AE SENIOR THESIS 2012-13

TECHNICAL REPORT 1

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The Optimus

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Executive Summary

The purpose of this technical report is to understand the structural system of the building and the its design process. This is achieved by studying the architectural and structural plans, sections, elevations and performing analytic calculations on the design.

The report summarizes the structural and architectural system of the building and shows the integration of the two systems. It goes more in depth and explains the building codes, materials, structural systems employed to achieve an efficient structure. As the building is located in India, it is designed using the indian codes. This report reads the building from the american code perspective and finds out differences between the two codes.

Besides codes, it summarizes the gravity, lateral and foundation systems of the building. It shows the interaction of the structural system to different loading conditions.

The reading and analyses of the structural system : columns and flat slabs using american codes ; ASCE 7-10 and ACI 318-11, shows the design can be optimized using the american codes. Also, the wind and seismic analysis on the building was carried out using wind data from the existing location and using equivalent location for gaining seismic information.

This report creates the base for further analysis of the structural systems to find the controlling conditions for the design in technical report 2 and eventually arrive at a detail analysis in technical report 3.

Building Introduction

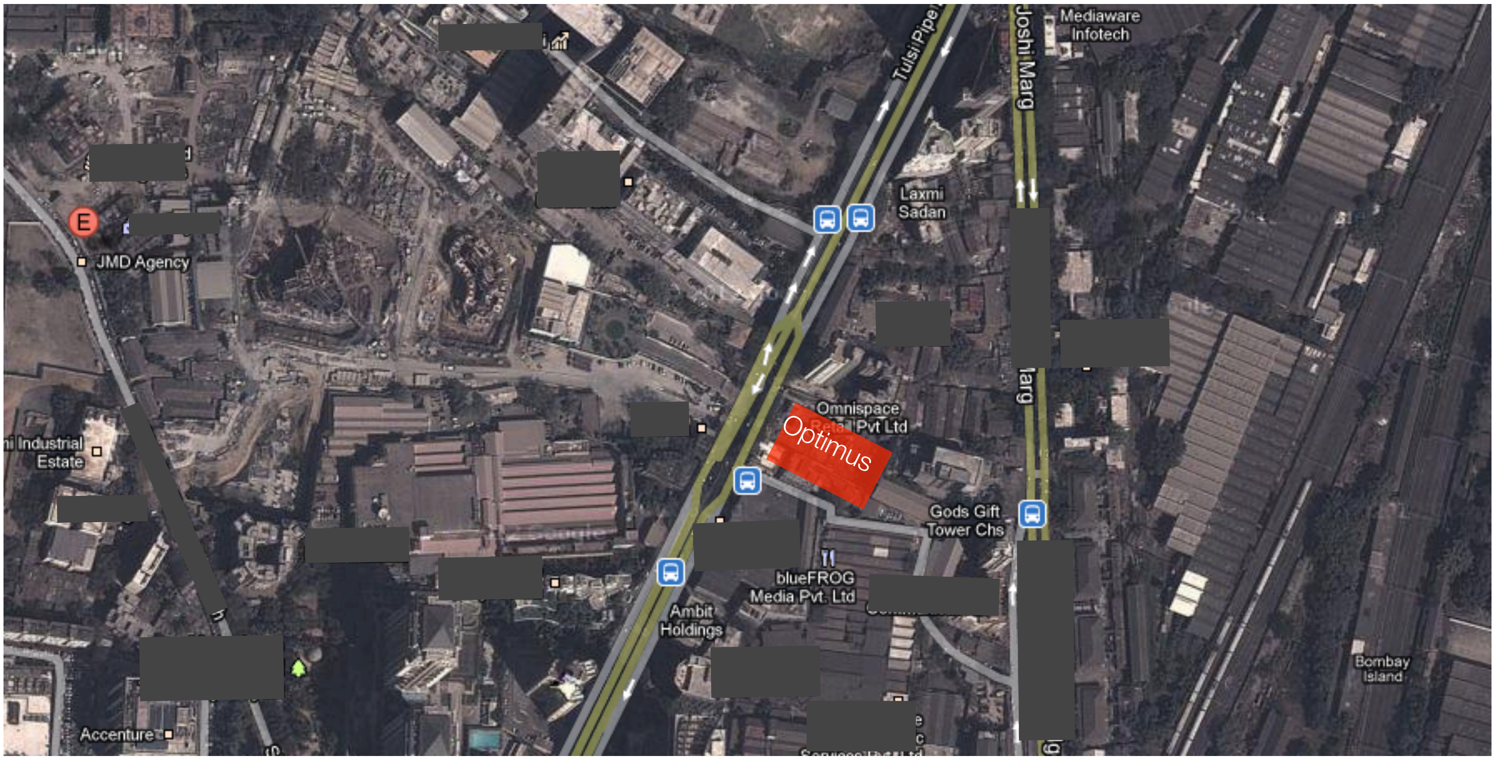


Figure 1 Aerial map from Google.com showing the location of the building site.

The Optimus, is a new building coming up in the city of Bombay, the economic capital of India. In a city that thrives on all kinds of businesses from small scale to large corporate companies, The Optimus will be catering medium size companies to set up their offices close to the business district of the city. The location is highly mixed use, as it contains residential towers, large shopping malls, office buildings and factories. While the future of the location is going to be marked by tall skyscrapers soaring about 100 stories, The Optimus is designed to provide a much humble yet modern look to fit in the fabric of the city.


Figure 2 Rendering showing roof garden

The design of The Optimus in the interior and exterior is very functional as well as aesthetic. It makes an efficient use of space within tight boundaries of the site and provides spacious floor space to its inhabitants. To cater the requirements of the offices, it offers open and customizable floor space. The spacing of the structural and architectural elements offers flexible partitioning for office spaces. 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 5 level as garden, lobby and office. The office spaces start from 6 to 16th story and 17th story contains a roof garden.


Figure 3 Rendering of the building entrance

Just as 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 designed to fit the location which also becomes an architectural feature of the building. Three kinds of materials decorate the facade: metal, stone and plants. As the north facade of the building faces a tall residential tower, all the office space is moved to the south facade and giving a better view of stone and green wall to the residents of the adjacent tower. The south facade is dominated by a bold and modern look with metal cladding and windows pushed inside to provide solar shading in the interior. The front facade that faces 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.

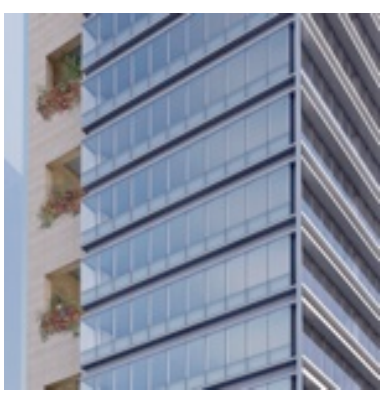


Figure 4 Rendering of the building facade

The structure of the building is something that complements the architectural beauty. A successful building is achieved when its structure and architecture integrate without compromise, and this applies to The Optimus. In order to provide the celebration of facade, open floor plan and efficient floor area, the structure plays a very significant role. All the columns in the floor area are pushed to the exterior so that interior is open and at the same time no column is visible in the exterior to provide different architectural features on the facade. In this way the structural system of building does not compromise the architecture but celebrates it.

Structural System Overview

The structural systems of The Optimus has been optimized to increase floor space area, to celebrate the architecture and decrease the overall cost of the building without compromising safety. In order to achieve these goals, concrete was chosen as a prime material to support the building. The properties of concrete allows fluidity in design, room for design changes during construction and makes the construction process cheaper by employing the ample of labour force available at a cheaper cost. All the structural systems from foundation to slabs come together to improve efficiency in design and safety.

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. Besides excavation, the soils report consists of soil bearing capacity of the soil, water table information, properties of soils and rocks at different levels below ground.

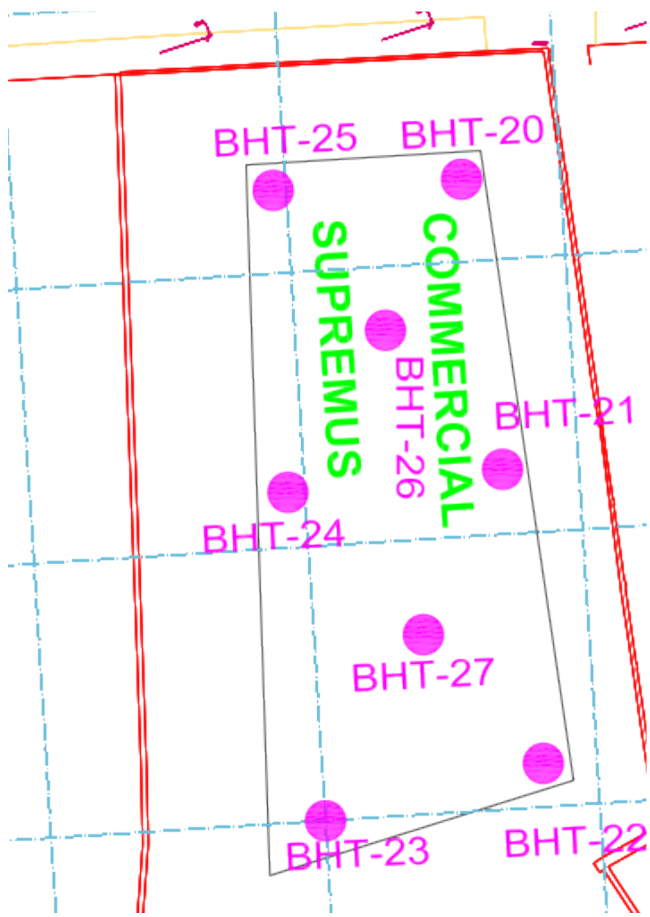


Figure 5 Boring test map on the building site.

8 boreholes were drilled and soil properties were analyzed in a lab. 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. The soils report explained that the soil and clay below ground would exert lateral pressures on the basement walls. To account for these lateral pressures, the reinforced concrete frame and the main structure of the

building will internally support the basement walls. Therefore, the basement walls were designed for hydrostatic pressure as well as the earth pressure. 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

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

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 is 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 the weak axis and 1.5 ft to 7 ft in the strong axis direction. 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.

The columns are tied together with beams, girders and mainly the flat slab system of the floor framing. The 8 - 12 in slabs connect to the columns with drop panels ranging about 2 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.

Lateral System

The wind and seismic forces are handled by the extensive shear walls that exist around the stairwells and elevator core. These reinforced concrete shear walls range from 8 in to 20 in thickness are designed to resist lateral and torsional forces due to wind and seismic loads. These walls span from basement to the top of the building and are connected using link beams. In N-S direction of the building, the shear wall and to some point the strong axis of the columns help in resisting the lateral forces. This is because width of the building is small in the N-S direction and strong axis of columns provide support to the shear walls through the connection with the slab. In the long side of the building i.e. the E-W direction the long and strong axis of the shear walls seem adequate to control drifts and resist forces in the E-W direction.

Floor and Roof system

The floor system of the entire building is a flat slab system that avoids adding beams to connect to columns and thus creates an even surface for office ceiling and floor spaces. The slab thickness range from 8 in in the office spaces to 12 in in parking and heavily loaded spaces. The thickness is not only controlled by loads but also by deflections and punching shear. The slabs are reinforced in top and bottom to account for positive moments in the span and negative moments at the slab-column connection. At places where deflection is an issue hidden beams are added to reduce deflection and avoid any undulating surfaces in the ceiling.The roof system is also a flat slab with waterproofing done to tackle the heavy monsoons of the city.

Design Codes

As the building is located in India, the Indian Standard (IS) code is used to design The Optimus. However, in this report the American codes are used for checks and analysis. This will 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

IS Code	Description
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 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 1

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. In vertical structural the strength of the materials increases as the required strength of the member increases. This helps in improving efficiency by increasing material strength instead of increasing the size of the member.

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 = $57000\sqrt{f'c}$ (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 = 5.5×10^{-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 used from the IS Code whereas the report uses ASCE 7-10 provisions to calculate live loads. The superimposed dead loads are used the same that is on the project 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 will give more conservative results because it accounts for contingencies in construction and materials used on the project. The tables below shows 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

- Parking Space and Drive-way

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
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

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	7.25	151.4
Live Load	4.00	100

- Entrance Lobby, Elevator lobbies

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	2.00	41.8
Live Load	3.00	100

- Mechanical Floor

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	2.00	41.8
Live Load	7.50 Non-reducible	150 non-reducible

- Electrical room over ground floor

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	2.00	41.8
Live Load	13.50 non-reducible	282 non-reducible

- Stairs

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	1.50	31.33
Live Load	3.00	100

- Toilet rooms

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	4.50	94
Live Load	2.00	40

- Typical Office

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	3.00	62.7
Live Load	4.00	100

- Retail over ground floor

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	4.575	95.6
Live Load	4.00	100

- Eatery and Utility

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	3.00	62.7
Live Load	5.00	100

- Outdoor Utility over Level 105, 107 and similar

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	5.625	117.5
Live Load	5.00	100

- Planted Terrace

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	12.50	261.1
Live Load	3.00	100

- Amenity / Fitness Center

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	3.50	73.10
Live Load	5.00	100

- Water tank over level 119

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	3.50	73.1
Live Load	35 non-reducible	731 non-reducible

- Electrical Panel room at ground floor

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^3)
Superimposed Dead Load	2.00	41.8
Live Load	13.50 non-reducible	282 non-reducible

- Roof

	IS Code (kN/ m^2)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Superimposed Dead Load	5.50	114.9
Live Load	3.00 Non-reducible	100 non-reducible

- Peripheral loads

	IS Code (kN/ m)	ACI 318-11 / ASCE 7-10 (lb / ft^2)
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 had 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="16">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 $50m^2$</td><td>0</td></tr>
<tr><td>$50m^2$ to $100 m^2$</td><td>5</td></tr>
<tr><td>$100m^2$ to $150 m^2$</td><td>10</td></tr>
<tr><td>$150m^2$ to $200 m^2$</td><td>15</td></tr>
<tr><td>$200m^2$ to $250m^2$</td><td>20</td></tr>
<tr><td>Over $250 m^2$</td><td>25</td></tr>
</table>

Gravity Framing System

The existing system was analyzed used the Live loads and live load reduction from ASCE 7-10 and procedures from ACI 318-11. The typical gravity members: columns and slabs were analyzed and can be found in Appendices 1 to 3 of the report.

In the analysis the dead loads were calculated using unit weight of concrete as 150 lb/ft^3. As mentioned, superimposed loads were used from the loads provided in the structural design criteria of the building. As the mechanical and electrical systems of the building will be designed and installed after the spaces are sold out, the loads of the mechanical and electrical cannot be accurately determined. This is the reason for applying higher superimposed and live loads in the areas where mechanical and electrical system will be installed.

Column spot check

Column C16 on level 7 at grid C-3 was check for axial loads and biaxial moments. Loads from level 8 to 17 were used as per ASCE 7-10 provisions. Live load reduction for columns was carried out according the the ASCE 7-10 code. The columns was check for pure axial and pure bending moments due to moments from the slabs. it was found that the capacity of columns is much higher than pure axial force and moment in major and minor axis directions. This shows that pure axial and moments form the slabs did not control the column design.

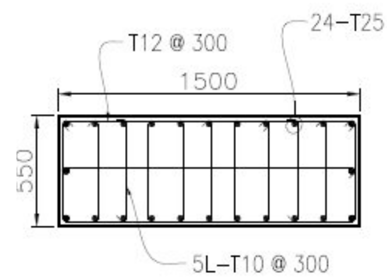


Figure 6 Cross-section of Column C16 at Level 7

Several other factors would be controlling the columns design. As the P-M interaction curve gives a high capacity for combined axial and bending moment, the design could be based on axial loads and bending moments due to lateral loads. A combination of axial force and lateral force due to wind or seismic could be a controlling factor for the design of the columns.

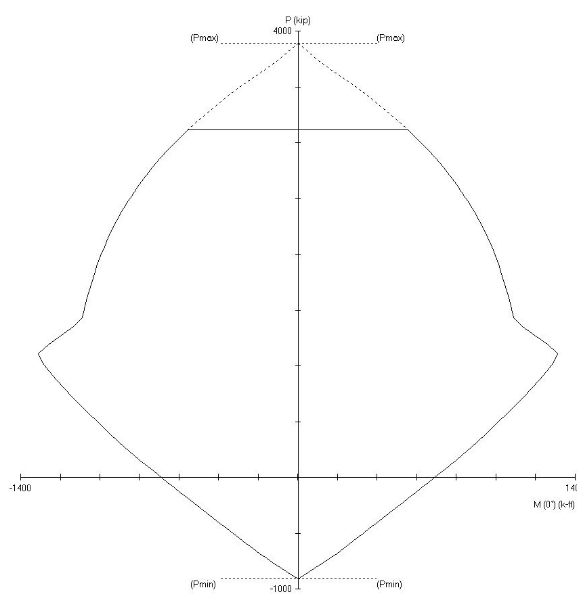


Figure 7: P-M interaction curve of Column C16

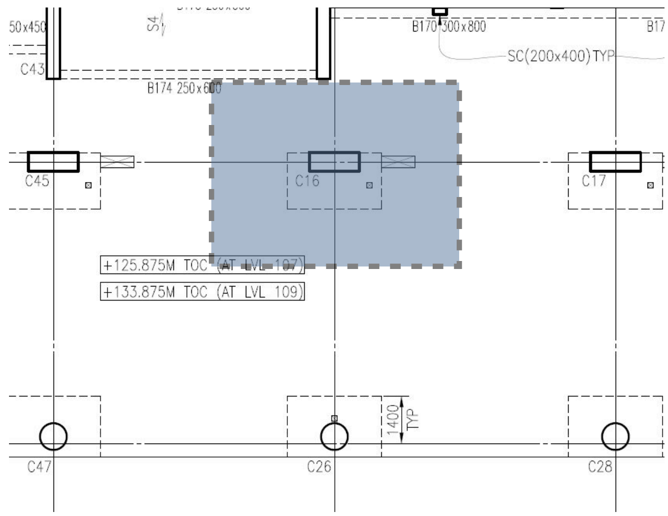


Figure 8: Column C16 in plan at Level 7

Punching Shear Spot check

Column C16 was checked for punching shear using the loads determined in column spot check. As column C16 has a 10 in drop panel, the punching shear was performed about the critical area of the drop panel. The results suggest that the shear capacity of the section was calculated to be much higher than the existing loads. This suggests direct shear does not control the thickness of slab or the drop panel. As the drop panel is moved towards the south, the longer span of the slab, unbalanced moments in the N-S direction could be a controlling factor for slab or drop panel thickness. A drop panel could also be used to control deflections in the weak axis direction because long term deflection also significantly affect the design of the slab.

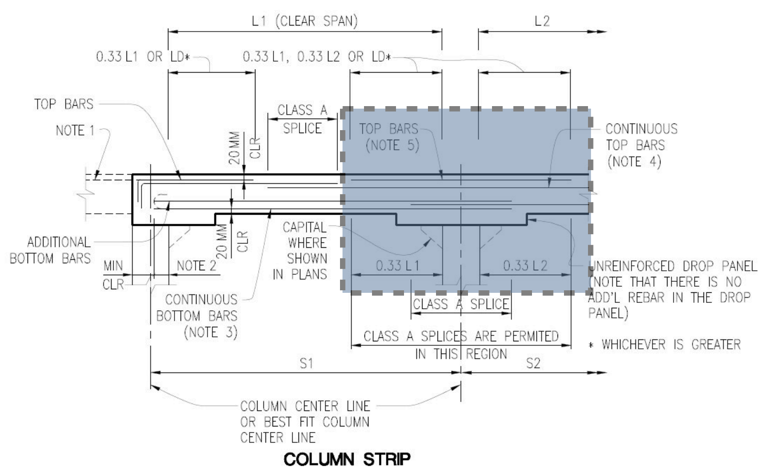


Figure 9: Typical cross-section of column drop panel

Slab moment and reinforcement check

Positive and negative moments were calculated in the slab using the moment coefficients from Direct Design method of ACI 318-11 and compared with the moment capacity of the reinforcement in the slab. A columns strip from grid B to D along grid 3 was analyzed. The results show a high negative reinforcement was used near the column but adequate bottom reinforcement was used as the values of moment capacity and actual moments were close. Slab reinforcement is also increased to reduce long term slab deflections.

A reason why the capacity of the structural members is high is the assumption that the indian code is the more conservative to account for variation in use of material because the material like the bricks may not be as per the standards. Also, there are variations in the construction process because human errors by the labor force on the site.

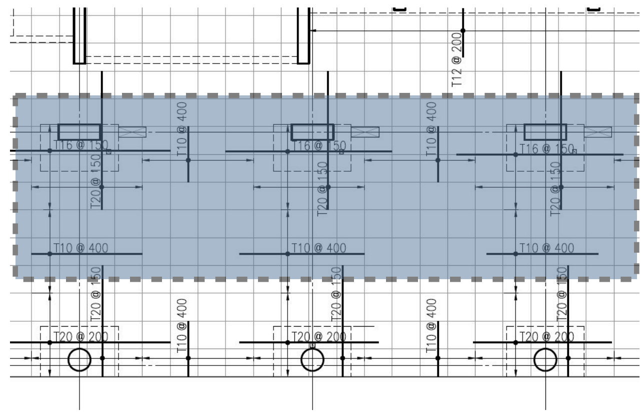


Figure 10: Column strip and top reinforcement in slab at level 7

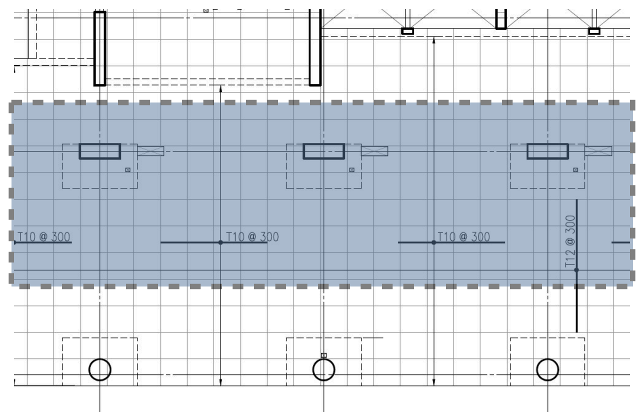


Figure 11: Column strip and bottom reinforcement in slab at level7

Wind Loads

Wind loads were calculated using ASCE 7-10, Chapter 7 using directional procedure for buildings of all heights in comparison to the IS 875 code in India. The basic wind speed was taken as 44 m/s that is taken from the wind data of the location. All the assumptions made were realistic and conservative to calculate wind pressures in the building. Please refer to Appendix to review the list of assumptions and calculations made to calculate shear forces in wind.

The wind forces are transferred through the shear wall in the core of the building. Also, the columns due to their large size seem to play some part in resisting lateral loads which will be figured in future reports.

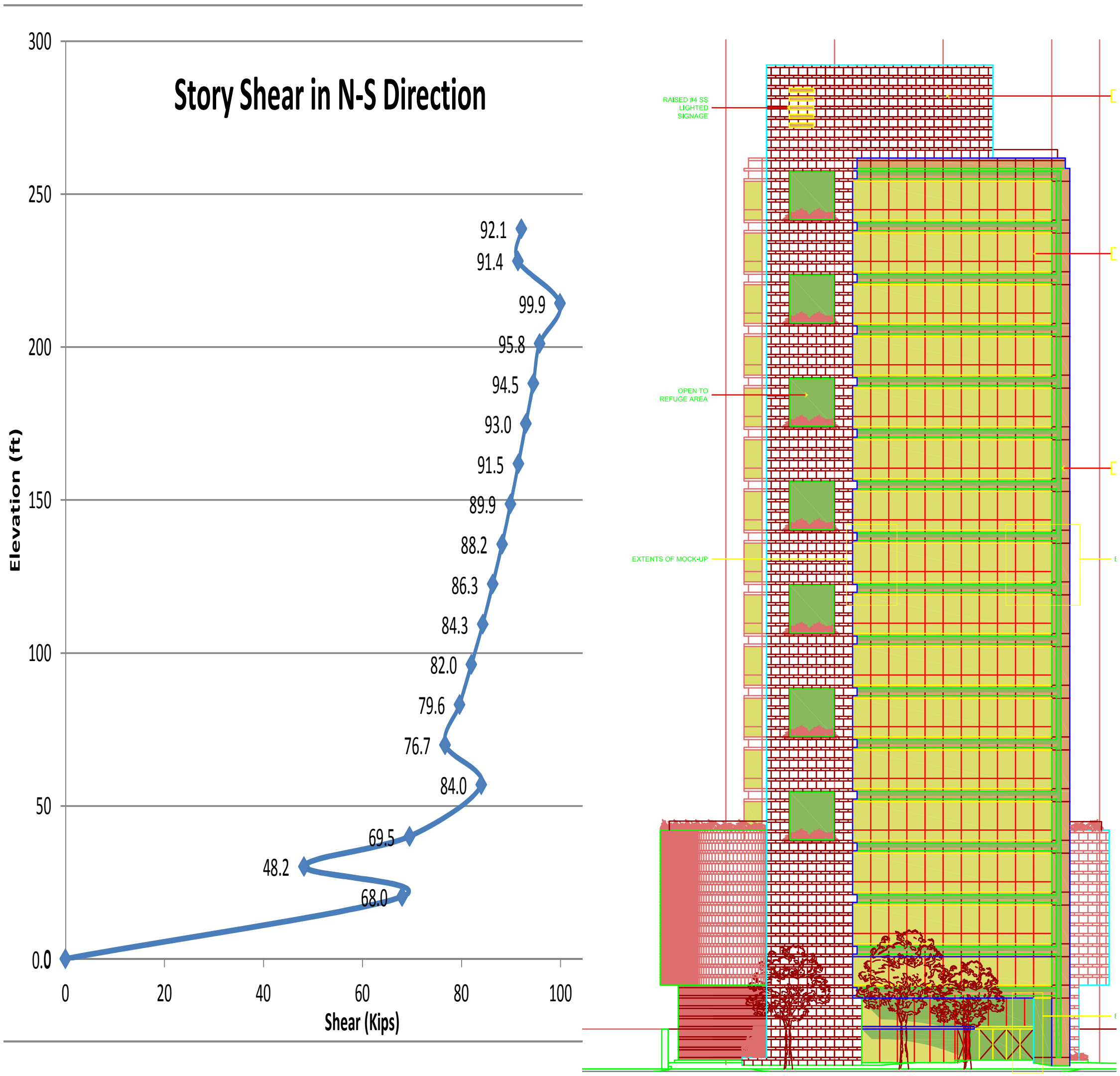


Figure 12: Story shear forces in N-S direction. The elevation shows east facade.

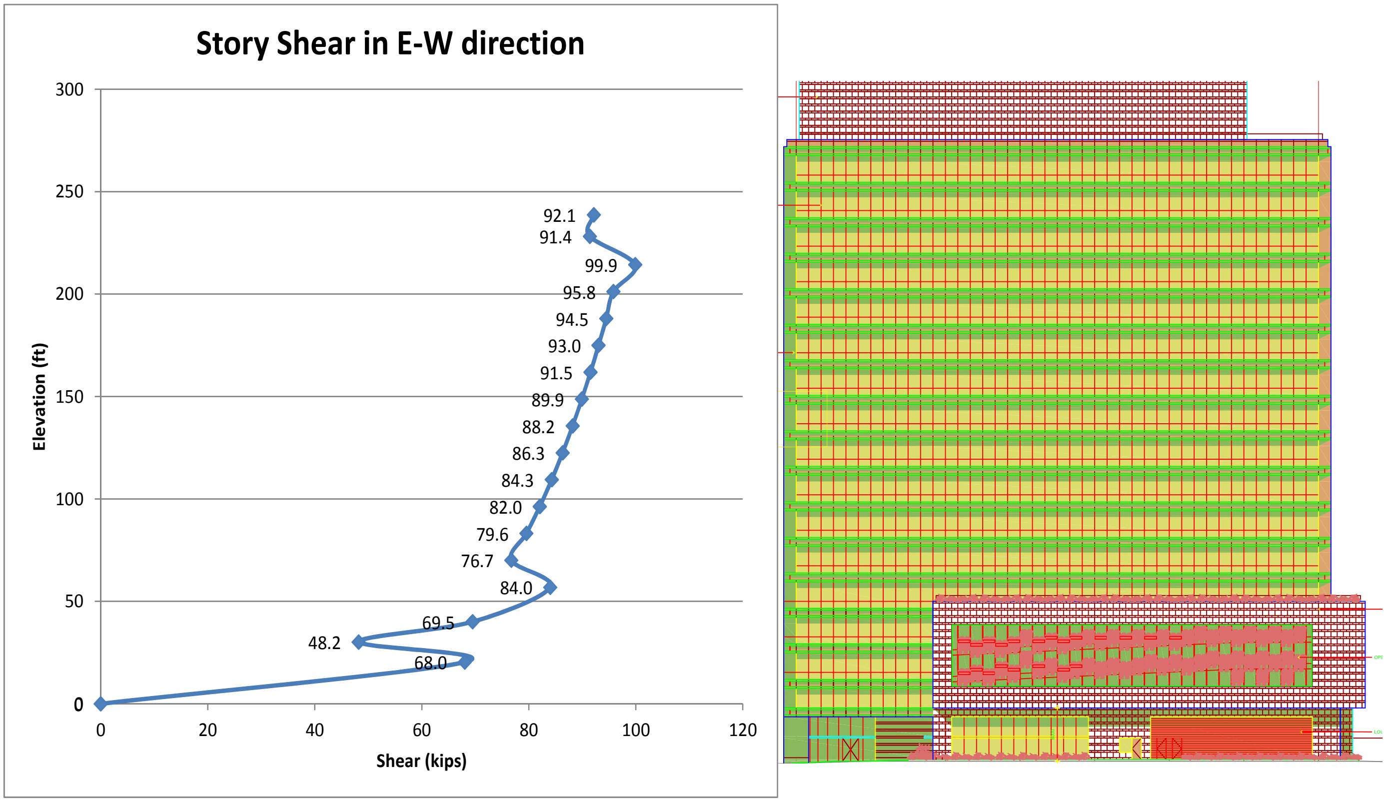


Figure 13: Story shear forces in E-W direction. The elevation shows south facade.

Seismic Loads

ASCE 7-10 was also used to calculate seismic loads on the building. Section 12.8, Equivalent lateral force procedure was used to calculate the base shear in comparison to the response spectrum analysis used in the IS code. The procedures in the two codes are similar but they differ in making assumptions to calculate the seismic response coefficient.

A major assumption was made in the location to calculate seismic base shear. As ASCE 7-10 does not provide mapped spectral response parameters for location in India, New york city was chosen as an equivalent location because the seismic behavior of the two locations is similar. This was also seen in the global seismic risk maps on the maps provided by USGS.The two locations also resemble each other in terms of their geographical features where both are islands surrounded by land mass in major mart of their perimeters.

The effective seismic weight of the building was calculated by making many simplified assumptions on each floor. Refer to the excel sheet in the appendix to look at the seismic weight calculations. Also refer to the appendix to look at the list of assumptions made.

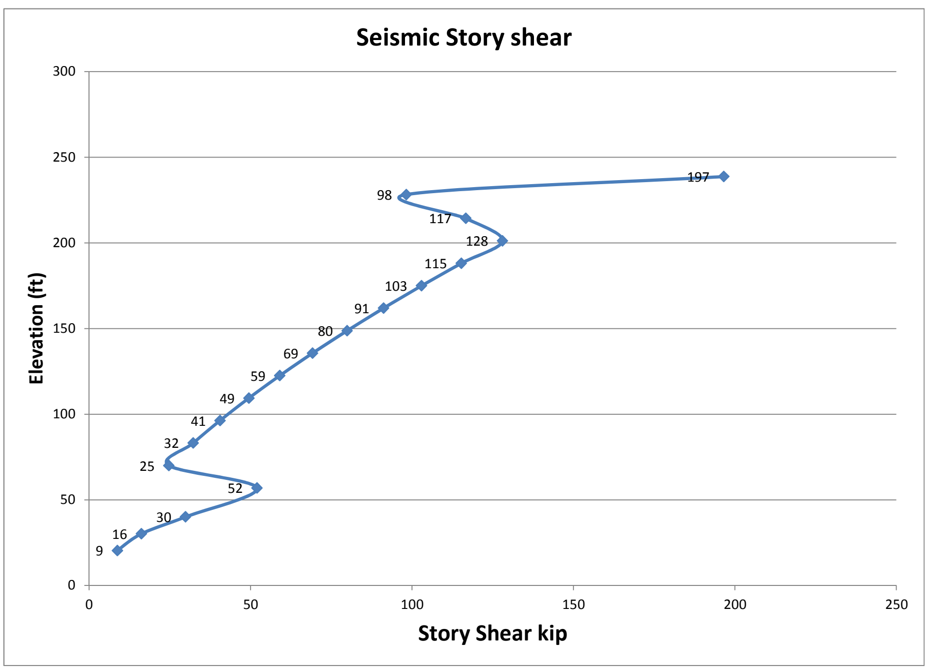


Figure 14: Graph of seismic story shear vs elevation of the building.

Conclusion

Tech 1 report analyzed the existing structural system of The Optimus. It summarized the foundations, gravity and lateral system of the building. The existing system was designed using the indian codes. However, this report used american codes to analyze the structural system.

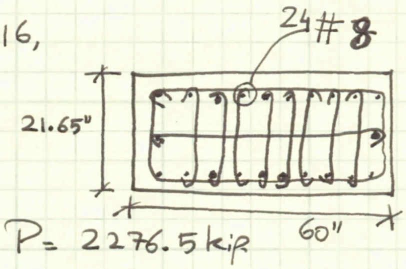
From the spot checks, it was found that the indian codes is conservative as compared to american codes. This is because of the contingencies that exist in the construction process in India. However, a deeper analysis will be done in technical reports 2 and 3 to confirm this finding.

Wind and seismic loads were calculated using ASCE 7-10 provisions and it was found that seismic loads control the design because of higher base shear. However, further analysis of story drifts will help understand the role of wind the design of the lateral system.

In conclusion, a further analysis into the structural systems will confirm or modify the findings from this report and will lead to a deeper understand of the interaction of the systems.

Appendix 1- Column Spot Check

Spot Chek → Column C16,



- Axial load check

From excel sheet, P = 2276.5 kip

Column capacity.

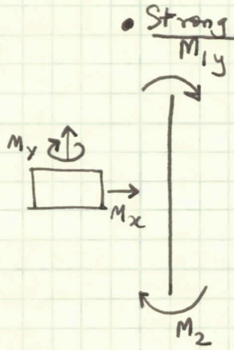
According to ACI 318-11 section 10.3.6.2, the column has tie reinforcement.

$$\phi P_{n_{max}} = 0.80\phi\left[0.85 f'_c \left(A_g - A_{st}\right) + f_y A_{st}\right]$$

$$= (0.8)(0.9)\left[(0.85)(5)\left(1299 - 18.96\right) + (60)(18.96)\right]$$

$$= 4735.9 \text{ kip} < 2276.5 \text{ kip} \quad \text{OK}$$

- Moments due to slabs above and below.
  - Strong axis moments (M_y)



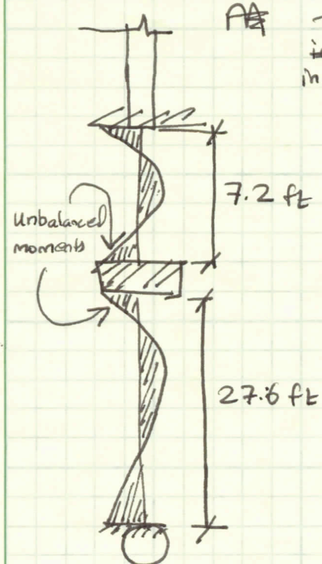
M_{1y} = Moments from slab on level 8.
Slab unbalanced ~~be~~ moments are the one transferred to column.

Assuming slab-column connection as fixed-fixed.

$M_{1y} = 0$. because the spans on the sides of column are equal so there is no unbalanced moment in slabs.

M_{1x} = Weak axis moments. As the span in y-direction are different on opposite sides of column, the column will contain unbalanced moments from slab.

- Calculating unbalanced moment on level 7 & 8.



~~AB~~ Total distributed load = 1.2D + 1.6L
~~in two str~~ in 1ft strip.

$= 1.2(285.2\,psf) + 1.6(100\,psf)$

$= 502.2\ psf \times 1\ ft$

$= \underline{502.2\ lb/ft}$

$$M = \left[\frac{(502.2)(27.6)^2}{12} - \frac{(7.2)^2(502.2)}{12}\right]$$

$\underline{M = 29.7\ k\text{-}ft}$

Total moment from above and below

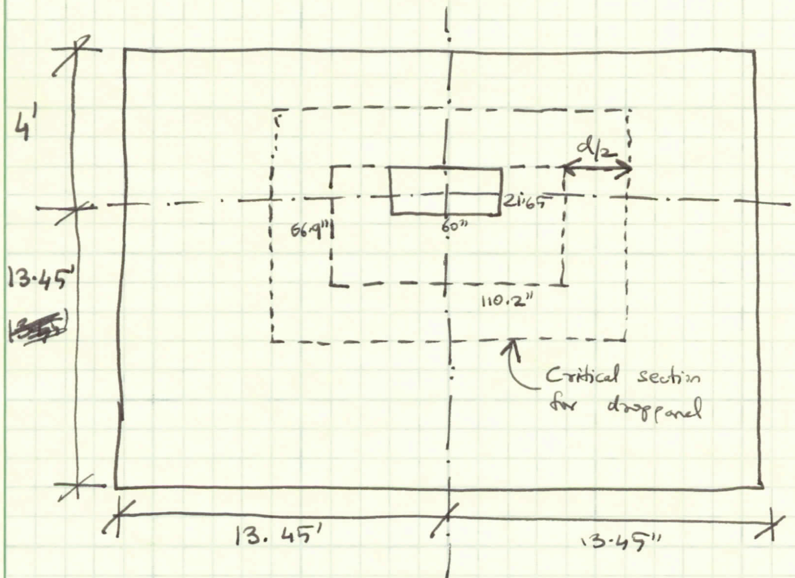
$= 2 \times 29.7 \approx \underline{60\ k\text{-}ft}$

The value of M falls well in the P-M curve.

Appendix 2 : Punching Shear

Spot Check → Punching shear for Column-slab connection at level 7.

- Following method shown in Mc Gregor textbook and equations from ACI 318-11.



$d_{\text{drop panel}} = 7.5''$

$d_{\text{slab}} = 5.7''$

$q_u = 1.2(222.5\ \text{psf}) + 1.6(100\ \text{psf}) = 427\ \text{psf}$

- Checking shear about drop panel.

$$V_u = (0.427\ \text{ksf})\left[(17.45' \times 26.9') - (6.2' \times 9.81')\right]$$

$$\underline{V_u = 174.5\ \text{kip}}$$

$$\phi V_{c_1} = 4\lambda\sqrt{f'_c}\, b_o d$$

$$= (4)(1)\sqrt{5000} \times (384.2)(7.5)$$

$$\phi V_{c_1} = 815\ \text{k}$$

$$b_o = 2(117.7 + 74.4)$$

$$= \underline{384.2\ \text{in}}$$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

$$\phi V_{c_2} = \left(2 + \frac{4}{\beta}\right) \lambda \sqrt{f'_c}\, b_o d \qquad \beta = \frac{60}{21.65} = 2.77$$

$$= \left(2 + \frac{4}{2.77}\right) \frac{(1)\sqrt{5000}\,(384.2)(7.5)}{1000}$$

$$= \underline{701.7 \text{ kip}}$$

$$\phi V_{c_3} = \left(\frac{\alpha_s d}{b_o} + 2\right) \lambda \sqrt{f'_c}\, b_o d \qquad \alpha_s = 40 \text{ for interior column}$$

$$= \left(\frac{(40)(7.5)}{384.2} + 2\right)(1)\left(\sqrt{5000}\right)(384.2)(7.5)$$

$$= 566.6 \text{ kip}$$

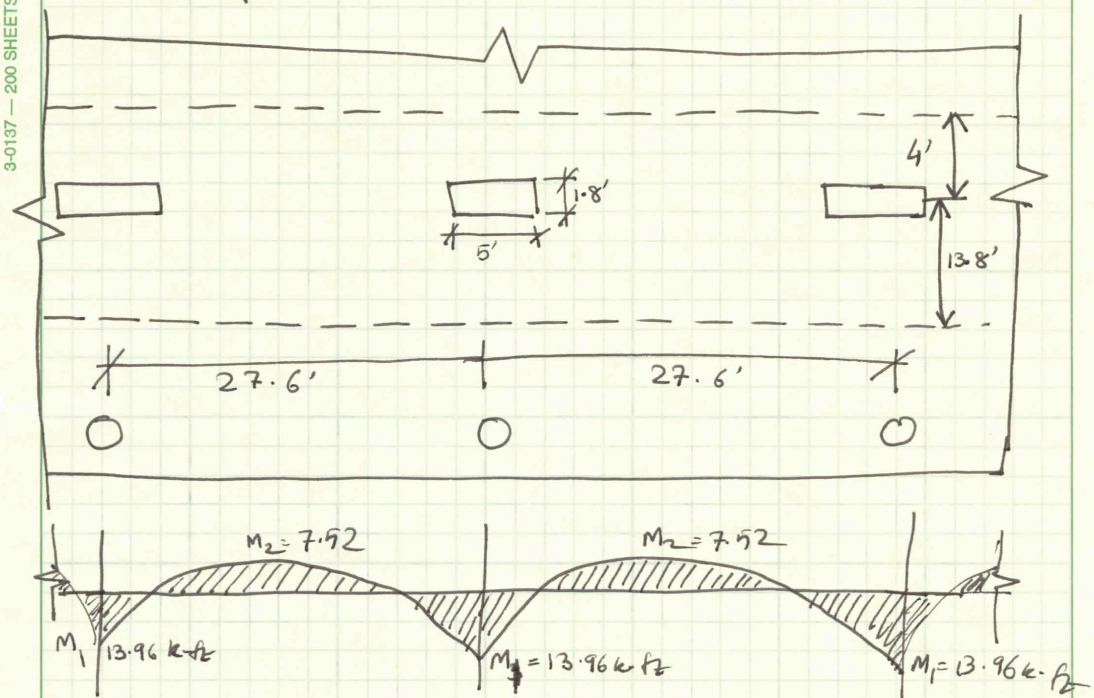
$$\phi V_c = \min \begin{vmatrix} \phi V_{c_1} \\ \phi V_{c_2} \\ \phi V_{c_3} \end{vmatrix} \implies \phi V_c = 566.6 \text{ kip} > 174.5\text{k}.$$

$\underline{OK}$

Appendix 3: Slab moments

Spot Check: Slab reinforcement at level 7.

- Checking slab reinforcement in a ~~EFL~~ column strip of slab at level 7.



$q = 427$ psf = load

$l_n = 27.6' - 5' = 22.6'$ = clear span

$l_2 = 17.8'$ → width of strip

Using ACI 318-11 Moment coefficients in Direct design method, the moments for interior span & column strip are:- (Section 13.6.3.2)

$$M_1 = 0.65 M_0 = \frac{(0.65)(0.427)(17.8)(22.6)}{8} = \underline{13.96} \text{ k-ft}$$

$$M_2 = 0.35 M_0 = \frac{(0.35)(0.427)(17.8)(22.6)}{8} = \underline{7.52} \text{ k-ft}$$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Reinforcement provided in a 17.8' column strip

Bottom reinforcement = T10@ 300 mm = #3 bars at 1.4' spacing

= 12 #3 bars

Top reinforcement = T16@ 150 mm = #6 bar at 0.5' spacing around the column.

= 35 #6 bars.

but the bars are added in 3m = 9.8' width of column.

i.e. $\boxed{\text{19 \#6 bars in a width of 9.8'}}$.

Calculating moment capacity

Bottom reinforcement.

$A_s = 12 \times 0.11 = \underline{1.32\ in^2}$

$M_u = \phi f_y\, jd\, A_s$

$= (0.9)(60)(3.88)(1.32)$

$M_u = 3276.6$ k-in

$= \underline{23\ k\text{-}ft} > 13.96$ k-ft / 7.52 k-ft

OK

$jd = d - \frac{a}{2}$

$a = \beta_1 c = \dfrac{0.003\, d\, \beta_1}{\varepsilon_t + 0.003}$

$= \dfrac{(0.003)(5.7)(0.85)}{0.005 + 0.003}$

$= 1.82$ in

$jd = 5.7 - 1.82$

$= \underline{3.88''}$

Top reinforcement.

$A_s = 19 \times \cancel{0.75}\ 0.44 = \underline{8.36\ in^2}$

$jd = 3.88''$ (at slab-column connection)

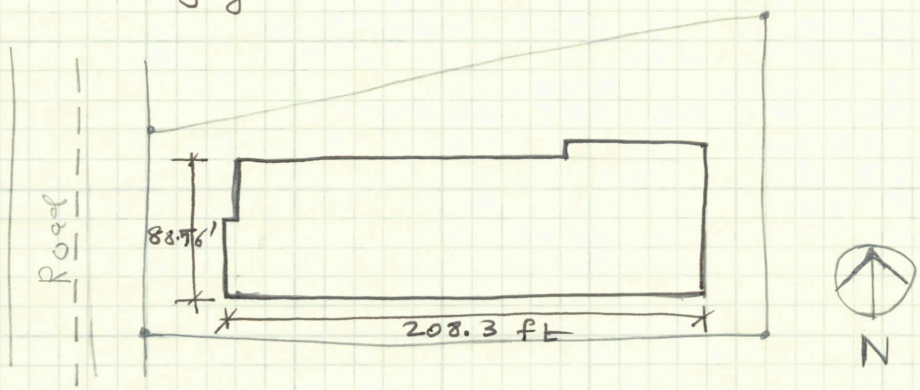
$M_u = (0.9)(60)(3.88)(19)(0.44)(1/12) \rightarrow \phi f_y\, jd\, A_s$

$M_u = \underline{146}$ k-ft > 13.96 k-ft

Appendix 4: Wind Loads

Tech 1 | Wind Load Calculations | Punit G. Das | Page 1

Simplifying the footprint

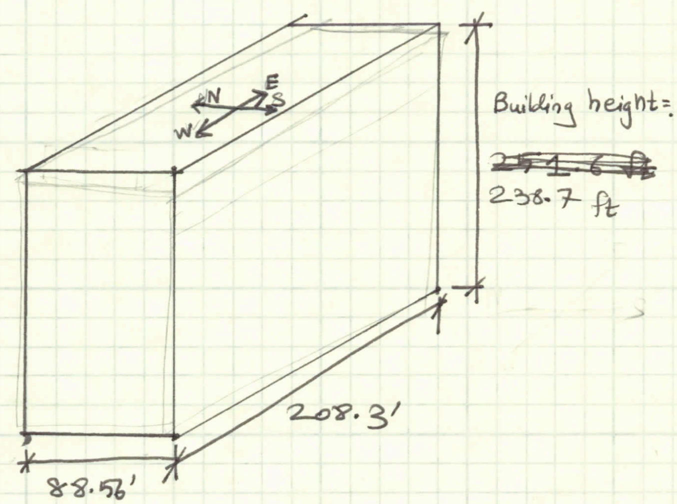


N-S Direction wind (short)

L = 208.3 ft
B = 88.56 ft

E-W direction Wind (long)

L = 208.3 ft
B = 88.56 ft



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~~ 1.5-1 (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 144.4 ft/s

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

• Exposure Category → C (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.

• 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 and~~ Table 26.6-1

3

- Gust effect factor → 0.85 (Section 26.9.1)
  ~~The~~ The natural frequency of building is below 1 Hz, which means the building is rigid. So gust effect factor is to be used as 0.85.

- Enclosure classification → Enclosed building. (Section 26.10)
  The building is enclosed on all sides with ~~glass facade~~ glass or stone or concrete facade.

- Internal pressure coefficient → $GC_{pi} = \pm 0.18$ (Section 26.11)
  for enclosed building

(4) Velocity pressure exposure coefficient.

<u>Table 27.3-1.</u>

Height above ground = 177.6m / 582.5ft

Exposure → C → 1.77 = K_z

4

(5) Velocity Pressure

$$q_z = 0.00256 K_z K_{zt} K_d V^2$$
$$q_h = 0.00256 K_h K_{zt} K_d V^2$$

Calculations on excel sheet

(6) External Pressure coefficients.

Referring Table 27.4-1.

• Wall pressure coefficients

Windward → Long side E-W

→ $\frac{L}{B} = \frac{88.56}{208.3} = 0.43$

Windward → Short side N-S

→ $\frac{L}{B} = \frac{208.3}{88.56} = 2.35$

Windward → E-W → $C_p = 0.8$
Windward → N-S → $C_p = 0.8$
] with q_z

Leeward → E-W → $C_p = -0.5$
Leeward → N-S → $C_p = -0.28$
] with q_h interpolation between -0.3 and -0.2

Side wall → $C_p = -0.7$] with q_h

5

• Roof pressure coefficient

N-S Wind

$$\frac{h}{L} = \frac{238.7}{88.56} = 2.7 > 1.0 \Rightarrow$$

0 to $\frac{h}{2}$ → $\boxed{C_p = -1.3 \text{ (reducible)}}$

$> \frac{h}{2}$ → $\boxed{C_p = -0.7}$

Roof area = 208.3 × 88.56 = 18,447.04 sq ft > 1000 sq ft

So C_p values are reducible by 0.8 factor

E-W Wind

$$\frac{h}{L} = \frac{238.7}{208.3} = 1.15 > 1.0$$

0 to $\frac{h}{2}$ → −1.3 (reducible)
Roof area > 1000 sq ft

$> \frac{h}{2}$ → −0.7

(7) Wind pressures for rigid building

Section 27.4-1

Refer to excel sheet for calculation

For windward, leeward, side walls and roof

$$p = q_{h/z} G C_p - q_i G C_{p_i} \quad (lb/ft^2)$$

E-W Direction						
Windward pressure Cp =0.8					Story Shear	
Level	Elevation (ft)	q_z (lb/ft^2)	p (+internal pressure) psf	p (-internal pressure) psf	Tributary Area ft^2	Story force (kip)
Ground	0.00	38.6	20.3	16.9	3142.8	68.0
1A	20.34	41.1	21.6	18.0		
2A	30.18	44.6	23.5	19.6	2049.7	48.2
3A	40.02	47.4	25.0	20.8	2784.1	69.5
4,5	56.91	51.0	26.9	22.4	3125.7	84.0
6	70.03	53.3	28.1	23.4	2732.9	76.7
7	83.15	55.2	29.1	24.2	2732.9	79.6
8	96.27	57.0	30.0	25.0	2732.9	82.0
9	109.39	58.5	30.8	25.7	2732.9	84.3
10	122.51	59.9	31.6	26.3	2732.9	86.3
11	135.63	61.2	32.3	26.9	2732.9	88.2
12	148.75	62.4	32.9	27.4	2732.9	89.9
13	161.87	63.6	33.5	27.9	2732.9	91.5
14	174.99	64.6	34.0	28.4	2732.9	93.0
15	188.11	65.6	34.6	28.8	2732.9	94.5
16	201.23	66.5	35.1	29.2	2732.9	95.8
17	214.35	67.4	35.5	29.6	2811.5	99.9
18	228.22	68.3	36.0	30.0	2538.2	91.4
19	238.72	69.0	36.3	30.3	2534.8	92.1
Leeward pressure Cp =-0.5					Total Shear	1515.0
Level	Elevation (ft)	q_z (lb/ft^2)	p (+internal pressure)	p (-internal pressure)		
All	238.72	69.0	-39.9	10.8		
Side wall pressure Cp =-0.7						
Level	Elevation (ft)	q_z (lb/ft^2)	p (+internal pressure)	p (-internal pressure)		
All	238.72	69.0	-51.6	22.8		
Roof Pressures						
level	Elevation (ft)	qz (lb/ft2)	p (+internal pressure)	p (-internal pressure)		
0 to h/2	0 to 104.15 ft	69.0	-86.8	-65.7		
>h/2	>104.15	69.0	-51.6	-30.5		

N-s Direction						
Windward pressure Cp =0.8					Story Shear	
Level	Elevation (ft)	q_z (lb/ft^2)	p (+internal pressure) psf	p (-internal pressure) psf	Tributary Area ft^2	Story force (kip)
Ground	0.00	38.6	20.3	16.9	3142.8	68.0
1A	20.34	41.1	21.6	18.0		
2A	30.18	44.6	23.5	19.6	2049.7	48.2
3A	40.02	47.4	25.0	20.8	2784.1	69.5
4,5	56.91	51.0	26.9	22.4	3125.7	84.0
6	70.03	53.3	28.1	23.4	2732.9	76.7
7	83.15	55.2	29.1	24.2	2732.9	79.6
8	96.27	57.0	30.0	25.0	2732.9	82.0
9	109.39	58.5	30.8	25.7	2732.9	84.3
10	122.51	59.9	31.6	26.3	2732.9	86.3
11	135.63	61.2	32.3	26.9	2732.9	88.2
12	148.75	62.4	32.9	27.4	2732.9	89.9
13	161.87	63.6	33.5	27.9	2732.9	91.5
14	174.99	64.6	34.0	28.4	2732.9	93.0
15	188.11	65.6	34.6	28.8	2732.9	94.5
16	201.23	66.5	35.1	29.2	2732.9	95.8
17	214.35	67.4	35.5	29.6	2811.5	99.9
18	228.22	68.3	36.0	30.0	2538.2	91.4
19	238.72	69.0	36.3	30.3	2534.8	92.1
Leeward pressure Cp =-0.28					Total Shear	1515.0
Level	Elevation (ft)	q_z (lb/ft^2)	p (+internal pressure)	p (-internal pressure)		
All	238.72	69.0	-27.0	2.3		
Side wall pressure Cp =-0.7						
Level	Elevation (ft)	q_z (lb/ft^2)	p (+internal pressure)	p (-internal pressure)		
All	238.72	69.0	-51.6	22.8		
Roof Pressures						
level	Elevation (ft)	qz (lb/ft2)	p (+internal pressure)	p (-internal pressure)		
0 to h/2	0 to 44.3 ft	69.0	-86.8	-65.7		
>h/2	>44.3	69.0	-51.6	-30.5		

Appendix 5: Seismic Forces

Seismic Loads | Technical Report 1 | Punit G. Das | Page 1

Using ASCE 7-10, Section 12.8 Equivalent Lateral force procedure.

- Seismic Base Shear Calculation (Section 12.8.1)

$$V = C_S W$$

V = Seismic base shear
C_S = Seismic response coefficient
W = effective seismic weight

- Calculation of C_S (seismic response co-efficient)

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

S_{DS} = design spectral response acceleration parameter at short periods
R = Response modification factor.
I_e = Importance factor

$$S_{DS} = \frac{2}{3} S_{MS} \quad \text{(Section 11.4.4)}$$

$$S_{MS} = F_a S_S$$

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

$$S_{M1} = F_V S_1$$

S_{MS} = Maximum Considered earthquake Spectral response Acceleration parameter for short periods

S_{M1} = Maximum considered earthquake spectral response acceleration parameter for 1 s period.

$S_S = 25\%g$ for NYC (New york city)

$S_1 = 7.3\%g$ for NYC (New york city)

F_a, F_V = Site coefficients

S_S, S_1 = Mapped spectral response acceleration parameter at short periods, at a period of 1 second.

Assuming Newyork, NY because ~~the~~ ~~earth~~ its seismic behaviour is closest to that of Mumbai, India.

2

Site Class

The site contains Hard rock so site class A according to Table 20.3-1, Chapter 20.

From Table 11.4-1,

for site class A, $S_S = 0.25 \rightarrow F_a = 0.8$

for site class A, $S_1 = 0.073 \rightarrow F_v = 0.8$

$$S_{MS} = F_a S_S = (0.8)(0.25) = 0.2$$

$$S_{M1} = F_v S_1 = (0.8)(0.073) = 0.0584$$

$$S_{DS} = \frac{2}{3} S_{MS} = 0.133$$

$$S_{D1} = \frac{2}{3} S_{M1} = 0.0389$$

• ~~Calc~~ From figure 22-12

$T_L = 6$ for New York NY

so, $T = 1.21 < T_L$

$$\underset{\geq 0.01}{0.044 S_{DS} I_e} < C_S = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} < \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

$$(0.044) \times (0.133) \times (1) < \frac{0.133}{\left(\frac{4}{1}\right)} < \frac{0.0389}{(1.21)\left(\frac{4}{1}\right)}$$

• Calculating time period of building acc to section 12.8.2.1

$$T_a = C_t h_n^x$$

From Table 12.8-2

Concrete frame with shear walls $\rightarrow C_t = 0.02$

$x = 0.75$

h_n = structural height = 238.7 ft

$$T_a = (0.02)(238.7)^{0.75} = 1.21\ s$$

$R = 4$ for ordinary concrete shear wall Table 12.2-1

$I = 1$ from Table 1.5-2

3

$0.44 S_{DS} I_e \geq 0.01$

so, $0.01 < 0.03325 < 0.00837$ ok

$C_S = 0.03325$

Calculating effective seismic weight, follow section 12.7.2.

- Refer to excel sheet for seismic wt. calculation

$W = \cancel{28,173.2}\ 39420$ kips

$V = C_S W = 0.03325 \times \cancel{28,173.2}\ 39420$

$V = \cancel{936.8 \text{ kips}}\ 1310.7$ kips

- Vertical distribution of seismic forces (Section 12.8.3) (Refer to excel sheet attached)

$F_x = C_{vx} V \longrightarrow$ (equation 12.8-11)

C_{vx} = Vertical distribution factor.

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(equation 12.8-12)}$$

Period of building in N-S direction — 1.53 s $\rightarrow k = 1.56$

E-W direction — ~~0.0267~~ 1.0s $k = 0.5$

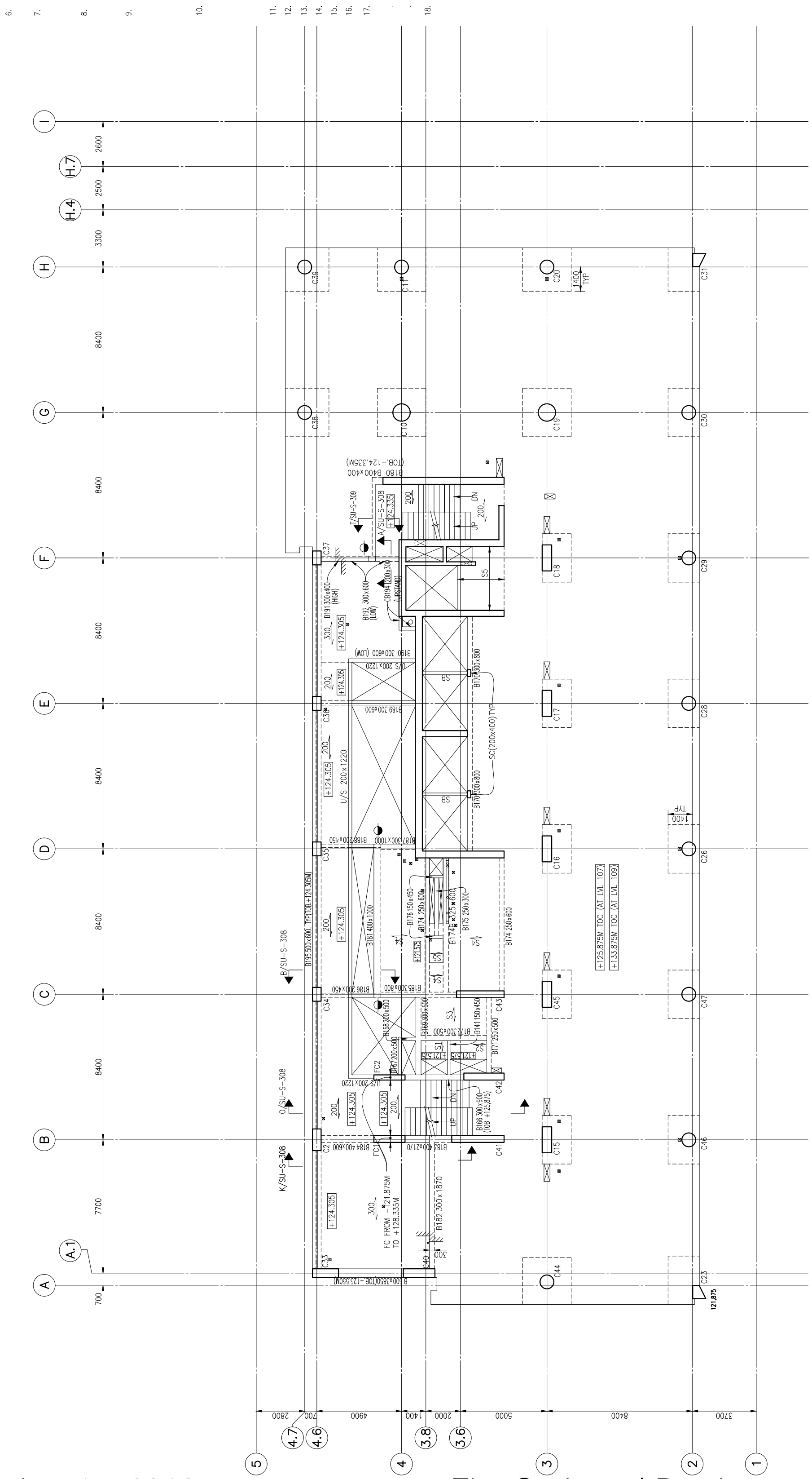
Floor weights				
	average slab thickness (ft)	total floor area (ft^2)	total dead load	25% live
1A	1.15	1860	320.292	18.6
2A	1.15	1860	320.292	18.6
3A	1.15	1860	320.292	18.6
Level5	0.98	1860	274.536	46.5
6	0.98	1400	206.64	35
7	0.98	1400	206.64	35
8	0.98	1400	206.64	35
9	0.98	1400	206.64	35
10	0.98	1400	206.64	35
11	0.98	1400	206.64	35
12	0.98	1400	206.64	35
13	0.98	1400	206.64	35
14	0.98	1400	206.64	35
15	0.98	1400	206.64	35
16	0.98	1400	206.64	35
17	0.98	1400	206.64	35
roof	0.98	1400	206.64	35
		total	3921.732	557.3
		Total load (kip)		4479.0

Columns				
level	Total cross-sectional area (ft^2)	total height (ft)	total volume	total weight kip
1A	310.5	56.91	17669.9	2650.5
2A	310.5	56.91	17669.9	2650.5
3A	310.5	56.91	17669.9	2650.5
4,5	310.5	56.91	17669.9	2650.5
6	152.55	16.89	2576.9	386.5
7	152.55	16.89	2576.9	386.5
8	152.55	16.89	2576.9	386.5
9	152.55	16.89	2576.9	386.5
10	152.55	16.89	2576.9	386.5
11	152.55	16.89	2576.9	386.5
12	152.55	16.89	2576.9	386.5
13	152.55	16.89	2576.9	386.5
14	152.55	16.89	2576.9	386.5
15	152.55	16.89	2576.9	386.5
16	152.55	16.89	2576.9	386.5
17	152.55	13.87	2116.5	317.5
18	152.55	10.50	1601.2	240.2
19	152.55	13.84	2111.5	316.7
			total	15728.18958

Shear walls				
level	totl cross section area (ft^2)	tota height (ft)	total volume	total weight kip
1A	640.8	9.84	6305.2	945.8
2A	640.8	9.84	6305.2	945.8
3A	640.8	16.89	10823.9	1623.6
4,5	640.8	16.89	10823.9	1623.6
6	418.8	16.89	7074.8	1061.2
7	418.8	16.89	7074.8	1061.2
8	418.8	16.89	7074.8	1061.2
9	418.8	16.89	7074.8	1061.2
10	418.8	16.89	7074.8	1061.2
11	418.8	16.89	7074.8	1061.2
12	418.8	16.89	7074.8	1061.2
13	418.8	16.89	7074.8	1061.2
14	418.8	16.89	7074.8	1061.2
15	418.8	16.89	7074.8	1061.2
16	418.8	16.89	7074.8	1061.2
17	418.8	13.87	5810.9	871.6
18	418.8	10.50	4396.0	659.4
19	418.8	13.84	5797.2	869.6
			Total	19212.733

Story Forces							k=	1.56
Level	w_x	$\sum w_i$	h_x	h_x^k	$w_i h_i^k$	$\sum w_i h_i^k$	Cvx	Fx
1A	3614.9	39419.95	20.34	109.87	397179	59563375	0.01	8.7
2A	3614.9	39419.95	30.18	203.36	735131	59563375	0.01	16.2
3A	4292.7	39419.95	40.02	315.85	1355852	59563375	0.02	29.8
5	4320.6	39419.95	56.91	547.10	2363806	59563375	0.04	52.0
6	1482.7	39419.95	70.03	756.18	1121222	59563375	0.02	24.7
7	1482.7	39419.95	83.15	988.48	1465671	59563375	0.02	32.3
8	1482.7	39419.95	96.27	1242.32	1842041	59563375	0.03	40.5
9	1482.7	39419.95	109.39	1516.33	2248331	59563375	0.04	49.5
10	1482.7	39419.95	122.51	1809.41	2682898	59563375	0.05	59.0
11	1482.7	39419.95	135.63	2120.63	3144362	59563375	0.05	69.2
12	1482.7	39419.95	148.75	2449.20	3631544	59563375	0.06	79.9
13	1482.7	39419.95	161.87	2794.42	4143418	59563375	0.07	91.2
14	1482.7	39419.95	174.99	3155.68	4679080	59563375	0.08	103.0
15	1482.7	39419.95	188.11	3532.44	5237727	59563375	0.09	115.3
16	1482.7	39419.95	201.23	3924.23	5818640	59563375	0.10	128.0
17	1224.1	39419.95	214.35	4330.58	5301160	59563375	0.09	116.7
18	934.6	39419.95	228.22	4775.72	4463255	59563375	0.07	98.2
19	1743.6	39419.95	238.72	5122.74	8932057	59563375	0.15	196.6
						total base shear		1310.7

Appendix 6: Level 7 plan



MARK	THICKNESS	BOTTOM REINFORCEMENT ALONG N-S SPAN	BOTTOM REINFORCEMENT ALONG E-W SPAN	TOP REINFORCEMENT ALONG N-S SPAN	TOP REINFORCEMENT ALONG E-W SPAN	REMARKS
S1	150	–	–	T8 @ 250	T10 @ 200	
S2	150	T10 @ 250	T8 @ 300	T8 @ 300	T10 @ 300	
S3	150	T10 @ 250	T12 @ 250	–	T10 @ 300	
S4	150	T16 @ 250	T10 @ 300	–	–	
S5	325	T12 @ 250	T12 @ 250	T10 @ 300	T10 @ 300	