AE SENIOR THESIS 2012-13

FINAL REPORT

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Structural Option
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Signature Boutique Offices
India

The Optimus

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Executive Summary

This final senior thesis report presents a redesign of a building with existing structure in concrete to a structure in steel. The existing building is called The Optimus. It is a 17 story office building with 5 stories of parking garage, ground floor retail and a recreation space at the roof. The building is 252 ft tall and located in India. It is a part of a huge redevelopment project that consists of residential and commercial spaces. The flat slab floor system provides an open floor plan and customizable space for the offices. The building has a large glass and metal facade, a stone wall and a green wall as part of the building envelope. The main gravity system consists of flat slabs supported on reinforced gravity columns and lateral system is a reinforced concrete shear wall located around the elevator shafts.

Major part of this report presents redesign of the structural system of the building in steel. This is being done to study the advantages of a steel building over concrete in India where, concrete is the first choice in building material. However, as the country is progressing, the cities are getting denser and riche; this is currently putting pressure on the construction industry to building more efficient, taller and innovative structures. One of the solution to this challenge is to switch the building material from concrete to steel.

The existing concrete gravity columns are converted to steel columns. Interior columns are steel columns encased with reinforced concrete. The lateral system is converted from existing reinforced concrete shear walls in the interior to braced frames with HSS braced and steel wide flange encased with reinforced concrete columns. The braced frames are moved to the exterior of the building. Also, a typical steel connection for moment frame is designed as part of the structural system. The site for redesign of the building is Mumbai, India and the structural redesign is carried out using ASCE 7-10 specifications and AISC Manual specifications.

The amount of changes in the structural system has a huge impact on the architecture of the building. Hence, as part of the first breadth, the integration of structure with architecture is being analyzed. Each structural redesign has and impact on the architecture which affects the interior and exterior of the building. Therefore, the co-ordination of architecture and structure is discussed in the report.

A part of the integration of structure and architecture is the building facade. The redesign in structure has completely transformed the facade of the building. As part of the second breadth analysis, the architecture of the facade of the building is analyzed in response the structural changes. The facade of the existing building was design to maintain a healthy indoor environment by controlling amount of sunlight and heat penetrating into the building. Therefore, the report further discusses the strategies to achieve an equally comfortable indoor environment.

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I would also like to thank my parents, and my close friends in US and back home for their relentless support during this whole process.

Thesis Abstract

The Optimus

Punit Das
Structural Option | 2012-2013

GENERAL INFORMATION

- Function — Offices + Retail + Parking
- Total Area — 430,000 sq. ft
- Height — 230 ft
- Floors — Ground + 17 floors
- Construction — January 2012 - October 2013
- Project Delivery — Design - Bid - Build

PROJECT TEAM

- Owner + Project Manager + General Contractor — Lodha Group
- Lead Architect — Pei Cobb Freed and Partners Architects
- Local Architect — Edifice Consultants Pvt. Ltd.
- Structural Engineer — Leslie E. Robertson Associates RLLP
- MEP + Fire Protection Consultant — Spectral Consultants Pvt. Ltd.
- Lighting Designer — George Sexton Associates

Vertical Transportation — Barker Mohandas

ARCHITECTURE

- 2 basements + 4 floors of parking space
- Ground floor retail
- Office spaces from 5th to 16th floor
- Roof: Gymnasium, Cafeteria and Garden
- 3 typical floor plans for different office requirements
- South façade windows for daylighting and panoramic views
- Utility areas located at north façade
- Parking spaces pushed to the rear of the building to show a unified front façade
- Maximum use of building footprint by integrating functional spaces inside the building mass

STRUCTURE

- Quality interior and exterior spaces due to Architecture and Structure integration
- Reinforced concrete frame with concrete core wall and flat slab system
- Column cross sections chosen to fit the functionality of spaces
- Rectangular columns ranging from 18 x 18 inches to 18 x 80 inches in parking spaces to provide maximum parking space
- Circular columns with about 20 inch diameter in office spaces to provide open floor plan and improve aesthetic quality
- Lateral system consists of elevator core and stairwell core with 12 - 20 inches thick reinforced concrete shear walls
- 8 in flat slabs with 16 in drop panels as required
- 50 to 80 inches thick Mat Foundation with pressure slab to resist hydrostatic pressure

Roof Garden

FACADE

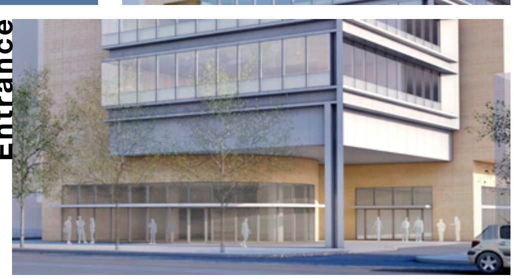
- Different façade systems used to highlight building mass
- Locally available decorative stone envelopes utility areas
- Architectural green wall wraps around parking spaces facing residential apartments
- Green wall acts as a sound and air barrier between parking areas and surrounding areas
- Metal and Glass curtain wall envelopes the south and west façade
- Windows on south façade pushed 2ft inwards to provide solar shading
- Windows on west façade extrude outwards to maximize daylighting

Facade

MEP and LIGHTING SYSTEMS

- Dedicated mechanical and electrical room at each floor eliminating roof top centralized mechanical spaces
- Tenant specific HVAC system selected after floors are rented out
- Main MEP rooms located on the ground floor to provide ease of access
- Energy efficient lighting provided with collaboration of architect and lighting consultant
- LED fixtures and Compact fluorescents used in office spaces and lobby areas
- Metal halides used in public spaces

Entrance



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Overview of existing conditions

Building Introduction

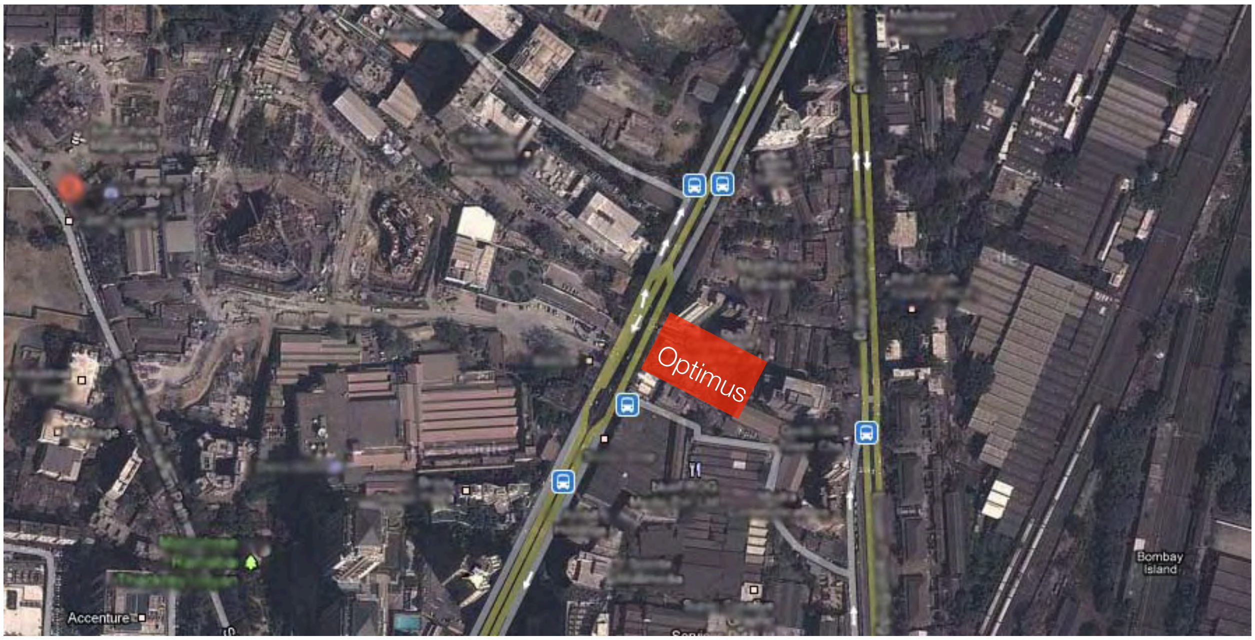


Figure 1 Aerial map from Google.com showing the location of the building site.

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 5 stories of parking and ground floor retail.

Architecture


Figure 2 Rendering showing roof garden

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 5^{th} level as garden, lobby and office. The office spaces start from 6 to 17^{th} story and 18^{th} story contains a roof garden.


Figure 3 Rendering of the building entrance

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. 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.

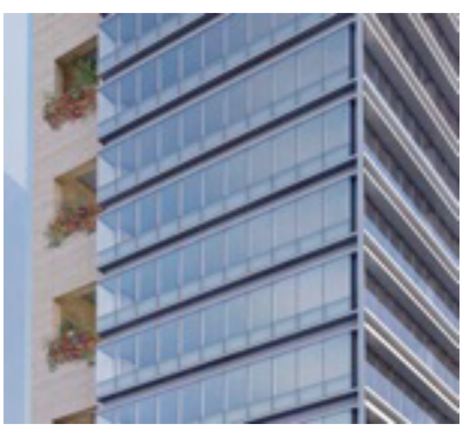


Figure 4 Rendering of the building facade

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.

Structural System

Structural system of The Optimus is designed by Leslie E. Robertson Associates R.L.L.P. 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 easily 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.

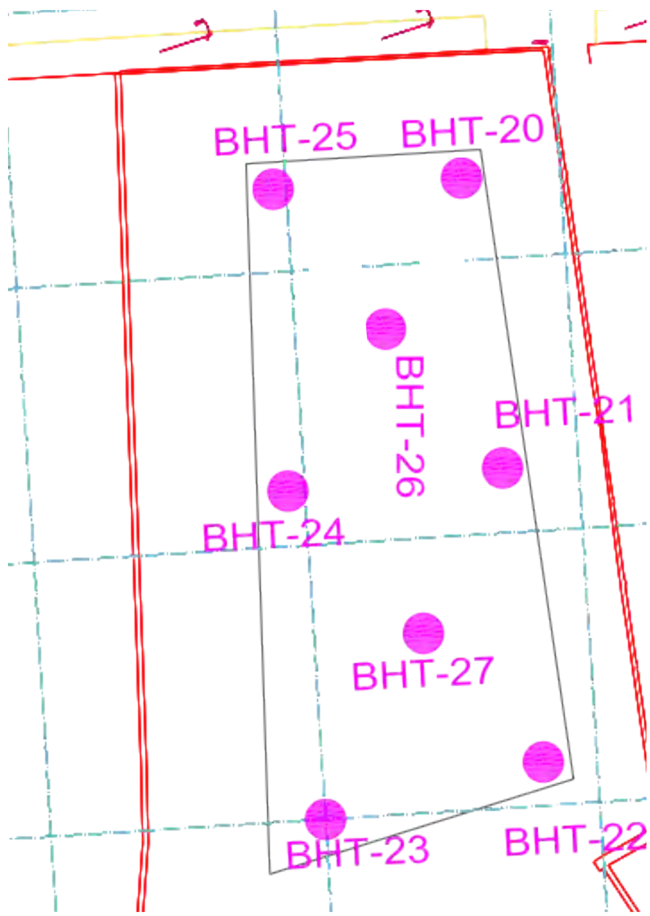


Figure 5 Boring test map on the building site.

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. Shoring piles were built to retain soil from excavation area during construction of basement floors. 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

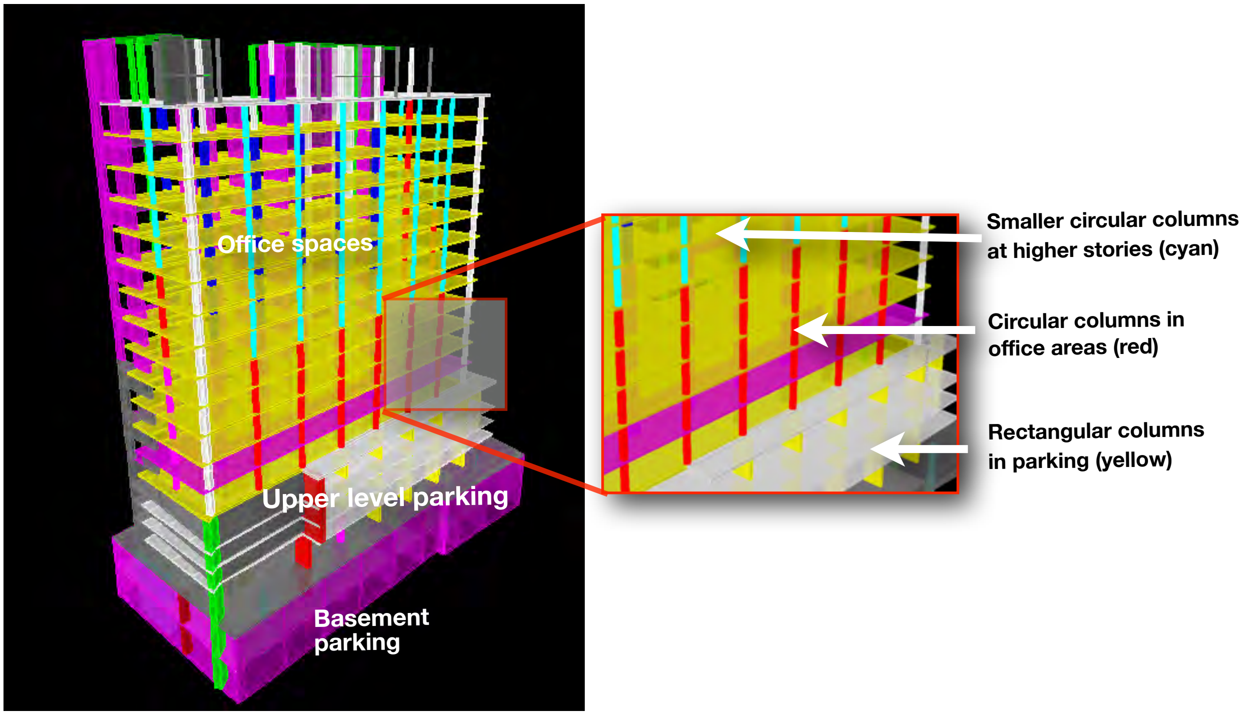


Figure 6: ETABS model, 3D view extruded.

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 columns and 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

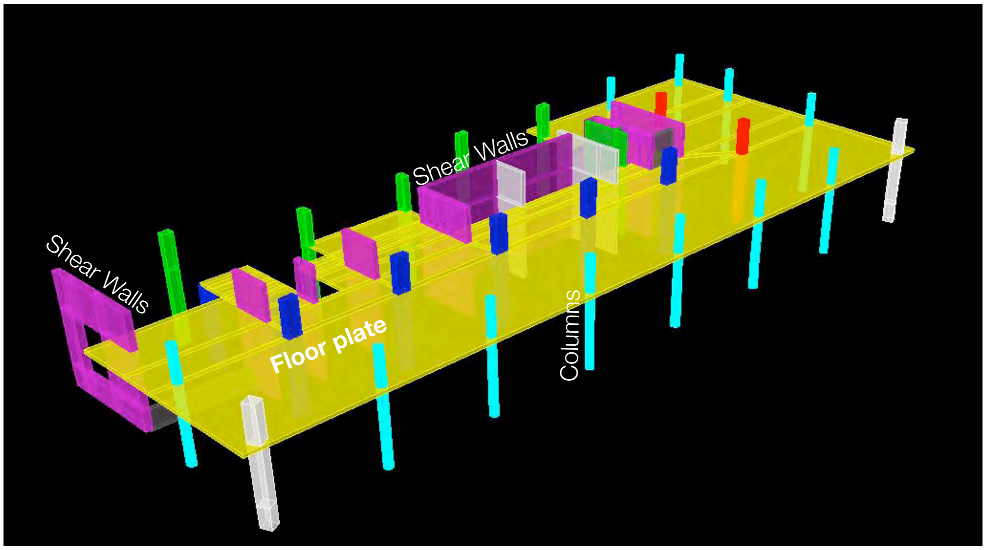


Figure 7: ETABS model, 3D view of floor plan.

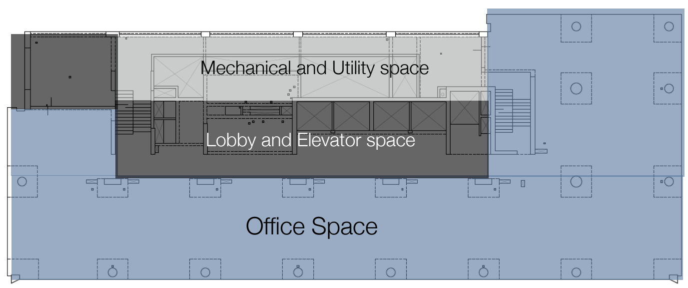


Figure 8: Division of floor space area for typical office floor.

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 the 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.

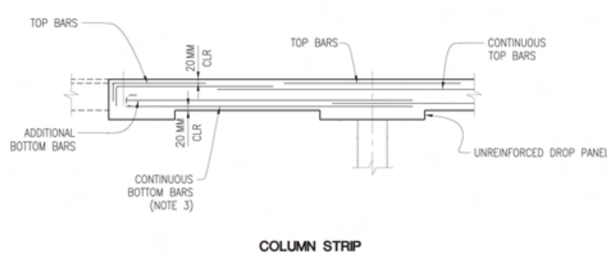


Figure 8: Section of column strip for typical slab

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.

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

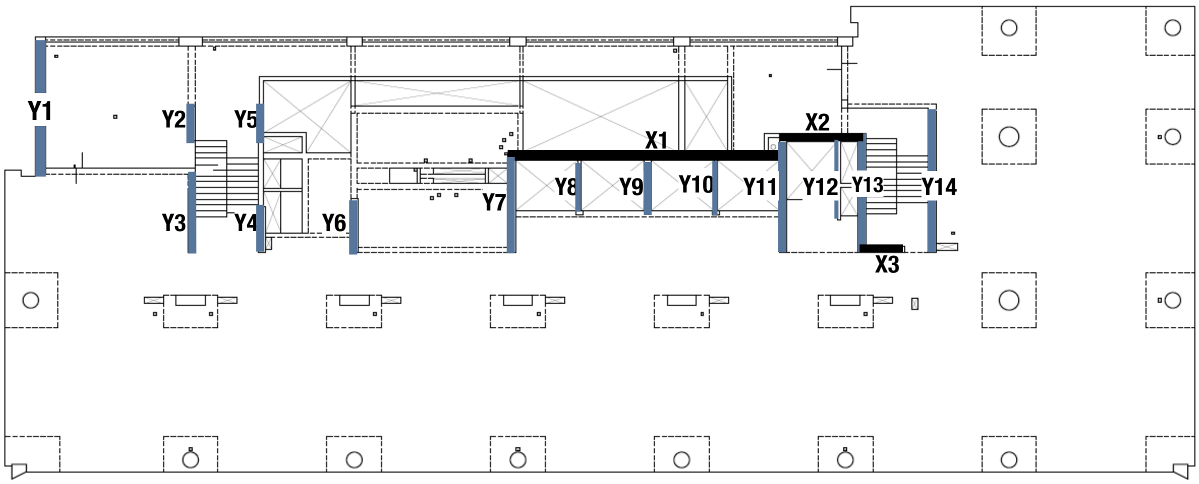


Figure 9: Shear Walls labelled for a typical office floor plan.

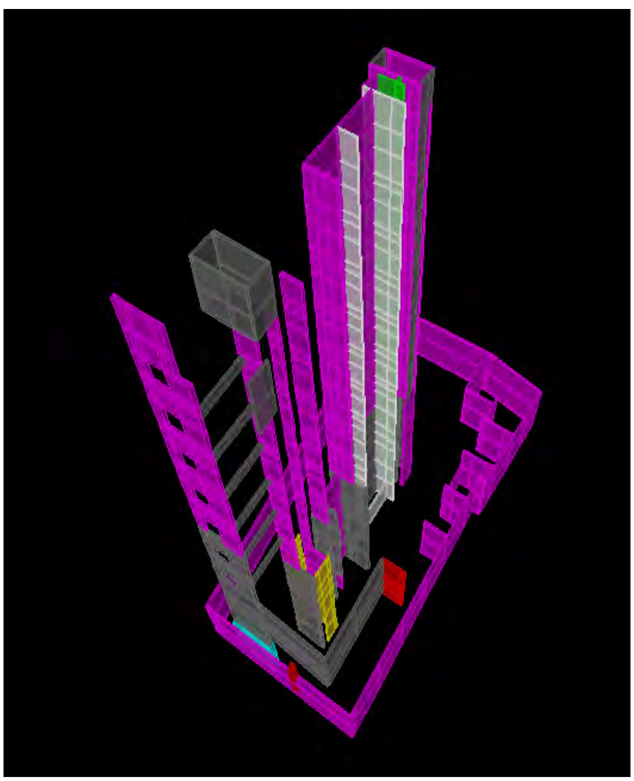


Figure 10: Shear walls in 3D extruded view.

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.

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 2950	Indian Standard Code of Practice for Design and Construction of Raft Foundation (Part –1)

IS Code	Description
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 redesign:

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.
AISC Steel Construction Manual	Steel Design code

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

Concrete					
Indian Code			American Code		
Concrete Grade	f'c (psi)	Ec (ksi)	Equivalent Concrete type	f'c	$E_c = 57000\sqrt{f'c}$ (ksi)
M60	7000	5614.3	High strength concrete 28 days	7000 psi	4768.9
M40	4700	4584.3	Ordinary ready mix	5000 psi	4030.5
M15	1750	2807.2	Ordinary ready mix	3000 psi	3122.01
fck is 28 compressive strength for 150mmx150mm cube. Poission's ratio = 0.2 Coefficient of thermal expansion = 9.9x10-0.6 per deg C.			f'c - specified compressive strength of concrete. Coefficient of thermal expansion = 5.5×10^{-6} per deg F. Poissions ratio = 0.2		
Reinforcement					
According to IS: 1786 Fe 415 (Fy = 415 MPa/ 60 ksi) or Fe 500 (Fy = 500 MPa) steel bars are used.			According to ASTM A615, deformed and plain carbon steel bars are used with Fy = 60 ksi.		

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

	ACI 318-11 / ASCE 7-10 (lb / ft^3)
Normal weight Concrete	150
Floor finishes / Plasters	140

Loading Area	Type of Load	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Parking Space and Drive-way	Superimposed Dead Load	36.6
	Live Load (vehicles)	40 non-reducible
	Live Load (fire truck over ground floor)	300 (AASHTO LRFD Bridge design standards) - non-reducible
Covered Entryway over ground floor	Superimposed Dead Load	151.4
	Live Load	100
Entrance Lobby, Elevator lobbies	Superimposed Dead Load	41.8
	Live Load	100
Mechanical Floor	Superimposed Dead Load	41.8
	Live Load	150 non-reducible
Electrical room over ground floor	Superimposed Dead Load	41.8
	Live Load	282 non-reducible
Stairs	Superimposed Dead Load	31.33
	Live Load	100

Loading Area	Type of Load	ACI 318-11 / ASCE 7-10 (lb / ft^2)
Restrooms	Superimposed Dead Load	94
	Live Load	40
Typical Office	Superimposed Dead Load	62.7
	Live Load	100
Retail over ground floor	Superimposed Dead Load	95.6
	Live Load	100
Eatery and Utility	Superimposed Dead Load	62.7
	Live Load	100
Outdoor Utility over Level 105, 107 and similar	Superimposed Dead Load	117.5
	Live Load	100
Planted Terrace	Superimposed Dead Load	261.1
	Live Load	100
Amenity / Fitness Center	Superimposed Dead Load	73.10
	Live Load	100
Water tank over level 119	Superimposed Dead Load	73.1
	Live Load	731 non-reducible
Electrical Panel room at ground floor	Superimposed Dead Load	41.8
	Live Load	282 non-reducible
Roof	Superimposed Dead Load	114.9
	Live Load	100 non-reducible
Peripheral loads	Superimposed Dead line load over wall surface	15.7

Proposed Redesign in Steel

Problem Statement

The existing structural design of The Optimus is adequately optimized according to the requirements of the owner, architect, structural engineer and all the professionals involved in the project. This fact has been proven in the technical reports. Also, the structural system is integrally designed with all other systems. Overall, the Optimus fits well with the type of construction that is widely accepted and used all over India.

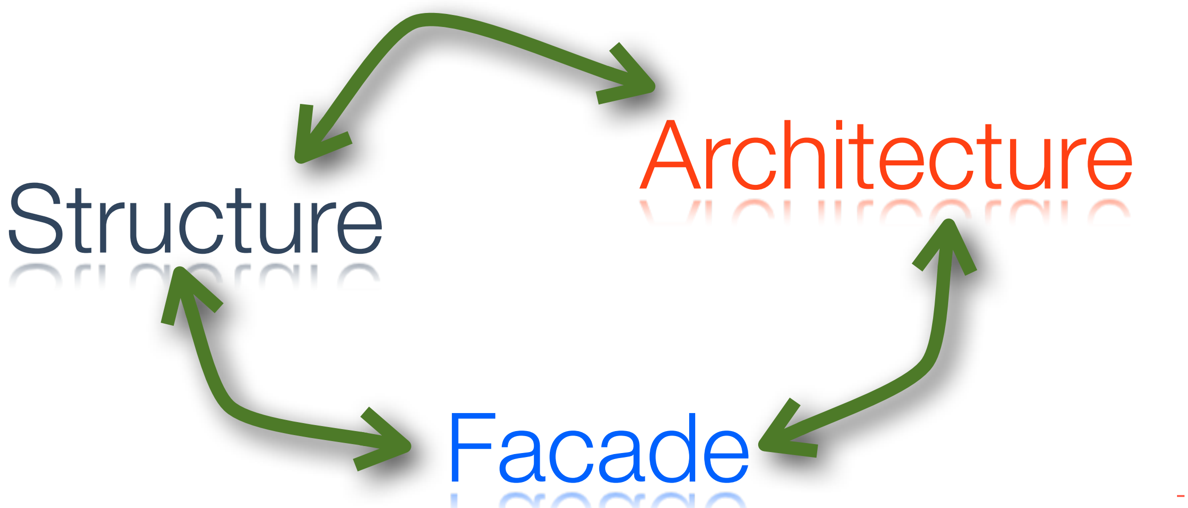
Majority of the buildings in India are constructed using reinforced concrete. This is because, labor and resources for concrete construction are easily available. The knowledge and problem solving help for designing concrete structures is also readily available due to widespread accepted concrete design. Concrete design is also given primary importance while teaching in universities across India. Structural engineers lean towards the more profitable choice because of the deeply accepted methods of concrete construction among architects and owners.

As compared to concrete, steel is hardly a preferred choice in construction of multi-story buildings in India. The skyline of major metropolitan cities is dominated with concrete buildings. Steel is only used in industry buildings and infrastructure projects like bridges, railway stations and airports. In india, as the cities growing denser, bigger and richer, the demand for taller buildings is rising. Each year the city of Mumbai comes up with a taller building. At present, there are 3 supertall building projects planned in Mumbai which plan to go taller than the empire state building. The requirement for taller and more refined buildings has generated the need for new technologies. These challenges are being met with new resources and designs. One of the ways to face the challenge of taller and sustainable buildings is by harnessing the benefits of steel construction. One of the examples is evident in the recent completion of the tallest steel building in India - The Sunshine towers in Mumbai. Having studied the advantages of steel over concrete, it was decided to explore how The Optimus would respond when converted from Concrete to a steel structure.

Proposed Solution

In the future, as India is going to be advancing towards taller and sustainable buildings, the use of steel as a major building material is going to be inevitable. Therefore, in order to learn the design using steel and explore the pros and cons of steel over concrete The Optimus was redesigned in Steel. The steel and composite structural system of The Optimus consists of steel gravity members and a composite braced frame system. This report presents the pros and cons of using steel to build a multi-story building in Mumbai, India.

The progress of construction industry in India is not only based on use of steel, but also in the refinement of the construction process. This refinement is being met with greater integration of disciplines in the construction process. In order to highlight this integration, this report also presents the integration of structural engineering with architecture and facade design as a part of the breadth study.



Proposed Solution: Structural Depth

The solution presents a new structural system redesigned in Steel. The existing reinforced - concrete gravity columns have been converted to steel and steel wide-flange with concrete encased gravity columns. The flat slab floor system is converted to a composite steel deck over wide-flange beam floor system. The lateral force resisting system has been completely redesigned and optimized from a reinforced concrete shear wall around the elevator shafts to a steel braced composite wide flange encased in concrete system. Instead of placing the lateral force resisting system around the elevator shafts, it has been moved to the exterior perimeter of the building where, the continuous cross-bracing also serves as an architectural element. The lateral system also consists of two moment-resisting frames in the North-South direction. A typical girder-column connection of the moment frame is also part of the design in the report. The redesign in steel system has been compared to the existing system in concrete. This comparison is based on cost of the structural system and architecture of the building.

Proposed Solution: Architectural Breadth

On a real construction project, a slight change in the structural system requires thorough co-ordination with the architect. The transition from concrete to steel transforms all the architectural features of the building. The building goes from looking a monolithic concrete structure to a tectonic structure in steel. In this report several architectural modifications to the existing architecture has been carried out to adapt to the structural system and vice versa. This creates an integrated architectural-structural system to achieve economic efficiency, maximum rentable space and an ambient environment for the inhabitants. This report highlights some major structure-architecture integrations carried out while redesigning the steel structure.

Proposed Solution: Building facade analysis

Along with architecture, the building facade has undergone significant transformation to respond the change in the building structure. Daylighting and energy analysis are few of the major criteria for designing the facade of the building. The building facade is modified to control to amount daylight and heat penetrating in the building to achieve high levels of human comfort. Finally, as India is trying to catch up with race for sustainability; LEED rating is getting more and more prevalent. Therefore, this report studies how the redesign in steel has helped in making the building more sustainable.

Structural Depth Study

The structural depth includes analysis and redesign of gravity and lateral system of The Optimus in steel. A logical linear design and analysis process was followed to achieve an efficient design. The overall goal of the depth was to design an efficient gravity and lateral system, to design a typical moment connection, revaluate loads on the foundation and finally compare the market cost of the steel and concrete structural system in India.

The first step towards design a structural system is to define all the loads and load combinations to be used in the process. While switching from concrete to steel system, the existing superimposed dead loads, live loads and mechanical loads were kept constant. This is because the intent of the thesis is to study the outcome of a constant loading condition on a different building material - steel. ASCE 7-10 code was used to acquire loads that were not specified in design criteria of the existing system. These loads include wind and seismic loads and analysis procedures. The design load combinations were used from ASCE 7-10. Using these loads, the steel and composite-steel structural system was designed using AISC specifications and design tables. The following list illustrates the procedure that was followed after defining all the loads :-

1. Schematic design and layout of columns in steel
2. Design of the floor system and revaluate schematic column design
3. Produce a Finite Element Model of the system in ETABS and apply gravity, wind and seismic loads calculated previously
4. Define and layout the lateral force resisting system
5. Use ASCE 7-10 to calculate wind loads using Directional procedure

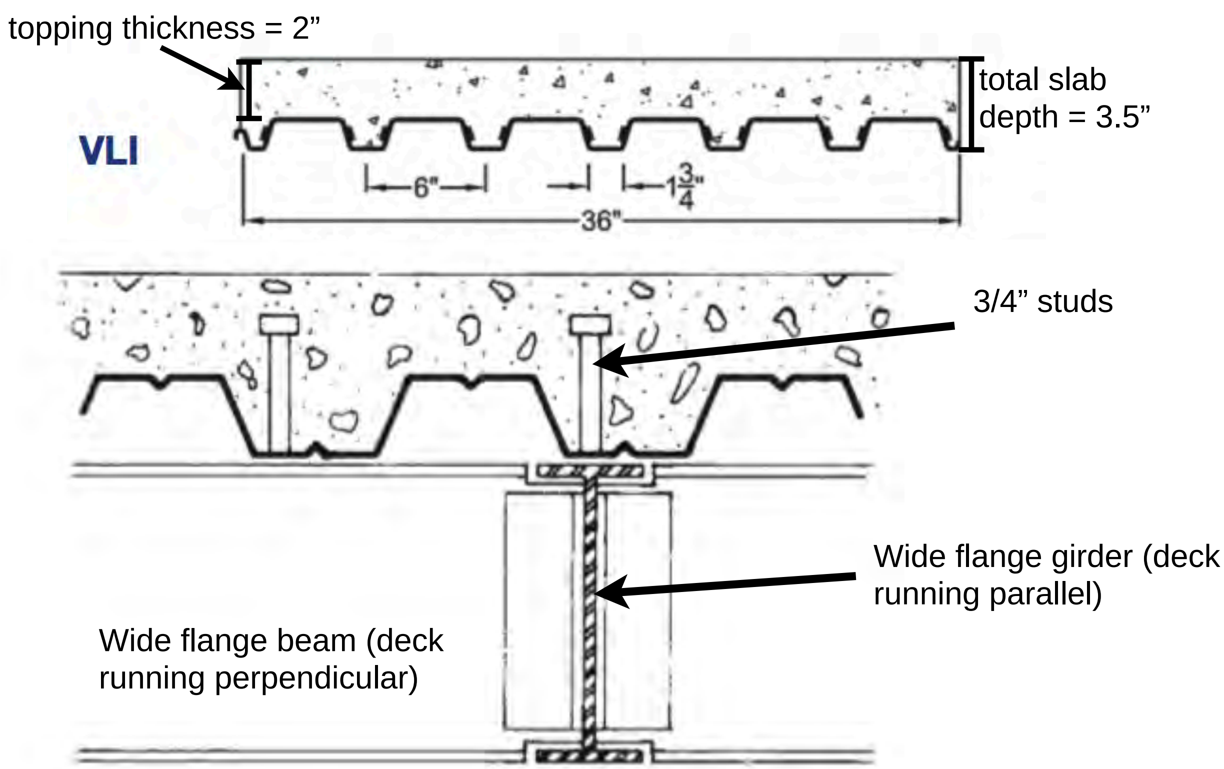
6. Calculate seismic loads using ASCE -7-10 Equivalent Lateral Force Procedure
7. Perform Modal response spectrum analysis to find base shears and story drifts
8. Determine critical wind and seismic load cases and apply it to ASCE 7-10 design load combinations
9. Reevaluate the gravity system using the loads from ETABS
10. Determine critical design load cases and design the lateral force resisting system to the required load capacity and to control lateral drifts.
11. Determine critical loads on moment frame connections and design a typical connection for moment frame.
12. Calculate critical loads on the foundations

A fairly efficient system was achieved using this design process. However, greater efficiency can be achieved by increasing the design iterations and revaluation process using the calculation power of a FEM modeling software like ETABS and manually created MS Excel design sheets. The conjunction of these two strategies has helped make the process faster and accurate.

Gravity System: Redesign and Analysis

The Gravity system of this building includes composite floor system, composite concrete-encased steel columns in the interior and steel wide flange columns at the perimeter. The intent of the design to achieve maximum advantage of the properties of steel without increasing the cost of the structure. Therefore, composite sections were used instead of heavy steel sections for an efficient design. Dead loads of the new system were used for design while superimposed dead loads were used from the existing design implying that the architectural elements were not compromised due to change in structural material. Loads not mentioned in the existing system were extracted from ASCE 7-10 design loads. ASCE 7-10 was also used to get LRFD load combinations to determine critical design loads. The critical design load 1.2Dead + 1.6Live has been used to design the floor system and gravity columns.

Composite Floor system design



As a first step towards designing the gravity system, columns were laid out using the existing architectural drawings and the location where concrete columns were replaced with steel sections. A logical decision was made to use composite floor system instead of regular metal decking on wide-flange section. The composite floor system was selected to reduce beam depths by using the compression capacity of concrete on metal decking placed over the wide-flange sections. AISC Specifications and load tables were used to design a partially-composite beam section and control live load deflections. As the grids are 27’6” squares, the beams were oriented in East-West direction. This orientation prevents design of girders in East-west direction which is exposed to the exterior and vice-versa. It will also give an unobstructed view to the outside. Also, the orientation in this direction facilitates running mechanical ducts and electrical wiring for the office floor. Composite Floor design was performed for typical floor spaces - parking levels, typical office floors, restrooms, mechanical areas and roof.

Column Design

1	1.4(D+SDL)
2	1.2(D+SDL) + 1.6L + 0.5RL
3A	1.2(D+SDL) + 1.6RL + L
3B	1.2(D+SDL) + 1.6RL + 0.5WX + 0.5LX
3C	1.2(D+SDL) + 1.6RL + 0.5WY + 0.5LY
4A	1.2(D+SDL) + 1.0WX +1.0LX + L + 0.5LR
4B	1.2(D+SDL) + 1.0WY +1.0LY + L + 0.5LR
5	1.2(D+SDL) + 1.0E + L
6A	0.9(D+SDL) + 1.0WX + 1.0LX
6B	0.9(D+SDL) + 1.0WY + 1.0LY
7	0.9(D+SDL) + 1.0E

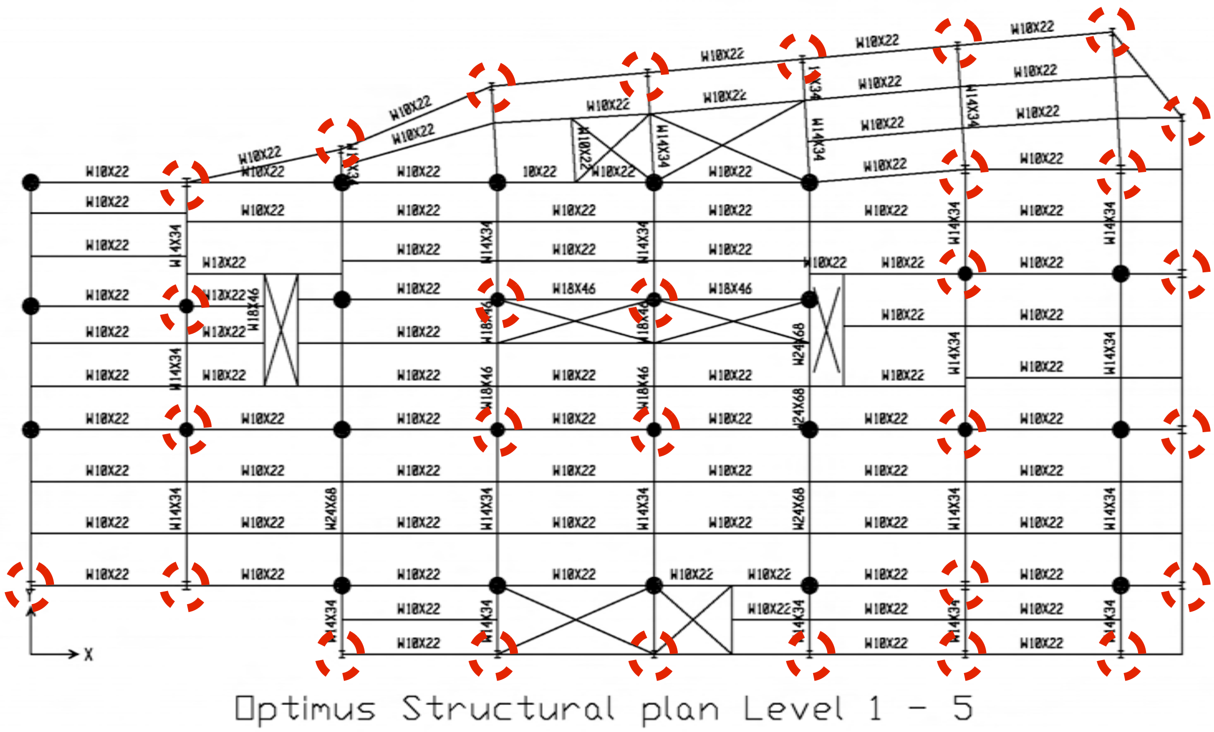
WX and LX =critical X dir wind load
E = critical seismic loads

The design of the floor system facilitated the calculation of floor dead loads that was further used in the design of columns. Initially, steel columns were manually designed using regular dead loads and superimposed dead loads transferred to the columns via floor girders. A finite element model was created using the manually designed columns and composite floor system. Additional superimposed dead loads from facade, brick walls etc were added to ETABS model which increased load demand from gravity columns. Therefore, in order to design gravity columns with increased load capacity and greater efficiency, it was decided to reduce the cross-sectional area of columns and encased it with reinforced concrete section to take advantage of composite behavior of concrete and steel. Also, it was decided to avoid heavy steel sections due to higher cost of steel as compared to concrete. Hence, a composite section would help in balancing the cost and strength of columns.

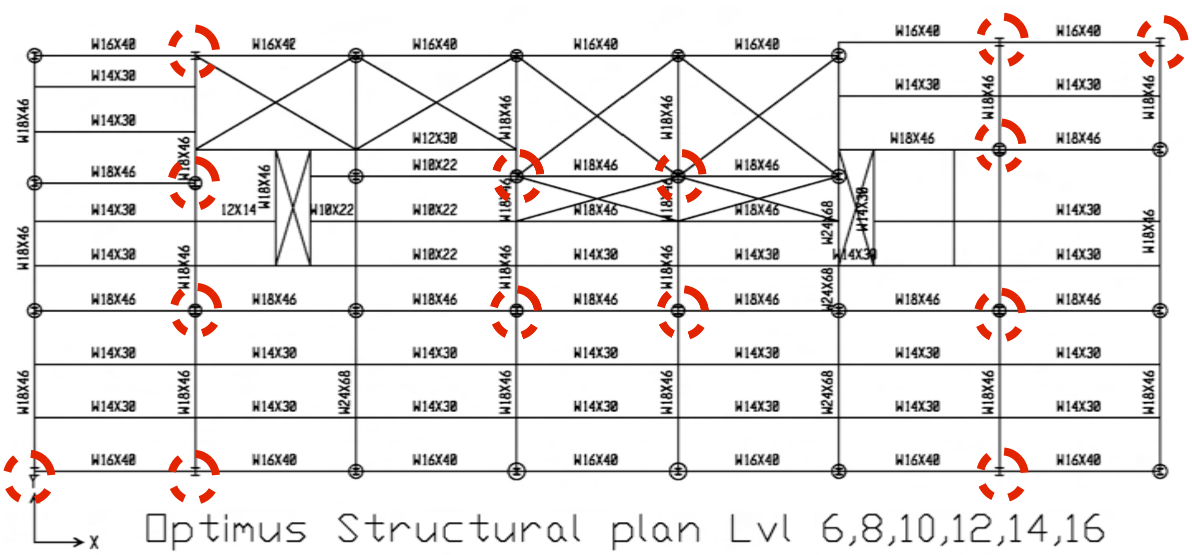
Design summary of critical interior gravity column							
Story	Column	Critical Load Combos	P (kip)	Member	ϕPn (kip)	DCR ratio	
LEVEL 1	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-3706	W14x176 dia28	3750	0.99	Composite columns- Reinforced concrete encasing steel wide flange columns. The size of steel section and concrete encasing decreases as loads decrease with increasing height for structural efficiency.
LEVEL 2	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-3572	W14x176 dia28	3750	0.95	
LEVEL 3	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-3402	W14x176 dia26	3625	0.94	
LEVEL 4	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-3271	W14x176 dia26	3625	0.90	
LEVEL 5	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-3132	W14x145 dia 26	3315	0.94	
LEVEL 6	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-2918	W14x145 dia 26	3315	0.88	
LEVEL 7	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-2691	W14x145 dia 26	3315	0.81	
LEVEL 8	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-2464	W14x120 dia 22	2587	0.95	
LEVEL 9	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-2236	W14x120 dia 22	2587	0.86	
LEVEL 10	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-2010	W14x120 dia 22	2587	0.78	
LEVEL 11	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-1782	W14x120 dia 22	2587	0.69	
LEVEL 12	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-1556	W14x120 dia 22	2815	0.55	
LEVEL 13	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-1329	W14x120	1400	0.95	Composite section was not required for loads from level 13 to roof.
LEVEL 14	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-1105	W14x120	1400	0.79	
LEVEL 15	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-875	W14x90	1050	0.83	
LEVEL 16	C1	1.2(D+SDL) + 1.6L + 0.5Roof L	-651	W14x90	1050	0.62	
LEVEL 17	C1	1.2(D+SDL) + 1.6RL + L	-460	W14x61	599	0.77	
ROOF	C1	1.2(D+SDL) + 1.6RL + L	-278	W14x61	599	0.46	

Design summary of critical edge gravity column						
Story	Column	Critical Load Combos	P (kip)	Member	ϕPn (kip)	DCR ratio
LEVEL 1	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-2201	W14x257	2660	0.83
LEVEL 2	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-2121	W14x257	2660	0.80
LEVEL 3	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-2042	W14x176	2090	0.98
LEVEL 4	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1964	W14x176	2090	0.94
LEVEL 5	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1887	W14x176	2090	0.90
LEVEL 6	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1737	W14x176	2090	0.83
LEVEL 7	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1658	W12x152	1690	0.98
LEVEL 8	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1462	W12x152	1690	0.87
LEVEL 9	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1383	W12x152	1690	0.82
LEVEL 10	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1188	W12x152	1690	0.70
LEVEL 11	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-1110	W12x106	1170	0.95
LEVEL 12	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-916	W12x106	1170	0.78
LEVEL 13	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-837	W12x106	1170	0.72
LEVEL 14	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-643	W12x72	806	0.80
LEVEL 15	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-566	W12x72	806	0.70
LEVEL 16	C40	1.2(D+SDL) + 1.6L + 0.5Roof L	-372	W12x50	413	0.90

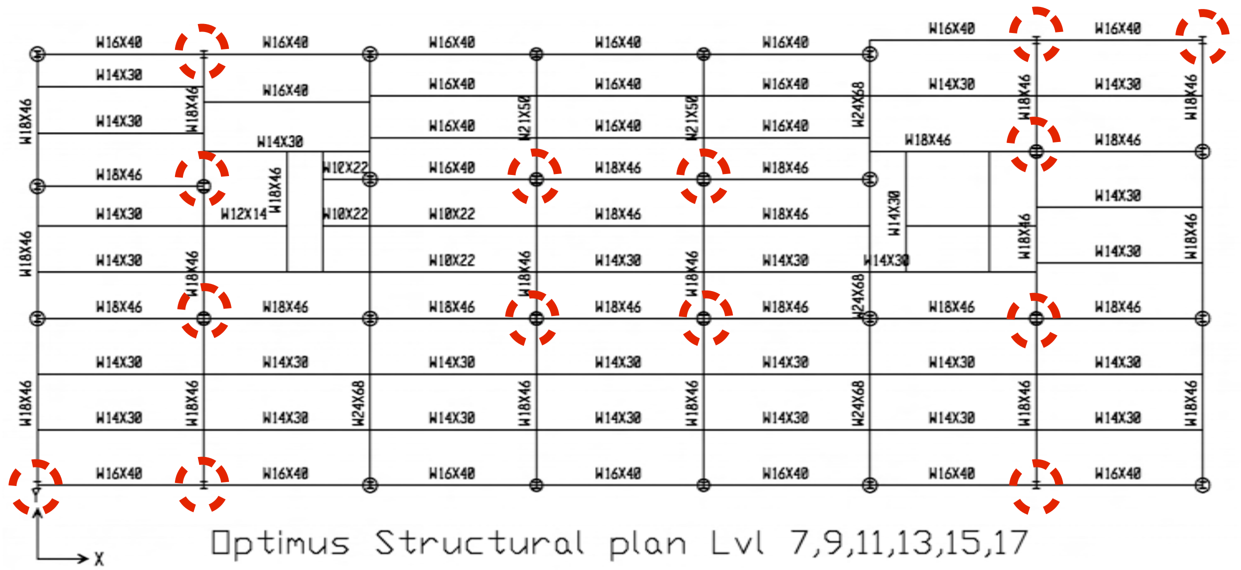
The following floor plans show the typical floor system layout for parking and two types of typical office floors. All columns encircled in the perimeter are non-composite wide flange columns mentioned in the table above. Similarly, the interior columns mentioned in the table above are encircled in the floor plans. All the other columns that are not encircled are part of the lateral system and also support gravity loads. However, due to added loads from winds and seismic behavior these columns carry higher loads as compared to gravity system.



Gravity Column Layout for parking level



Gravity Column Layout for typical office 1



Gravity Column Layout for typical office 2

Lateral System: Redesign and Analysis

The design of lateral system was performed in conjunction with the revaluation of gravity system after developing a finite element model using ETABS. In the design of lateral system, the use of finite element model was made to perform complex calculation like lateral drifts and member forces. Wind loads were calculated manually using ASCE 7-10 Directional procedure and story shears along with eccentricities were applied to ETABS model. After calculating wind loads, seismic analysis was performed using ASCE 7-10 Equivalent Lateral Force Procedure and Modal response spectrum analysis. Eventually, wind loads were the controlling load cases for lateral system design. Also, P-delta analysis was performed in ETABS which amplified the story drifts and was used for designing the lateral system.

Wind Loads Analysis

Wind loads were calculated using ASCE 7-10 Directional procedure. As the building is being redesigned in Mumbai (India), all the wind load parameters were obtained for the new location of the building. The design wind speed was found to be 98.4 miles/hour. The elevations of the building are not of the same dimension as the building gets narrower above 5th level. Therefore, the average lengths and breadths were calculated and used in the directional procedure. A detailed report on the wind load parameters can be found the wind load analysis appendix.

According to the directional procedure, wind pressures in North-South and East-West direction were calculated followed by story forces. These forces were used as wind loads cases and entered into ETABS using the load combinations specified in ASCE 7-10. The following tables are arranged in the procedure in which each calculation was performed. For further detailed calculation, please refer to the Wind load analysis appendix.

27.3.2 Velocity Pressure

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

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

Velocity pressure Calculation			
Story	Elevation (ft)	K_z	q_z (lb/ft^2)
Ground	0	1.0	21.7
1	20	1.1	22.8
2	33	1.2	24.9
3	46	1.3	26.4
4	59	1.3	27.5
5	72	1.4	28.5
6	85	1.4	29.3
7	98	1.4	30.1
8	111	1.5	30.7
9	124	1.5	31.3
10	137	1.5	31.9
11	150	1.5	32.4
12	163	1.6	32.9
13	176	1.6	33.3
14	189	1.6	33.7
15	202	1.6	34.1
16	215	1.6	34.5
17	228	1.7	34.8
Roof (level 18)	241	1.7	35.2

Using the wind load parameters, velocity pressures were calculated at each level for the new building. These values were further used to calculate wind pressures in East-West and North-South direction. The building was considered flexible and partially enclosed because the approximate natural frequency calculated was less than 1 Hz. Also, the facade will be permitted to have windows which proves that building was partially enclosed.

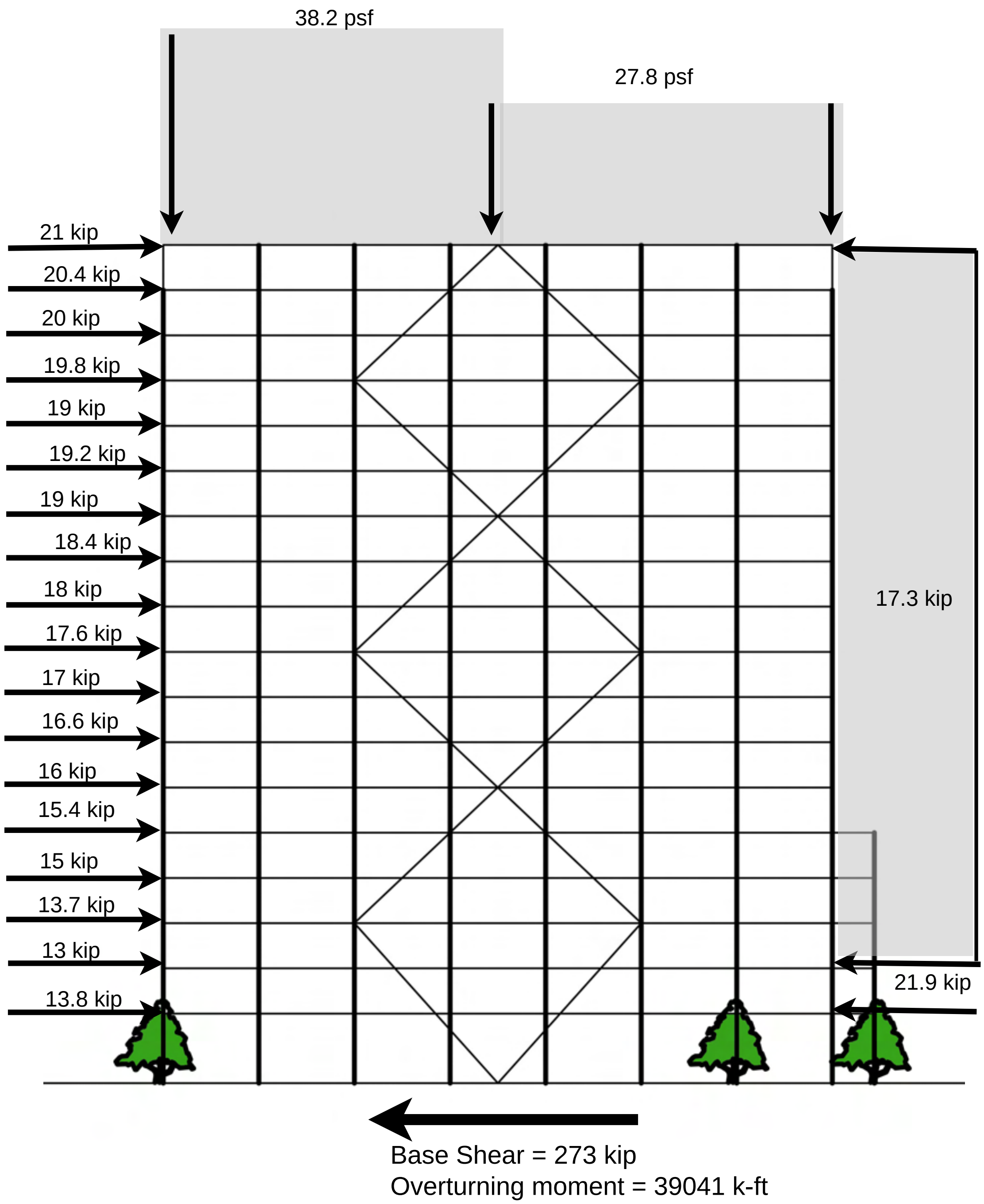
27.4.2 Enclosed and Partially Enclosed Flexible Buildings

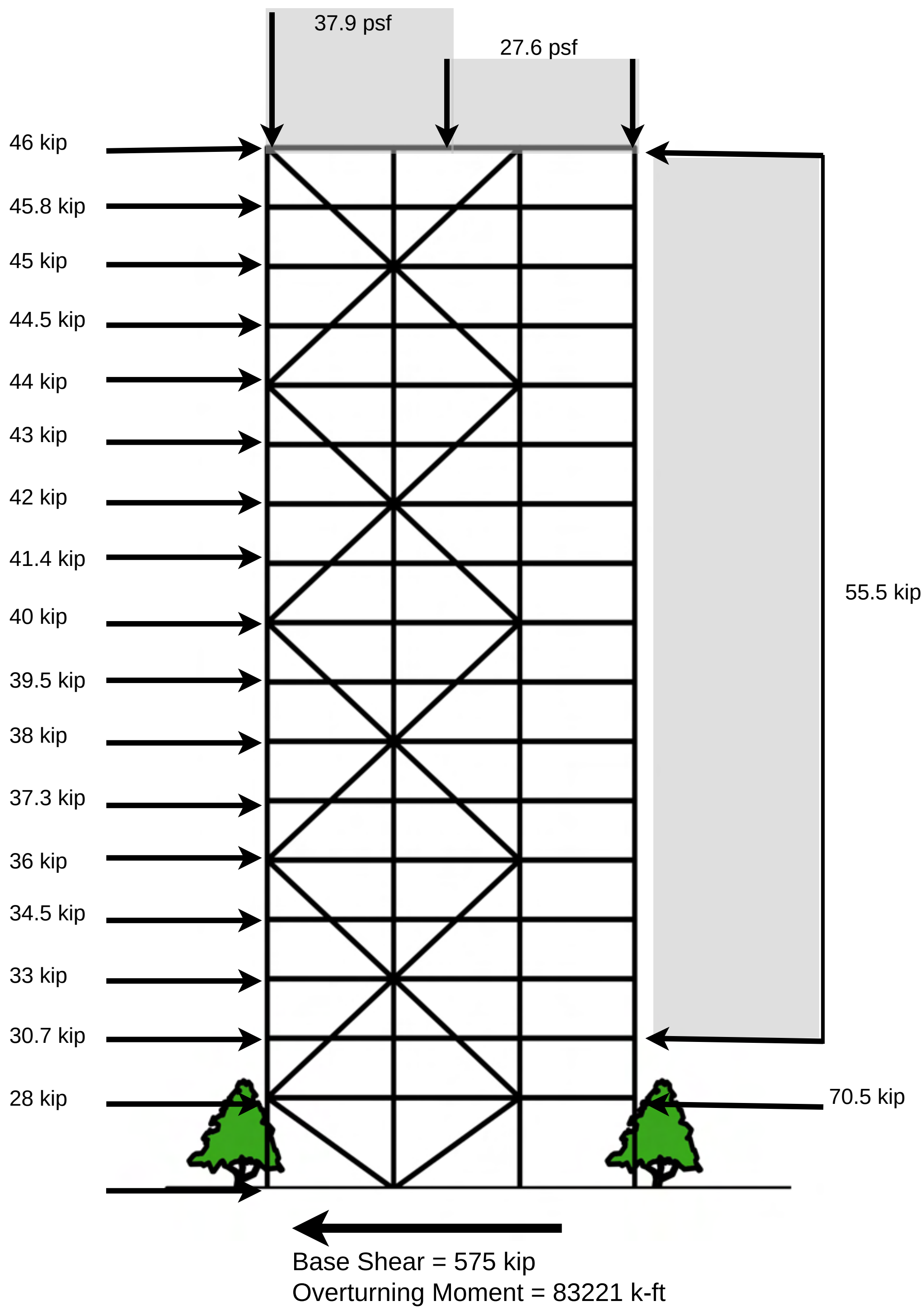
Design wind pressures for the MWFRS of flexible buildings shall be determined from the following equation:

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

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**		**Net pressure (+)**	**Net pressure (-)**
				+Gcpi*qi	**-Gcpi*qi**		
Ground	0	21.7	15.14	6.33	-6.33	8.80	21.47
1	20	22.8	15.92	6.33	-6.33	9.58	22.25
2	33	24.9	17.36	6.33	-6.33	11.03	23.70
3	46	26.4	18.40	6.33	-6.33	12.06	24.73
4	59	27.5	19.21	6.33	-6.33	12.88	25.54
5	72	28.5	19.89	6.33	-6.33	13.55	26.22
6	85	29.3	20.47	6.33	-6.33	14.14	26.80
7	98	30.1	20.98	6.33	-6.33	14.65	27.32
8	111	30.7	21.44	6.33	-6.33	15.11	27.77
9	124	31.3	21.86	6.33	-6.33	15.53	28.19
10	137	31.9	22.24	6.33	-6.33	15.91	28.57
11	150	32.4	22.59	6.33	-6.33	16.26	28.93
12	163	32.9	22.92	6.33	-6.33	16.59	29.26
13	176	33.3	23.23	6.33	-6.33	16.90	29.56
14	189	33.7	23.52	6.33	-6.33	17.19	29.85
15	202	34.1	23.80	6.33	-6.33	17.46	30.13
16	215	34.5	24.05	6.33	-6.33	17.72	30.39
17	228	34.8	24.30	6.33	-6.33	17.97	30.63
Roof (level 18)	241	35.2	24.54	6.33	-6.33	18.20	30.87
Leeward pressure Cp =-0.29							
Level	Elevation (ft)	q_z (lb/ft^2)	Wind pressure	internal pressure		Net pressure	Net pressure
				+Gcpi*qi	-Gcpi*qi		
All	241.00	35.2	**-8.9**	6.3	-6.3	**-15.2**	**-2.6**
Side wall pressure Cp =-0.7							
Level	**Elevation (ft)**	**q_z (lb/ft^2)**	**Wind pressure**	**internal pressure**		**Net pressure**	**Net pressure**
				+Gcpi*qi	**-Gcpi*qi**		
all	241.00	35.2	**-21.5**	6.3	-6.3	**-27.8**	**-15.1**
Roof pressures							
Level	**Elevation (ft)**	**q_z (lb/ft^2)**	**Wind pressure**	**internal pressure**		**Net pressure**	**Net pressure**
				+Gcpi*qi	**-Gcpi*qi**		
0 to h/2 (Cp=-1.04)	241.00	35.2	**-31.9**	6.3	-6.3	**-38.2**	**-25.6**
0 to h/2 (Cp=-0.18)	241.00	35.2	**-21.5**	6.3	-6.3	**-27.8**	**-15.1**

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	21.7	15.0	6.33	-6.33	8.65	21.32
1	20	22.8	15.8	6.33	-6.33	9.43	22.09
2	33	24.9	17.2	6.33	-6.33	10.86	23.53
3	46	26.4	18.2	6.33	-6.33	11.88	24.55
4	59	27.5	19.0	6.33	-6.33	12.69	25.35
5	72	28.5	19.7	6.33	-6.33	13.36	26.02
6	85	29.3	20.3	6.33	-6.33	13.94	26.60
7	98	30.1	20.8	6.33	-6.33	14.44	27.11
8	111	30.7	21.2	6.33	-6.33	14.90	27.56
9	124	31.3	21.6	6.33	-6.33	15.31	27.98
10	137	31.9	22.0	6.33	-6.33	15.69	28.36
11	150	32.4	22.4	6.33	-6.33	16.04	28.71
12	163	32.9	22.7	6.33	-6.33	16.37	29.03
13	176	33.3	23.0	6.33	-6.33	16.67	29.34
14	189	33.7	23.3	6.33	-6.33	16.96	29.62
15	202	34.1	23.6	6.33	-6.33	17.23	29.89
16	215	34.5	23.8	6.33	-6.33	17.49	30.15
17	228	34.8	24.1	6.33	-6.33	17.73	30.40
Roof (level 18)	241	35.2	24.3	6.33	-6.33	17.96	30.63
Leeward pressure Cp =-0.5							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure**	internal pressure		**Net pressure**	**Net pressure**
				+Gcpi*qi	-Gcpi*qi		
All	241.00	35.2	**-15.2**	6.3	-6.3	**-21.5**	**-8.9**
Side wall pressure Cp =-0.7							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure**	internal pressure		**Net pressure**	**Net pressure**
				+Gcpi*qi	-Gcpi*qi		
all	241.00	35.2	**-21.3**	6.3	-6.3	**-27.6**	**-14.9**
Roof pressures							
Level	levation (ft	q_z (lb/ft^2)	**Wind pressure**	internal pressure		**Net pressure**	**Net pressure**
				+Gcpi*qi	-Gcpi*qi		
0 to h/2 (Cp=-1.04)	241.00	35.2	**-31.6**	6.3	-6.3	**-37.9**	**-25.3**
0 to h/2 (Cp=-0.18)	241.00	35.2	**-21.3**	6.3	-6.3	**-27.6**	**-14.9**


38.2 psf
27.8 psf
21 kip
20.4 kip
20 kip
19.8 kip
19 kip
19.2 kip
19 kip
18.4 kip
18 kip
17.6 kip
17 kip
16.6 kip
16 kip
15.4 kip
15 kip
13.7 kip
13 kip
13.8 kip
17.3 kip
21.9 kip
Base Shear = 273 kip
Overturning moment = 39041 k-ft


37.9 psf
27.6 psf
46 kip
45.8 kip
45 kip
44.5 kip
44 kip
43 kip
42 kip
41.4 kip
40 kip
39.5 kip
38 kip
37.3 kip
36 kip
34.5 kip
33 kip
30.7 kip
28 kip
55.5 kip
70.5 kip
Base Shear = 575 kip
Overturning Moment = 83221 k-ft

Level	WX	LX	WY	LY	ex	ey	WMX	WMY
Ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	13.78	21.90	30.88	70.48	12.07	0.50	2.69	183.50
2	12.50	17.26	28.03	55.53	12.14	0.53	2.38	152.10
3	13.67	17.26	30.67	55.53	10.24	2.56	11.88	132.33
4	14.59	17.26	32.74	55.53	12.24	0.55	2.64	162.08
5	15.36	17.26	34.47	55.53	10.39	2.63	12.89	140.23
6	16.02	17.26	35.96	55.53	12.33	0.57	2.83	169.15
7	16.60	17.26	37.27	55.53	10.51	2.72	13.83	146.34
8	17.12	17.26	38.45	55.53	12.38	0.60	3.09	174.54
9	17.60	17.26	39.51	55.53	10.58	2.82	14.73	150.86
10	18.03	17.26	40.49	55.53	12.54	0.62	3.29	180.60
11	18.43	17.26	41.39	55.53	10.82	2.91	15.57	157.29
12	18.80	17.26	42.23	55.53	12.68	0.64	3.45	185.91
13	19.15	17.26	43.02	55.53	10.90	2.97	16.24	161.13
14	19.48	17.26	43.76	55.53	1.98	3.00	16.55	29.46
15	19.79	17.26	44.46	55.53	2.23	3.01	16.73	33.45
16	20.08	17.26	45.12	55.53	2.20	3.02	16.93	33.23
17	20.36	17.26	45.75	55.53	2.18	3.03	17.12	33.12
Roof (level 18)	20.63	17.26	46.36	55.53	2.07	3.07	17.47	31.65

All these loads were input in ETABS as windload cases and the most critical load case was determined. This critical case was further used in ASCE design load combinations for designing the lateral force resisting system

Load Case definitions

WX =	Windward force in X dir (kip)
WY =	Windward force in Y dir (kip)
LX =	Leeward force in X dir (Kip)
LY =	Leeward force in Y dir (kip)
WMX =	(WX + LX) x 0.15ey (kip)
WMY =	(WY + LY) x 0.15ex (kip)

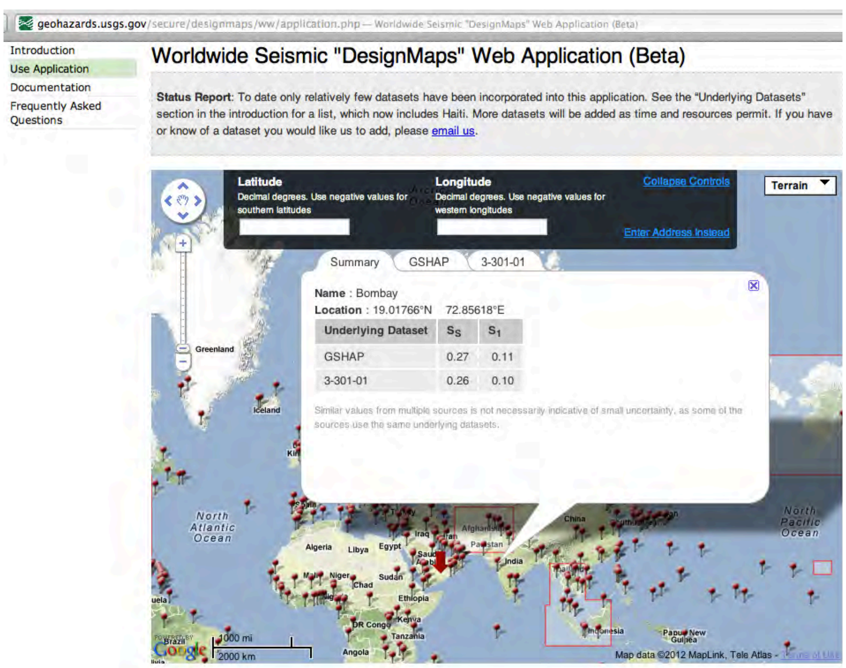
The story forces shown in the diagram above were converted to wind load case and defined in ETABS at each level. These cases were converted to a load combination as specified in Figure 27.4-8 in ASCE 7-10. The story shears from the wind load combinations were compared and critical load combination was further compared with the seismic loads. This comparison is shown further in the Lateral System Design section.

Windcase1A	WX + LX
Windcase1B	WY + LY
Windcase2A	0.75WX + 0.75LX + 0.75WINDMX
Windcase2B	0.75WX + 0.75LX - 0.75WINDMX
Windcase2C	0.75WY + 0.75LY + 0.75WINDMY
Windcase2D	0.75WY + 0.75LY - 0.75WINDMY
Windcase3	0.75WX + 0.75LX + 0.75WY + 0.75LY
Windcase4A	0.563WX + 0.563LX + 0.563WY +0.563LY + 0.563WINDMX + 0.563WMY
Windcase4B	0.563WX + 0.563LX + 0.563WY +0.563LY - 0.563WINDMX + 0.563WMY
Windcase4C	0.563WX + 0.563LX + 0.563WY +0.563LY + 0.563WINDMX - 0.563WMY
Windcase4D	0.563WX + 0.563LX + 0.563WY +0.563LY - 0.563WINDMX - 0.563WMY

These load combinations were found to be critical because center of mass and center of rigidity are near to each other causing lower eccentricity and hence, lower torsional shear.

Seismic Load Analysis

Seismic base shears were calculated using Equivalent lateral force procedure and Modal response spectrum analysis using specifications from ASCE 7-10. As ASCE 7-10 does not provide the short and long period design response spectrum values, United States Geological Survey website was used to find the values for city of Mumbai. This website provides Worldwide Seismic Design maps which are regularly updated. This value was used in both the analysis procedures.



Initially, the base shear and story forces were calculated using the equivalent lateral force procedure from ASCE 7-10. According to tables 11.6-1 and 11.6-2 in ASCE 7-10, the building falls in the Seismic Design Category A. Lateral forces for SDC A is calculated as Fx = 0.01 Wx as specified in section 1.4.3 in ASCE 7-10. Therefore according to this equation, the Seismic base shear value is 440.85 kips for an effective seismic weight of 44085 kips. However, it was assumed that the building structure is critical and it was important to look at the base shear and story drift values from Equivalent Lateral Force procedure and Response Spectrum Analysis according to the scope of the thesis. Also, it was important to include the effects of P-delta analysis. Continuing with the equivalent lateral force procedure, a response modification coefficient of 5 was chosen for loads in North-South direction and East-west direction. This is because of the lateral system in North-South direction consists of moment-resisting frames and it is specified in Section 12.2.3.1 of ASCE 7-10 to use lower response spectrum coefficient incase of dual systems. Also, the base shear values calculated further prove that fact that moment-resisting frames carry more than 25% of prescribed seismic forces. The use of R value of 5 means that special detailing of the structure would be required according to specifications of ASCE 7-10. However, due to time constraints this task was not undertaken in this assignment. The Equivalent Lateral Force Procedure was also followed in the FEM model in ETABS in parallel to the manual calculations. The base shear results of ETABS were very close to the the values of the manual calculation which confirmed the accuracy of the FEM model as well as the correctness of the manual calculations. This model was further used to calculate drifts due to the equivalent lateral force procedure.

Story Shears due to critical seismic loadcase in East-West direction			
Story	VX	T	MY
ROOF	-75	3306	-973
LEVEL 17	-133	6041	-2698
LEVEL 16	-174	7715	-4955
LEVEL 15	-220	9923	-7818
LEVEL 14	-253	11273	-11109
LEVEL 13	-290	13007	-14876
LEVEL 12	-315	14052	-18974
LEVEL 11	-344	15389	-23440
LEVEL 10	-363	16181	-28156
LEVEL 9	-383	17143	-33136
LEVEL 8	-396	17694	-38289
LEVEL 7	-410	18330	-43617
LEVEL 6	-418	18663	-49052
LEVEL 5	-430	19219	-54638
LEVEL 4	-436	19494	-60300
LEVEL 3	-439	19671	-66011
LEVEL 2	-441	19768	-71749
LEVEL 1	-442	19810	-80594

Story Shears due to critical seismic loadcase in North-south direction			
Story	VX	T	MY
ROOF	-75	3306	-973
LEVEL 17	-133	6041	-2698
LEVEL 16	-174	7715	-4955
LEVEL 15	-220	9923	-7818
LEVEL 14	-253	11273	-11109
LEVEL 13	-290	13007	-14876
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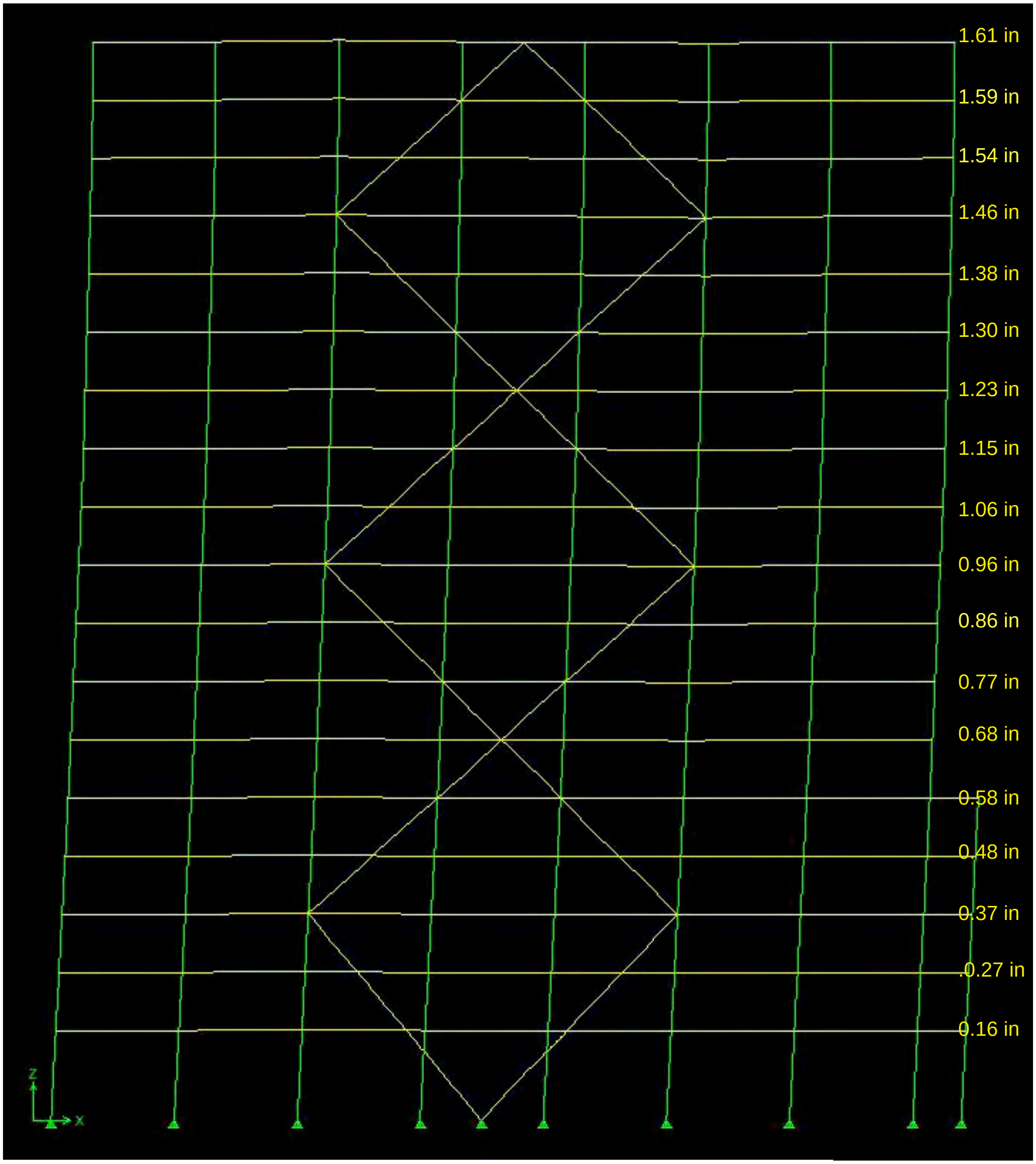
LEVEL 4	-436	19494	-60300
LEVEL 3	-439	19671	-66011
LEVEL 2	-441	19768	-71749
LEVEL 1	-442	19810	-80594

The next step of seismic analysis was to calculate base shears and drifts using Modal Response spectrum analysis. Base shear was calculated by using SRSS (Square root sum of squares method) for 10 fundamental periods of vibration. The resultant base shear was much lower than 85% of base shear due to Equivalent lateral force procedure and therefore the response spectrum load case was scaled to match base shear calculated using Equivalent Lateral Force Procedure. This scaled load case was further used to calculate story drifts by applying P-delta analysis.

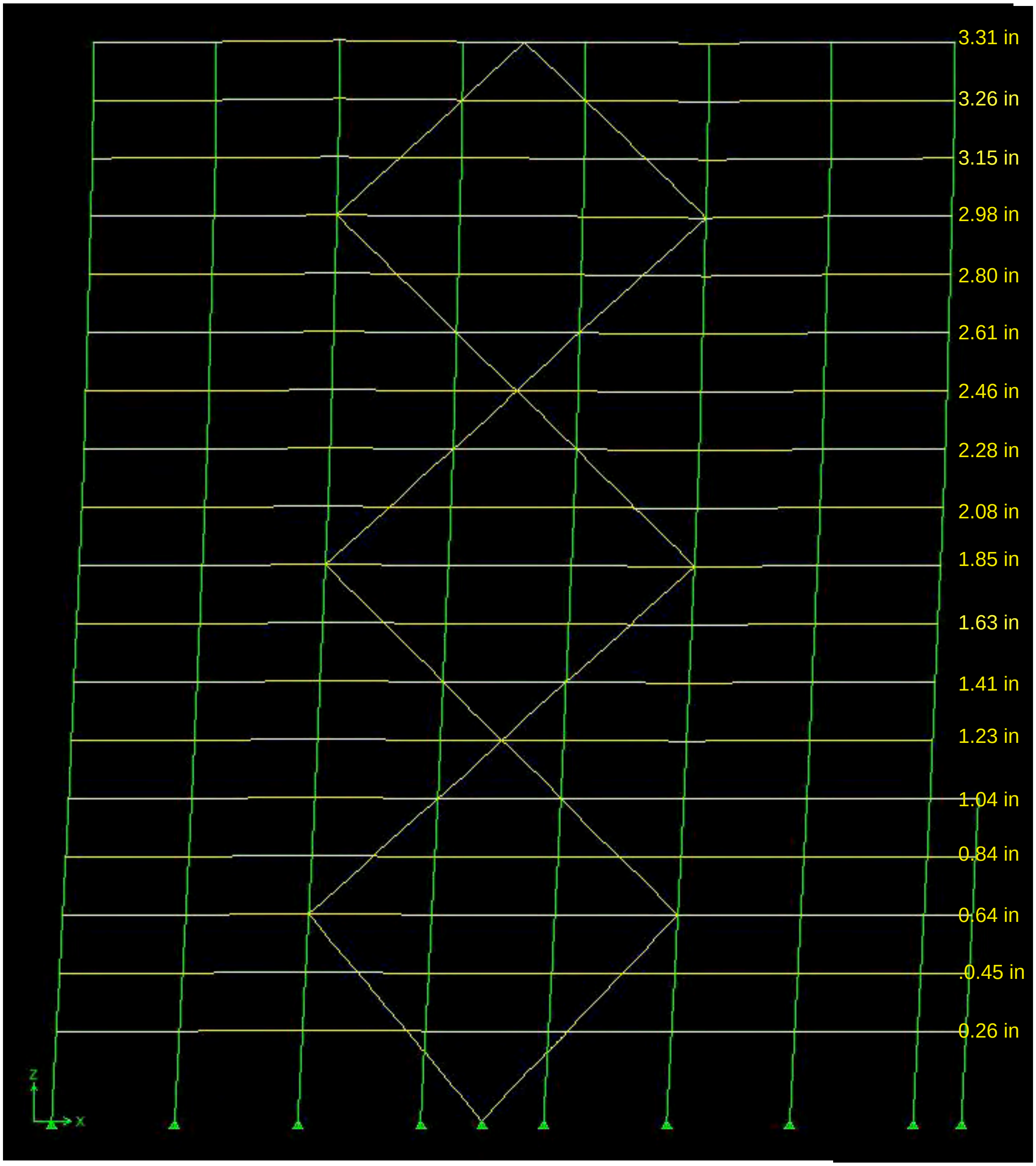
The table below shows story shears using response spectrum analysis and proves that the base shears due to response spectrum analysis s lower than 85% of base shear due to Equivalent lateral force procedure. This also results in lower story drifts. As the seismic loads will further be compared to wind loads, it was chosen to be conservative by comparing the base shear due to Equivalent lateral force procedure rather than Modal Response Spectrum Analysis. This was conservative because the latter had lower base shears.

Story Shears due to Seismic Response Spectrum Analysis					
Story	VX	VY	T	MX	MY
ROOF	43	65	92165	868	564
LEVEL 17	78	111	156850	2371	1604
LEVEL 16	100	135	189710	4204	2982
LEVEL 15	115	151	213162	6252	4578
LEVEL 14	126	159	224939	8368	6242
LEVEL 13	139	168	238478	10495	8004
LEVEL 12	148	174	247790	12658	9898
LEVEL 11	157	182	259467	14830	11944
LEVEL 10	164	189	271191	17040	14114
LEVEL 9	173	199	290021	19276	16384
LEVEL 8	179	208	306085	21592	18720
LEVEL 7	188	220	325526	23987	21115
LEVEL 6	195	229	338971	26471	23609
LEVEL 5	208	251	371740	29022	26253
LEVEL 4	218	270	400697	31756	29041
LEVEL 3	228	290	432489	34710	31956
LEVEL 2	238	308	462937	37877	34999
LEVEL 1	248	321	487028	43146	39931

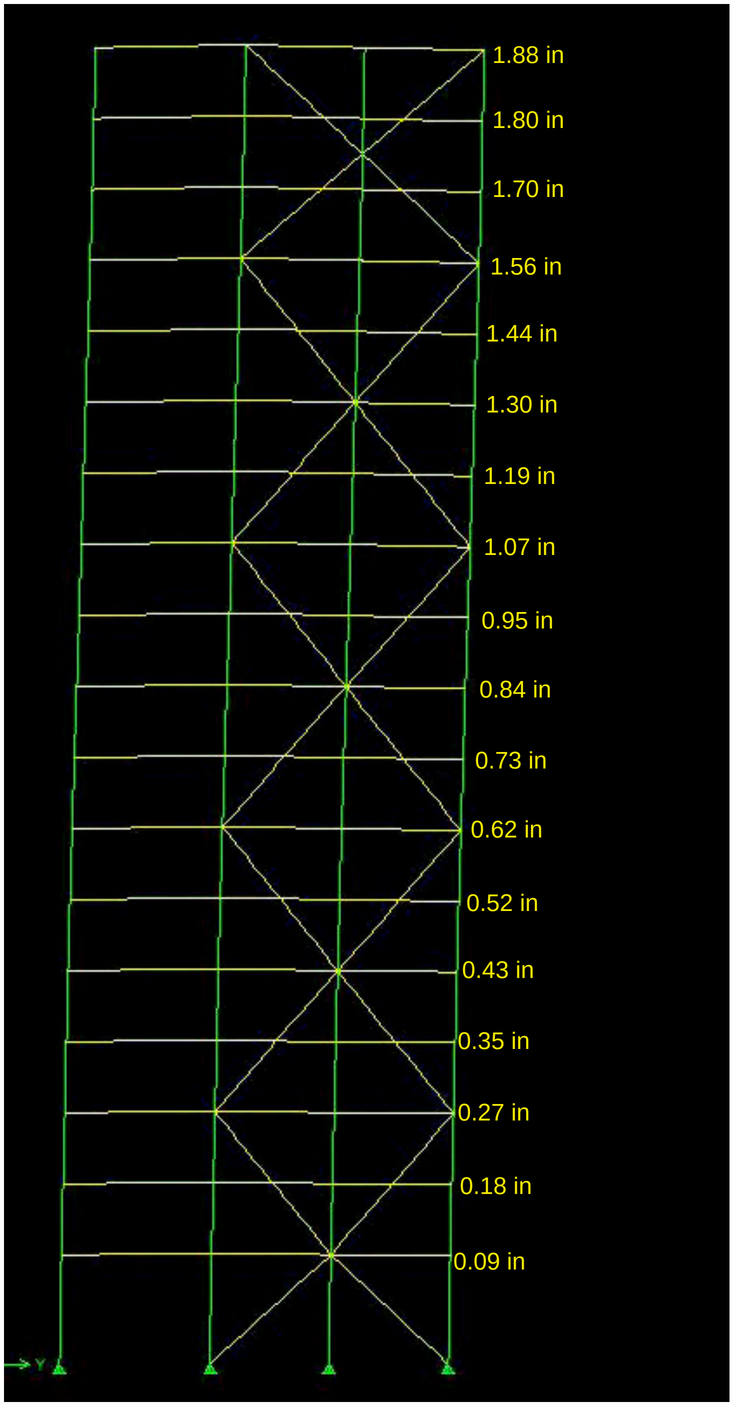
The following images show deflections in X and Y directions due to the two seismic load analysis. Critical load combinations were compared from the two analysis procedures. For Equivalent Lateral Force Procedure critical load combination was due to loads in X and Y direction with 5% accidental eccentricities. The load case with 5% accidental eccentricity was also controlling for Model response spectrum analysis and shown in the pictures below.



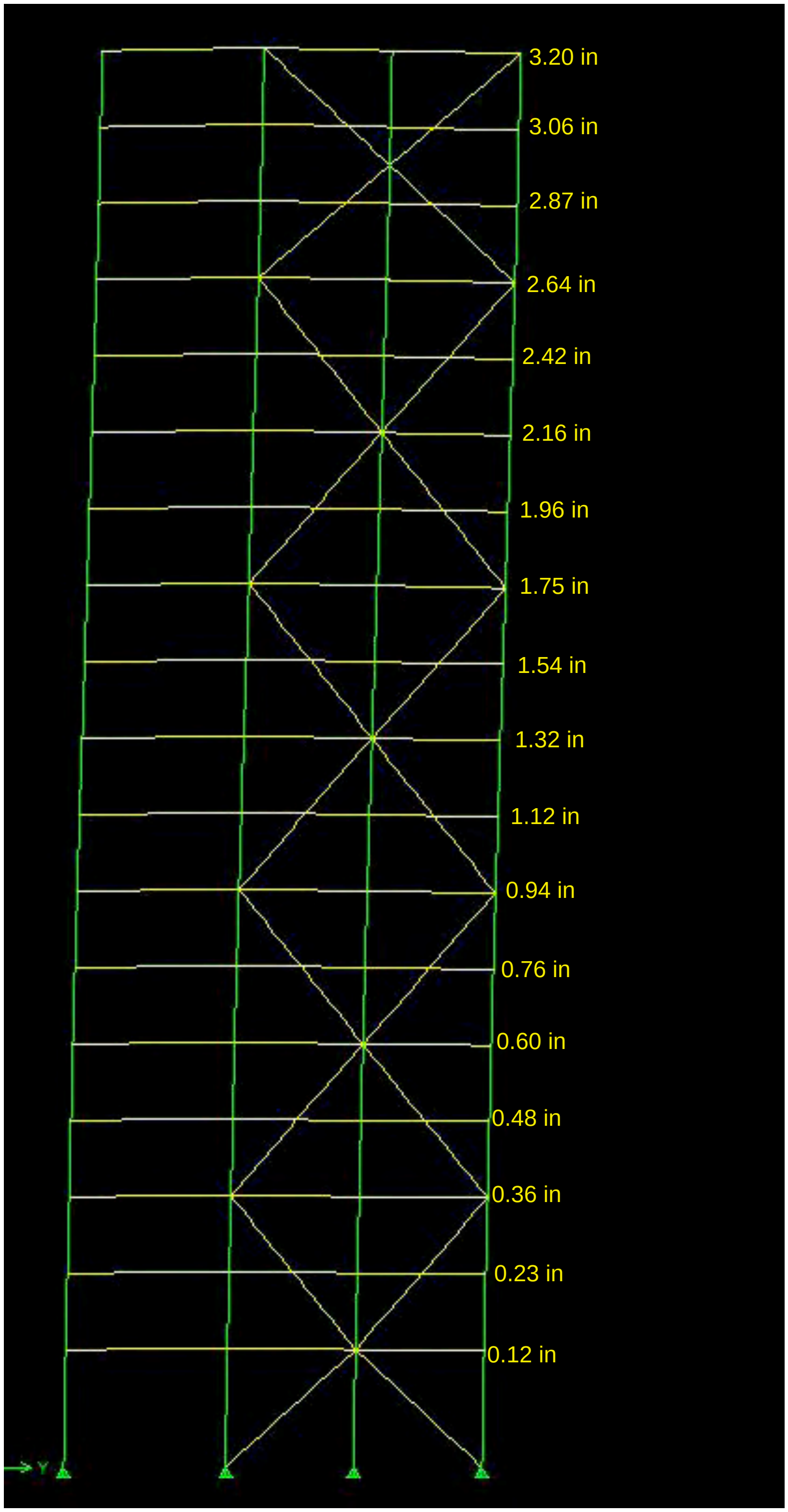
Deflections due to Response spectrum analysis in East-West direction



Deflections due to critical seismic load case in East-West direction using Equivalent Lateral force procedure



Deflections due to Response Spectrum analysis in North-South direction

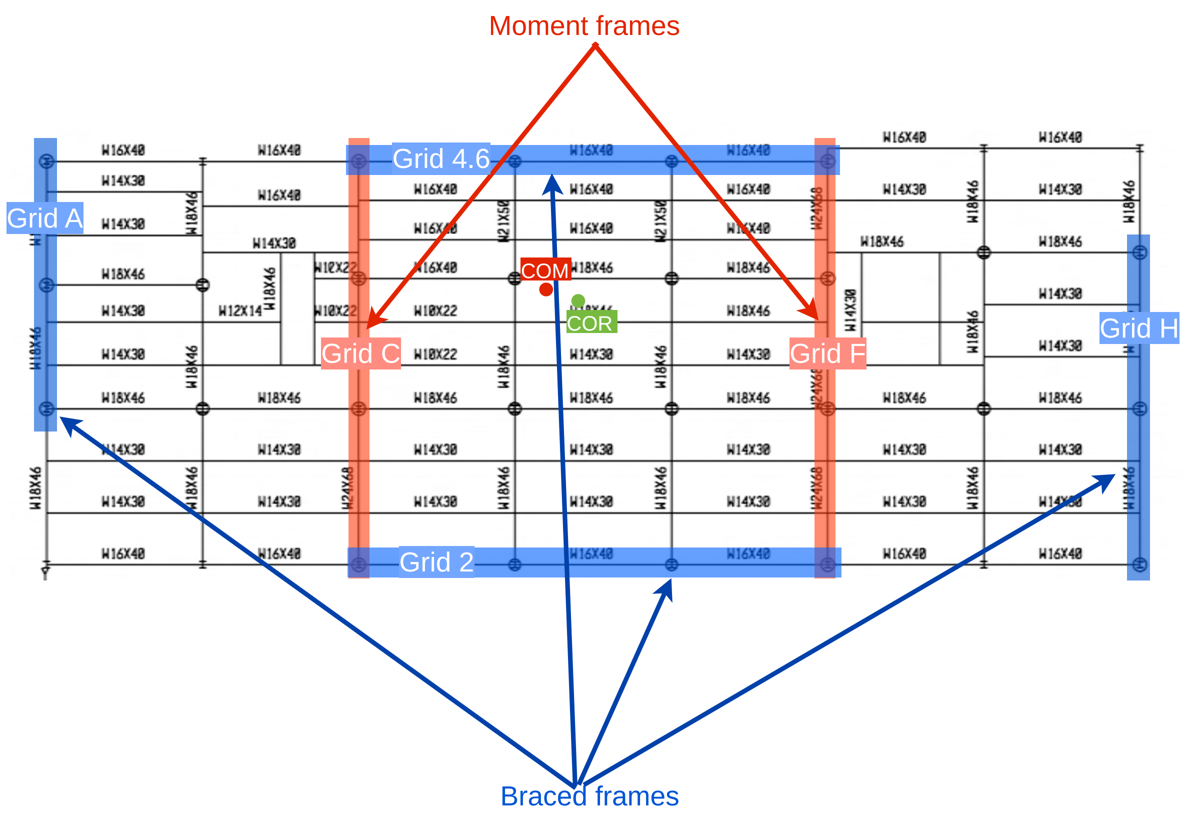


Deflections due to critical seismic load case in North-South direction using Equivalent Lateral force procedure

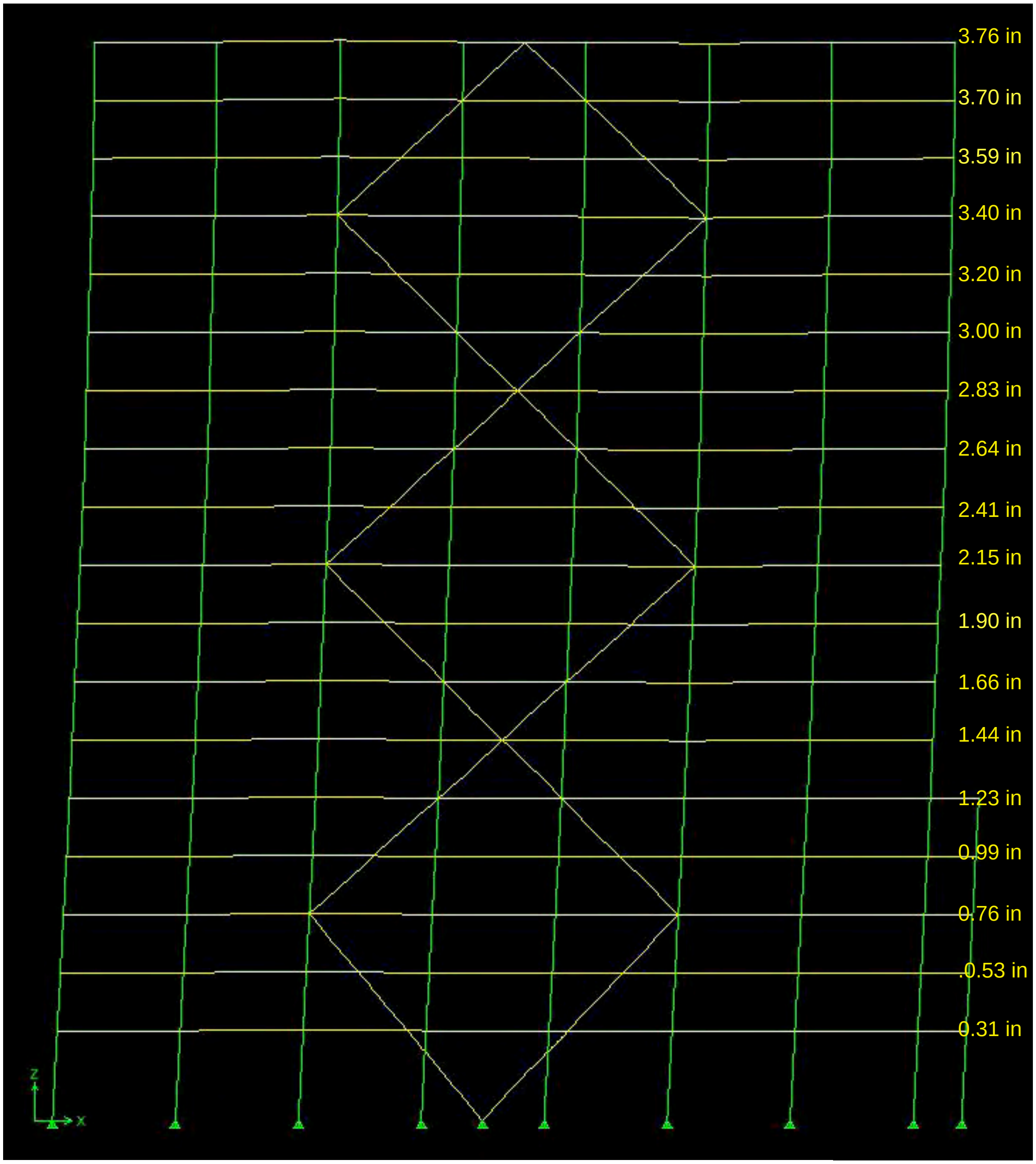
Equivalent lateral force procedure has higher story drifts after looking at the diagrams above. This proves that Equivalent lateral force procedure controls seismic loads. Having compared the two analysis procedures, the critical load combination of the critical analysis procedure was further compared to critical wind load combination which is shown in the next section.

Lateral System Design

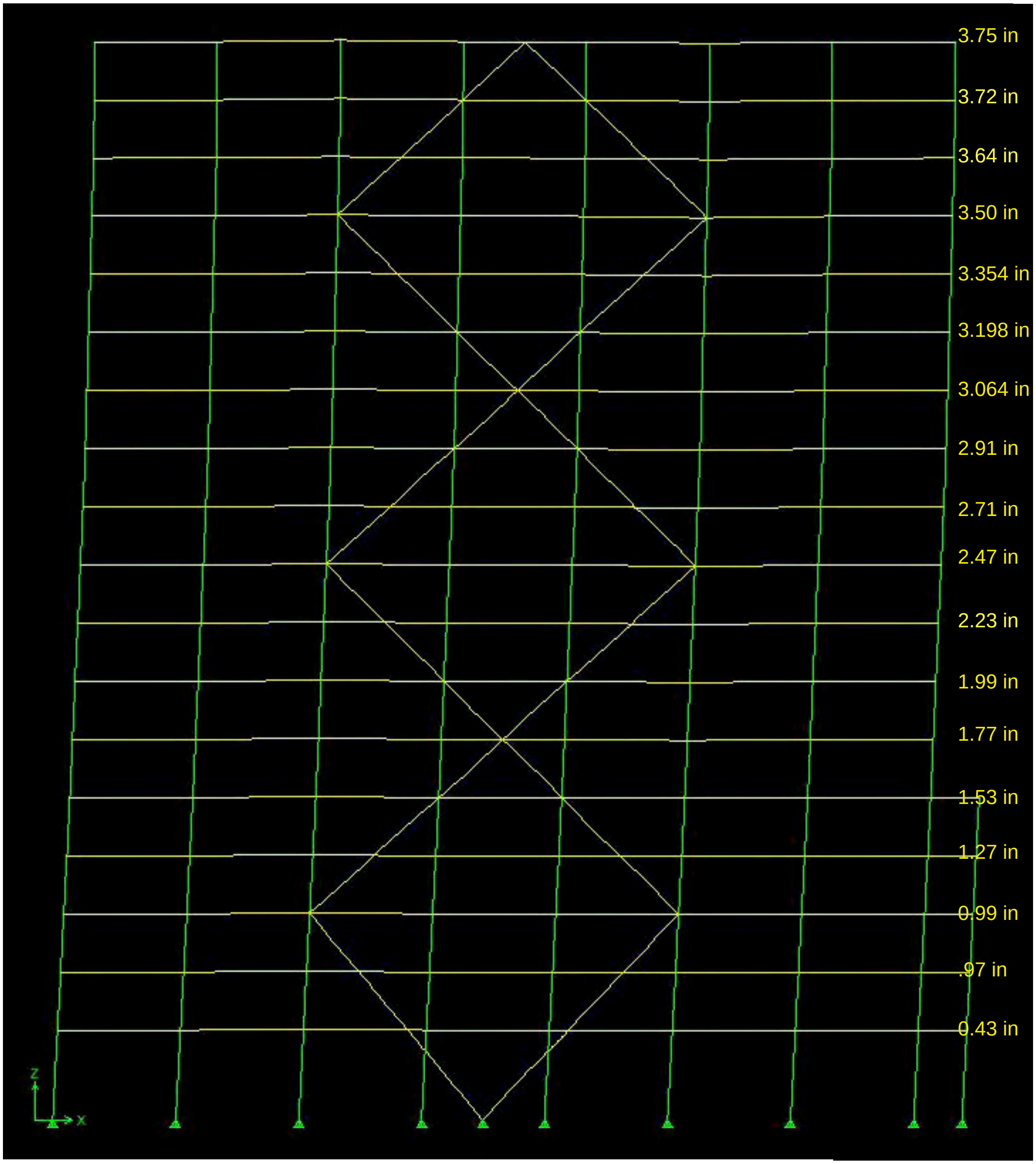
After lateral loads were analyzed, critical load combination from the critical analysis procedure was determined. As part of the design procedure, the critical load combinations from wind and seismic analysis were compared. This comparison was used to decide if the lateral system design was controlled by wind or seismic loads. P-delta effects were taken into consideration while comparing the two load cases - wind and seismic. The following load diagrams show story drifts from P-delta analyses of critical load combination of wind and seismic loads.



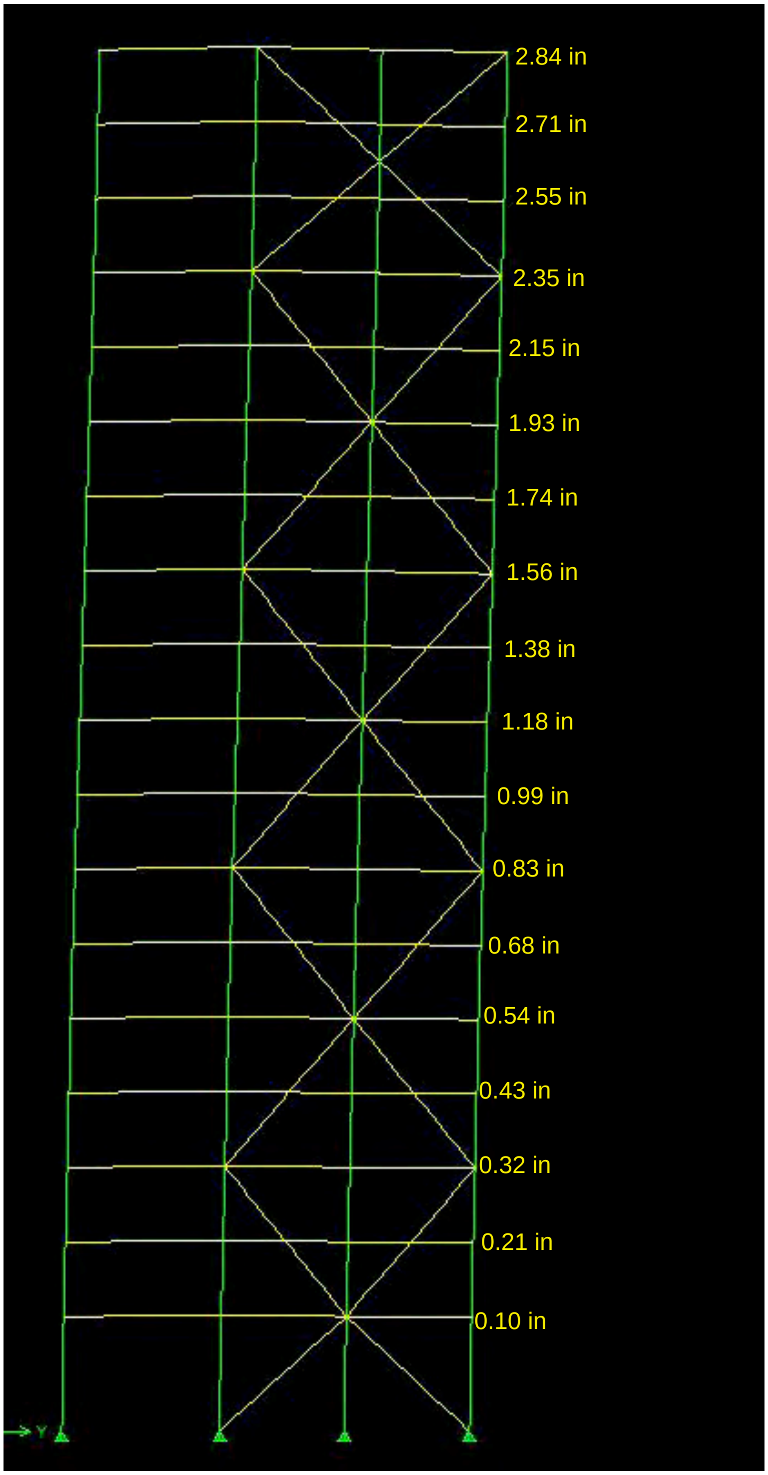
Lateral system: Moment and Braced frames layout



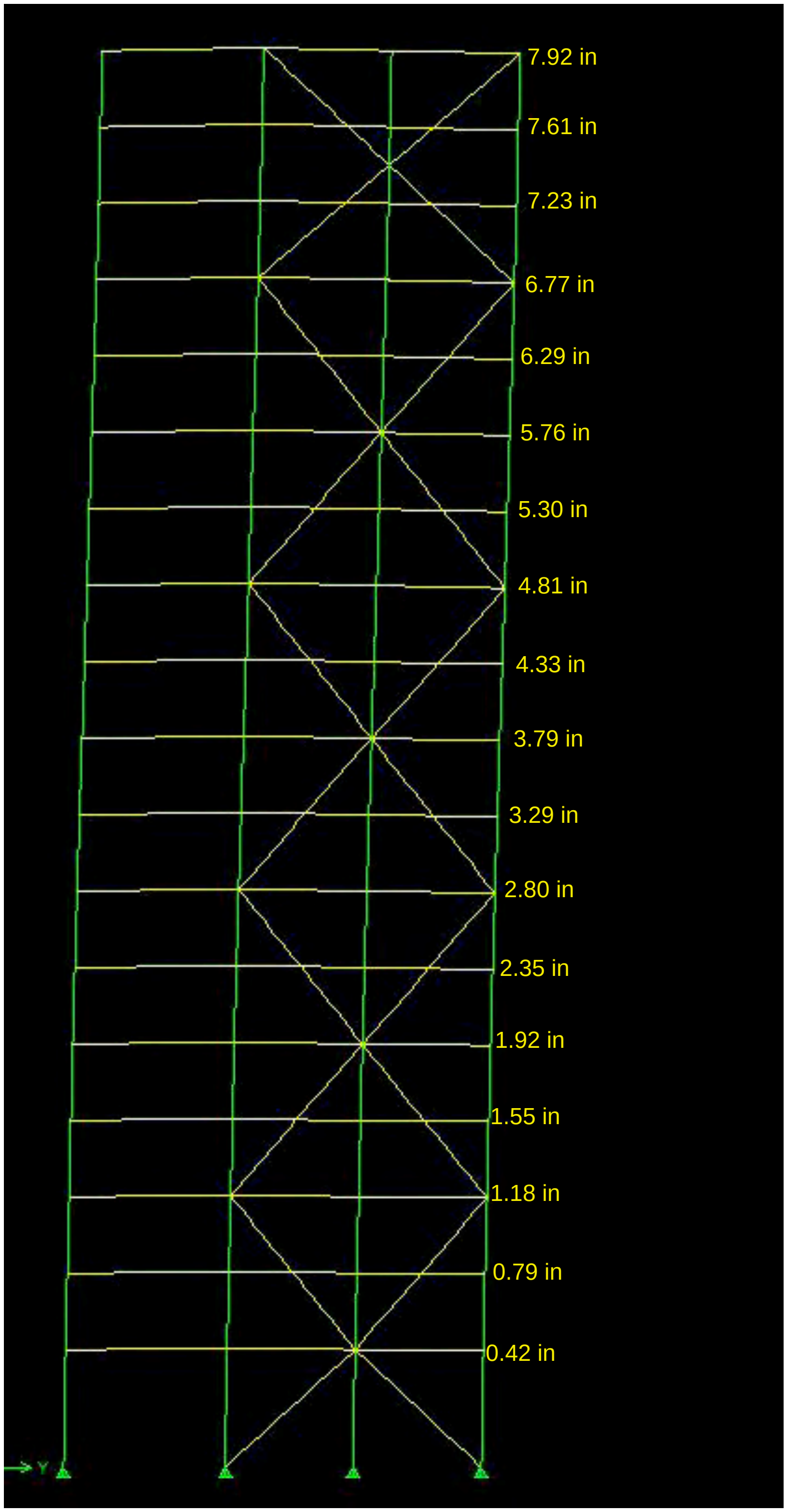
Deflections due to critical seismic load case in East-West direction
(Including P-delta effects)



Deflections due to critical Windcase in East-West direction
(Including P-delta effects)



Deflections due to critical seismic load case in North-South direction (including P-delta effects)



Deflections due to critical Windcase in North-South direction
(Including P-delta effects)

Story Shears due to critical wind loadcase in East-West direction			
Story	VX	T	MY
ROOF	-38	1851	-503
LEVEL 17	-76	3689	-1534
LEVEL 16	-113	5512	-3129
LEVEL 15	-150	7320	-5254
LEVEL 14	-187	9112	-7905
LEVEL 13	-223	10888	-11041
LEVEL 12	-259	12647	-14720
LEVEL 11	-295	14386	-19018
LEVEL 10	-330	16104	-23917
LEVEL 9	-365	17800	-29349
LEVEL 8	-399	19473	-35283
LEVEL 7	-433	21121	-41696
LEVEL 6	-466	22739	-48650
LEVEL 5	-499	24576	-56250
LEVEL 4	-531	26371	-64407
LEVEL 3	-562	28113	-73037
LEVEL 2	-592	29786	-82157
LEVEL 1	-627	31788	-97033

Story Shears due to critical wind loadcase in North-south direction			
Story	VY	T	MX
ROOF	-102	-9804	1413
LEVEL 17	-203	-19547	4284
LEVEL 16	-304	-29229	8578
LEVEL 15	-404	-38842	14340
LEVEL 14	-503	-48388	21527
LEVEL 13	-602	-57859	30171
LEVEL 12	-699	-67253	40238
LEVEL 11	-796	-76566	51673
LEVEL 10	-892	-85793	64433
LEVEL 9	-987	-94923	78549
LEVEL 8	-1081	-103952	94001
LEVEL 7	-1174	-112865	110783
LEVEL 6	-1266	-121653	128760
LEVEL 5	-1356	-130982	147941
LEVEL 4	-1444	-140129	168382
LEVEL 3	-1530	-149082	190057
LEVEL 2	-1617	-158014	212754
LEVEL 1	-1715	-168219	249503

Story Shears due to critical seismic loadcase in East-West direction			
Story	VX	T	MY
ROOF	-75	3306	-973
LEVEL 17	-133	6041	-2698
LEVEL 16	-174	7715	-4955
LEVEL 15	-220	9923	-7818
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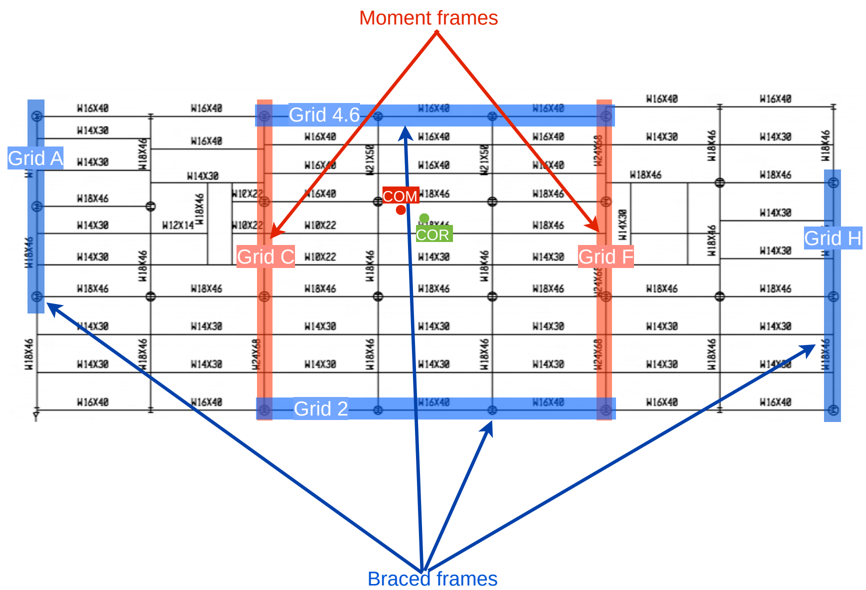
LEVEL 3	-439	19671	-66011
LEVEL 2	-441	19768	-71749
LEVEL 1	-442	19810	-80594

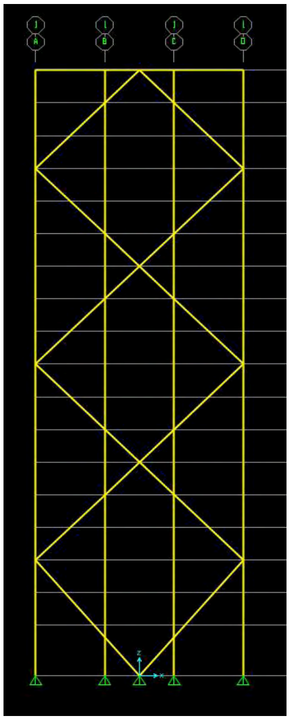
Comparing the drift vales it was clear that wind was controlling over seismic loads and the lateral system was designed using Wind loads. Wind load 1A and 1B were the controlling loads and they were used to calculate design forces in the members. The layout of the columns was made to achieve least eccentricity by distributing the lateral frames in a way to reduce distance between the Center of Mass and Center of Rigidity.



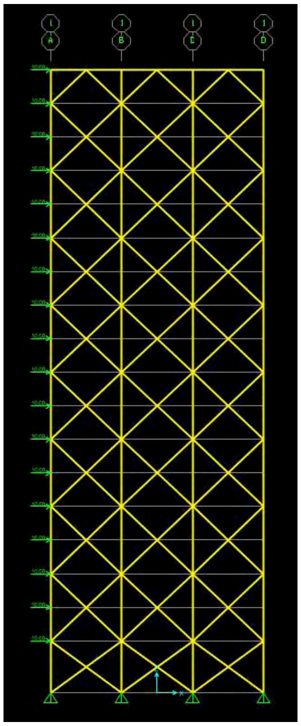
Lateral system: Moment and Braced frames layout

The layout of the lateral system frames helped reduce torsional effects which would be higher of the frames were located in the interior like that of the existing system. The braced frames consist of steel HSS braces and steel wide flange columns encased with reinforced -concrete. The columns not only support compression due to lateral overturning moments, but also gravity loads. Therefore, the compression forces on these columns require the use of very heavy steel columns. To avoid this, it was decided to encase reinforced concrete around the steel wide flange columns of the lateral system. There were four types of braces analyzed to select the most efficient brace. The criteria for the most efficient brace was to achieve maximum stiffness per unit length of the brace. This 4 types of braces were analyzed in SAP for deflection and stiffness.

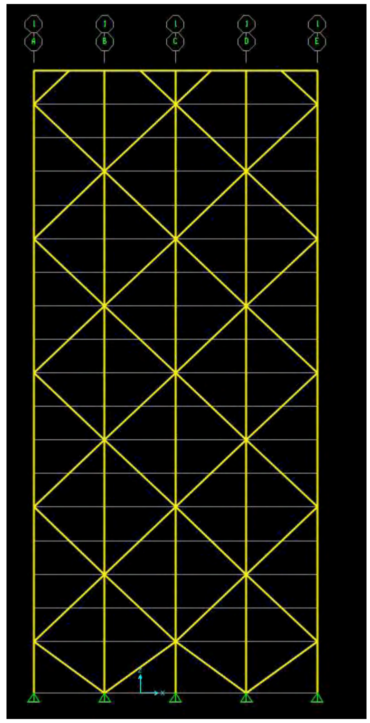
Efficiency in bracing					
	Force (kip)	displacement (in)	stiffness	steel brace length	stiffness per unit length
R1	180	9.3	19	691.2	0.028
R2	180	6.5	28	2020.6	0.014
R3	180	4.1	44	1422.7	0.031
R4	180	28.6	6	1292	0.005



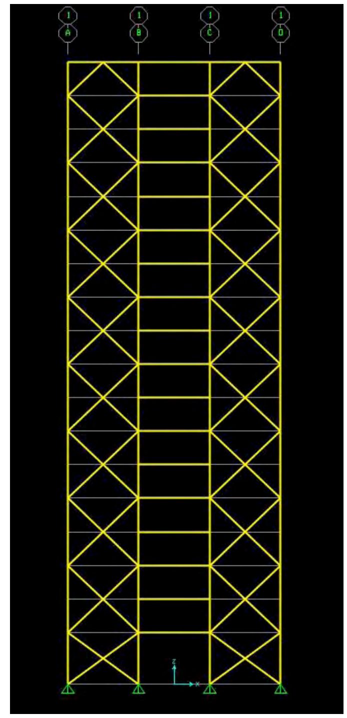
SAP Frame R1



SAP Frame R2

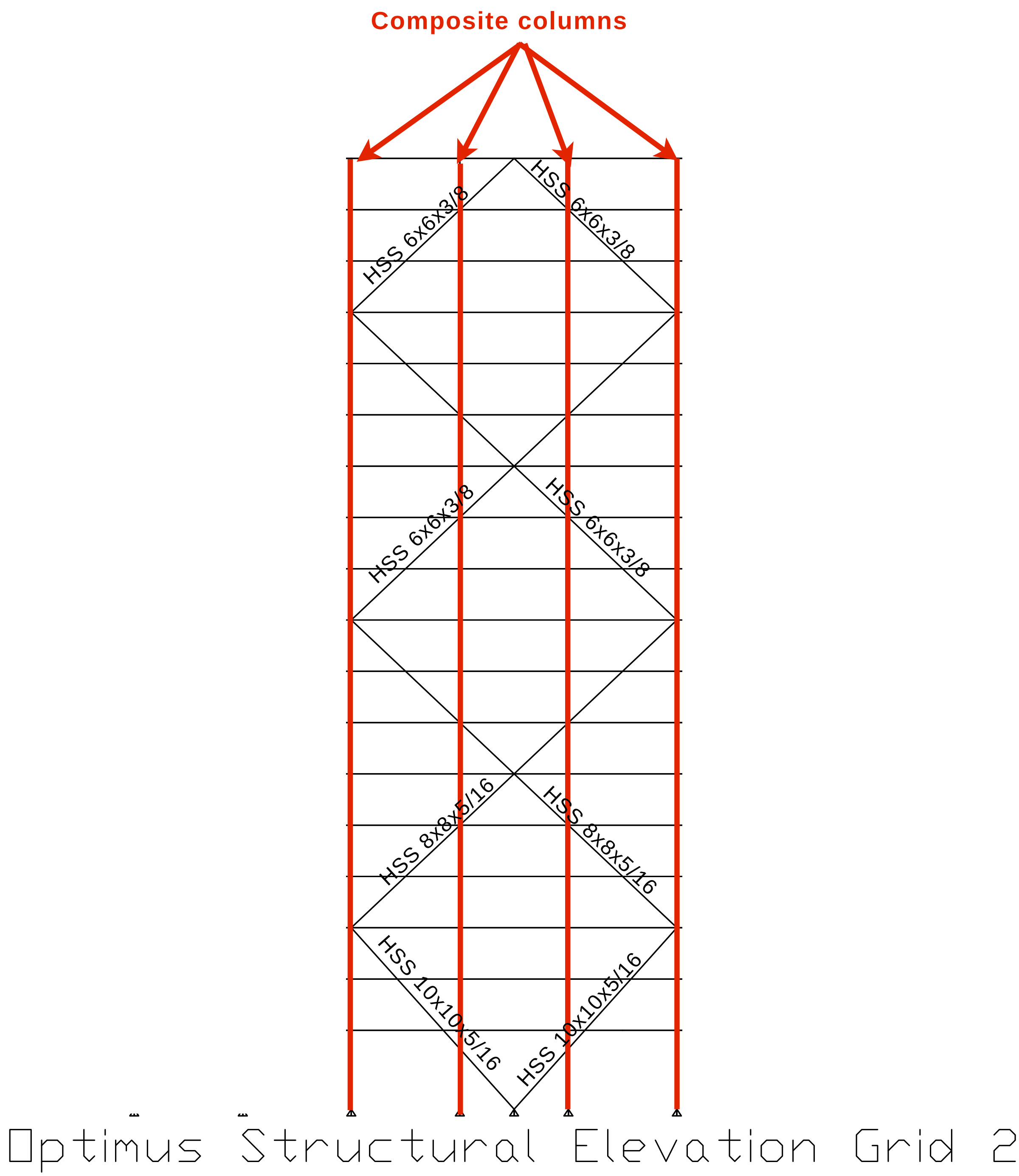


SAP Frame R3



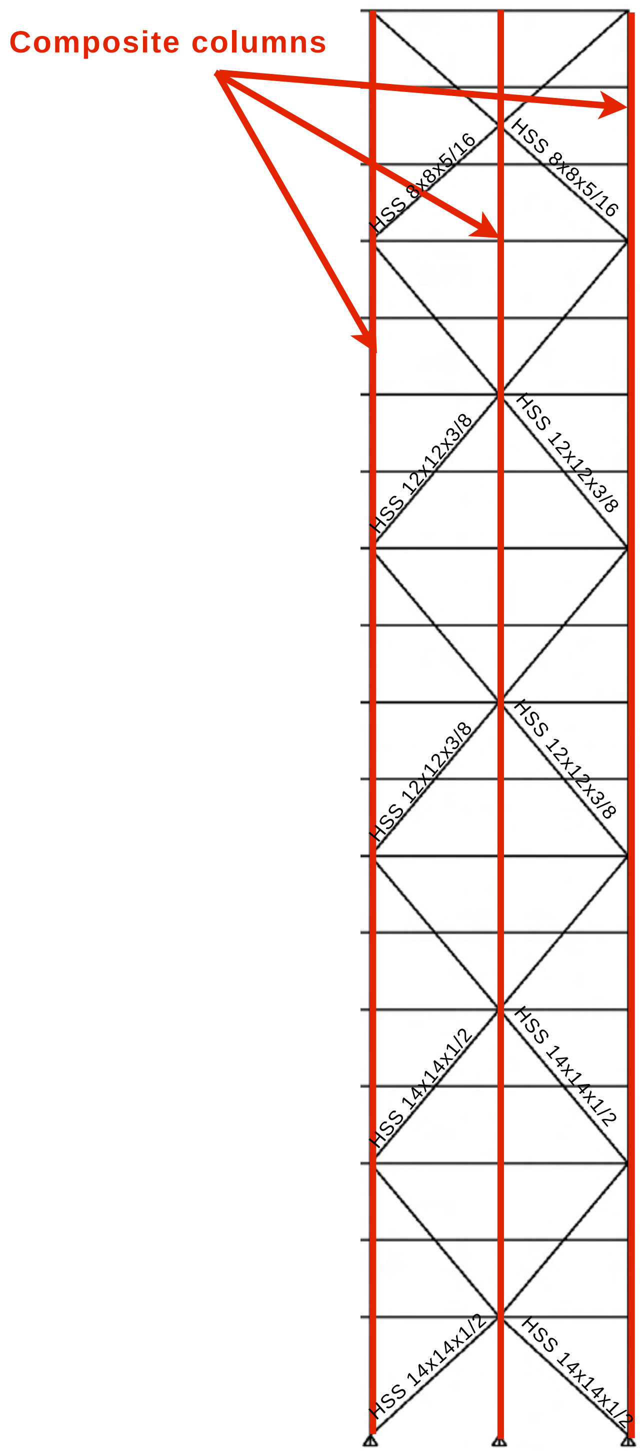
SAP Frame R4

Lateral system design: Grid 2 and 4.6 Column design summary				
Story	P	Member	ΦPn	DCR
LEVEL 1	-2752.4	W12x120 dia32 8#10	2800	0.98
LEVEL 2	-2638.14	W12x120 dia30 8#8	2800	0.94
LEVEL 5	-2098.38	W12x120 dia30 8#8	2800	0.75
LEVEL 7	-1789.52	W12x87 dia24 8#8	1900	0.94
LEVEL 9	-1526.7	W12x87 dia24 8#8	1900	0.80
LEVEL 11	-1132.68	W12x72 dia24 8#8	1800	0.63
LEVEL 13	-858.84	W12x72 dia24 8#8	1800	0.48
LEVEL 15	-589.8	W12x58 dia22 8#8	1200	0.49
LEVEL 17	-290.11	W12x58 dia22 8#8	1200	0.24



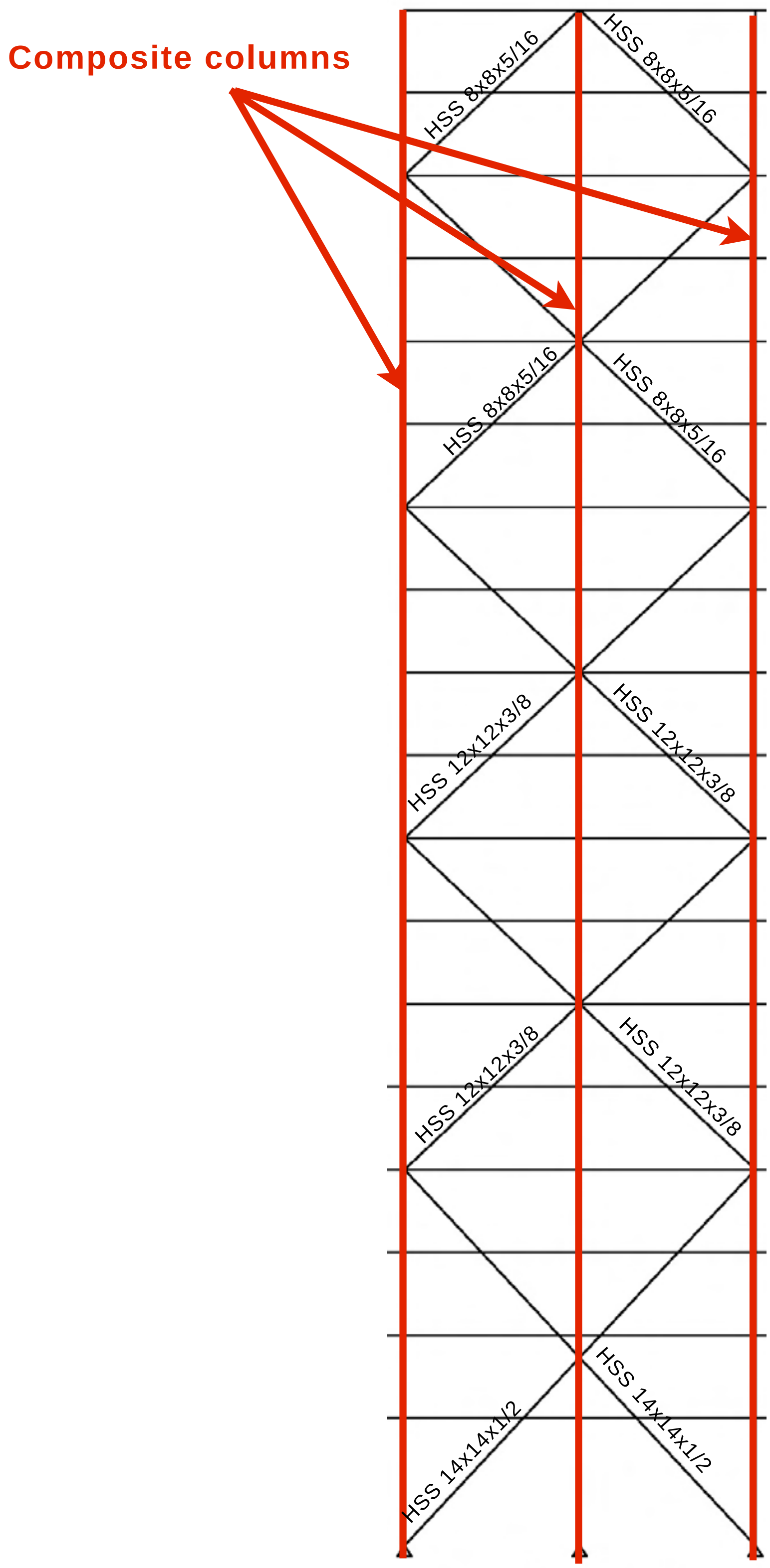
Lateral System: Braced frame Grid 2 and 4.6

Lateral system design : Grid A column design summary				
Story	P	Member	ΦPn	DCR
LEVEL 1	-3190.81	W12x120 dia34 8#10	3500	0.91
LEVEL 2	-3134.15	W12x120 dia34 8#10	3500	0.90
LEVEL 4	-2175.3	W12x120 dia34 8#10	3500	0.62
LEVEL 6	-2060.86	W12x120 dia28 8#8	2600	0.79
LEVEL 8	-1537.09	W12x120 dia28 8#8	2600	0.59
LEVEL 10	-1315.42	W12x58 dia22 8#8	1500	0.88
LEVEL 12	-906.59	W12x58 dia22 8#8	1500	0.60
LEVEL 14	-684.15	W12x58 dia22 8#8	1500	0.46
LEVEL 16	-372.39	W12x45 dia20 8#8	1000	0.37
LEVEL 17	-275.6	W12x45 dia20 8#8	1000	0.28
ROOF	-196.63	W12x45 dia20 8#8	1000	0.20



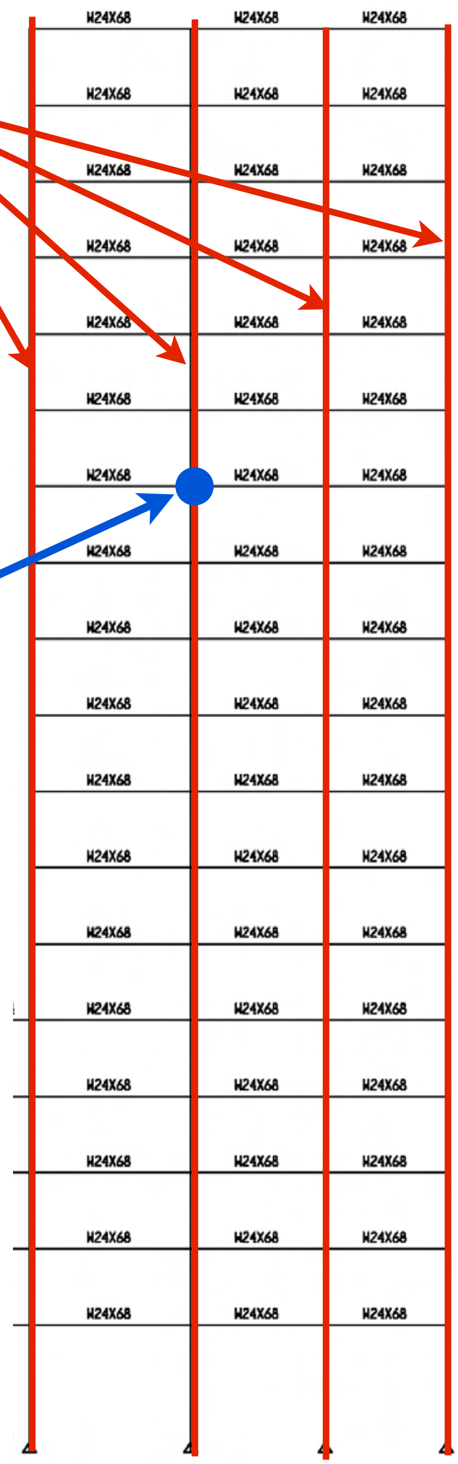
Lateral System: Braced frame Grid A

Lateral system design: Grid H column design summary				
Story	P	Member	ΦPn	DCR
LEVEL 1	-3445.48	W12x120 dia34 8#10	3500	0.98
LEVEL 2	-3361.65	W12x120 dia34 8#10	3500	0.96
LEVEL 4	-3264.42	W12x120 dia34 8#10	3500	0.93
LEVEL 6	-2315.79	W12x120 dia28 8#8	2600	0.89
LEVEL 8	-2125.79	W12x120 dia28 8#8	2600	0.82
LEVEL 10	-1409.82	W12x58 dia22 8#8	1500	0.94
LEVEL 11	-1294.75	W12x58 dia22 8#8	1500	0.86
LEVEL 12	-1213.71	W12x58 dia22 8#8	1500	0.81
LEVEL 14	-675.77	W12x45 dia20 8#8	1000	0.68
LEVEL 16	-479.41	W12x45 dia20 8#8	1000	0.48
ROOF	-201.42	W12x45 dia20 8#8	1000	0.20



Lateral System: Braced frame Grid H

Lateral system design: Moment frame Grid C and F column design				
Story	P	Member	ΦPn	DCR
LEVEL 1	-3362.13	W12x120 dia34 8#10	3500	0.96
LEVEL 2	-3219.96	W12x120 dia34 8#10	3500	0.92
LEVEL 4	-2940.66	W12x120 dia34 8#10	3500	0.84
LEVEL 6	-2569.28	W12x120 dia28 8#8	2600	0.99
LEVEL 8	-2163.94	W12x120 dia28 8#8	2600	0.83
Level 9		W12x120 dia28 8#8	2600	0.83
LEVEL 10	-1761.57	W12x120 dia28 8#8	2600	0.68
LEVEL 12	-1362.4	W12x58 dia22 8#8	1500	0.91
LEVEL 14	-975.92	W12x58 dia22 8#8	1500	0.65
LEVEL 16	-590.67	W12x44 dia20 8#8	1000	0.59
ROOF	-291.99	W12x44 dia20 8#8	1000	0.29

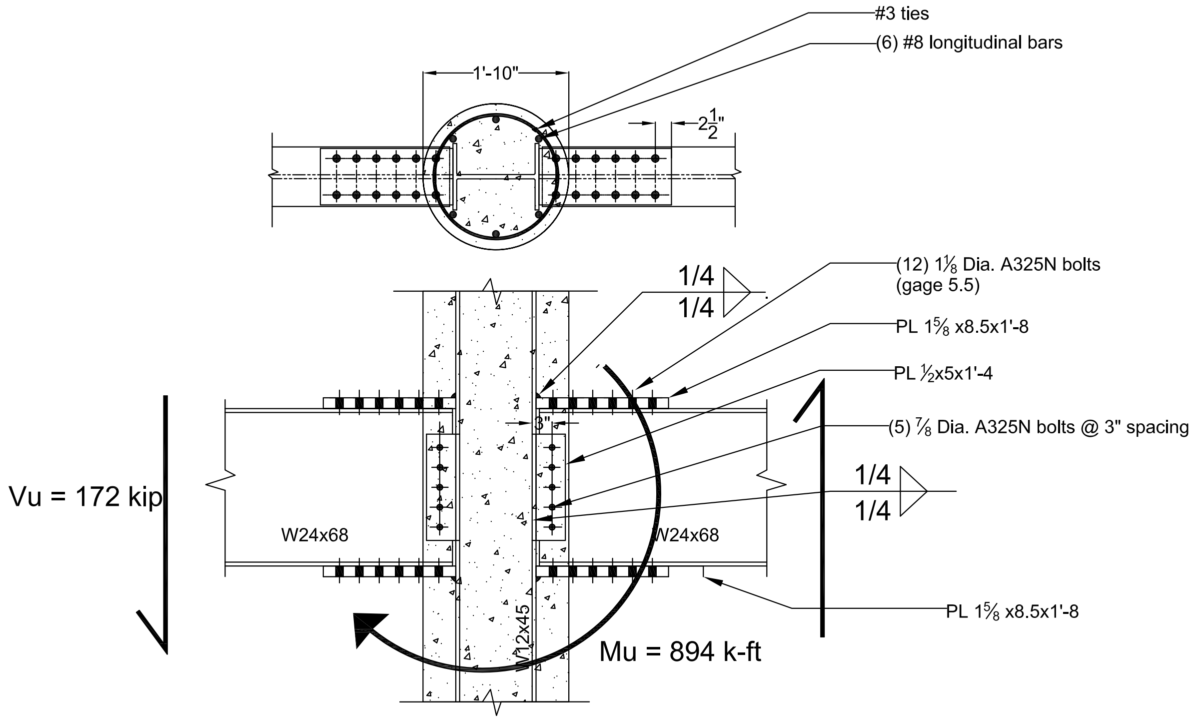


All W24x68 girders to columns are flange plate bolted-web bolted moment-resisting connection. A typical connection of this type is designed and presented in the following section.

In order to model this type of frame in ETABS, panel zone were defined where the stiffness of columns was assigned to the connection and rigid zone factor of 0.5 was assigned. For the frames that were not moment resisting, moment releases were assigned and beams were design as simply supported. This strategy was obtained to achieve conservative moment in the beams and connections for design.

Lateral System: Moment frame Grid C and F

Moment frame connection design



Flange plate bolted and web bolted moment connection

In order to design a typical moment connection, maximum design loads were obtained from ETABS panel zone deformation output. The design includes a W24x68 girder connected to a W21x45 columns encased in reinforced concrete to form a 20 inch circular column. The girder is connected to column flanged using 1 5/8 inch thick ASTM A36 steel plates bolted to beam flange and welded to column flange. The beam web is connected to column flange using ASTM A36 1/2in plate bolted to beam and welded to columns flange. 1/4 in is used a standard weld size for all welds and 1 1/8 in bolts for flange bolting and 7/8 in bolts are used for web bolting. In the design, it was assumed that additional stiffeners and doubler plates were not required because the reinforced concrete encasing added significant amount of stiffness to the column to avoid flange and web local bending and crippling.

Foundation Revaluation

A significant change in loading condition does not occur due to transition of a superstructure from concrete to steel. The dead static load of the structure was calculated to reevaluate the foundations. In the concrete system this load includes dead weight of floors, beams, columns shear walls and in the redesigned steel system it includes composite floor system, composite and non composite columns, braces and steel beams. The FEM model for concrete and steel was used to calculate the weight of the superstructure. The weight of concrete structure is 25000 kip while that of steel is reduced to 23000 kip. Therefore, a significant load reduction is not observed because of the used of composite system. Due to time constraints to research on design procedures for MAT foundation revaluation, the foundation redesign could not be undertaken. However, a significant reduction in thickness of the foundation would not take place. Also, additional components like - foundation base plate, column to foundation connection would be required to redesign the foundation for a steel structural system.

Cost-Benefit analysis

In today's scenario, the cost of a steel high-rise building is higher as compared to concrete. According to the cost information of concrete and steel in India presented in the Turner and Townsend report, the cost of the redesign of The Optimus in steel is 18% higher as compared to concrete. Other limitations faced in the redesign was avoid the used to heavy steel sections due to lack of availability, lack of experience in steel design among contractors and engineers. So, a client has to go though these hassles plus the additional financial cost of the building to building a steel structure.

	Steel		Concrete	
Cost per sq ft	INR 906		INR 768	
Level	Area (sq ft)	Cost of floor	Area	Cost of floor
Ground	21000	INR 19,026,000	21000	INR 16,128,000
1	21000	INR 19,026,000	21000	INR 16,128,000
2	21000	INR 19,026,000	21000	INR 16,128,000
3	21000	INR 19,026,000	21000	INR 16,128,000
4	21000	INR 19,026,000	21000	INR 16,128,000
5	21000	INR 19,026,000	21000	INR 16,128,000
6	12390	INR 11,225,340	12390	INR 9,515,520
7	15000	INR 13,590,000	15000	INR 11,520,000
8	12390	INR 11,225,340	12390	INR 9,515,520
9	15000	INR 13,590,000	15000	INR 11,520,000
10	12390	INR 11,225,340	12390	INR 9,515,520
11	15000	INR 13,590,000	15000	INR 11,520,000
12	12390	INR 11,225,340	12390	INR 9,515,520
13	15000	INR 13,590,000	15000	INR 11,520,000
14	12390	INR 11,225,340	12390	INR 9,515,520
15	15000	INR 13,590,000	15000	INR 11,520,000
16	12390	INR 11,225,340	12390	INR 9,515,520
17	15000	INR 13,590,000	15000	INR 11,520,000
Roof	15000	INR 13,590,000	15000	INR 11,520,000
Total Cost		INR 276,638,040		INR 234,501,120
	Steel is	INR 42,136,920	higher	
Cost of steel structure is		18%	higher as compared to concrete	

However, the question lies if this additional cost is worth the investment. The answer is yes because a the additional cost can be regained with the advantages that steel design provides. A steel building provides more carpet area due to smaller size of structural elements like columns and shear walls. Moreover, shear walls can be completely eliminated with the use of bracing system. This strategy has been applied in redesign of The Optimus and the increase in rentable space increases the total rent per month by 2%. In addition, the steel superstructure will increase the speed of construction. Therefore, the spaces can be rented much earlier in the bustling demand for office spaces in Mumbai. The redesign in steel will also reduced the cost of building facade. In the concretes structure, a special facade has been designed for architecture and solar shading strategies. However, a special strategy has been used in The Optimus where, the exterior structural steel frame is used as an architectural element as well as a solar shading element. To sum up the discussion, it would be a challenge to convince the clients in Mumbai to construct a steel building but the benefits will surely lure them to look into steel as an option. However, in future steel will become a prevalent material as new challenges surface in the construction industry in India.

Cost comparison based on Carpet area						
Average Rent of commercial office space =			INR 129	per square ft. per month		
	Steel			Concrete		
Level	Area (sq ft)	Area of structure per floor (sq ft.)	Cost of floor	Area	Area of structure per floor (sq ft.)	Cost of floor
Ground	21000	164	INR 2,687,834	21000	703	INR 2,618,313
1	21000	164	INR 2,687,834	21000	703	INR 2,618,313
2	21000	164	INR 2,687,834	21000	703	INR 2,618,313
3	21000	164	INR 2,687,834	21000	703	INR 2,618,313
4	21000	164	INR 2,687,834	21000	703	INR 2,618,313
5	21000	164	INR 2,687,834	21000	703	INR 2,618,313
6	12390	109	INR 1,584,249	12390	376	INR 1,549,839
7	15000	109	INR 1,920,939	15000	376	INR 1,886,529
8	12390	109	INR 1,584,249	12390	376	INR 1,549,839
9	15000	109	INR 1,920,939	15000	376	INR 1,886,529
10	12390	109	INR 1,584,249	12390	376	INR 1,549,839
11	15000	42	INR 1,929,582	15000	301	INR 1,896,171
12	12390	42	INR 1,592,892	12390	301	INR 1,559,533
13	15000	42	INR 1,929,582	15000	301	INR 1,896,171
14	12390	42	INR 1,592,892	12390	301	INR 1,559,481
15	15000	42	INR 1,929,582	15000	241	INR 1,903,937
16	12390	42	INR 1,592,892	12390	241	INR 1,567,221
17	15000	42	INR 1,929,582	15000	241	INR 1,903,911
Roof	15000	42	INR 1,929,582	15000	241	INR 1,903,911
		Total rent	INR 39,148,218			INR 38,322,788
Rent in a Steel structure is higher by			INR 825,430	2% higher as compared to concrete		

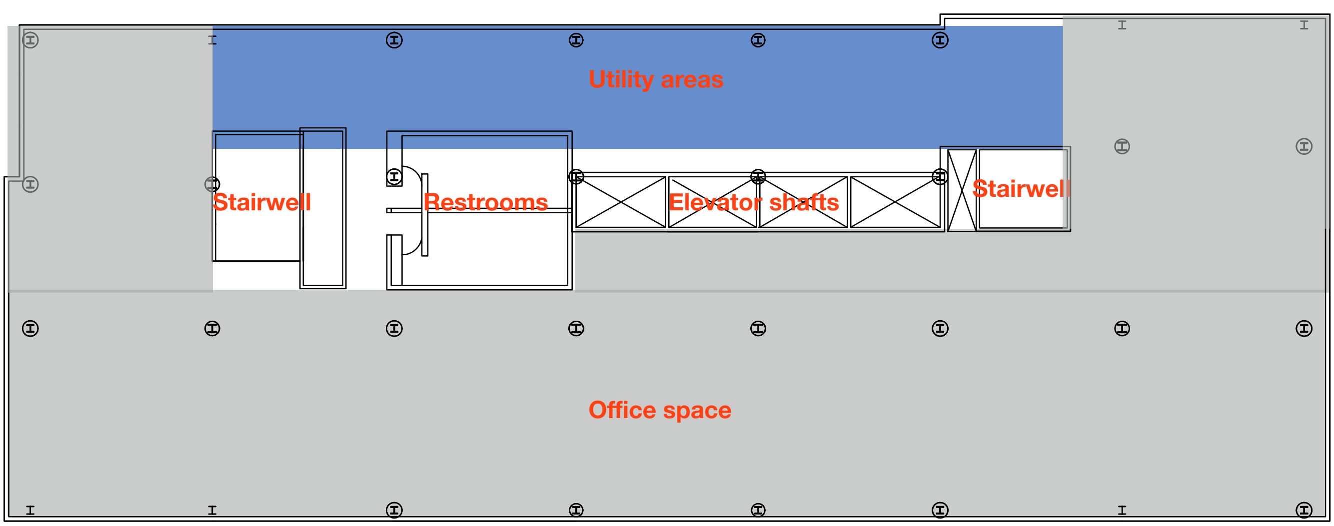
Breadth Study

Architectural redesign and implications

Structural system design in steel has a significant effect on the architectural design of the building. The architect has to make a decision whether to expose the structure or conceal it with architectural encasing and claddings. The existing structural system is concealed from the exterior with a facade. From the interior, the concrete gives an empty effect with clear undulated ceilings and smooth columns. However, it was decide to celebrate the structural system in the redesign of the structure in steel. This requires co-ordination between architect and structural engineer. The exposure of structural steel creates a tectonic visual effect. From the interior, the slender columns give ample of open space and unonstructucd view. On the other hand, exposing the structural members on the exterior makes to look light and transparent. A disadvantage of exposing the steel structural elements is that it becomes susceptible to fire. Therefore, a special coating called Intumescent paint is applied to steel structural members. Because this paint is expensive, the use of encased concrete on steel members will help reduce the use of this paint. This is one of the ways, every structural decision in the redesign of The Optimus was taken keeping into mind its integration with the architecture.

Layout of structural elements: Columns and braced frames

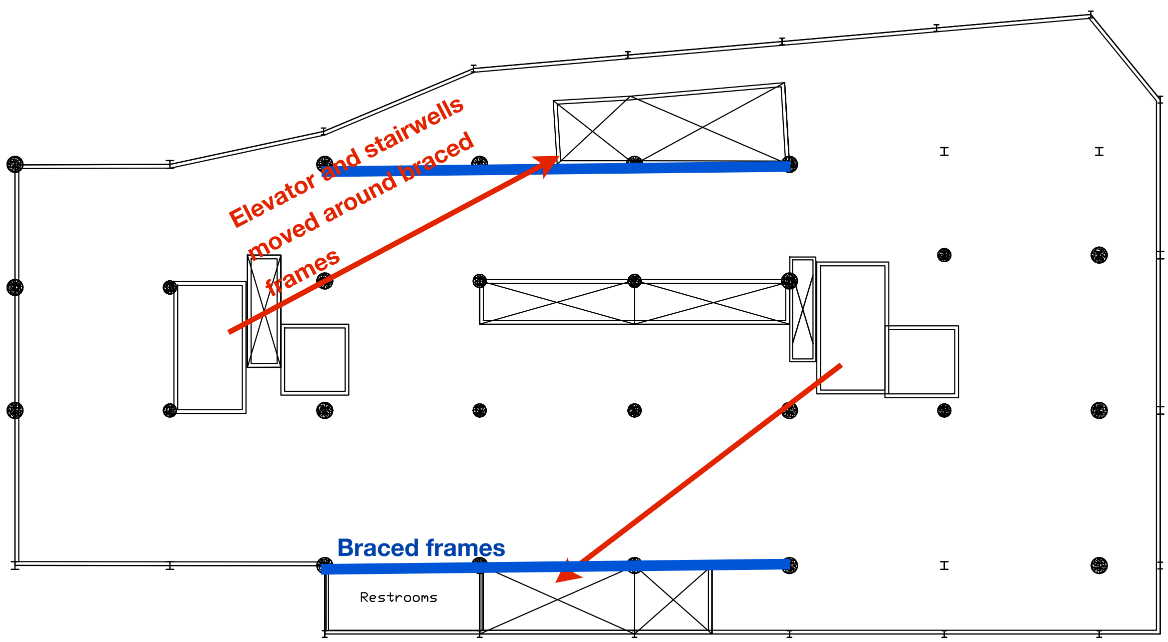
The existing building promises office space that is completely customizable by the occupants. The occupants have the liberty to select their own floor finishes, partition walls and elements that could significantly increase the floor loads. In order to maintain this design decision, higher loads were assumed in the office spaces. These loads significantly increased the loads on the columns which had to be encased in reinforced concrete for increased load capacity. Now, the redesign composite floors system can handle loads form the exquisite marble and granite flooring that Indian's are fond of. The layout of the columns and bracing system was kept in mind to create an open floor plan. Therefore, 3 types of columns systems were designed - Interior gravity columns, exterior gravity columns and columns that are part of the lateral system. One of the greatest advantages to the architecture of the building came from the lateral braced frames. Not only, this system increased interior space by eliminating hefty reinforced shear walls, but also moving the braced frames to the exterior completely transformed the interior space.



Typical office floor plan

Parking spaces

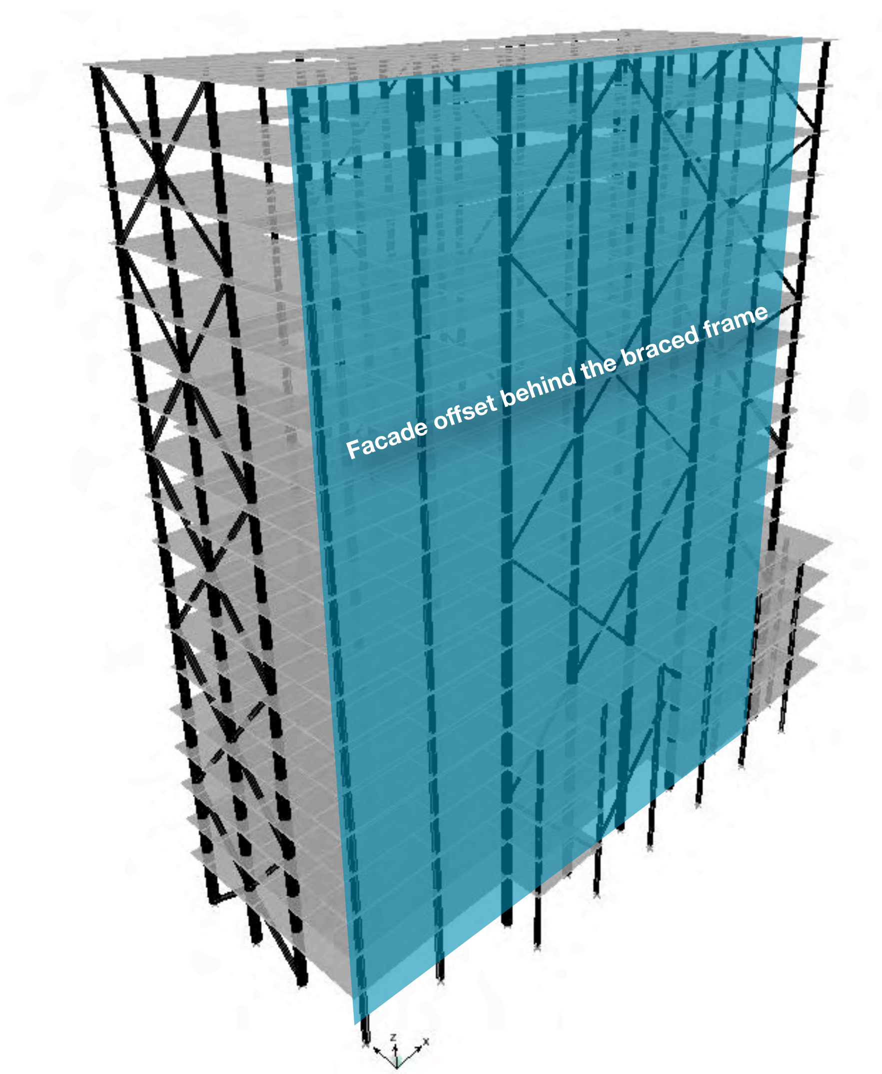
The transformation of reinforced concrete shear walls to braced frames and its relocation to the exterior also gave an advantage to the parking spaces. In the existing structure, the continuity of the structural shear walls forced to keep the stairwells and elevator shafts at one location - the interior. However, in the redesigned steel structure, the elevator shafts and the stairwell is relocated to spaces where it would not be able to park cars. This has increased parking spaces which has a great added advantage to the revenue.



Typical parking level plan

Architecture of the facade

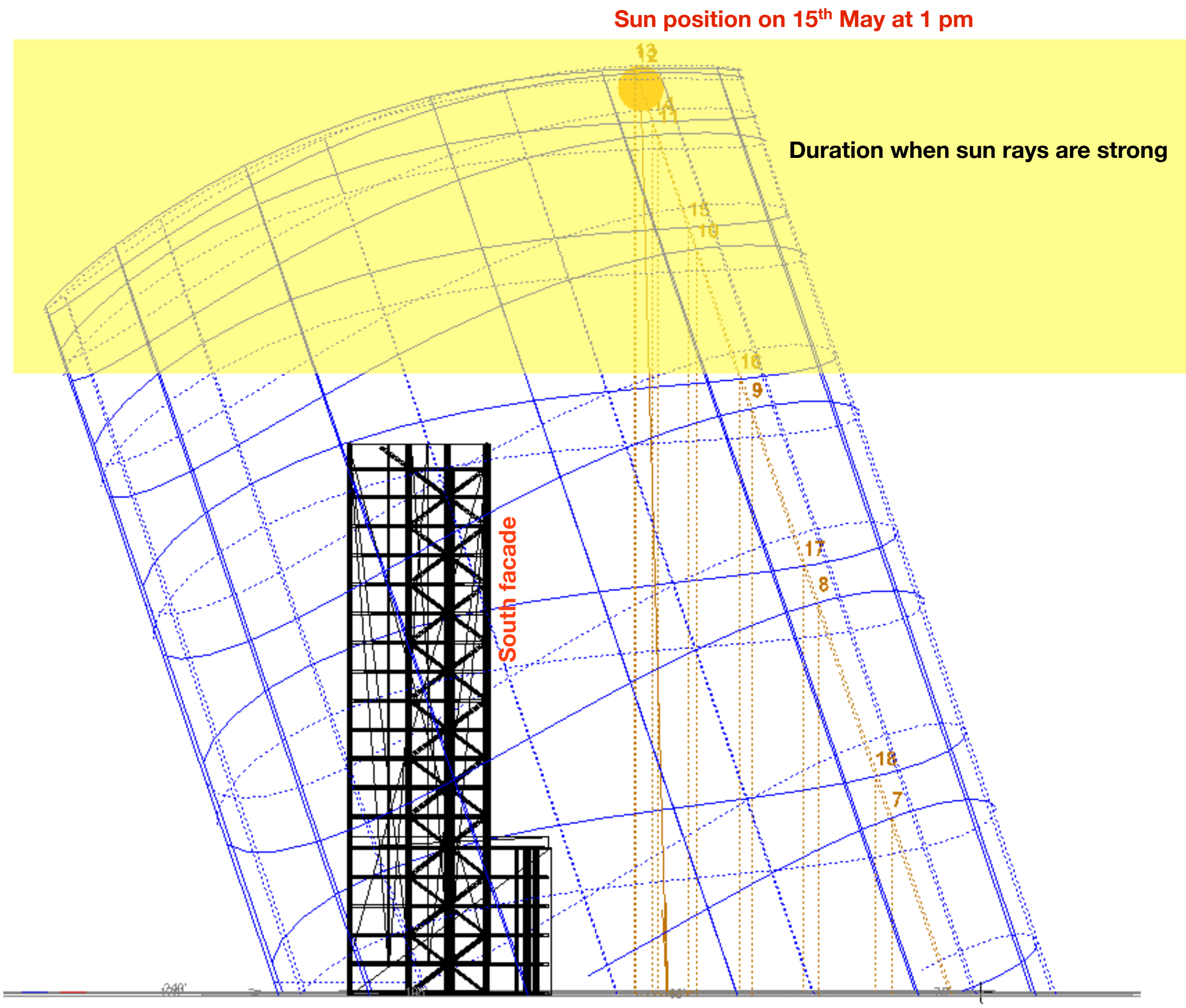
The relocation of the braced lateral system to the exterior has had significant effect on the building facade. The tectonic expression of exposed hollow structural sections as braces has transformed the facade. This itself acts as building facade where the facade mullions are attached to the beams and braces and offset towards the interior. To avoid over expression of the braces on the exterior, only the south facade has exposed steel to the exterior. The east and west facades has millions offset to the exterior to create a feeling of expanded interior space.



Breadth 2: Facade modifications

The architecture of the facade with exposed brace frames and steel beams and mullions offset in the interior is a traditional indian strategy to light the space with daylight without over exposure from the sun. In indian architectural terminology it is called “Jaali” or “Lattice”.

Daylighting



Ecotect Analysis: Sun path visualization

The intent behind the use of exterior exposed bracing, column and beams system was to control the amount of sunlight penetrating into the space at the south facade. The south facade is where the office spaces are located that provide panoramic views of the city. With exposed and extruded structure towards the exterior, the glass and mullion supported at beams, columns and braces is offset in the interior controlling harsh sunlight on the south facade. Autodesk Ecotect software proved to be more advantageous than Project Vasari for visualizing the daylighting on the building facade. This software was used to observe the annual sun path facing the south facade and its effect on the interior comfort. The location of the sun is at a higher altitude because Mumbai is located closer to the equator as compared to North-east of USA. Therefore, the location of the sun is at a high altitude angle when the sun is at its highest power. The ecotect analysis image shows that the location of the sun is at a high angle on 15th May at 1 pm of the day. This is one of the hotter days in the calendar of city of Mumbai.The design of the facade has facilitated to manage harsh sunlight into the office and achieve an optimum light level during the day.

LEED

The LEED rating system is slow getting popular in India due to globalization of the construction industry. The redesign in steel can help secure significant LEED points. Also, by using the holistic approach of integration of architecture, structure and facade can help achieve better quality indoor environments. In terms of structure, steel is a material with high recyclable capacity as compared to concrete. The application of architectural and daylighting strategies used can also help achieve LEED points. Therefore, a switch to steel structural system can have advantages over multiple disciplines and Architecture and Facade are just a few of many.

Conclusion

The existing structure of The Optimus was redesigned from concrete to steel. Existing superimposed dead loads were not changed as it was expected that, no change would be made to et use of architectural elements. Due to this, increased loads were obtained in interior gravity columns. In response to these increased loads, reinforced concrete encasing was applied to the gravity columns. The layout of the gravity columns was performed to integrate with the architecture to achieve an open floor plan suitable for customizable space.

The lateral system was re-designed from concrete shear walls to steel braced frames and moved from interior to the perimeter of the building for improved interior space and less torsion in the structure. The lateral system on the exterior also became the facade of the building. This facade acts as support for glass mullions as wells provides solar shading on the south facade.

The structural design was performed using finite element modeling in ETABS and design was carried out used excel sheets. The purpose of FEM modeling was to perform complex calculations to make the design process faster.

The cost benefit analysis was performed using the numbers created obtained from Turner and Townsend Report on comparison between concrete and steel structures in India. From the analysis it we found that, the steel structure is expensive as compared to concrete. However, the future benefits of steel construction outweigh the cost advantage of concrete.

Finally, the integration of architecture, structure and faced was carried out by analyzing the design changes made to architecture and facade due to structural redesign.

References

Turner & Townsend, and JSW Severfield Structures Ltd. *Commercial Building in India: Comparison between Concrete and Steelwork Structures.* Rep. Mumbai: n.p., 2009. Print.

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Appendices

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Typical Office Floor design

DL	38	psf
SDL	62.7	psf
LL	100	psf
factored load with LL NR	281	psf
factored load with LL reduced	267	psf

Partially composite Section

Live load reduction

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

K_{LL} =	2	Table 4.2, ASCE 7-10
A_T =	253.9	sq. ft.
$K_{LL}A_T$ =	508	Live load can be reduced
LL=	100	psf
LL reduced=	**92**	**psf**

Select steel decking using Vulcraft tables

1

Deck span	3	
Clear span	9' 6"	
LL NR	100	
Unshored span	9' 6"	
Selected	1.5VL20	
topping thicknes	2	in
total slab depth	3.5	in
Slab dead load	33	psf

2 Selecting wide flange section

orientation **deck ribs are perpendicular to span for beams**

span	27.6	ft
spacing	9.2	ft
b'	3.45	ft
b eff=	6.9	ft
w factored=	2.5	k / ft
Vu	68	kip
Mu	234	kip ft

PNA has to be below top of flange for practially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

Qn=	17.2	kip	3/4 in stud for 4 ksi NW concrete		from table 3.21 AISC Manual
rib width - wr	2.5	in	wr / hr	1.6667	Weak studs per rib (conservative)
rib height - hr	1.5	in			1 stud per rib

Trial sections

Section	Y2	ϕMp	∑ Qn	# studs
W14X30	3.1	249	248	15

W14x30 beam with 15 studs and 1.5VL20 steel decking

Deflection and additional strength checks

LL deflection				**Unshored strength**				
w_{LL} =	0.9	k / ft		beam self wt.	30	lb/ft		
I_{LB} =	603.0	in^4		w_u =	0.69	k/ft		
	from AISC Manual table 3-20			M_u =	66.15	k-ft		
				$\varphi M_{n\ unshored}$ =	177	k ft	for W14x30	OK
Δ_{LL} =	0.687	in	OK	**Wet concrete deflection**				
$\Delta_{LL\ max}$ =	0.92	in		w_c =	0.33	k/ft		
Check self-weight assumption of 5psf				I =	291	in^4		
Weight of beam	36	lb/ft		Δ_{LL} =	0.516	in	OK	
Weight of beam in psf=	1	psf	Assumption is sufficient	$\Delta_{LL\ max}$ =	1.38	in		

orientation deck ribs are parallel to span for girder

span	27.6	ft
spacing	27.6	ft
b'	3.45	ft
b eff=	6.9	ft
w factored=	4.9	k / ft
Vu	136	kip
Mu	468	kip ft

PNA has to be below top of flange for pratially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

	Qn=	21.5	kip	3/4 in stud for 4 ksi NW		from table 3.21 AISC Manual
rib width	wr	2.5	in	wr / hr	1.6667	
rib height	hr	1.5	in			

Trial sections

Section	Y2	ϕMp	∑ Qn @ PNA 3	# studs
W18X46	2.63	548	492	23

W18x46 girder with 23 studs per rib and 1.5VL steel decking

Deflection and additional strength checks

LL deflection				**Unshored strength**				
w_{LL} =	2.8	k / ft		beam self wt.	46	lb/ft		
I_{LB} =	1451.8	in^4		w_u =	2.03	k/ft		
	from AISC Manual table 3-20			M_u =	193.43	k-ft		
				$\varphi M_{n\ unshored}$ =	340	k ft	for W18x40	OK
Δ_{LL} =	0.856	in	OK	**Wet concrete deflection**				
$\Delta_{LL\ max}$ =	0.920	in		w_c =	0.96	k/ft		
Check self-weight assumption of 5psf				I =	712	in^4		
Weight of beam	46	lb/ft		Δ_{LL} =	0.605	in	OK	
Weight of beam in psf=	2	psf	Assumption is sufficient	$\Delta_{LL\ max}$ =	1.38	in		

Typical Parking floor design

DL	38	psf
SDL	36	psf
LL	40	psf
factored load	152.8	psf

Partially composite Section

1 Select steel decking using Vulcraft tables

Deck span	3	
Clear span	9' 6"	
LL NR	40	
Unshored span	9' 6"	
Selected	1.5VL20	We can use the same used for office floors
topping thicknes	2	in
total slab depth	3.5	in
Slab dead load	33	psf

2 Selecting wide flange section

orientation **deck ribs are perpendicular to span for beams**

span	27.6	ft
spacing	9.2	ft
b'	3.45	ft
b eff=	6.90	ft
w factored=	1.4	k / ft
Vu	38.8	kip
Mu	133.9	kip ft

PNA has to be below top of flange for pratially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

Qn=	17.2	kip	3/4 in stud for 4 ksi NW concrete		from table 3.21 AISC Manual
rib width - wr	2.5	in	wr / hr	1.7	Weak studs per rib (conservative)
rib height - hr	1.5	in			1 stud per rib

Trial sections

Section	**Y2**	**φMp**	∑ Qn @ PNA 4	# studs	
W10x22	3.2	161	169	10	**W10x22 beam with 10 studs and 1.5VL20 steel decking**

Deflection and additional strength checks

LL deflection

wLL =	0.4	k / ft	
ILB =	264.0	in4	
	from AISC Manual table 3-20		
ΔLL =	0.628	in	OK
ΔLL max =	0.92	in	

Check self-weight assumption of 5psf

Unshored strength

beam self wt.	22	lb/ft		
wu =	0.69	k/ft		
Mu =	65.2	k-ft		
φMn unshore	97.5	k ft	for W14x30	OK

Wet concrete deflection

wc =	0.33	k/ft
I =	118	in4

Weight of beam	22	lb/ft		ΔLL =	1.242	in	OK
Weight of beam in psf=	1	psf	Assumption is sufficient	ΔLL max =	1.38	in	

orientation **deck ribs are parallel to span for girder**

span	27.6	ft
spacing	27.6	ft
b'	3.45	ft
b eff=	6.9	ft
w factored=	2.8	k / ft
Vu	78	kip
Mu	268	kip ft

PNA has to be below top of flange for pratially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

	Qn=	21.5	kip	3/4 in stud for 4 ksi NW		from table 3.21 AISC Manual
rib width	wr	2.5	in	wr / hr	1.7	
rib height	hr	1.5	in			

Trial sections

Section	Y2	ϕMp	∑ Qn @ PNA 4	# studs
W14x34	3.0	320	270	13

W14x34 girder with 13 studs per rib and 1.5VL20 steel decking

Deflection and additional strength checks

LL deflection

w_{LL} =	1.1	k / ft	
I_{LB} =	691.0	in^4	
	from AISC Manual table 3-20		
Δ_{LL} =	0.719	in	OK
$\Delta_{LL\ max}$ =	0.920	in	

Check self-weight assumption of 5psf

Weight of beam	34	lb/ft	
Weight of beam in psf=	1	psf	Assumption is sufficient

Unshored strength

beam self wt.	34	lb/ft		
w_u =	2.02	k/ft		
M_u =	192.05	k-ft		
$\varphi M_{n\ unshored}$ =	205	k ft	for W14x30	OK

Wet concrete deflection

w_c =	0.94	k/ft	
I =	340	in^4	
Δ_{LL} =	1.251	in	OK
$\Delta_{LL\ max}$ =	1.38	in	

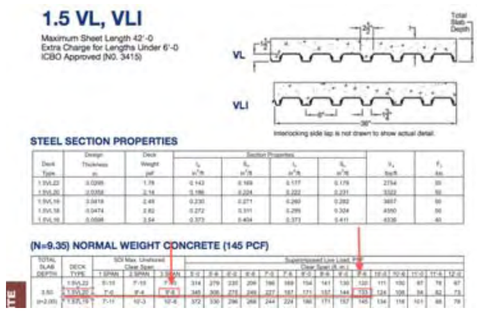
Typical Mechanical Floor design

DL	38	psf
SDL	41.8	psf
LL	150	psf
factored load with LL NR	336	psf

Partially composite Section

1 Select steel decking using Vulcraft tables

Deck span	3	
Clear span	9' 6"	
LL NR	150	
Unshored span	9' 6"	
Selected	1.5VL20	
topping thicknes	2	in
total slab depth	3.5	in
Slab dead load	33	psf



2 Selecting wide flange section

orientation **deck ribs are perpendicular to span for beams**

span	27.6	ft
spacing	9.2	ft
b'	3.45	ft
b eff=	6.9	ft
w factored=	3.1	k / ft
Vu	85	kip
Mu	294	a

PNA has to be below top of flange for practially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

Qn=	17.2	kip	3/4 in stud for 4 ksi NW concrete		from table 3.21 AISC Manual
rib width - wr	2.5	in	wr / hr	1.6667	Weak studs per rib (conservative)
rib height - hr	1.5	in			1 stud per rib

Trial sections

Section	Y2	ϕMp	∑ Qn	# studs
W16X40	2.9	423	325	19

W16x40 beam with 19 studs and 1.5VL20 steel decking

Deflection and additional strength checks

LL deflection			**Unshored strength**		
w_{LL} =	1.4	k / ft	beam self wt.	40	lb/ft
I_{LB} =	1030.0	in^4	w_u =	0.71	k/ft
	from AISC Manual table 3-20		M_u =	67.29	k-ft

				$\varphi M_{n\ unshored}$ =	274	k ft	for W14x30 OK
Δ_{LL} =	0.603	in	OK	**Wet concrete deflection**			
$\Delta_{LL\ max}$ =	0.92	in		w_c =	0.34	k/ft	
Check self-weight assumption of 5psf				I =	518	in^4	
Weight of beam	40	lb/ft		Δ_{LL} =	0.299	in	OK
Weight of beam in psf=	1	psf	Assumption is sufficient	$\Delta_{LL\ max}$ =	1.38	in	

orientation deck ribs are parallel to span for girder

span	27.6	ft
spacing	27.6	ft
b'	3.45	ft
b eff=	6.9	ft
w factored=	6.2	k / ft
Vu	171	kip
Mu	588	kip ft

PNA has to be below top of flange for pratially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

	Qn=	21.5	kip	3/4 in stud for 4 ksi NW		from table 3.21 AISC Manual
rib width	wr	2.5	in	wr / hr	1.6667	
rib height	hr	1.5	in			

Trial sections

Section	Y2	ϕMp	∑ Qn @ PNA 3	# studs
W21x50	2.51	658	560	27

W21x50 girder with 27 studs per rib and 1.5VL steel decking

Deflection and additional strength checks

LL deflection				**Unshored strength**			
w_{LL} =	4.1	k / ft		beam self wt.	50	lb/ft	
I_{LB} =	2040.0	in^4		w_u =	2.04	k/ft	
	from AISC Manual table 3-20			M_u =	193.88	k-ft	
				$\varphi M_{n\ unshored}$ =	413	k ft	OK
Δ_{LL} =	0.914	in	OK	**Wet concrete deflection**			
$\Delta_{LL\ max}$ =	0.920	in		w_c =	0.96	k/ft	
Check self-weight assumption of 5psf				I =	984	in^4	
Weight of beam	50	lb/ft		Δ_{LL} =	0.440	in	OK
Weight of beam in psf=	2	psf	Assumption is sufficient	$\Delta_{LL\ max}$ =	1.38	in	

Restroom Composite beam design

DL	38	psf
SDL	100	psf
LL	40	psf
factored load with LL reduced	206	psf

Partially composite Section

1 Select steel decking using Vulcraft tables

Deck span	3	
Clear span	7.67	
LL NR		
Unshored span	7.67	
Selected Deck	1.5VL20	
topping thicknes	2	in
total slab depth	3.5	in
Slab dead load	33	psf

Live load reduction

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

K_{LL} =	2	Table 4.2, ASCE 7-10
A_T =	756.3	sq. ft.
$K_{LL}A_T$ =	1513	Live load can be reduced
LL=	40	psf
LL reduced=	**25**	**psf**

2 Selecting wide flange section

orientation **deck ribs are perpendicular to span for beams**

span	27.5	ft
spacing	7.67	ft
b'	3.44	ft
b eff=	6.9	ft
w factored=	1.6	k / ft
Vu	44	kip
Mu	150	kip ft

PNA has to be below top of flange for practially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

Qn=	17.2	kip	3/4 in stud for 4 ksi NW concrete		from table 3.21 AISC Manual
rib width - wr	2.5	in	wr / hr	1.6667	weak stud
rib height - hr	1.5	in			1 stud per rib

Trial sections

Section	Y2	ϕMp	∑ Qn @ PNA 4	# studs
W10 x 22	3.2	161	169	10

W10x22 beam with 10 studs and 1.5VL20 steel decking

Deflection and additional strength checks

LL deflection

w_{LL} =	0.3	k / ft	
I_{LB} =	282.0	in^4	
	from AISC Manual table 3-20		
Δ_{LL} =	0.483	in	OK
$\Delta_{LL\ max}$ =	0.91666667	in	

Check self-weight assumption of 5psf

Weight of beam	22	lb/ft	
Weight of beam in psf=	1	psf	Assumption is sufficient

Unshored strength

beam self wt.	22	lb/ft		
w_u =	0.58	k/ft		
M_u =	54.41	k-ft		
$\varphi M_{n\ unshored}$ =	97.5	k ft	for W14x30	OK

Wet concrete deflection

w_c =	0.28	k/ft	
I =	118	in^4	
Δ_{LL} =	1.035	in	OK
$\Delta_{LL\ max}$ =	1.375	in	

Typical roof design

DL	38	psf
SDL	115	psf
LL	100	psf
factored load with LL NR	344	psf

Partially composite Section

1 Select steel decking using Vulcraft tables

Deck span	3	
Clear span	9' 6"	
LL NR	100	
Unshored span	9' 6"	
Selected	1.5VL20	
topping thicknes	2	in
total slab depth	3.5	in
Slab dead load	33	psf

2 Selecting wide flange section

orientation deck ribs are perpendicular to span for beams

span	27.6	ft
spacing	9.2	ft
b'	3.45	ft
b eff=	6.9	ft
w factored=	3.2	k / ft
Vu	87	kip
Mu	301	kip ft

PNA has to be below top of flange for practially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

Qn=	17.2	kip	3/4 in stud for 4 ksi NW concrete		from table 3.21 AISC Manual
rib width - wr	2.5	in	wr / hr	1.6667	Weak studs per rib (conservative)
rib height - hr	1.5	in			1 stud per rib

Trial sections

Section	Y2	ϕMp	∑ Qn	# studs
W14x30	2.8	381	386	23

W14x30 beam with 23 studs and 1.5VL20 steel decking

Deflection and additional strength checks

LL deflection			**Unshored strength**		
w_{LL} =	0.9	k / ft	beam self wt.	38	lb/ft
I_{LB} =	802.0	in^4	w_u =	0.70	k/ft
	from AISC Manual table 3-20		M_u =	67.07	k-ft

				$\varphi M_{n\ unshored}$ =	231	k ft	OK
Δ_{LL} =	0.516	in	OK	**Wet concrete deflection**			
$\Delta_{LL\ max}$ =	0.92	in		w_c =	0.34	k/ft	
Check self-weight assumption of 5psf				I =	385	in^4	
Weight of beam	38	lb/ft		Δ_{LL} =	0.399	in	OK
Weight of beam in psf=	1	psf	Assumption is sufficient	$\Delta_{LL\ max}$ =	1.38	in	

orientation — deck ribs are parallel to span for girder

span	27.6	ft
spacing	27.6	ft
b'	3.45	ft
b eff=	6.9	ft
w factored=	6.3	k / ft
Vu	174	kip
Mu	602	kip ft

PNA has to be below top of flange for pratially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

	Qn=	21.5	kip	3/4 in stud for 4 ksi NW		from table 3.21 AISC Manual
rib width	wr	2.5	in	wr / hr	1.6667	
rib height	hr	1.5	in			

Trial sections

Section	Y2	ϕMp	∑ Qn @ PNA 3	# studs
W21x50	2.66	658	473	22

W21x50 girder with 22 studs per rib and 1.5VL steel decking

Deflection and additional strength checks

LL deflection				**Unshored strength**			
w_{LL} =	2.8	k / ft		beam self wt.	50	lb/ft	
I_{LB} =	1940.0	in^4		w_u =	2.04	k/ft	
	from AISC Manual table 3-20			M_u =	193.88	k-ft	
				$\varphi M_{n\ unshored}$ =	413	k ft	OK
Δ_{LL} =	0.641	in	OK	**Wet concrete deflection**			
$\Delta_{LL\ max}$ =	0.920	in		w_c =	0.96	k/ft	
Check self-weight assumption of 5psf				I =	984	in^4	
Weight of beam	50	lb/ft		Δ_{LL} =	0.440	in	OK
Weight of beam in psf=	2	psf	Assumption is sufficient	$\Delta_{LL\ max}$ =	1.38	in	

TypicalComposite Edge beam

DL	38 psf		Façade load	0.0157 k/ft
SDL	62.7 psf		factored faça	0.01884
LL	100 psf			

factored load with LL NR 281 psf

Partially composite Section

1 **Select steel decking using Vulcraft tables**

Deck span	3
Clear span	9' 6"
LL NR	100
Unshored span	9' 6"
Selected	1.5VL20
topping thicknes	2 in
total slab depth	3.5 in
Slab dead load	33 psf

Live load reduction

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

K_{LL} =	1	Table 4.2, ASCE 7-10
A_T =	253.9	sq. ft.
$K_{LL}A_T$ =	254	Live load cannot be reduced
LL=	100	psf
LL reduced=	**119**	**psf**

2 **Selecting wide flange section**

orientation **deck ribs are perpendicular to span for EDGE beams**

span	27.6	ft
spacing	9.2	ft
b'	1.64	ft
b eff=	3.3	ft
w factored=	2.6	k / ft
Vu	72	kip
Mu	248	kip ft

PNA has to be below top of flange for practially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

Qn=	17.2	kip		3/4 in stud for 4 ksi NW concrete	from table 3.21 AISC Manual
rib width - wr	2.5	in	wr / hr	1.6667	Weak studs per rib (conservative)
rib height - hr	1.5	in			1 stud per rib

Trial sections

Section	Y2	ϕMp	∑ Qn	# studs
W16X40	2.3	274	325	19

W16x40 beam with 19 studs and 1.5VL20 steel decking

Deflection and additional strength checks

LL deflection

w_{LL} =	0.9	k / ft
I_{LB} =	937.0	in^4

Unshored strength

beam self wt.	40	lb/ft
w_u =	0.71	k/ft

	from AISC Manual table 3-20		
Δ_{LL} =	0.442	in	OK
$\Delta_{LL\ max}$ =	0.92	in	

Check self-weight assumption of 5psf

Weight of beam	40	lb/ft	
Weight of beam in psf=	1	psf	Assumption is sufficient

M_u =	67.29	k-ft		
$\varphi M_{n\ unshored}$ =	274	k ft	for W14x30	OK

Wet concrete deflection

w_c =	0.34	k/ft	
I =	518	in^4	
Δ_{LL} =	0.299	in	OK
$\Delta_{LL\ max}$ =	1.38	in	

orientation — deck ribs are parallel to span for EDGE girder

span	27.6	ft
spacing	27.6	ft
b'	1.64	ft
b eff=	3.3	ft
w factored=	2.6	k / ft
Vu	72	kip
Mu	248	kip ft

PNA has to be below top of flange for pratially composite action.
Using table 3.19 from AISC Manual
Available locaions for PNA for economic section according to AISC Manual - 2, 3, 4, 5

	Qn=	21.5	kip	3/4 in stud for 4 ksi NW	from table 3.21 AISC Manual
rib width	wr	2.5	in	wr / hr	1.6667
rib height	hr	1.5	in		

Trial sections

Section	Y2	ϕMp	∑ Qn @ PNA 4	# studs
W18x46	2.01	513	400	19

W18x46 girder with 19 studs per rib and 1.5VL steel decking

Deflection and additional strength checks

LL deflection

w_{LL} =	2.8	k / ft	
I_{LB} =	1380.0	in^4	
	from AISC Manual table 3-20		
Δ_{LL} =	0.900	in	OK
$\Delta_{LL\ max}$ =	0.920	in	

Check self-weight assumption of 5psf

Weight of beam	46	lb/ft	
Weight of beam in psf=	2	psf	Assumption is sufficient

Unshored strength

beam self wt.	46	lb/ft		
w_u =	2.03	k/ft		
M_u =	193.43	k-ft		
$\varphi M_{n\ unshored}$ =	340	k ft	for W18x40	OK

Wet concrete deflection

w_c =	0.96	k/ft	
I =	612	in^4	
Δ_{LL} =	0.704	in	OK
$\Delta_{LL\ max}$ =	1.38	in	

Design sheet of critical gravity composite column at Level 1 and 2

Pu compression =	3706	kip
unbraced length	20	ft
Mu=	0	k ft

following section I2 in ASCE 7-10

Steel section info.			**Steel reinf**			**Concrete**		
KL=	20	ft	Fysr=	60	ksi	Circular cross section		
Wide flange			bar size (long) =	#8		d =	**28**	**in**
W14x176			d long bar =	1	in	Ag =	615.8	in^2
Fy=	50	ksi	A bar =	0.79	in^2	5 ksi NW wt		
As =	51.8	in^2	# of bars	8		f'c =	5	ksi
Es =	29000	ksi	bar size (tie) =	#3	in			
Is =	2140	in^4	d bar (tie) =	0.375	in			
			Asr =	6.32	in^2	Ac =	557.6	in^2
			Es =	29000	ksi	wc =	145	pcf
			I_{sr} =	623.2	in^4	Ec =	3904.2	ksi
			reinf ratio	0.0103	OK	Ic =	120687.4	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section

As=	51.8	in^2	% area of steel core	8.41	OK
A tot =	615.8	in^2			

C1 =	0.27	OK
EI eff =	198316596	
P no =	5339.1	kip
Pe =	33981.0	kip
P np / Pe =	0.1571	Use part a

part a :	phi Pn =	3749.5	kip			
	DCR=	0.99	OK	~~part b :~~	~~29801.35~~	~~kip~~

Design sheet of critical gravity composite column at Level 3 and 4

Pu compression =	3402	kip
unbraced length	13	ft
Mu=	0	k ft

following section I2 in ASCE 7-10

Steel section info.			**Steel reinf**			**Concrete**		
KL=	13	ft	Fysr=	60	ksi	Circular cross section		
Wide flange			bar size (long) =	#8		d =	**26**	**in**
W14x176			d long bar =	1	in	Ag =	530.9	in^2
Fy=	50	ksi	A bar =	0.79	in^2	5 ksi NW wt		
As =	51.8	in^2	# of bars	8		f'c =	5	ksi
Es =	29000	ksi	bar size (tie) =	#3	in			
Is =	2140	in^4	d bar (tie) =	0.375	in			
			Asr =	6.32	in^2	Ac =	472.8	in^2
			Es =	29000	ksi	wc =	145	pcf
			I_{sr} =	537.2	in^4	Ec =	3904.2	ksi
			reinf ratio	0.0119	OK	Ic =	89727.0	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section

As=	51.8	in^2	% area of steel core	9.76	OK
A tot =	530.9	in^2			

C1 =	0.30	OK
EI eff =	174061413	
P no =	4978.6	kip
Pe =	70591.6	kip
P np / Pe =	0.0705	Use part a

part a :	phi Pn =	3625.4	kip			
	DCR=	0.94	OK	~~part b :~~	~~61908.84~~	~~kip~~

Design sheet of critical gravity composite column at Levels 5, 6, and 7

Pu compression =	3132	kip
unbraced length	13	ft
Mu=	0	k ft

following section I2 in ASCE 7-10

Steel section info.			**Steel reinf**			**Concrete**		
KL=	13	ft	Fysr=	60	ksi	Circular cross section		
Wide flange			bar size (long) =	#8		d =	**26**	**in**
W14x145			d long bar =	1	in	Ag =	530.9	in^2
Fy=	50	ksi	A bar =	0.79	in^2	5 ksi NW wt		
As =	42.7	in^2	# of bars	8		f'c =	5	ksi
Es =	29000	ksi	bar size (tie) =	#3	in			
Is =	1710	in^4	d bar (tie) =	0.375	in			
			Asr =	6.32	in^2	Ac =	481.9	in^2
			Es =	29000	ksi	wc =	145	pcf
			I_{sr} =	537.2	in^4	Ec =	3904.2	ksi
			reinf ratio	0.0119	OK	Ic =	89727.0	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section

As=	42.7	in^2	% area of steel core	8.04	OK
A tot =	530.9	in^2			

C1 =	0.26	OK
EI eff =	149438068	
P no =	4562.3	kip
Pe =	60605.5	kip
P np / Pe =	0.0753	Use part a

part a :	phi Pn =	3315.6	kip			
	DCR=	0.94	OK	~~part b :~~	~~53150.99~~	~~kip~~

Design sheet of critical gravity composite column at Levels 8, 9 and 10

Pu compression =	2009	kip
unbraced length	13	ft
Mu=	0	k ft

following section I2 in ASCE 7-10

Steel section info.			Steel reinf			Concrete		
KL=	13	ft	Fysr=	60	ksi	Circular cross section		
Wide flange			bar size (long) =	#8		d =	**22**	**in**
W14x120			d long bar =	1	in	Ag =	380.1	in^2
Fy=	50	ksi	A bar =	0.79	in^2	5 ksi NW wt		
As =	35.3	in^2	# of bars	8		f'c =	5	ksi
Es =	29000	ksi	bar size (tie) =	#3	in			
Is =	1380	in^4	d bar (tie) =	0.375	in			
			Asr =	6.32	in^2	Ac =	338.5	in^2
			Es =	29000	ksi	wc =	145	pcf
			I_{sr} =	388.3	in^4	Ec =	3904.2	ksi
			reinf ratio	0.0166	OK	Ic =	45996.1	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section

As=	35.3	in^2	% area of steel core	9.29	OK
A tot =	380.1	in^2			

C1 =	0.29	OK
EI eff =	97524193	
P no =	3582.9	kip
Pe =	39551.5	kip
P np / Pe =	0.0906	Use part a

part a :	phi Pn =	2587.2	kip			
	DCR=	0.78	OK	~~part b :~~	~~34686.66~~	~~kip~~

Design information for critical column

Pu =	1300	kip		
unbraced length =	13.0	ft	156.0	inches

rx/ry=	1.67
KL y eq =	7.8

Compression check

for KL=	13.0	ft	
Pu=	1300	kip	
try W14x120			
ΦPn =	1400	kip	from AISC Table 4-1
radius of gyration =	3.74	in	
KL/r=	41.71		
4.71 x sqrt(E/Fy)	118.3		
Ag =	35.3	in^2	
b/t=	7.8		
lambda r =	33.7		
Non slender	**Use equation equation E3-2**		
Fe	164.51	kip	
Fcr	40.92	kip	
ΦPn=	1300	kip	OK
DCR	1.00		

Please refer to AISE Spec Chapter E3 when performing this calculation

Design information for critical column

Pu =	1887	kip		
unbraced length =	13.0	ft	156.0	inches

rx/ry=	1.6
KL y eq =	8.1

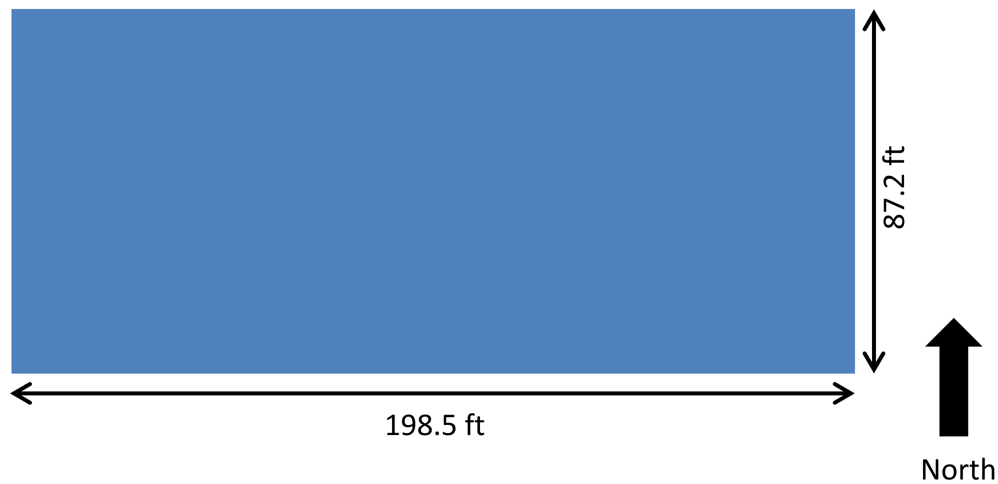
Compression check

for KL=	13.0	ft	
Pu=	1887	kip	
try W14x176			
ΦPn =	2090	kip	from AISC Table 4-1
radius of gyration =	4.02	in	
KL/r=	38.81		
4.71 x sqrt(E/Fy)	118.3		
Ag =	51.8	in^2	
b/t=	5.97		
lambda r =	33.7		
Non slender	**Use equation equation E3-2**		
Fe	190.06	kip	
Fcr	41.57	kip	
ΦPn=	1938	kip	OK
DCR	0.97		

Please refer to AISE Spec Chapter E3 when performing this calculation

Building Information

Dimensional Information



N-S direction		
L=	87.2	ft
B=	198.5	ft

E-W direction		
L=	198.5	ft
B=	87.2	ft

Mean roof height= 228 ft

floor to floor height 13 ft
height from grount to 1st level 20 ft

Calculating average length and width of the building				
height			length	width
from	to	total heigh		
ground	level 5	72	203	109.85
level 5	roof	156	196.4	76.71
		average	198.5	87.2

Step 1
Risk Category of building from Table: 1.5-1 ASCE 7-10: Category 2

Step 2
Basic wind speed
Mumbai: 98.4 miles/hr

Step 3
Wind Load parameters

a	Wind directionality factor Kd	d	Enclosure classification
b	Exposure category	e	Internal pressure coefficient

c Topographic factor f Gust effect factor

a Wind directionality factor Table 26.6-1 ASCE 7-10

K_d= 0.85

b Exposure Category Section 26.7.3

building is located close to the arabian sea and wind flows for a distance of atleast 1 mile

Category - D

c Topographic factor Section 26.8.2 ASCE 7-10

K_{zt} = 1.0

d Enclosure Classification

Building is completely enclosed

e Internal pressure coefficient

Gc_{pi} +/-0.18

f Gust effect factor Section 26.9 ASCE 7-10

refer to gust effect factor sheet

Insert Gust effect factor sheet

f Gust effect factor Section 26.9 ASCE 7-10

N-S direction

h	228.00	ft
B	198.50	ft
L	87.18	ft

Approximate natural frequency

for structural steel with other lateral force resisting system

N-S direction E-W direction	n_a =	0.3289	75/h

Assuming this building is flexible or dynamically sensitive building

Using section 26.9.5 ASCE 7-10

G_f =	**0.86326**	
g_Q	3.4	
g_V	3.4	
g_R	3.92	
$I_{z\ bar}$ =	0.12	
c =	0.15	
R=	0.0031	
Q=	0.835676	
B	198.50	ft
h	228.00	ft
$L_{z\ bar}$ =	776.44	
ℓ =	650	ft
z bar	136.8	ft
ε bar =	0.125	
N_1 =	1.888864	

R_n	0.092
R_h	0.316
R_B	0.000
R_L	0.259
β	0.05

R_h	
η =	2.55
$V_{z\ bar}$ =	135.22
1/η =	0.392
1/2η^2	0.077
1-e^(-2η)	0.994

R_B	
η =	40614.43
$V_{z\ bar}$ =	135.22
1/η =	0.00
1/2η^2	0.00
1-e^(-2η)	1.00

R_L	
η =	3.27
$V_{z\ bar}$ =	135.22
1/η =	0.31
1/2η^2	0.05
1-e^(-2η)	1.00

$L_{z\ bar}$ =	776.44	
ℓ =	650	ft
z bar =	136.8	ft
ε bar =	0.125	
$V_{z\ bar}$ =	135.22	
b bar	0.8	
z bar	136.8	ft
α bar	0.111111	
V	98.4	ft/sec

f Gust effect factor Section 26.9 ASCE 7-10

E-W direction

h	228.00	ft
B	87.18	ft
L	198.50	ft

Approximate natural frequency
for structural steel with other lateral force resisting system

N-S direction
E-W direction n_a = 0.3289

Assuming this building is flexible or dynamically sensitive building

Using section 26.9.5 ASCE 7-10

G_f =	0.87182	
g_Q	3.4	
g_V	3.4	
g_R	3.92	
$I_{z\ bar}$ =	0.12	
c =	0.15	
Q=	0.858443	
B	87.18	ft
h	228.00	ft
$L_{z\ bar}$ =	776.44	
l =	650	ft
z bar	136.8	ft
ε bar =	0.125	
N_1 =	1.888864	

R=	0.0044
R_n	0.092
R_h	0.316
R_B	0.000
R_L	0.125
β	0.05

R_h	
η =	2.55
$V_{z\ bar}$ =	135.22
1/η =	0.392
1/2η^2	0.077
1-e^(-2η)	0.994

R_B	
η =	17836.64
$V_{z\ bar}$ =	135.22
1/η =	0.00
1/2η^2	0.00
1-e^(-2η)	1.00

R_L	
η =	7.44
$V_{z\ bar}$ =	135.22
1/η =	0.13
1/2η^2	0.01
1-e^(-2η)	1.00

$L_{z\ bar}$ =	776.44	
l =	650	ft
z bar =	136.8	ft
ε bar =	0.125	
$V_{z\ bar}$ =	135.22	
b bar	0.8	
z bar	136.8	ft
α bar	0.111111	
V	98.4	ft/sec

N-S Direction	G=	0.86
E-W direction	G=	0.87

Step 4 & 5
Velocity pressure exposure coefficient

Velocity pressure Calculation			
Story	Elevation (ft)	K_z	q_z (lb/ft^2)
Ground	0	1.0	21.7
1	20	1.1	22.8
2	33	1.2	24.9
3	46	1.3	26.4
4	59	1.3	27.5
5	72	1.4	28.5
6	85	1.4	29.3
7	98	1.4	30.1
8	111	1.5	30.7
9	124	1.5	31.3
10	137	1.5	31.9
11	150	1.5	32.4
12	163	1.6	32.9
13	176	1.6	33.3
14	189	1.6	33.7
15	202	1.6	34.1
16	215	1.6	34.5
17	228	1.7	34.8
Roof (level 18)	241	1.7	35.2
qh=	35.18		

Exposure category : D

K_{zt}	1.00
K_d	0.85
V	98.40

story height	13	ft
height from grount to 1st level	20	ft

α =	11.5
z_g =	700

Step 6
External pressure coefficient Figure 27.4-1 ASCE 7-10

Walls

N-S direction

L=	87.2	ft
B=	198.5	ft
L/B	0.4	

	C_p	Use with
Windward wal	0.8	qz
Leeward wall	-0.5	qh
Side wall	-0.7	qh

E-W direction

L=	198.5	ft
B=	87.2	ft
L/B	2.3	

	C_p	Use with
Windward wal	0.8	qz
Leeward wall	-0.29	qh
Side wall	-0.7	qh

Roofs

h=	228.0	ft			
N-S direction				**E-W direction**	
h/L	2.62			h/L	1.15
θ < 10				**θ < 10**	
horizontal dist from windward edge		Cp			
0 to h/2	<= 100 sq.	-1.3	-0.18		
	250 sq. ft.	-1.17	-0.18		
	>= 1000 sq	-1.04	-0.18		
> h/2		-0.7	-0.18		

Step 7 **Wind pressures**

Roof pressure coefficient		
0 to h/2	-1.04	-0.18
> h/2	-0.7	-0.18
	G_f =	0.86326

C_p Pressure coefficients	
for windward wall	0.8
for leeward wall	-0.5
for side wall	-0.7

Internal pressure coefficient			
+ Gcpi	0.18		
- Gc_{pi}	-0.18		
		q_h =	35.18

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 +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure (+)**	**Net pressure (-)**
Ground	0	21.7	15.0	6.33	-6.33	8.65	21.32
1	20	22.8	15.8	6.33	-6.33	9.43	22.09
2	33	24.9	17.2	6.33	-6.33	10.86	23.53
3	46	26.4	18.2	6.33	-6.33	11.88	24.55
4	59	27.5	19.0	6.33	-6.33	12.69	25.35
5	72	28.5	19.7	6.33	-6.33	13.36	26.02
6	85	29.3	20.3	6.33	-6.33	13.94	26.60
7	98	30.1	20.8	6.33	-6.33	14.44	27.11
8	111	30.7	21.2	6.33	-6.33	14.90	27.56
9	124	31.3	21.6	6.33	-6.33	15.31	27.98
10	137	31.9	22.0	6.33	-6.33	15.69	28.36
11	150	32.4	22.4	6.33	-6.33	16.04	28.71
12	163	32.9	22.7	6.33	-6.33	16.37	29.03
13	176	33.3	23.0	6.33	-6.33	16.67	29.34
14	189	33.7	23.3	6.33	-6.33	16.96	29.62
15	202	34.1	23.6	6.33	-6.33	17.23	29.89
16	215	34.5	23.8	6.33	-6.33	17.49	30.15
17	228	34.8	24.1	6.33	-6.33	17.73	30.40
Roof (level 18)	241	35.2	24.3	6.33	-6.33	17.96	30.63
Leeward pressure Cp =-0.5							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure**	internal pressure +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure**	**Net pressure**
All	241.00	35.2	**-15.2**	6.3	-6.3	**-21.5**	**-8.9**
Side wall pressure Cp =-0.7							
Level	Elevation (ft)	q_z (lb/ft^2)	**Wind pressure**	internal pressure +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure**	**Net pressure**
all	241.00	35.2	**-21.3**	6.3	-6.3	**-27.6**	**-14.9**
Roof pressures							
Level	levation (ft	q_z (lb/ft^2)	**Wind pressure**	internal pressure +Gcpi*qi	internal pressure -Gcpi*qi	**Net pressure**	**Net pressure**
0 to h/2 (Cp=-1.04)	241.00	35.2	**-31.6**	6.3	-6.3	**-37.9**	**-25.3**
0 to h/2 (Cp=-0.18)	241.00	35.2	**-21.3**	6.3	-6.3	**-27.6**	**-14.9**

floor height	13	ft	ground to 1st floor height	20	ft
B=	198.5	ft			

N-S Direction

Story Force due to Windward pressure

Level	Elevation (ft)	Net Wind pressure (psf)	trib height below	trib height above	Total trib height	Story shear (lb / ft)	Story shear (kip)	Overturning moment (kip-ft)
Ground	0	8.7	0.0	10.0	10.0	**86.5**	**0**	**0**
1	20	9.4	10.0	6.5	16.5	**155.5**	**30.9**	**617.5**
2	33	10.9	6.5	6.5	13.0	**141.2**	**28**	**925**
3	46	11.9	6.5	6.5	13.0	**154.5**	**30.7**	**1410.6**
4	59	12.7	6.5	6.5	13.0	**165.0**	**33**	**1932**
5	72	13.4	6.5	6.5	13.0	**173.7**	**34.5**	**2482.1**
6	85	13.9	6.5	6.5	13.0	**181.2**	**36**	**3057**
7	98	14.4	6.5	6.5	13.0	**187.8**	**37.3**	**3652.8**
8	111	14.9	6.5	6.5	13.0	**193.7**	**38**	**4268**
9	124	15.3	6.5	6.5	13.0	**199.1**	**39.5**	**4899.6**
10	137	15.7	6.5	6.5	13.0	**204.0**	**40**	**5547**
11	150	16.0	6.5	6.5	13.0	**208.5**	**41.4**	**6208.9**
12	163	16.4	6.5	6.5	13.0	**212.8**	**42**	**6884**
13	176	16.7	6.5	6.5	13.0	**216.7**	**43.0**	**7571.6**
14	189	17.0	6.5	6.5	13.0	**220.5**	**44**	**8271**
15	202	17.2	6.5	6.5	13.0	**224.0**	**44.5**	**8980.9**
16	215	17.5	6.5	6.5	13.0	**227.3**	**45**	**9701**
17	228	17.7	6.5	6.5	13.0	**230.5**	**45.8**	**10431.9**
Roof (level 18)	241	18.0	6.5	6.5	13.0	**233.5**	**46**	**11172**

Story Forces due to Leeward pressure

Level	Elevation (ft)	Net Wind pressure (psf)	Trib height	Story shear (lb / ft)	Story shear (kip)	Overturning moment (kip-ft)
ground	0	-21.5	10.0	-215.2	**0.0**	**0**
1	20	-21.5	16.5	**-355.04**	**70.5**	**1410**
All	241	-21.5	13.0	-279.7	**55.5**	**13382**

Total Base shear in N-S direction	**575**	**kip**
Total Overturning moment in N-S direction	**83221**	**kip-ft**

Roof pressure coefficient			C_p Pressure coefficients		Internal pressure coefficient		
0 to h/2	-1.04	-0.18	for windward wall	0.8	+ Gcpi	0.18	
> h/2	-0.7	-0.18	for leeward wall	-0.29	- Gc_{pi}	-0.18	
	G_f =	0.87182	for side wall	-0.7		q_h =	35.18

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**	**-Gcpi*qi**	**Net pressure (+)**	**Net pressure (-)**
Ground	0	21.7	15.14	6.33	-6.33	8.80	21.47
1	20	22.8	15.92	6.33	-6.33	9.58	22.25
2	33	24.9	17.36	6.33	-6.33	11.03	23.70
3	46	26.4	18.40	6.33	-6.33	12.06	24.73
4	59	27.5	19.21	6.33	-6.33	12.88	25.54
5	72	28.5	19.89	6.33	-6.33	13.55	26.22
6	85	29.3	20.47	6.33	-6.33	14.14	26.80
7	98	30.1	20.98	6.33	-6.33	14.65	27.32
8	111	30.7	21.44	6.33	-6.33	15.11	27.77
9	124	31.3	21.86	6.33	-6.33	15.53	28.19
10	137	31.9	22.24	6.33	-6.33	15.91	28.57
11	150	32.4	22.59	6.33	-6.33	16.26	28.93
12	163	32.9	22.92	6.33	-6.33	16.59	29.26
13	176	33.3	23.23	6.33	-6.33	16.90	29.56
14	189	33.7	23.52	6.33	-6.33	17.19	29.85
15	202	34.1	23.80	6.33	-6.33	17.46	30.13
16	215	34.5	24.05	6.33	-6.33	17.72	30.39
17	228	34.8	24.30	6.33	-6.33	17.97	30.63
Roof (level 18)	241	35.2	24.54	6.33	-6.33	18.20	30.87
Leeward pressure Cp =-0.29							
Level	Elevation (ft)	q_z (lb/ft^2)	Wind pressure	internal pressure +Gcpi*qi	-Gcpi*qi	Net pressure	Net pressure
All	241.00	35.2	**-8.9**	6.3	-6.3	**-15.2**	**-2.6**
Side wall pressure Cp =-0.7							
Level	**Elevation (ft)**	**q_z (lb/ft^2)**	**Wind pressure**	**internal pressure +Gcpi*qi**	**-Gcpi*qi**	**Net pressure**	**Net pressure**
all	241.00	35.2	**-21.5**	6.3	-6.3	**-27.8**	**-15.1**
Roof pressures							
Level	**Elevation (ft)**	**q_z (lb/ft^2)**	**Wind pressure**	**internal pressure +Gcpi*qi**	**-Gcpi*qi**	**Net pressure**	**Net pressure**
0 to h/2 (Cp=-1.04)	241.00	35.2	**-31.9**	6.3	-6.3	**-38.2**	**-25.6**
0 to h/2 (Cp=-0.18)	241.00	35.2	**-21.5**	6.3	-6.3	**-27.8**	**-15.1**

floor height	13	ft	ground to 1st floor height	20	ft
B=	87.1753	ft			

E-W direction

Story Force due to Windward pressure

Level	Elevation (ft)	Net Wind pressure (psf)	trib height below	trib height above	Total trib height	Story shear (lb / ft)	Story shear (Kip)	Overturning moment (kip-ft)
Ground	0	8.8	0.0	10.0	10.0	**88.0**	**0**	**0**
1	20	9.6	10.0	6.5	16.5	**158.1**	**13.8**	**275.7**
2	33	11.0	6.5	6.5	13.0	**143.4**	**13**	**413**
3	46	12.1	6.5	6.5	13.0	**156.8**	**13.7**	**628.9**
4	59	12.9	6.5	6.5	13.0	**167.4**	**15**	**861**
5	72	13.6	6.5	6.5	13.0	**176.2**	**15.4**	**1106.0**
6	85	14.1	6.5	6.5	13.0	**183.8**	**16**	**1362**
7	98	14.7	6.5	6.5	13.0	**190.5**	**16.6**	**1627.1**
8	111	15.1	6.5	6.5	13.0	**196.4**	**17**	**1901**
9	124	15.5	6.5	6.5	13.0	**201.8**	**17.6**	**2181.9**
10	137	15.9	6.5	6.5	13.0	**206.8**	**18**	**2470**
11	150	16.3	6.5	6.5	13.0	**211.4**	**18.4**	**2764.5**
12	163	16.6	6.5	6.5	13.0	**215.7**	**19**	**3065**
13	176	16.9	6.5	6.5	13.0	**219.7**	**19.2**	**3370.7**
14	189	17.2	6.5	6.5	13.0	**223.5**	**19**	**3682**
15	202	17.5	6.5	6.5	13.0	**227.0**	**19.8**	**3997.6**
16	215	17.7	6.5	6.5	13.0	**230.4**	**20**	**4318**
17	228	18.0	6.5	6.5	13.0	**233.6**	**20.4**	**4643.0**
Roof (level 18)	241	18.2	6.5	6.5	13.0	**236.7**	**21**	**4972**

Story Forces due to Leeward pressure

Level	Elevation (ft)	Net Wind pressure (psf)	Trib height	Story shear (lb / ft)	Story shear (Kip)	Overturning moment (kip-ft)
Ground	0	-15.2	10.0	**-152**	0.0	**0**
1	20	-15.2	16.5	**-251**	21.9	**438**
All	241	-15.2	13.0	**-198**	17.3	**4159**

Total Base shear in N-S direction	**273**	**kip**
Total Overturning moment in N-S direction	**39041**	**kip-ft**

Center of Mass and Center of rigidity output from ETABS

		COM		COR		moment arm	
Story	Diaphragm	XCM	YCM	XCR	YCR	x (ft)for force acting in N-S direction	y (ft) for force acting in E-W direction
LEVEL 1	D1	1166.4	572.8	1311.3	566.8	12.1	0.5
LEVEL 2	D1	1166.9	573.1	1312.5	566.7	12.1	0.5
LEVEL 3	D1	1190.0	511.5	1312.8	480.7	10.2	2.6
LEVEL 4	D1	1166.8	573.2	1313.7	566.6	12.2	0.6
LEVEL 5	D1	1189.4	511.9	1314.0	480.3	10.4	2.6
LEVEL 6	D1	1166.7	573.4	1314.6	566.6	12.3	0.6
LEVEL 7	D1	1189.1	512.1	1315.2	479.4	10.5	2.7
LEVEL 8	D1	1167.3	573.7	1315.9	566.6	12.4	0.6
LEVEL 9	D1	1189.4	512.2	1316.4	478.4	10.6	2.8
LEVEL 10	D1	1166.4	574.0	1316.9	566.5	12.5	0.6
LEVEL 11	D1	1187.8	512.8	1317.6	477.9	10.8	2.9
LEVEL 12	D1	1165.9	574.1	1318.1	566.5	12.7	0.6
LEVEL 13	D1	1188.0	512.9	1318.8	477.2	10.9	3.0
LEVEL 14	D1	1343.3	619.9	1319.6	655.9	2.0	3.0
LEVEL 15	D1	1346.5	619.9	1319.8	656.0	2.2	3.0
LEVEL 16	D1	1346.5	619.9	1320.1	656.2	2.2	3.0
LEVEL 17	D1	1346.5	619.9	1320.4	656.3	2.2	3.0
ROOF	D1	1345.5	619.4	1320.6	656.3	2.1	3.1

Load Case definitions

WX = Windward force in X dir (kip)
WY = Windward force in Y dir (kip)
LX = Leeward force in X dir (Kip)
LY = Leeward force in Y dir (kip)
WMX = (WX + LX) x 0.15ey (kip)
WMY = (WY + LY) x 0.15ex (kip)

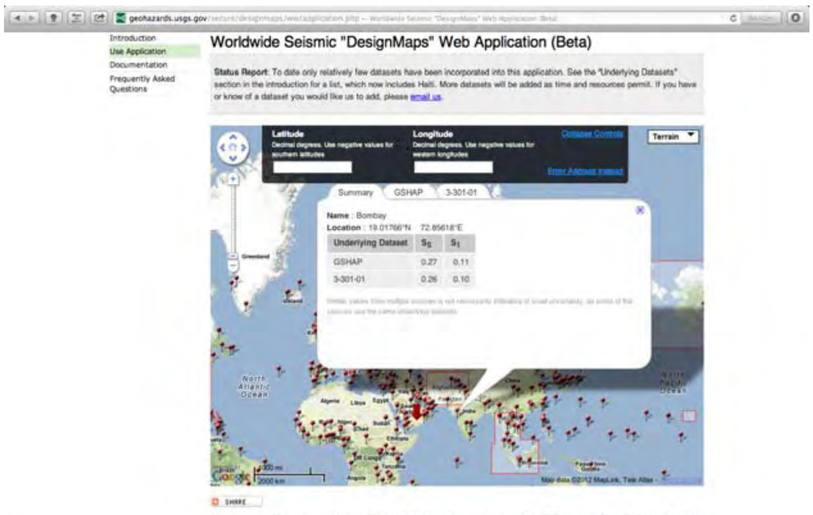
Level	WX	LX	WY	LY	ex	ey	WMX	WMY
Ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	13.78	21.90	30.88	70.48	12.07	0.50	2.69	183.50
2	12.50	17.26	28.03	55.53	12.14	0.53	2.38	152.10
3	13.67	17.26	30.67	55.53	10.24	2.56	11.88	132.33
4	14.59	17.26	32.74	55.53	12.24	0.55	2.64	162.08
5	15.36	17.26	34.47	55.53	10.39	2.63	12.89	140.23
6	16.02	17.26	35.96	55.53	12.33	0.57	2.83	169.15
7	16.60	17.26	37.27	55.53	10.51	2.72	13.83	146.34
8	17.12	17.26	38.45	55.53	12.38	0.60	3.09	174.54
9	17.60	17.26	39.51	55.53	10.58	2.82	14.73	150.86
10	18.03	17.26	40.49	55.53	12.54	0.62	3.29	180.60
11	18.43	17.26	41.39	55.53	10.82	2.91	15.57	157.29
12	18.80	17.26	42.23	55.53	12.68	0.64	3.45	185.91
13	19.15	17.26	43.02	55.53	10.90	2.97	16.24	161.13
14	19.48	17.26	43.76	55.53	1.98	3.00	16.55	29.46
15	19.79	17.26	44.46	55.53	2.23	3.01	16.73	33.45
16	20.08	17.26	45.12	55.53	2.20	3.02	16.93	33.23
17	20.36	17.26	45.75	55.53	2.18	3.03	17.12	33.12
Roof (level 18)	20.63	17.26	46.36	55.53	2.07	3.07	17.47	31.65

All these loads were input in ETABS as windload cases and the most critical load case was determined. This critical case was further used in ASCE design load combinations for designing the lateral force resisting system

Seimic Base shear calculation in East-West direction (ASCE 7-10)

Building Information

story height	13	ft
ground to 1st floor height	20	ft
structural height or height of MWFRS	228	ft



Seismic Response Coefficient (Cs)

Section 11.4.4

S_s=	0.27		S_{MS}=	0.22
S_1=	0.11		S_{M1}=	0.09
Site Class	A	(Chapter 20)	S_{DS}=	0.14
F_a=	0.80		S_{D1}=	0.06
F_v=	0.80			

Section 12.8.2.1

Using period from ETABS model

$T_a = C_t h_n^x$ =	1.1735	seconds
C_t =	0.02	for all structural systems according to AISC table 12.8-2
x =	0.75	
h_n =	228	height from base of the structure to the top of LFRS
T_L=	6.00	seconds figure 22.12

$R=$ 5 from table 12.2.1 for composite moment resisting

Risk category 2 from table 1.5-1

$I_e=$ 1.00 from table 1.5-2

$C_s=$ 0.029

T<TL use equation 12.8.3

Equation 12.8.3

$C_s=$ 0.0100

Cs in eq. 12.8-2 exceeds result from eq. 12.8.3

Equation 12.8.5

$C_s=$ 0.0063 don't use this equation

$S_1=$ 0.11 do not use equation equation 12.8.6

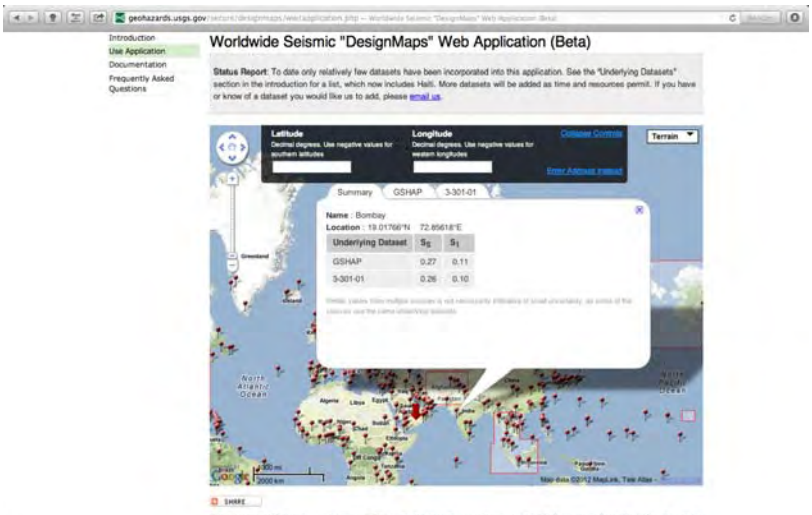
So, use $C_s=$ 0.0100

Effective seismic weight	44085.18	
Seismic base shear	**441**	**kip**

Seimic Base shear calculation in North-South direction (ASCE 7-10)

Building Information

story height	13	ft
ground to 1st floor height	20	ft
structural height or height of MWFRS	228	ft



Seismic Response Coefficient (Cs)

Section 11.4.4

S_s=	0.27		S_{MS}=	0.22
S_1=	0.11		S_{M1}=	0.09
Site Class	A	(Chapter 20)	S_{DS}=	0.14
F_a=	0.80		S_{D1}=	0.06
F_v=	0.80			

Section 12.8.2.1

Using period from ETABS model

$T_a = C_t h_n^x$ =	1.1735	seconds
C_t =	0.02	for all structural systems according to AISC table 12.8-2
x =	0.75	
h_n =	228	height from base of the structure to the top of LFRS
T_L=	6.00	seconds figure 22.12

R=	5	from table 12.2.1 for steel and concrete composite concentric braced frames
Risk category	2	from table 1.5-1
I_e=	1.00	from table 1.5-2
C_s=	0.029	

T<TL use equation 12.8.3

Equation 12.8.3	
C_s=	0.0100
Cs in eq. 12.8-2 exceeds result from eq. 12.8.3	

Equation 12.8.5		
C_s=	0.0063	don't use this equation

S_1=	0.11	do not use equation equation 12.8.6

So, use	**C_s=**	**0.0100**

Effective seismic weight	44085.18	
Seismic base shear	**441**	**kip**

S_{D1} = 0.06

Mode	Period	UX	UY	Sa = S_{D1}/T	Sa/(R/I)	$(Cm \times UX\%)^2$	$(Cm \times UY\%)^2$
1	4.3869	79.6490	0.0007	0.0137	0.0023	3.30E-06	2.55E-16
2	3.5891	0.0007	68.2716	0.0167	0.0028	3.80E-16	3.62E-06
3	2.3741	0.0023	1.0950	0.0253	0.0042	9.39E-15	2.13E-09
4	1.5274	13.2184	0.0001	0.0393	0.0065	7.49E-07	4.29E-17
5	1.2041	0.0000	20.5580	0.0498	0.0083	0.00E+00	2.92E-06
6	0.8863	3.3211	0.0012	0.0677	0.0113	1.40E-07	1.83E-14
7	0.8239	0.0531	0.2240	0.0728	0.0121	4.15E-11	7.39E-10
8	0.6660	0.0000	4.9878	0.0901	0.0150	0.00E+00	5.61E-07
9	0.6353	1.5776	0.0007	0.0945	0.0157	6.17E-08	1.21E-14
10	0.5164	0.8321	0.0000	0.1162	0.0194	2.60E-08	0.00E+00
11	0.4855	0.0041	1.3530	0.1236	0.0206	7.13E-13	7.77E-08
12	0.4532	0.0063	0.4850	0.1324	0.0221	1.93E-12	1.15E-08
	Sum =	98.6647	96.9771				

Cm,x =	SQSS =	0.002	<85^Cs	Required to scale forces and drifts by $0.85^*C_sW/V_t$ according to ASCE 7-10 12.9.4.1 and 2
Cm,y =	SQSS =	0.003	<85^Cs	
CS, ELFP =	0.008			

0.85*CsW/Vt = 0.08066 i.e. need to scale drifts and forces by 8.1%

Efficiency in bracing					
	Force (kip)	**displacement (in)**	**stiffness**	**steel brace length**	**stiffness per unit length**
R1	180	9.3	19	691.2	0.028
R2	180	6.5	28	2020.6	0.014
R3	180	4.1	44	1422.7	0.031
R4	180	28.6	6	1292	0.005

Design information for critical composite column (braced frame)

Pu compression =	2423	kip
unbraced length	20	ft
Mu=	200	k ft

following section I2 in ASCE 7-10

KL=	20	ft	**Steel reinf**			**Concrete**		
Wide flange			d bar tie			Circular cross section		
W12 x 120			Fysr=	60	ksi	d =	**28**	**in**
Fy=	50	ksi	d long bar	1.27	in	Ag =	615.8	in^2
As =	35.2	in^2	d ties =	0.375	in	5 ksi NW wt		
Es =	29000	ksi	A bar =	1.27	in^2	f'c =	5	ksi
Is =	1070	in^4	# of bars	8		b	16	in
			d from NA	5.49	in	h	16	in
			Asr =	10.16	in^2	Ac =	570.4	in^2
			Es =	29000	ksi	wc =	145	pcf
			Isr (for 4 ba	185.8024	in^4	Ec =	3904.2	ksi
			reinf ratio	0.0165	OK	Ic =	5461.3	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section **OK**

As=	35.2	in^2	% area of steel core	5.72	%
A tot =	615.8	in^2			

C1 =	0.2162499	OK
EI eff =	38335097	
P no =	4793.8	kip
Pe =	6568.6	kip
P np / Pe =	0.7298	Use part a

part a :	phi Pn =	2649.0	kip	~~part b :~~	~~5760.676~~	~~kip~~
	DCR=	0.91				

Tension Check

phi Pn =	2133	kip

Design information for critical composite column (braced frame)

Pu compression =	2293	kip
unbraced length	13	ft
Mu=	228.3	k ft

following section I2 in ASCE 7-10

KL=	13	ft	**Steel reinf**			**Concrete**		
Wide flange			d bar tie			Circular cross section		
W12 x 106			Fysr=	60	ksi	d =	**24**	**in**
Fy=	50	ksi	d long bar	1	in	Ag =	452.4	in^2
As =	31.2	in^2	d ties =	0.375	in	5 ksi NW wt		
Es =	29000	ksi	A bar =	1	in^2	f'c =	5	ksi
Is =	933	in^4	# of bars	8		b	16	in
			d from NA	5.625	in	h	16	in
			Asr =	8.00	in^2	Ac =	413.2	in^2
			Es =	29000	ksi	wc =	145	pcf
			Isr (for 4 ba	139.1289	in^4	Ec =	3904.2	ksi
			reinf ratio	0.0177	OK	Ic =	5461.3	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section OK

As=	31.2	in^2	% area of steel core	6.90	%
A tot =	452.4	in^2			

C1 =	0.2404174	OK
EI eff =	34200640	
P no =	3796.1	kip
Pe =	13870.3	kip
P np / Pe =	0.2737	Use part a

part a :	phi Pn =	2538.9	kip	~~part b :~~	~~12164.22~~	~~kip~~
	DCR=	0.90				

Tension Check

phi Pn =	1836	kip

Design information for critical composite column (braced frame)

Pu compression =	1846	kip
unbraced length	13	ft
Mu=	322.0	k ft

following section I2 in ASCE 7-10

KL=	13	ft	**Steel reinf**			**Concrete**		
Wide flange			d bar tie			Circular cross section		
	W12 x 87		Fysr=	60	ksi	d =	**24**	**in**
Fy=	50	ksi	d long bar	1	in	Ag =	452.4	in^2
As =	25.6	in^2	d ties =	0.375	in	5 ksi NW wt		
Es =	29000	ksi	A bar =	1	in^2	f'c =	4	ksi
Is =	740	in^4	# of bars	8		b	16	in
			d from NA	5.625	in	h	16	in
			Asr =	8.00	in^2	Ac =	418.8	in^2
			Es =	29000	ksi	wc =	145	pcf
			Isr (for 4 b	139.1289	in^4	Ec =	3492.1	ksi
			reinf ratio	0.0177	OK	Ic =	5461.3	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section **OK**

As=	25.6	in^2	% area of steel core	5.66	%
A tot =	452.4	in^2			

C1 =	0.2152143	OK
EI eff =	27581788	
P no =	3183.9	kip
Pe =	11186.0	kip
P np / Pe =	0.2846	Use part a

part a :	phi Pn =	2119.7	kip	~~part b :~~	~~9810.08~~	~~kip~~
	DCR=	0.87				

Tension Check

phi Pn =	1584	kip

Design information for critical composite column (braced frame)

Pu compression =	1414	kip
unbraced length	13	ft
Mu=	264.5	k ft

following section I2 in ASCE 7-10

KL=	13	ft	**Steel reinf**			**Concrete**		
Wide flange			d bar tie			Circular cross section		
	W12 x 72		Fysr=	60	ksi	d =	**24**	**in**
Fy=	50	ksi	d long bar	1	in	Ag =	452.4	in^2
As =	21.1	in^2	d ties =	0.375	in	5 ksi NW wt		
Es =	29000	ksi	A bar =	1	in^2	f'c =	4	ksi
Is =	597	in^4	# of bars	8		b	16	in
			d from NA	5.625	in	h	16	in
			Asr =	8.00	in^2	Ac =	423.3	in^2
			Es =	29000	ksi	wc =	145	pcf
			Isr (for 4 ba	139.1289	in^4	Ec =	3492.1	ksi
			reinf ratio	0.0177	OK	Ic =	5461.3	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section OK

As=	21.1	in^2	% area of steel core	4.66	%
A tot =	452.4	in^2			

C1 =	0.1949618	OK
EI eff =	23048546	
P no =	2974.2	kip
Pe =	9347.5	kip
P np / Pe =	0.3182	Use part a

part a :	phi Pn =	1952.5	kip	~~part b :~~	~~8197.731~~	~~kip~~
	DCR=	0.72				

Tension Check

phi Pn =	1382	kip

Design information for critical composite column (braced frame)

Pu compression =	2651	kip
unbraced length	20	ft
Mu=	180.7	k ft

following section I2 in ASCE 7-10

KL=	20	ft	**Steel reinf**			**Concrete**		
Wide flange			d bar tie			Circular cross section		
W12 x 120			Fysr=	60	ksi	d =	**28**	**in**
Fy=	50	ksi	d long bar	1.27	in	Ag =	615.8	in^2
As =	35.2	in^2	d ties =	0.375	in	5 ksi NW wt		
Es =	29000	ksi	A bar =	1.27	in^2	f'c =	5	ksi
Is =	1070	in^4	# of bars	10		b	16	in
			d from NA	5.49	in	h	16	in
			Asr =	12.70	in^2	Ac =	567.9	in^2
			Es =	29000	ksi	wc =	145	pcf
			Isr (for 4 ba	185.8024	in^4	Ec =	3904.2	ksi
			reinf ratio	0.0206	OK	Ic =	5461.3	in^4

Compression Analysis

Cross-sectional area of steel core shall comprise of atleast 1% of the total composite cross section **OK**

As=	35.2	in^2	% area of steel core	5.72	%
A tot =	615.8	in^2			

C1 =	0.2167395	OK
EI eff =	38345537	
P no =	4935.4	kip
Pe =	6570.4	kip
P np / Pe =	0.7512	Use part a

part a :	phi Pn =	2703.0	kip	~~part b :~~	~~5762.245~~	~~kip~~
	DCR=	0.98				

Tension Check

phi Pn =	2270	kip

Grid 2 Level 1, 2, 3

Design information for critical brace

P =	330.4	kip		
unbraced length =	18.4	ft	220.6	inches

Limit states

Tension check					
tension rupture			**tension yield**		
Fu =	58	ksi	Fy=	46	ksi
Ag min =	6.33	in^2	Ag min =	7.98	in^2
	Does not control			Controls	
try 10 x 10 x 5/16	Ag = 11.1	in^2	OK	OK	

Compression check			
for KL=	18.4	ft	
Pu=	330.4	kip	
try 10 x 10 x 5/16			
ΦPn =	367	kip	from AISC Table 4-4
radius of gyration =	3.94	in	
KL/r=	55.99		
4.71 x sqrt(E/Fy)	118.3		
Ag =	11.1	in^2	
b/t=	31.4		
lambda r =	33.7		
Non slender	**Use equation equation E3-2**		
Fe	91.29	kip	
Fcr	37.25	kip	
ΦPn=	372.2	kip	OK
DCR	0.89		

Please refer to AISE Spec Chapter E3 when performing this calculation

Grid 2 Level 4 to 9

Design information for critical brace

Pu =	253	kip		
unbraced length =	18.4	ft	220.8	inches

Limit states

Tension check						
tension rupture				**tension yield**		
Fu =	58	ksi		Fy=	46	ksi
Ag min =	4.85	in^2		Ag min =	6.11	in^2
	Does not control				Controls	
try 8x8x5/16	Ag =	8.76	in^2	OK	OK	

Compression check				
for KL=	18	ft		
Pu=	253	kip		
try 8x8x5/16				
ΦPn =	254	kip	OK	from AISC Table 4-4
radius of gyration =		3.13	in	
KL/r=	70.54			Please refer to AISE Spec Chapter E3
4.71 x sqrt(E/Fy)	118.3			when performing this calculation
Ag =	8.76	in^2		
b/t=	24.5			
lambda r =	33.7			
Non slender		**Use equation equation E3-2**		
Fe	57.52	kip		
Fcr	32.91	kip		
ΦPn=	259	kip	OK	
DCR	0.97			

Grid 2 Level 10 to roof

Design information for critical brace

Pu =	150.9	kip			
unbraced length =	18.4	ft		220.8	inches

Limit states

Tension check

tension rupture				**tension yield**		
Fu =	58	ksi		Fy=	46	ksi
Ag min =	2.89	in^2		Ag min =	3.64	in^2
	Does not control				Controls	
try 6x6x 3/8	Ag =	7.58	in^2	OK	OK	

Compression check

for KL=	18	ft		
Pu=	150.9	kip		
try 6x6x 3/8				
ΦPn =	160	kip	OK	from AISC Table 4-4
radius of gyration =		2.31	in	
KL/r=	95.58		Please refer to AISE Spec Chapter E3	
4.71 x sqrt(E/Fy)	118.3		when performing this calculation	
Ag =	7.58	in^2		
b/t=	14.2			
lambda r =	33.7			
Non slender		**Use equation equation E3-2**		
Fe	31.33	kip		
Fcr	24.88	kip		
ΦPn=	170	kip	OK	
DCR	0.89			

Grid A Level 1 to 5

Design information for critical brace

P =	770	kip		
unbraced length =	18.4	ft	220.6	inches

Limit states

Tension check						
tension rupture				**tension yield**		
Fu =	58	ksi		Fy=	46	ksi
Ag min =	14.75	in^2		Ag min =	18.60	in^2
	Does not control				Controls	
try 14x14x 1/2	Ag =	24.6	in^2	OK	OK	

Compression check			
for KL=	18.4	ft	
Pu=	770	kip	
try 14x14x 1/2			
ΦPn =	896	kip	from AISC Table 4-4
radius of gyration =	5.49	in	
KL/r=	40.19		
4.71 x sqrt(E/Fy)	118.3		
Ag =	24.6	in^2	
b/t=	27.1		
lambda r =	33.7		
Non slender		**Use equation equation E3-2**	
Fe	177.24	kip	
Fcr	41.26	kip	
ΦPn=	913.6	kip	OK
DCR	0.84		

Please refer to AISE Spec Chapter E3 when performing this calculation

Grid A Level 6,7

Design information for critical brace

P =	580	kip		
unbraced length =	18.4	ft	220.6	inches

Limit states

Tension check						
tension rupture				**tension yield**		
Fu =	58	ksi		Fy=	46	ksi
Ag min =	11.11	in^2		Ag min =	14.01	in^2
	Does not control				Controls	
try 12x12x1/2	Ag =	20.9	in^2	OK	OK	

Compression check			
for KL=	18.4	ft	
Pu=	580	kip	
try 12x12x1/2			
ΦPn =	725	kip	from AISC Table 4-4
radius of gyration =	4.68	in	
KL/r=	47.14		
4.71 x sqrt(E/Fy)	118.3		
Ag =	20.9	in^2	
b/t=	22.8		
lambda r =	33.7		
Non slender		**Use equation equation E3-2**	
Fe	128.80	kip	
Fcr	39.61	kip	
ΦPn=	745.1	kip	OK
DCR	0.78		

Please refer to AISE Spec Chapter E3 when performing this calculation

Grid A Level 8 to 12

Design information for critical brace

P =	517	kip		
unbraced length =	18.4	ft	220.6	inches

Limit states

Tension check					
tension rupture			**tension yield**		
Fu =	58	ksi	Fy=	46	ksi
Ag min =	9.90	in^2	Ag min =	12.49	in^2
	Does not control			Controls	
try 12x12x3/8	Ag =	16 in^2	OK	OK	

Compression check			
for KL=	18.4	ft	
Pu=	517	kip	
try 12x12x3/8			
ΦPn =	557	kip	from AISC Table 4-4
radius of gyration =	4.73	in	
KL/r=	46.64		
4.71 x sqrt(E/Fy)	118.3		
Ag =	16	in^2	
b/t=	31.4		
lambda r =	33.7		
Non slender		**Use equation equation E3-2**	
Fe	131.57	kip	
Fcr	39.74	kip	
ΦPn=	572.2	kip	OK
DCR	0.90		

Please refer to AISE Spec Chapter E3 when performing this calculation

Grid A Level 13 to roof

Design information for critical brace

P =	190	kip		
unbraced length =	18.4	ft	220.6	inches

Limit states

Tension check					
tension rupture			**tension yield**		
Fu =	58	ksi	Fy=	46	ksi
Ag min =	3.64	in^2	Ag min =	4.59	in^2
	Does not control			Controls	
try 8 x 8x 5/16	Ag =	8.76 in^2	OK	OK	

Compression check			
for KL=	18.4	ft	
Pu=	190	kip	
try 8 x 8x 5/16			
ΦPn =	244	kip	from AISC Table 4-4
radius of gyration =	3.13	in	
KL/r=	70.48		
4.71 x sqrt(E/Fy)	118.3		
Ag =	8.76	in^2	
b/t=	24.5		
lambda r =	33.7		
Non slender	**Use equation equation E3-2**		
Fe	57.61	kip	
Fcr	32.93	kip	
ΦPn=	259.6	kip	OK
DCR	0.73		

Please refer to AISE Spec Chapter E3 when performing this calculation

1	1.4(D+SDL)	
2	1.2(D+SDL) + 1.6L + 0.5RL	
3A	1.2(D+SDL) + 1.6RL + L	
3B	1.2(D+SDL) + 1.6RL + 0.5WX + 0.5LX	put the critical X dir wind load here
3C	1.2(D+SDL) + 1.6RL + 0.5WY + 0.5LY	
4A	1.2(D+SDL) + 1.0WX +1.0LX + L + 0.5LR	put the critical X dir wind load here
4B	1.2(D+SDL) + 1.0WY +1.0LY + L + 0.5LR	
5	1.2(D+SDL) + 1.0E + L	put critical earth quake load here
6A	0.9(D+SDL) + 1.0WX + 1.0LX	put the critical X dir wind load here
6B	0.9(D+SDL) + 1.0WY + 1.0LY	
7	0.9(D+SDL) + 1.0E	put critical earth quake load here

Brace Design summary for lateral system grid 2				
Story	P	Member	ΦPn	DCR
LEVEL 1	-330	10x10x5/16	367	0.90
LEVEL 2	-317	10x10x5/16	367	0.86
LEVEL 3	-315	10x10x5/16	367	0.86
LEVEL 4	-253	8x8x5/16	254	1.00
LEVEL 6	-235	8x8x5/16	254	0.93
LEVEL 8	-214	8x8x5/16	254	0.84
LEVEL 10	-151	6x6x 3/8	160	0.94
LEVEL 12	-160	6x6x 3/8	160	1.00
LEVEL 14	-148	6x6x 3/8	160	0.92
LEVEL 16	81	6x6x 3/8	160	-0.50
ROOF	-51	6x6x 3/8	160	0.32

Column design summary for lateral system grid 2				
Story	P	Member	ΦPn	DCR
LEVEL 1	-2752.4	W12x120 dia32 8#10	2800	0.98
LEVEL 2	-2638.14	W12x120 dia30 8#8	2800	0.94
LEVEL 5	-2098.38	W12x120 dia30 8#8	2800	0.75
LEVEL 7	-1789.52	W12x87 dia24 8#8	1900	0.94
LEVEL 9	-1526.7	W12x87 dia24 8#8	1900	0.80
LEVEL 11	-1132.68	W12x72 dia24 8#8	1800	0.63
LEVEL 13	-858.84	W12x72 dia24 8#8	1800	0.48
LEVEL 15	-589.8	W12x58 dia22 8#8	1200	0.49
LEVEL 17	-290.11	W12x58 dia22 8#8	1200	0.24

Brace Design summary for lateral system grid A				
Story	P	member	ΦPn	DCR
LEVEL 1	-769.87	14x14x1/2	896	0.86
LEVEL 2	-745.19	14x14x1/2	896	0.83
LEVEL 4	-690.42	14x14x1/2	896	0.77
LEVEL 6	-579.9	14x14x1/2	896	0.65
LEVEL 8	-516.9	12x12x3/8	557	0.93
LEVEL 10	-379.99	12x12x3/8	557	0.68
LEVEL 12	-317.64	12x12x3/8	557	0.57
LEVEL 14	-186.68	8x8x5/16	244	0.77
LEVEL 16	-90.77	8x8x5/16	244	0.37
ROOF	63.71	8x8x5/16	244	-0.26

Column design summary for lateral system grid A				
Story	P	Member	ΦPn	DCR
LEVEL 1	-3190.81	W12x120 dia34 8#10	3500	0.91
LEVEL 2	-3134.15	W12x120 dia34 8#10	3500	0.90
LEVEL 4	-2175.3	W12x120 dia34 8#10	3500	0.62
LEVEL 6	-2060.86	W12x120 dia28 8#8	2600	0.79
LEVEL 8	-1537.09	W12x120 dia28 8#8	2600	0.59
LEVEL 10	-1315.42	W12x58 dia22 8#8	1500	0.88
LEVEL 12	-906.59	W12x58 dia22 8#8	1500	0.60
LEVEL 14	-684.15	W12x58 dia22 8#8	1500	0.46
LEVEL 16	-372.39	W12x45 dia20 8#8	1000	0.37
LEVEL 17	-275.6	W12x45 dia20 8#8	1000	0.28
ROOF	-196.63	W12x45 dia20 8#8	1000	0.20

Beam design summary grid C and H			
Story	M3 kip-ft	Member	
LEVEL 1	0.0	W21x62	
LEVEL 2	-517.1	W21x62	
LEVEL 3	-501.6	W21x62	
LEVEL 4	-450.0	W21x62	
LEVEL 5	-625.3	W24x68	critical beam
LEVEL 6	-630.1	W24x68	critical beam
LEVEL 7	-757.2	W21x93	
LEVEL 8	-606.1	W24x68	
LEVEL 9	-738.3	W21x93	
LEVEL 10	-632.6	W24x68	critical beam
LEVEL 11	-711.3	W21x93	
LEVEL 12	-637.2	W24x68	critical beam
LEVEL 13	-682.6	W24x68	critical beam
LEVEL 14	-647.2	W24x68	critical beam
LEVEL 15	-652.2	W24x68	critical beam
LEVEL 16	-643.1	W24x68	critical beam
LEVEL 17	-652.2	W24x68	critical beam
ROOF	-640.2	W24x68	critical beam

Column design summary for lateral system grid C and H				
Story	P	Member	ΦPn	DCR
LEVEL 1	-3362.13	W12x120 dia34 8#10	3500	0.96
LEVEL 2	-3219.96	W12x120 dia34 8#10	3500	0.92
LEVEL 4	-2940.66	W12x120 dia34 8#10	3500	0.84
LEVEL 6	-2569.28	W12x120 dia28 8#8	2600	0.99
LEVEL 8	-2163.94	W12x120 dia28 8#8	2600	0.83
Level 9		W12x120 dia28 8#8	2600	0.83
LEVEL 10	-1761.57	W12x120 dia28 8#8	2600	0.68
LEVEL 12	-1362.4	W12x58 dia22 8#8	1500	0.91
LEVEL 14	-975.92	W12x58 dia22 8#8	1500	0.65
LEVEL 16	-590.67	W12x44 dia20 8#8	1000	0.59
ROOF	-291.99	W12x44 dia20 8#8	1000	0.29

Brace Design summary for lateral system grid H				
Story	P	member	ΦPn	DCR
LEVEL 1	-814.26	14x14x1/2	896	0.91
LEVEL 2	-800.85	14x14x1/2	896	0.89
LEVEL 3	-594.41	14x14x1/2	896	0.66
LEVEL 4	499.15	12x12x1/2	725	-0.69
LEVEL 6	-424.31	12x12x3/8	557	0.76
LEVEL 8	355.16	12x12x3/8	557	-0.64
LEVEL 10	-299.05	12x12x3/8	557	0.54
LEVEL 12	249.28	8x8x5/16	244	-1.02
LEVEL 14	-145.49	8x8x5/16	244	0.60
LEVEL 16	140.27	8x8x5/16	244	-0.57
LEVEL 17	-130.27	8x8x5/16	244	0.53

Column design summary for lateral system grid H				
Story	P	Member	ΦPn	DCR
LEVEL 1	-3445.48	W12x120 dia34 8#10	3500	0.98
LEVEL 2	-3361.65	W12x120 dia34 8#10	3500	0.96
LEVEL 4	-3264.42	W12x120 dia34 8#10	3500	0.93
LEVEL 6	-2315.79	W12x120 dia28 8#8	2600	0.89
LEVEL 8	-2125.79	W12x120 dia28 8#8	2600	0.82
LEVEL 10	-1409.82	W12x58 dia22 8#8	1500	0.94
LEVEL 11	-1294.75	W12x58 dia22 8#8	1500	0.86
LEVEL 12	-1213.71	W12x58 dia22 8#8	1500	0.81
LEVEL 14	-675.77	W12x45 dia20 8#8	1000	0.68
LEVEL 16	-479.41	W12x45 dia20 8#8	1000	0.48
ROOF	-201.42	W12x45 dia20 8#8	1000	0.20

Steel Connection : Girder to Column Moment Connection

Flange plate bolted and web bolted moment connection

Design information

Mu =	894	k-ft	Vu =	172	kip
Pu =	294.5	kip			

Girder / Beam

W24x68		
d	23.7	in
bf	8.97	in
tf	0.585	in
tw	0.415	in
Sx	177	in
Ag =	20	in^2

Column

W12x58		
d	12.2	in
bf	10	in
tf	0.64	in
tw	0.36	in
k des =	1.02	in
Ag =	17	in^2

Girder/Beam and Column

Fy =	50	ksi
Fu =	65	ksi
Gauge length=	5.5	in

Plates

Fy=	36	ksi
Fu=	58	ksi

Trial flange plate and bolt size

PL 1 1/5 x 8.5 x 20"		
plate thickness=	1 5/8	in
Length	20	in
Width	8.75	in

Flange plate bolts

# of rows	6	
Diameter	1 1/8	in
Bolt type	A325N	
bolt spacing	3	in

Trial web plate and bolt size

PL 3/8 x 5 x 12"		
plate thickness=	1/2	in
Length	16	in
Width	5	in

Web plate bolts

# of rows	5	
Diameter	7/8	in
Bolt type	A325N	
bolt spacing	3	in
# of bolts per line	5	

Weld size to column= D =	4	sixteenth inch

Reduced Strength for members with holes in the tension flange AISC Spec Eq F-13-1

A fg =	5.247	in^2	
A fn =	3.785	in^2	
FuAfn =	246		Nominal flexural strength must not be greater than reduced Mn
Fy/Fu =	0.769		
Yt =	1.0		
YtFyAfg =	262		

Reduced ϕMn =	622	k-ft

Beam web limit states

Single plate web connection

Shear strength of bolts				
Φr_n =	**24.3**	**kip/bolt**	from Table 7-1 AISC Manual	
Bearing strength of bolts (Bearing in plate controls over beam web)				
Vertical edge distance =		3	in	
lc =	2.531	in		
	Φ1.2 lc t Fu =	55.05	kip	
	Φ2.4 dt Fu =	45.68	kip	
	Φr_n =	**45.68**	**kip**	**at top bolt**
	From AISC Manual Table 7-4			
	s=	3	in	
	bolt dia =	7/8	in	
	plate thickne:	0.500	in	
	$\Phi r_{n \text{ per in plate thk}}$ =	91.4	kip per in plate thickness	
	Φr_n =	**45.70**	kip	**at middle bolts**
Bolt bearing at top bot controls				
Eccentrically loaded bolt group condition				
	Vu =	172		
	Φr_n =	45.68	kip	
C_{min} =	Vu / ϕr_n =	**3.77**		
	e_x =	3	in	
	s =	3	in	
	C =	3.9	from table 7-6 AISC Manual	
	OK			

Design data: Web plate 1/2 thick with 5 rows of bolts and 3" of vertical edge distance works. Total length of web plate is 15"

Plate Shear Yield			
Fy =	36	ksi	
A_{gv} =	8.00	in^2	Changed plate length to 16"
Vu =	172	kip	OK
ΦR_n =	**172.8**	kip	
Plate Shear Rupture			
Fu =	58	ksi	
Bolt hole area=	0.5	in^2	
A_{nv} =	6.50	in^2	
ΦR_n =	**226.2**	kip	OK

Block Shear Rupture Strength of the Web Plate			
plate thick =	1/2	in	
Leh =	3	in	
Lev =	3.5	in	
n =	5	# of bolts	
Using AISC Tables 9-3a, 9-3b, 9-3c			
Tabls 9-3a	ΦFu Ant / t =	109	kip/in
	ΦU_{bs} Fu Ant =	**54.5**	kip
Tabls 9-3b	Φ0.6 Fy Agv/t =	338	kip/in
	ΦU_{bs} Fu Ant =	**169.0**	kip
Tabls 9-3c	Fu Ant / t =	274	kip/in
	ΦU_{bs} Fu Ant =	**137.0**	kip
Shear rupture controls			
Use	137.0	for ΦR_n	
ΦR_n =	**191.5**	kip	OK

Web plate to Column Flange Weld Shear Strength			
Vu =	172	kip	
ΦR_n =	1.392 D l 2		
D =	4	sixteenth inches	
l =	16	in	
ΦR_n =	**178.2**	kip	OK

Web plate Rupture strength at Welds (AISC manual using Eq. 9-2)			
D=	4	sixteenth in	
F_{EXX} =	70	ksi	
Fu (column)=	65	ksi	
t_{min} =	**0.190**	in	**OK**
t_{web} =	0.415	in	

Beam flange limit states

Tension flange plate and connection

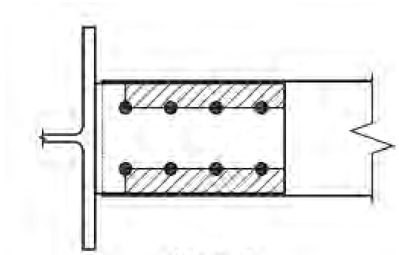
$P_{uf} = M_u/d =$ **452.7** kip

Bolt strength		
Using AISC Table 7-1		
Φr_n =	**40.3**	kip/bolt
Bearing on flange using AISC Table 7-5		
Edge dist=	2	in
t flange =	3/5	in
Φr_n =	89.6	kip per bolt per in thickness
Φr_n =	**52.42**	kip/bolt
Bearing on plate using AISC table 7-5		
Edge dist =	2	in
t plate =	1 5/8	in
Φr_n =	79.9	kip per bolt per in thickness
Φr_n =	**129.84**	kip/bolt
Bolt strength	controls	
Number of bolts required =	12	

Flange plate Tensile yield			
Fy=	50	ksi	
Ag=	14.22	in^2	
ΦR_n =	**640**	kip	**OK**

Flange plate tensile rupture			
Fu=	65	ksi	
An =	10.16	in^2	
Ae =	10.16	in^2	
ΦR_n =	**495.1**	kip	OK

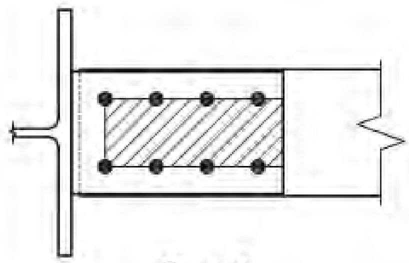
Flange plate Block Shear Rupture



Flange Plate Block Shear Rupture Case 1					
t plate =	1 5/8	in	Bolt dia=	1 1/8	in
Leh =	1.5	in	Gauge length	5.5	in
Lev =	2	in	plate length =	20	in
n =	6	# of bolts			
plate width =	8.75	in			
Fy =	36	ksi			
Fu =	58	ksi			

ΦU_{bs} Fu Ant =	**141.4**	kip	Tension component
0.6Φ Fy Agv =	**473.9**	kip	Shear Yield component
Φ0.6 Fu Ant =	**471.8**	kip	Shear Rupture Component

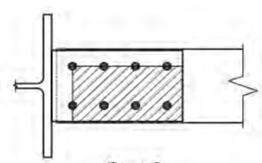
Shear rupture controls			
Use	471.8	for ΦR_n	
ΦR_n =	**613.2**	kip	OK



Flange Plate Block Shear Rupture Case 2

t plate =	1 5/8	in	Bolt dia=	1 1/8	in
Leh =	5.5	in	Gauge length	5.5	in
Lev =	2	in	plate length =	20	in
n =	6	# of bolts			
plate width =	8.5	in			
Fy =	36	ksi			
Fu =	58	ksi			
	ΦU_{bs} Fu Ant =	**300.4**	kip	Tension component	
	0.6Φ Fy Agv =	**947.7**	kip	Shear Yield component	
	Φ0.6 Fu Ant =	**768.7**	kip	Shear Rupture Component	
Shear rupture controls					
Use	768.7	for ΦR_n			
ΦR_n =	**1069.1**	kip	OK		

Flange Plate Block Shear Rupture Case 3



t plate =	1 5/8	in	Bolt dia=	1 1/8	in
Leh =	5.5	in	Gauge length	5.5	in
Lev =	2	in	plate length =	20	in
n =	6	# of bolts			
plate width =	8.5	in			
Fy =	36	ksi			
Fu =	58	ksi			
	ΦU_{bs} Fu Ant =	**711.29**	kip	Tension component	
	0.6Φ Fy Agv =	**292.9**	kip	Shear Yield component	
	Φ0.6 Fu Ant =	**451.5**	kip	Shear Rupture Component	
Shear yield controls					
Use	292.9	for ΦR_n			

ΦR_n =	**1004.2**	kip	OK		

Beam Flange Block Shear

tf =	3/5	in	Bolt dia=	1 1/8	in
Leh =	1.7	in	Gauge length	5.5	in
Lev =	1.5	in	plate length =	20	in
	ΦU_{bs} Fu Ant =	**99.0**	kip	Tension component	
	0.6Φ Fy Agv =	**513.3**	kip	Shear Yield component	
	Φ0.6 Fu Ant =	**432.1**	kip	Shear Rupture Component	
Shear rupture controls					
Use	432.1	for ΦR_n			
ΦR_n =	**531.0**	kip	OK		

Fillet weld to supporting column flange

length of weld = width of plate =

D_{min} =	**12.39**	16ths of inch		
Use	**1**	in	8.75	

Connecting elements rupture strength (size as fillet weld

t_{min} =	**0.003**	in	OK

Compression flange plate and connection

K=	0.65	AISC *Specification Commentary* Table C-A-7.1
L=	2.5	(1/2 in. edge distance and 2 in. setback)
r =	0.469	
KL/r =	**3.464**	<=25
ΦP_n =	**460.7**	kip OK

Panel Zone Shear considering frame stability

$P_{u\ axial\ above}$ =	294.5	kip	Ag =	17	in^2
$P_{u\ axial\ below}$ =	294.5	kip	P_c = FyAg =	850	kip
Pu =	452.7	kip			
$P_{u\ axial}$ =	294.5	kip	Pu <= 0.75Pc	True use equation 1	
V_u =	452.7	kip	Pu > 0.75Pc	False	

tcf =	0.64	in
db	23.7	in
dc	12.2	in
Ag	17	in^2

FyAg =	850	kip	
Fy=	50	ksi	
tf	0.64	in	
dc =	12.2	in	
tw=	0.36	in	
bcf =	10	in	
Equation 1			
ΦR_v =	**147.3**	kip	**Doubler plate required**
Equation 2			
ΦR_v =	**218.6**	kip	**Doubler plate required**

Because of reinforced concrete encasing, doubler plates are not required. The reinforced concrete creates an added stiffness to the connection which eliminates the requirement of doubler plates and additional stiffners.