AE SENIOR THESIS 2012-13

TECHNICAL REPORT 3

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India

The Optimus

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Executive Summary

The third technical report consists of analysis of the lateral force resisting system of The Optimus. The 17 story, 252 ft tall reinforced concrete building is supported by reinforced concrete 8” flat slabs on reinforced concrete columns. The flat slabs contain 8” deep drop panels around the columns. The building facade is supported at the perimeter of the floor slabs. The 1st to the 3rd level of the building consist of parking spaces and 5th to 17th level consists of office spaces. The roof of the building houses a garden and a gymnasium. The structure of the building has been optimized and designed to fit with the architecture of the building.

The lateral force resisting system is an important part of the structural system that helps stabilize the building and supports the building form wind loads and earthquake effects. The lateral system of The Optimus consists of a reinforced shear wall core that spans from the base of the building to the roof. The Optimus contains 14 shear walls spanning North-South direction and 3 shear walls spanning in East-West direction. All the walls are concentrated around the elevator shaft and stairwells where continuity can be achieved in the wall system.

The lateral system analysis in this report starts with calculation of wind loads using ASCE 7-10 Directional procedure. The basic wind speed used to calculate wind loads is acquired from the weather data of the existing location of the building. Seismic loads are calculated using Equivalent Lateral Force Procedure in ASCE 7-10 and by relocating the building to a location with similar seismic behavior. The building was relocated because ASCE 7-10 does not contain seismic information for the existing location of the building.

The wind loads were distributed vertically to determine force at each story and the force at a typical story was distributed horizontally. Similarly, vertical distribution of seismic base shear was calculated. The horizontal distribution of wind and seismic loads was carried out using stiffness values of each shear wall. It was found that two shear walls were critical in carrying maximum shear and the largest wall was checked for shear and overturning moment. Also. the stiffness of every story was used to calculate drifts due to seismic and wind loads due to cracked as and un-cracked section of shear walls. It was found that the drifts at the top story were under the limiting deflection defined in ASCE 7-10.

Although finite element modeling of the building was performed, the report was not able to acquire appropriate results from the computer model. This was because of lack of simplification of the building elements. It was found that simplification of floor diaphragms to rigid-membrane elements and proper mesh size would help simplify the model for future computer analysis.

Building Introduction

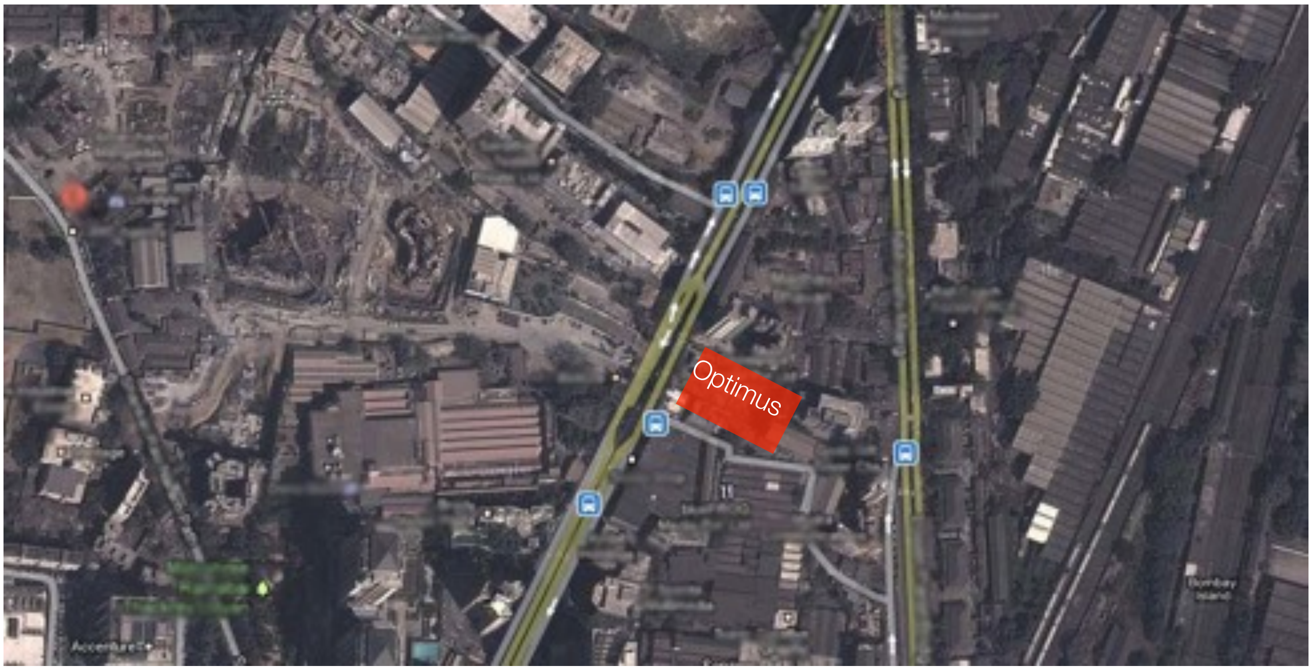


Figure 1 Aerial map from Google.com showing the location of the building site.

The Optimus is a new building rising in the economic capital of India. The building is owned by Lodha Group, one of the prime developers in the city and is designed by Pei Cobb Freed and Partners Architects LLP, New York. It is part of the large redevelopment project that used to be a textile mill. The project consists of residential buildings, offices, parking garages and retail spaces. The Optimus is mainly an office building designed to cater the needs of small and medium size companies who look for office spaces in the business district of the city. It is 17 stories tall with 4 stories of parking and ground floor retail.

The design of The Optimus is functional and elegant. Although the building is located in tight boundaries it makes efficient use of space by expanding vertically. To cater the requirements of the offices, it offers open and customizable floor space. The spacing of the structural and architectural elements offer flexible partitioning for office areas. The building provides recreational facilities that include a gymnasium, roof garden, green balcony spaces at every floor and a garden at the lobby area. The 2 basements and first 3 levels are dedicated to parking with 5th level as garden, lobby and office. The office spaces start from 6 to 17th story and 18th story contains a roof garden.

Figure 2 Rendering showing roof garden

Just like the interior, the exterior of the building is efficient in utilizing the available resources at the same time maintaining its aesthetic qualities. The envelope of the building is designed to fit into the fabric of the city which also becomes an important architectural feature of the building. Three kinds of materials decorate the facade: metal, stone and plants. The north facade, that faces residential apartments, provides a view of green wall to the apartment buildings and the south facade provides a panoramic view of the city to all the office spaces.

Figure 3 Rendering of the building entrance

The south facade is dominated by a bold and modern look with metal cladding and windows offset inside to provide solar shading in the interior. The front facade facing the main street shows a play of all materials on the facade: stone, metal and green wall giving a rich look to the building front.

The structure of the building complements the architectural features. A successful building is achieved when its structure and architecture integrate without compromise. The structure plays an important role in facilitating the show of different materials on the facade and in achieving an open floor plan. Most of the columns in the floor area are pushed to the exterior so that interior is open. The facade forms the skin of the building concealing the columns and overall structural system of the building. This facilitates different architectural features in the exterior and interior of the building.

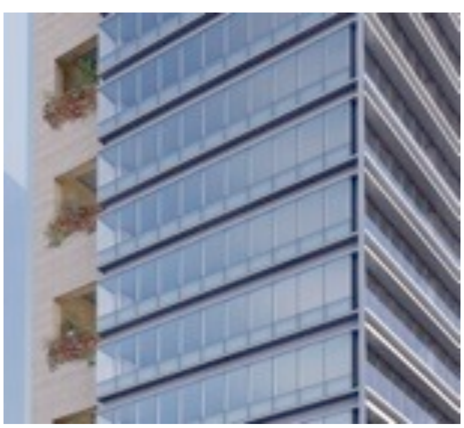


Figure 4 Rendering of the building facade

Structural System Overview

Structural system of The Optimus is designed by Leslie E. Robertson Associates (Mumbai). It has been optimized to increase floor space area, to celebrate the architecture and economize the overall cost of the building. In order to achieve these goals, reinforced concrete was chosen as a prime material to design the structural members. The properties of concrete allow fluidity in design. It also facilitates design changes during construction. Concrete is a preferred material over steel for construction in India because it is readily available. Also, the labor for concrete based construction is cheaper as compared to steel The structural system of the building consists of flat slabs supported by columns and shear walls that sit on a mat foundation.

Foundations

The geotechnical investigation report was performed by Shekhar Vaishampayan Geotechnical Consultants Pvt. Ltd. and special care was taken to avoid disturbances to adjacent buildings as the site is tightly surrounded by factories and residential buildings. As the building has two basement floors, the geotechnical investigation included excavation qualities of the site. The quality and the bearing capacity of the soil was determined.

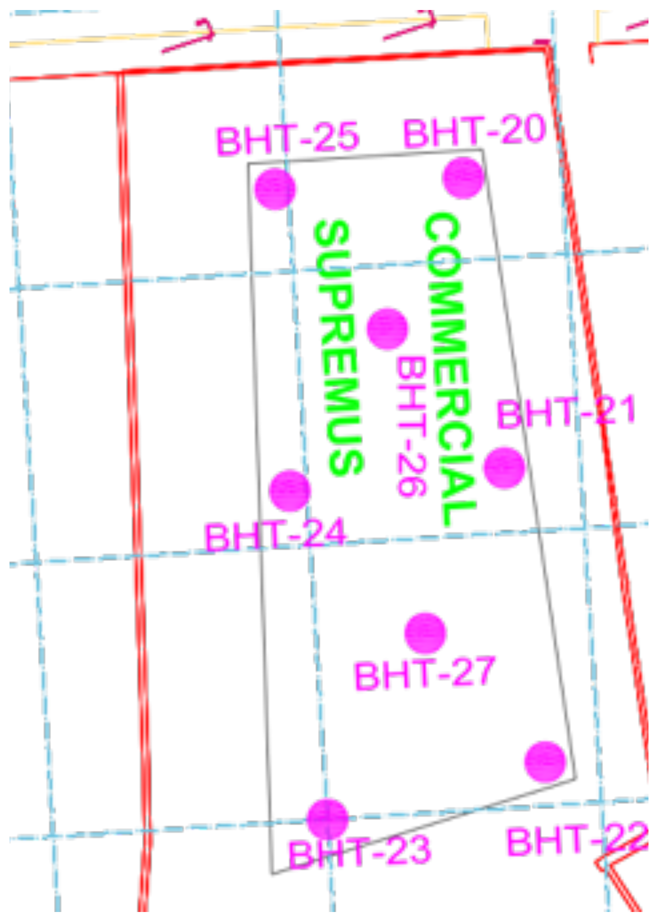


Figure 5: Test Boring Plan

In order to perform the analysis eight boreholes were drilled and soil samples were collected and analyzed. It was discovered that soil properties consisted of filled up soil, medium to stiff clay, weathered rock and highly to slightly weathered tuff. The minimum depth of excavation was determined to be 12.5 m / 41 feet below ground level. The basement raft was decided to be placed 10 m / 33 ft below ground level. Lateral pressures due to soil and water table was determined and basement retaining walls were designed to support these pressures. The ground water table was determined to be present at a depth of 1.00 m / 3.3 ft below ground. This was a conservative figure chosen by the geotechnical consultant to account for the built of water pressures during heavy monsoon season in the city.

Gravity Framing System

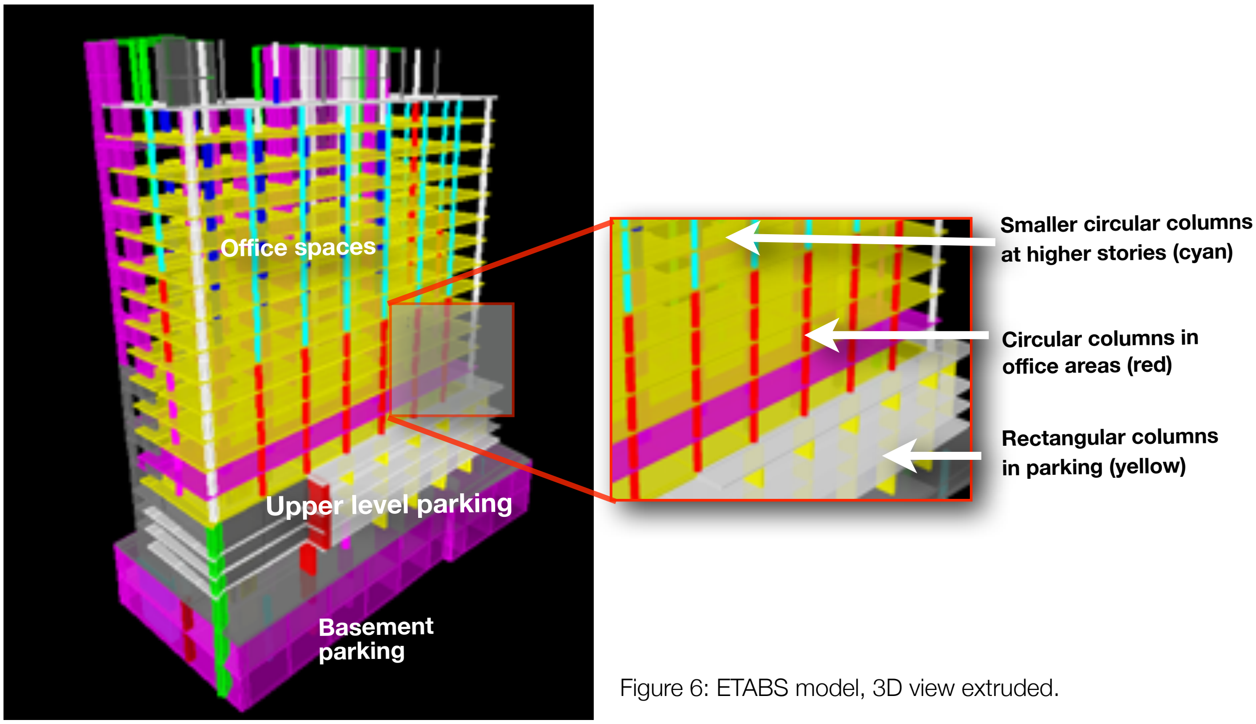


Figure 6: ETABS model, 3D view extruded.

The reinforced concrete framing system of The Optimus is developed to fit different types of floor spaces from the basement to top floor. The column, beam and slab system are chosen to fit with the architecture of the building as well as to act as architectural elements.

Architecture and structural system integration is seen in the columns of the building that change its cross sectional properties and layout as the space progresses from basement to the top of the building. The columns from the basement to the level 5 are rectangular and oriented parallel to the parking spaces. These rectangular columns transition to circular and square columns in office spaces from level 5 to the top level. This transition occurs with the use of transfer girders, columns brackets and adjustments to account for eccentricity in the columns. The columns sizes range from 1.5 ft to 3 ft in width and 1.5 ft to 7 ft in length. Circular columns range from 1.5 ft to 3 ft in diameter in the office areas. the building has a peculiar column with cross section of a parallelogram. This column is located at the entrance of the building and defines the corner of the building from the base to the top adding to the architecture.

Beams integrated with flat slab are present in the parking areas.Transfer girders are present at the fifth level where the floor plan changed from parking to office. Beams are also used to transfer lateral loads from facade to the shear walls. The 8 - 12 inch slabs connect to the columns with drop panels ranging about 8 in additional depth. Drop panels mainly exist at parking spaces and thin drops are added at slabs in office spaces. The slabs also create interaction between the columns and core walls of the building and help distributing gravity loads.

Floor System

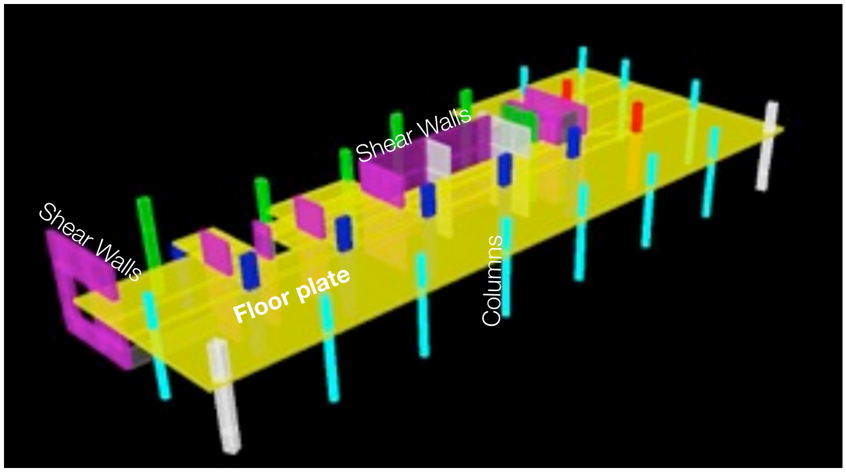


Figure 7: ETABS model, 3D view of floor plan.

Floor system of The Optimus typically consist of two-way flat slabs with drop panels. Flat slabs provide a floor to ceiling height of about 10 to 15 feet which provides ample of space for mechanical ducts and electrical wiring. Besides the floor live loads, the flat slabs support facade that is attached to the perimeter of the slabs. The slabs also help transfer lateral loads from the facade to the shear walls around the stairwell and elevator.

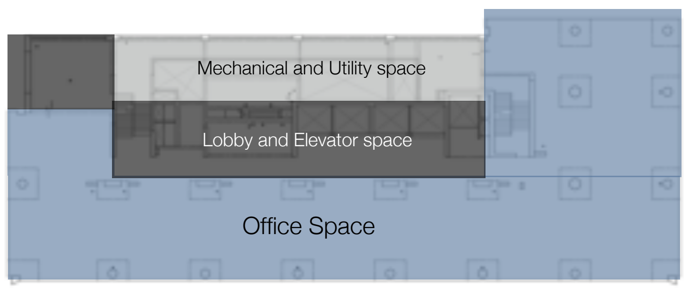


Figure 8: Division of floor space area for typical office floor.

The slabs are 8” thick and typical size of drop panel is 4’6”x4’6” x 8”. The primary purpose of the drop panel is to reduce deflections and punching shear in 27’6” long spanning slabs. A secondary purpose is to help the slab increase the moment carrying capacity. However, this is majorly carried by the top and bottom reinforcement. The drop panels are not reinforced which proves that it does not provides minimum support in transferring slab moments to columns.

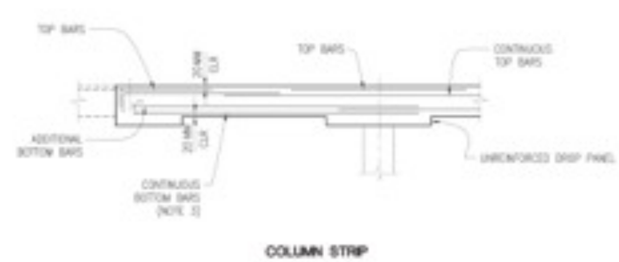


Figure 8: Section of column strip for typical slab

Slab depths have been increased to 11.5” in fire areas also called refuge areas where there is a higher chance of live load occurring during a fire. The utility areas that house mechanical equipment have thicker slabs to support mechanical and electrical equipments. The slabs in parking spaces have larger drop panels and additional hidden beams to support live load due to vehicles.

Lateral System

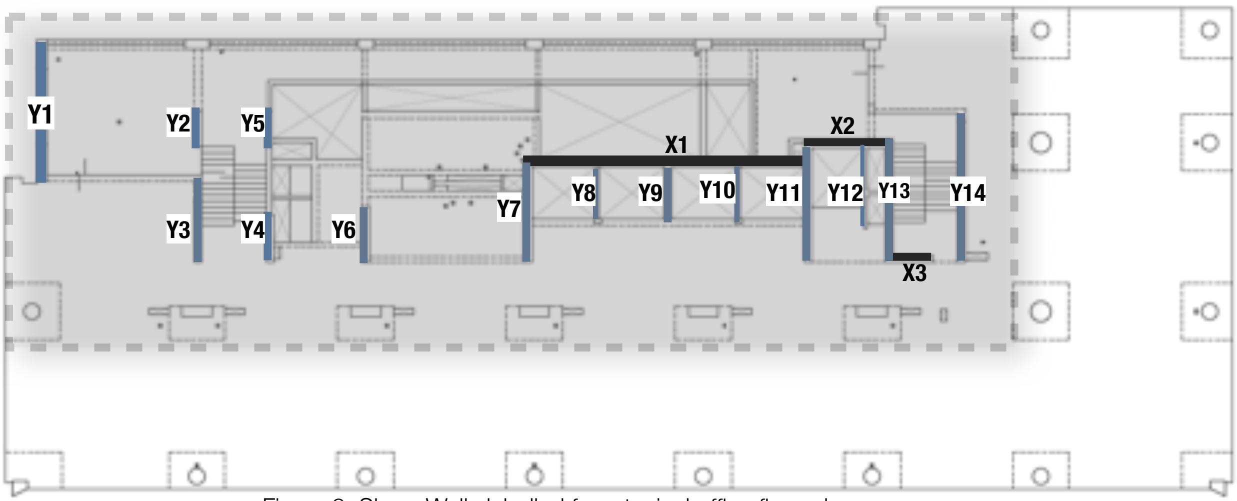


Figure 9: Shear Walls labelled for a typical office floor plan.

The Main Lateral Force Resisting System consists of shear walls present at the core of the building. The shear walls envelope the elevator and stairwell which is the best way to achieve continuity in the walls from bottom to the top without adding obstructions in the floor area. The walls span from the base to of the building to the roof and range 8 inch to 20 inch thick. The walls connect to each other through the floor slab or link beams to act as a unified system against wind and seismic forces. There are 14 short length walls in the North-South direction and 3 long shear walls in the East-West direction. The shear wall X1 in the East-West direction is a major element that is 47 ft long 16 inch thick supporting the transverse loads. The wall Y1 is a major element in supporting loads due to torsion because the wall is located farthest from the center of rigidity giving a larger moment arm.

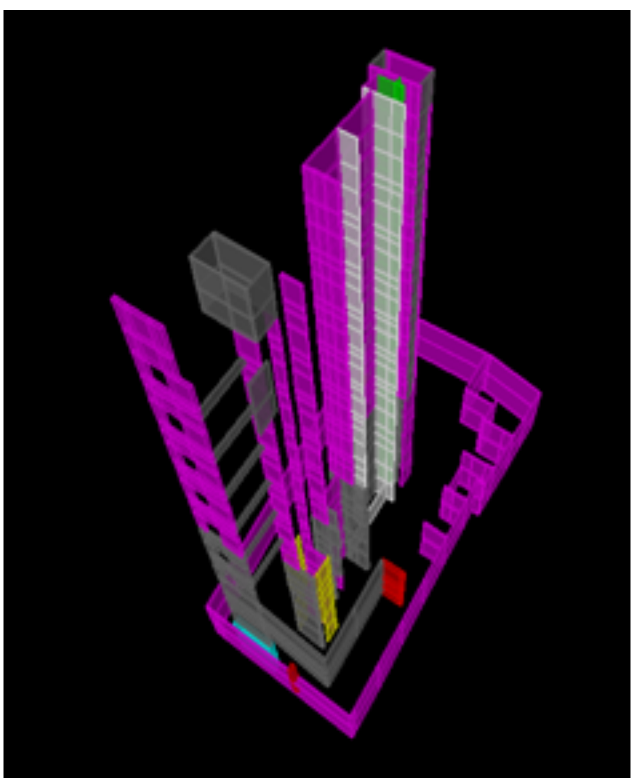


Figure 10: Shear walls in 3D extruded view.

Design Codes

As the building is located in India, the Indian Standard (IS) code is used to design The Optimus. However, the American codes are used in this report while performing analysis. This will also provide a comparison between the two codes and also a look into the design from the perspective of the american rules.

- Minimum design loads for Buildings other than seismic loads

IS Code	Description
IS 875 (Part 1): 1987	Dead loads
IS 875 (Part 2): 1987	Imposed loads
IS 875 (Part 3): 1987	Wind loads
IS 875 (Part 5): 1987	Special loads and load combinations

- Seismic Provisions for buildings

IS Code	Description
IS 1893: 2002	Criteria for earthquake resistance design of structure
IS 4326: 1993	Earthquake resistant design and Construction of Buildings - Code of Practice
IS 13920: 1993	Ductile Detailing of Reinforced concrete Structures subjected for Seismic Forces - Code of Practice

- Building code requirements for Structural Concrete:

IS Code	Description
IS 456: 2000	Plain and Reinforced Concrete - Code of practice
SP 16	Structural use of concrete. Design charts for singly reinforced beams, doubly reinforced beams and columns.
SP 34	Handbook on Concrete Reinforcement & Detailing
IS 1904	Indian Standard Code of practice for design and construction foundations in Soil: General Requirements

IS Code	Description
IS 2950	Indian Standard Code of Practice for Design and Construction of Raft Foundation (Part –1)
IS 2974	Code of practice for design & construction of machine foundation
IS 2911	Code of practice for design & construction of Pile foundation (Part I 1o IV)

- Building code used for Structural Steel

IS Code	Description
IS 800: 1984	Code of practice for general construction in Steel

- Design codes to be used for Tech 3

American codes to analyze the existing conditions.

American Code	Description
ACI 318-11	Concrete Design Code
ASCE 7-10	Minimum design loads for Buildings and Structures for Dead, Live, Wind and Seismic loads.

Materials

Materials used on this project help achieve efficiency in the structural system. This is achieved by economizing the use of material with respect to increasing height. Hence, higher strength concrete is used in the shear walls and columns in the lower floors. As we go higher, the material strength decreases.

Use of the material	Indian Code	American Code
	Material	Equivalent Material
Raft and pile foundations	M40	5000 psi
PCC	M15	3000 psi
slabs and beams	M40	5000 psi
Perimeter basement wall except Grid A	M40	5000 psi
Perimeter basement wall on Grid A	M60	7000 psi
Walls, Columns and Link beams from foundation for 5th floor	M60	7000 psi
Walls, Columns and Link beams from 5th floor to above	M40	5000 psi

<table>
<tr><th colspan="6">Concrete</th></tr>
<tr><th colspan="3">Indian Code</th><th colspan="3">American Code</th></tr>
<tr><th>Concrete Grade</th><th>f’c (psi)</th><th>Ec (ksi)</th><th>Equivalent Concrete type</th><th>f’c</th><th>Ec = 57000√f’c (ksi)</th></tr>
<tr><td>M60</td><td>7000</td><td>5614.3</td><td>High strength concrete 28 days</td><td>7000 psi</td><td>4768.9</td></tr>
<tr><td>M40</td><td>4700</td><td>4584.3</td><td>Ordinary ready mix</td><td>5000 psi</td><td>4030.5</td></tr>
<tr><td>M15</td><td>1750</td><td>2807.2</td><td>Ordinary ready mix</td><td>3000 psi</td><td>3122.01</td></tr>
<tr><td colspan="3">fck is 28 compressive strength for 150mmx150mm cube.
Poission’s ratio = 0.2
Coefficient of thermal expansion = 9.9x10-0.6 per deg C.</td><td colspan="3">f’c - specified compressive strength of concrete.
Coefficient of thermal expansion = $5.5x10^{-6}$ per deg F.
Poissions ratio = 0.2</td></tr>
<tr><th colspan="6">Reinforcement</th></tr>
<tr><td colspan="3">According to IS: 1786 Fe 415 (Fy = 415 MPa/ 60 ksi) or Fe 500 (Fy = 500 MPa) steel bars are used.</td><td colspan="3">According to ASTM A615, deformed and plain carbon steel bars are used with Fy = 60 ksi.</td></tr>
</table>

Gravity Loads

The dead, superimposed and live loads used on the project are referred to IS Code provisions whereas the report uses ASCE 7-10 provisions to calculate live loads. The superimposed dead loads that are used are provided by the structural engineer because they are loads from actual materials like floor finishes used on the project. The difference in live loads and calculation procedures like Live load reduction will cause difference in analysis results. However, the assumption is that indian code gives conservative results because it accounts for contingencies in construction and materials used on the project. The tables below show the difference in loading values between the IS code and ASCE 7-10 provisions.

- Typical Dead Loads

	IS Code (kN/ m^3)	**ACI 318-11 / ASCE 7-10 (lb / ft^3)**
Normal weight Concrete	25.00	150
Floor finishes / Plasters	20.00	140

Loading Area	**Type of Load**	**IS Code (kN/ m^2)**	**ACI 318-11 / ASCE 7-10 (lb / ft^2)**
Parking Space and Drive-way	Superimposed Dead Load	1.75	36.6
	Live Load (vehicles)	2.50 non-reducible	40 non-reducible
	Live Load (fire truck over ground floor)	15.00 non-reducible	300 (AASHTO LRFD Bridge design standards) - non-reducible
Covered Entryway over ground floor	Superimposed Dead Load	7.25	151.4
	Live Load	4.00	100
Entrance Lobby, Elevator lobbies	Superimposed Dead Load	2.00	41.8
	Live Load	3.00	100
Mechanical Floor	Superimposed Dead Load	2.00	41.8
	Live Load	7.50 Non-reducible	150 non-reducible
Electrical room over ground floor	Superimposed Dead Load	2.00	41.8
	Live Load	13.50 non-reducible	282 non-reducible

Loading Area	Type of Load	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Stairs	Superimposed Dead Load	1.50	31.33
	Live Load	3.00	100
Toilet rooms	Superimposed Dead Load	4.50	94
	Live Load	2.00	40
Typical Office	Superimposed Dead Load	3.00	62.7
	Live Load	4.00	100
Retail over ground floor	Superimposed Dead Load	4.575	95.6
	Live Load	4.00	100
Eatery and Utility	Superimposed Dead Load	3.00	62.7
	Live Load	5.00	100
Outdoor Utility over Level 105, 107 and similar	Superimposed Dead Load	5.625	117.5
	Live Load	5.00	100
Planted Terrace	Superimposed Dead Load	12.50	261.1
	Live Load	3.00	100
Amenity / Fitness Center	Superimposed Dead Load	3.50	73.10
	Live Load	5.00	100
Water tank over level 119	Superimposed Dead Load	3.50	73.1
	Live Load	35 non-reducible	731 non-reducible
Electrical Panel room at ground floor	Superimposed Dead Load	2.00	41.8
	Live Load	13.50 non-reducible	282 non-reducible
Roof	Superimposed Dead Load	5.50	114.9
	Live Load	3.00 Non-reducible	100 non-reducible

Loading Area	Type of Load	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Peripheral loads	Superimposed Dead line load over wall surface	0.75	15.7

- Live load reduction

According to IS 875 (part 2) - 1987, section 3.2, live load has been reduced.

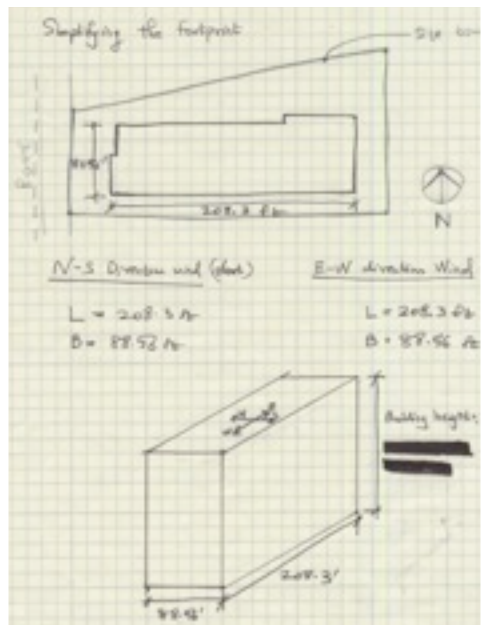
<table>
<tr><th colspan="2">IS Code</th><th>ASCE 7-10</th></tr>
<tr><td colspan="2">Walls, columns, piers, their supports and foundation:</td><td rowspan="17">Reduction in live loads is carried out as per the provision in ASCE 7-10 Section 4.7.2/</td></tr>
<tr><td>Number of floors supported</td><td>% reduction in total live load</td></tr>
<tr><td>1</td><td>0</td></tr>
<tr><td>2</td><td>10</td></tr>
<tr><td>3</td><td>20</td></tr>
<tr><td>4</td><td>30</td></tr>
<tr><td>5 to 10</td><td>40</td></tr>
<tr><td>over 10</td><td>50</td></tr>
<tr><td colspan="2">Beams, girders and trusses</td></tr>
<tr><td>Supported Area</td><td>% reduction in total live load</td></tr>
<tr><td>less than 50m²</td><td>0</td></tr>
<tr><td>50m² to 100 m²</td><td>5</td></tr>
<tr><td>100m² to 150 m²</td><td>10</td></tr>
<tr><td>150m² to 200 m²</td><td>15</td></tr>
<tr><td>200m² to 250m²</td><td>20</td></tr>
<tr><td>Over 250 m²</td><td>25</td></tr>
</table>

Lateral Loads

Wind and Seismic loads were calculated using ASCE 7-10 provisions. Wind pressures were used to find story forces and seismic base shear and mass of stories was used to find the story force due to seismic loads. The calculations are performed manually in Appendix 1.

Wind Loads

The wind loads were calculated using the ASCE 7-10 Part 1 of the MWFRS Directional Procedure (Chapter 27). This procedure was chosen as appropriate for hand calculations and computer analysis because the building height is greater than 60 ft and is fairly enclosed. The windward, leeward, sidewall and roof pressured were also calculated using the directional procedure.



The basic wind speed (98.4 miles/hour) was determined from the weather data of the existing location in India. The behavior of the wind is dominated by the location of the building that is closer to the sea. This was the reason why exposure category D was chosen for wind pressure coefficient. Other parameters were chosen based on the location, the shape of the building and the simplifications made for ease of calculation. The exterior walls of the building were projected onto East-West and North-South planes and building was simplified to a cuboid. The mean roof height of the building is the distance from the ground to the top of the ceiling of the roof gymnasium, also termed as parapet wall by the architect. The envelope of the roof top gymnasium was termed as parapet walls because it's the part of the building where shear walls don't exist and a separate roof top structure.

The simplification made also affected the calculation of the natural frequency of the building. Consequently, this affect the determinations of gust effects on the building. As the the lateral force resisting system of the building consist of shear wall core, the following formula was used from ASCE 7-10 to calculate the natural frequency of the building.

$$n_a = 385(C_w)^{0.5}/h$$

$$C_w = \frac{100}{A_B}\sum_{i=1}^{n}\left(\frac{h}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

The net wind pressures were calculated using gust effect factors, wind load parameters and internal pressures of the building. Wind pressures resulted in higher base shear in the North-South direction as compared to East-West direction which is evident because of the slender shape in the building and higher surface area in the north-south direction. The story forces are collected by the facade which is supported at the perimeter of the floor slab. The floor slab is flat slab assumed to be a rigid diaphragm. The forces from the facade are transferred to the shear walls through the floor slab.

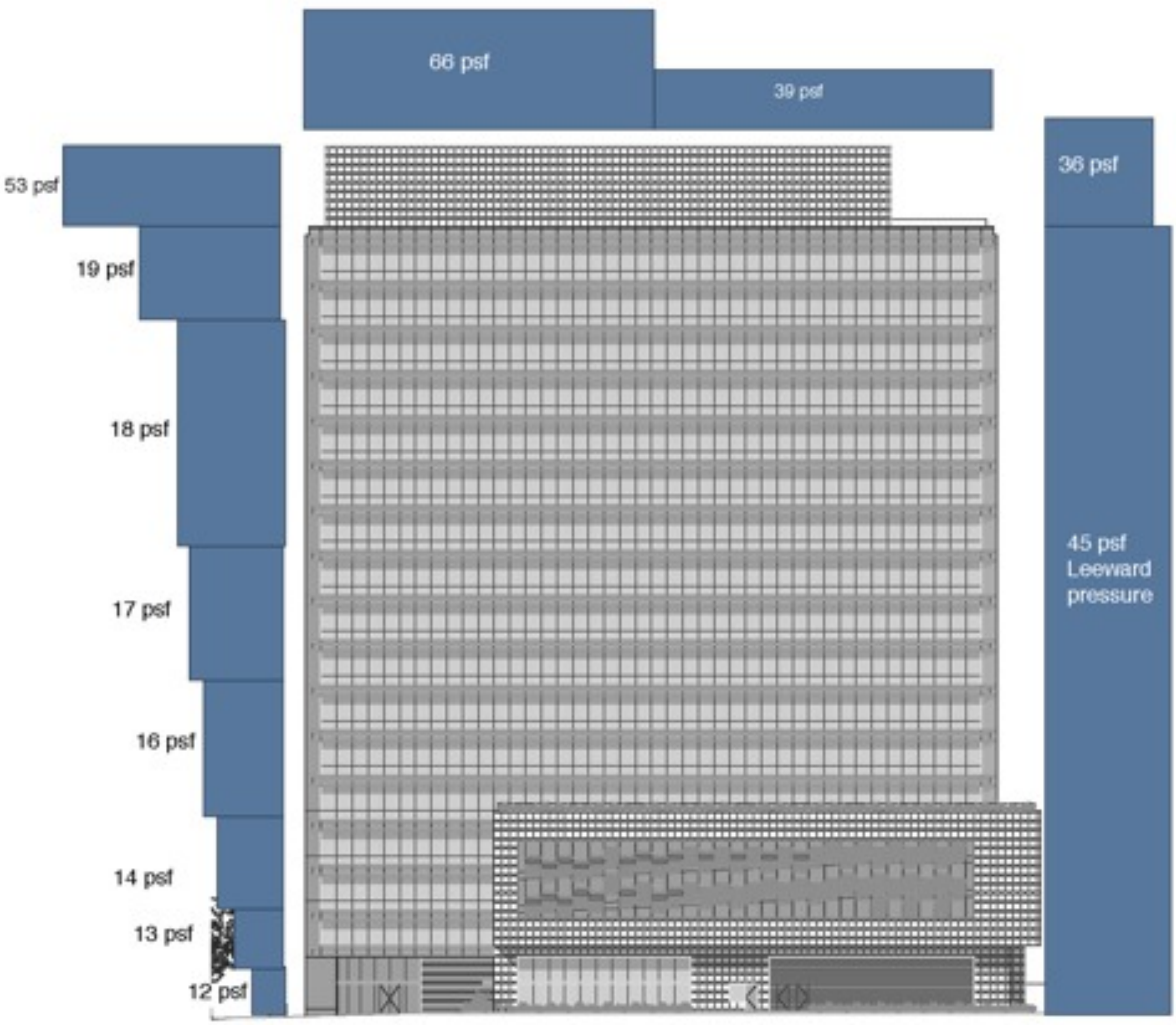


Figure 11: Wind pressure diagram in East-West Direction

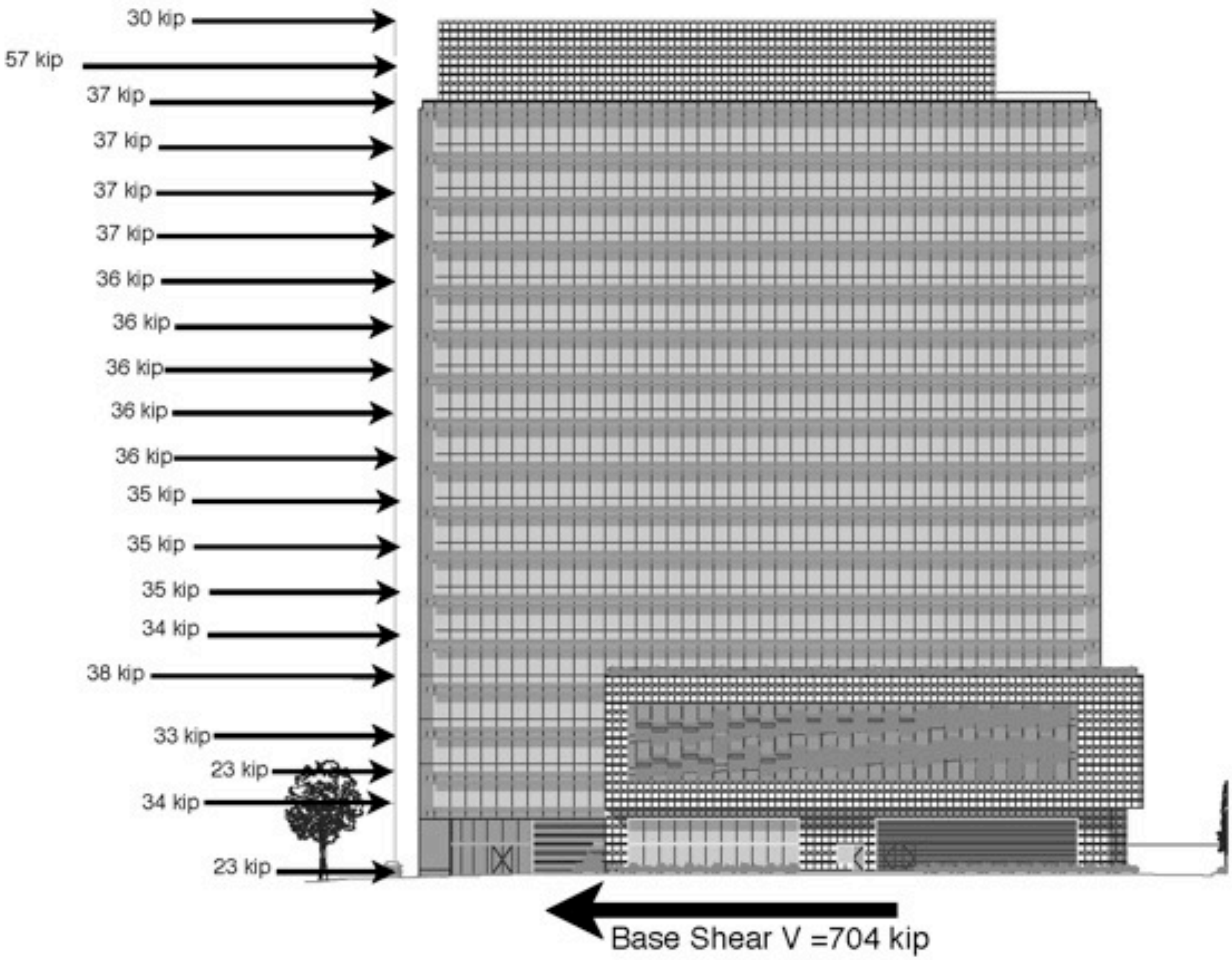


Figure 12: Story Force diagram due to wind loads in East-West Direction

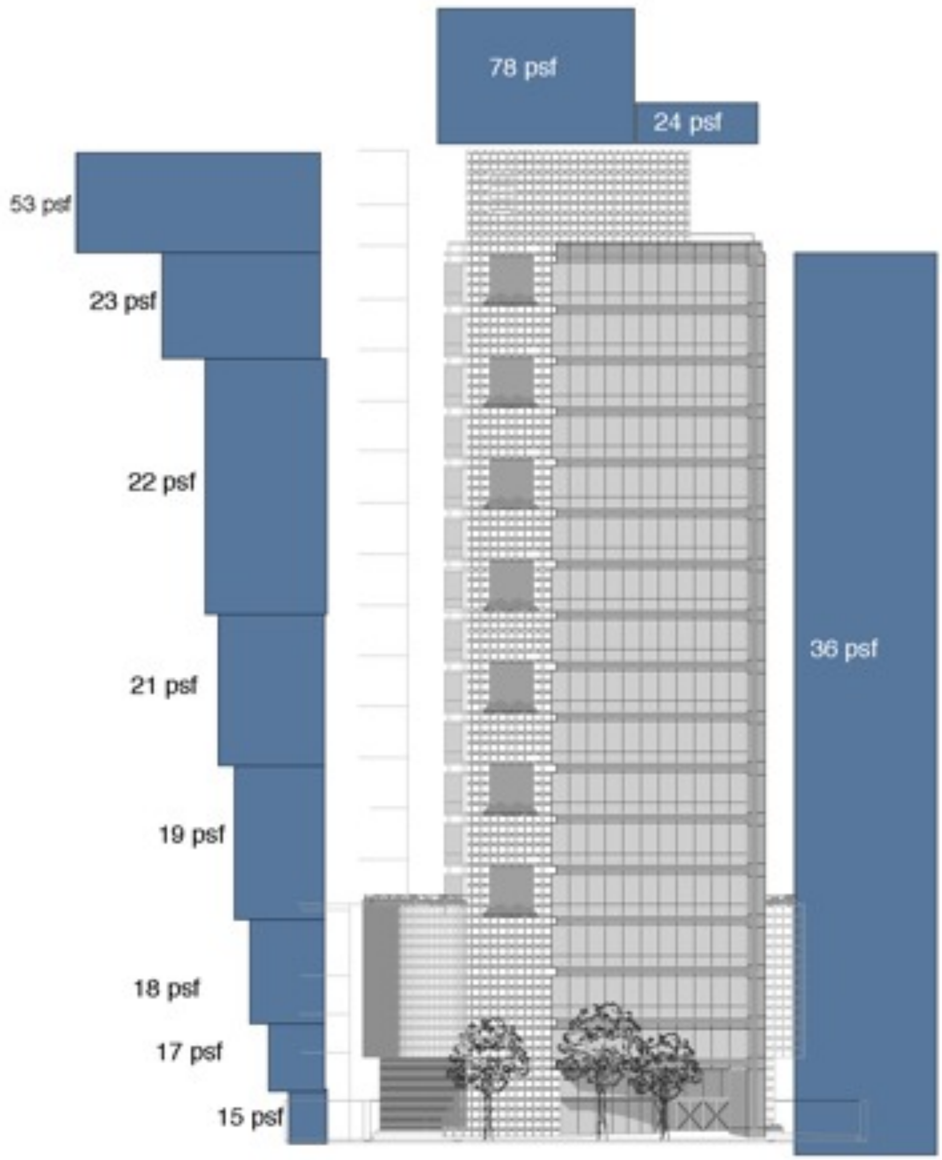


Figure 13: Wind pressure diagram in North-South Direction

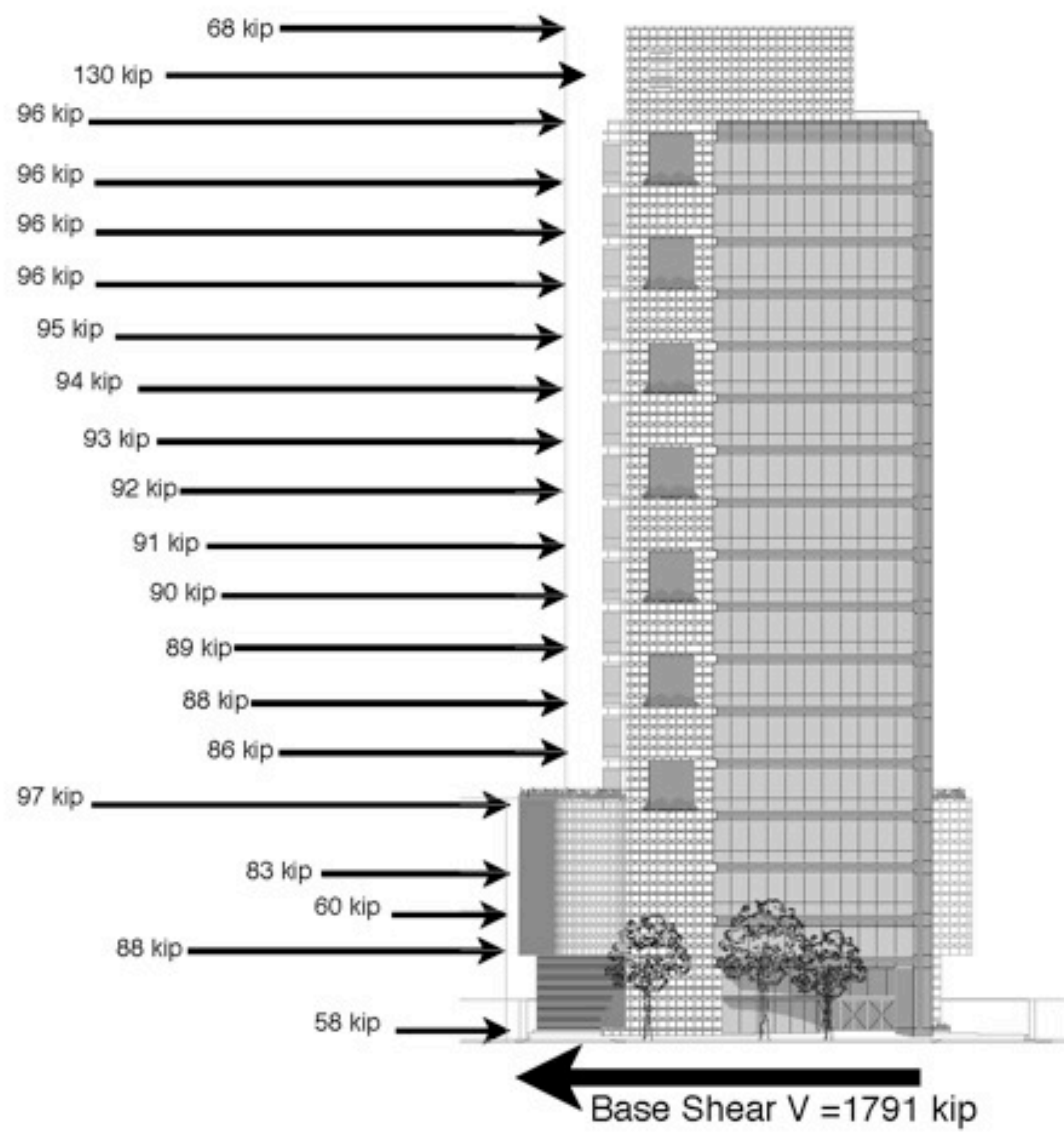


Figure 14: Story Force diagram due to wind loads in North-South Direction

Seismic loads

The seismic loads were calculated using Equivalent Lateral Force Procedure from ASCE 7-10 Chapter 12.8. In order to calculate seismic base shear, the ground accelerations were not available for the existing location in ASCE 7-10. Therefore, the building was relocated to a location with similar seismic activity - New York. It was determined from USGS seismic world maps that the seismic behavior of New York is similar to the existing location of the building. However, the soil characteristics remained the same that was available in the geotechnical report of the location of the building. As the lateral system of the building is a reinforced shear wall, it falls in Seismic Design Category C according to Table 12.2-1, ASCE 7-10 and Seismic Risk Category I in ASCE 7-10. The approximate time period of the building was calculated to be 1.41 seconds using the following formula provided in ASCE 7-10.

Approximate fundamental period		
$T_a = C_t h_n^x =$	1.405	seconds
$C_t =$	0.02	for all structural systems according to table 12.8-2
$x =$	0.75	
$h_n =$	289.8	height from base of the structure to the top of shear walls

The ground accelerations and time period was used to calculate the seismic design coefficient. This was further used factor the seismic weight of the building to get the effective seismic weight and finally, the seismic base shear of the building. According to ASCE 7-10, the seismic weight of the building consist of self-weight of members, superimposed dead loads and 25% of the live loads. As the formula for time period does not differ for North-South and East-West directions, the seismic coefficient remains the same and also, the base shear. Conceptually, the building will have a higher time period in the North-South direction of the slender shape and consequently, a lower base shear. However, it would be a conservative approach to use same base shear in both directions. Also, using this conservative approach did not make seismic loads to control the lateral system. Generally, it is assumed that the lateral system tall buildings is controlled by wind loads which was found to be true in the analysis that is explained further in the report.

The seismic base shear was vertically distributed according to the mass of each story and further, distributed horizontally among the shear walls according to the stiffness. The stiffer shear walls attracted greater shear forces. The calculation of horizontal distribution of forces and story drifts for wind and seismic loads are explained and compared further in analysis section of the report.

Effective seismic weight							
Floor	Slab	beams	shear walls	SDL	live loads	façade	Total
1A 2A	2302.0	721.8	491.5	752.4	1541.8	9.7	11638.3
3A	2302.0	721.8	491.5	667.9	1368.7	9.7	5561.5
5	2064.4	1249.3	491.5	1289.0	513.9	9.7	5617.7
6 8	1248.8	194.3	491.5	779.8	1243.7	8.6	7933.3
7 9	1460.8	194.3	491.5	777.41	325.76	8.6	6516.5
10 12 14 16	1248.8	194.3	491.5	779.8	1243.7	8.6	15866.6
11 13 15 17	1547.7	194.3	491.5	885.6	1412.5	8.6	18160.5
Roof (level 18)	2064.4	194.3	491.5	1289.0	513.9	9.7	4562.7
			Effective seismic weight				**75857.1**

Center of Mass at LVL7						
Load distribution		Load (PSF or LF)		total weight (kip)	COM	
		DL (kip)	SDL (kip)		X	Y
Area of slab	A	662.56	356.10	1018.67	78.40	18.50
	B	41.75	26.60	68.35	15.35	43.91
	C	89.67	38.09	127.76	17.20	62.07
	D	59.82	38.12	97.94	48.80	69.95
	E	30.09	9.58	39.67	35.90	51.70
	F	11.74	9.98	21.72	53.20	44.80
	G	36.49	20.67	57.16	70.85	46.85
	H	30.65	19.53	50.18	105.65	40.50
	I	16.89	6.62	23.51	133.90	41.40
	J	32.56	13.83	46.39	131.05	65.17
	K	23.97	10.18	34.15	112.92	48.60
	L	112.85	61.29	174.14	169.90	69.82
	M	311.76	166.81	478.57	178.50	30.65
Length of Shear wall	X1	99.74		99.74	108.10	53.25
	X2	30.77		30.77	137.70	56.35
	X3	12.43		12.43	147.70	37.59
	Y1	61.90		61.90	6.02	61.85
	Y2	12.52		12.52	31.36	58.84
	Y3	21.01		21.01	31.36	42.05
	Y4	16.13		16.13	43.17	40.90
	Y5	9.41		9.41	43.00	58.88
	Y6	19.31		19.31	58.78	41.65
	Y7	35.65		35.65	85.29	45.50
	Y8	6.89		6.89	96.69	48.25
	Y9	15.84		15.84	108.17	48.25
	Y10	6.89		6.89	108.99	48.25
	Y11	37.07		37.07	119.17	47.05
	Y12	19.39		19.39	128.62	49.80
	Y13	37.26		37.26	132.46	47.10
	Y14	49.24		49.24	144.22	48.70
	Total weight		2729.65			
	COM in X direction			**100.2**	**ft**	
	COM in Y direction			**37.3**	**ft**	

Center of rigidity at LVL7					
Shear Wall	K	x	y	%K	K * (x or y)
X1	989	108.10	53.25	69.80	106939.94
X2	305	137.70	56.35	21.52	41996.23
X3	123	147.70	37.59	8.68	18166.55
Y1	614	6.02	61.85	17.78	37965.15
Y2	124	31.36	58.84	3.58	7273.13
Y3	208	31.36	42.05	6.03	8748.23
Y4	160	43.17	40.90	4.62	6524.51
Y5	93	43.00	58.88	2.69	5466.89
Y6	191	58.78	41.65	5.54	7962.35
Y7	353	85.29	45.50	10.24	16081.10
Y8	68	96.69	48.25	1.97	3287.56
Y9	157	108.17	48.25	4.54	7561.39
Y10	68	108.99	48.25	1.97	3287.56
Y11	368	119.17	47.05	10.65	17294.47
Y12	192	128.62	49.80	5.57	9572.77
Y13	369	132.46	47.10	10.70	17399.92
Y14	488	144.22	48.70	14.14	23775.67
Total stiffness in X	1417.25				
Total stiffness in Y	3452.87				
COR in X	**117.91**				
COR in Y	**49.87**				

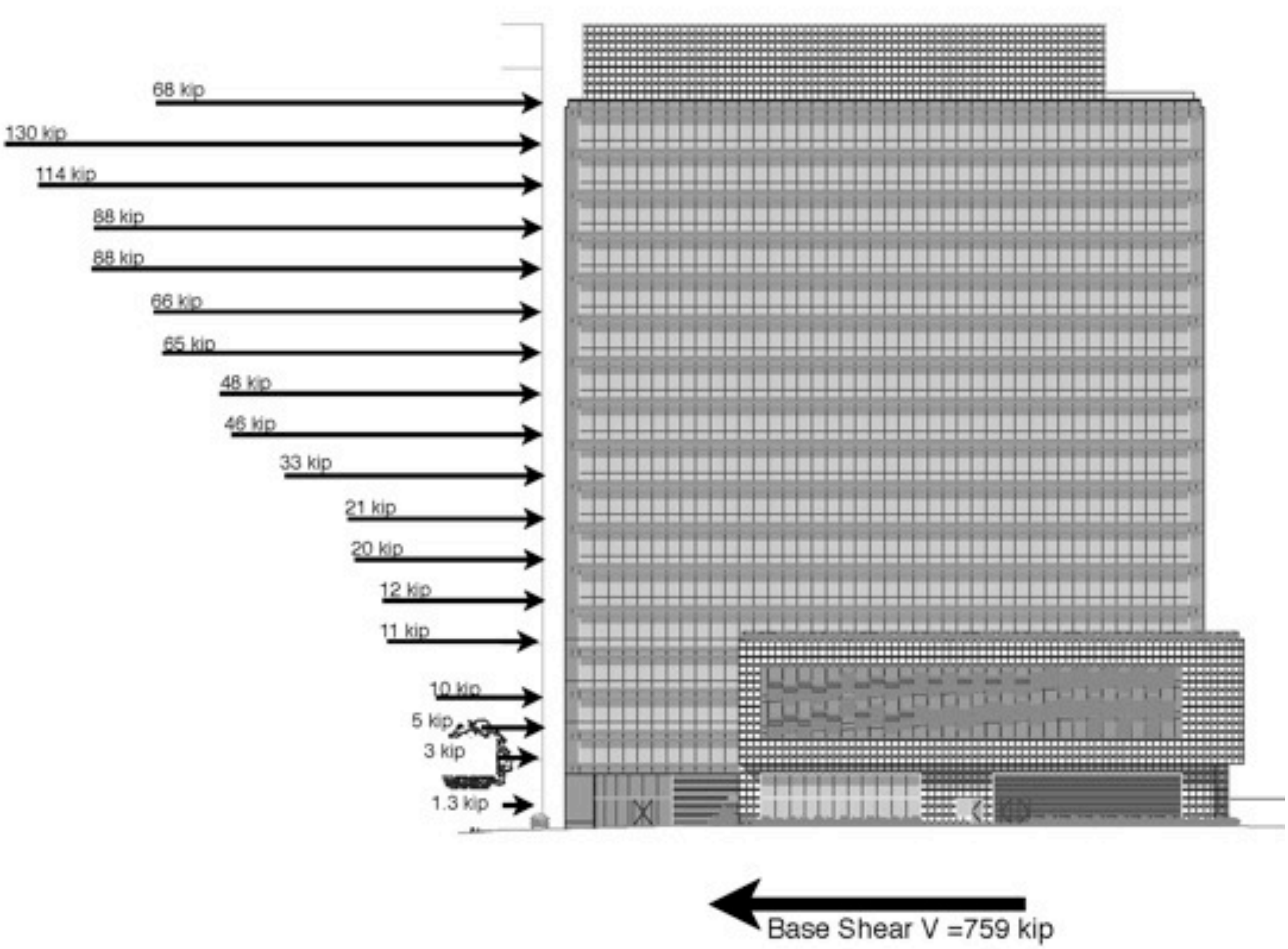


Figure 15: Story Force diagram for seismic forces in East-West Direction

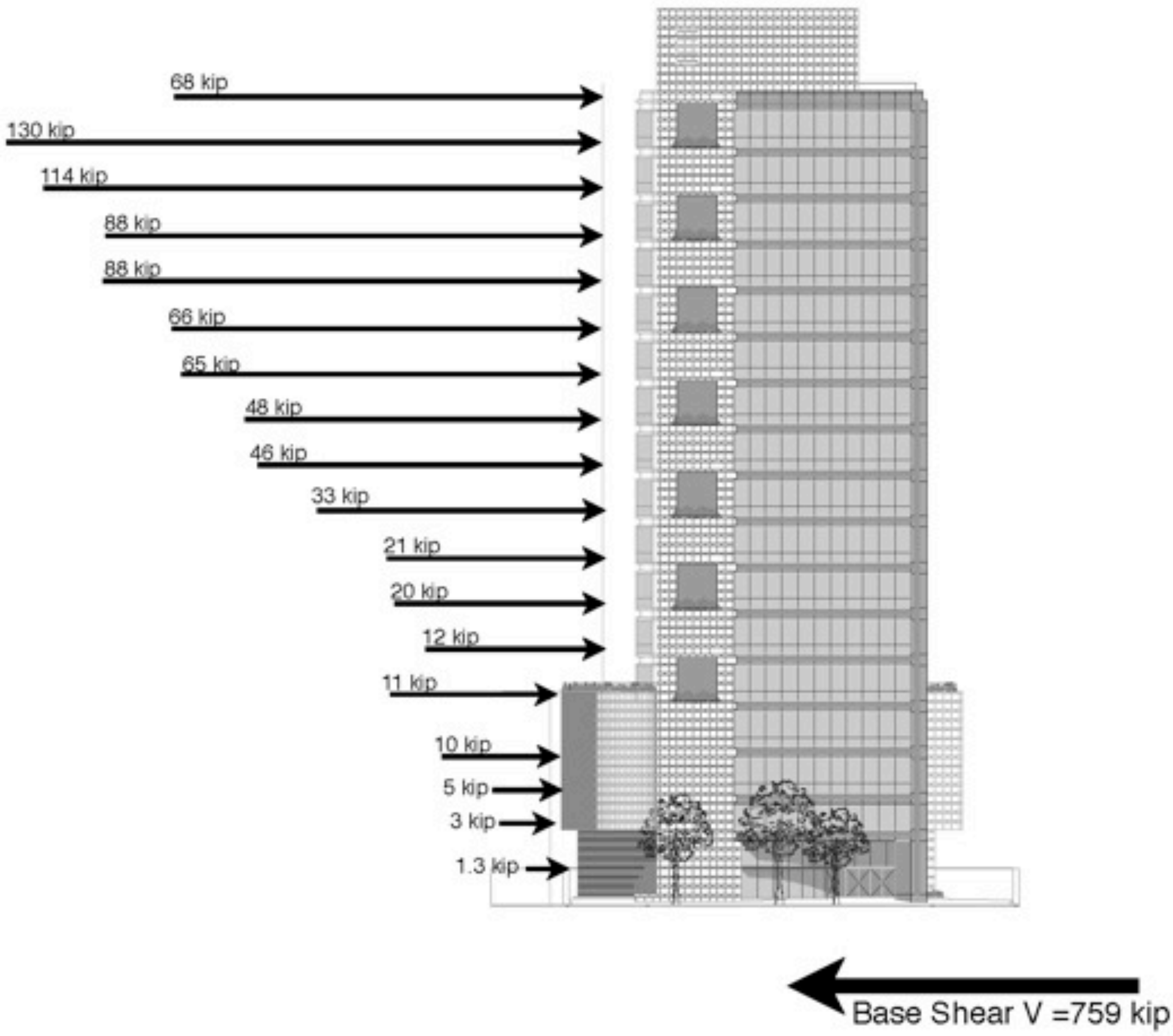


Figure 15: Story Force diagram for seismic forces in North-South Direction

Analysis of Lateral Loads

The lateral force resisting system of The Optimus was analyzed using the wind and seismic loads. Internal shear forces were determined in shear walls using relative stiffnesses. Also, story drifts were calculated for each story and deflection of the top story was checked. The internal forces and story drifts were also used to determine the controlling loads - wind or seismic. Finally, controlling load was used to determine critical shear wall member that was further checked for shear and overturning moment.

In order to calculate internal forces in shear walls, it was required to calculate the Center of Mass (COM) of the stories and Center of Rigidity (COR) of the shear walls at each story. It is a complex processs to calculate COM and COR for every story of a 17 story building. Therefore, the process was simplified by calculating COM and COR for a typical story (Level 7). This was assumed to be the same for every story in the building.

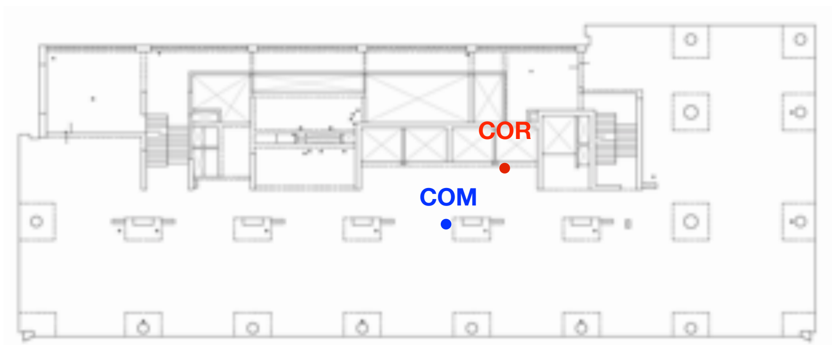
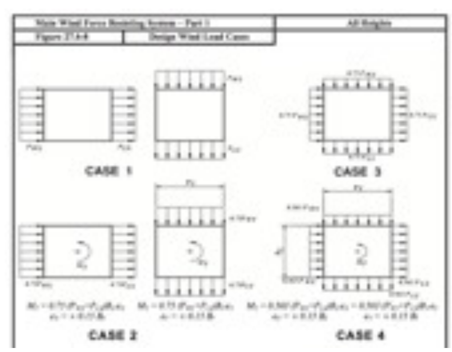


Figure 16: Typical office floor plan showing Center of Mass (COM) and Center of Rigidity (COR)

The design wind load cases from ASCE 7-10, figure 27.4-8 were used to calculate wind load effects due to shear and torsion. These forces were applied to every wall at level 7 and the horizontal load distribution was determined using wall stiffnesses. The critical wall was the one that had largest stiffness. The critical load case in East-West direction was wind forces in East-West direction without eccentricity. The wind load in this load case, induced maximum shear in the shear wall labelled as X1 which is the largest shear wall in the structure - 47 ft long and 20 inch thick. The wind in North-South direction with a positive eccentricity induces maximum shear in shear wall labelled as Y1 because it is furthest from the center of rigidity providing a large moment arm to resist torsional shear.



The vertical story forces due to seismic loads at Level 7 were used to calculate internal shear forces in walls due to direct shear and shear due to accidental torsion. The walls Y1 in North-South direction and X1 in East-West direction were determined as critical walls similar to wind loads. As the base shear due to wind loads was higher than that due to seismic loads, the wall X1 was checked for shear and overturning moments. The wall X1 has added reinforcement at the ends which resist overturning moments in positive and negative directions. As a spot check, the wall X1 was check for its shear and moment capacity. The shear and moment reinforcement at the boundary elements (the ends of the shear wall) were taken into consideration for a conservative approach. By applying maximum shear, the wall passed in shear and overturning moment and the reinforcement was determined to be adequate.

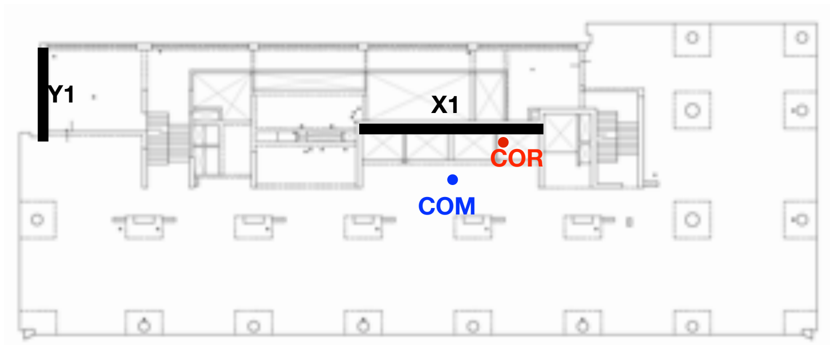


Figure 17: Critical Shear walls X1 and Y1

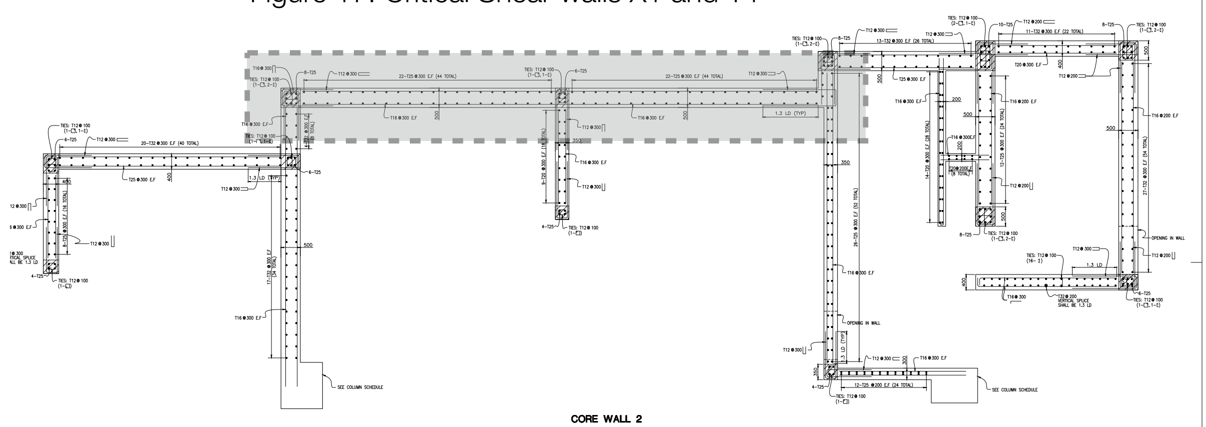


Figure 18: Cross-section of core wall at the ground level. Shear Wall X1 highlighted.

As a building becomes taller, it becomes more flexible at higher stories. This causes large deflections at higher stories which affect the comfort level of the inhabitants. The ASCE 7-10, Commentary Chapter C mentions that the lateral drift of the building should be in the order of h/600 to h/400. The story drifts were calculated for wind and seismic forces. This calculation was carried out by determining stiffness in shear walls at each story due to unit distributed loads. The unit distributed wind and seismic loads were applied to the stiffness to find story drift. For ease of calculation and to reduce complexity, the cross-section of shear wall was assumed to be consistent from the base of the structure to the roof. The thickness and modulus of elasticity controlled the calculation of stiffness. From the base to level 5, the walls had higher thickness and higher strength concrete (5000 psi). Thickness in walls decreased after level 5 and 4000 psi concrete was used as material for walls. Using the provisions of ACI 318-11, section 10.10.4.1, un-cracked and cracked modifiers were applied to the gross moment of inertia of the shear walls and drifts were calculated. It was found that the drift due to wind loads in a cracked section was maximum - 10.2 inches at the roof of the building. Although this value is higher than the drift limit in ASCE 7-10 - h/400 = 8.67 in, it can be termed as an overestimated value. The reason for overestimation of story drift is that additional stiffness at lower stories was disregarded as well as the stiffness offered by the large column at the lower stories was disregarded for ease of calculation. Hence, additional stiffness due to columns and additional shear walls in the base would help reduce the deflection at the roof.

Story drifts in X and Y directions for Seismic Loads							
Level	height	Kx (kip/in)	Ky	Fx	Fy	delta x	delta y
1A	20	5838.90	4362.44	0.01	0.01	0.0000	0.0000
2A	30	643.11	564.88	0.02	0.02	0.0000	0.0000
3A	40	281.93	234.60	0.04	0.04	0.0001	0.0002
5	57	72.91	48.17	0.05	0.05	0.0007	0.0010
6	70	30.13	18.30	0.07	0.07	0.0022	0.0037
7	83	16.28	8.37	0.08	0.08	0.0046	0.0083
8	96	9.50	5.27	0.13	0.13	0.0134	0.0242
9	109	5.89	3.19	0.14	0.14	0.0230	0.0424
10	123	3.84	2.04	0.21	0.21	0.0569	0.1014
11	136	2.60	1.36	0.29	0.29	0.1116	0.2126
12	149	1.82	0.94	0.30	0.30	0.1675	0.3226
13	162	1.31	0.67	0.41	0.41	0.3153	0.6116
14	175	0.97	0.49	0.42	0.42	0.4956	0.8517
15	188	0.73	0.37	0.56	0.56	0.7646	1.5022
16	201	0.56	0.28	0.56	0.56	0.9961	1.9646
17	214	0.44	0.22	0.72	0.72	1.6581	3.2617
Roof (level 18)	228.28	0.34	0.17	0.78	0.78	2.2792	4.5247

Story drifts due to Wind loads for un-cracked walls

Story drifts in X and Y directions for Wind Loads							
Level	height	Kx (kip/in)	Ky	Fx	Fy	delta x	delta y
1A	20	5838.90	4362.44	0.60	0.24	0.0003	0.0001
2A	30	643.11	564.88	0.50	0.20	0.0008	0.0003
3A	40	281.93	234.60	0.70	0.28	0.0025	0.0013
5	57	72.91	48.17	0.48	0.19	0.0065	0.0039
6	70	30.13	18.30	0.55	0.21	0.0182	0.0116
7	83	16.28	8.37	0.56	0.22	0.0342	0.0231
8	96	9.50	5.27	0.57	0.22	0.0596	0.0416
9	109	5.89	3.19	0.57	0.22	0.0874	0.0687
10	123	3.84	2.04	0.58	0.22	0.1515	0.1182
11	136	2.60	1.36	0.59	0.25	0.2260	0.1665
12	149	1.82	0.94	0.59	0.25	0.3259	0.2424
13	162	1.31	0.67	0.60	0.25	0.4564	0.3420
14	175	0.97	0.49	0.60	0.25	0.6295	0.4695
15	188	0.73	0.37	0.61	0.25	0.8337	0.6310
16	201	0.56	0.28	0.61	0.24	1.0839	0.8150
17	214	0.44	0.22	0.63	0.24	1.4524	1.1065
Roof (level 18)	228.28	0.34	0.17	1.76	0.74	5.1550	4.3047

Story drifts due to Seismic loads for un-cracked walls

Story drifts in X and Y directions for Seismic Loads							
Level	height	Kx (kip/in)	Ky	Fx	Fy	delta x	delta y
1A	20	1470.18	2157.23	0.01	0.01	0.0000	0.0000
2A	30	462.53	322.63	0.02	0.02	0.0001	0.0001
3A	40	185.58	111.32	0.04	0.04	0.0002	0.0004
5	57	43.38	25.21	0.05	0.05	0.0011	0.0019
6	70	17.10	9.44	0.07	0.07	0.0038	0.0071
7	83	8.96	4.79	0.08	0.08	0.0087	0.0163
8	96	5.11	2.68	0.13	0.13	0.0246	0.0475
9	109	3.12	1.61	0.14	0.14	0.0433	0.0837
10	123	2.01	1.03	0.21	0.21	0.1027	0.2006
11	136	1.35	0.69	0.29	0.29	0.2146	0.4219
12	149	0.94	0.48	0.30	0.30	0.3260	0.6406
13	162	0.67	0.34	0.41	0.41	0.6126	1.2162
14	175	0.50	0.25	0.42	0.42	0.8507	1.6846
15	188	0.37	0.19	0.56	0.56	1.4975	2.9909
16	201	0.28	0.14	0.56	0.56	1.9554	3.9138
17	214	0.22	0.11	0.72	0.72	3.2622	6.5407
Roof (level 18)	228.28	0.17	0.09	0.78	0.78	4.4926	9.0218

Story drifts due to Wind loads for cracked walls

Story drifts in X and Y directions for Wind Loads							
Level	height	Kx (kip/in)	Ky	Fx	Fy	delta x	delta y
1A	20	1470.18	2157.23	0.60	0.24	0.0004	0.0001
2A	30	462.53	322.63	0.50	0.20	0.0011	0.0006
3A	40	185.58	111.32	0.70	0.28	0.0038	0.0025
5	57	43.38	25.21	0.48	0.19	0.0110	0.0074
6	70	17.10	9.44	0.55	0.21	0.0321	0.0226
7	83	8.96	4.79	0.56	0.22	0.0623	0.0452
8	96	5.11	2.68	0.57	0.22	0.1107	0.0818
9	109	3.12	1.61	0.57	0.22	0.1837	0.1375
10	123	2.01	1.03	0.58	0.22	0.2888	0.2181
11	136	1.35	0.69	0.59	0.23	0.4345	0.3302
12	149	0.94	0.48	0.59	0.23	0.6305	0.4813
13	162	0.67	0.34	0.60	0.23	0.8874	0.6798
14	175	0.50	0.25	0.60	0.23	1.2170	0.9348
15	188	0.57	0.19	0.61	0.23	1.6324	1.2564
16	201	0.28	0.14	0.61	0.24	2.1475	1.6555
17	214	0.22	0.11	0.63	0.24	2.8574	2.2054
Roof (level 18)	228.28	0.17	0.09	1.76	0.74	10.1611	8.5833

Story drifts due to Seismic loads for cracked walls

Finite Element Model

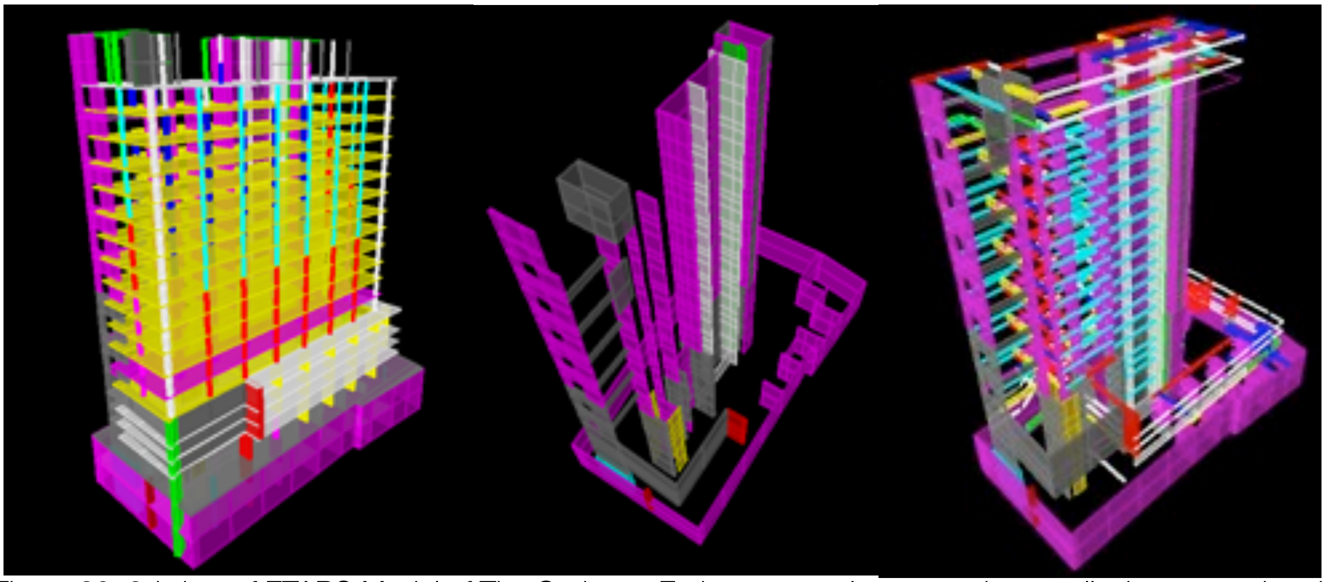


Figure 20: 3d view of ETABS Model of The Optimus: Entire structural system, shear walls, beams and and shear walls

A finite element model of the building was made using ETABS. The model was intended to be used for finding more accurate values for internal forces and story drifts. Accuracy in results is achieved by modeling every structural element of the building in ETABS with precision. Although, the entire building was modeled in ETABS, it became complex enough to fix the errors that gave skewed results.

Although modeling every structural element in the building is important for accurate results, it is also important to keep the model relatively simple. Making appropriate simplifications in the model helps in faster run time, easier debugging and reliable results. In the ETABS model, the beams were modeled as line elements, floors, shear walls were modeled as shell elements. A 48 inch mesh size was chosen to auto-mesh the floors and shear walls. It is anticipated that the complications in the computer model was caused because of the use of auto-mesh for shell elements like floor slabs and shear walls. Using manual meshing would have ensured proper alignment of the every node which can be an in issue while using automatic meshing. Hence, simplifications in the model will result in a reasonable model with accurate results. For further analysis it is planned that the floors will be modeled as rigid-diaphragm membrane elements. By using rigid-diaphragm membrane elements, modeling program will disregard the effects of out-of-plane forces in floor diaphragms. This will help in reducing complexity of the model without much affect on results due to lateral forces. These simplifications help in saving modeling time while performing a schematic-level analysis on a building structure where the behavior of a structure is more of a concern than getting accurate results. In order to attain, accurate and precise results which are more useful at the design-development stage; manual meshing of shell elements is required.

Conclusion

Thorough calculations of the effects of wind and seismic forces on The Optimus have resulted in conclusion that the lateral force resisting system is sufficient to carry the lateral loads at it's existing location. The internal forces due to direct shear and torsional forces reveal that the long shear wall at the core, spanning East-West and the shear wall at the East facade were critical elements. The largest shear wall out of these two was checked for shear and overturning moment and it found to be adequate in carrying the required shear and overturning moments.

It was found that the wind loads controlled the design of the lateral force resisting elements. The reason for wind controlling lateral system members is that it induces higher base shear and story drifts as compared to that due to seismic loads. The drifts were calculated for cracked and un-cracked wall sections and it was found that the drifts at the top story were slightly above the limiting deflection in ASCE 7-10. It was found that the higher deflection was attained because of the assumption of using same shear wall layout from the base to the top to reduce complexity in calculation. If the additional stiffness at the bottom stories were taken into consideration, then the deflection would fall under the limiting drift as specified in ASCE 7-10.

Although the computer model was useful in acquiring the results due to modeling errors, it was found that simplifications in modeling elements like the floors and walls would help in reducing complexity and achieving better and accurate results.

Appendix 1: Wind Loads

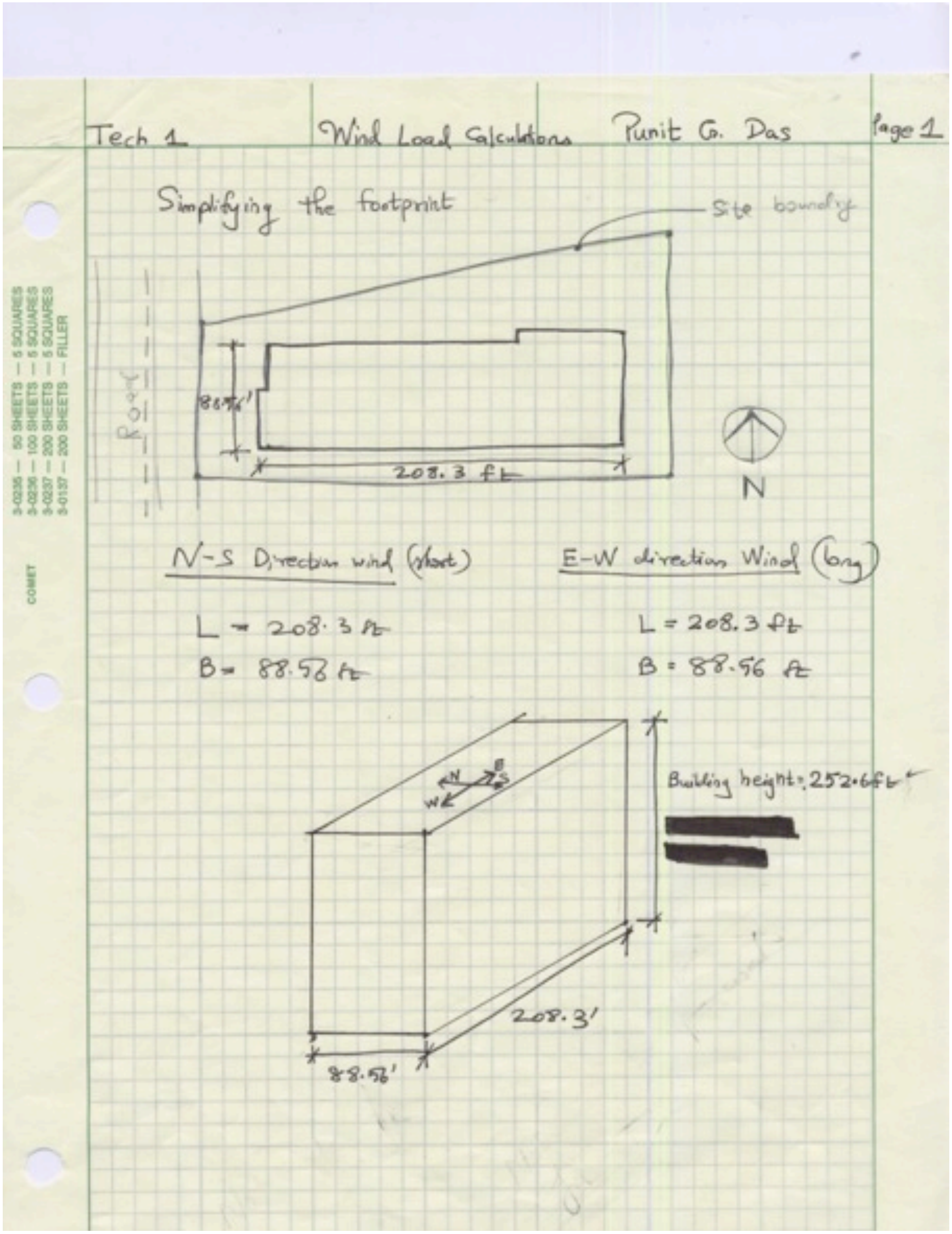


Table 27.2-1 Steps to Determine MWFRS Wind Loads for Enclosed, Partially Enclosed and Open Buildings of All Heights

Step 1: Determine risk category of building or other structure, see Table 1.4-1

Step 2: Determine the basic wind speed, V, for the applicable risk category, see Figure 26.5-1A, B or C

Step 3: Determine wind load parameters:
- Wind directionality factor, K_d, see Section 26.6 and Table 26.6-1
- Exposure category, see Section 26.7
- Topographic factor, K_{zt}, see Section 26.8 and Table 26.8-1
- Gust Effect Factor, G, see Section 26.9
- Enclosure classification, see Section 26.10
- Internal pressure coefficient, (GC_{pi}), see Section 26.11 and Table 26.11-1

Step 4: Determine velocity pressure exposure coefficient, K_z or K_h, see Table 27.3-1

Step 5: Determine velocity pressure q_z or q_h, Eq. 27.3-1

Step 6: Determine external pressure coefficient, C_p or C_N
- Fig. 27.4-1 for walls and flat, gable, hip, monoslope or mansard roofs
- Fig. 27.4-2 for domed roofs
- Fig. 27.4-3 for arched roofs
- Fig. 27.4-4 for monoslope roof, open building
- Fig. 27.4-5 for pitched roof, open building
- Fig. 27.4-6 for troughed roof, open building
- Fig. 27.4-7 for along-ridge/valley wind load case for monoslope, pitched or troughed roof, open building

Step 7: Calculate wind pressure, p, on each building surface
- Eq. 27.4-1 for rigid buildings
- Eq. 27.4-2 for flexible buildings
- Eq. 27.4-3 for open buildings

27.3.2 Velocity Pressure

Velocity pressure, q_z, evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \text{ (lb/ft}^2\text{)} \qquad (27.3\text{-}1)$$

[In SI: $q_z = 0.613 K_z K_{zt} K_d V^2$ (N/m²); V in m/s]

where

K_d = wind directionality factor, see Section 26.6
K_z = velocity pressure exposure coefficient, see Section 27.3.1
K_{zt} = topographic factor defined, see Section 26.8.2
V = basic wind speed, see Section 26.5
q_z = velocity pressure calculated using Eq. 27.3-1 at height z
q_h = velocity pressure calculated using Eq. 27.3-1 at mean roof height h.

The numerical coefficient 0.00256 (0.613 in SI) shall be used except where sufficient climatic data are available to justify the selection of a different value of this coefficient for a design application.

27.4 WIND LOADS—MAIN WIND FORCE-RESISTING SYSTEM

27.4.1 Enclosed and Partially Enclosed Rigid Buildings

Design wind pressures for the MWFRS of buildings of all heights shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \text{ (lb/ft}^2\text{) (N/m}^2\text{)} \qquad (27.4\text{-}1)$$

where

$q = q_z$ for windward walls evaluated at height z above the ground
$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h
$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings
$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering shall be treated as an opening in accordance with Section 26.10.3. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$)
G = gust-effect factor, see Section 26.9
C_p = external pressure coefficient from Figs. 27.4-1, 27.4-2 and 27.4-3
(GC_{pi}) = internal pressure coefficient from Table 26.11-1

q and q_i shall be evaluated using exposure defined in Section 26.7.3. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces as defined in Figs. 27.4-1, 27.4-2 and 27.4-3.

Tech 1 | Wind load Calculations | Punit G Das | Page 2

Using Directional procedure for buildings of all heights for Wind loads for MWFRS.

- Method specified in ASCE-7-10, Chapter 27.

Following steps from Table 27.2-1, ASCE 7-10.

(1) Risk Category of building → Table ~~1.5 1~~ 1.5-1 (Chapter 1) → II

Failure of Building has a risk to human life but does not ~~to~~ fall into category III and IV.

(2) • Basic wind speed → $V = 44\ m/s$ or [redacted] 98.4 m/hr

This information is ~~ac~~ supplied by the Structural engineer and ~~is~~ it is ~~the~~ ~~info~~ taken from the wind data & in indian code.

• Exposure Category → D (Section 26.7.3)

The mean roof height is greater than 30 ft and the ground surface roughness ~~does not~~ conform to exposure category D. because the building is close to the sea.

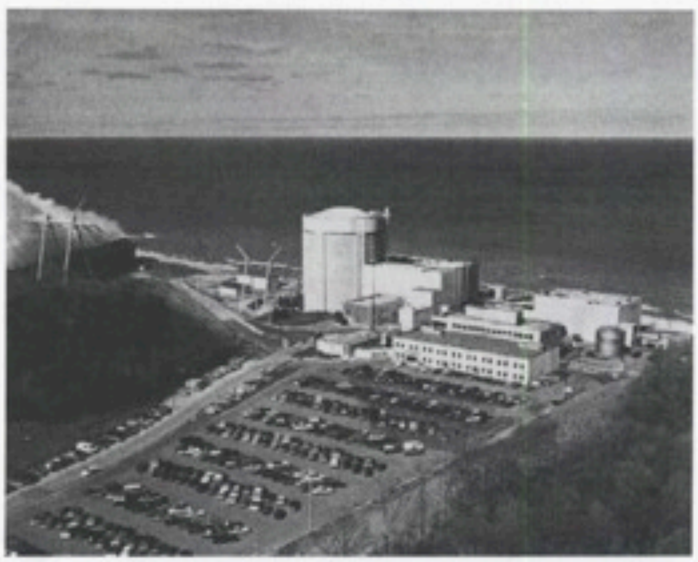
• Topographic factor → ~~$K_{zt} = 0.85$~~ $K_{zt} = 1.0$ (Section 26.8.1 and 26.8.2)

The site topography is relatively flat and does not have hills or escarpments.

• Wind directionality factor → $K_d = 0.85$ (Section 26.6.1 Table 26.6-1.

Pg 3.5

EXPOSURE C OPEN TERRAIN WITH SCATTERED OBSTRUCTIONS HAVING HEIGHTS GENERALLY LESS THAN 30 FT FOR MOST WIND DIRECTIONS, ALL 1-STORY STRUCTURES WITH A MEAN ROOF HEIGHT LESS THAN 30 FT IN THE PHOTOGRAPH ARE LESS THAN 1500 FT OR TEN TIMES THE HEIGHT OF THE STRUCTURE, WHICHEVER IS GREATER, FROM AN OPEN FIELD THAT PREVENTS THE USE OF EXPOSURE B.



EXPOSURE D A BUILDING AT THE SHORELINE (EXCLUDING SHORELINES IN HURRICANE-PRONE REGIONS) WITH WIND FLOWING OVER OPEN WATER FOR A DISTANCE OF AT LEAST 1 MILE. SHORELINES IN EXPOSURE D INCLUDE INLAND WATERWAYS, THE GREAT LAKES, AND COASTAL AREAS OF CALIFORNIA, OREGON, WASHINGTON, AND ALASKA.

Wind load calculations 6

- Enclosure Classification → enclosed building
  because all the wind pressure is applied on the curtain wall that encloses the entire building.
- Internal pressure coefficient → $GC_{pi} = \pm 0.18$
  (from ASCE 7-10, table 26.11-1)

(4) Velocity pressure exposure coefficient

- Height above ground → 252.6 ft
- exposure → D

From Table 26.9-1,

$z_g = 700$ ft, $z = 252.6$ ft

$15\text{ft} < z < 700\text{ft}$

- So, $K_z = 2.01(z/z_g)^{2/\alpha}$, $\alpha = 11.5$

(5) • Velocity Pressure

$q_z = 0.00256\, K_z K_{zt} K_d V^2$ (Refer to excel sheet for calculations)

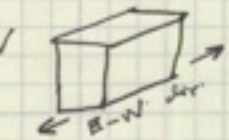
(6) External pressure coefficients

Next page

Wind load calculation 7

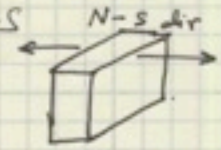
• Wall pressure co-efficients

* Windward (to be used with q_z) → E-W



$\frac{L}{B} = \frac{208.3}{88.56}$, $C_p = 0.8$

→ N-S



$\frac{L}{B} = \frac{88.56}{208.3}$, $C_p = 0.8$

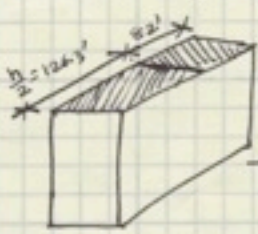
* Leeward for (q_h) → E-W → $\frac{L}{B} = 2.35$, $C_p = -0.28$ (using interpolation)

→ N-S → $\frac{L}{B} = 0.425$ $C_p = -0.5$, with q_h

* Sidewall (for q_h) → $C_p = -0.7$ with q_h

* Roof (for q_h) E-W wind → $\frac{h}{L} = \frac{\cancel{208}\ 252.6}{208.3} = 1.213$

N-S wing → $\frac{h}{L} = \frac{252.6}{88.56} = 2.852$



Roof slope $\theta < 10°$

→ E-W wind → $\frac{h}{L} > 1.0$, 0 to $\frac{h}{2}$ = $C_p = (-1.3)(0.8) = -1.04$ & (-0.18)

$> \frac{h}{2}$ = $C_p = (-0.7)(0.8) = -0.56$ & (-0.18)

Area $(0 \text{ to } \frac{h}{2})$ = (26.3')(88.56') > 1000 sq. ft

→ Reduction factor = 0.8

Area $(> \frac{h}{2})$ = ([illegible]')(88.56') > 1000 sq. ft

Pg 75

CHAPTER 27 WIND LOADS ON BUILDINGS—MWFRS (DIRECTIONAL PROCEDURE)

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-1 (cont.)	External Pressure Coefficients, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
250 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of L/B, h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 26.9.4.
6. Refer to Figure 27.4-2 for domes and Figure 27.4-3 for arched roofs.
7. Notation:
   - B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   - L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   - h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   - z: Height above ground, in feet (meters).
   - G: Gust effect factor.
   - q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   - θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

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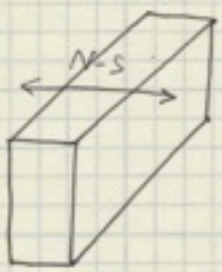
Wind pressure coefficiants (C_p)

	E-W wind	N-S wind
Windward (Use with q_z)	0.8	0.8
Leeward (Use with q_h)	-0.28	-0.5
Sidewall (Use with q_h)	-0.7	-0.7
Roof (Use with q_h) (0 ft to h/2 ft)	-1.04, -0.18	-1.04, -0.18
Roof (Use with q_h) (>h/2 ft)	-0.56, -0.18	-

Wind load calculations 9



N-S wind $\longrightarrow$ $\frac{h}{L} = 2.852 > 1.0$

$0 - \frac{h}{2} = 0$ to $88.56'$, $C_p = -1.3 \times 0.8 = \underline{-1.04, -0.18}$

Area $= (88.56')(208.3') >> 1000$.
Reduction factor $= 0.8$

(7) Wind pressures for flexible building

equation ~~26.7~~ 27.4-2

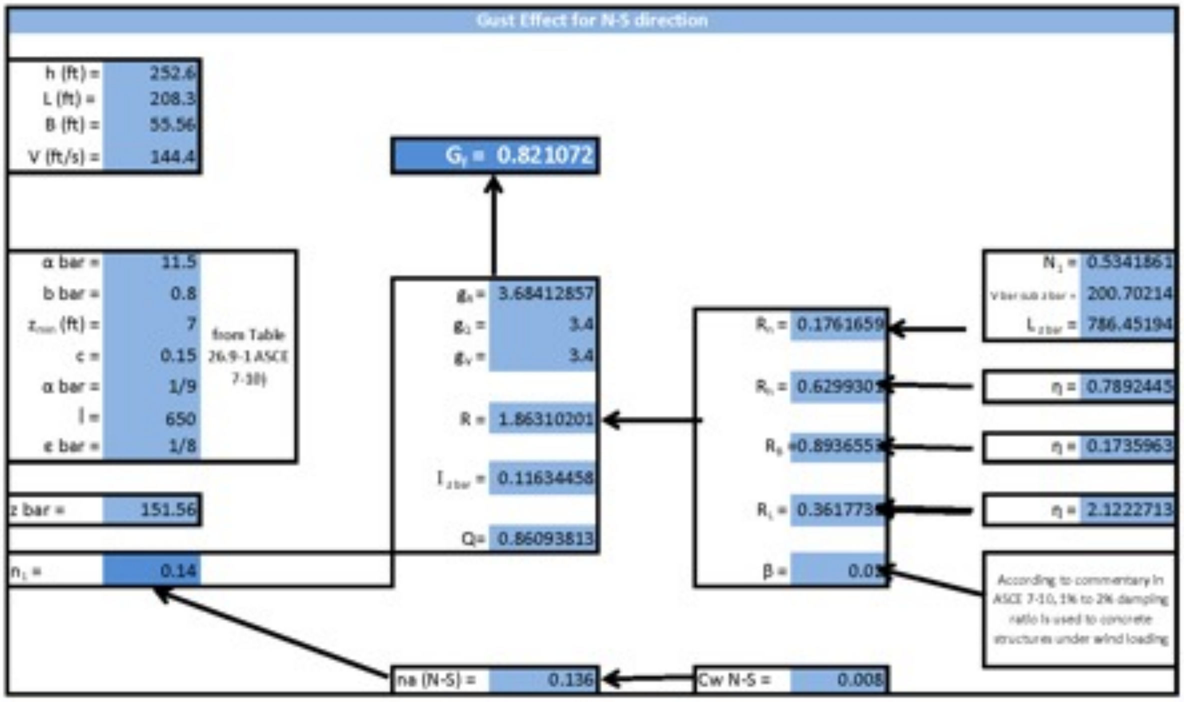
- $p = q G_f C_p - q_i (GC_{pi}) \longrightarrow 27.4\text{-}2$

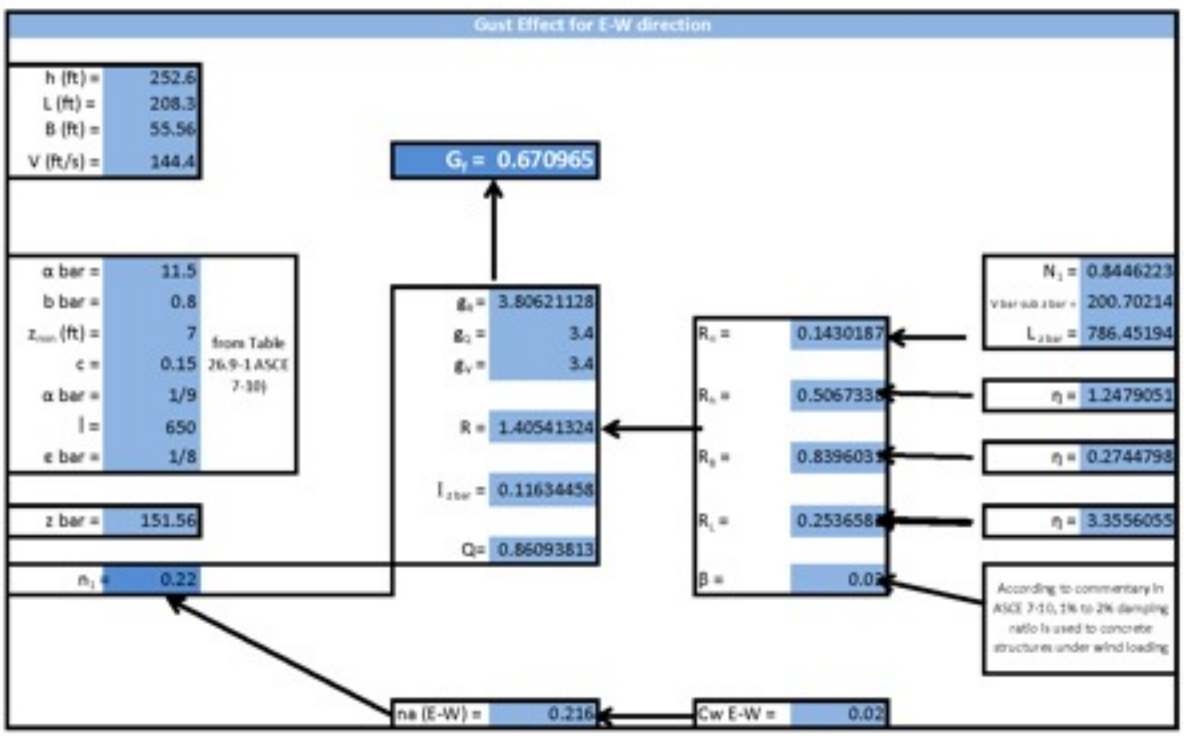
for parapet

$p = q_p (GC_{pi}) \longrightarrow 27.4\text{-}4$

(Refer to excel sheet for calculations)

Velocity pressure Calculation			
Story	Elevation (ft)	K_z	q_z (lb/ft^2)
Ground	0.00	1.03	21.7
1A	20.34	1.09	22.9
2A	30.18	1.16	24.5
3A	40.03	1.22	25.7
5	56.92	1.30	27.4
6	70.05	1.35	28.4
7	83.17	1.39	29.2
8	96.29	1.42	30.0
9	109.42	1.46	30.7
10	122.54	1.48	31.3
11	135.66	1.51	31.8
12	148.79	1.54	32.4
13	161.91	1.56	32.8
14	175.03	1.58	33.3
15	188.16	1.60	33.7
16	201.28	1.62	34.1
17	214.40	1.64	34.5
Roof (level 18)	228.28	1.65	34.9
Parapet Lvl 19	240.09	1.67	35.2
PH parapet	252.62	1.68	35.5
h =	240.09		
K_h =	1.67		
q_h =	75.71		


Gust Effect for N-S direction
h (ft) = 252.6
L (ft) = 208.3
B (ft) = 55.56
V (ft/s) = 144.4
Gf = 0.821072
α bar = 11.5
b bar = 0.8
zmin (ft) = 7
c = 0.15
α bar = 1/9
l = 650
ε bar = 1/8
from Table 26.9-1 ASCE 7-10)
gR = 3.68412857
gQ = 3.4
gv = 3.4
R = 1.86310201
Iz bar = 0.11634458
Q= 0.86093813
Rn = 0.1761659
Rh = 0.6299301
RB = 0.8936655
RL = 0.361773
β = 0.05
N1 = 0.5341861
V bar sub z bar = 200.70214
Lz bar = 786.45194
η = 0.7892445
η = 0.1735963
η = 2.1222713
According to commentary in ASCE 7-10, 1% to 2% damping ratio is used to concrete structures under wind loading
z bar = 151.56
n1 = 0.14
na (N-S) = 0.136
Cw N-S = 0.008


Gust Effect for E-W direction
h (ft) = 252.6
L (ft) = 208.3
B (ft) = 55.56
V (ft/s) = 144.4
Gf = 0.670965
α bar = 11.5
b bar = 0.8
zmin (ft) = 7
c = 0.15
α bar = 1/9
l = 650
ε bar = 1/8
from Table 26.9-1 ASCE 7-10)
gR = 3.80621128
gQ = 3.4
gv = 3.4
R = 1.40541324
Iz bar = 0.11634458
Q= 0.86093813
Rn = 0.1430187
Rh = 0.5067338
RB = 0.8396033
RL = 0.2536588
β = 0.02
N1 = 0.8446223
V bar sub z bar = 200.70214
Lz bar = 786.45194
η = 1.2479051
η = 0.2744798
η = 3.3556055
According to commentary in ASCE 7-10, 1% to 2% damping ratio is used to concrete structures under wind loading
z bar = 151.56
n1 = 0.22
na (E-W) = 0.216
Cw E-W = 0.02

Approximate fundamental period for wind loads								
Area of base	AB=	20557.6		structural height		hn = (ft)	228.3	
Shear Wall name	Area	Area (ft^2)	Height	height ft (hi)		Di		Σ
X1	**A1**	**61.57**	**h1**	**228.3**	**D1**	**47**	2.9912054	
X2	A2	19.00	h2	228.30	D2	14.50	0.09	3.09
X3	A3	7.68	h3	228.30	D3	7.80	0.01	
Y1	A4	38.21	h4	228.30	D4	23.30	0.47	
Y2	A5	7.73	h5	228.30	D5	5.90	0.01	
Y3	A6	12.97	h6	228.30	D6	9.90	0.03	
Y4	A7	9.96	h7	228.30	D7	7.60	0.01	
Y5	A8	5.81	h8	228.30	D8	5.90	0.00	
Y6	A9	11.92	h9	228.30	D9	9.10	0.02	
Y7	A10	22.01	h10	228.30	D10	16.80	0.14	1.57
Y8	A11	4.25	h11	228.30	D11	8.50	0.01	
Y9	A12	9.78	h12	228.30	D12	8.50	0.02	
Y10	A13	4.25	h13	228.30	D13	8.50	0.01	
Y11	A14	22.89	h14	228.30	D14	19.90	0.21	
Y12	A15	11.97	h15	228.30	D15	14.60	0.06	
Y13	A16	23.00	h16	228.30	D16	20.00	0.21	
Y14	A17	30.39	h17	228.30	D17	23.20	0.37	
E-W direction	Ta=	3.54	seconds	**N-S Direction**	Ta=	4.96	seconds	
	Cw=	**0.015**			**Cw=**	**0.008**		

N-S Direction							
Windward pressure Cp =0.8							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure		**Net pressure (+)**	**Net pressure (-)**
				+Gcpi*qi	-Gcpi*qi		
Ground	0.00	21.7	**14.3**	13.6	-13.6	**27.9**	**0.6**
1A	20.34	22.9	**15.0**	13.6	-13.6	**28.7**	**1.4**
2A	30.18	24.5	**16.1**	13.6	-13.6	**29.7**	**2.5**
3A	40.03	25.7	**16.9**	13.6	-13.6	**30.5**	**3.3**
5	56.92	27.4	**18.0**	13.6	-13.6	**31.6**	**4.4**
6	70.05	28.4	**18.6**	13.6	-13.6	**32.3**	**5.0**
7	83.17	29.2	**19.2**	13.6	-13.6	**32.8**	**5.6**
8	96.29	30.0	**19.7**	13.6	-13.6	**33.3**	**6.1**
9	109.42	30.7	**20.1**	13.6	-13.6	**33.8**	**6.5**
10	122.54	31.3	**20.5**	13.6	-13.6	**34.2**	**6.9**
11	135.66	31.8	**20.9**	13.6	-13.6	**34.5**	**7.3**
12	148.79	32.4	**21.2**	13.6	-13.6	**34.9**	**7.6**
13	161.91	32.8	**21.6**	13.6	-13.6	**35.2**	**7.9**
14	175.03	33.3	**21.9**	13.6	-13.6	**35.5**	**8.2**
15	188.16	33.7	**22.1**	13.6	-13.6	**35.8**	**8.5**
16	201.28	34.1	**22.4**	13.6	-13.6	**36.0**	**8.8**
17	214.40	34.5	**22.6**	13.6	-13.6	**36.3**	**9.0**
Roof (level 18)	228.28	34.9	**22.9**	13.6	-13.6	**36.5**	**9.3**
Parapet Lvl 19	240.09	35.2	**52.7**	-	-	**52.7**	**-**
PH parapet	252.62	35.5	**53.2**	-	-	**53.2**	**-**
Leeward pressure Cp =-0.5							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure		**Net pressure (+)**	**Net pressure (-)**
				+Gcpi*qi	-Gcpi*qi		
All	240.09	75.7	**-31.1**	13.6	-13.6	**-17.5**	**-44.7**
Parapet (level 19 and top)	252.62	35.5	**-35.5**	-	-	**-35.5**	**-**
Side wall pressure Cp =-0.7							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure		**Net pressure (+)**	**Net pressure (-)**
				+Gcpi*qi	-Gcpi*qi		
All	240.09	75.7	**-43.5**	13.6	-13.6	**-29.9**	**-57.1**
Roof Pressures Cp=-1.04, -0.18							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure		**Net pressure (+)**	**Net pressure (-)**
				+Gcpi*qi	-Gcpi*qi		
0 to h/2 (Cp=-1.04)	240.09	75.7	**-64.7**	13.6	-13.6	**-51.0**	**-78.3**
0 to h/2 (Cp=-0.18)	240.09	75.7	**-11.2**	13.6	-13.6	**2.4**	**-24.8**

E-W Direction							
Windward pressure Cp =0.8							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure (+)**	**Net pressure (-)**
Ground	0.00	21.7	**11.6**	13.6	-13.6	**25.3**	**-2.0**
1A	20.34	22.9	**12.3**	13.6	-13.6	**25.9**	**-1.3**
2A	30.18	24.5	**13.2**	13.6	-13.6	**26.8**	**-0.5**
3A	40.03	25.7	**13.8**	13.6	-13.6	**27.4**	**0.2**
5	56.92	27.4	**14.7**	13.6	-13.6	**28.3**	**1.1**
6	70.05	28.4	**15.2**	13.6	-13.6	**28.9**	**1.6**
7	83.17	29.2	**15.7**	13.6	-13.6	**29.3**	**2.1**
8	96.29	30.0	**16.1**	13.6	-13.6	**29.7**	**2.5**
9	109.42	30.7	**16.5**	13.6	-13.6	**30.1**	**2.8**
10	122.54	31.3	**16.8**	13.6	-13.6	**30.4**	**3.2**
11	135.66	31.8	**17.1**	13.6	-13.6	**30.7**	**3.5**
12	148.79	32.4	**17.4**	13.6	-13.6	**31.0**	**3.7**
13	161.91	32.8	**17.6**	13.6	-13.6	**31.3**	**4.0**
14	175.03	33.3	**17.9**	13.6	-13.6	**31.5**	**4.2**
15	188.16	33.7	**18.1**	13.6	-13.6	**31.7**	**4.5**
16	201.28	34.1	**18.3**	13.6	-13.6	**31.9**	**4.7**
17	214.40	34.5	**18.5**	13.6	-13.6	**32.1**	**4.9**
Roof (level 18)	228.28	34.9	**18.7**	13.6	-13.6	**32.3**	**5.1**
Parapet Lvl 19	240.09	35.2	**52.7**	-	-	**52.7**	**-**
PH parapet	252.62	35.5	**53.2**	-	-	**53.2**	**-**
Leeward pressure Cp =-0.28							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure (+)**	**Net pressure (-)**
All	240.09	35.2	**-45.1**	6.3	-6.3	**-38.8**	**-51.4**
Parapet (level 19 and top)	252.62	35.5	**-35.5**	-	-	**-35.5**	**-**
Side wall pressure Cp =-0.7							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure (+)**	**Net pressure (-)**
All	240.09	75.7	**-35.6**	13.6	-13.6	**-21.9**	**-49.2**
Roof Pressures Cp=-0.56, -0.18							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure ($q*G_f*C_p$)**	internal pressure +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure (+)**	**Net pressure (-)**
0 to h/2 (Cp=-1.04)	240.09	75.7	**-52.8**	13.6	-13.6	**-39.2**	**-66.5**
0 to h/2 (Cp=-0.18)	240.1	75.7	**-52.8**	13.6	-13.6	**-39.2**	**-66.5**
>h/2 (Cp=-0.56)	240.09	75.7	**-52.8**	13.6	-13.6	**-39.2**	**-66.5**
>h/2 (Cp=-0.18)	240.09	75.7	**-52.8**	13.6	-13.6	**-39.2**	**-66.5**

N-S Direction						
Story Force due to Windward pressure						
Level	Elevation (ft)	**Net Wind pressure (psf)**	Area below (ft^2)	Area above (ft^2)	Tributary area	**Story shear (kip)**
Ground	0.00	27.9	**0.0**	2072.8	2072.8	**57.8**
1A	20.34	28.7	**2072.8**	1003.0	3075.7	**88.2**
2A	30.18	29.7	**1003.0**	1003.0	2005.9	**59.6**
3A	40.03	30.5	**1003.0**	1721.7	2724.7	**83.2**
5	56.92	31.6	**1721.7**	1337.3	3059.0	**96.7**
6	70.05	32.3	**1337.3**	1337.3	2674.5	**86.3**
7	83.17	32.8	**1337.3**	1337.3	2674.5	**87.8**
8	96.29	33.3	**1337.3**	1337.3	2674.5	**89.1**
9	109.42	33.8	**1337.3**	1337.3	2674.5	**90.3**
10	122.54	34.2	**1337.3**	1337.3	2674.5	**91.4**
11	135.66	34.5	**1337.3**	1337.3	2674.5	**92.4**
12	148.79	34.9	**1337.3**	1337.3	2674.5	**93.3**
13	161.91	35.2	**1337.3**	1337.3	2674.5	**94.1**
14	175.03	35.5	**1337.3**	1337.3	2674.5	**94.9**
15	188.16	35.8	**1337.3**	1337.3	2674.5	**95.7**
16	201.28	36.0	**1337.3**	1337.3	2674.5	**96.3**
17	214.40	36.3	**1337.3**	1414.2	2751.4	**99.8**
Roof (level 18)	228.28	36.5	**1414.2**	1203.5	2617.7	**95.6**
Parapet Lvl 19	240.09	52.7	**1203.5**	1277.09	2480.64	**130.8**
PH parapet	252.62	53.2	**1277.1**	0	1277.09	**67.9**
					Base Shear	**1791.3**
Story Forces due to Leeward pressure						
Level	Elevation (ft)	**Net Wind pressure (psf)**	Tributary area	**Story shear (kip)**		
All	240	45	2674.5	**120**		
Parapet (level 19 and top)	252.6	35.5	2610.6	**92.6**	**Base Shear**	212.2

E-W Direction						
Story Force due to Windward pressure						
Level	Elevation (ft)	**Net Wind pressure (psf)**	Area below (ft^2)	Area above (ft^2)	Tributary area	**Story shear (kip)**
Ground	0.00	25.3	**0.0**	900.7	900.7	**22.8**
1A	20.34	25.9	**900.7**	435.8	1336.5	**34.6**
2A	30.18	26.8	**435.8**	435.8	871.7	**23.3**
3A	40.03	27.4	**435.8**	748.2	1184.0	**32.5**
5	56.92	28.3	**748.2**	581.1	1329.3	**37.6**
6	70.05	28.9	**581.1**	581.1	1162.2	**33.5**
7	83.17	29.3	**581.1**	581.1	1162.2	**34.1**
8	96.29	29.7	**581.1**	581.1	1162.2	**34.5**
9	109.42	30.1	**581.1**	581.1	1162.2	**35.0**
10	122.54	30.4	**581.1**	581.1	1162.2	**35.4**
11	135.66	30.7	**581.1**	581.1	1162.2	**35.7**
12	148.79	31.0	**581.1**	581.1	1162.2	**36.0**
13	161.91	31.3	**581.1**	581.1	1162.2	**36.3**
14	175.03	31.5	**581.1**	581.1	1162.2	**36.6**
15	188.16	31.7	**581.1**	581.1	1162.2	**36.9**
16	201.28	31.9	**581.1**	581.1	1162.2	**37.1**
17	214.40	32.1	**581.1**	614.5	1195.6	**38.4**
Roof (level 18)	228.28	32.3	**614.5**	523.0	1137.5	**36.8**
Parapet Lvl 19	240.09	52.7	**523.0**	554.95	1077.94	**56.8**
PH parapet	252.62	53.2	**555.0**	0	554.95	**29.5**
					Base Shear	**703.6**
Story Forces due to Leeward pressure						
Level	Elevation (ft)	**Net Wind pressure (psf)**	Tributary area	**Story shear (kip)**		
All	240	51	1162	**60**		
Parapet (level 19 and top)	252.6	35.5	1109.9	**39.4**	**Base Shear**	99.1

Case 1 North-South Wind									
Total K for direct shear	3452.87		Windward force -	87.82		Leeward force -	118.58		
Shear Wall	d ft	Ki (kip/in)	Ki d² (kip ft)	Σ(ki d²) (kip-ft)	P	Moment arm	Direct Shear	Torsional shear	Total Shear
X1	3.35	989.27	11069				0.00	0.82	0.82
X2	5.79	304.98	10224				0.00	0.44	0.44
X3	-13.06	123.00	20988				0.00	-0.40	-0.40
Y1	-111.91	613.83	7686779				-36.87	-16.96	-53.83
Y2	-86.56	123.61	926047				-7.42	-2.64	-10.07
Y3	-86.56	208.04	1558613				-12.50	-4.45	-16.94
Y4	-74.75	156.52	891228				-9.58	-2.94	-12.53
Y5	-74.75	92.85	518725				-5.58	-1.71	-7.29
Y6	-59.14	191.17	668522	14868314	207.4	17.7	-11.48	-2.79	-14.27
Y7	-61.33	351.43	1417277				-21.23	-5.53	-26.75
Y8	-21.22	68.14	30662				-4.09	-0.36	-4.45
Y9	-9.75	156.71	14901				-6.41	-0.38	-6.79
Y10	1.72	68.14	200				-4.09	0.03	-4.06
Y11	12.79	367.58	60130				-22.08	1.16	-20.92
Y12	22.25	192.22	95120				-11.55	1.06	-10.49
Y13	26.16	369.41	252814				-22.19	2.39	-19.80
Y14	38.00	488.21	704671				-29.32	4.58	-24.74
								Max shear	-53.8

Case 1 East-West Wind									
Total K for direct shear	1417.2		Windward force -	38.1		Leeward force -	58.8		
Shear Wall	d ft	Ki (kip/in)	Ki d² (kip ft)	Σ(ki d²) (kip-ft)	P	Moment arm	Direct Shear	Torsional shear	Total Shear
X1	3.35	989.27	11069				-65.51	0.26	-65.25
X2	5.79	304.98	10224				-20.20	0.14	-20.06
X3	-13.06	123.00	20988				-8.15	-0.13	-8.27
Y1	-111.91	613.83	7686779				0.00	-5.48	-5.48
Y2	-86.56	123.61	926047				0.00	-0.85	-0.85
Y3	-86.56	208.04	1558613				0.00	-1.44	-1.44
Y4	-74.75	156.52	891228				0.00	-0.95	-0.95
Y5	-74.75	92.85	518725				0.00	-0.55	-0.55
Y6	-59.14	191.17	668522	14868314	93.9	12.63	0.00	-0.90	-0.90
Y7	-61.33	351.43	1417277				0.00	-1.78	-1.78
Y8	-21.22	68.14	30662				0.00	-0.12	-0.12
Y9	-9.75	156.71	14901				0.00	-0.12	-0.12
Y10	1.72	68.14	200				0.00	0.01	0.01
Y11	12.79	367.58	60130				0.00	0.17	0.17
Y12	22.25	192.22	95120				0.00	0.34	0.34
Y13	26.56	366.41	252854				0.00	0.77	0.77
Y14	38.00	488.21	704671				0.00	1.48	1.48
								Max shear	-65.2

Case 2: North-South Wind + eccentricity									
Total K for direct shear	3452.87		By-	208.90	ft	ey-	31.25		
Shear Wall	d ft	Ki (kip/in)	Ki d² (kip ft)	Σ(ki d²) (kip-ft)	75% P	moment arm	Direct shear	Torsional shear	total shear
X1	3.35	989.27	11069				0.00	1.69	1.69
X2	5.79	304.98	10224				0.00	0.90	0.90
X3	-13.06	123.00	20988				0.00	-0.82	-0.82
Y1	-111.91	613.83	7686779				-27.65	-35.17	-62.82
Y2	-86.56	123.61	926047				-5.57	-5.48	-11.05
Y3	-86.56	208.04	1558613				-9.37	-9.22	-18.59
Y4	-74.75	156.52	891228				-7.19	-6.11	-13.29
Y5	-74.75	92.85	518725				-4.18	-3.55	-7.74
Y6	-59.14	191.17	668522	14868314	155.5	48.645	-8.61	-5.79	-14.40
Y7	-61.33	351.43	1417277				-15.92	-11.46	-27.38
Y8	-21.22	68.54	30662				-3.07	-0.74	-3.81
Y9	-9.75	156.71	14901				-7.06	-0.78	-7.84
Y10	1.72	68.54	200				-3.07	0.06	-3.01
Y11	12.79	367.58	60530				-16.56	2.41	-14.15
Y12	22.25	592.22	95520				-8.66	2.19	-6.47
Y13	26.16	369.40	252814				-16.64	4.95	-11.69
Y14	38.00	488.21	704671				-21.99	9.50	-12.49
								Max shear	-62.8

Case 2: North-South Wind - eccentricity									
Total K for direct shear	3452.87		By-	208.90	ft	ey-	-31.25		
Shear Wall	d ft	Ki (kip/in)	Ki d² (kip ft)	Σ(ki d²) (kip-ft)	75% P	moment arm	Direct shear	Torsional shear	total shear
X1	3.35	989.27	11069				0.00	-0.47	-0.47
X2	5.79	304.98	10224				0.00	-0.25	-0.25
X3	-13.06	123.00	20988				0.00	0.23	0.23
Y1	-111.91	613.83	7686779				-27.65	9.73	-17.92
Y2	-86.56	123.61	926047				-5.57	1.52	-4.05
Y3	-86.56	208.04	1558613				-9.37	2.55	-6.82
Y4	-74.75	156.52	891228				-7.19	1.69	-5.50
Y5	-74.75	92.85	518725				-4.18	0.98	-3.20
Y6	-59.14	191.17	668522	14868314	155.5	-13.345	-8.61	1.60	-7.01
Y7	-61.33	351.43	1417277				-15.92	3.17	-12.75
Y8	-21.22	68.54	30662				-3.07	0.20	-2.86
Y9	-9.75	156.71	14901				-7.06	0.22	-6.84
Y10	1.72	68.54	200				-3.07	-0.02	-3.09
Y11	12.79	367.58	60530				-16.56	-0.67	-17.23
Y12	22.25	592.22	95520				-8.66	-0.61	-9.27
Y13	26.16	369.40	252814				-16.64	-1.37	-18.01
Y14	38.00	488.21	704671				-21.99	-2.63	-24.62
								Max shear	-28.8

Case 2: East-West Wind + eccentricity									
Total K for direct shear	1417.25		Bx-	88.56	ft	ex	13.28		
Shear Wall	d ft	Ki (kip/in)	Ki d² (kip ft)	Σ(ki d²) (kip-ft)	75% P	moment arm	Direct shear	Torsional shear	total shear
X1	3.35	989.27	11069				-49.54	0.41	-48.73
X2	5.79	304.98	10224				-15.15	0.22	-14.93
X3	-13.06	123.00	20988				-6.11	-0.20	-6.31
Y1	-111.91	613.83	7686779				0.00	-8.40	-8.40
Y2	-86.56	123.61	926047				0.00	-1.31	-1.31
Y3	-86.56	208.04	1558613				0.00	-2.21	-2.21
Y4	-74.75	156.52	891228				0.00	-1.46	-1.46
Y5	-74.75	92.85	518725				0.00	-0.85	-0.85
Y6	-59.14	191.17	668522	14868314	70.4	25.654	0.00	-1.39	-1.39
Y7	-61.33	351.43	1417277				0.00	-2.75	-2.75
Y8	-21.22	68.14	30662				0.00	-0.18	-0.18
Y9	-9.75	156.71	14901				0.00	-0.19	-0.19
Y10	1.72	68.14	200				0.00	0.05	0.05
Y11	12.79	367.58	60130				0.00	0.58	0.58
Y12	22.25	192.22	95120				0.00	0.52	0.52
Y13	26.16	369.41	252854				0.00	1.19	1.19
Y14	38.00	488.21	704671				0.00	2.28	2.28
								Max shear	-48.7

Case 2: East-West Wind - eccentricity									
Total K for direct shear	1417.25		Bx	88.56	ft	ex	-13.28		
Shear Wall	d ft	Ki (kip/in)	Ki d² (kip ft)	Σ(ki d²) (kip-ft)	75% P	moment arm	Direct shear	Torsional shear	total shear
X1	3.35	989.27	11069				-49.54	0.41	-48.73
X2	5.79	304.98	10224				-15.15	0.22	-14.93
X3	-13.06	123.00	20988				-6.11	-0.20	-6.31
Y1	-111.91	613.83	7686779				0.00	-8.40	-8.40
Y2	-86.56	123.61	926047				0.00	-1.31	-1.31
Y3	-86.56	208.04	1558613				0.00	-2.21	-2.21
Y4	-74.75	156.52	891228				0.00	-1.46	-1.46
Y5	-74.75	92.85	518725				0.00	-0.85	-0.85
Y6	-59.14	191.17	668522	14868314	70.4	-0.654	0.00	-1.39	-1.39
Y7	-61.33	351.43	1417277				0.00	-2.75	-2.75
Y8	-21.22	68.14	30662				0.00	-0.18	-0.18
Y9	-9.75	156.71	14901				0.00	-0.19	-0.19
Y10	1.72	68.14	200				0.00	0.05	0.05
Y11	12.79	367.58	60130				0.00	0.58	0.58
Y12	22.25	192.22	95120				0.00	0.52	0.52
Y13	26.16	369.41	252854				0.00	1.19	1.19
Y14	38.00	488.21	704671				0.00	2.28	2.28
								Max shear	-48.7

Case 2: North-South Wind - eccentricity									
Total K for direct shear	3452.87		By=	208.30	ft	ey=	-31.25		
Shear Wall	d ft	Ki (kip/in)	Ki di^2(kip ft)	ΣKi di^2(kip ft)	75% P	moment arm	Direct shear	Torsional shear	total shear
X1	3.35	989.27	11069	14868314	125.1	-13.545	0.00	-0.38	-0.38
X2	5.79	304.98	10224				0.00	-0.20	-0.20
X3	-13.06	123.00	20988				0.00	0.18	0.18
Y1	-111.91	613.83	7686779				-22.23	7.83	-14.41
Y2	-86.56	123.61	926047				-4.48	1.22	-3.26
Y3	-86.56	208.04	1558613				-7.54	2.05	-5.48
Y4	-74.75	159.52	891228				-5.78	1.36	-4.42
Y5	-74.75	92.85	518725				-3.36	0.79	-2.57
Y6	-59.14	191.17	668522				-6.92	1.29	-5.64
Y7	-63.33	353.43	1417277				-12.80	2.55	-10.25
Y8	-21.22	68.14	30692				-2.47	0.16	-2.30
Y9	-9.75	156.71	14913				-5.68	0.17	-5.50
Y10	1.72	68.14	200				-2.47	-0.01	-2.48
Y11	12.79	367.58	60130				-13.31	-0.54	-13.85
Y12	22.25	192.22	95120				-6.96	-0.49	-7.45
Y13	26.16	369.43	252814				-13.38	-1.10	-14.48
Y14	38.00	488.21	704971				-17.68	-2.11	-19.80
								Max shear	-19.8

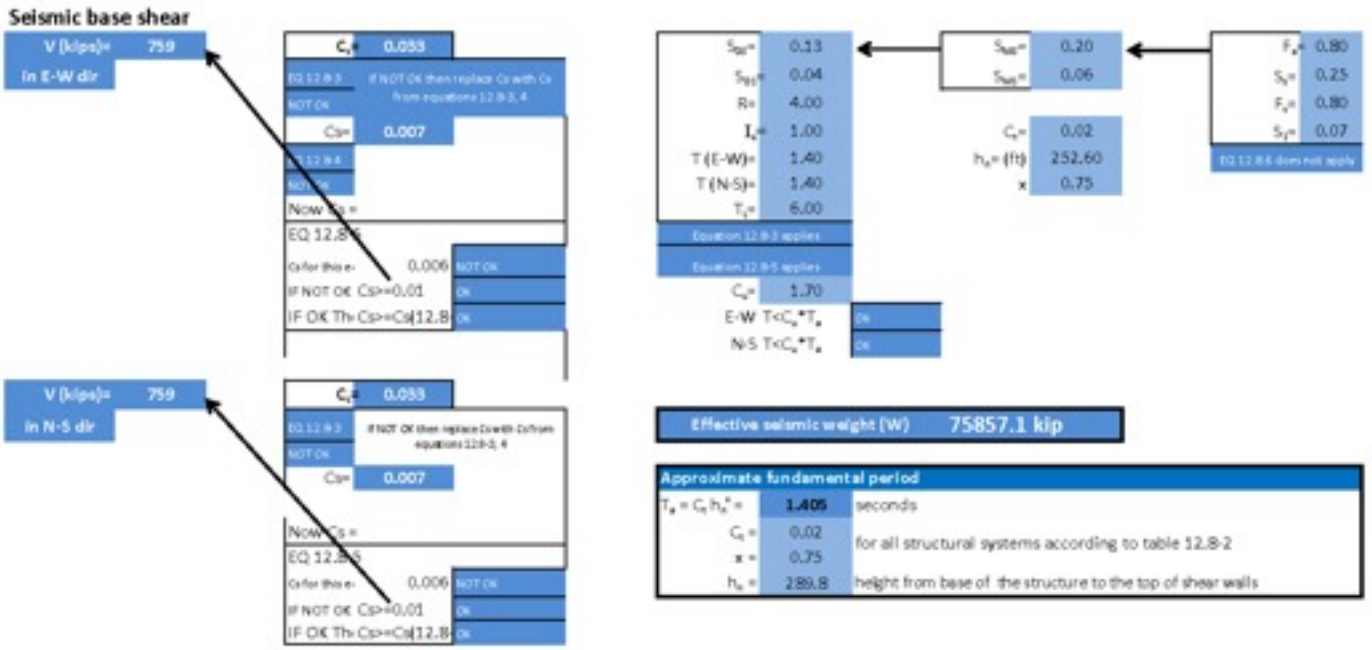
Case 2: East-West Wind - eccentricity									
Total K for direct shear	1417.25		Bx	88.56	ft	ex	-13.28		
Shear Wall	d ft	Ki (kip/in)	Ki di^2(kip ft)	ΣKi di^2(kip ft)	75% P	moment arm	Direct shear	Torsional shear	total shear
X1	3.35	989.27	11069	14868314	62.6	-0.654	-43.73	0.36	-43.37
X2	5.79	304.98	10224				-13.48	0.19	-13.29
X3	-13.06	123.00	20988				-5.44	-0.18	-5.61
Y1	-111.91	613.83	7686779				0.00	-7.50	-7.50
Y2	-86.56	123.61	926047				0.00	-1.17	-1.17
Y3	-86.56	208.04	1558613				0.00	-1.97	-1.97
Y4	-74.75	159.52	891228				0.00	-1.30	-1.30
Y5	-74.75	92.85	518725				0.00	-0.76	-0.76
Y6	-59.14	191.17	668522				0.00	-1.23	-1.23
Y7	-63.33	353.43	1417277				0.00	-2.44	-2.44
Y8	-21.22	68.14	30692				0.00	-0.16	-0.16
Y9	-9.75	156.71	14913				0.00	-0.17	-0.17
Y10	1.72	68.14	200				0.00	0.01	0.01
Y11	12.79	367.58	60130				0.00	0.51	0.51
Y12	22.25	192.22	95120				0.00	0.47	0.47
Y13	26.16	369.43	252814				0.00	1.06	1.06
Y14	38.00	488.21	704971				0.00	2.03	2.03
								Max shear	-43.4

Case 3: 75% N-S wind + 75% E-W wind													
Total K for direct shear NS		3452.9		Total K for EW direction		1417.2							
Shear Wall	d ft	Ki (kip/in)	Ki d_i^2 (kip ft)	$\Sigma K_i d_i^2$ (kip ft)	75%P NS	75%P EW	Moment arm NS	Moment arm EW	Direct Shear	Total Moment	Torsional shear	total shear
X1	3.35	989.27	11069						-49		1	-48
X2	5.79	304.98	10224						-15		0	-15
X3	-13.06	123.00	20988						-6		0	-7
Y1	-111.91	613.83	7686779						-28		-17	-44
Y2	-86.56	123.61	926047						-6		-3	-8
Y3	-86.56	208.04	1558613						-9		-4	-14
Y4	-74.75	159.52	891228						-7		-3	-10
Y5	-74.75	92.85	518725						-4		-2	-6
Y6	-59.14	191.17	668522	14868314	155.5	70.4	17.7	12.63	-9	3642.2429	-3	-11
Y7	-63.33	353.43	1417277						-16		-5	-21
Y8	-21.22	68.14	30692						-3		0	-3
Y9	-9.75	156.71	14913						-7		0	-7
Y10	1.72	68.14	200						-3		0	-3
Y11	12.79	367.58	60130						-17		1	-15
Y12	22.25	192.22	95120						-9		1	-8
Y13	26.16	369.43	252814						-17		2	-14
Y14	38.00	488.21	704971						-22		5	-17
											Max shear	-48.3

Case 4: +75% NS wind with eccentricity - 75% EW wind with eccentricity												
Total K for direct shear NS	3452.9				By= 208.3 ft		ey= 31.2 ft					
Total K for EW direction	1417.2				Bx= 88.6 ft		ex -13.3 ft					
Shear Wall	d ft	Ki (kip/in)	Ki d_i^2 (kip ft)	$\Sigma K_i d_i^2$ (kip ft)	75%P NS	75%P EW	Moment arm NS	Moment arm EW	Direct Shear	Total Moment	Torsional shear	total shear
X1	3.35	989.27	11069						-49		2	-47
X2	5.79	304.98	10224						-15		1	-14
X3	-13.06	123.00	20988						-6		-1	-7
Y1	-111.91	613.83	7686779						-13		-35	-47
Y2	-86.56	123.61	926047						-3		-5	-8
Y3	-86.56	208.04	1558613						-4		-9	-13
Y4	-74.75	159.52	891228						-3		-6	-9
Y5	-74.75	92.85	518725						-2		-4	-5
Y6	-59.14	191.17	668522	14868314	155.5	70.4	48.945	-0.654	-4	7567.2302	-6	-10
Y7	-63.33	353.43	1417277						-7		-11	-19
Y8	-21.22	68.14	30692						-1		-1	-2
Y9	-9.75	156.71	14913						-3		-1	-4
Y10	1.72	68.14	200						-1		0	-1
Y11	12.79	367.58	60130						-7		2	-5
Y12	22.25	192.22	95120						-4		2	-2
Y13	26.16	369.43	252814						-8		5	-3
Y14	38.00	488.21	704971						-10		9	-1
										Max shear	-47.5	

Case 4: -75% NS wind with ecentricity + 75% EW wind with eccentricity												
Total K for direct shear	3452.9				By= 208.3 ft		ey= -31.2					
Total K for EW direction	1417.2				Bx= 88.6 ft		ex 13.3					
Shear Wall	d ft	Ki (kip/in)	Ki d_i^2 (kip ft)	$\Sigma K_i d_i^2$ (kip ft)	75%P NS	75%P EW	Moment arm NS	Moment arm EW	Direct Shear	Total Moment	Torsional shear	total shear
X1	3.35	989.27	11069						-49		-0.06	-49
X2	5.79	304.98	10224						-15		-0.03	-15
X3	-13.06	123.00	20988						-6		0.03	-6
Y1	-111.91	613.83	7686779						-13		1.31	-11
Y2	-86.56	123.61	926047						-3		0.20	-2
Y3	-86.56	208.04	1558613						-4		0.34	-4
Y4	-74.75	159.52	891228						-3		0.23	-3
Y5	-74.75	92.85	518725						-2		0.13	-2
Y6	-59.14	191.17	668522	14868314	155.5	70.4	-13.545	25.914	-4	-282.74	0.21	-4
Y7	-63.33	353.43	1417277						-7		0.43	-7
Y8	-21.22	68.14	30692						-1		0.03	-1
Y9	-9.75	156.71	14913						-3		0.03	-3
Y10	1.72	68.14	200						-1		0.00	-1
Y11	12.79	367.58	60130						-7		-0.09	-8
Y12	22.25	192.22	95120						-4		-0.08	-4
Y13	26.16	369.43	252814						-8		-0.18	-8
Y14	38.00	488.21	704971						-10		-0.35	-10
										Max shear	-49.2	

Appendix 2: Seismic Loads



Effective seismic weight							
Floor	**Slab**	**beams**	**shear walls**	**SDL**	**live loads**	**façade**	**Total**
1A 2A	2302.0	721.8	491.5	752.4	1541.8	9.7	11638.3
3A	2302.0	721.8	491.5	667.9	1368.7	9.7	5561.5
5	2064.4	1249.3	491.5	1289.0	513.9	9.7	5617.7
6 8	1248.8	194.3	491.5	779.8	1243.7	8.6	7933.3
7 9	1460.8	194.3	491.5	777.41	325.76	8.6	6516.5
10 12 14 16	1248.8	194.3	491.5	779.8	1243.7	8.6	15866.6
11 13 15 17	1547.7	194.3	491.5	885.6	1412.5	8.6	18160.5
Roof (level 18)	2064.4	194.3	491.5	1289.0	513.9	9.7	4562.7
			Effective seismic weight				**75857.1**

Center of Mass at LVL7						
Load distribution		Load (PSF or LF)		total weight (kip)	COM	
		DL (kip)	SDL (kip)		X	Y
Area of slab	A	662.56	356.10	1018.67	78.40	18.50
	B	41.75	26.60	68.35	15.35	43.91
	C	89.67	38.09	127.76	17.20	62.07
	D	59.82	38.12	97.94	48.80	69.95
	E	30.09	9.58	39.67	35.90	51.70
	F	11.74	9.98	21.72	53.20	44.80
	G	36.49	20.67	57.16	70.85	46.85
	H	30.65	19.53	50.18	105.65	40.50
	I	16.89	6.62	23.51	133.90	41.40
	J	32.56	13.83	46.39	131.05	65.17
	K	23.97	10.18	34.15	112.92	48.60
	L	112.85	61.29	174.14	169.90	69.82
	M	311.76	166.81	478.57	178.50	30.65
Length of Shear wall	X1	99.74		99.74	108.10	53.25
	X2	30.77		30.77	137.70	56.35
	X3	12.43		12.43	147.70	37.59
	Y1	61.90		61.90	6.02	61.85
	Y2	12.52		12.52	31.36	58.84
	Y3	21.01		21.01	31.36	42.05
	Y4	16.13		16.13	43.17	40.90
	Y5	9.41		9.41	43.00	58.88
	Y6	19.31		19.31	58.78	41.65
	Y7	35.65		35.65	85.29	45.50
	Y8	6.89		6.89	96.69	48.25
	Y9	15.84		15.84	108.17	48.25
	Y10	6.89		6.89	108.99	48.25
	Y11	37.07		37.07	119.17	47.05
	Y12	19.39		19.39	128.62	49.80
	Y13	37.26		37.26	132.46	47.10
	Y14	49.24		49.24	144.22	48.70
	Total weight		2729.65			
	COM in X direction			100.2	ft	
	COM in Y direction			37.3	ft	

Center of rigidity at LVL7					
Shear Wall	K	x	y	%K	K * (x or y)
X1	989	108.10	53.25	69.80	106939.94
X2	305	137.70	56.35	21.52	41996.23
X3	123	147.70	37.59	8.68	18166.55
Y1	614	6.02	61.85	17.78	37965.15
Y2	124	31.36	58.84	3.58	7273.13
Y3	208	31.36	42.05	6.03	8748.23
Y4	160	43.17	40.90	4.62	6524.51
Y5	93	43.00	58.88	2.69	5466.89
Y6	191	58.78	41.65	5.54	7962.35
Y7	353	85.29	45.50	10.24	16081.10
Y8	68	96.69	48.25	1.97	3287.56
Y9	157	108.17	48.25	4.54	7561.39
Y10	68	108.99	48.25	1.97	3287.56
Y11	368	119.17	47.05	10.65	17294.47
Y12	192	128.62	49.80	5.57	9572.77
Y13	369	132.46	47.10	10.70	17399.92
Y14	488	144.22	48.70	14.14	23775.67
Total stiffness in X	1417.25				
Total stiffness in Y	3452.87				
COR in X	**117.91**				
COR in Y	**49.87**				

Stiffness of shear walls on LVL 7									
Shear Wall	h	thickness (in)	length (l) inches	A	Ix	Iy	E	G	K
X1	95	15.72	564.00	61.57	235022049		4030	1831.8	989.3
X2	95	15.72	174.00	19.00	6901111		4030	1831.8	305.0
X3	95	11.81	93.60	7.68	806905		4030	1831.8	123.0
Y1	95	19.68	279.60	38.21		35847209	4030	1831.8	613.8
Y2	95	15.72	70.80	7.73		464912	4030	1831.8	123.6
Y3	95	15.72	118.80	12.97		2196446	4030	1831.8	208.0
Y4	95	15.72	91.20	9.96		993701	4030	1831.8	159.5
Y5	95	11.81	70.80	5.81		349217	4030	1831.8	92.8
Y6	95	15.72	109.20	11.92		1705844	4030	1831.8	191.2
Y7	95	15.72	201.60	22.01		10733538	4030	1831.8	353.4
Y8	95	6.00	102.00	4.25		530604	4030	1831.8	68.1
Y9	95	13.80	102.00	9.78		1220389	4030	1831.8	156.7
Y10	95	6.00	102.00	4.25		530604	4030	1831.8	68.1
Y11	95	13.80	238.80	22.89		15660326	4030	1831.8	367.6
Y12	95	9.84	175.20	11.97		4409772	4030	1831.8	192.2
Y13	95	13.80	240.00	23.00		15897600	4030	1831.8	369.4
Y14	95	15.72	278.40	30.39		28266952	4030	1831.8	488.2

Vertical Distribution of Seismic Forces in E-W direction												
k= 2.0												
Level	wi	hi	hx	w * hx^k	Σwi * hi^k	Cvx	Fx	V kips	M (overturning)	By	5% By	Mz
1A	5819	20.3	20.34	2407753		0.00	1.31	758.6	0.0	208.3	10.42	13.7
2A	5819	9.8	30.18	5301567		0.00	2.89	757.3	58.8	208.3	10.42	30.1
3A	5562	9.8	40.03	8910104		0.01	4.86	754.4	146.6	208.3	10.42	50.6
5.00	5618	16.9	56.92	18202355		0.01	9.92	749.5	397.0	208.3	10.42	103.3
6.00	3967	13.1	70.05	19462059		0.01	10.61	739.6	603.7	208.3	10.42	110.5
7.00	3258	13.1	83.17	22537804		0.02	12.28	729.0	860.3	208.3	10.42	127.9
8.00	3967	13.1	96.29	36779789		0.03	20.04	716.7	1667.0	208.3	10.42	208.8
9.00	3258	13.1	109.42	39007431		0.03	21.26	696.7	2046.9	208.3	10.42	221.4
10.00	3967	13.1	122.54	59562681	1391993350	0.04	32.46	675.4	3551.5	208.3	10.42	338.1
11.00	4540	13.1	135.66	83558154		0.06	45.54	642.9	5579.9	208.3	10.42	474.2
12.00	3967	13.1	148.79	87810734		0.06	47.85	597.4	6491.8	208.3	10.42	498.4
13.00	4540	13.1	161.91	119017856		0.09	64.86	549.6	9650.1	208.3	10.42	675.5
14.00	3967	13.1	175.03	121523949		0.09	66.22	484.7	10722.4	208.3	10.42	689.7
15.00	4540	13.1	188.16	160732853		0.12	87.59	418.5	15331.4	208.3	10.42	912.3
16.00	3967	13.1	201.28	160702325		0.12	87.58	330.9	16477.8	208.3	10.42	912.1
17.00	4540	13.1	214.40	208703145		0.15	113.73	243.3	22892.2	208.3	10.42	1184.5
Roof (level 18)	4563	13.9	228.28	237772791		0.17	129.57	129.6	27781.2	208.3	10.42	1349.5
	Total torsional moment =	7900.5	k-ft		Total overturning moment =		124258.7	k-ft	Total base shear =		758.6	kip

Vertical Distribution of Seismic Forces in N-S direction												
k= 2.0												
Level	wi	hi	hx	w * hx^k	Σwi * hi^k	Cvx	Fx	V kips	M (overturning)	By	5% By	Mz
1A	5819	20.3	20.3	2407753		0.00	1.31	758.6	0.0	88.6	4.43	5.8
2A	5819	9.8	30.2	5301567		0.00	2.89	757.3	58.8	88.6	4.43	12.8
3A	5562	9.8	40.0	8910104		0.01	4.86	754.4	146.6	88.6	4.43	21.5
5.00	5618	16.9	56.9	18202355		0.01	9.92	749.5	397.0	88.6	4.43	43.9
6.00	3967	13.1	70.0	19462059		0.01	10.61	739.6	603.7	88.6	4.43	47.0
7.00	3258	13.1	83.2	22537804		0.02	12.28	729.0	860.3	88.6	4.43	54.4
8.00	3967	13.1	96.3	36779789		0.03	20.04	716.7	1667.0	88.6	4.43	88.8
9.00	3258	13.1	109.4	39007431		0.03	21.26	696.7	2046.9	88.6	4.43	94.2
10.00	3967	13.1	122.5	59562681	1391993350	0.04	32.46	675.4	3551.5	88.6	4.43	143.8
11.00	4540	13.1	135.7	83558154		0.06	45.54	642.9	5579.9	88.6	4.43	201.7
12.00	3967	13.1	148.8	87810734		0.06	47.85	597.4	6491.8	88.6	4.43	212.0
13.00	4540	13.1	161.9	119017856		0.09	64.86	549.6	9650.1	88.6	4.43	287.3
14.00	3967	13.1	175.0	121523949		0.09	66.22	484.7	10722.4	88.6	4.43	293.4
15.00	4540	13.1	188.2	160732853		0.12	87.59	418.5	15331.4	88.6	4.43	388.0
16.00	3967	13.1	201.3	160702325		0.12	87.58	330.9	16477.8	88.6	4.43	388.0
17.00	4540	13.1	214.4	208703145		0.15	113.73	243.3	22892.2	88.6	4.43	503.8
Roof (level 18)	4563	13.9	228.3	237772791		0.17	129.57	129.6	27781.2	88.6	4.43	574.0
	Total torsional moment =	3360.5	k-ft		Total overturning moment =		124258.7	k-ft	Total base shear =		758.6	kip

Forces in Shear Walls due to Base shear in N-S direction at Level 7									
Total K for direct shear		3452.60		Shear force	728.99				
Shear Wall	d ft	Ki (kip/in)	Ki di^2(kip ft)	ΣKi di^2(kip ft)	P	Moment arm	Direct Shear	Torsional shear	Total Shear
X1	3.30	989.30	11069				0.0	2.8	2.8
X2	5.80	305.00	10224				0.0	1.5	1.5
X3	-13.10	123.00	20988				0.0	-1.4	-1.4
Y1	-111.90	613.80	7686779				-129.6	-59.6	-189.2
Y2	-86.60	123.60	926047				-26.1	-9.3	-35.4
Y3	-86.60	208.00	1558613				-43.9	-15.6	-59.5
Y4	-74.70	159.50	891228				-33.7	-10.3	-44.0
Y5	-74.70	92.80	518725				-19.6	-6.0	-25.6
Y6	-59.10	191.20	668522	14868314	729.0	17.7	-40.4	-9.8	-50.2
Y7	-63.30	353.40	1417277				-74.6	-19.4	-94.0
Y8	-21.20	68.10	30692				-14.4	-1.3	-15.6
Y9	-9.80	156.70	14913				-33.1	-1.3	-34.4
Y10	1.70	68.10	200				-14.4	0.1	-14.3
Y11	12.80	367.60	60130				-77.6	4.1	-73.5
Y12	22.20	192.20	95120				-40.6	3.7	-36.9
Y13	26.20	369.40	252814				-78.0	8.4	-69.6
Y14	38.00	488.20	704971				-103.1	16.1	-87.0
								Max shear	-189.2
							So, Y1 is the critical wall in NS direction		

Forces in Shear Walls due to Base shear in E-W direction at Level 7									
Total K for direct shear		1417.30		Shear force	728.99				
Shear Wall	d ft	Ki (kip/in)	Ki di^2(kip ft)	ΣKi di^2(kip ft)	P	Moment arm	Direct Shear	Torsional shear	Total Shear
X1	3.30	989.30	11069				-508.8	2.0	-506.8
X2	5.80	305.00	10224				-156.9	1.1	-155.8
X3	-13.10	123.00	20988				-63.3	-1.0	-64.3
Y1	-111.90	613.80	7686779				0.0	-42.4	-42.4
Y2	-86.60	123.60	926047				0.0	-6.6	-6.6
Y3	-86.60	208.00	1558613				0.0	-11.1	-11.1
Y4	-74.70	159.50	891228				0.0	-7.4	-7.4
Y5	-74.70	92.80	518725				0.0	-4.3	-4.3
Y6	-59.10	191.20	668522	14868314.27	729.0	12.6	0.0	-7.0	-7.0
Y7	-63.30	353.40	1417277				0.0	-13.8	-13.8
Y8	-21.20	68.10	30692				0.0	-0.9	-0.9
Y9	-9.80	156.70	14913				0.0	-0.9	-0.9
Y10	1.70	68.10	200				0.0	0.1	0.1
Y11	12.80	367.60	60130				0.0	2.9	2.9
Y12	22.20	192.20	95120				0.0	2.6	2.6
Y13	26.20	369.40	252814				0.0	6.0	6.0
Y14	38.00	488.20	704971				0.0	11.5	11.5
								Max shear	-506.8
						So, X1 is the critical shear wall in EW direction			

Appendix 4: Spot check of Shear Wall

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(i) Checking Moment Strength at base of Shear wall X1

$\%k$ for wall X1 $= \frac{989}{(989+305+123)} \times 100 = 69.8\%$

Total Base Shear in EW direction = 729k

Shear in X1 wall = (0.698) (729)
= 508.8 k

* $M_u = (1.0)(508.8k)\underbrace{\left(\frac{2}{3}\right)(228.3ft)}_{\text{Centroid of lateral forces}} = 77{,}439.4 k\cdot ft$

* $d = l_w - \left(\begin{array}{c}\text{distance from edge to centroid}\\ \text{of reinforcement}\end{array}\right) = (47)(12) - 15.7 = 548.3 in$

* Factored axial load = $0.9N_0$ = 8640 k

Area of Wall = 61.6 ft^2 = 9x6.8 (Approx 7 column)
Area of a column = 9 ft^2
so tributary area of 7 columns: 7 x 761.8 = 5332.3 ft^2
DL = 100 psf (approx)
so DL on Shear wall X1 assuming that its tributary area is equivalent to that of 7 columns. → $\frac{(100 psf)(5332.3)(18)}{1000}$ = 9598

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$* \; T = A_s f_y = (6.32)(60) = \underline{379.2\,k}$

$8T25 = 8\#8 = 6.32\ in^2$

$* \; a = \dfrac{T + N_u}{0.85 f'_c b} = \dfrac{(379.2) + (8640)}{(0.85)(4.7)(19.7)} = \underline{114.6\ in}$

$* \; c = \dfrac{a}{\beta_1} = \dfrac{114.6}{0.85} = \underline{134.8\ in}$

$$* \; \phi M_n = \phi \left[T\left(d - \frac{a}{2}\right) + N_u \left(\frac{l_w - a}{2}\right)\right]$$

$$= \frac{0.9}{12}\left[379.2\left(548.3 - \frac{114.6}{2}\right) + 8640\left(\frac{(47)(12) - 114.6}{2}\right)\right]$$

$$= \frac{0.9}{12}\left[186187.2 + 1941408\right]$$

$$= \underline{159569.6\ k\text{-}ft} > M_u \quad \underline{OK}$$

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(2) Capacity based shear strength.

$$*\ N_{pr} = N_D + N_L = 9600 + \frac{(5332.3)}{1000}(1.0)(18) = \underline{19{,}198\,k}$$

$$*\ a = \frac{T + N_u}{0.85 f'_c b} = \frac{379.2 + 19198}{(0.85)(4.7)(19.7)} = \underline{248.8 \text{ in}}$$

$$*\ M_{pr} = T\left(d - \frac{a}{2}\right) + N_{pr}\left(\frac{l_w - a}{2}\right)$$

$$= (379.2)\left(548.3 - \frac{248.8}{2}\right) + 19{,}198\left(\frac{(47)(12) - 248.8}{2}\right)$$

$$= \left(160742.9 + 3025604.8\right) \text{ k-in}$$

$$M_{pr} = 265528.9 \text{ k-ft}$$

$$V_u = \frac{M_{pr}}{0.5 h_w} = \frac{265528.9}{(0.5)(228.3)} = \underline{2326\,k}$$

(3) Shear strength

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y\right)$$

$$\rho_t = \frac{A_v}{h s_2} = \frac{(3)(0.2) + (2)(0.31)}{(19.7)(7.86)} = 0.0079 > 0.0025 \text{ ok}$$

(ACI 11.9.9.2)

$V_n = (19.7)(47)(12)\left[(2)(1)\sqrt{4700} + (0.0079)(60{,}000)\right]$

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$\phi V_n = (0.75) V_n = 5092$ kip $>$ V_u (capacity based)

OK

Thus, 19.7in thick shear wall in E-W direction has adequate strength to support vertical and horizontal loading during earthquake

Appendix 5: Story Drift

Story Drift | Tech Report 3 | 1

Procedure to calculate story drift.

(1.) Using story forces in wind and seismic calculation

F = story forces (kips)

h_x = height of each story (in)

h_i = height of story from base (in)

t = thickness of shear wall (in)

l = length of shear wall (in)

E = Modulus of elasticity (ksi)

I = Moment of inertia (in^4)

G = Shear modulus (in^3)

k = Stiffness due to unit distributed load $\frac{(k/in)}{in}$

$\left.\right\}$ for shear walls

X1, X2, X3 — Shear walls in East-West direction

Y1, Y2, ... Y14 — Shear walls in North-South direction.

Δ = story drift

w = distributed wind/seismic load (k/in)

A = Cross-sectional area of the wall

$\alpha = \frac{\text{Max shear}}{\text{Average shear}}$

2

Steps

(1)

$$\Delta = \frac{K}{P}$$

$$K = \frac{1}{\left[\frac{wh_i^4}{8EI} + \frac{wh_i^2 \alpha}{2AG}\right]}$$

$I = 0.7 I_g$ for uncracked walls
$I = 0.35 I_g$ for cracked walls
ACI 10.10.4.1

I, G, K were calculated at each story using t, l, A and E.

Finally $\Delta = \frac{K}{P}$ ← $P = F/h_x$

Complete excel sheet calculations for story drift can be made available upon request. Due to large number of tables and sheets, the files are not attached to this file.