
90	WINDX	226.15
90	COMBINEDWIND	55.49
113	WINDX	741.77
113	COMBINEDWIND	408.09
115	WINDX	706.47
115	COMBINEDWIND	453.21
117	WINDX	736.03
117	COMBINEDWIND	462.35
122	WINDX	882.32
122	WINDY	66.67
122	COMBINEDWIND	711.77
123	WINDX	-239.89
123	WINDY	151.33
124	WINDX	112.1
124	WINDY	203.12
124	COMBINEDWIND	236.42
128	WINDX	202.03
128	COMBINEDWIND	52.24
146	WINDY	112.58

Table 11: Column uplift forces

Member Checks

Spot checks were done for two diagonal bracing members and two columns at critical locations. Level 19 was considered a critical location because this location could potentially be a weak point in the distribution of forces to the lower levels due to some of the frames stopping at this level. Where the column and diagonal bracing is connected to the foundation was also considered critical due to the complicated connections. Spot checks for the columns and diagonal members all proved that the members have sufficient strength. Because these were strength checks, the loads were multiplied by the appropriate coefficients. Calculations of the spot checks can be found in appendix D.

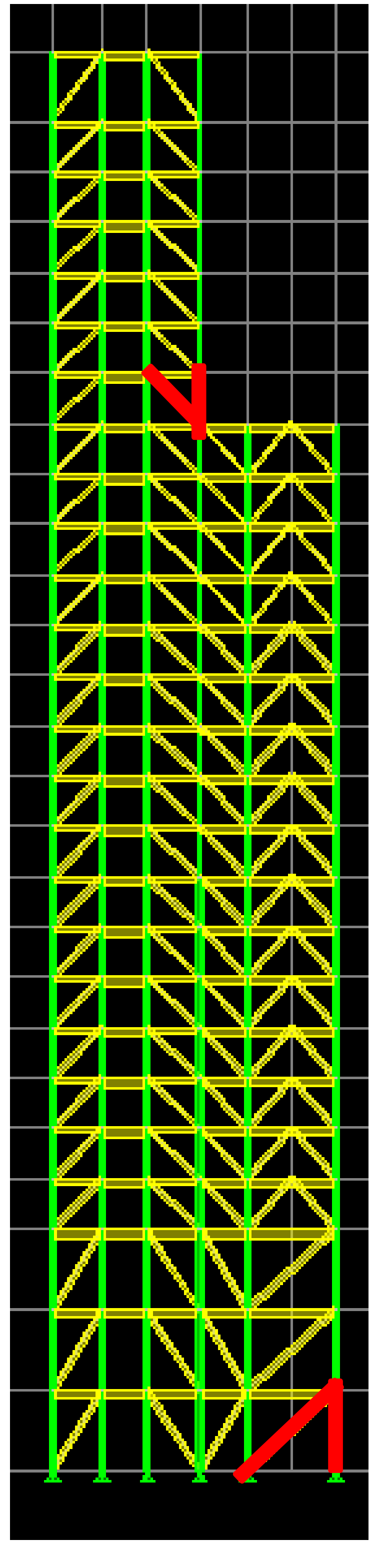

Figure 21: Checked members in red

Conclusion

To conclude this report of *"Lateral System Analysis and Confirmation Design,"* it was found that the lateral system of Res Tower II is adequate in both strength criteria and serviceability requirements. This conclusion was made by evaluating the system using both hand calculations and an Etabs computer model. Using a computer model provided a way to check hand calculations and a more accurate method of determining the lateral load path.

The computer model was used to determine relative stiffness of each frame, drift of the entire structure and the loads used to spot check members by hand. Four spot checks were completed to confirm that the model output was reasonably accurate. Two diagonal members and two columns were checked and proven adequate at two critical locations. The base level and level 20 were considered critical due to discontinuous frames at level 20 and large forces being transferred to the foundation at the base level.

Through hand checks and computer models, Res Tower II was proven to be adequate for strength and serviceability requirements.

Appendices:

A.1: Wind Pressures

North-South										
Floor	Elev	z	kz	q	windward (psf)	windward (p	windward	leeward (ps	leeward (plf)	leeward (k)
1	46	0	0.85	22.38	16.13	1720.32	13.763	-12.26	-1307.65	-10.461
2	62	16	0.86	22.64	16.32	1740.56	27.687	-12.26	-1307.65	-20.922
3	78	32	0.99	26.12	18.83	2007.72	23.963	-12.26	-1307.65	-16.999
4	88	42	1.05	27.65	19.93	2125.11	20.664	-12.26	-1307.65	-13.076
5	98	52	1.10	28.91	20.84	2222.25	21.737	-12.26	-1307.65	-13.076
6	108	62	1.14	29.96	21.60	2303.21	22.627	-12.26	-1307.65	-13.076
7	118	72	1.18	31.02	22.36	2384.17	23.437	-12.26	-1307.65	-13.076
8	128	82	1.22	32.02	23.08	2461.08	24.226	-12.26	-1307.65	-13.076
9	138	92	1.24	32.75	23.61	2517.75	24.894	-12.26	-1307.65	-13.076
10	148	102	1.27	33.31	24.01	2560.25	25.390	-12.26	-1307.65	-13.076
11	158	112	1.29	33.97	24.49	2610.85	25.855	-12.26	-1307.65	-13.076
12	168	122	1.32	34.62	24.96	2661.44	26.361	-12.26	-1307.65	-13.076
13	178	132	1.34	35.28	25.44	2712.04	26.867	-12.26	-1307.65	-13.076
14	188	142	1.36	35.89	25.87	2758.59	27.353	-12.26	-1307.65	-13.076
15	198	152	1.38	36.28	26.16	2788.95	27.738	-12.26	-1307.65	-13.076
16	208	162	1.39	36.70	26.46	2821.33	28.051	-12.26	-1307.65	-13.076
17	218	172	1.41	37.23	26.84	2861.81	28.416	-12.26	-1307.65	-13.076
18	228	182	1.43	37.73	27.20	2900.26	28.810	-12.26	-1307.65	-13.076
19	238	192	1.45	38.13	27.49	2930.62	29.154	-12.26	-1307.65	-13.076
20	248	202	1.46	38.51	27.77	2960.58	29.456	-12.26	-1307.65	-13.076
21	258	212	1.48	38.88	36.61	3904.01	34.323	-21.18	-2258.24	-17.829
22	268	222	1.49	39.25	36.89	2674.55	32.893	-21.18	-1156.03	-17.071
23	278	232	1.50	39.62	37.17	2694.55	26.845	-21.18	-1156.03	-11.560
24	288	242	1.52	39.99	37.44	2714.55	27.046	-21.18	-1156.03	-11.560
25	298	252	1.53	40.35	37.71	2733.98	32.711	-21.18	-1156.03	-13.872
26	312	266	1.55	40.79	38.04	2757.98	41.202	-21.18	-1156.03	-17.340
27	328	282	1.57	41.30	38.42	2785.41	41.562	-21.18	-1156.03	-17.340
ROOF	342	296	1.59	41.74	38.75	2809.42	19.666	-21.18	-1156.03	-8.092

East-West										
Floor	Elev	z	kz	q	windward (psf)	windward (plf)	windward (k)	leeward (psf)	leeward (plf)	leeward (k)
1	46	0	0.85	22.38	15.54	3782.14	30.257	-11.81	-2874.87	-22.999
2	62	16	0.86	22.64	15.73	3826.64	60.870	-11.81	-2874.87	-45.998
3	78	32	0.99	26.12	18.14	4413.98	52.683	-11.81	-2874.87	-37.373
4	88	42	1.05	27.65	19.20	4672.06	45.430	-11.81	-2874.87	-28.749
5	98	52	1.10	28.91	20.08	4885.64	47.788	-11.81	-2874.87	-28.749
6	108	62	1.14	29.96	20.81	5063.62	49.746	-11.81	-2874.87	-28.749
7	118	72	1.18	31.02	21.54	5241.61	51.526	-11.81	-2874.87	-28.749
8	128	82	1.22	32.02	22.24	5410.69	53.261	-11.81	-2874.87	-28.749
9	138	92	1.24	32.75	22.75	5535.28	54.730	-11.81	-2874.87	-28.749
10	148	102	1.27	33.31	23.13	5628.72	55.820	-11.81	-2874.87	-28.749
11	158	112	1.29	33.97	23.59	5739.96	56.843	-11.81	-2874.87	-28.749
12	168	122	1.32	34.62	24.05	5851.20	57.956	-11.81	-2874.87	-28.749
13	178	132	1.34	35.28	24.50	5962.44	59.068	-11.81	-2874.87	-28.749
14	188	142	1.36	35.89	24.92	6064.78	60.136	-11.81	-2874.87	-28.749
15	198	152	1.38	36.28	25.20	6131.52	60.982	-11.81	-2874.87	-28.749
16	208	162	1.39	36.70	25.49	6202.72	61.671	-11.81	-2874.87	-28.749
17	218	172	1.41	37.23	25.86	6291.71	62.472	-11.81	-2874.87	-28.749
18	228	182	1.43	37.73	26.20	6376.25	63.340	-11.81	-2874.87	-28.749
19	238	192	1.45	38.13	26.48	6442.99	64.096	-11.81	-2874.87	-28.749
20	248	202	1.46	38.51	26.75	6508.85	64.759	-11.81	-2874.87	-28.749
21	258	212	1.48	38.88	27.87	6780.62	66.447	-18.69	-4548.95	-37.119
22	268	222	1.49	39.25	28.13	3544.34	51.625	-18.69	-2672.50	-36.107
23	278	232	1.50	39.62	28.39	3577.63	35.610	-18.69	-2672.50	-26.725
24	288	242	1.52	39.99	28.66	3610.91	35.943	-18.69	-2672.50	-26.725
25	298	252	1.53	40.35	28.91	3643.25	43.557	-18.69	-2672.50	-32.070
26	312	266	1.55	40.79	29.23	3683.19	54.968	-18.69	-2672.50	-40.088
27	328	282	1.57	41.30	29.59	3728.84	55.567	-18.69	-2672.50	-40.088
ROOF	342	296	1.59	41.74	29.91	3768.78	26.381	-18.69	-2672.50	-18.708

A.2: Hand Calculations

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ASCE 7-10: DESIGN CRITERIA

RISK CATEGORY (TABLE 1.5.1) : III

BASIC WIND SPEED (FIG 26.5-1B) : $V = 140$ MPH

DIRECTIONALITY FACTOR (TABLE 26.6-1) : $K_d = 0.85$

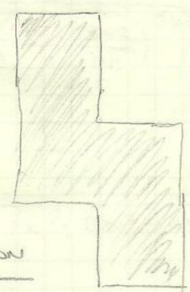
EXPOSURE CATEGORY (SPECIFIED IN PLANS) : C

TOPOGRAPHIC FACTOR (SECTION 26.8) : $K_{zt} = 1.0$

GUST FACTOR: (CAN'T ASSUME RIGID) (26.9)

26.9.2.1 : BUILDING HEIGHT = 342' > 300' ∴ 26.9.3 CAN NOT BE USED

DUE TO SLENDER FLOOR PLAN & STEEL FRAME SYSTEM } ASSUME FLEXIBLE



N ↑

NORTH-SOUTH DIRECTION

$g_Q = g_v = 3.4$

$g_R = \sqrt{2\ln(3{,}600\, n_1)} + \dfrac{0.577}{\sqrt{2\ln(3600\, n_1)}}$

n_1 = NATURAL FREQUENCY

$n_1 = \dfrac{1}{T_a}$; T_a = NATURAL PERIOD

(P.90) FROM 12.8.2.1 (SEISMIC DESIGN)

$T_a = C_t h_n^{x}$ $\quad C_t = 0.02$ (CONCENTRICALLY BRACED FRAMES)

$x = 0.75$

$n_1 = \dfrac{1}{(0.02)(\cancel{342}\ 296)^{0.75}}$ $\quad h_n = \cancel{342\text{ ft}}\ 296$

$n_1 = \cancel{0.6287}\ 0.701$ Hz < 1.0 ∴ FLEXIBLE ASSUMPTION CORRECT

$\sqrt{2\ln[3600(\cancel{0.6287}\ 0.701)]} = \cancel{3.9305}\ 3.958$

$g_R = \dfrac{\cancel{3.9305}}{3.958} + \dfrac{0.577}{\cancel{3.9305}\ 3.958} \Rightarrow g_R = \cancel{4.077}\ 4.104$

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(p. 255)

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \qquad \bar{z} = 0.6h \qquad h = 342$$

$$\bar{z} = 205.2 \geq z_{min} = 15 \text{ ft} \quad \underline{\text{ok}}$$

(TABLE 26.9-1)

$z_{min} = 15$

$c = 0.20$

$\ell = 500$

$\bar{\epsilon} = 1/5.0$

$$I_{\bar{z}} = (0.20)\left(\frac{33}{205.2}\right)^{1/6}$$

$$I_{\bar{z}} = 0.1475$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

* B = HORIZONTAL LENGTH $\perp$ TO WIND DIRECTION $= 54.583''$

$$L_{\bar{z}} = 500\left(\frac{205.2}{33}\right)^{1/5}$$

$$L_{\bar{z}} = 720.614$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{54.583 + 342}{720.614}\right)^{0.63}}}$$

$$Q = 0.8355$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47R_L)}$$

$$R_n = \frac{7.47N_1}{(1 + 10.3N_1)^{5/3}} \qquad N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V \qquad \bar{b} = 0.65 \quad \bar{\alpha} = 1/6.5$$

$$\bar{V}_{\bar{z}} = (0.65)\left(\frac{205.2}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)(140) \qquad V = 140$$

$$\bar{V}_{\bar{z}} = 176.797$$

$$N_1 = \frac{(0.6287)(720.64)}{176.797} \Rightarrow N_1 = 2.563$$

$$R_n = \frac{7.47(2.563)}{[1 + (10.3)(2.563)]^{5/3}} \Rightarrow R_n = 0.07688$$

TYLER MEEK | AE SENIOR THESIS | WIND CALCS | 3

R_h: $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.6287)(342)/176.797 \Rightarrow \eta = 5.594$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) = \frac{1}{5.594} - \frac{1}{2(5.594)^2}\left[1 - e^{-2(5.594)}\right]$$

$R_h = 0.16277$

R_B: $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.6287)(54.585)/176.797 \Rightarrow \eta_1 = 0.8929$

$$R_B = \frac{1}{0.8929} - \frac{1}{2(.8929)^2}\left(1 - e^{-2(0.8929)}\right) \Rightarrow R_B = 0.5980$$

R_L: $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$ $\quad$ L = HORIZONTAL DIMENSION // TO WIND DIRECTION

$$\eta = \frac{15.4(0.6287)(142.958)}{176.797} \qquad L = 142.958'$$

$\eta = 7.8288$

$$R_L = \frac{1}{7.8288} - \frac{1}{2(7.8288)^2}\left(1 - e^{-2(7.8288)}\right) \Rightarrow R_L = 0.1196$$

DAMPING RATIO (β) : 1.0 RECOMMENDED BY ISO (1997) ON PAGE 521 OF ASCE 7-10

$$R = \sqrt{\left(\frac{1}{0.01}\right)(0.07688)(0.16277)(0.5980)\left[0.53 + 0.47(0.1196)\right]}$$

$R = 0.6623$

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$$G_f = 0.925\left[\frac{1 + 1.7(0.1475)\sqrt{(3.4^2)(0.8355^2) + (3.4^2)(0.6623)^2}}{1 + 1.7(3.4)(0.1475)}\right]$$

1.8526

$\boxed{G_f = 0.953}$ FOR NORTH-SOUTH, TALL TOWER

TYLER MEEK | AE SENIOR THESIS | WIND CALCS | 4

ENCLOSURE CLASSIFICATION ⇒ ENCLOSED ∴ $GC_{pi} = \pm 0.18$
↳ NO OPERABLE WINDOWS

C_p: WALL PRESSURE COEFFICIENTS

			USED WITH
WIND WARD	→	0.8	q_z
LEE WARD	→	-0.35	q_h
SIDE WALL	→	-0.7	q_h

$L/B = 2.6$
ONLY FOR TALL TOWER

27.4.2 ENCLOSED FLEXIBLE BUILDINGS

$$P = q\,G_f C_p - q_i\,(GC_{pi})$$

$q = q_z$ FOR WINDWARD

$q = q_h$ FOR LEEWARD

$q_i = q_h$ FOR ENCLOSED

$$q = 0.00256\,K_z K_{zt} K_d V^2 \qquad K_h = 1.632$$

$$q_h = 69.6$$

WINDWARD:

$$P = q_z(0.953)(0.8) - 69.6(\pm 0.18)$$

$$P = (0.7624\,q_z + 12.53)\ \text{psf}$$

LEEWARD:

$$P = 69.6(0.953)(-0.35) - 69.6(\pm 0.18)$$

$$P = -35.74\ \text{psf}$$

N-S DIRECTION

FOR TALL TOWER ONLY

Appendix B: Seismic Calculations

TYLER MEEK | AE SENIOR THESIS | SIESMIC CALCS | 1

ASCE7-10

(p.65) 11.4: SEISMIC GROUND MOTION

GEOTECH REPORT SPECIFIES SITE CLASS S3 FROM MASSACHUSETTS BUILDING CODE

TABLE 1612.4.1

S3 ⇒ • UNDRAINE SHEAR STRENGTH ≥ 1000 psf
• CONTAINING 40 - 100 ft MEDIUM STIFF CLAY

ASSUMPTION SITE CLASS D: STIFF SOIL

$S_{MS} = F_a S_S$ FROM USGS DESIGN TOOLS: GROUND MOTION CALCULATOR

$S_{M1} = F_v S_1$

$S_S = 0.415$

$S_1 = 0.156$

(TABLE 11.4-1) $F_a = 1.468$

(11.4-2) $F_v = 2.176$

(11.4.3) $S_{MS} = F_a S_S = 0.60922$ $S_{M1} = F_v S_1 = 0.33946$

(11.4.4) $S_{DS} = \frac{2}{3} S_{MS} = 0.40615$

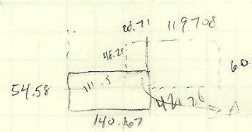
$S_{D1} = \frac{2}{3} S_{M1} = 0.22630$

(11.5.1) $I_e = 1.25$ (RISK CATEGORY III)

(TABLE 11.6-1) SEISMIC DESIGN CATEGORY ⇒ D (MORE SEVERE CASE)

USE EQUIVALENT LATERAL FORCE PROCEDURE (12.8)

$V = C_s W$



A = 16285 sf FOR 1-20

TYLER MEEK | AE SENIOR THESIS | SEISMIC CALCS 2

$V = C_s W$ $\quad W = 34449^k$ (CALCULATED USING EXCEL SPREADSHEET)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$R = 3.25$ PER TABLE 12.2-1
- CONCENTRICALLY BRACED FRAMES (ORDINARY)

$$C_s = \frac{0.40615}{\left(\frac{3.25}{1.25}\right)} \Rightarrow C_s = 0.1562$$

$\therefore V = (34449)(0.1562) \Rightarrow V = 5381^k$

(12.8.2.1) $\quad T_a = C_t h_n^x \Rightarrow T_a = 1.427$ (WORKED OUT IN WIND CALCS)

(12.8.3) VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} V$ $\qquad k = 1.4635$ (12.8.3)

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i}$$

CALCULATIONS COMPLETED USING SPREAD SHEET

ADD TO END OF REPORT

SEISMIC CALCULATIONS

$$\frac{0.3-0.2}{14-1.5} = \frac{0.3-0.22630}{14-C_u}$$

$C_u = 1.474$

STORY SHEAR IN X-DIRECTION

$$C_s = \min \begin{cases} S_{DS} / (R/I) \\ S_{D1} / (T \cdot R/I) \\ S_{D1} \cdot T_L / [T^2 \cdot R/I] \end{cases} = \begin{matrix} 0.10154 \\ 0.0548 \Leftarrow \text{CONTROLS} \\ 0.31811 \end{matrix}$$

$S_{DS} = 0.40615$

$S_{D1} = 0.22630$

$R = 5$ (SPECIFIED IN BUILDING PLANS)

$I = 1.25$

$T_L = 6$ (ASCE7-05) (FIG 22-15)

$$T = \min \begin{cases} C_u T_a = 1.033 \leftarrow \text{CONTROLS} \\ T_{MODEL} = 2.4020 \end{cases}$$

$C_u = 1.474$

$T_a = 0.701$

$$\boxed{V_{bx} = C_s W = 1798\ k}$$

Y - DIRECTION

T_{MODEL} DOES NOT CONTROL $\therefore V_{bx} = \boxed{V_{by} = 1798\ k}$

$T_{MODEL} = 1.9598 > 1.033$

Appendix C: Impact of Foundation

C

IMPACT ON FOUNDATION

3'-9" REINFORCED CONCRETE MAT FOUNDATION

$= (150 \text{ lb/ft}^3)(3.75 \text{ ft}) = 562.5 \text{ lb/ft}^2$

9" UNREINFORCED CONCRETE TRENCH

$= (145 \text{ lb/ft}^3)(0.75) = 108.75 \text{ lb/ft}^2$

REINFORCING CAGE BELOW LATERAL COLUMNS:

3'-6" x 15'-0"

USING DIMENSIONS OF CAGE & NEGLECTING THE EXTRA 9" OF UNREINFORCED CONCRETE, WEIGHT OF FOUNDATION IN CAGE AREA =

$(562.5 \text{ lb/ft}^2)(3.5')(15') = 29.5^k$

$29.5^k >$ ANY UPLIFT FORCE FOUND IN MODEL

Appendix D: Spot Checks

SPOT CHECKS

D

COLUMN @ BASE LEVEL: AXIAL ONLY

$$\left.\begin{array}{l} W14 \times 211 \\ KL = 16 \text{ ft} \end{array}\right\} \phi P_n = 2370 \text{ k} \gg 468 \text{ k} \;\therefore\; \text{ok}$$

COMBINED LOADING:

$P = 468$ k

$M_{rx} = 715$ ft-k

$M_{ry} = 1.29$ ft-k

$$pP_r + b_x M_{rx} + b_y M_{ry} \le 1.0$$

$$(0.422 \times 10^{-3})(468) + (0.613 \times 10^{-3})(715) + (1.20 \times 10^{-3})(1.29) = 0.637$$

$$0.637 < 1.0 \;\therefore\; \text{OK}$$

COLUMN @ LEVEL 20: W14x74

$P = 80.95$ k

$M_{rx} = 242.6$ ft-k

$M_{ry} = 11.34$ ft-k

$p = 1.58 \times 10^{-3}$

$b_x = 2.15 \times 10^{-3}$

$b_y = 5.85 \times 10^{-3}$

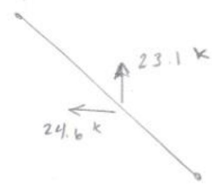
$$pP_r + b_x M_{rx} + b_y M_{ry} = 0.716 < 1.0 \;\therefore\; \text{OK}$$

D

SPOT CHECKS

DIAGONAL BRACING MEMBER @ LEVEL 20 : W8x31

COMPRESSION :



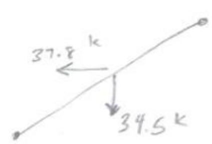
AXIAL = 33.75^k (C)

KL = 15 ft

$\phi P_n = 230^k > (33.75)(1.6) = 54^k$ ∴ OK

DIAGONAL BRACING MEMBER @ BASE : W12x79

COMPRESSION :



AXIAL = 51.18^k (C)

KL = 25 ft

@ KL = 26 ft, $\phi P_n = 486^k > 1.6(51.18^k) = 82^k$

∴ OK

Typical Story Height = 10 ft

Typical Upper Level = 22

CASE 1

NORTH SOUTH		**Direct:**	F_T =	42.10048 k	**Eccentric:**	F = k*d*Θ		Θ = P*e/Σ(K*d^2) =	0.022726 rad	**TOTAL FORCE:**			
Windward (psf)	Leeward (psf)		K	FORCE		K	d = distance to Cp	FORCE					
36.89	-21.18	F_7	0.182304	5.498212 k	F_7	0.182304	4.0417	0.016745		F_1	0.979464 k	SIGN CONVENTION
		F_8	0.182304	5.498212 k	F_8	0.182304	14.0417	0.058177		F_2	0.207537 k	(+) Left
(plf)	(plf)	F_9	0.601308	18.13519 k	F_9	0.601308	5.4583	0.074591		F_3	0.785959 k	(-) Right
368.90	211.79	F_{10}	0.062892	1.896803 k	F_{10}	0.062892	12.625	0.018045		F_4	1.170624 k	
		F_{11}	0.367116	11.07207 k	F_{11}	0.367116	36.25	0.302443		F_7	5.514957 k	
Cp =	36.25									F_8	5.556389 k	
CR =	40.502				Σ(K*d^2) =	7876.773				F_9	18.20978 k	
e = CR - Cp =	4.252	sign convention = ccw (+)								F_{10}	1.914848 k	
										F_{11}	10.76962 k	

East West		**Direct:**	F_T =	58.99829 k	**Eccentric:**	F = k*d*Θ		Θ = P*e/Σ(K*d^2) =	0.127333 rad	**TOTAL FORCE:**		
Windward (psf)	Leeward (psf)		K	FORCE		K	d = distance to Cp	FORCE				
28.13	-18.69	F_1	0.684094	12.76245 k	F_1	0.684094	63	5.487779		F_1	18.25023 k	
		F_2	0.672457	12.54535 k	F_2	0.672457	13.58	1.162799		F_2	13.70815 k	(+) Up
(plf)	(plf)	F_3	1	18.65598 k	F_3	1	34.5834	4.403598		F_3	14.25238 k	(-) Down
281.30	186.94	F_4	0.805881	15.03451 k	F_4	0.805881	63.9167	6.558816		F_4	8.47569 k	
										F_7	0.093821 k	
Cp =	63									F_8	0.325954 k	
CR =	80									F_9	0.417921 k	
e = CR - Cp =	17	sign convention = ccw (+)								F_{10}	0.101104 k	
										F_{11}	1.69454 k	

CASE 2

NORTH SOUTH		Direct:	F_T = 31.57536 k		Eccentric:	F = k*d*Θ		Θ = P*e/Σ(K*d²) =	0.060639 rad	TOTAL FORCE:		
Windward (psf)	Leeward (psf)		K	FORCE		K	d = distance to Cp	FORCE				
36.89	-21.18	F_7	0.182304	4.123659 k	F_7	0.182304	4.0417	0.04468		F_1	2.613421 k	SIGN CONVENTION
		F_8	0.182304	4.123659 k	F_8	0.182304	14.0417	0.155228		F_2	0.553755 k	(+) Left
(plf)	(plf)	F_9	0.601308	13.60139 k	F_9	0.601308	5.4583	0.199025		F_3	2.097106 k	(-) Right
368.90	211.79	F_{10}	0.062892	1.422602 k	F_{10}	0.062892	12.625	0.048148		F_4	3.123476 k	
		F_{11}	0.367116	8.30405 k	F_{11}	0.367116	36.25	0.806984		F_7	4.168339 k	
Cp =	36.25									F_8	4.278887 k	
CR =	40.502				Σ(K*d²) =	7876.773				F_9	13.80042 k	
e = CR - Cp (+/-)0.15Bx =	15.127	sign convention = ccw (+)								F_{10}	1.47075 k	
										F_{11}	7.497066 k	

East West		Direct:	F_T = 44.24872 k		Eccentric:	F = k*d*Θ		Θ = P*e/Σ(K*d²) =	0.201673 rad	TOTAL FORCE:		
Windward (psf)	Leeward (psf)		K	FORCE		K	d = distance to Cp	FORCE				
28.13	-18.69	F_1	0.684094	9.57184 k	F_1	0.684094	63	8.691674		F_1	18.26351 k	
		F_2	0.672457	9.409014 k	F_2	0.672457	13.58	1.841668		F_2	11.25068 k	(+) Up
(plf)	(plf)	F_3	1	13.99199 k	F_3	1	34.5834	6.974523		F_3	7.017463 k	(-) Down
281.30	186.94	F_4	0.805881	11.27588 k	F_4	0.805881	63.9167	10.38801		F_4	0.887873 k	
										F_7	0.148596 k	
Cp =	63									F_8	0.516254 k	
CR =	80									F_9	0.661914 k	
e = CR - Cp (+/-)0.15By =	35.9	sign convention = ccw (+)								F_{10}	0.160131 k	
										F_{11}	2.683853 k	

CASE 3

NORTH SOUTH

Windward (psf)	Leeward (psf)
36.89	-21.18
(plf)	(plf)
368.90	211.79

Cp = 36.25

CR = 40.502

e = CR - Cp = 4.252

sign convention = ccw (+)

Direct: F_T = 31.57536 k

	K	FORCE	
F_7	0.182304	4.123659	k
F_8	0.182304	4.123659	k
F_9	0.601308	13.60139	k
F_{10}	0.062892	1.422602	k
F_{11}	0.367116	8.30405	k

Eccentric: F = k*d*Θ Θ = P*e/Σ(K*d^2) = 0.017045 rad

	K	d = distance to Cp	FORCE
F_7	0.182304	4.0417	0.012559
F_8	0.182304	14.0417	0.043632
F_9	0.601308	5.4583	0.055943
F_{10}	0.062892	12.625	0.013534
F_{11}	0.367116	36.25	0.226832

Σ(K*d^2) = 7876.773

TOTAL FORCE:

F_1	0.734598	k
F_2	0.155653	k
F_3	0.589469	k
F_4	0.877968	k
F_7	4.136218	k
F_8	4.167292	k
F_9	13.65733	k
F_{10}	1.436136	k
F_{11}	8.077217	k

SIGN CONVENTION
(+) Left or
(-) Right

East West

Windward (psf)	Leeward (psf)
28.13	-18.69
(plf)	(plf)
281.30	186.94

Cp = 63

CR = 80

e = CR - Cp = 17

sign convention = ccw (+)

Direct: F_T = 44.24872 k

	K	FORCE	
F_1	0.684094	9.57184	k
F_2	0.672457	9.409014	k
F_3	1	13.99199	k
F_4	0.805881	11.27588	k

Eccentric: F = k*d*Θ Θ = P*e/Σ(K*d^2) = 0.0955 rad

	K	d = distance to Cp	FORCE
F_1	0.684094	63	4.115834
F_2	0.672457	13.58	0.872099
F_3	1	34.5834	3.302699
F_4	0.805881	63.9167	4.919112

TOTAL FORCE:

F_1	13.68767	k
F_2	10.28111	k
F_3	10.68929	k
F_4	6.356768	k
F_7	0.070366	k
F_8	0.244466	k
F_9	0.313441	k
F_{10}	0.075828	k
F_{11}	1.270905	k

(+) Left or
(-) Right

TOTAL FORCE:

F_1	14.42227
F_2	10.43677
F_3	11.27876
F_4	7.234736
F_7	4.206584
F_8	4.411757
F_9	13.97078
F_{10}	1.511964
F_{11}	9.348123

CASE 4

NORTH SOUTH

Windward (psf)	Leeward (psf)
36.89	-21.18
(plf)	(plf)
368.90	211.79

Cp =	36.25
CR =	40.502
e = CR - Cp (+/-)0.15Bx =	15.127

sign convention = ccw (+)

Direct: $F_T =$ 23.70257 k

	K	FORCE	
F_7	0.182304	3.095493	k
F_8	0.182304	3.095493	k
F_9	0.601308	10.21011	k
F_{10}	0.062892	1.0679	k
F_{11}	0.367116	6.233573	k

Eccentric: $F = k*d*\Theta$ $\Theta = P*e/\Sigma(K*d^2) =$ 0.04552 rad

	K	d = distance to Cp	FORCE
F_7	0.182304	4.0417	0.03354
F_8	0.182304	14.0417	0.116524
F_9	0.601308	5.4583	0.149401
F_{10}	0.062892	12.625	0.036143
F_{11}	0.367116	36.25	0.605776

$\Sigma(K*d^2) =$ 7876.773

TOTAL FORCE:

F_1	1.961808	k
F_2	0.415685	k
F_3	1.574228	k
F_4	2.344689	k
F_7	3.129033	k
F_8	3.212018	k
F_9	10.35951	k
F_{10}	1.104043	k
F_{11}	5.627798	k

SIGN CONVENTION

(+) Left
or
(-) Right

TOTAL FORCE:

F_1	15.67162
F_2	8.861197
F_3	6.842004
F_4	3.011186
F_7	3.240579
F_8	3.599552
F_9	10.85639
F_{10}	1.224248
F_{11}	7.642477

East West

Windward (psf)	Leeward (psf)
28.13	-18.69
(plf)	(plf)
281.30	186.94

Cp =	63
CR =	80
e = CR - Cp (+/-)0.15By =	35.9

sign convention = ccw (+)

Direct: $F_T =$ 33.21604 k

	K	FORCE	
F_1	0.684094	7.185261	k
F_2	0.672457	7.063033	k
F_3	1	10.50332	k
F_4	0.805881	8.464427	k

Eccentric: $F = k*d*\Theta$ $\Theta = P*e/\Sigma(K*d^2) =$ 0.151389 rad

	K	d = distance to Cp	FORCE
F_1	0.684094	63	6.52455
F_2	0.672457	13.58	1.382479
F_3	1	34.5834	5.235542
F_4	0.805881	63.9167	7.79793

TOTAL FORCE:

F_1	13.70981	k
F_2	8.445512	k
F_3	5.267776	k
F_4	0.666497	k
F_7	0.111546	k
F_8	0.387534	k
F_9	0.496877	k
F_{10}	0.120205	k
F_{11}	2.014679	k

(+) Left
or
(-) Right

(+) Up
(-) Down

k
k
k
k
k
k
k
k
k

(+) Up
(-) Down

(+) Up
(-) Down

k
k
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k

(+) Up
(-) Down