

OFFICE BUILDING-G

Eastern United States

Technical Report III



Carl Hubben

Structural Option

Advisor: Dr. Ali Memari

November 29, 2010

Table of Contents

Executive Summary.....	2
Introduction	3
Gravity System	3
Lateral System:	4
Foundation System:	5
Structural Materials	6
Code and Design Requirements	7
Design Codes:.....	7
Thesis Codes:.....	7
Report Content	8
As-Designed System.....	9
Design Loads and Deflection Limits	10
Wind:.....	10
Seismic:	14
Computer Model.....	16
Load Cases.....	18
Load Path	20
Shear	20
Torsion	22
Strength Checks	26
Drift and Displacement	28
Overturning.....	30
Conclusion.....	31
Appendix A.....	33
Appendix B.....	40
Appendix C.....	42
Appendix D.....	44

Executive Summary

Technical Report III is a lateral system analysis and design confirmation report of the existing lateral system of Office Building-G. The purpose of this report is to gain a broader understanding of the lateral system by determining which lateral loads will control the design, how the lateral loads are distributed, and verify the lateral load resisting system has been sufficiently designed for strength and serviceability. The lateral system of Office Building-G consists of a concrete shear wall core.

Preliminary hand calculations were performed to investigate and determine the relative stiffness of each lateral load resisting shear wall. It was concluded that the lateral loads were distributed accordingly to the relative stiffness of the shear walls in each respective direction. The shear wall stiffness was then used to perform a hand calculation to determine the center of rigidity, CR. This location was compared to the buildings center of mass, CM, and it was determined that the building has a natural eccentricity. This creates an inherent torsional force in wind loads and seismic loads which must be resisted by the lateral elements. The design of the shear walls must account for the direct forces as well as the torsional forces in their design.

A computer model was created in ETABS to verify the hand calculations. The structure of Office Building-G was created using a rigid diaphragm with a distributed floor mass, accounting for the dead load of the structure. With this applied mass, only the shear wall core element had to be modeled. Line loads were applied on the exterior of the diaphragm to account for the weight of the building façade. When the load combinations were applied the model, a single load combination did not control the strength design and serviceability so multiple design cases were analyzed. For strength design, load case $0.9D + 1.0W$ controlled in the North/South (Y) direction due to the large tributary width and load case $1.2D + 1.0L + 1.0E$ controlled in the East/West (X) direction.

Technical Report III concluded that no major concerns were found in the lateral design of Office Building-G with regard to torsion, shear, drift, displacement, and overturning. Future analysis should be performed in which all loads are applied to the entire structure to see how the systems interact with each other and to determine if different load cases control the design.

Introduction

Due to owner restrictions, the building name, location and tenant of Office Building-G cannot be disclosed. Neighboring an existing metro station, this 14 story building will become one the tallest of the modest skyline. Beneath the superstructure is a below grade, 4-story parking garage with space for 662 cars. On the first two floors of the building, a larger floor plan is used to accommodate for rentable space for retail, a restaurant, a bank and a loading dock. Typical floors have a square footage of 25,376 sf with a floor to floor height of 12'-3". The roof of the mechanical penthouse is 195 ft above grade and the gross square footage of the superstructure and garage combined is 649,461 sf.

The southern façade of the building is a curved glass curtain wall, breaking the mold of precast concrete panels the other three sides of the building follow. There is a setback on the first floor of the glass façade, exposing the exterior row of columns. On the first and second floor, the restaurant has a glass façade with concrete pilasters between the panes of glass.

Gravity System

Gravity loads are carried down the building through a combination of interior and exterior concrete columns and a shear wall core. The typical floor system is a cast-in-place concrete one-way slab. Thickness changes based on loading conditions but the typical floor is a 7", 5000 psi normal weight concrete slab. On the first floor, there is a 12" concrete slab designed for fire separation between the parking garage and superstructure. The slab system carries the loads to post-tensioned concrete beams with spans between 41'-5" and 45'-1 1/4".

The post tensioned beams range in width from 18" to 48" and have a maximum depth of 24". In Office Building-G, the typical girder is 18" deep by 48" wide. Forces in the beams are between 162 kips to 675 kips. These beams collect the floor loads from the slab and distribute their reactions to the columns supporting them.

Rectangular and round concrete columns then transfer the loads down the strictly followed grid. Typical floors have columns sizes of 24" x 24", 24" x 30", and 30" diameter. Smaller columns are used in the mechanical penthouse due to the much lower loads they are carrying. On above grade floors, higher strength concrete is placed below columns and shear walls in the slab to accommodate for any possibility of punching shear. In the parking garage, 8" drop panels are used instead of the different concrete strengths. The typical floor plan shown in Figure 1 below highlights the post-tensioned beams in yellow, the reinforced beams in purple, shear walls in green and blue, and the columns in red.

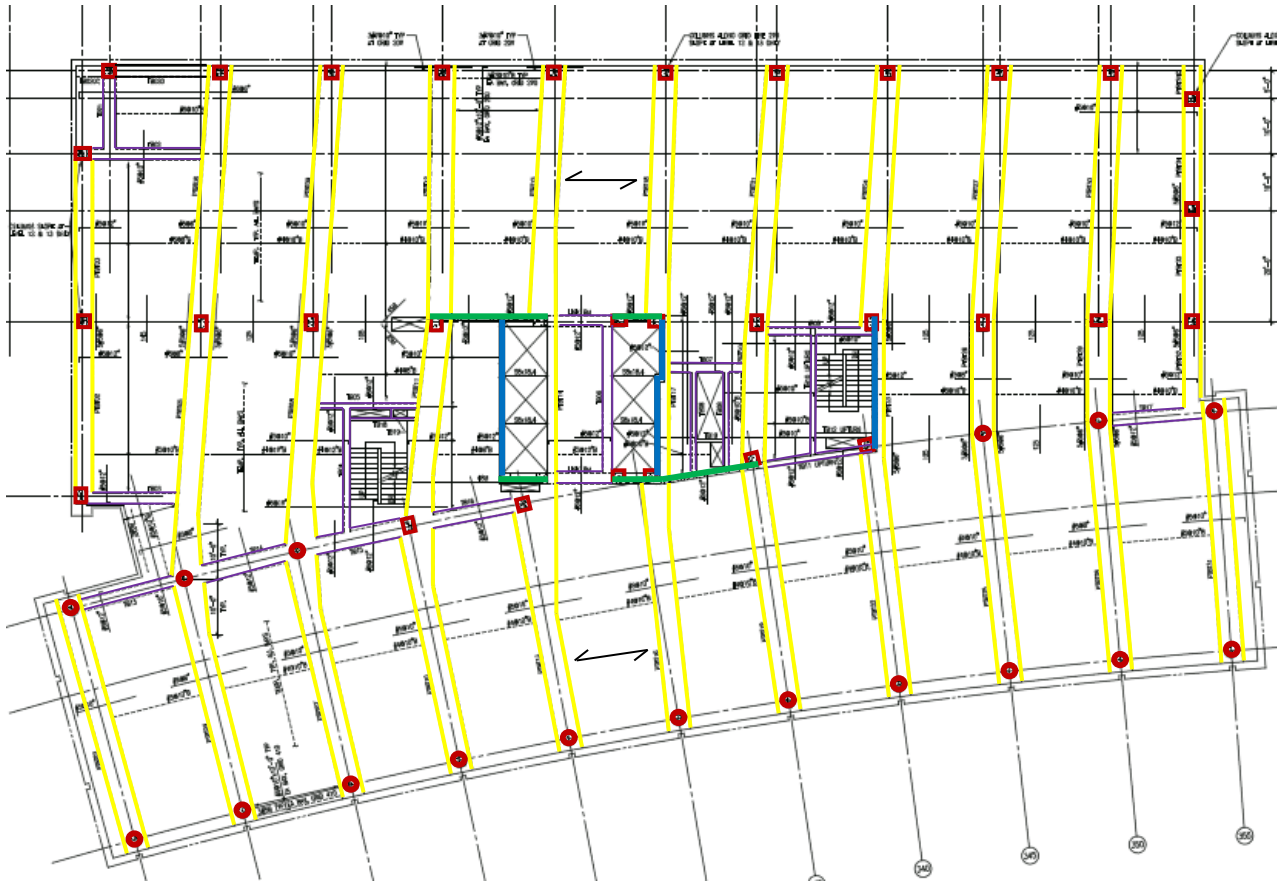


Figure 1

The above floor plan displays the skewed nature of Office Building-G. This condition was ignored during the design of the existing system as well as the alternative proposals. However, when comparing the systems at the end of the report, the advantages and disadvantages of the alternative systems on a skewed grid was considered.

Lateral System:

Wind and seismic forces are resisted by an internal shear wall core. The core is made of reinforced concrete walls which have a consistent floor plan from the bottom floor of the parking garage up to the slab of the roof. Basement shear walls were designed with $f'c = 10,000$ psi, levels 1-4 use $f'c = 8,000$ psi, and levels 5-14 use $f'c = 5,000$ psi. Precast concrete beams attached to concrete columns using precast lateral connections provide the required resistance for the mechanical penthouse and elevator machine room.

Lateral forces are engaged by the shear walls through the use of floor diaphragms. The building façade collects wind forces that are then transferred to the respective floor diaphragm. Forces then travel through the diaphragm until the shear walls are engaged, at which point the forces are distributed based on the relative stiffness of the walls.

Foundation System:

Schnabel engineering performed a geotechnical study for the location of Office Building-G which determined the possible foundation systems as spread footings, caissons or geopiers. The engineers of SK&A decided to use a system of spread footings under the columns, shear walls and along the perimeter concrete bearing wall. Square footage and depth of the footings are based on the load carrying capability of the soil and the vertical load on the column.

Service loads on the columns ranged greatly depending on whether or not the column extended up into the superstructure of the building. Based on the structure above the foundation, the load capacity of soil was determined to support a range of 3,000 psf to 10,000 psf. Loads on the footings varied between 60 kips to 3075 kips, once again depending on which part of Office Building-G they are supporting.

Structural Materials

Structural Materials			
Material	Element	Level	Strength
Cast-in-Place Concrete	Spread Footings	Foundation	$f'_c = 10,000$ psi
			$f'_c = 6,000$ psi
			$f'_c = 3,000$ psi
	Foundation Walls	B4	$f'_c = 5,000$ psi
		B3-B1	$f'_c = 4,000$ psi
	Shear Walls	B4-B1	$f'_c = 10,000$ psi
		L1-L4	$f'_c = 8,000$ psi
		L5-L7	$f'_c = 6,000$ psi
		L8-L14	$f'_c = 5,000$ psi
	Columns	B4-B1	$f'_c = 10,000$ psi
			$f'_c = 6,000$ psi
		L1-L4	$f'_c = 10,000$ psi
			$f'_c = 8,000$ psi
			$f'_c = 6,000$ psi
	L5-L7	$f'_c = 6,000$ psi	
L8-Roof	$f'_c = 5,000$ psi		
Reinforced Beams	ALL	$f'_c = 5,000$ psi	
Post-Tensioned Beams	ALL	$f'_c = 5,000$ psi	
Tendons	Post-Tensioned Beams	ALL	$F_u = 270$ ksi
Reinforcing Steel	Concrete	ALL	$F_y = 60$ ksi
Structural Steel	Elevator Framing - A36	ALL	$F_y = 36$ ksi
	Bolts - A325	ALL	$F_u = 120$ ksi

Code and Design Requirements

Design Codes:

National Model Code:

Local building code based on the 2006 International Building Code

Sections: 1603.1.1-1603.1.7, 1603.2, 1607.11, 1608.1, 1608.7, 1608.8, 1609.1

Design Codes:

American Concrete Institute (ACI) 318-08, Building Code Requirements for Structural Concrete and Commentary

ACI 301, Specifications for Structural Concrete for Buildings

ACI 347, Standard Recommended Practice for Concrete Formwork

American Institute for Steel Construction (AISC), Specification for the design, fabrication and erection of structural steel for buildings

Thesis Codes:

National Model Code:

International Building Code, 2006

Design Codes:

ACI 318-08, Building Code Requirements for Structural Concrete and Commentary

American Institute for Steel Construction (AISC), Specification for the design, fabrication and erection of structural steel for buildings

Structural Standards:

American Standards of Civil Engineers (ASCE), ASCE 7-10 Minimum Design Loads for Buildings and Other Structures

Report Content

Technical Report III is an in-depth analysis of the lateral system used in Office Building-G. This report follows the order in which tasks were performed to confirm the existing design as an adequate structure. Below is a list of the topics covered in this report along with a brief description of what they entailed:

- **As-Designed System**
 - This section is a more detailed description of the lateral system and the structural techniques applied.
- **Design Loads**
 - These values were taken from Technical Report I and the calculations associated with them can be found in Report I.
- **Computer Model**
 - ETABS was used to model the existing structure of Office Building-G and assumptions associated with the model can be found in this section.
- **Load Cases**
 - Various ASCE 7-10 load cases were analyzed and chosen for analysis based on the potential to control lateral system elements.
- **Load Path**
 - Computer analysis and hand calculations were used to confirm the expected load path as well as the forces in the lateral members.
- **Strength Checks**
 - Computer results and hand checks were used to confirm forces in lateral members were not greater than the allowable strengths.
- **Drift and Displacement**
 - Story drift and total building displacement is analyzed and compared to the allowable amounts.
- **Overturning**
 - The magnitude of overturning moment is calculated and possible effects on the foundation design are considered.

As-Designed System

As mentioned in the Introduction, Office Building-G relies on an internal shear wall core to resist lateral forces. The core surrounds the elevators and stair cases used for vertical transportation throughout the height of the building. This allows for the core to keep the same floor plan from the bottom floor of the basement to the 14th floor of the superstructure. Figure 2 is a plan view of the core.

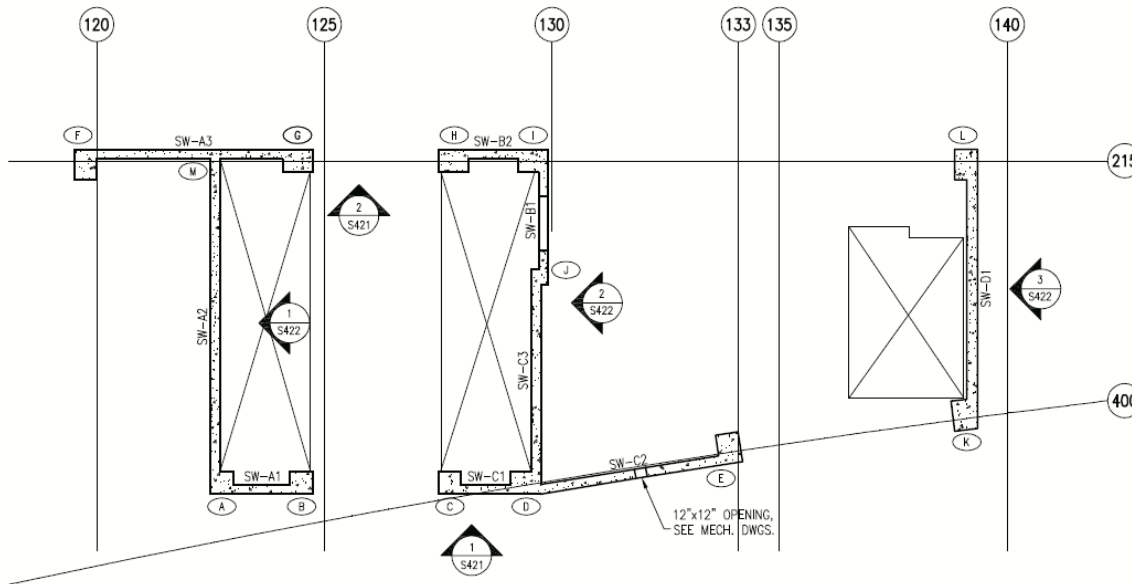


Figure 2

Within the core, there are shear walls, link beams and columns. The link beams are not shown in the figure above but they are located between columns G-H and B-C. Both are 30” deep by 24” wide. All of the shear walls are 10” thick and the columns are 24” x 30” with their orientation depending on the plan above.

Columns on the end of the shear walls create a much more effect shear wall in which more strength is achieved in a smaller area. A shear wall will resolve bending forces into a compression and tension load which will concentrate itself at the chords at each end of the wall. The columns at either end of the shear wall be designed to resist these forces and are much more effective at distributing these loads than a stand-alone shear wall would be.

Design Loads and Deflection Limits

A detailed description of how the gravity and lateral forces on the building were determined is provided below.

Superimposed Dead Loads		
Load Description	Load Location	Design Load
Superimposed	All	5 - Mech/Elec/Ceiling
Curtain Wall	Levels 1-14	25 - Vertical Surface

Floor Live Loads			
Load Description	Load Location	Design Load (psf)	ASCE 7-10 Load (psf)
Office	Levels 1-14	80 20 - Partitions	80

Live Load deflection limitation will be $L/360$

Service Load deflection limitation will be $L/240$

Construction Load deflection limitation will be $L/180$

Wind:

ASCE 7-10 was used for the determination of the wind loads for the Main Wind-Force Resisting System (MWRFS) of Office Building-G. Loads were calculated in the North-South and in the East-West direction due to the roughly rectangular shape of the building. The forces were determined using the Chapter 27 guidelines for Enclosed and Partially Enclosed Rigid Buildings.

The first step in calculating wind loads is determining if the building is flexible or rigid. This classification is based on the natural frequency of the structure. ASCE 7-10 allows for an estimation of a buildings frequency based on relationships between the building height and characteristics of the lateral force resisting system. Through this estimation it was determined that the natural frequency of Office Building-G > 1 defining the building as rigid.

The building is fairly square on three sides but the curved southern façade creates a scenario where the West wall has a greater length than the East wall. If the curvature had been so severe that the West wall was wider than the North wall is deep, an additional wind load would have had to be calculated. Since this is not the case and $L/B < 1$ a single wind load calculation can be used for both the East and West loads. Using the similar rationale, the North-South wind loads were calculated using the worst case for the different geometries of the building. The

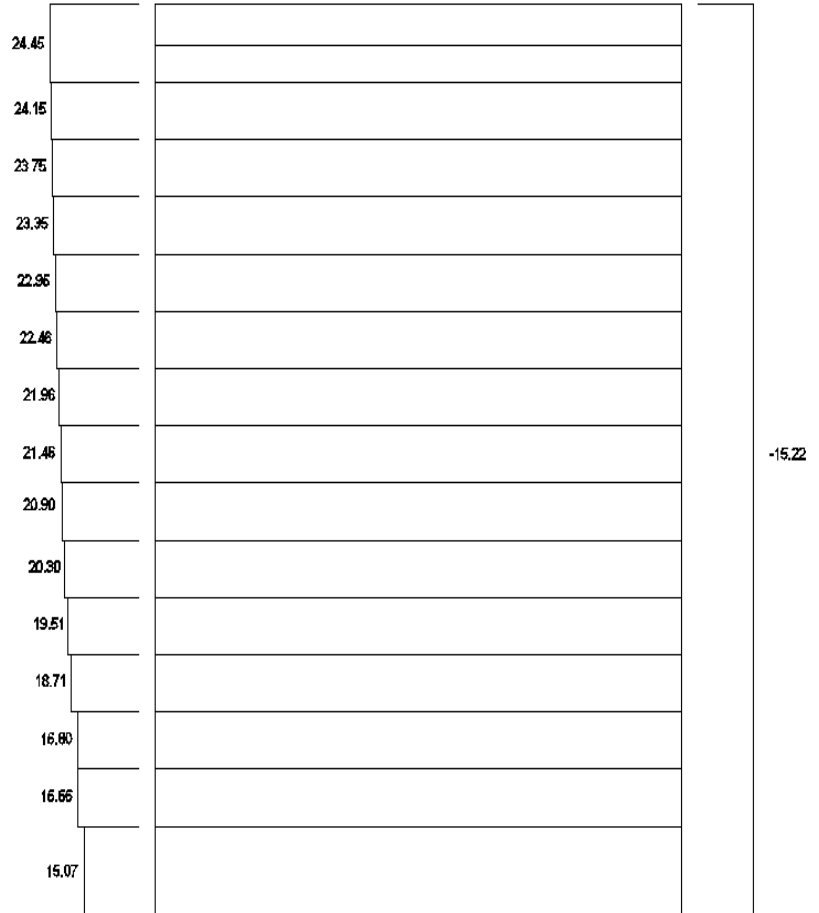
building receives the largest wind force in the North-South directions, as these are the longer façades of the building normal to the wind.

For the computer analysis wind load, an automatic load calculation was performed by the computer program ETABS. Wind loading information was put into the load calculator and ETABS determined the loads based off of ASCE 7-05 guidelines. Although ASCE 7-05 was used by ETABS, an ASCE 7-10 wind speed of 120 mph was entered, a change from 90 mph in ASCE 7-05. Having ETABS calculate the different load cases possible from wind loading was a major advantage. This is described in more detail in the Load Case section of this report.

In order to check that the wind forces calculated by hand and those determined by ETABS are of the same magnitude, a hand comparison of the largest possible forces was performed and can be found in Appendix A.

North-South Wind Distribution

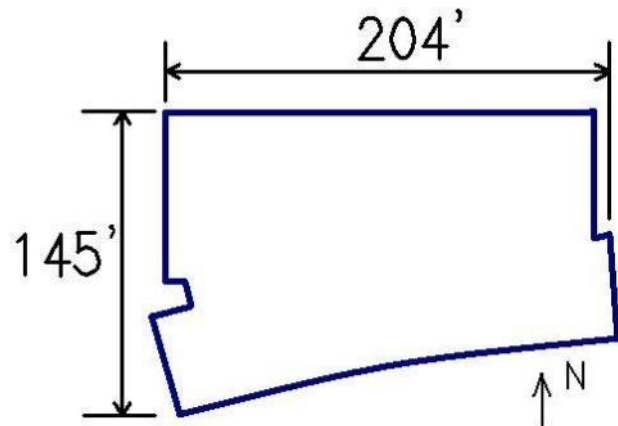
North-South			
Story Level	Story Height (ft)	Windward pz (psf)	Leeward ph (psf)
Roof	195	24.45	-15.22
Elevator	186.0	24.45	-15.22
Penthouse	178.3	24.15	-15.22
14	166.0	23.75	-15.22
13	153.8	23.35	-15.22
12	141.5	22.95	-15.22
11	129.3	22.46	-15.22
10	117.0	21.96	-15.22
9	104.8	21.46	-15.22
8	92.5	20.90	-15.22
7	80.3	20.30	-15.22
6	68.0	19.51	-15.22
5	55.8	18.71	-15.22
4	43.5	16.80	-15.22
3	31.3	16.66	-15.22
2	19.0	15.07	-15.22
1	0	15.07	-15.22



L=204'

B=145'

The larger B value of Office Building-G's dimensions was used in order to decrease the L/B ratio. The C_p multiplier used when calculating Leeward Wind forces is inversely related to this ratio so when L/B decrease, C_p and thus the wind force, increase.



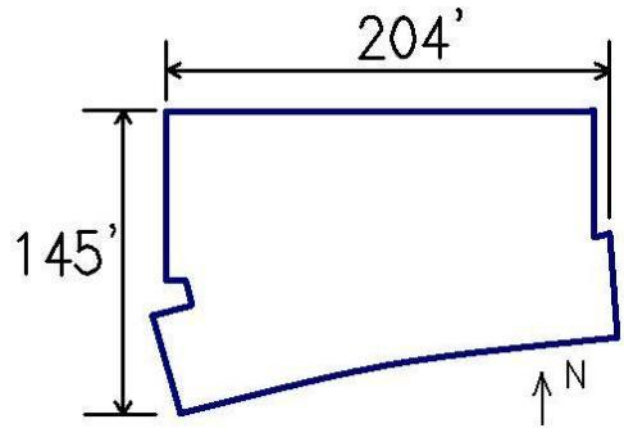
East-West Wind Distribution

L=145'

B=204

As noted above, a conservative value for the L/B ratio should be used to ensure the greatest value of C_p . In this particular instance the L value used does not matter because both ratios (either 114'/204' or 145'/204') result in a value of less than one, resulting in a C_p value of -0.5.

East-West			
Story Level	Story Height (ft)	Windward pz (psf)	Leeward ph (psf)
Roof	195	24.45	-17.20
Elevator	186.0	24.45	-17.20
Penthouse	178.3	24.15	-17.20
14	166.0	23.75	-17.20
13	153.8	23.35	-17.20
12	141.5	22.95	-17.20
11	129.3	22.46	-17.20
10	117.0	21.96	-17.20
9	104.8	21.46	-17.20
8	92.5	20.90	-17.20
7	80.3	20.30	-17.20
6	68.0	19.51	-17.20
5	55.8	18.71	-17.20
4	43.5	16.80	-17.20
3	31.3	16.66	-17.20
2	19.0	15.07	-17.20
1	0	15.07	-17.20



Seismic:

The Equivalent Lateral Force Procedure is in chapters 11 and 12 of ASCE 7-10 and these were referenced during the calculation of the seismic loads for Office Building-G. General design parameters of the building are a site classification of type D, a seismic design category of B, and a seismic importance factor of 1.0.

The first step taken in determining the seismic forces of the building was to determine the seismic response coefficient; C_s . C_s is based on a variety of factors that take into account the lateral system of the building as well as its geographical. The lateral system of the building is classified as ordinary reinforced concrete shear walls, corresponding to a response modification factor of $R=5$. When determined, C_s can then be multiplied by the total dead load weight of the building to yield the seismic base shear.

The next step was to consider all of the possible areas that could contribute to the dead weight of the building. The building elements considered were: slabs, beams, columns, shear walls, exterior walls, partitions, and imposed MEP loads. These loads were either a pound per square foot or a total per floor, depending on the nature of the element. It should be noted that partitions included in a 100 psf live load for office space but since they are secured to the floor of the structure it was assumed that they will not move freely in the instance of an earthquake.

A typical floor plan was used to determine many of the weights calculated. This yielded a very reasonable estimate because Office Building-G follows a typical floor plan design and variations are uncommon and minor.

Shear forces and the corresponding overturning moments at each floor were calculated and the total of these forces can be compared to the wind forces. The table below displays the story force, the total shear at that level, as well as the moment caused by seismic forces. An additional table provides values of a rotational moment that could result due to a slight eccentricity of the seismic forces. These seismic loads are for both North-South and East-West directions. There would have been different seismic loads had one direction had a lateral system consisting of shear walls while the other utilized a different system like moment frames for example. Reference the Appendix A for a complete list of values used and calculations.

Seismic Forces						
Level	Height (ft)	W_x (k)	$w_i * h_i^k$	f_x (k)	V_x (k)	Turnover Moment (k-ft)
Roof	195	126	97,684	5	5	879
Elevator	186	421	307,496	14	19	2640
Penthouse	178	5457	3,777,032	153	172	27258
14	166	5300	3,353,492	135	307	22445
13	154	5300	3,044,317	123	430	18872
12	142	5300	2,741,529	111	540	15641
11	129	5300	2,445,532	99	639	12744
10	117	5300	2,156,798	87	726	10174
9	105	5300	1,875,881	76	801	7923
8	93	5300	1,603,449	65	866	5980
7	80	5300	1,340,323	54	920	4337
6	68	5300	1,087,537	44	964	2982
5	56	5300	846,448	34	998	1903
4	44	5300	618,927	25	1023	1086
3	31	5300	407,758	16	1039	514
2	19	5300	217,642	9	1048	167
Total Base Shear (k)=					1048	
Total Over Turning Moment (k-ft)=					135544	

Computer Model

For the majority of the lateral analysis performed on Office Building-G, a CSI structural analysis program, ETABS, was used. ETABS was used because it is capable of analyzing a structure under a variety of different load combinations and display results in a user friendly, graphical way. Certain assumptions were taken when modeling the existing conditions of the building.

The first assumption taken in the computer model which differs from the actual structure is the general shape of the floor plan. The model created uses a rectangle to estimate the floor plan geometry of Office-Building-G. This was done to simplify the grid lines used. Modeling techniques described in the following paragraphs allowed an accurate model to be creating without including every member so the floor plan only needed to represent the basic extents and square footage of Office-Building G.

Another assumption in the ETABS model was aligning one of the shear walls to be parallel to the loads it was designed to resist. The original shear wall is 9° off of the horizontal, thus the majority of the concrete will be resisting forces acting in the X direction. In ETABS, the wall was modeled parallel to the other X direction shear walls as well as slightly shorter to compensate for change in orientation. It should be noted that there was no shear wall added in the Y direction to account for the concrete in the off axis wall that would have resisted forces in this direction.

As mentioned above, a technique was used to model the lateral resisting members only and not include any of the gravity elements. This was done by creating a rigid floor diaphragm and assigning a mass per unit area value to in. The mass per unit area was based off of the dead load of the building and structure, therefore accounting for the missing building elements. This created a much simpler model while not taking away any of the accuracy. A typical floor plan of the model is shown below in Figure 3.

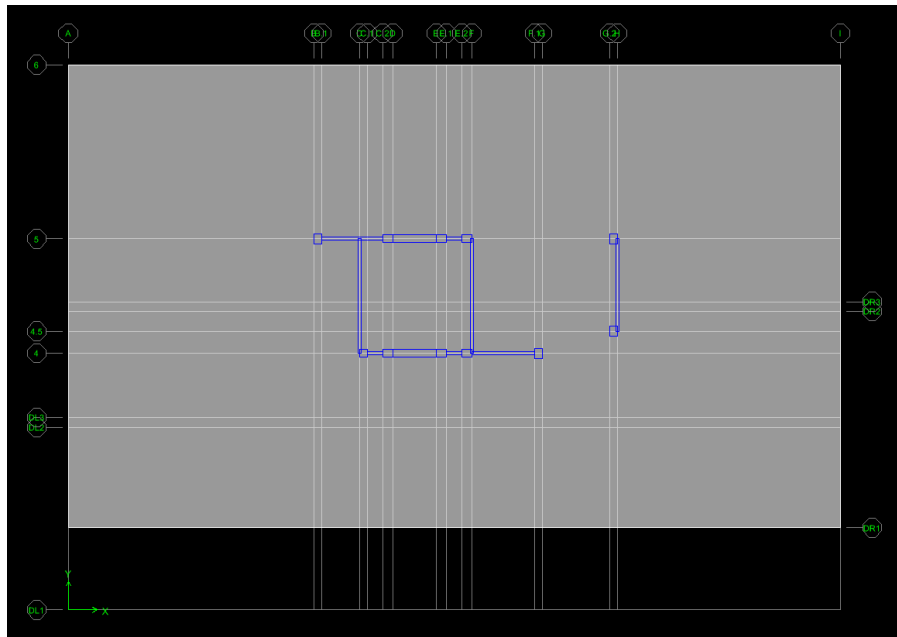


Figure 3

Another assumption taken to simplify the ETABS model while still gaining the accurate information desired was not including the below grade floors. The lateral forces in the superstructure do not transfer into the below grade levels due to the base shear reaction of the slab-on-grade. This being the case, the four levels of shear wall below the first story provide no resistance to the lateral loads experienced by the superstructure. The shear walls were modeled to have a fixed connection at the base of the structure and these reactions were analyzed to determine the possibility of uplift forces.

Load Cases

ASCE 7-10 section 2.3, Combining Factored Loads Using Strength Design, was used in determining which load cases would be applied to Office Building-G. The load combinations considered are listed below.

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

Typically, when only gravity loads are being considered, load case 2 will control. However, when lateral forces are being analyzed, cases 4-7 may control based on the magnitude of the forces and whether overturning moment is considered.

Figure 27.4-8 of ASCE 7-10 describes the different loading conditions for wind on a building. All four of the cases for the Main Wind Force Resisting System must be considered in the analysis of the lateral system. These cases account for the effects that wind has on a structure when wind blows from two different directions and are applied slightly off axis. As shown in Figure 4, cases 2 and 4 consider the torsional loads that can be induced by wind loading.

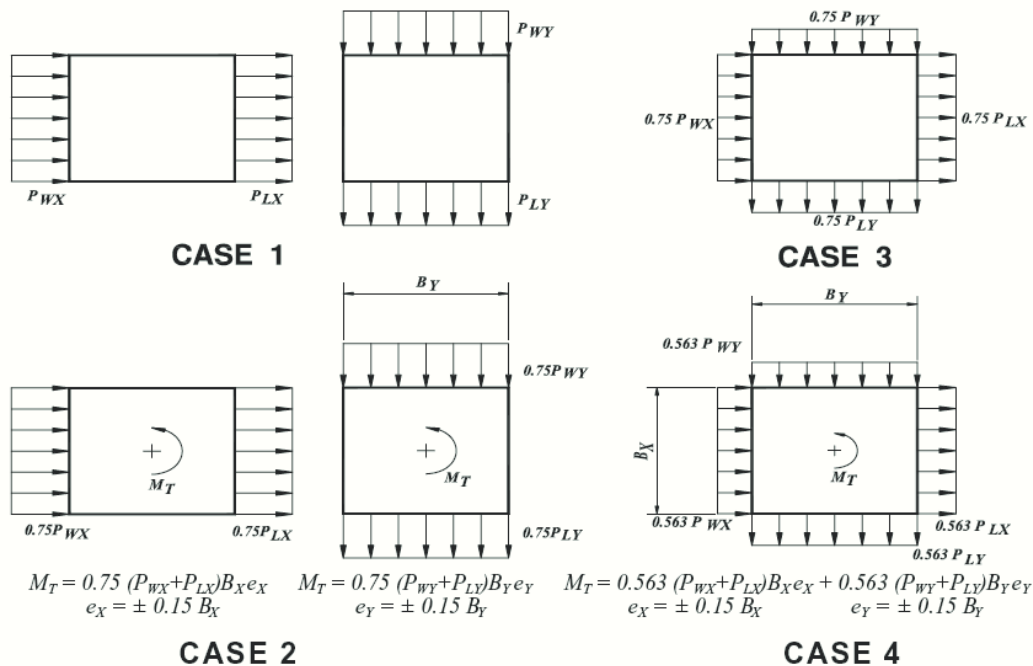


Figure 4

Due to the multiple cases and combinations the building could possibly experience an automatic calculation of the wind loads was used in the ETABS analysis. This was done by putting in general building and site information and allowing the program to calculate the possible loads. The controlling computer generated cases were checked and confirmed.

In total, there are 12 different wind loads that Office Building-G can expect to experience. To account for these possibilities, 12 iterations of each of the above load combinations which included a wind component were input into ETABS. Load combination 7 was also given an X and Y case when entered into ETABS. The multiple loads cases for wind and earthquake forces changed the number of load combinations from 7 to 43. A complete list of the load combinations, forces used and the confirming wind calculations for Office Building-G can be found in Appendix A.

Controlling Cases

The load combinations which had the greatest effect on the structure were determined based on the forces in the shear wall core as well as the diaphragm deflections. Based on the diaphragm deflections of the entered load combinations, combination 42 ($0.9D + 1.0E$) caused the largest deflection in the X-direction and combination 41 ($1.2D + 1.0L + 1.0E$) caused the largest in the Y-direction. Shear forces in the Y-Direction are controlled by load combination 29, ($0.9D + 1.0W$). This can be attributed to the larger width of the building in this direction. The X-Direction wind forces do not control because the tributary width is not large enough to create a force larger than the seismic force. Load combination 40 ($1.2D + 1.0L + 1.0E$) creates the largest shear forces in the X-Direction.

Load Path

When wind loads act on a building, the façade collects the shear per story and transfers the force into the floor diaphragm. Since seismic loads are created primarily by the inertial effect of the dead load, the force is applied to the center of mass on each floor. Whether a lateral force is due to wind or an earthquake, the loads end up in the floor diaphragm. The shear is then distributed into the lateral elements as a direct force or a force due to torsion. In directly loaded cases, the forces are distributed based on the relative stiffness of the member. For torsional forces, the relative stiffness as well as the distance from the applied force determines the amount of force in each element.

Shear

In the lateral system of a building, the load distribution is determined by the relative stiffness of the lateral force resisting elements. Stiffness, K , is the ratio between an applied load and the deflection of the member in the direction of the load. Relative stiffness is the ratio of an individual element's stiffness and the total stiffness in that direction. Since individual K values are divided by the total, the sum of all of the relative stiffness's in a particular direction should add up to one. The relative stiffness is what determines the percentage of direct shear that each element will resist. Shear due to torsion will be discussed further in the next section. Figure 5 below is a floor plan which labels the lateral resisting elements along with their relative stiffness. The calculations used to determine the relative stiffness can be found in Appendix B.

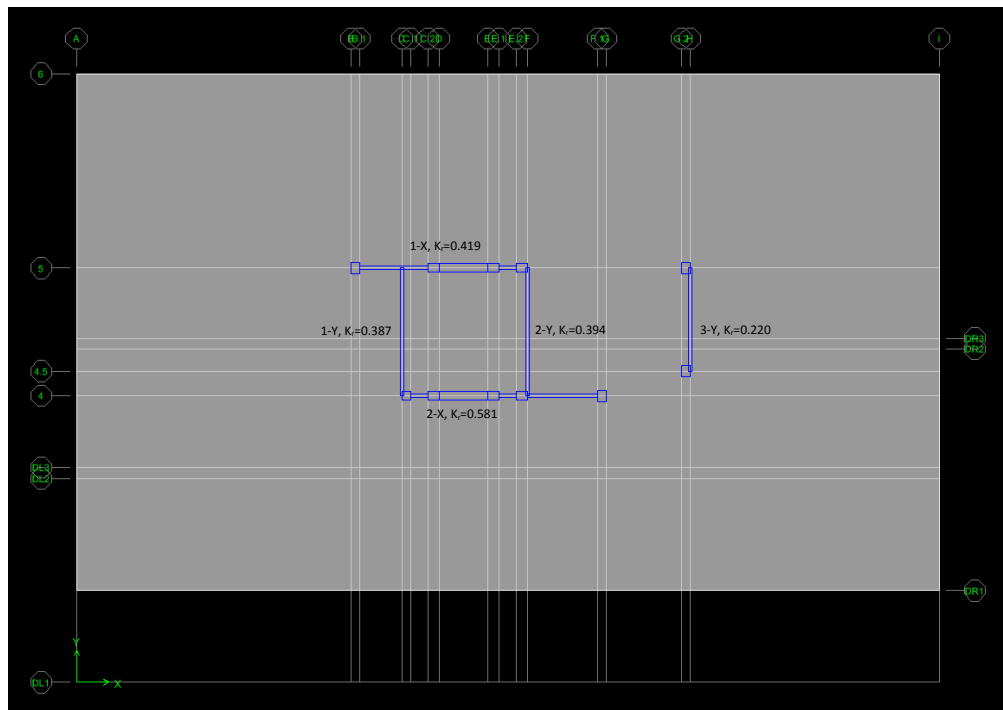


Figure 5

A check to determine if the ETABS model was distributing the loads in the assumed fashion was performed. This consisted of taking section cuts on the 1st floor of shear walls in both directions. Using the force in one shear wall, divided by the total force applied in that direction, the percent of the total force can be determined. This %Force should closely match the relative stiffness of the wall. The results from this check can be seen below.

X-Direction Load Case 40			
Shear Wall	Force	% Force	K
1-X	494.4	0.483521	0.419
2-X	528.1	0.516479	0.581
Total	1022.5	1	1

Y-Direction Load Case 29			
Shear Wall	Force	% Force	K
1-Y	494	0.409279	0.387
2-Y	467	0.38691	0.394
3-Y	246	0.203811	0.22
Total	1207	1	1

Although the % Force does not exactly equal the previously calculated stiffness, K, the general trend and distribution of forces is noticeable. The slight difference in these two values can be attributed to the way in which the stiffness was calculated. Shear walls with perpendicular shear walls at either end are much more rigid due to the increased moment of inertia due to the end conditions. This geometry was accounted for in the calculation of the Y-Direction K values and the distribution of forces closely matches the relative stiffness of the members. However, in the X-Direction the effects of neighboring shear walls were not accounted for in the stiffness determination. This approximation was made because the perpendicular walls were not expected to have a large effect on the elements. In order to have % Force match K in the X-Direction, a calculation which accounts for the perpendicular shear walls should be performed.

In the X-Direction, the controlling shear force comes from load combination 40, 1.2D + 1.0L + 1.0E. Since earthquake forces are the lateral force, the total shear in the X-Direction at story 1 should be equal to the base shear of the structure. The base shear is 1048 k, slightly larger than the 1022.5 k, the total from shear wall 1-X and 2-X. This is an expected result because although the vast majority of lateral forces are resisted by the parallel elements, out of plane shear walls will take a certain amount of load.

The applied forces in the Y-Direction from load combination 29, 0.9D + 1.0W, are shown in the below table.

Load Combination 29 Wind Forces					
Case	Story	Diaphragm	FX (k)	FY (k)	MZ (k-in)
AUTOWIND-2	STORY14	D1	0	50	0
AUTOWIND-2	STORY13	D1	0	99.16	0

AUTOWIND-2	STORY12	D1	0	97.83	0
AUTOWIND-2	STORY11	D1	0	96.42	0
AUTOWIND-2	STORY10	D1	0	94.92	0
AUTOWIND-2	STORY9	D1	0	93.31	0
AUTOWIND-2	STORY8	D1	0	91.58	0
AUTOWIND-2	STORY7	D1	0	89.69	0
AUTOWIND-2	STORY6	D1	0	87.6	0
AUTOWIND-2	STORY5	D1	0	85.27	0
AUTOWIND-2	STORY4	D1	0	82.6	0
AUTOWIND-2	STORY3	D1	0	79.44	0
AUTOWIND-2	STORY2	D1	0	75.5	0
AUTOWIND-2	STORY1	D1	0	84.22	0
Total				1207.54	

The total applied force is equal to the total resisted force by the shear walls in the Y-Direction. This is expected due to the modeling technique described above. The out of plane walls were included in determining the stiffness of the Y-Direction walls. This means that the Y-Direction shear walls will show all of the applied forces even though the out of plane walls are responsible for resisting the load.

Torsion

Torsional forces are created when an eccentric load is applied to the structure. An eccentric load can be defined as any load not directly applied to the center of rigidity. In seismic loads, an inherent eccentricity and an accidental eccentricity are combined. In wind loads, the force is applied at the geometric center of the story. Due to the way in which Office Building-G was modeled, the center of mass is also the geometric center of the floor plan, creating an inherent torsion due to wind. Additionally, certain wind cases will induce an intentional eccentricity by applying the forces at 15% the buildings width from the center of rigidity.

Inherent torsional forces are created in a building when the center of rigidity and center of mass are not in the same location. This puts the diaphragm in an eccentric loading condition and the floor diaphragm will rotate around the center of rigidity. The eccentricity of the CM and CR acts as a moment arm for the applied forces so with a greater eccentricity, larger torsional forces are created. The below table is in inches and displays the center of mass, center of rigidity and the eccentricity of each floor. Since the lateral system is identical for each story, all of the calculated values are the same for each floor.

	Center of Rigidity				Center of Mass				Eccentricity	
	ETABS Output		Hand Calculations		ETABS Output		Hand Calculations		ETABS Values	
	X	Y	X	Y	X	Y	X	Y	e _x	e _y
14	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
13	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
12	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
11	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
10	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
9	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
8	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
7	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95

6	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
5	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
4	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
3	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
2	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95
1	1269	686.8	1241	712.3	1222.6	732.75	-	-	46.4	-45.95

This rotational force will need to be resisted by the lateral members. Members furthest from the center of rigidity are more efficient in resisting torsional forces than closer elements because the larger moment arm of the element allows it to resist torsion with a smaller force. Hand calculations for the relative stiffness and center of rigidity can be found in Appendix B.

Wind

The table below displays the shear forces due to wind loading in the 1st story shear walls of Office Building-G. Combination 16 is a wind load applied with no additional eccentricity and combination 18 applied with an intentional eccentricity.

Torsional Effects		
Shear Wall 1st Floor	Combination 16 Shear (Inherent eccentricity)	Combination 18 Shear (Intentional eccentricity)
1-X	-316.7	-366.4
2-X	-401.2	-172.5
1-Y	-32.4	-106.22
2-Y	25.4	6.55
3-Y	6.95	99.7

It is important to note that the magnitudes of the applied forces in combination 16 are larger than those in combination 18 which will slightly skew the comparison but the values are relatively close and the general concept is displayed. Both of the load combinations are being applied in the X-Direction which explains the larger forces in the X-Direction. There is an inherent eccentricity in Office Building-G’s design because the geometric center is not in the same location as the center of rigidity. The natural eccentricity creates a small torsional force which is resisted by all of the lateral walls, explaining the small forces in the Y direction. Load combination 18 is applied with an intentional eccentricity of 15% the building width from the center of rigidity. This creates a moment around the center of rigidity as shown below in Figure 6.

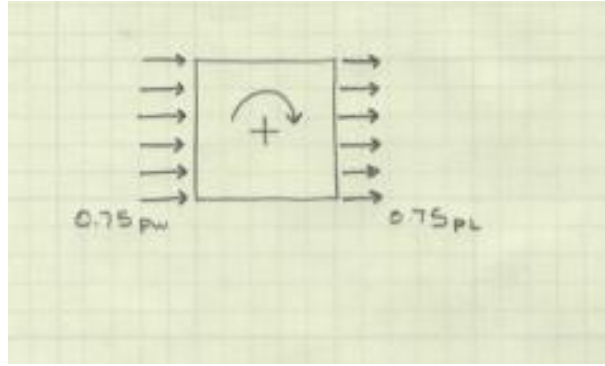


Figure 6

The torsional effects are noticeable in the two X-Direction shear walls. In combination 16, the walls are resisting the wind load with reactions to the left. When torsion is introduced, these shear walls must now account for the torsional forces as well as the direct ones. Figure 7 below shows the direction of the forces due to inherent torsion in blue arrows and the direction of the forces due to accidental torsional in red.

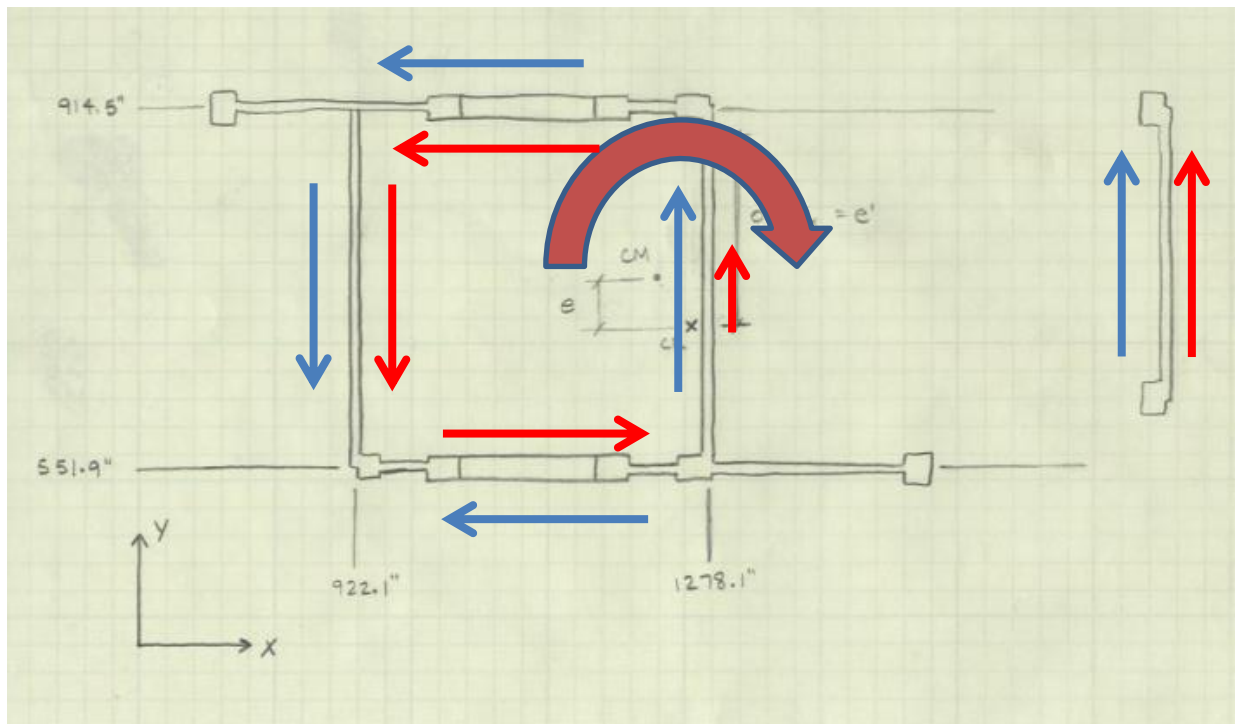


Figure 7

Shear wall 1-X has an additive effect while the forces in shear wall 2-X are in the opposite direction, causing the overall force in the wall to decrease. All of the shear walls have a predictable increase or decrease in the forces they resist based on their stiffness and distance from the center of rigidity. Shear wall 2-Y saw a decrease in force. This is slightly unexpected because the torsional moment increased. A

possible explanation of this reduction is the change in distance between the applied force and shear wall 2-Y, causing the wall to be less effective in resisting the torsional forces.

Seismic

Seismic torsional moments have the same effect on the shear in the lateral elements as shown in Figure 7. The difference between wind and seismic torsion is in the in which they are created. Earthquake loads are reliant on the dead load of the floors and their height above ground. The loads are applied at the center of mass of each floor, creating an eccentricity, inducing an inherent torsional moment. There can also be an accidental eccentricity caused by the displacement of the center of mass equal to 5% of the building width perpendicular to the direction of the applied forces. The magnitude of these forces is shown in the below table.

Floor	North-South Torsional Forces				East-West Torsional Forces				
	Lateral Force (k)	Inherent (ft-k)	Accidental (ft-k)	Total (ft-k)	Lateral Force (k)	Inherent (ft-k)	Accidental (ft-k)	Total (ft-k)	
14	153	590.4	1557.6	2148.0	153	-584.7	1107.1	1691.8	
13	136	524.2	1382.9	1907.1	136	-519.2	982.9	1502.1	
12	123	475.9	1255.4	1731.3	123	-471.3	892.3	1363.6	
11	111	428.6	1130.5	1559.1	111	-424.4	803.6	1228.0	
10	99	382.3	1008.5	1390.8	99	-378.6	716.8	1095.4	
9	87	337.2	889.4	1226.6	87	-333.9	632.2	966.1	
8	76	293.2	773.6	1066.8	76	-290.4	549.8	840.2	
7	65	250.7	661.2	911.9	65	-248.2	470.0	718.2	
6	54	209.5	552.7	762.2	54	-207.5	392.9	600.4	
5	44	170.0	448.5	618.5	44	-168.4	318.8	487.1	
4	34	132.3	349.1	481.4	34	-131.0	248.1	379.1	
3	25	96.8	255.2	352.0	25	-95.8	181.4	277.2	
2	16	63.7	168.2	231.9	16	-63.1	119.5	182.6	
1	9	34.0	89.8	123.8	9	-33.7	63.8	97.5	
Total				14511.4	Total				11429.5

The larger moment in the North-South Direction can be attributed to the larger dimension of the floor plan in the East-West Direction.

Strength Checks

The lateral forces at level 1 were calculated for each wall participating in resisting the lateral loads. These loads show the distribution of forces in the lateral system but a strength check of these elements must be performed to determine they are capable of resisting the applied forces. ACI 381-08 section 21.9.4.1 was used to determine the available strength of the shear walls. A table confirming the strengths of the walls as adequate is shown below.

Shear Wall Check at Level 1								
Wall Number	Direct Shear (k)	Torsional Shear (k)	Total Shear (k)	Vertical Reinforcing	Acv (in ²)	α_c	ρ_t	ϕV_n (k)
1-X	302.937	-42.59290452	260.3441	#5 @ 12"	5492	3	0.002585	1744
2-X	420.063	20.17891544	440.24192	#5 @ 12"	6800	3	0.002585	2159
1-Y	467.109	-233.5512568	233.55774	#5 @ 12"	5171	3	0.002585	1642
2-Y	475.558	6.161601403	481.7196	#5 @ 12"	5171	3	0.002585	1642
3-Y	265.54	179.0808457	444.62085	#5 @ 12"	3836	3	0.002585	1218

Hand calculations for the shear wall check can be found in Appendix C. The equation used accounts for the higher shear strength of walls with higher shear-to-moment ratios. For comparison forces, the available shear in the walls is greater than the applied force, confirming the design as adequate. Figure 8 on the next page shows the typical reinforcing of the shear wall core.

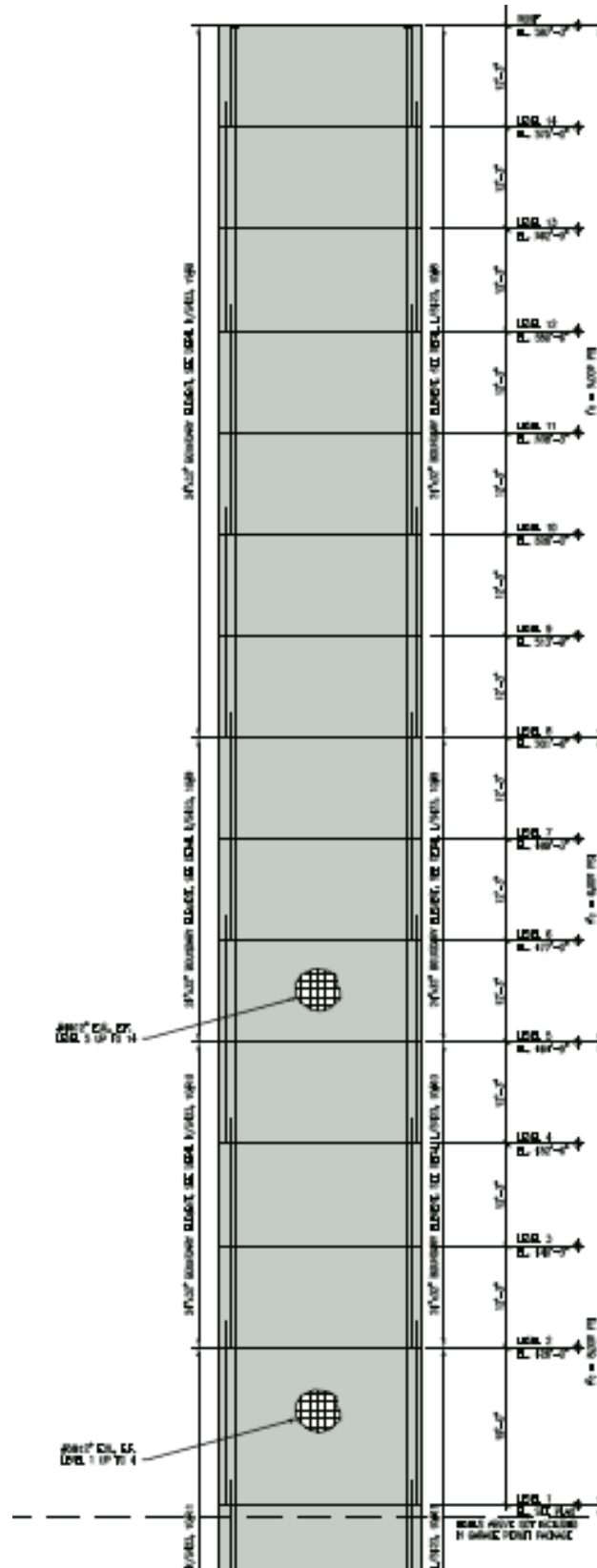


Figure 8

Drift and Displacement

Story drift and lateral displacement are not a concern in the strength design of a member but they need to be considered as a serviceability requirement. There are no wind drift requirements directly addressed in ASCE 7-10, H/400 has become a standard in engineering practice. The seismic drift requirement is based on the building occupancy category. Office Building-G is limited to a seismic drift of 2".

The allowable deflection and drift due to wind of Office Building G is:

Deflection: $\Delta = H/400 = 2139/400 = 5.35''$ **Drift:** $\Delta = H/400 = 147/400 = 0.3675''$

ETABS was used to calculate the story drift and the allowable limits compared to the actual maximum drifts are shown in the tables below. The drifts per floor are significantly less than the requirements necessary.

Wind Story Drift			
Story	Allowable Drift (in)	Actual Drift X (in)	Actual Drift Y (in)
14	0.36750	0.00062	0.00107
13	0.36750	0.00067	0.00114
12	0.36750	0.00070	0.00118
11	0.36750	0.00074	0.00122
10	0.36750	0.00078	0.00126
9	0.36750	0.00081	0.00129
8	0.36750	0.00083	0.00130
7	0.36750	0.00083	0.00127
6	0.36750	0.00082	0.00123
5	0.36750	0.00079	0.00116
4	0.36750	0.00072	0.00103
3	0.36750	0.00064	0.00090
2	0.36750	0.00053	0.00073
1	0.57000	0.00016	0.00025

Seismic Story Drift			
Story	Allowable Drift (in)	Actual Drift X (in)	Actual Drift Y (in)
14	2.0	0.000819	0.001162
13	2.0	0.000874	0.001195
12	2.0	0.00092	0.001214
11	2.0	0.000962	0.001226
10	2.0	0.000995	0.001228
9	2.0	0.001015	0.001215
8	2.0	0.001016	0.001183
7	2.0	0.000987	0.00112
6	2.0	0.000948	0.001046

5	2.0	0.000882	0.000947
4	2.0	0.000777	0.000812
3	2.0	0.000668	0.000672
2	2.0	0.000524	0.000503
1	2.0	0.000224	0.000195

The wind displacement in the North/South Direction (Y-Direction) was calculated by hand and was found to be 1.81". Hand calculations and an Excel spread sheet can be found in Appendix D. These results were compared to the ETABS results which yielded a slightly smaller deflection of 1.02". Both of these deflections are well below the serviceability requirement of 5.35" calculated above.

When determining the deflection due to wind, the wind limitation is a serviceability requirement which does not require the loads to be factored. Wind deflections are an estimate of how the building is actually going to perform so the actual deflection, not how loads inflated by factors of safety will deflect the building. In the seismic deflections, the building may fail if a drift greater than the allowable is formed so the factored loads are used to ensure a conservative estimate. The deflections per floor are shown in the table below.

Wind Displacement (Y-Direction)					
Floor	Story Height (in)	Height Above Ground (in)	Δ Flex (in)	Δ Shear (in)	Lateral Displacement (in)
14	147	2139	0.2404	0.0105	0.2509
13	147	1992	0.3851	0.0193	0.4045
12	147	1845	0.3019	0.0177	0.3196
11	147	1698	0.2319	0.0160	0.2480
10	147	1551	0.1740	0.0144	0.1884
9	147	1404	0.1269	0.0128	0.1397
8	147	1257	0.0894	0.0113	0.1006
7	147	1110	0.0550	0.0089	0.0639
6	147	963	0.0351	0.0075	0.0426
5	147	816	0.0208	0.0062	0.0270
4	147	669	0.0096	0.0043	0.0139
3	147	522	0.0044	0.0032	0.0076
2	147	375	0.0015	0.0022	0.0037
1	147	228	0.0004	0.0015	0.0019
Total Displacement =					1.8123

Overturning

Overturning moments are important forces to check because they can have greatly affect the design of the foundation. The moments are created by the lateral forces acting at each story level at some height above the foundation. These moments are transferred into axial loads which are transferred through the lateral members into the foundation. The moments create a couple, equal but opposite forces acting a certain distance from each other. The compressive forces push the foundation down in the same direction as gravity but the tension forces resist gravity. If these forces get too large the gravity forces will not have the necessary weight to keep the foundation from lifting up.

The magnitude of overturning moments can be estimated through multiplying the shear force at each story by the height of the story above the foundation. The load combination with the largest upward force on the foundation is the one which should be used to determine the story forces creating the overturning moment. Office Building-G experienced an upward reaction of 450k due to load combination 32, 0.9D + 1.0W. The estimated overturning moments are calculated below using the story shears from load case 32.

		Overturning Moment					
		North/South Wind		East/West Wind		Seismic	
Floor	Height	Story Force (k)	Moment (ft-k)	Story Force (k)	Moment (ft-k)	Story Force (k)	Moment (ft-k)
14	178	37.5	6675	29.97	5334.66	172	30616
13	166	74.37	12345.42	59.43	9865.38	307	50962
12	154	73.37	11298.98	58.63	9029.02	430	66220
11	142	72.31	10268.02	57.79	8206.18	540	76680
10	129	71.19	9183.51	56.89	7338.81	639	82431
9	117	69.98	8187.66	55.93	6543.81	726	84942
8	105	68.68	7211.4	54.89	5763.45	801	84105
7	93	67.27	6256.11	53.76	4999.68	866	80538
6	80	65.7	5256	52.51	4200.8	920	73600
5	68	63.95	4348.6	51.11	3475.48	964	65552
4	56	61.95	3469.2	49.51	2772.56	998	55888
3	44	59.58	2621.52	47.61	2094.84	1023	45012
2	31	56.63	1755.53	45.25	1402.75	1039	32209
1	19	63.17	1200.23	50.48	959.12	1048	19912
Total Overturning Moment (ft-k)			90077.18		71986.54		848667

Based on the dead load of the parking garage below the superstructure of Office Building-G, overturning is not a concern. The weight of the garage structure and the size of the supporting foundation will compensate for the upward force of 450k thus removing any possibility of uplift or overturning. This means that controlling design considerations for the foundations was a gravity only loading case.

Conclusion

Technical Report III is a comprehensive examination of the lateral force resisting system in Office Building-G. This report covered the existing superstructure, design loads and deflections, controlling load cases, load paths, strength checks, estimated drifts and displacements, and overturning effects. Upon completion of the report a broad understanding of the lateral system of Office Building-G was obtained.

All of the lateral loads applied to Office Building-G are resisted by an internal shear wall core. This core does not change in plan for the entire height of the building. The lower stories of the building have a higher concrete strength due to the story shear forces accumulating at the bottom of the building. Although it is commonly assumed that lateral forces are resisted by parallel elements, the analysis of Office Building-G yielded results which showed out of plane elements having a large effect on the parallel elements ability to resist the forces. This is due to the added stiffness that out-of-plane walls offer to the in-plane walls.

Using variation on ASCE 7-10 load cases, 43 load combinations were analyzed by ETABS to determine which controlled the design of the structure. In the North/South (Y) Direction, wind forces control the buildings shear forces due to the large tributary area the wind is acting on. For X-Direction shear stresses and the building deflection, load combinations with seismic forces controlled Office Building-G.

The lateral loads in Office Building-G were found to be distributed based on the relative stiffness of the elements. Shear walls with a larger relative stiffness would take a higher percentage of the load compared to elements with a lower relative stiffness. For torsional moments, the force distribution is based on relative stiffness as well as the distance the element is to the center of rigidity. Shear walls further from the CR are more effective at resisting torsional loads due to a larger moment arm.

Checks on strength, drift, deflection, and overturning moments were performed for Office Building-G. Based on the hand calculations performed to confirm the ETABS results, each of the requirements were met. The overturning analysis resulted in upward forces on the base reaction in the ETABS model but the values were not large enough to warrant a design consideration for overturning. The dead load of the parking garage and foundation below the upward force will compensate for this force.

Through a comparison between computer generated results and hand calculations it was determined that an accurate computer model was generated. With an accurate model created, hand calculations were able to confirm the as designed lateral structure has sufficient strength to resist loads applied by the controlling load cases. Through the techniques used in Technical Report III, the existing design was justified and determined to be sufficient.

Differences in hand calculations and computer models are inevitable when checking the design of a structure. Office Building-G is no different. The important part when comparing results is to understand where the difference between the two numbers is coming from and determining if this difference is appropriate based on the assumptions made. This section of the report will discuss any inconsistent hand calculations and computer model results suggest reasons for the difference between the two.

Period of Vibration:

A buildings period of vibration is a relationship between the stiffness and the dead load. The equation, $T = 2\pi \sqrt{\frac{M}{k}}$, reveals that as the mass of the building increases, the period will also increase. Inversely, if the stiffness of the structure increases, the period will decrease. ASCE 7-10 describes a method which can be used to estimate the period by limiting it to the product of C_u and T_a . The calculations for this can be found in Appendix ____ and yielded a result of 1.66s. The ETABS maximum modal period of vibration was found to be 2.33s, a number greater than $C_u T_a$, creating a scenario in which the structure is more flexible than the code permits.

Appendix A

Load Combinations							
ASCE Combination	ETABS Combination	Type	Case	Factor	Case Type	Sort ID	
1	1	ADD	DEAD	1.4	Static	1	
	2	2	ADD	DEAD	1.2	Static	2
		2		LIVE	1.6	Static	3
3	2		ROOF	0.5	Static	4	
	3	ADD	DEAD	1.2	Static	5	
	3		LIVE	1	Static	6	
	3		ROOF	1.6	Static	7	
	4	ADD	DEAD	1.2	Static	8	
	4		AUTOWIND	0.5	Static	9	
	4		ROOF	1.6	Static	10	
	5	ADD	DEAD	1.2	Static	11	
	5		AUTOWIND-2	0.5	Static	12	
	5		ROOF	1.6	Static	13	
	6	ADD	DEAD	1.2	Static	14	
	6		AUTOWIND-3	0.5	Static	15	
	6		ROOF	1.6	Static	16	
	7	ADD	DEAD	1.2	Static	17	
	7		AUTOWIND-4	0.5	Static	18	
	7		ROOF	1.6	Static	19	
	8	ADD	DEAD	1.2	Static	20	
	8		AUTOWIND-5	0.5	Static	21	
	8		ROOF	1.6	Static	22	
	9	ADD	DEAD	1.2	Static	23	
	9		AUTOWIND-6	0.5	Static	24	
	9		ROOF	1.6	Static	25	
	10	ADD	DEAD	1.2	Static	26	
	10		AUTOWIND-7	0.5	Static	27	
	10		ROOF	1.6	Static	28	
	11	ADD	DEAD	1.2	Static	29	
	11		AUTOWIND-8	0.5	Static	30	
	11		ROOF	1.6	Static	31	
	12	ADD	DEAD	1.2	Static	32	
	12		AUTOWIND-9	0.5	Static	33	
	12		ROOF	1.6	Static	34	
	13	ADD	DEAD	1.2	Static	35	
	13		AUTOWIND-10	0.5	Static	36	
	13		ROOF	1.6	Static	37	
	14	ADD	DEAD	1.2	Static	38	
	14		AUTOWIND-11	0.5	Static	39	
	14		ROOF	1.6	Static	40	
	15	ADD	DEAD	1.2	Static	41	
	15		AUTOWIND-12	0.5	Static	42	
	15		ROOF	1.6	Static	43	

ASCE Combination	ETABS Combination	Type	Case	Factor	Case Type	Sort ID
4	16	ADD	DEAD	1.2	Static	44
	16		AUTOWIND	1	Static	45
	16		ROOF	0.5	Static	46
	16		LIVE	1	Static	47
	17	ADD	DEAD	1.2	Static	48
	17		AUTOWIND-2	1	Static	49
	17		ROOF	0.5	Static	50
	17		LIVE	1	Static	51
	18	ADD	DEAD	1.2	Static	52
	18		ROOF	0.5	Static	53
	18		LIVE	1	Static	54
	18		AUTOWIND-3	1	Static	55
	19	ADD	DEAD	1.2	Static	56
	19		ROOF	0.5	Static	57
	19		LIVE	1	Static	58
	19		AUTOWIND-4	1	Static	59
	20	ADD	DEAD	1.2	Static	60
	20		ROOF	0.5	Static	61
	20		LIVE	1	Static	62
	20		AUTOWIND-5	1	Static	63
	21	ADD	DEAD	1.2	Static	64
	21		ROOF	0.5	Static	65
	21		LIVE	1	Static	66
	21		AUTOWIND-6	1	Static	67
	22	ADD	DEAD	1.2	Static	68
	22		ROOF	0.5	Static	69
	22		LIVE	1	Static	70
	22		AUTOWIND-7	1	Static	71
	23	ADD	DEAD	1.2	Static	72
	23		ROOF	0.5	Static	73
	23		LIVE	1	Static	74
	23		AUTOWIND-8	1	Static	75
	24	ADD	DEAD	1.2	Static	76
	24		ROOF	0.5	Static	77
	24		LIVE	1	Static	78
	24		AUTOWIND-9	1	Static	79
	25	ADD	DEAD	1.2	Static	80
	25		ROOF	0.5	Static	81
	25		LIVE	1	Static	82
	25		AUTOWIND-10	1	Static	83
	26	ADD	DEAD	1.2	Static	84
	26		ROOF	0.5	Static	85
	26		LIVE	1	Static	86
	26		AUTOWIND-11	1	Static	87
	27	ADD	DEAD	1.2	Static	88
	27		ROOF	0.5	Static	89
	27		LIVE	1	Static	90
	27		AUTOWIND-12	1	Static	91

ASCE Combination	ETABS Combination	Type	Case	Factor	CaseType	SortID
6	28	ADD	DEAD	0.9	Static	92
	28		AUTOWIND	1	Static	93
	29	ADD	DEAD	0.9	Static	94
	29		AUTOWIND-2	1	Static	95
	30	ADD	DEAD	0.9	Static	96
	30		AUTOWIND-3	1	Static	97
	31	ADD	DEAD	0.9	Static	98
	31		AUTOWIND-4	1	Static	99
	32	ADD	DEAD	0.9	Static	100
	32		AUTOWIND-5	1	Static	101
	33	ADD	DEAD	0.9	Static	102
	33		AUTOWIND-6	1	Static	103
	34	ADD	DEAD	0.9	Static	104
	34		AUTOWIND-7	1	Static	105
	35	ADD	DEAD	0.9	Static	106
	35		AUTOWIND-8	1	Static	107
	36	ADD	DEAD	0.9	Static	108
	36		AUTOWIND-9	1	Static	109
	37	ADD	DEAD	0.9	Static	110
	37		AUTOWIND-10	1	Static	111
	38	ADD	DEAD	0.9	Static	112
	38		AUTOWIND-11	1	Static	113
	39	ADD	DEAD	0.9	Static	114
	39		AUTOWIND-12	1	Static	115
5	40	ADD	DEAD	1.2	Static	116
	40		LIVE	1	Static	117
	40		QUAKEEX	1	Static	118
	41	ADD	DEAD	1.2	Static	119
	41		LIVE	1	Static	120
	41		QUAKEY	1	Static	121
7	42	ADD	DEAD	0.9	Static	122
	42		QUAKEEX	1	Static	123
	43	ADD	DEAD	0.9	Static	124
	43		QUAKEY	1	Static	125

Wind Forces

Case	Story	Diaphragm	FX (k)	FY (k)	MZ (k-in)
AUTOWIND	STORY14	D1	29.97	0	0
AUTOWIND	STORY13	D1	59.43	0	0
AUTOWIND	STORY12	D1	58.63	0	0
AUTOWIND	STORY11	D1	57.79	0	0
AUTOWIND	STORY10	D1	56.89	0	0
AUTOWIND	STORY9	D1	55.93	0	0
AUTOWIND	STORY8	D1	54.89	0	0
AUTOWIND	STORY7	D1	53.76	0	0
AUTOWIND	STORY6	D1	52.51	0	0
AUTOWIND	STORY5	D1	51.11	0	0
AUTOWIND	STORY4	D1	49.51	0	0
AUTOWIND	STORY3	D1	47.61	0	0
AUTOWIND	STORY2	D1	45.25	0	0
AUTOWIND	STORY1	D1	50.48	0	0
AUTOWIND-2	STORY14	D1	0	50	0
AUTOWIND-2	STORY13	D1	0	99.16	0
AUTOWIND-2	STORY12	D1	0	97.83	0
AUTOWIND-2	STORY11	D1	0	96.42	0
AUTOWIND-2	STORY10	D1	0	94.92	0
AUTOWIND-2	STORY9	D1	0	93.31	0
AUTOWIND-2	STORY8	D1	0	91.58	0
AUTOWIND-2	STORY7	D1	0	89.69	0
AUTOWIND-2	STORY6	D1	0	87.6	0
AUTOWIND-2	STORY5	D1	0	85.27	0
AUTOWIND-2	STORY4	D1	0	82.6	0
AUTOWIND-2	STORY3	D1	0	79.44	0
AUTOWIND-2	STORY2	D1	0	75.5	0
AUTOWIND-2	STORY1	D1	0	84.22	0
AUTOWIND-3	STORY14	D1	22.48	0	4941.017
AUTOWIND-3	STORY13	D1	44.57	0	9798.052
AUTOWIND-3	STORY12	D1	43.97	0	9666.745
AUTOWIND-3	STORY11	D1	43.34	0	9527.619
AUTOWIND-3	STORY10	D1	42.67	0	9379.448
AUTOWIND-3	STORY9	D1	41.95	0	9220.679
AUTOWIND-3	STORY8	D1	41.17	0	9049.281
AUTOWIND-3	STORY7	D1	40.32	0	8862.53
AUTOWIND-3	STORY6	D1	39.38	0	8656.629
AUTOWIND-3	STORY5	D1	38.33	0	8426.035

Case	Story	Diaphragm	FX (k)	FY (k)	MZ (k-in)
AUTOWIND-7	STORY14	D1	22.48	-37.5	0
AUTOWIND-7	STORY13	D1	44.57	-74.37	0
AUTOWIND-7	STORY12	D1	43.97	-73.37	0
AUTOWIND-7	STORY11	D1	43.34	-72.31	0
AUTOWIND-7	STORY10	D1	42.67	-71.19	0
AUTOWIND-7	STORY9	D1	41.95	-69.98	0
AUTOWIND-7	STORY8	D1	41.17	-68.68	0
AUTOWIND-7	STORY7	D1	40.32	-67.27	0
AUTOWIND-7	STORY6	D1	39.38	-65.7	0
AUTOWIND-7	STORY5	D1	38.33	-63.95	0
AUTOWIND-7	STORY4	D1	37.13	-61.95	0
AUTOWIND-7	STORY3	D1	35.71	-59.58	0
AUTOWIND-7	STORY2	D1	33.94	-56.63	0
AUTOWIND-7	STORY1	D1	37.86	-63.17	0
AUTOWIND-8	STORY14	D1	22.48	37.5	0
AUTOWIND-8	STORY13	D1	44.57	74.37	0
AUTOWIND-8	STORY12	D1	43.97	73.37	0
AUTOWIND-8	STORY11	D1	43.34	72.31	0
AUTOWIND-8	STORY10	D1	42.67	71.19	0
AUTOWIND-8	STORY9	D1	41.95	69.98	0
AUTOWIND-8	STORY8	D1	41.17	68.68	0
AUTOWIND-8	STORY7	D1	40.32	67.27	0
AUTOWIND-8	STORY6	D1	39.38	65.7	0
AUTOWIND-8	STORY5	D1	38.33	63.95	0
AUTOWIND-8	STORY4	D1	37.13	61.95	0
AUTOWIND-8	STORY3	D1	35.71	59.58	0
AUTOWIND-8	STORY2	D1	33.94	56.63	0
AUTOWIND-8	STORY1	D1	37.86	63.17	0
AUTOWIND-9	STORY14	D1	16.87	-28.15	14034.148
AUTOWIND-9	STORY13	D1	33.46	-55.82	27829.761
AUTOWIND-9	STORY12	D1	33.01	-55.08	27456.807
AUTOWIND-9	STORY11	D1	32.54	-54.28	27061.64
AUTOWIND-9	STORY10	D1	32.03	-53.44	26640.787
AUTOWIND-9	STORY9	D1	31.49	-52.53	26189.829
AUTOWIND-9	STORY8	D1	30.9	-51.56	25703
AUTOWIND-9	STORY7	D1	30.26	-50.49	25172.564
AUTOWIND-9	STORY6	D1	29.56	-49.32	24587.738
AUTOWIND-9	STORY5	D1	28.77	-48.01	23932.772

Carl Hubben

Advisor: Dr. Ali Memari

AUTOWIND-3	STORY4	D1	37.13	0	8162.105
AUTOWIND-3	STORY3	D1	35.71	0	7850.073
AUTOWIND-3	STORY2	D1	33.94	0	7460.692
AUTOWIND-3	STORY1	D1	37.86	0	8322.33
AUTOWIND-4	STORY14	D1	22.48	0	-4941.017
AUTOWIND-4	STORY13	D1	44.57	0	-9798.052
AUTOWIND-4	STORY12	D1	43.97	0	-9666.745
AUTOWIND-4	STORY11	D1	43.34	0	-9527.619
AUTOWIND-4	STORY10	D1	42.67	0	-9379.448
AUTOWIND-4	STORY9	D1	41.95	0	-9220.679
AUTOWIND-4	STORY8	D1	41.17	0	-9049.281
AUTOWIND-4	STORY7	D1	40.32	0	-8862.53
AUTOWIND-4	STORY6	D1	39.38	0	-8656.629
AUTOWIND-4	STORY5	D1	38.33	0	-8426.035
AUTOWIND-4	STORY4	D1	37.13	0	-8162.105
AUTOWIND-4	STORY3	D1	35.71	0	-7850.073
AUTOWIND-4	STORY2	D1	33.94	0	-7460.692
AUTOWIND-4	STORY1	D1	37.86	0	-8322.33
AUTOWIND-5	STORY14	D1	0	37.5	13754.563
AUTOWIND-5	STORY13	D1	0	74.37	27275.342
AUTOWIND-5	STORY12	D1	0	73.37	26909.819
AUTOWIND-5	STORY11	D1	0	72.31	26522.524
AUTOWIND-5	STORY10	D1	0	71.19	26110.055
AUTOWIND-5	STORY9	D1	0	69.98	25668.08
AUTOWIND-5	STORY8	D1	0	68.68	25190.951
AUTOWIND-5	STORY7	D1	0	67.27	24671.081
AUTOWIND-5	STORY6	D1	0	65.7	24097.906
AUTOWIND-5	STORY5	D1	0	63.95	23455.988
AUTOWIND-5	STORY4	D1	0	61.95	22721.274
AUTOWIND-5	STORY3	D1	0	59.58	21852.652
AUTOWIND-5	STORY2	D1	0	56.63	20768.714
AUTOWIND-5	STORY1	D1	0	63.17	23167.3
AUTOWIND-6	STORY14	D1	0	37.5	-13754.563
AUTOWIND-6	STORY13	D1	0	74.37	-27275.342
AUTOWIND-6	STORY12	D1	0	73.37	-26909.819
AUTOWIND-6	STORY11	D1	0	72.31	-26522.524
AUTOWIND-6	STORY10	D1	0	71.19	-26110.055
AUTOWIND-6	STORY9	D1	0	69.98	-25668.08
AUTOWIND-6	STORY8	D1	0	68.68	-25190.951
AUTOWIND-6	STORY7	D1	0	67.27	-24671.081
AUTOWIND-6	STORY6	D1	0	65.7	-24097.906

AUTOWIND-9	STORY4	D1	27.87	-46.5	23183.124
AUTOWIND-9	STORY3	D1	26.81	-44.73	22296.846
AUTOWIND-9	STORY2	D1	25.48	-42.51	21190.874
AUTOWIND-9	STORY1	D1	28.42	-47.42	23638.215
AUTOWIND-10	STORY14	D1	16.87	-28.15	-14034.148
AUTOWIND-10	STORY13	D1	33.46	-55.82	-27829.761
AUTOWIND-10	STORY12	D1	33.01	-55.08	-27456.807
AUTOWIND-10	STORY11	D1	32.54	-54.28	-27061.64
AUTOWIND-10	STORY10	D1	32.03	-53.44	-26640.787
AUTOWIND-10	STORY9	D1	31.49	-52.53	-26189.829
AUTOWIND-10	STORY8	D1	30.9	-51.56	-25703
AUTOWIND-10	STORY7	D1	30.26	-50.49	-25172.564
AUTOWIND-10	STORY6	D1	29.56	-49.32	-24587.738
AUTOWIND-10	STORY5	D1	28.77	-48.01	-23932.772
AUTOWIND-10	STORY4	D1	27.87	-46.5	-23183.124
AUTOWIND-10	STORY3	D1	26.81	-44.73	-22296.846
AUTOWIND-10	STORY2	D1	25.48	-42.51	-21190.874
AUTOWIND-10	STORY1	D1	28.42	-47.42	-23638.215
AUTOWIND-11	STORY14	D1	16.87	28.15	14034.148
AUTOWIND-11	STORY13	D1	33.46	55.82	27829.761
AUTOWIND-11	STORY12	D1	33.01	55.08	27456.807
AUTOWIND-11	STORY11	D1	32.54	54.28	27061.64
AUTOWIND-11	STORY10	D1	32.03	53.44	26640.787
AUTOWIND-11	STORY9	D1	31.49	52.53	26189.829
AUTOWIND-11	STORY8	D1	30.9	51.56	25703
AUTOWIND-11	STORY7	D1	30.26	50.49	25172.564
AUTOWIND-11	STORY6	D1	29.56	49.32	24587.738
AUTOWIND-11	STORY5	D1	28.77	48.01	23932.772
AUTOWIND-11	STORY4	D1	27.87	46.5	23183.124
AUTOWIND-11	STORY3	D1	26.81	44.73	22296.846
AUTOWIND-11	STORY2	D1	25.48	42.51	21190.874
AUTOWIND-11	STORY1	D1	28.42	47.42	23638.215
AUTOWIND-12	STORY14	D1	16.87	28.15	-14034.148
AUTOWIND-12	STORY13	D1	33.46	55.82	-27829.761
AUTOWIND-12	STORY12	D1	33.01	55.08	-27456.807
AUTOWIND-12	STORY11	D1	32.54	54.28	-27061.64
AUTOWIND-12	STORY10	D1	32.03	53.44	-26640.787
AUTOWIND-12	STORY9	D1	31.49	52.53	-26189.829
AUTOWIND-12	STORY8	D1	30.9	51.56	-25703
AUTOWIND-12	STORY7	D1	30.26	50.49	-25172.564
AUTOWIND-12	STORY6	D1	29.56	49.32	-24587.738

Carl Hubben

Advisor: Dr. Ali Memari

AUTOWIND-6	STORY5	D1	0	63.95	-23455.988
AUTOWIND-6	STORY4	D1	0	61.95	-22721.274
AUTOWIND-6	STORY3	D1	0	59.58	-21852.652
AUTOWIND-6	STORY2	D1	0	56.63	-20768.714
AUTOWIND-6	STORY1	D1	0	63.17	-23167.3

AUTOWIND-12	STORY5	D1	28.77	48.01	-23932.772
AUTOWIND-12	STORY4	D1	27.87	46.5	-23183.124
AUTOWIND-12	STORY3	D1	26.81	44.73	-22296.846
AUTOWIND-12	STORY2	D1	25.48	42.51	-21190.874
AUTOWIND-12	STORY1	D1	28.42	47.42	-23638.215

Earthquake Forces				
Case	Story	Diaphragm	FX (k)	FY (k)
QUAKEX	STORY14	D1	153	0
QUAKEX	STORY13	D1	135	0
QUAKEX	STORY12	D1	123	0
QUAKEX	STORY11	D1	111	0
QUAKEX	STORY10	D1	99	0
QUAKEX	STORY9	D1	87	0
QUAKEX	STORY8	D1	76	0
QUAKEX	STORY7	D1	65	0
QUAKEX	STORY6	D1	54	0
QUAKEX	STORY5	D1	44	0
QUAKEX	STORY4	D1	34	0
QUAKEX	STORY3	D1	25	0
QUAKEX	STORY2	D1	16	0
QUAKEX	STORY1	D1	9	0
QUAKEY	STORY14	D1	0	153
QUAKEY	STORY13	D1	0	135
QUAKEY	STORY12	D1	0	123
QUAKEY	STORY11	D1	0	111
QUAKEY	STORY10	D1	0	99
QUAKEY	STORY9	D1	0	87
QUAKEY	STORY8	D1	0	76
QUAKEY	STORY7	D1	0	65
QUAKEY	STORY6	D1	0	54
QUAKEY	STORY5	D1	0	44
QUAKEY	STORY4	D1	0	34
QUAKEY	STORY3	D1	0	25
QUAKEY	STORY2	D1	0	16
QUAKEY	STORY1	D1	0	9

CONFIRMING ETABS	WIND LOADS	1/1
------------------	------------	-----

MAXIMUM X-DIRECTION FORCE
59.43 K

- CASE 1 OF FIGURE 27.4-8

OFFICE BUILDING-G VALUES

$P_{WX} @ 178.3' = 24.15 \text{ psf}$

$P_{LX} @ 178.3' = -17.2 \text{ psf}$

WINDWARD :

$24.15(12.25)(145) = 36 \text{ K}$

LEEWARD :

$-17.2(12.25)(145) = 25.7$

TOTAL = 61.7 \approx 59.43 \therefore Good

MAXIMUM Y-DIRECTION FORCE = 99.16 K

- CASE 1 OF FIGURE 27.4-8 (Y-DIRECTION)

OFFICE BUILDING-G VALUES

$P_{WX} = 24.45 \text{ psf}$; $P_{LX} = -15.22$

WINDWARD :

$24.45(12.25)(209) = 61.1 \text{ K}$

LEEWARD :

$-15.22(12.25)(209) = 38.0$

TOTAL = 99.1 K \approx 99.16 K \therefore Good

Appendix B

DETERMINING	RELATIVE STIFFNESS
WALL 3-Y ETABS SINGLE ELEMENT RESULTS	
$P = 1000 \text{ K}$	$K = P/U = 1000 / 0.1463 = 6835.3 \text{ K/in}$
$U = 0.1463 \text{ in}$	
WALL 2-Y ETABS	
$P = 1000$	$K = P/U = 1000 / 0.0816 = 12254.9$
$U = 0.0816$	
WALL 1-Y ETABS	
$P = 1000$	$K = 1000 / 0.0830 = 12048.2$
$U = 0.0830$	
	$\Sigma K = 31138.4$
$K_{REL} = \frac{K_i}{\Sigma K}$	
$K_{3y} = \frac{6835.3}{31138.4} = 0.220$	} $\Sigma K_e \approx 1.0$
$K_{2y} = \frac{12254.9}{31138.4} = 0.394$	
$K_{1y} = \frac{12048.2}{31138.4} = 0.387$	

WALL 1-X ETABS
 $P = 1000K$
 $U = 0.0601$
 $K = \frac{1000}{0.0601} = 16638.9$

WALL 2-X ETABS
 $P = 1000K$
 $U = 0.0434$
 $K = \frac{1000}{0.0434} = 23041.5$

$\Sigma K = 39680.4$

$K_{1x} = \frac{16638.9}{39680.4} = 0.419$
 $K_{2x} = \frac{23041.5}{39680.4} = 0.581$ } $\Sigma K_x = 1.0$

CENTER OF RIGIDITY:

$\bar{y} = \frac{\Sigma K_x y_i}{\Sigma K_x} = \frac{16638.9(76.2) + 23041.5(47.2)}{39680.4} = 59.36'$

$59.36 \times 12 = 712.3 \text{ in}$

$\bar{x} = \frac{\Sigma K_y x_i}{\Sigma K_y} = \frac{12048.2(76.8) + 12254.9(106.5) + 6835.3(144.8)}{31138.4} = 103.4$

$103.4 \times 12 = 1241.0 \text{ in}$

Appendix C

TORSIONAL SHEAR (3-Y)

LOAD CASE : 17 (1.2D + 0.5L₂ + 1.0L + 1.0W)

DIRECT SHEAR = STORY FORCE x REL STIFFNESS

$$\text{TORSIONAL SHEAR} = \frac{k_j d_j e^P}{\sum k_i d_i^2}$$

WALL 3-Y @ LEVEL 1 (N/S)

STORY SHEAR = 905.65

CR : X = 1269" , Y = 686.8" e_x = 46.4"

CM : X = 1222.6" , Y = 732.75"

d_j = 1738 - 1269 = 469"

$$\sum k_i d_i^2 = 166384(914.5 - 686.8)^2 + 230915(608.9 - 686.8)^2$$

$$= 1.003 \times 10^9$$

DIRECT SHEAR = 1207 (.22) = 265.54^k

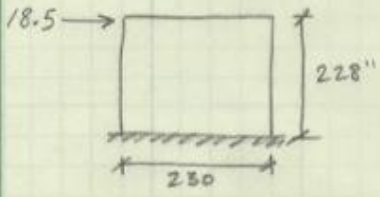
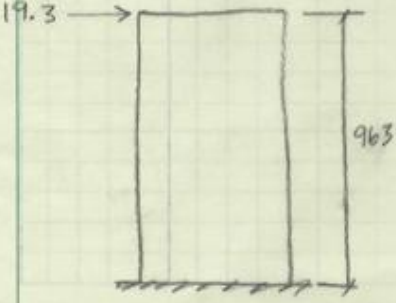
$$\text{TORSIONAL SHEAR} = \frac{6835(469)(46.4)1207}{1.003 \times 10^9} = 179^k$$

TOTAL = 445

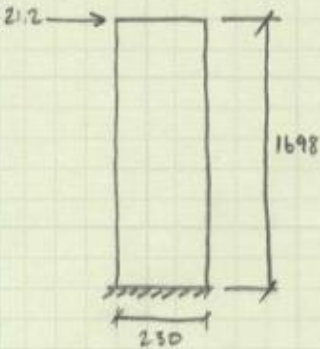
SHEAR STRENGTH (3-Y)

$$\phi V_n = \phi A_{cv} [\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y]$$
$$\phi = 0.75$$
$$A_{cv} = 3836. \text{ in}^2$$
$$\sqrt{f'_c} = \sqrt{8000}/1000 = 0.0894$$
$$\lambda = 1.0$$
$$\alpha_c = 3.0 \text{ FOR } H/L = 1.5 \quad 209/298 = 0.694$$
$$\rho_t = \frac{A_v}{S_n} = \frac{0.31}{12(10)} = 0.00258$$
$$\phi V_n = 0.75 (3836) [3.0(1.0)0.0894 + 0.00258(60)]$$
$$\phi V_n = 1217 \text{ K}$$

Appendix D

DISPLACEMENT	(WALL 3-Y)
$\Delta = \Delta_{flex} + \Delta_{shear}$ $\Delta = \frac{Ph^3}{3E_c I} + \frac{2.78Ph}{E_r A}$	$E_c = 33 \times 145^{1.5} \sqrt{f'_c}$ $E_r = 0.4 E_c$ $A = lt$ $I = \frac{bh^3}{12}$
$E_c = 5153.6 \text{ ksi}$ $E_r = 2061.4 \text{ ksi}$ $A = 230(10) = 2300 \text{ in}^2$ $I = \frac{24(294^3)}{12} - \frac{14(230^3)}{12} = 5.0824 \times 10^7 - 1.9195 \times 10^7 = 3.663 \times 10^7$	
<p>WALL 3-Y RELIEVES 22% OF LOAD BASED ON RELATIVE STIFFNESS</p>	
<p>STORY 1 DISPLACEMENT</p>	
	$\Delta_{flex} = \frac{Ph^3}{3E_c I} = \frac{18.5(228^3)}{3(5153.6)(3.66 \times 10^7)}$ $= 0.00388$ $\Delta_{shear} = \frac{2.78Ph}{E_r A} = \frac{2.78(18.5)(228)}{2061.4(2300)}$ $= 0.002473$ $\Delta = 0.00286 \text{ in}$
<p>STORY 6 DISPLACEMENT</p>	
	$\Delta_{flex} = \frac{Ph^3}{3E_c I} = \frac{19.3(963^3)}{3(4463.2)(3.66 \times 10^7)}$ $= 0.0351$ $\Delta_{shear} = \frac{2.78(19.3)(963)}{17953(2300)}$ $= 0.01258$ $\Delta = 0.0477 \text{ in}$

STORY II DISPLACEMENT



$$\Delta_{flex} = \frac{Ph^3}{3E_c I} = \frac{21.2(1698^3)}{3(4074.3)(3.66 \times 10^9)}$$

$$= 0.232 \text{ in}$$

$$\Delta_{shear} = \frac{2.78(21.2)(1698)}{1629.7(2300)} = 0.0267$$

$$\Delta = 0.259 \text{ in}$$

ALLOWABLE DRIFT = $\frac{1}{400} = \frac{2139}{400} = 5.3475'' \gg 1.9''$

∴ OK

ETAB RESULT FOR DRIFT in Y-DIRECTION

$$\Delta = 0.8391 \text{ in}$$