Thesis Final Report

Redesign of Gravity and Lateral System for Boston Relocation

PricewaterhouseCoopers Building
Oslo, Norway

James Wilson
Structural Option
AE 481W Senior Thesis
The Pennsylvania State University
Faculty Consultant: Professor M. Kevin Parfitt

PricewaterhouseCoopers
Oslo - Norway

PwC building viewed from the Opera House

GENERAL STATISTICS

Building name: PricewaterhouseCoopers (PwC)
Location & Site: Bjørvika B10A, Oslo, Norway
Building Occupant Name: PricewaterhouseCoopers
Occupancy: Office building
Size: 14 000 m2
Stories: 12 stories above grade, 2 stories below
Cost: $50 million - construction cost
Date of completion: November 2008
Project delivery method: DBB with CM agent

ARCHITECTURE

- Spaces include:
    - Cafeteria with outdoor patio *- 12th floor, rooftop*
    - Office/conference rooms *- 3rd - 11th floor*
    - Reception, shops and 154 person auditorium *- 1st & 2nd floor*
    - Technical rooms and parking *- below grade*
- Envelope consists of curtainwall glazing, metal paneling and tar paper roof.
- First building in the "Barcode", which is a concept based on a series of parallel buildings, each with varied volume and materialistic expression.

STRUCTURE

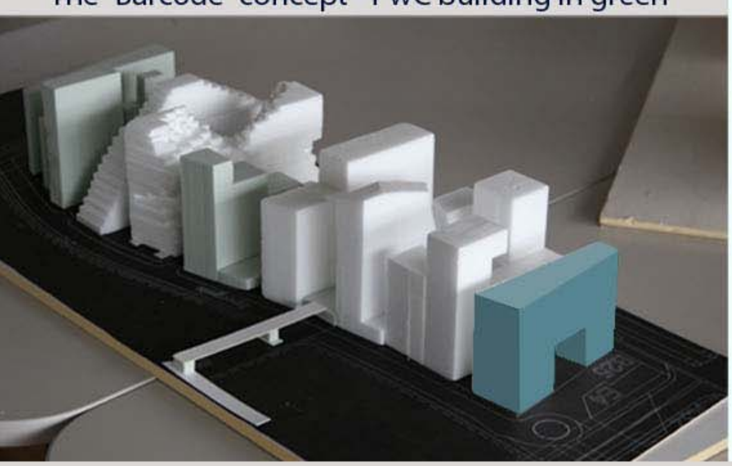
- Precast concrete decking on steel beams and composite columns.
- Lateral resistance provided by reinforced concrete shear walls.
- Sub grade floors comprised of reinforced concrete.
- Pile foundation driven between 100ft and 130ft to bedrock.

M.E.P. SYSTEMS

- Each office/conference room controls VAV with reheat air system.
- Two air handling units per floor.
- District water heating / cooling.
- Building atomization system controls all technical installations.
- 230/400V 3 phase 4 wire system.
- Diesel fuel powered emergency generator.

"Face of the Barcode toward the West"

The "Barcode" concept - PwC building in green



PROJECT TEAM

Developer: Oslo S Utvikling
Architect: A-Lab
Construction Manager: Vedal Prosjekt AS
Structural Engineer: Multiconsult AS
Electrical Engineer: Ingeniør Per Rasmussen
Mechanical Engineer: Erichsen & Horgen AS
Interior Architect: Arktitektene AS / Beate Ellingsen
Fire Safety Consultant: NEAS Brannconsult AS
Geotechnical Consultant: Geovita AS
Ecconomics Consultant: Bygganalyse AS

James Wilson
option - structural

Executive Summary

The PricewaterhouseCoopers (PwC) building is a 12 story office building located in downtown Oslo, Norway. It is the first building to be completed in a two million square foot development known as the “BARCODE”. The superstructure consists of hollowcore concrete plank decking on a steel frame with cast in place concrete shear walls at the core. There is a five story opening at the center of the façade, created by the use of three steel trusses. The cast in place concrete substructure extends two stories below grade and acts as a base to distribute overturning moments to pile foundations.

In the following report the PwC building was hypothetically relocated to Dorchester Avenue in Boston, MA and the structural system was redesigned. The report encompasses a redesign of the gravity and lateral system for the superstructure. Although there were many factors that needed to be considered, the design attempts to balance structural performance, economy and architectural expression. Determining the most suitable structural design for the location of Boston provides a basis for comparison with that of Oslo, from which advantages and disadvantages of each system were brought in to focus.

With the guidance from design professionals the most viable floor system was concluded to be composite concrete deck on composite steel beams and girders. After determining a framing plan that conformed to the architectural layout, decking and steel members were sized in accordance with the applicable design codes. With the use of composite action in the beams and girders it was possible to reduce steel member sizes, thus yielding in a more economic solution and kept structural depth to a minimum. The proposed design resulted in a structural depth of 19.25”, which is 5” deeper than the existing design.

The redesign of the lateral force resisting system was performed using steel as the choice of material. Amongst other reasons, steel was selected because of its compatibility with the steel framing chosen in the redesign of the floor system. Much effort was devoted towards determining a structure that met design criteria. The resulting structure uses concentrically braced chevron frames at the core with moment frames acting as outriggers to perimeter columns. Despite efforts, it was concluded that the design was an uneconomic solution because of the large axial forces in the columns, induced by the narrow aspect ratio of the core. Given more time to explore the use of braced frames in combination with moment frames, a more economic steel structure could likely be determined. If not, the most viable structural system for the PwC building, if hypothetically located in Boston, would be concrete shear walls at the core in combination with the proposed floor system.

Acknowledgements

I would like to thank the following individuals, professors, and firms for their generous support and assistance on this senior thesis:

+ Prof. M. Kevin Parfitt - *Thesis Consultant*
+ Svein Barstad **-** *Multiconsult AS*
+ Ole K. Vedal - *Vedal Prosjekt AS*
+ Bernt Aasenhuus - *Vedal Prosjekt AS*
+ Sven Hegstad - *Pareto Investments*
+ Andreas Solberg - *Contiga AS*
+ Pareto Investments for granting me permission to use the PwC building as the subject of this year long project.
+ All the professors at The Pennsylvania State University for their assistance over the past five years.

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1 – Existing Conditions

1.1 Building Architecture and the *BARCODE* Concept

In 2003 *Oslo S Utvikling* hosted an international architecture competition for the lot located south of the *Oslo S* train lines - between the outrun of *Akerselven* and *Middeladerparken*. The competition was jointly won by *MVRDV*, *Dark Arkitekter*, and *A-lab* with their proposal for the *BARCODE* development. The new *PricewaterhouseCoopers* (PwC) building is the first building to be completed in the *BARCODE* and will be "the face" of the *BARCODE* towards the west.

The *BARCODE* is a concept based on a series of parallel building aligned in a formation that will ensure a lot of air between buildings and provide good views onto and out of the site, says *A-lab* architect Mathias Eckman. The development will contain a row of eight to ten buildings, each with their own individual form and character. They will have to abide by certain formulas and guidelines set forth by the zoning plan that regulates shape, size, function, material use, public spaces, roofing, and entrances. There is a volume guide with specific principle forms that the buildings may take on. Each building must adhere to one of the principle forms and must be completely different from the adjacent buildings. The intention is to provide unique multifunctional architecture with a lot of light, variation and accessibility.

Figure 1: BARCODE Concept

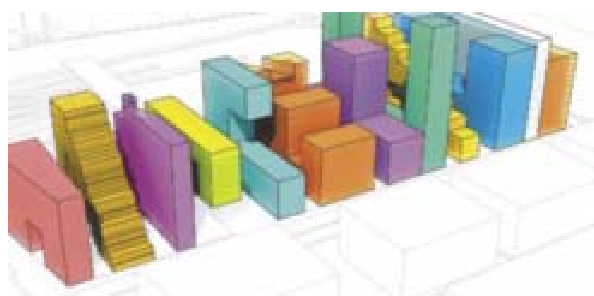


Figure 2: Image BARCODE Concept

- Images courtesy of Oslo S Utvikling

The exterior shape of the PwC building is simple and defined. The east side runs perpendicular to *Nydalen Alle* and the west side follows the property line, creating a rhombus like shape in plan. There are of two stories below grade and twelve stories above grade with a five story opening in the center of the façade, indicating the main entrance. The building envelope consists of curtainwall glazing, metal paneling and tar paper roof, intended to give off an impression of lightness, openness and technological sophistication. The story height is 12 ft which is similar for all the buildings in the *BARCODE* development.

The program inside mainly conforms to the needs of the professional services firm, PricewaterhouseCoopers. Technical rooms and parking are located on sub grade floors. The first three floors above grade contain an auditorium, a reception area, meeting rooms, and towards *Nydalen Alle*, shops and display rooms. The forth through the eleventh floors hold conference and office spaces. A grand cafeteria with spectacular views and outdoor dining options is located on the top floor. The core consists of a permanent technical zone that contains communication, technical installations and wet services, in addition to zones that can be designed differently depending on the need of the different departments.

1.2 Drawings

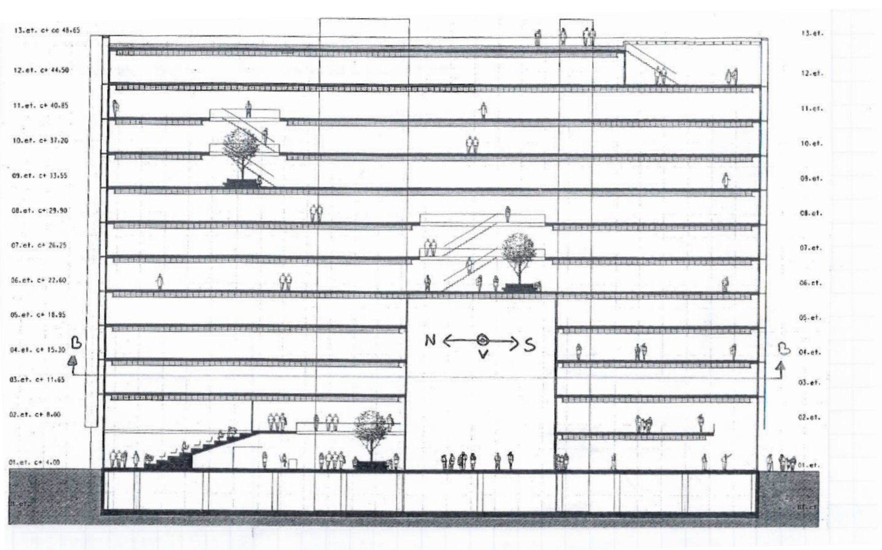


Figure 3: Building Section

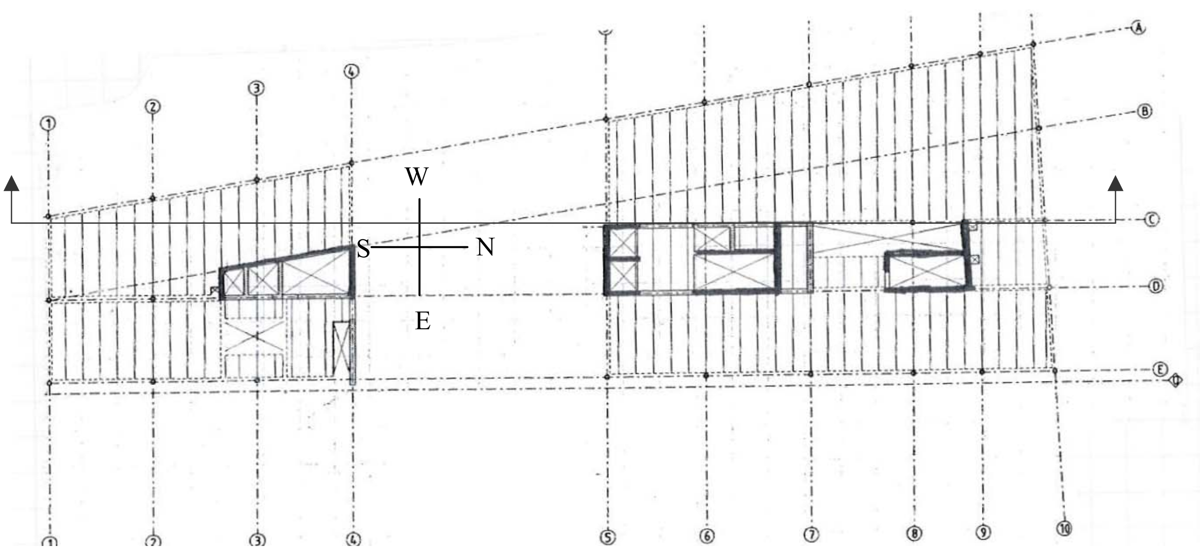


Figure 4: Typical framing plan for floors 1 – 4

1.3 Major Model Codes

+ Life Safety Code
+ Byggforsk
+ Norske Standarder

1.4 Zoning

+ Oslo Kommune S-4187, 16.11.2005, Regulerings bestemmelser for felt B10 i bjørvika
+ Oslo Kommune S-4099, 15.06.2004, Regulerings bestemmelser for Bjørvika - Bispevika - Lohaven

1.5 Mechanical Systems

The mechanical systems are intended to provide high quality indoor climate, while maintaining efficient energy use. The building contains many office and conference rooms under varied use that require premium indoor climate. Therefore flexible and adaptable control systems are implemented. The building developer also requested a solution that would be sustainable and keep energy consumption to a minimum. Some of the systems used to accommodate these criteria are district heating and cooling, a balanced ventilation system and a building automation system.

The building is heated along the perimeter with thin tube, hot water radiators. The radiators mainly account for the heat losses through the envelope of the building. Floors 5 through 11 contain approximately 70 radiators per floor, each radiator with a heating capacity of 600W/h. Further individual temperature adjustment is provided by variable air valve (VAV) with reheat air systems. Each office and conference room controls their own VAV with reheat air system. There are two main air handling units on each floor which supply air to the various spaces. The building is cooled using water provided from the river *Akerselva*. During the colder seasons, freecooling is used, which utilizes air directly from outside. All the buildings technical installations are zone controlled by a web-based building atomization system (BAS). This system regulates HVAC, lighting, electrical, safety and security systems.

Figure 5: Conference room ceiling

Figure 6: Perimeter Radiators

Figure 7: Control Display

The heating and cooling central is currently located in the basement of the building and is intended to be a temporary solution, however, it is capable of being permanent if need be. There are plans to build a central supply for the entire BARCODE district, which the PwC building could eventually take use of.

1.6 Electrical

The buildings electrical system runs on a on a 230/400V 3 phase 4 wire system. If power were to be lost during an emergency, power will be provided by a diesel generator located in the basement. Keeping energy consumption to a minimum was one of the architects and developers design goals; however with the clients' wishes for an all glass façade and high quality indoor climate their goal was achieved only to a certain degree. The buildings overall energy consumption is estimated to be 156 kWh/m2/year. With the growing focus on sustainable design, there will be made greater efforts to reduce energy consumption in the following BARCODE buildings.

1.7 Lighting

Lighting fixtures were chosen on a basis of providing the desired amount of light for intended use, energy consumption, aesthetics, flexibility and economy. Office and hallways are typically lit with suspended direct/indirect compact florescent lighting fixtures. Conference rooms typically use a combination of wallwashers and recessed direct compact florescent lighting. Public areas mainly use recessed circular compact florescent down lights.

Figure 8: Direct/indirect

Figure 9: wall washers

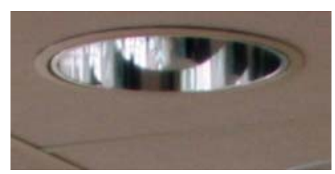


Figure 10: Down lights

1.8 Construction

The project delivery method chosen for the PwC building was design bid build with construction manager as agent. The developer, Oslo S Utvikling, was responsible for design engineers and sub contractors. This delivery method was chosen opposed to a general contractor because the market was strained during initial stages.

2 – Existing Structural System

2.1 Superstructure Floor System

The superstructure of the building consists of precast concrete plank decking on a steel frame with cast in place shear walls at the core. The decking consists of prestressed hollow core concrete plank (figure 11) with typical sections of 120cmx30cm and spans ranging from 10 to 20 meters. Due to the irregular buildings shape, many plank ends are cut at an angle. The decking has a 2" concrete topping which provides rigid diaphragm action to transfer lateral loads to the concrete shear walls.

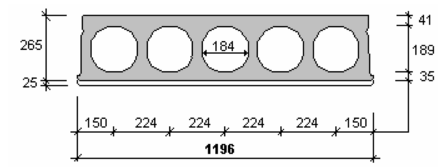


Figure 11: HD 265 hollow core plank

Along the interior of the building, planks typically rest on steel angles fastened to the concrete core (figure 13). Along the exterior, planks typically rest on the bottom flange of a special made steel girder (HSQ profile, figure 12). The girders are fabricated by precast engineer *Contiga AS* and conceal its flange and web within the plane of the slab, creating extremely low structural depth. Connections between beams and deck elements are made with cast in place concrete reinforced with stirrups that loop around shear tabs on the beams (figure 13)

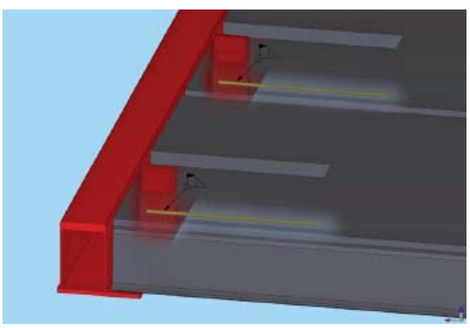


Figure 12: Principle connection of deck elements with one sided HSQ profile steel beam.

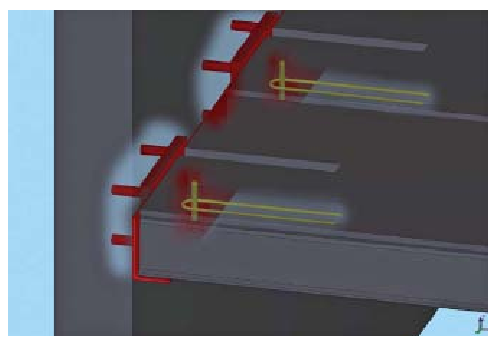


Figure 13: Principle connection of deck elements with interior concrete shear wall.

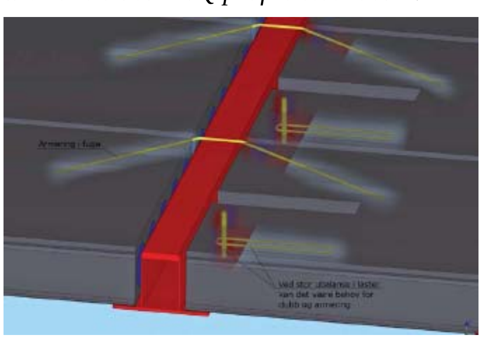


Figure 14: Principle connection of deck element with two sided HSQ beam

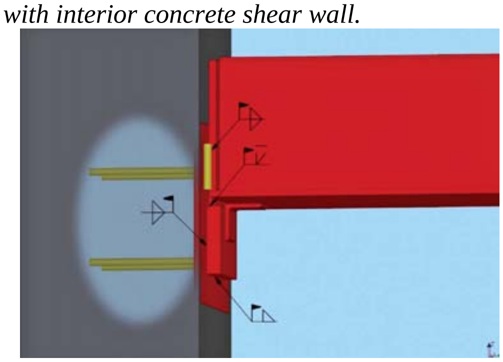


Figure 15: principle connection of steel beam with concrete shear wall

-images courtesy of Norsk Stålforbund and Betongelement Foreningen

2.2 Superstructure Columns

Hollow circular steel columns filled with reinforced concrete support the beams along the perimeter of the building. They are typically spaced at 7.2 m along the perimeter with sizes ranging from Ø406.4mm x 8mm at level 1 to Ø323.9 x 6.3 at level 12. The sequence of erection was to first lift the hollow steel columns into position with a crane and temporarily brace them. At the base, this was followed by welding the columns to steel plates with 6mm fillet welds (figure 17). After the beams were connected to the columns, the columns were grouted.

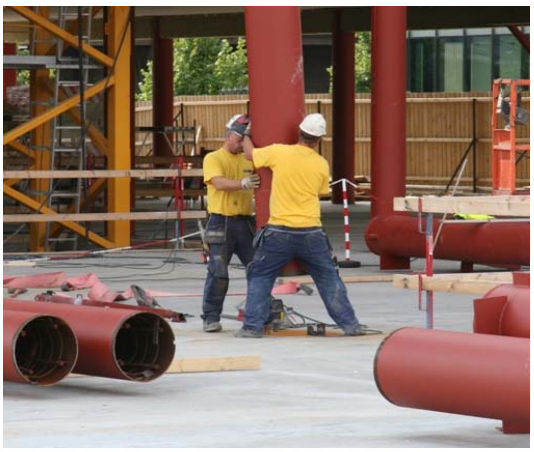


Figure 16: Placing hollow steel column on steel base plate

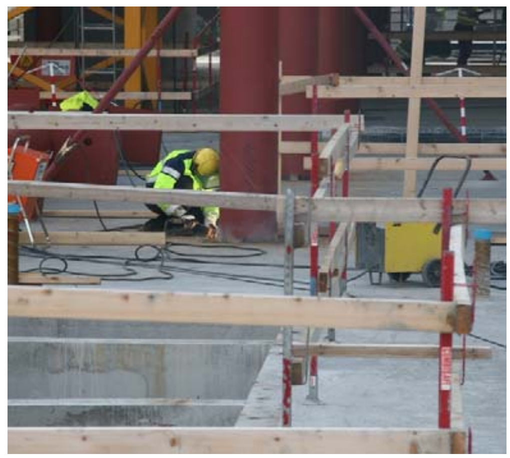


Figure 17: Welding column to steel plate

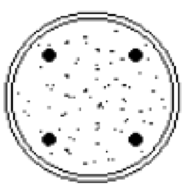


Figure 18: Typical column cross section

According to *Design guide for concrete filled columns* by *Corus UK limited,* advantages to concrete filled structural hollow sections are:

+ They provide architects and engineers with a robust and inherently fire resistant column.
+ During construction the steel sections dispenses with the need for formwork and erection schedule is not depended on concrete curing time.
+ During finishing concrete, filling is protected against mechanical damage.
+ When completed, columns provide greater usable floor area, higher visibility, reduced maintenance, and are aesthetically pleasing

2.3 The Grand Opening

The 5 story grand opening at the center of the façade is created by using three trusses comprised of hollow circular steel tubing for diagonal/vertical members and HSQ profiles for horizontal members (Figure 19,20,21). During construction the structure was supported by three temporary columns that were removed after the integrity of the truss was intact.

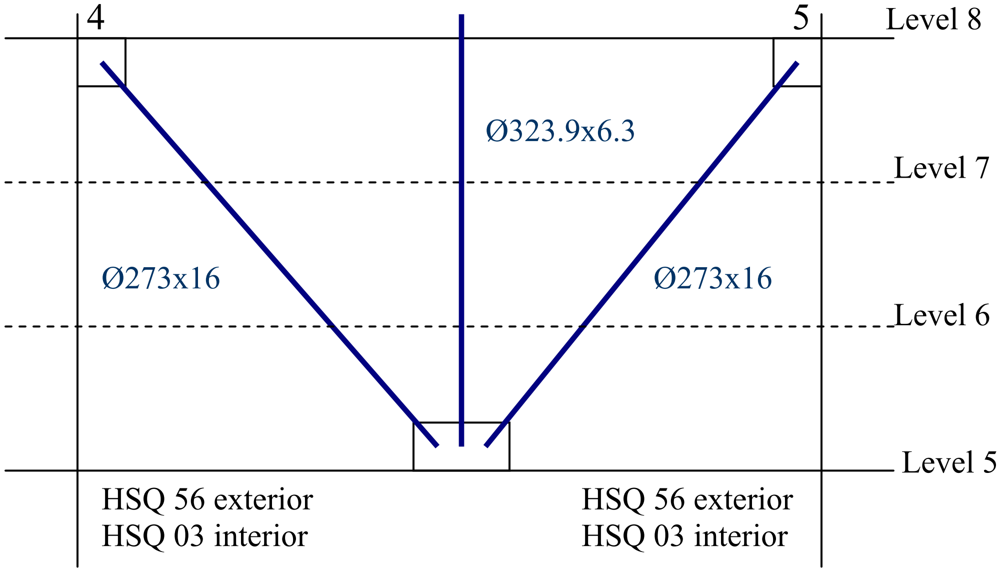


Figure 19: Truss Elevation

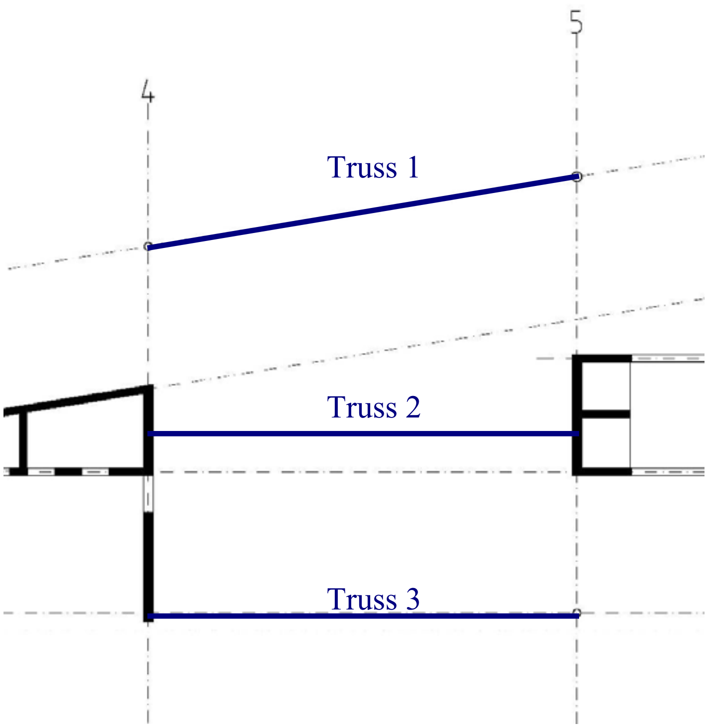


Figure 20: Truss Plan

Figure 21: Truss Images

2.4 Lateral System

Lateral resistance is provided by cast in place concrete shear walls located at the center of each leg of the building. Concrete plank decking acts as a rigid diaphragm that transfers loads to the shear walls. The building is tall and narrow in the short direction and therefore requires thick shear walls. Walls are typically 400mm thick in the short direction and 300mm in the long direction.

The narrow building shape also causes large overturning moments. Shear walls at the core are integrated into a cast in place concrete substructure which acts as a base to distribute the overturning moments to the foundation. The foundation uses steel and concrete piles to transfer axial tension, axial compression and lateral loads to the ground. Piles are driven between 100 and 130ft (30 and 40m) to bedrock.

Material Properties of Concrete used in shear walls:

Item	Norwegian Standard	Eurocode CEN	f_{ck} (ksi)	f_{ctm} (ksi)	E_{cm} (ksi)
Cast in place concrete	B35	C35/45	5	0.46	4 850

f_{ck} - *compressive cylinder strength at 28days*
f_{ctm} - *value of mean axial tensile strength of concrete*
E_{cm} – *Secant modulous of elasticity*

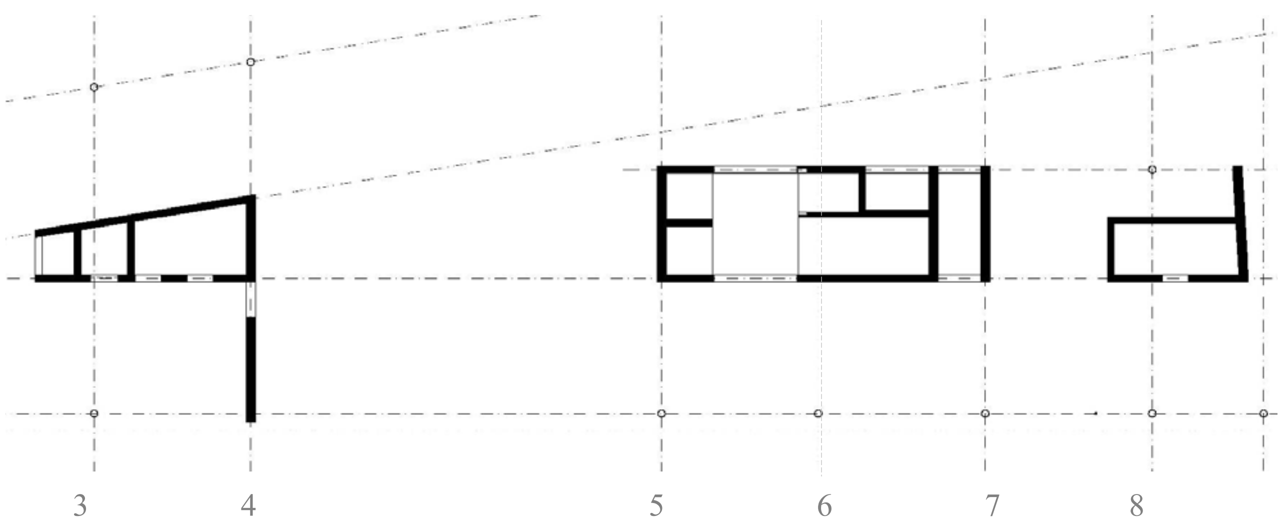


Figure 22: Typical Shear wall layout

2.5 Substructure

There are two stories below grade comprised of cast in place concrete. The lowest level has a slab thickness of 500mm with recessed areas for elevator shafts. Other floor slabs below grade are 300mm thick, with exception of slabs below outdoor areas, where slab thickness is increased to 400mm.

2.6 Foundations

The foundation uses five different types of piles driven between 100 and 130ft (30 and 40m) into bedrock. Pile capacities are dependent on pile type, connection type, and whether bending is about strong or weak axis.

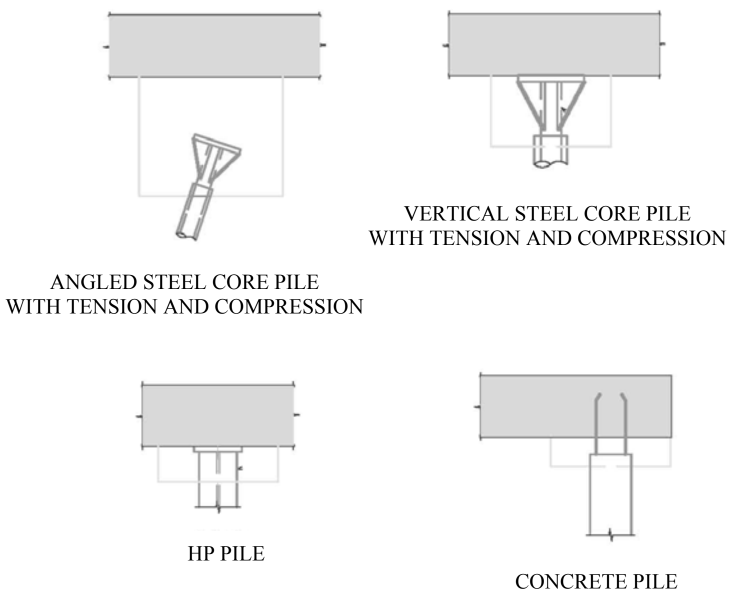


Figure23: Typical pile types

The *BARCODE* development was built in sections. This meant that the PwC building stood complete before the next building to the west had begun. Therefore uneven loads from ground pressure to the west were accounted for in its design.

Figure 24 : Image showing the next building in the BARCODE being constructed.

Figure 25: Excavated BARCODE site

2.7 Steel Materials

Tables below summarize the steel material properties used in the PwC building:

Metric

Item	Euronorm	ASTM	Fy (N/mm^2)	Fu (N/mm^2)	Ea (N/mm^2)	Va	Density (kg/m^3)
Columns	S355	A572Gr50	355	510	210 000	.3	7 850
Beams	S355	A572Gr50	355	510	210 000	.3	7 850
Reinforcing	B500C	-	-	500	210 000	-	-
Piles	HISAR460	*still need to determine*					

Imperial

Item	Euronorm	ASTM	Fy (ksi)	Fu (ksi)	Ea (ksi)	Va	Density (Ib/ft^3)
Columns	S355	A572Gr50	51	74	30 500	.3	50
Beams	S355	A572Gr50	51	74	30 500	.3	50
Reinforcing	B500C	-	-	72	30 500	-	-

Notes
1. Metric densities are converted to imperial form using 1 lb/ ft = 157 kg/m3
2. Metric material strengths are converted to imperial form using 1 psi = .006894 N/mm^2. Values are rounded down to nearest whole number.

2.8 Concrete Materials

Tables below summarize the concrete material properties used in the PwC building:

Metric

Item	Norwegian Standard	Eurocode CEN	f_{ck} (N/mm^2)	f_{ctm} (N/mm^2)	E_{cm} (N/mm^2)
Cast in place	B35	C35/45	35	3.2	33 500
Prefabricated	B45	C45/55	45	3.8	36 000
Columns	B45	C45/55	45	3.8	36 000

Imperial

Item	Norwegian Standard	Eurocode CEN	f_{ck} (ksi)	f_{ctm} (ksi)	E_{cm} (ksi)
Cast in place	B35	C35/45	5	0.46	4 850
Prefabricated	B45	C45/55	6.5	0.55	5 222
Columns	B45	C45/55	6.5	0.55	5 222

f_{ck} - compressive cylinder strength at 28days
f_{ctm} - value of mean axial tensile strength of concrete
E_{cm} – Secant modulous of elasticity

Notes
1. Metric material strengths are converted to imperial form using 1psi = .006894 N/mm^2. Values are rounded down to nearest whole number.

2.9 Codes and Reference Standards:

In the past Norway has operated using national design standards. As part of an effort to decrease trade barriers between EU countries the Eurocodes are currently being developed. The Eurocodes are unified design codes for buildings and civil engineering works for all of Europe. Norway is currently in the transition period where National and Eurocodes coexist. The Norwegian versions of the Eurocodes and the national annexes are still under production and aim to be completed by 2009. According to the time schedule, the transition will period last from year 2008 – 2010, after which national standards will be withdrawn.

The PwC building was designed in accordance with various sections and editions of the Norwegian Standards.

3 – Proposal

3.1 Problem Statement

Studies conducted on the existing conditions (Technical Report 1, 2, 3) determined the existing structural system to be optimal for the location of Oslo, Norway. However, if the PwC building were hypothetically moved to Dorchester Avenue in Boston, MA, it is likely that design and construction methods would change. Determining the most viable structural design for the location of Boston, MA will provide a basis for comparison with that of Oslo. From this study, advantages and disadvantages of each system can be brought in to focus in order to develop better engineering decisions in the future.

Boston was chosen as relocation site in order to limit the number of changed variables. Boston shares similar geographic characteristics to that of Oslo and therefore the redesign will experience similar design loads. However, there are still numerous other factors that dictate the choice of structural system. Some of these are local labor and design expertise, design codes and material availability. The report will not present an analysis of all the factors, but rather determine and present a structural design which is suitable for an office building in the Boston area.

3.2 Site Relocation

To keep the PwC building in context, it was decided to hypothetically move the entire BARCODE development to Dorchester Avenue, Boston, MA (figures 27-30).The existing building design relied heavily on its importance as an entity in the BARCODE as a whole. Therefore, architecturally it would not make sense to have the building as a standalone structure. Images below display a graphical representation of the hypothetical BACODE site if located along Dorchester Ave. The images are courtesy of Google Earth.

Figure 26: Site – Birds Eye View*

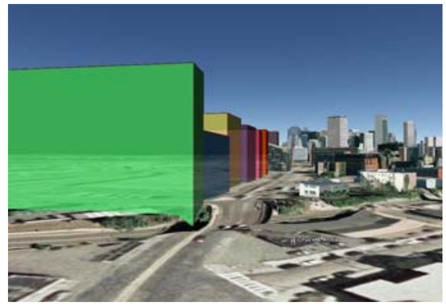


Figure 27: Site Looking North*

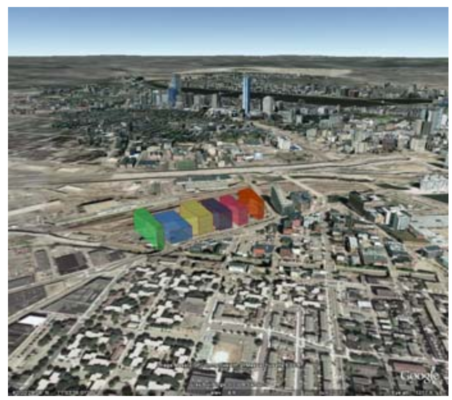


Figure 28: Site Looking West*

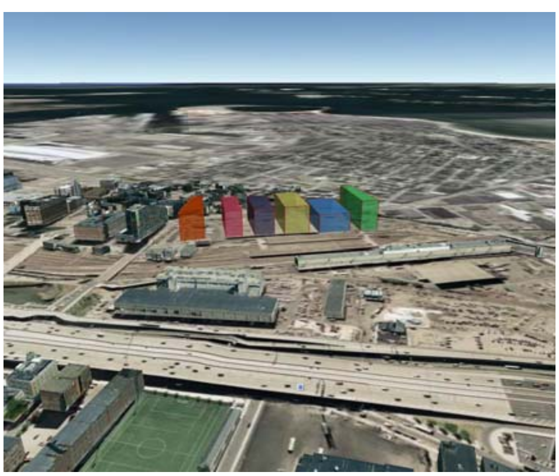


Figure 29: Site – Looking East*

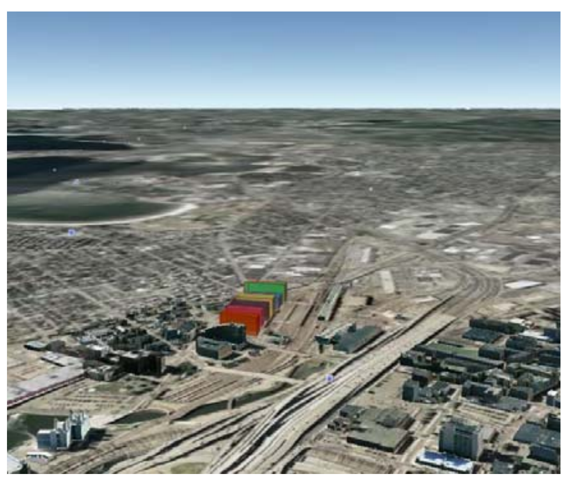


Figure 30: Site – Looking South / West*

3.3 Proposed Studies

This thesis will conduct an in depth study on composite concrete decking on a composite steel frame as an alternative to the existing gravity system. A change in gravity system and design loads consequently incurs a redesign of the lateral system. Alterations to construction cost, scheduling, and architecture, will also be addressed in this thesis.

3.3.2 Depth Studies:

Composite concrete deck on a steel frame will be studied as an alternative to the existing gravity system because the local labor expertise in Boston, MA potentially makes this a more economical solution. Important features shared in both existing and proposed structural solutions are low structural weight on foundations and ability to provide flexible floor layout for tennats/occupants. A proposed framing layout will be determined and modeled in RAM Structural System Steel Module, from which trial members will be determined. Hand calculations will be used to verify the determined results.

A redesign of the lateral system will be required due to different design loads incurred by change of site location and structural weight. Alternative steel solutions will be explored at a schematic level, however if a reasonable alternative cannot be found then a shear wall system at the core will be used. Technical report three determined that the existing lateral system experiences considerable torsion under both wind and seismic loads. The redesign will also explore methods of minimizing torsional effects, although this is not an easy task given the non-symmetrical layout. The proposed lateral system will be modeled in ETABS from which trial members can be determined.

3.3.2 Breadth Studies:

Speed of construction is important, because the PwC building must be completed before successive buildings in the BARCODE strip can be continued. Although an all steel structure is faster to erect than an all concrete solution, it will not be as fast as the existing prefabricated structure. A comparison study will be conducted whether the savings made by change in structure are outweighed by increased construction time. Determination of cost and schedule of the new structural system will be estimated using RS Means 2009. A sequencing schedule will also be conducted in Microsoft Project. Although values obtained will not provide for direct comparison with existing conditions, it will provide an indication as to whether the proposed design can provide cost savings.

A change in the structural system will potentially incur changes to the façade and floor plans. Any alterations made to the façade will attempt to keep the existing architectural expression in tact. The goal is to keep the simple defined form and maintain an expression of transparency and technological sophistication. The importance of the PwC building as a unique entity in the BARCODE strip as a whole is also critical. The rules and regulations defined by zoning will have to be studied, such that any alterations conform within the guidelines.

3.4 MAE Requirements

As required by the MAE program, this thesis will incorporate material from a graduate level class. I have chosen to incorporate material from AE 597A – *Computer Modeling of Building Structures*. This will be done through modeling the lateral system of the building using ETABS as structural modeling program. Below are guidelines provided by course instructor that will be followed to meet the MAE requirements.

Guidelines provided by instructor:

Develop a computer model of the lateral-force-resisting system and determine member demands due to Earthquake and/or Wind forces based on the permitted analytical procedures of the applicable building code.

The model shall represent the floor as a rigid or semi-rigid diaphragm. Structural walls and semi-rigid diaphragms shall be modeled with area elements. Beams and columns shall be modeled with line elements representing a 3-D frame element. Both the area and line elements shall account for flexural, shear, and axial deformations.

Where a 3-D building model is used with rigid floor diaphragms, a minimum of three degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure.

Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

The lateral force analysis shall consider inherent torsion, accidental torsion, and P-Delta effects. The story forces shall be distributed to the various vertical elements of the lateral-force-resisting system based on the relative lateral stiffness of the vertical resisting elements and the floor diaphragm.

- Guidelines provided by Dr. Andreas Lepage, The Pennsylvania State University

4 – Structural Depth

4.1 Introduction

This following section addresses the structural redesign of the PwC building. The redesign began with research on steel construction typical for the Boston area. Through this research it was confirmed that composite concrete deck on steel beams and girders is the preferred floor system for office buildings in the Boston area. Design loads for the relocated site were determined in accordance with ASCE 7-05. The most viable gravity and lateral systems were determined and designed for the relocated site.

Gravity system

A design using composite concrete deck on a steel frame was conducted for the PwC building. Decking was designed in accordance with the CMC Joist & Deck Design Manual. Optimal sizes for steel members supporting the deck were determined aided by a 3-D finite element program RAM Structural Systems. Members were spot checked by hand for strength and serviceability criteria.

Lateral system

A design using steel braced frames and rigid frames for lateral force resistance was conducted for the PwC building. Optimal member sizes were determined aided finite element program ETABS. Optimal members were checked for strength criteria under combined loading using RAM Structural Systems.

4.2 Goals and Criteria

Personal Thesis Goals:

- Learn about Norwegian building design through studying existing conditions of the PwC building.
- Learn about US steel design through relocating the PwC building to Boston, MA and redesigning the structural system.

Depth Study Design Goals:

- Determine loads on the structure in accordance with ASCE 7 – 05 for Boston location.
- Determine and design the most viable gravity system:
  - Study typical structural design for the Boston area.
  - Provide a framing layout that conforms to existing architectural plans.
  - Design beams, columns and girders with the help of RAM structural systems
  - Determine whether to use composite or non-composite beams and girders.
  - Verify design obtained from RAM by performing spot checks for strength criteria.
- Determine and design the most viable steel lateral system:
  - Determine a framing layout for a steel lateral system that conforms or enhances existing architectural layout.
  - Aided by structural modeling in ETABS, design a steel lateral system that effectively resists design loads and meets serviceability criteria.
  - Study the effects various parameters on fundamental period of the building in order to obtain a more efficient structural design.
  - Verify structural model of the lateral system created in ETABS by comparison with RAM model.

4.3 Hollow Core Concrete Plank vs. Composite Concrete Deck

Research into typical construction for the Boston area was conducted in order to confirm initial ideas presented in the *proposal solution* (section 3.3) of this report. Through an online discussion board hosted by the Architectural Engineering Department to aid students working on senior thesis, Robert McNamera provided information on floor systems typical for steel buildings in the Boston area. He informed me that “the hollow core plank system is not commonly used in Boston for office buildings. The hollow core plank system has been used in the metropolitan area for housing and the girder slab system is being marketed aggressively for housing as an alternate. Office buildings in the Boston area are generally constructed of composite lightweight concrete slabs on metal deck with composite steel beams and girders

The hollow core system is composed of pre stressed plank which limit flexibility for future alterations, and limit the ability to use composite steel beams and girders which makes for a deeper and more expensive floor. Boston is a mild seismic zone so the diaphragm action of the plank is a problem and would require a topping (typically 2") and the floor is usually heavier than the alternate metal deck and concrete system creating larger seismic lateral loads.

The selection is usually based mainly on economic factors but the lack of flexibility for future modifications and possible cutting of the pre stressed strands for tenant work is a concern. To get maximum economy from the plank system one wants to span as far as the specified depth will allow and this will result in a thicker floor and ultimately larger floor to floor heights than the metal deck system again resulting in higher costs.”[1]

As far as McNamara knew, “the plank system has been used on several steel frames in the Boston area but all of those projects were housing uses where the plank can offer another advantage by using the underside of the plank for a base for a sprayed ceiling. Of course this option doesn't work for the office use with the need for a mechanical duct plenum ” [1] .

[1] Information provided by Robert McNamara, McNamara/Salvia Inc, through AE Senior Thesis e-Studio – Structural Mentors discussion board/listserv.

4.4 – Design Loads

4.4.1 Gravity Design Loads

The following provides a summary of the gravity loads used for analysis in the structural redesign. Loads were largely determined in accordance with ASCE 7-05. To give the owner/tenant flexibility to place lightweight partitions and corridors anywhere, the floor load was designed for 80psf live load. Roof live and snow loads were conservatively assumed to be a uniformly distributed load of 100psf.

Dead Loads:

Material / Occupancy	Reference	Load
Light Weight Concrete	ACI 318	115pcf
Steel	AISC 13th ed.	Per Shape
Steel Deck	USD	2 psf
Façade	Design Value	15psf (180plf)
Floor, Ceiling, M.E.P		15psf

Live Loads:

Area	Reference	Unit Weight (psf)
Office spaces	ASCE 7 – 05, Table 4-1	50
Lobbies and first floor corridor	ASCE 7 – 05, Table 4-1	100
Corridors above first floor	ASCE 7 – 05, Table 4-1	80
Cafeteria	ASCE 7 – 05, Table 4-1	100
Partitions	ASCE 7 – 05, Table 4-1	15
Outdoor terrace	ASCE 7 – 05, Table 4-1	100
Auditorium	ASCE 7 – 05, Table 4-1	60

Snow Loads:

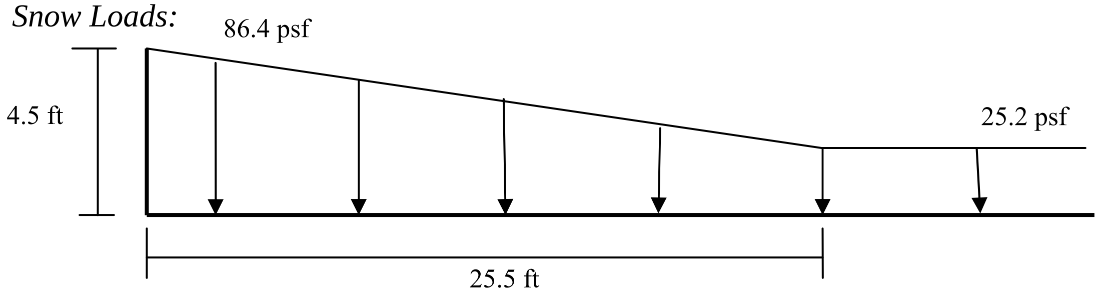


Figure 31: Snow Load at Parapet Walls - North /South Faces

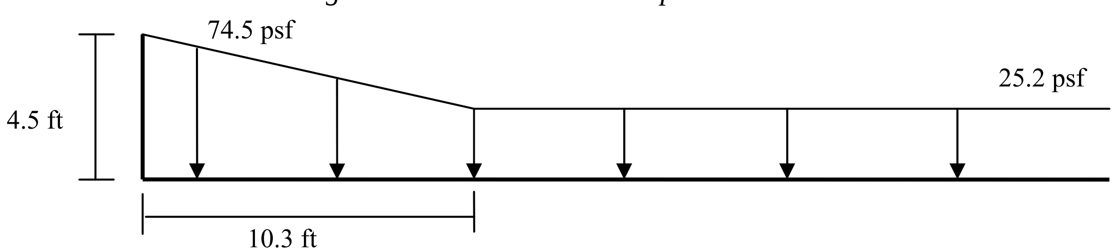
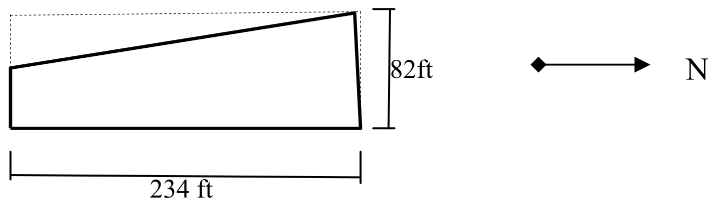


Figure 32: Snow Load at Parapet Walls - East /West Faces

4.4.2 Wind Loads

Wind loads were determined in accordance with the analytical procedure described in ASCE 7-05. For simplification purposes, calculation of wind pressures assumed the building to be a rectangular box using maximum dimensions (figure33).



Fiure33: simplified dimensions for wind pressure calculations

Wind pressures can automatically be calculated by RAM through entering location and site specific parameters. This was used to verify calculated design loads. From reviewing output reports of story base shears calculated by RAM, it was revealed that RAM calculated more conservative wind pressures. Although an in depth study was not conducted, it was determined from reviewing output, that RAM assumed a conservative external pressure coefficient, Cp, of -0.5. This coefficient accounts for the suction on the leeward side based on a ratio of base to length. As RAM was used to check the members for strength in the redesign of the lateral system, it was decided to be consistent and conservative, and use design loads with a Cp coefficient of -0.5.

ASCE 7-05 Calculation Summary: Wind Pressures - East / West Winds:

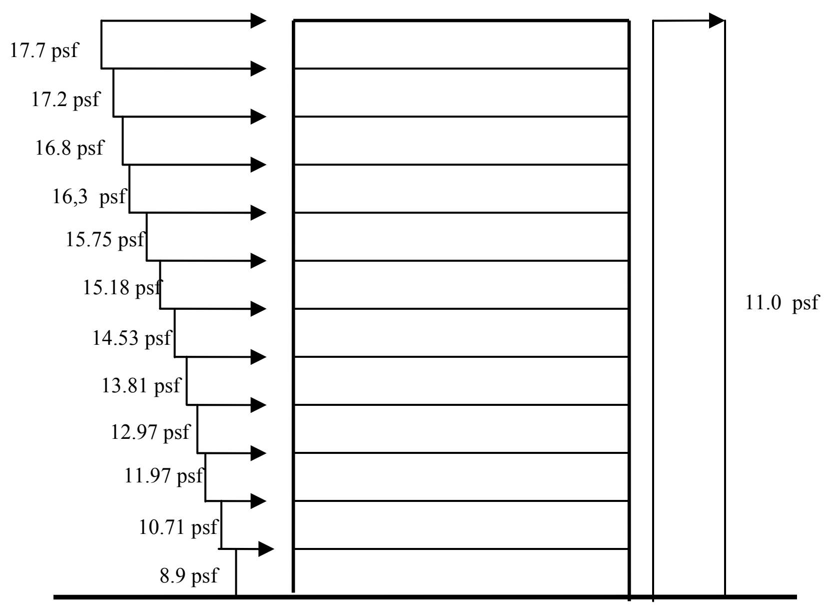


Figure 34: Pressure summary from East / West Winds (ASCE 7-05, Analytical Procedure)
For information on calculations see Appendix B

ASCE7-05 Calculation Summary: Wind Pressures - North / South Winds:

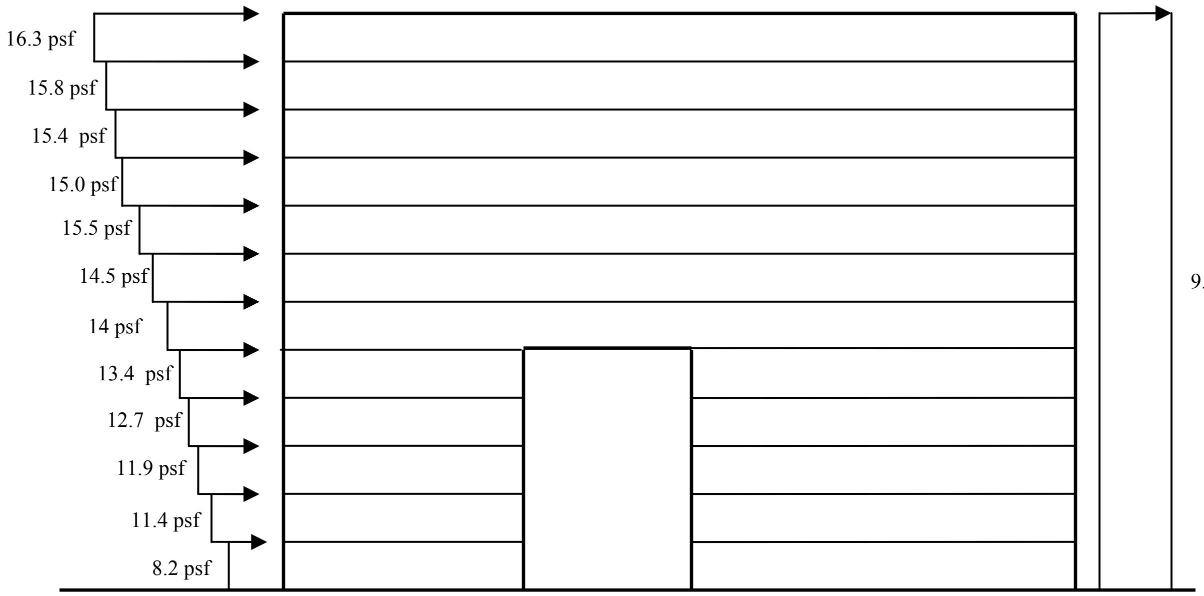


Figure 35: Pressure summary from North /South Winds (ASCE 7-05, Analytical Procedure)
For information on calculations see Appendix B

4.4.3 Seismic Loads

The following provides a summary of the seismic loads used for analysis of the lateral system redesign. Loads were determined in accordance with the ASCE 7-05 Analytical Procedure.

According to ASCE 7 – 05 the building is experiencing the following irregularities:

	12.3-1 Horizontal Structural Irregularities	
	Irregularity	**Must Comply with Reference Section:**
1a	Torsional Irregularity $\Delta 1$ (in.) = 1.68 $\Delta 2$ (in.) = 2.70 → $1.2((\Delta 1 + \Delta 2)/2) = 2.63 < \Delta 2$	12.7.3 16.2.2
3	Diaphragm Discontinuity Irregularity → Slit diaphragm at the bottom four stories	*12.7.3* *16.2.2*
5	Nonparallel Systems-Irregularity → Vertical lateral force resisting elements are not parallel or symmetric about major orthogonal axes.	*12.7.3* *16.2.2*

It was required by ASCE7-05 to perform a Modal Response Spectrum Analysis for the building specified in Section 12.9, however for simplification purposes of this thesis seismic loads were determined in accordance with the provisions of the Analytical Procedure, section 12.8. Load calculations were performed for both Ordinary Concentrically Brace Frames (OCBF) and Special Concentrically Braced Frames (SCBF)

Example Calculation 1 - Base Shear calculations for OCBF and SCBF:

SCBF Base Shear	
N/W and E/W Direction	
R	6
Cd	5
Ct	0.02
hn	147
x	0.75
Cu	1.6

Ta = ct*hn^x	0.84
T = Cu*Ta	1.35
Cs = Min:	
SDS / (R/I)	0.075
SD1 / (T(R/I))	0.020
(SD1*TL)/((T^2)*(R/I))	0.087

Weight	11176
Vb = Cs*W	219

OCBF Base Shear	
N/W and E/W Direction	
R	3.25
Cd	3.25
Ct	0.02
hn	147
x	0.75
Cu	1.6

Ta = ct*hn^x	0.84
T = Cu*Ta	1.35
Cs = Min:	
SDS / (R/I)	0.138
SD1 / (T(R/I))	**0.036**
(SD1*TL)/((T^2)*(R/I))	0.161

Weight	11176
Vb = Cs*W	404

Example calculation 2 - Story Force Distribution for R = 3.25 (OCBF):

T= 1.350 s
k= 1.425
V_b= 404 kips

Split forces on diapragm according to mass
% Mass North Leg = 0.32
% Mass South Leg = 0.68

Seismic Loads in North / South Direction - Ordinary braced frames																
i	h_i (ft)	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)		V_i (kips)	By (ft)		5%By (ft)		Ax		M_z (k-ft)	
Roof	12	144	1035	1231974	0.187	75		75	234		12		1.06		936	
12	12	132	1035	1088311	0.165	67		142	234		12		1.06		827	
11	12	120	1035	950098	0.144	58		200	234		12		1.00		681	
10	12	108	1035	817643	0.124	50		250	234		12		1.00		586	
9	12	96	1035	691308	0.105	42		293	234		12		1.00		495	
8	12	84	1035	571522	0.087	35		328	234		12		1.13		463	
7	12	72	1035	458811	0.070	28		356	234		12		1.39		457	
6	12	60	1035	353835	0.054	22		377	234		12		1.50		380	
Split Diaphragm						S	N		S	N	S	N	S	N	S	N
5	12	48	800	199060	0.030	3.9	8.3	390	74	106	4	5.3	1.0	1.15	15	50
4	12	36	800	132113	0.020	2.6	5.5	398	74	106	4	5.3	1.1	1.41	10	41
3	12	24	800	74134	0.011	1.5	3.1	402	74	106	4	5.3	1.0	1.46	5	24
2	12	12	800	27609	0.004	0.5	1.1	404	74	106	4	5.3	1.0	0.89	2	5
		Σ	11481	6596416		404										

Example Calculation 3 - Amplification Factor for R = 3.25 (OCBF):

Amplification Factor in the East-West Direction (Y Dir)														
i	h_i (ft)	h (ft)	δA (in.)		δB (in.)		δavg. (in.)		δmax (in.)		(δmax / (1.2*δavg))^2		Ax (1< Ax < 3)	
Roof	12	144	1.68		2.70		2.19		2.70		1.06		1.06	
12	12	132	1.50		2.42		1.96		2.42		1.06		1.06	
11	12	120	1.31		1.80		1.55		1.80		0.93		1.00	
10	12	108	1.10		1.50		1.30		1.50		0.92		1.00	
9	12	96	0.90		1.20		1.05		1.20		0.91		1.00	
8	12	84	0.70		1.24		0.97		1.24		1.13		1.13	
7	12	72	0.50		1.21		0.86		1.21		1.39		1.39	
6	12	60	0.34		0.94		0.64		0.94		1.50		1.50	
Diaphragm Splits			S	N	S	N	S	N	S	N	S	N	S	N
5	12	48	0.24	0.39	0.37	0.70	0.30	0.55	0.37	0.70	1.03	1.15	1.03	1.15
4	12	36	0.15	0.19	0.24	0.47	0.20	0.33	0.24	0.47	1.05	1.41	1.05	1.41
3	12	24	0.08	0.12	0.12	0.31	0.10	0.22	0.12	0.31	1.01	1.46	1.46	1.46
2	12	12	0.03	0.05	0.04	0.07	0.03	0.06	0.04	0.07	1.02	0.89	1.02	1.00

4.5 – Gravity System Redesign

4.5.1 Framing Plan

The structural redesign began with determining an initial framing plan. It was possible to use the existing locations for almost all perimeter columns and girders. The main effects of the redesign were replacing concrete planks with composite deck supported by steel beams and girders. It was decided to span the beams in the North-West direction in order to locate deeper girder members along the perimeter and towards the core, thus minimizing interruption of MEP and partition layout.

A beam spacing of 2.4m was a logical choice because much of the architectural plans lie on grids spaced at 7.2m (23.3ft), which was easily divided into three equal spans. A true redesign for the Boston area would present a framing plan using dimensions in practical fractions of imperial units. However, the structural framing layout was kept in logical metric fractions to conform to the existing the architectural layout. All grids modeled in computer programs were created in metric units. To much convenience, RAM and ETABS allowed switching between metric and imperial units depending on preferred operation or input. Therefore there was little need for manual conversions when using these programs.

Schematic first floor framing plan:

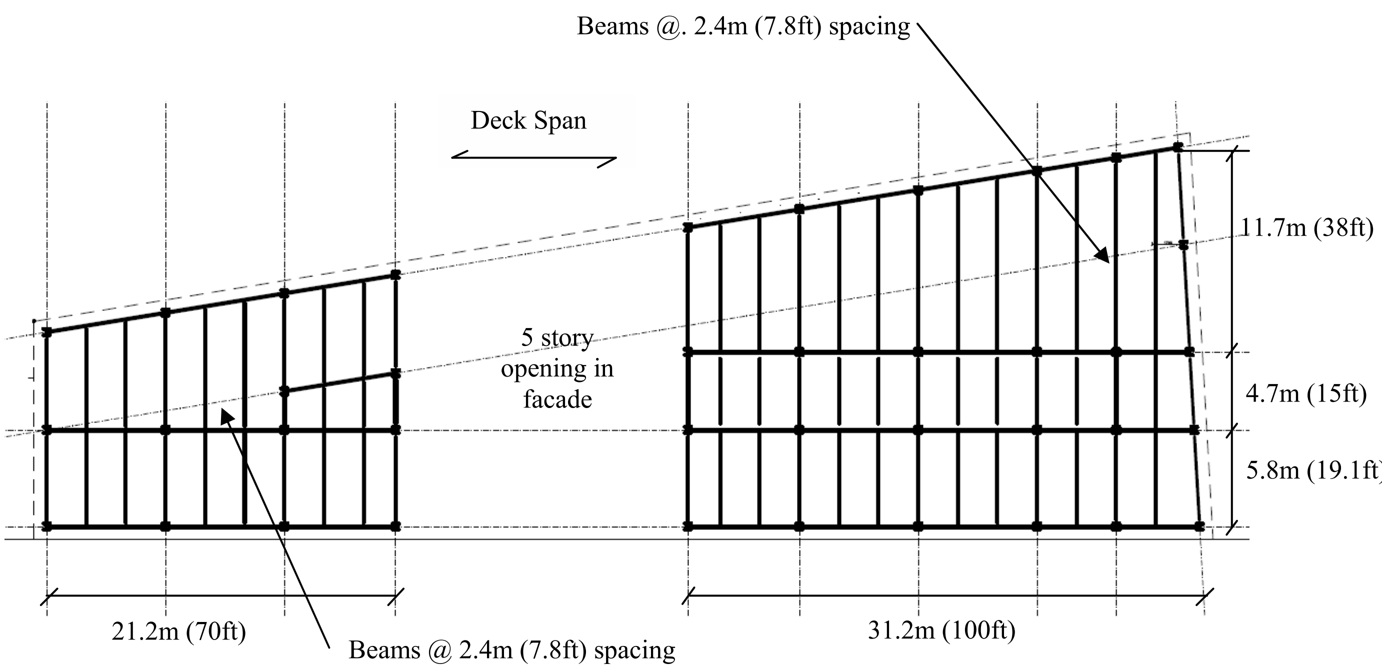


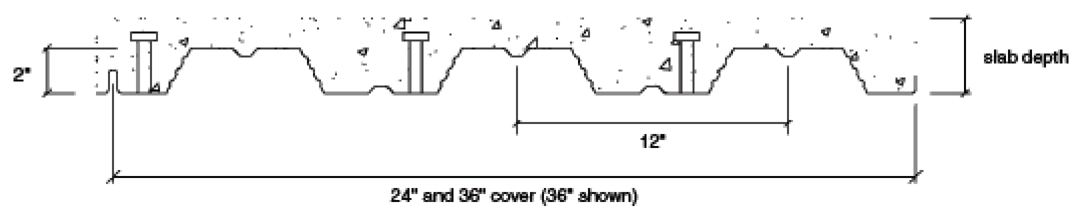
Figure36: Schematic first floor framing plan

4.5.2 Deck Design

The next step in the design process was determining the composite deck and concrete slab. Using the CMC Joist & Deck Design Manual, the thinnest result was a 3.25" lightweight concrete slab on *2" LOK-FLOOR* composite deck. The concrete topping provides a 2 hour fire rating without the need for fire protection.

Deck specifications summarized below were assembled from the CMC Joist & Deck Design Manual and Catalog of Steel Deck Products:

2" LOK-FLOOR 2"" x 12" deck F_y = 40 ksi f'_c = 3 ksi 115 pcf concrete



DECK PROPERTIES										
Gage	t	w	A_s	I_p	S_p	S_n	ϕR_{be}	ϕR_{bi}	ϕV_n	studs
20	0.0358	1.8	0.540	0.390	0.332	0.345	800	1360	2930	0.62

COMPOSITE PROPERTIES												
Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max Unshored Span, ft. 1 span	2 span	3 span	A_{wwf} in²/ft
5.25	71.55	40.0	0.354	41	1.52	7.4	52.00	5590	7.31	9.48	9.80	0.029

Slab Depth	ϕM_n in.k	Span "L" feet, Uniform Live Unfactored Service Loads, psf												
		6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
5.25	71.55	400	400	400	400	400	380	335	300	265	240	215	195	175

RESTRAINED ASSEMBLY RATINGS (HOURLY)			
2 (continued on page 65)			
U.L Design No	F.P.	Concrete Cover and Type	CMC Joist & Deck Product
D922	N	3 1/4" LW	LF2,LFC2,LF3,LFC3,NL,NLC

Product Codes:
LF2 = 2" LOK floor

Q_n (kips) for Studs in Steel Deck - LRFD*															
Profile	W_r in.	Concrete Density pcf	Studs per Corr.	3/4" ϕ Shear Stud's Avg. Nominal Shear Strength/Stud in a Deck Corrugation						1/2" ϕ Shear Stud's Avg. Nominal Shear Strength/Stud in a Deck Corrugation					
				Perpendicular to Beam			Parallel to Beam			Perpendicular to Beam			Parallel to Beam		
				f'c Concrete Compressive Strength, ksi											
				3.0	3.5	4.0	3.0	3.5	4.0	3.0	3.5	4.0	3.0	3.5	4.0
LOK FLOOR 1.5", 2", 3"	6	145	1	17.2			21.0	21.5	21.5	7.7			9.4	9.6	9.6
			2	16.5						7.3					
			3	13.1						5.8					
Inverted B-LOK	3.875	115	1	17.2			17.7	19.8	21.5	7.7			7.9	8.8	9.6
			2	16.5						7.2	7.3	7.3			
			3	13.1						5.8					

Welded Wire Fabric used in this manual**					
Conventional (USA)	Metric (International)	Wire Area (A_s)		Wire Diameter	
		in²/ft	mm²/m	inches	mm
6 x 6 - W2.0 x W2.0	152 x 152 - MW12.9 x MW12.9	0.040	84.7	0.160	4.05

CMC Joist & Deck

4.5.3 Beam and Girder Design

Once the deck was sized, the supporting steel framing members could be designed for the given loads. Beams and girders were sized in accordance with Load and Resistance Factor Design (LRFD) methods and the AISC Thirteenth Edition Steel Construction Manual. In accordance with ASCE7-05 sec 2.3 loads were multiplied by a load factor that incorporates both the likelihood of the loads occurring simultaneously at their maximum level and the margins against which failure if the structure is measured[2]. A 3D structural model of the gravity system was constructed in RAM Structural Systems as a design aid to efficiently determine optimal member sizes. It was chosen to use RAM because it is known to be a reliable and user friendly design aid for steel structures.

Member sizes obtained through RAM were spot checked with hand calculations for strength and serviceability criteria. In all cases optimal member sizes determined by hand calculations matched those determined through RAM.

An important design consideration was whether or not to use composite action between steel beams and concrete deck through the use of shear studs. There were a number of factors considered, which ultimately led to the decision to use a composite system. One of the decisive advantages of composite action is the reduction in steel member sizes and therefore structural depth. The girder slab system of the existing structure allows a structural depth of 14”. Therefore it was important to minimize structural depth in the redesign to keep the architectural features the same. With the use of composite action, the depth of all girders and beams were limited 14”, yielding an overall structural deck and frame sandwich of 19.25”

A second consideration was economy. A cost comparison, on basis of steel weight, was made on four selective beams between composite and non composite action. Composite members were assigned an additional 10 lbs per shear stud. The 10 lbs does not account for the actual weight of a shear stud, but is rather a comparison value that accounts for the cost and implementation of a single shear stud. In all four of the selected members the equivalent weight of the composite beam was less than that of a non-composite beam.

Member	Span (ft)	Composite			Non Composite		Most ecconomical by equiv. wt
		Least Wt. Mem.	# Studs	Equivalent Wt.	Least Wt. Mem.	Wt	
Typical Int. Beam	19.14	W12x14	8	348	W12x19	364	Composite
Typical Ext. Girder	23.6	W14x22	12	639	W14x30	708	Composite
Long span beam	38.5	W14x53	23	2271	W14x68	2618	Composite
Long Span Ext. Girder	23.9	W14x30	22	937	W14x43	1028	Composite

An inefficiency discovered in the composite design was the low composite action found in many members. This was mainly due to serviceability criteria under precomposite conditions. A precomposite deflection criterion was set to l/360. Amongst

[2] Geshwinder, Lewis F., Unified Design of Steel Structures, John Wiley & Sons, 2008

others, the reason for setting this criteria is to reduce the amount of additional concrete the contractor needs to pour under deflection of the beams. There are three ways to solve this; shoring, camber, and increasing member size. Shoring is not favorable because it comes with an increase of cost, schedule and chance of error. Cambering of beams typically becomes economical if spans are larger than 25ft. Most of the typical bays in the PwC building are less than 25ft. Therefore the design resulted in increased member sizes with a lower composite action.

4.6.4 Gravity Design Criteria:

Strength – ASCE 7-05 sec2.3 LRFD load combinations:

Load Combinations:

1. 1.4 Dead
2. 1.2 Dead + 1.6Live + 0.5 Roof Live
3. 1.2 Dead + 1.6 Roof Live + 0.5 Live

Serviceability - Deflection:

Composite:
- Construction Dead Load…………l/360
- Post Composite Live Load……....l/360
- Post Composite Superimposed ….l/240
- Net Total Load…………………...l/240

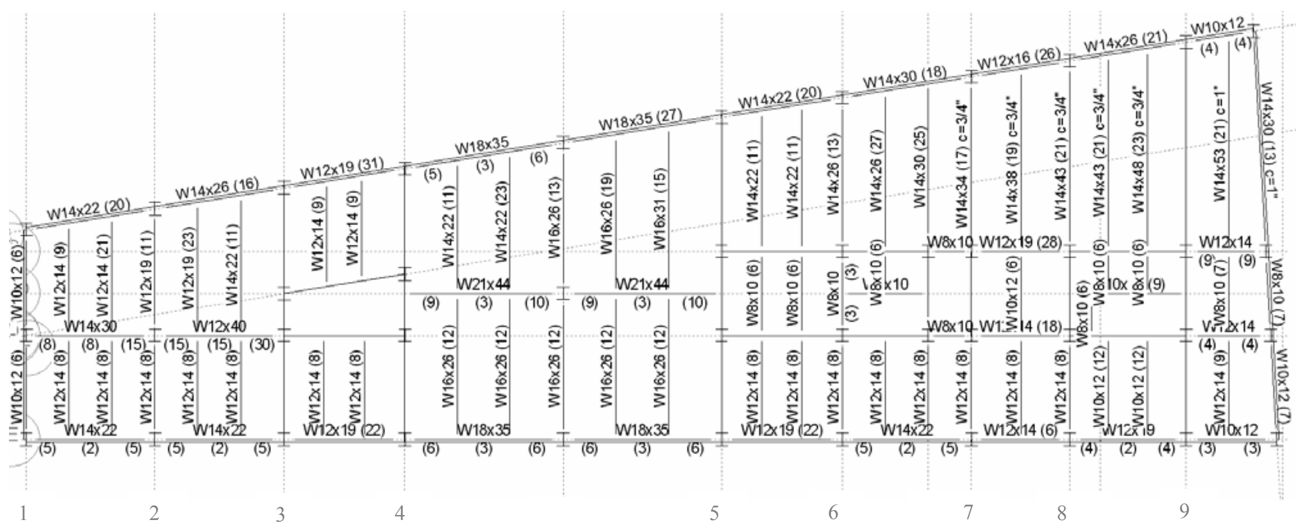
Non Composite:
- Dead Load………………………..l/360
- Live Load……... ………………..l/360
- Net Total Load…………………...l/240

Economy – Camber[3]

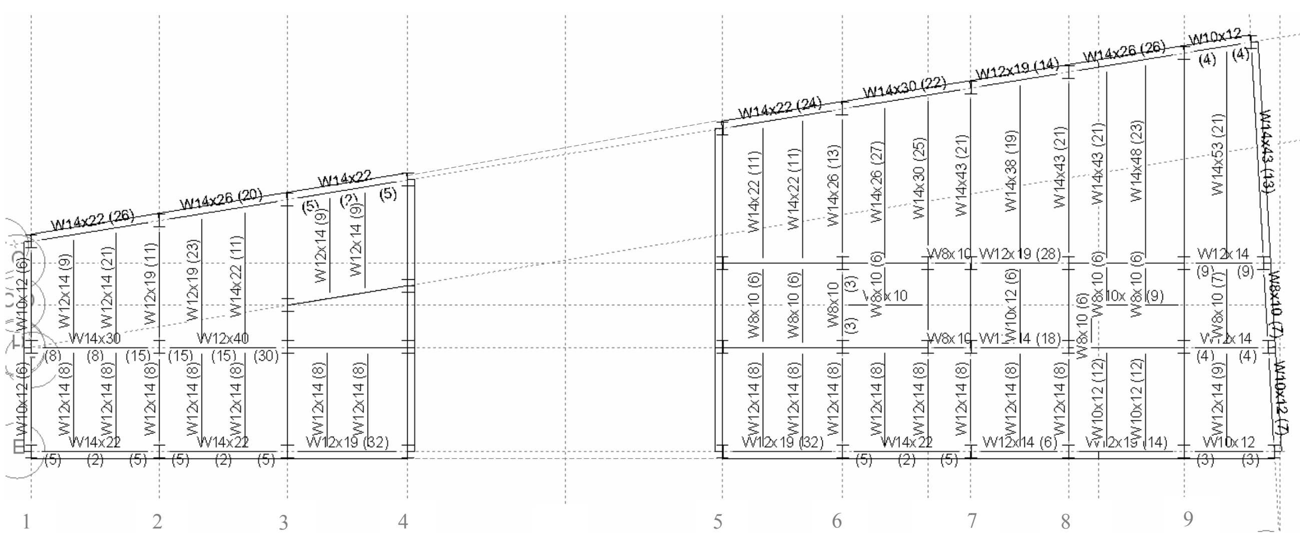
Do not camber: Beams less than 25ft
Beams that require less than 3/4” of camber
Beams in braced frames

[3] Dr. Lewis F Geschwinder, presentation slides on steel beam camber

4.5.5 Drawings – Gravity System Final Design



Fiure37: Typical Framing Plan Level 5-12



Fiure38: Typical Framing Plan Level 1-4

4.5.7 Column Design

The design process of the steel columns followed the same procedures as the beams and girders. Optimal column designs were obtained by RAM and select members were spot checked with hand calculations. It was decided to splice columns every two stories. For simplification of calculations, optimal member sizes were only determined every four stories. Columns were decided to be all W12's around the perimeter (resisting gravity loads only) and W14's at the core (resisting lateral and gravity loads). In retrospect, W12 members were oversized at top levels, even with the smallest W12 member selected. The design could rather have used W10's at top levels.

The table below summarizes calculations conducted by hand and by RAM:

Column E-6 Spot Check					
	Floor	Pu	KL (ft)	Least Wt. Mem.	PhiPn
Hand Calc.	1-4	166	12	W12x40	328
	5-8	311	12	W12x40	328
	9-Roof	451	12	W12x53	547
RAM	1-4	155	12	W12x40	328
	5-8	287	12	W12x40	328
	9-Roof	429	12	W12x53	547

Example Design- 4rth floor for Columns (Columns resisting gravity loads only are labeled. Columns of the lateral system are discussed in following section 4.6)

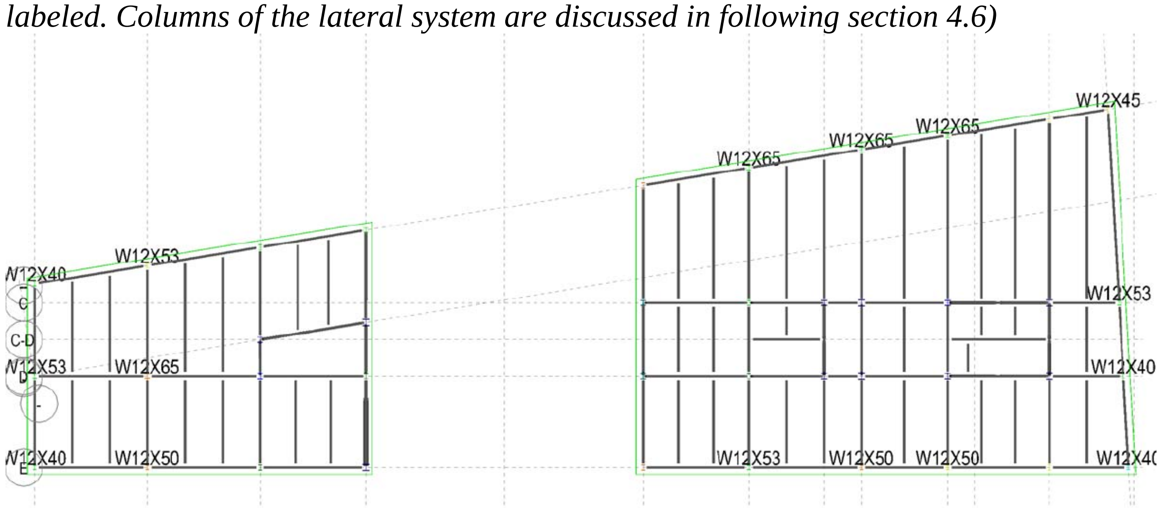


figure 39: 4rth floor column plan
Columns not labeled are part of the lateral design and presented in sec 4.7.10

4.6 – Lateral System Redesign

4.6.1 Introduction

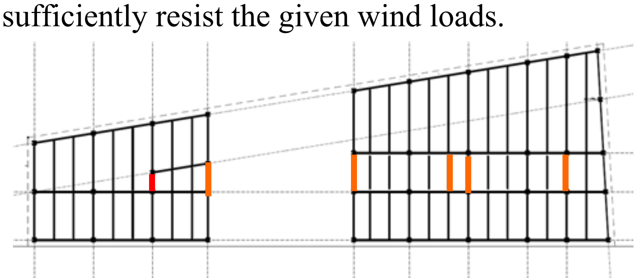
This following section discusses the redesign and analysis process of the lateral force resisting system. As discussed in the proposal it was decided to explore the use of steel as material for lateral force resistance as opposed to concrete. It was initially hypothesized that steel could potentially be a good alternative because of it's compatibility with the steel beams and girders used in floor system redesign. On the basis of architectural implications studied in "*section 5.Architecture*" of this report, it was decided to conceal braced frames within the core of the building and not expose them to the perimeter. To determine an efficient design, countless of hours were spent modeling framing variations. It was discovered that that it was very difficult to come up with a rigid, efficient and economic framing plan with the use of only braced frames at the core. Ultimately, it was discovered that the favorable performance of using rigid and braced frames combined, would provide more efficient resistance to lateral loading. The final design presented this report still has largely oversized columns towards the base due to the large axial tension and compression forces induced by the narrow core size in the East – West direction. Although an efficient design was not achieved in this study the use of steel is not completely dismissed. There are potentially good solutions which take use of rigid frames in combination with braced frames; however, due to limited time to complete this thesis an opportunity to further explore alternatives was not permitted.

4.6.2 Schematic Design

Schematic design began with estimating how many braced frames would be needed in the structure. By inspection it was assumed that deflection in the North/West direction would be critical. It was determined that there could be approximately 5.5 braced frames of 19ft length in that direction at the core (figure 40). A single braced frame was modeled in ETABS and wind loads were applied to the frame at each level (Figure 41). This study initially indicated that the braced frames at the core would sufficiently resist the given wind loads.

Braces = HSS10x10x.5
Beams = W18x86
Columns = W14x132
Deflection = 26.396in

26.396 / 5.5 = 4.8in
H/400 = 4.38in



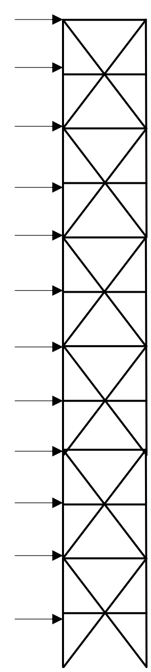


figure 40,41: Schematic Design

The initial framing plan of the lateral system attempted to mimic the existing concrete design by creating, where possible, square tube sections at the core (figure 42). This configuration was chosen because it did not impede the existing architectural layout and allowed connecting braced frames to contribute with out of plane forces, thus making the structure more rigid.

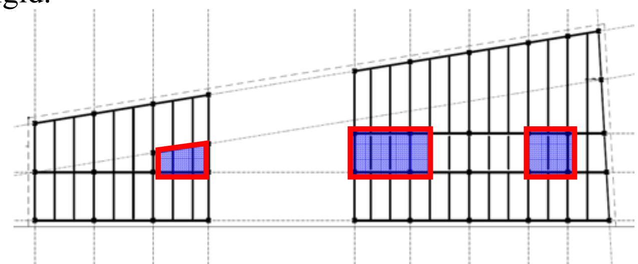
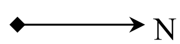


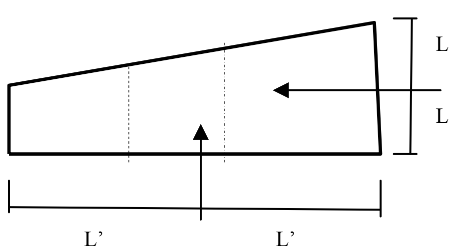
Figure 42: Schematic Design - Braced Frame Location

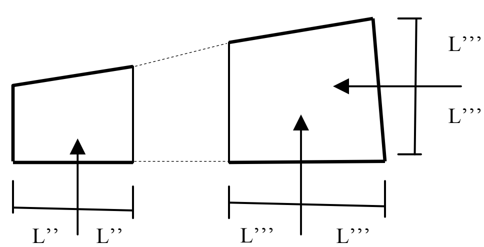
4.6.3 Modeling in ETABS

Schematic design was followed by creating a 3D structural model of the initial framing plan in ETABS. The model was simplified to only include members of the superstructure that provide lateral force resistance. This was done in order easily obtain direct analysis and minimize source of errors incurred with modeling gravity and substructure elements. The diaphragms were modeled as perfectly rigid such that the applied point loads would be distributed according to relative stiffness of the braced frames. Framing members were assigned properties of zero mass and a distributed mass including self weight and superimposed dead loads were also applied to the diaphragms. For simplicity, façade, beam and column loads were considered to be evenly distributed across the diaphragm at each story. Dead loads of girders, beams and columns were determined from takeoffs generated by RAM.

Lateral loads were manually applied as point loads to the rigid diaphragms. Wind loads at each level were applied to the centers of pressure at each story (figure 43, 44). Story forces were calculated by multiplying tributary area of façade by the wind pressures.

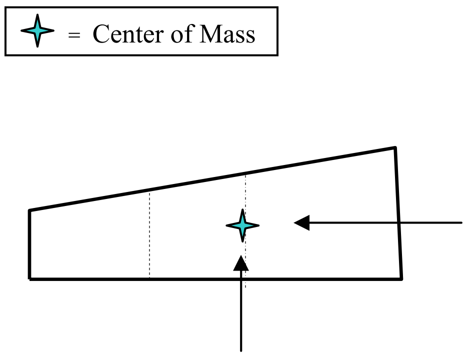


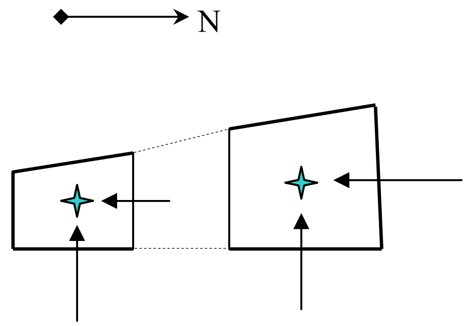




Fiure43: application of wind loads level 5-12 *Figure44: application of wind loads level 1-4*

Seismic loads were applied to the structure at centers of mass (figure45, 46).





Fiure45: application of seismic loads level 5-12 *Figure46: application of seismic loads level 1-4*

Since the model did not include the substructure, there are aspects of the lateral system that are not addressed in this thesis study and would need further investigation. Firstly, where the flexible braced frames meet the comparatively rigid concrete substructure, shear reversals will occur (figure 47). Secondly, the columns were modeled as pin connections at the base to a perfectly rigid substructure. Although the sub structure is relatively rigid, there will be increased deflections due to flexible properties of the substructure. Thirdly, the distribution of forces to the foundations will not be able to be fully assessed without modeling the irregular shaped substructure.

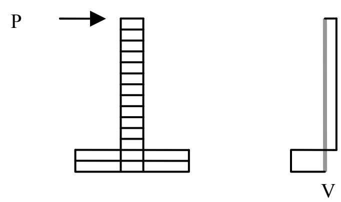


Figure 47: Concept of shear reversals

4.6.4 Effects of Various Parameters on the Fundamental Natural Period of the Structure

Initial trial designs of steel braced frames at the core modeled in ETABS yielded building fundamental natural periods (FNP) significantly longer than what was initially expected. Therefore it was decided to perform an investigation on different parameters effecting the fundamental period. Trial designs yielded FNP's in the range of 5-6 seconds. Based on the ASCE7-05 seismic design provisions, the FNP for steel structures with height of 146ft can be estimated to 1.35s. From historical data steel structures are estimated to have a FNP of N/10, 1.2 seconds. The FNP of a building depends on its mass and lateral stiffness. More flexible structures will experience lower seismic loads, however, also yield larger deformations[4]. The investigation on which parameters affected the building most was performed by changing single groups of members at a time and recording the change in the first mode FNP (see appendix A.6).

From this process it was concluded that that columns at gridlines 3 and 9 largely affect the FNP (figure 48a). This was likely due to the considerable torsion incurred by the irregular shape of the building as well as the narrow aspect ratio of the core. Although intuitive, it was also confirmed that columns at the lower levels contribute more to stiffness than upper levels, where overturning moments are smaller and axial forces in columns are smaller. It was also discovered that chevron bracing yielded a lower fundamental period than cross bracing and therefore this was used in the design.

Critical Column Members Critical Column Members

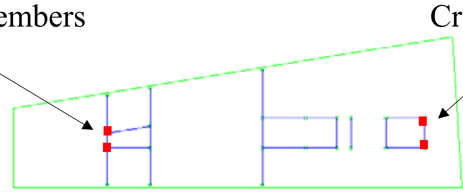


Figure 48a: Critical Memners

[4] Dr Pandya B N, Effects of Various Parameters on Fundemenal Natural Period of Reinforced Concrete Space Framed Structure< IE Jounral-CVVol 86 August 2006

4.6.5 Connections

Structures are designed to respond elastically during an earthquake in order to keep the gravity load carrying capacity in tact. The preferred braced frame design strategy, is to ensure that plastic deformations only occur in the braces, leaving the columns, beams, and connections undamaged, thus allowing the structure to survive a strong earthquakes without losing gravity-load resistance[5]. As connections dissipate energy in different ways, two types of connections were considered in the braced frame design; Ordinary Steel Concentrically Braced Frames (OCBF) and Special Steel Concentrically Braced Frames (SCBF).

The major performance difference between the two is the substantially improved deformation capacity of SCBS's due to more stringent detailing requirements. ASCE7-05 accounts for this larger energy dissipation capacity by allowing the use a higher Response Modification factor, R. This lowers the seismic design forces on the structure by a factor of 1.5 when compared to OCBF's. This can be a favorable design option, however the more stringent requirements come at an additional cost.

Modeling the two alternatives in ETABS revealed that OCBF's are a more economical solution for the given design due to the wind load predominantly controlling member sizes. Lateral loading conditions for SBFS (R = 3.25) and OCBFS (R = 6.0) were created and applied to two separate models with the same framing configuration. Each of the two models also included wind loads. Then, with the use of the "Steel Design" function in ETABS, optimal member sizes were automatically generated by ETABS based on strength and serviceability criteria. The result yielded similar member sizes despite one model experiencing almost half the seismic load. This indicated that member sizes were largely controlled by wind loads, and there was a small margin of return for the use of SCBF.

[5] Naeim, Seismic design of steel structures

4.6.6 Final Design Process

The final design was achieved by determining optimal member sizes in ETABS and checking for strength in RAM. Optimal members based on strength and drift criteria were generated by ETABS through using the "*steel design*" tool. This was done by assigning members to *Auto Select* groups and letting ETABS iterate the design until optimal members were selected. Columns were assigned to an Auto Select group containing only W14 members, girders were assigned W18's and braces HSS members. The deflection limit was set to H/400 under wind loading at the most South – East point of the diaphragm where initial studies had determined wind deflection to be the greatest.

Through the optimization and strength check, it was discovered that the columns at the base of the braced frame core were experiencing very large axial forces caused by lateral loads. Strength criteria required the members to be much larger under lateral loads than that required for gravity loading only, indicating an inefficient design. Therefore an extensive and time-consuming process of modeling different framing configurations was performed in order to determine a more efficient design.

Ultimately, it was discovered through research, guidance by thesis consultant[6], as well as trial and error that the favorable performance of using rigid and braced frames combined, provided more efficient resistance to lateral loading. The use of moment frames branching out on either side, also known as outriggers, considerably improved the drift performance of the building. "The basic structural response of outriggers is quite simple. When subjected to lateral loads, the column-restrained outriggers resist rotation of the core, causing lateral deflections and moments in the core to be smaller than if the freestanding core alone resisted the loading" [7]. Lateral resistance is provided by axial tension in the windward exterior columns and compression in the leeward exterior columns.

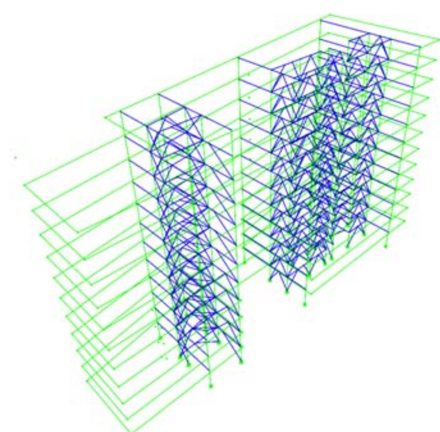


Figure 48b: ETABS Model

[6] Parfitt, Kevin, Senior Theisis Advisor, The Pennsylvania State University

[7] Taranath Bungale S, Wind and Earthquake resistant buildings structural analysis and design, CRC Press, 2004

The benefit of outriggers on the design was explored by creating a model of a single frame in ETABS and comparing the two alternatives under the same loading conditions (figure 49, 50). This revealed a drift reduction by a factor of almost 3. Through reading literature[6] it was found that this favorable reduction in drift is due to the different deflection characteristics of braced and rigid frames.

<u>Members</u>
Braces = HSS10x10x.5
Beams = W18x86
Columns = W14x132

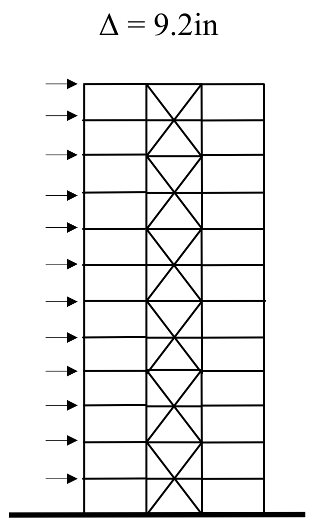


Figure 49: Braced Frame w. outriggers

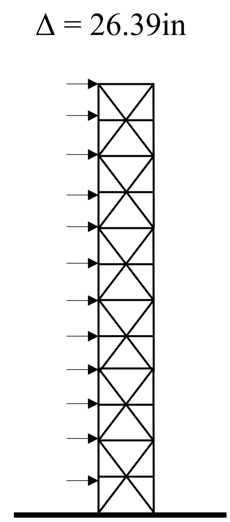


Figure 50: Braced Frame

Once the final framing plan and member sizes were achieved by ETABS they were updated into RAM frame to perform an integrated strength check. This was done to efficiently check all the strength capacities for combined loading and to update members that did not pass. The ETABS model was not used for this because gravity loads were not input into the ETABS model and therefore the atomized "design tool" would not account for combined loading.

Members were sized by RAM in accordance with the following ASCE 7-05 sec 2.3 LRFD load combinations:

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

Members were checked by RAM for the interaction equation H1-1a and H1-1b presented in the AISC design manual. Members were also checked with hand calculations using member forces obtained from the RAM model.

4.6.8 Seismic Drift

Seismic drifts at the center of mass were designed to be within the limits presented in ASCE7-05 section 12.8. All seismic drifts were within the allowable limit.

$$\delta_x = \frac{C_d\,\delta_{xe}}{I} \qquad (12.8\text{-}15)$$

$$\Delta_a = 0.020h_{sx} \qquad 12.12\text{-}1$$

Cd = 3.25 for OCBF's

Example calculation of permitted drift caused by seismic loading in the East/West dir:

Drift Calculation in East-West Direction												
i	h_i (ft)	h (ft)	Disp. of COM (in.)		δxe (in.)		δx (in.)		Δa (in.)		Result	
Roof	12	144	1.75		0.19		0.62		2.88		OK	
12	12	132	1.56		0.20		0.65		2.88		OK	
11	12	120	1.36		0.21		0.68		2.88		OK	
10	12	108	1.15		0.19		0.62		2.88		OK	
9	12	96	0.96		0.19		0.62		2.88		OK	
8	12	84	0.77		0.17		0.55		2.88		OK	
7	12	72	0.60		0.13		0.42		2.88		OK	
6	12	60	0.47		0.01		0.03		2.88		OK	
Split Diaphragm			N	S	S	N	S	N	S	N	S	N
5	12	48	0.47	0.45	0.14	0.14	0.46	0.46	2.88	2.88	OK	OK
4	12	36	0.33	0.31	0.14	0.13	0.46	0.42	2.88	2.88	OK	OK
3	12	24	0.19	0.18	0.11	0.11	0.36	0.34	2.88	2.88	OK	OK
2	12	12	0.08	0.08	0.08	0.08	0.26	0.24	2.88	2.88	OK	OK

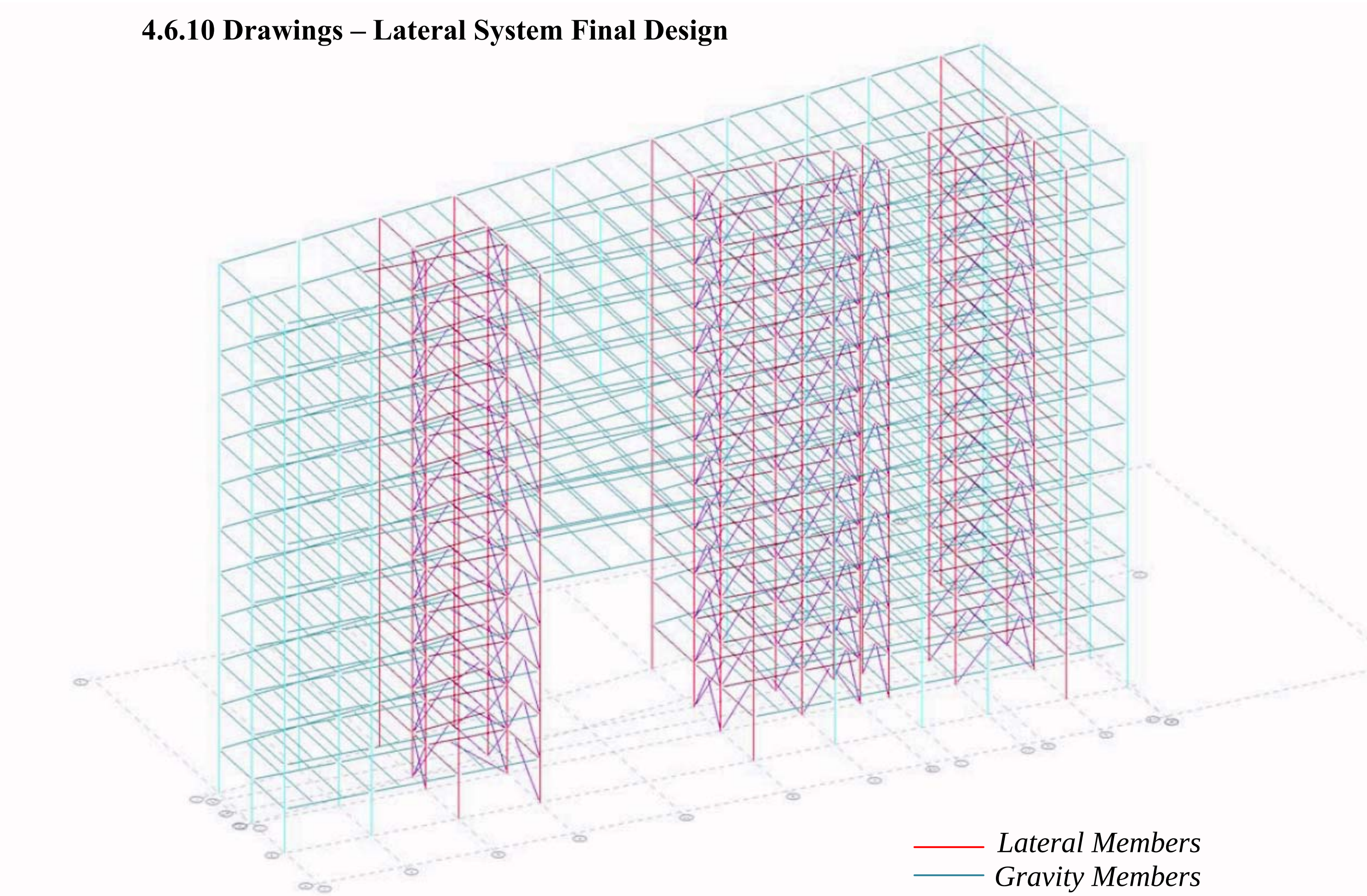
Figurex example calculation for drift in the East-West Direction

4.6.9 Wind Drift

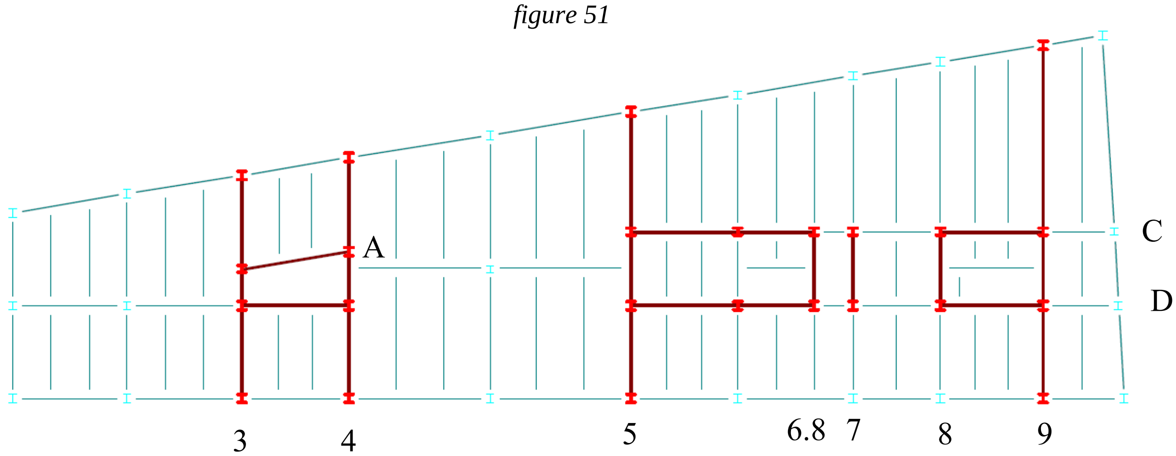
Maximum building drifts at extreme points of the diaphragm were designed to be within a limit of H/400. (H = building height in in)

Max wind deflection – Loading in Y Dir = 3.66in < H/400 = (146x12) / 400 = 4.32in **OK**
Max wind deflection – Loading in X Dir = 1.03in < H/400 = (146x12) / 400 = 4.32in **OK**

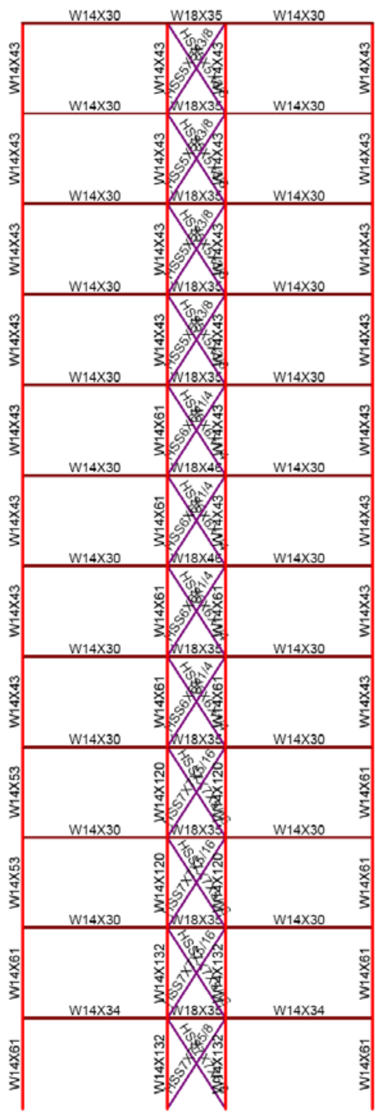
4.6.10 Drawings – Lateral System Final Design



3D View
figure 51



Framing Plans
figure 52

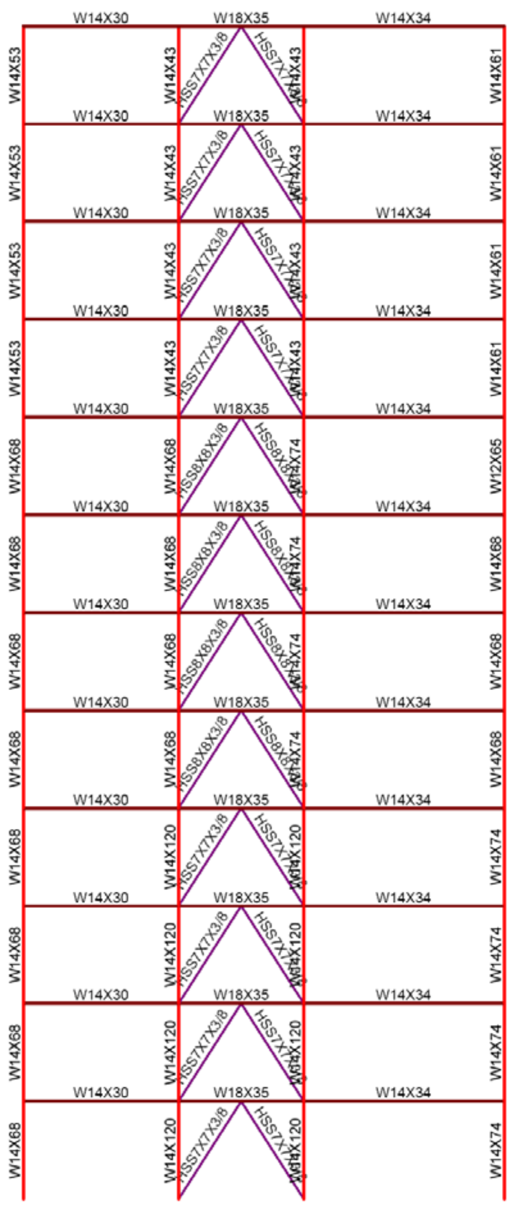


Elevation 3

Elevation 53

Elevation 4

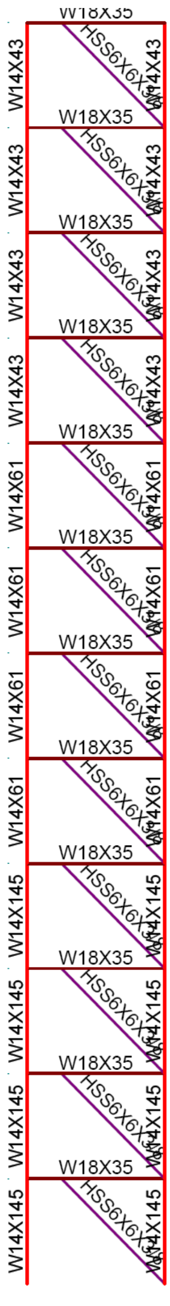
figure 54



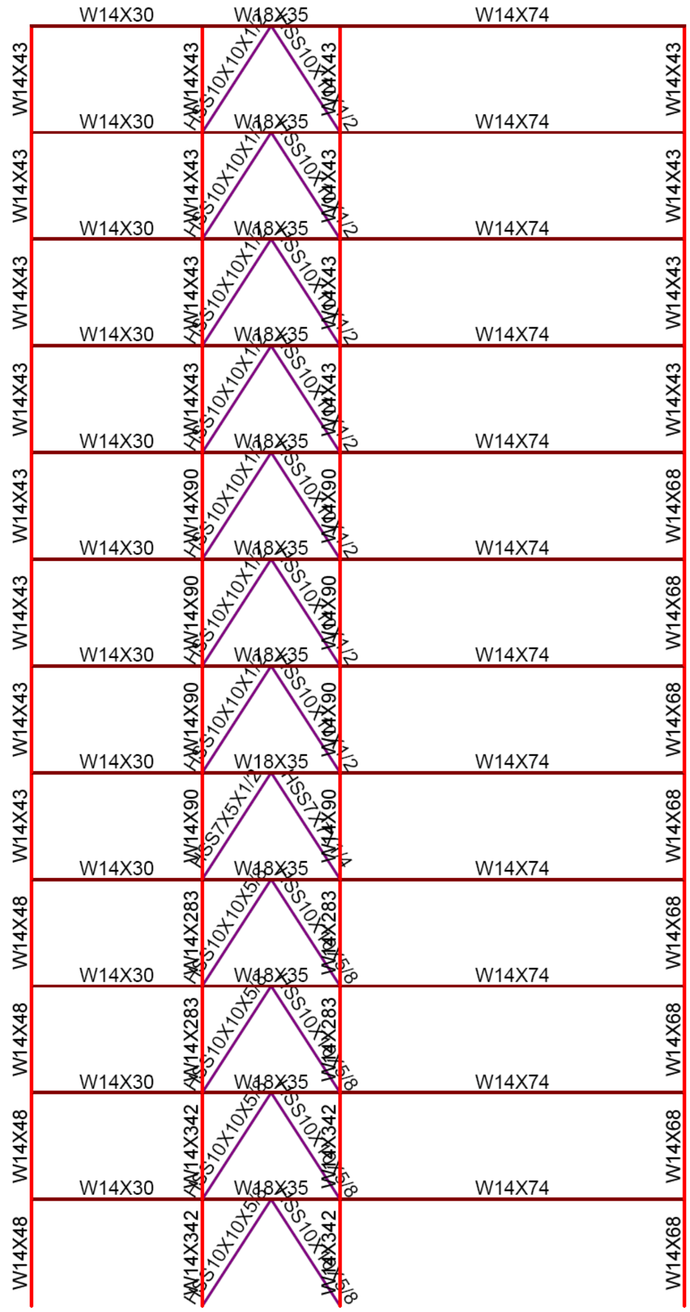
Elevation 5
Elevation 55

Elevation 6.8
figure 56

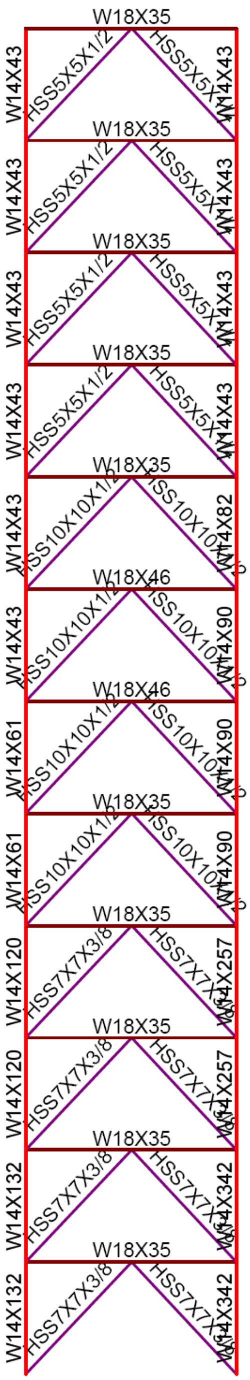
Elevation 7
figure 57



Elevation 8
Figure 58

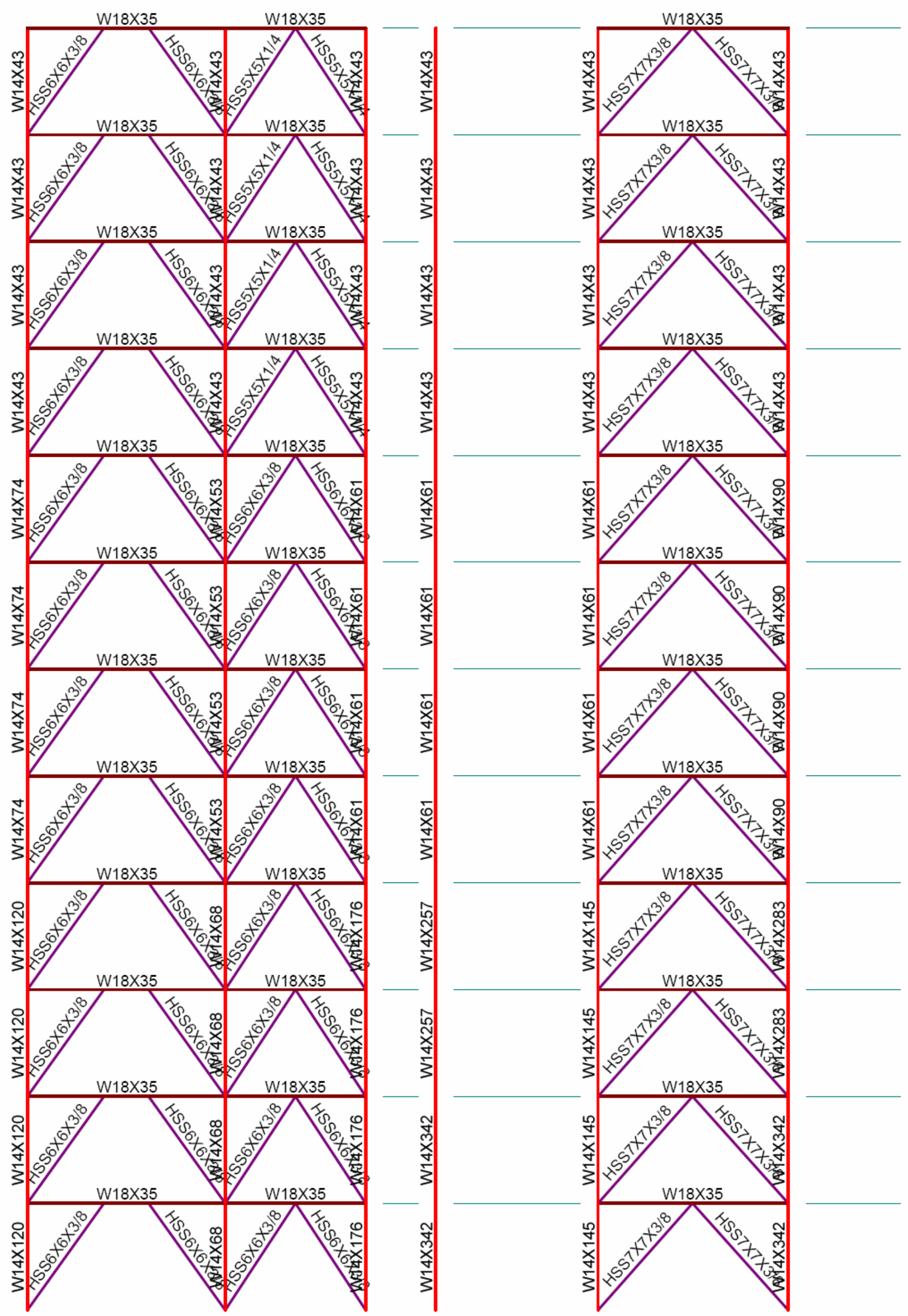


Elevation 9
figure 59



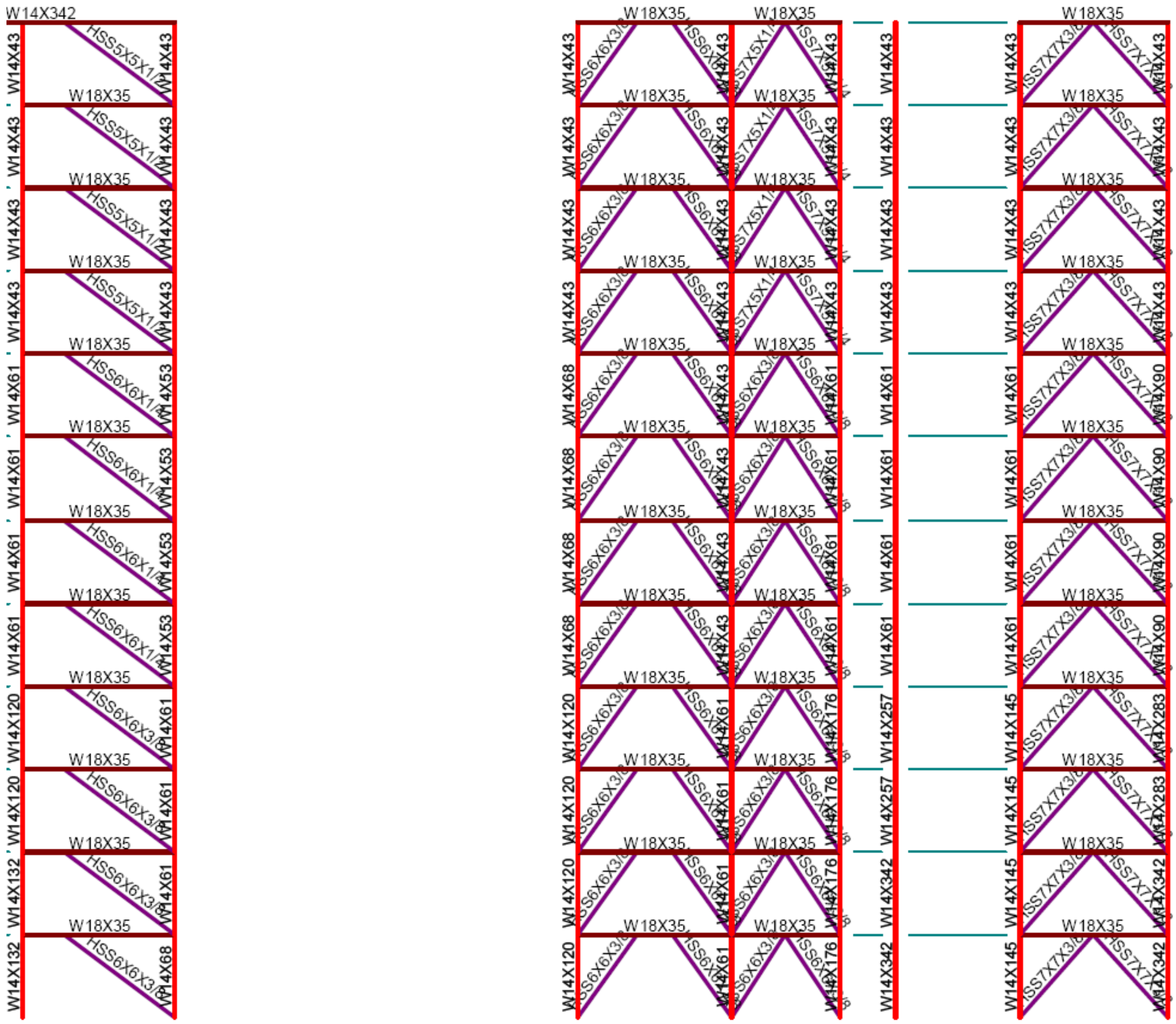
Elevation A

Figure 60



Elevation C

Figure 61



Elevation D

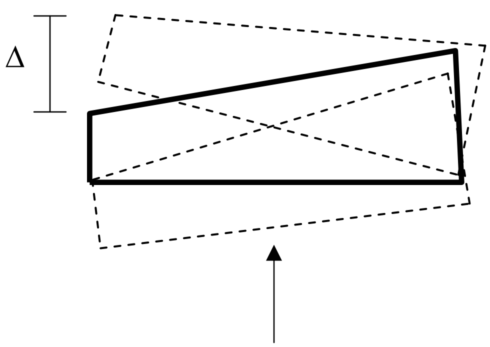
Figure62

4.6.11 Torsion

The building shape contains both plan and vertical irregularities which cause substantial torsional forces on the structure. The opening in the center of the façade creates a split diaphragm on the lower four levels. Therefore the center of rigidity shifts as you progress from level to level.

From studying deflection output from the ETABS model it was observed that under wind loading from the West, torsion creates larger deflection at the South end (figure 63). The opposite is observed under seismic loading from the same direction, where there are larger deflections at the North end (figure 64). This can be rationalized by the fact that the wedge plan shape causes the center of mass to be located further to the North with respect to the center of pressure. This also indicates that the center of rigidity is balanced between the center of pressure and the center of mass.

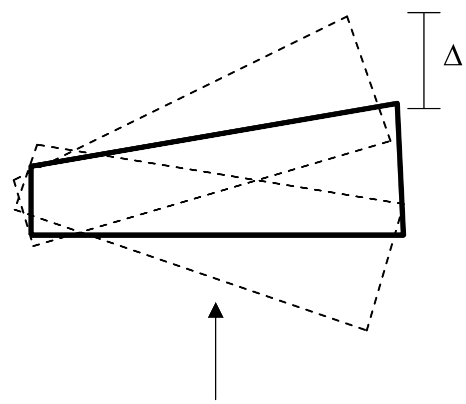
Wind - Larger Deflections at South End



Wind Force from West

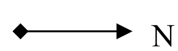
Figure 63: Behavior under wind loading

Seismic - Larger deflections at North End



Seismic Force from West

Figure 64: Behavior under seismic loading



4.6.12 Brace Design

Slenderness

"Braced Frames with very slender members must progressively drift further and further to be able to dissipate the same amount of energy under each cycle, which perhaps can lead to collapse due to second-order effects" [8]. Members were therefore designed in accordance with the slenderness criteria provided by the AISC Seismic Provisions.

$KL/r \leq 200$

Width to Thickness Ratio

"The plastic hinge that forms at mid-span of a buckled brace may develop large plastic rotations that could lead to local buckling and rapid loss of compressive capacity and energy dissipation during repeated cycles of inelastic deformations" [8]. The brace members were therefore required to comply with the width to thickness ratio provided in the AISC Seismic Provisions.

$b/t < \lambda$ $\qquad\qquad$ $\lambda_{ps} = 0.64\sqrt{(E/Fy)}$

[8] Naeim, Seismic design of steel structures

4.6.13 Impacts on foundations

Due to the narrow shape of the building, steel braced frames at the core may pose foundations issues. The columns at the core are experiencing very high axial forces due to overturning moment, especially when loaded in the wide direction. For the controlling load combination 1.2 Dead + 0.5 Live + 1.6 critical columns along gridline 3 were experiencing approximately 2000 kips of axial load.

The outriggers will contribute to distributing forces to the perimeter, however the girders used in the redesign were kept shallow in order to not interrupt MEP layout, therefore limiting the amount of force distributed to the perimeter. The substructure was not considered in this redesign, but assuming it is similar to the existing design, the two story cast in place concrete substructure will act as a base to distribute tension and compression forces from the core into to the piles.

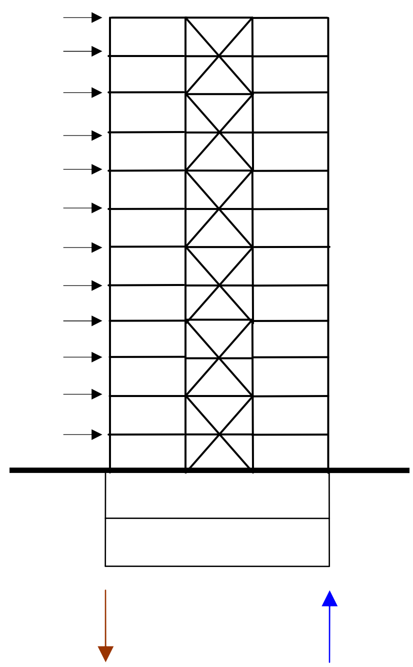


Figure 65: Overturning

4.6.14 Next Step

Due to limited time, further investigation of alternative framing solutions was not permitted. The proposed design yields an uneconomic structure due to the large column sizes required at the base of the core. Given more time, I would have liked to explore the use of moment frames around the perimeter and braced frames at the core (figure 66). This would allow for deep girder members at the perimeter and shallow interior beams, allowing MEP equipment go unobstructed. Perimeter moment frames would potentially alleviate the large axial forces in the columns and at the core due to the large overturning moments. This would also improve distribution of loads to the foundations. The increased number of moment frames could again result in an uneconomic solution, in which case a concrete shear walls at the core could be used.

Alternative design not explored in this report:

Location of Moment Frames
Location of Braced Frames

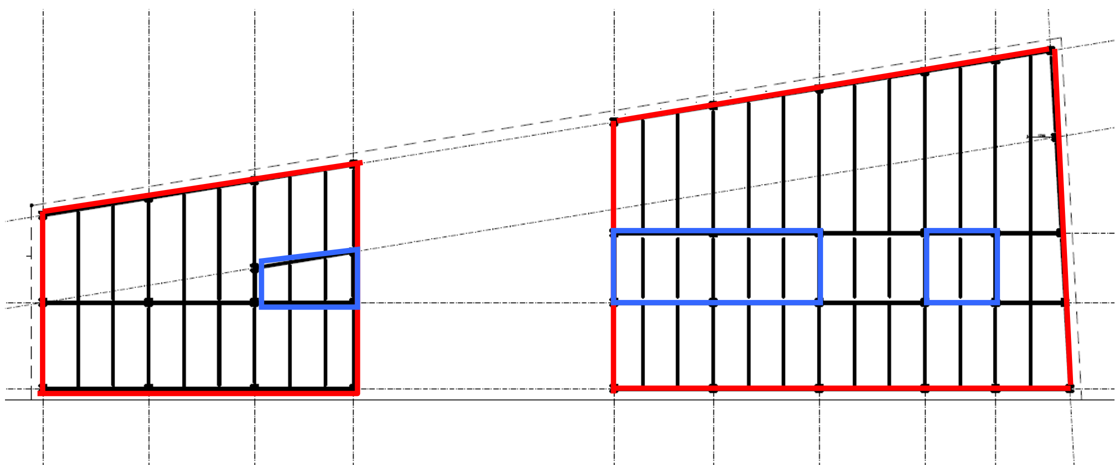


Figure 66: Alternative lateral force resisting design.
Moment frames at perimeter, braced frames at the core.

5 Architecture Breadth - Impacts of LFRS

5.1 Introduction

An important aspect of the lateral system redesign was its impacts on architecture. Initially, it was attempted to take advantage of the unique architectural properties of steel braced frames by exposing them to the exterior façade. It was ultimately decided to abandon this idea and rather conceal the braced frames at the core. With this came changes to the architectural layout at the South End core.

5.2 Braced Frame Location Study

Schematic diagrams (*figure 67*) imposed on floor plans were created to help determine the best locations for braced frames. An important feature of the PwC building is the wide hallway on all floors that circumnavigates the entire building *(Shown in orange on Figure 67)*. The hallway provides circulation as well as discussion and social space for the occupants. It was prioritized not to impede the hallway with steel braces. Therefore brace locations were limited to either the core or the exterior façade. Another hinder to braced frames was the auditorium and two story lobby at the North-West end of the building. This disallowed locating braces at the North end where they were needed to resist larger seismic loads created by larger mass at the North end. The 5 story opening at the center of the façade also disallowed braces to project to the base at the center of the building. It was ultimately concluded that brace locations were limited to either the core or the perimeter.

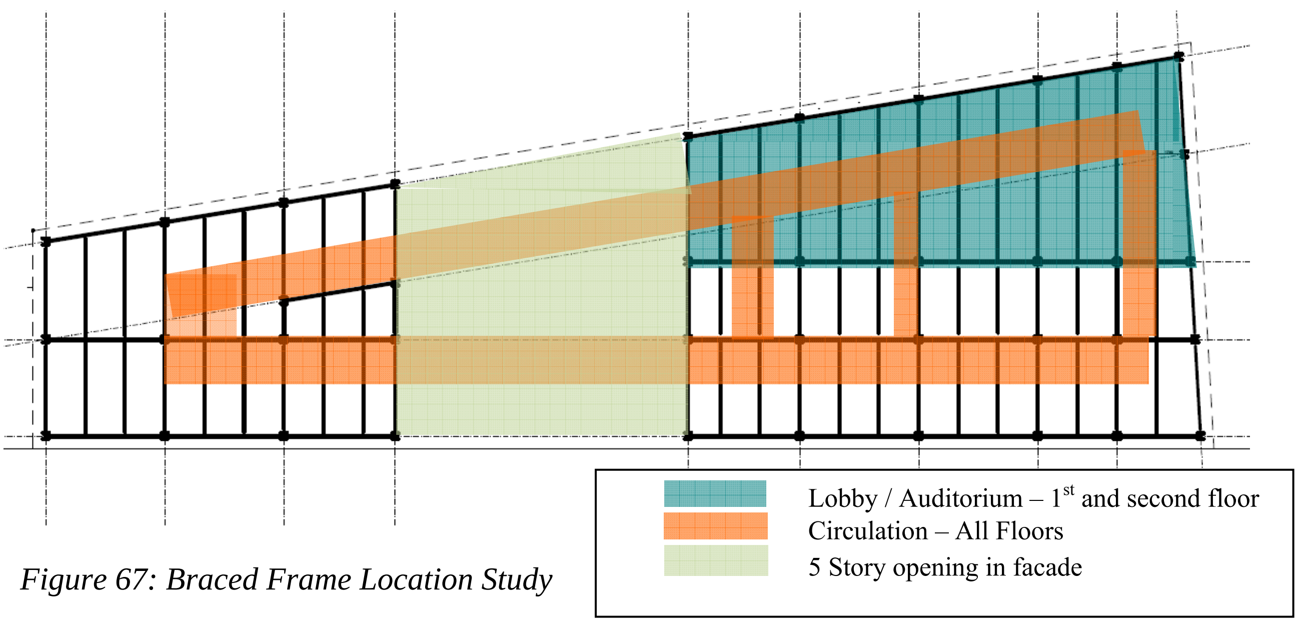


Figure 67: Braced Frame Location Study

5.3 Braced Frames at the Perimeter

Due to the narrow shape of the building it was hypothesized that a more efficient structural design could be achieved if the braced frames were brought to the perimeter and exposed to the facades (figure 68). The architectural idea behind this design was bring an expression of technological sophistication to the exterior façade. This approach was abandoned for a number of reasons;

1. Braces obstructed prominent views out of the building.
2. There would be a loss of usable floor space due to the braces.
3. The architectural massing would appear more segmented and less unified due to the vertical prominence of the braces, which was an unfavorable expression with respect to the ideology of the BARCODE concept (See Sec 1.1 of this report for discussion on the BARCODE concept).

Schematic design showing location of braced frames at the perimeter:

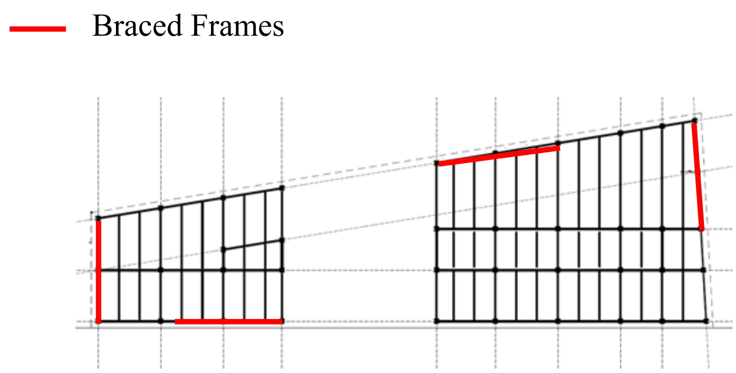


Figure 68: Braced frames at perimeter

The following images were created to study the visual implications of exposing braced frames to the exterior facades. It was during this study that it was discovered that the braces created an unfavorable segmentation of the architectural massing. Images were created by taking plans out of AutoCad and coloring them using Adobe Photoshop.

Existing West Façade

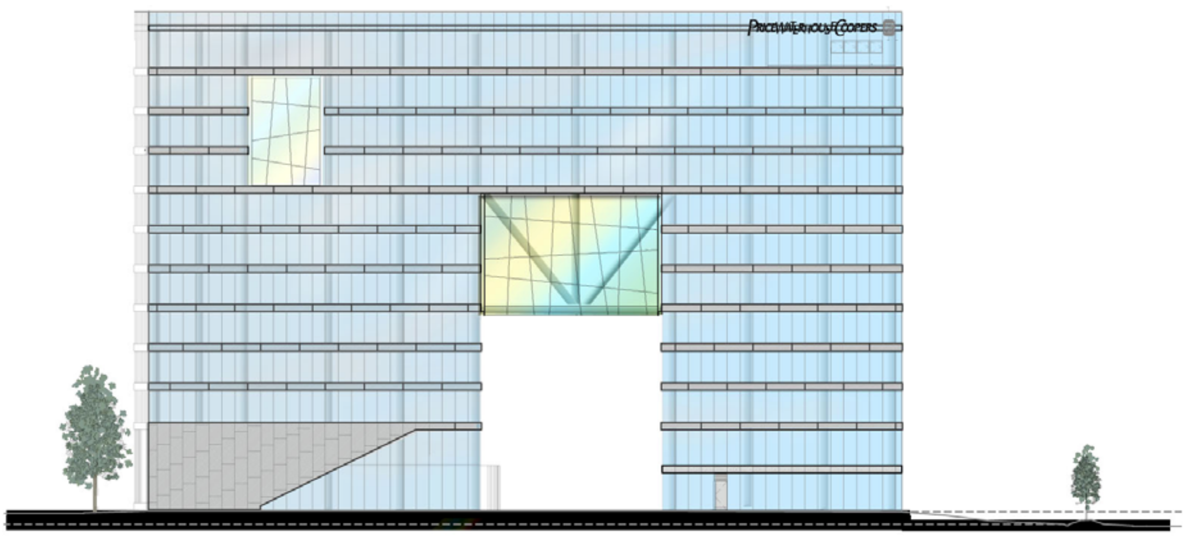


Figure69: West Façade - Existing Design

Proposed West Façade

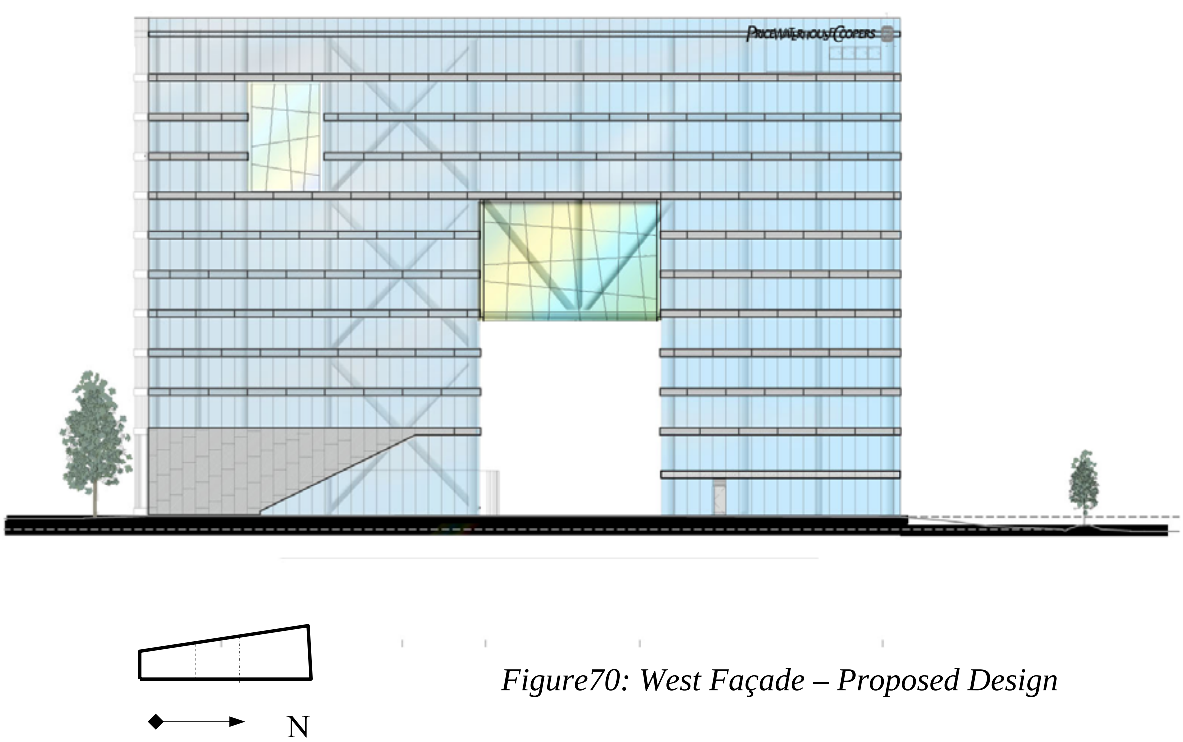


Figure70: West Façade – Proposed Design

Existing North Façade **Proposed North Façade**

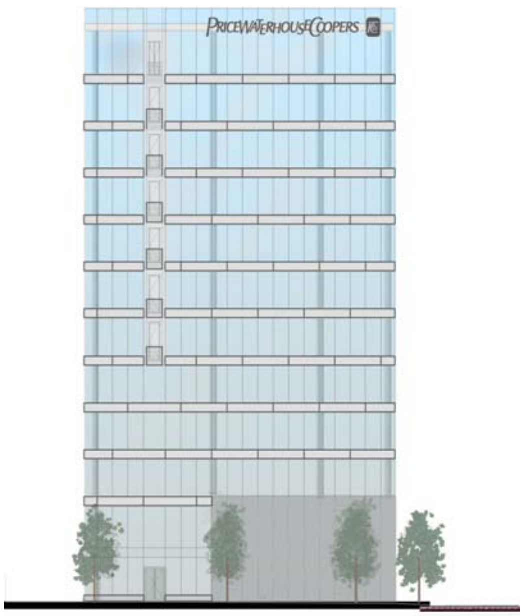


Figure70: North Façade - Existing Design

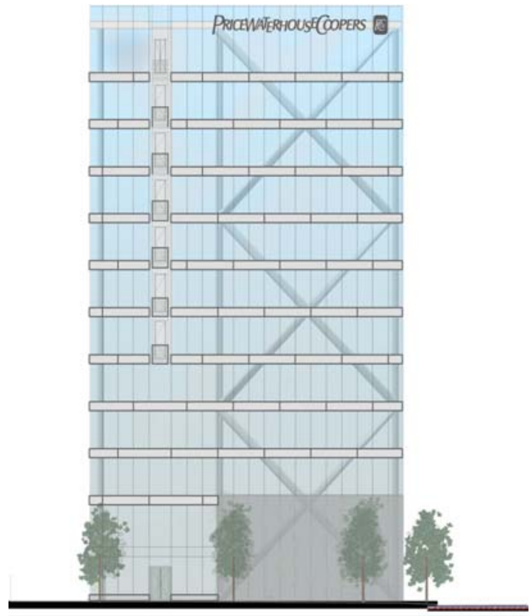
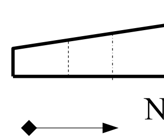


Figure71: North Façade - Proposed Design

Existing South Façade **Proposed South Façade**

Figure72: South Façade - Existing Design

Figure73: South Façade - Proposed Design



Existing East Façade

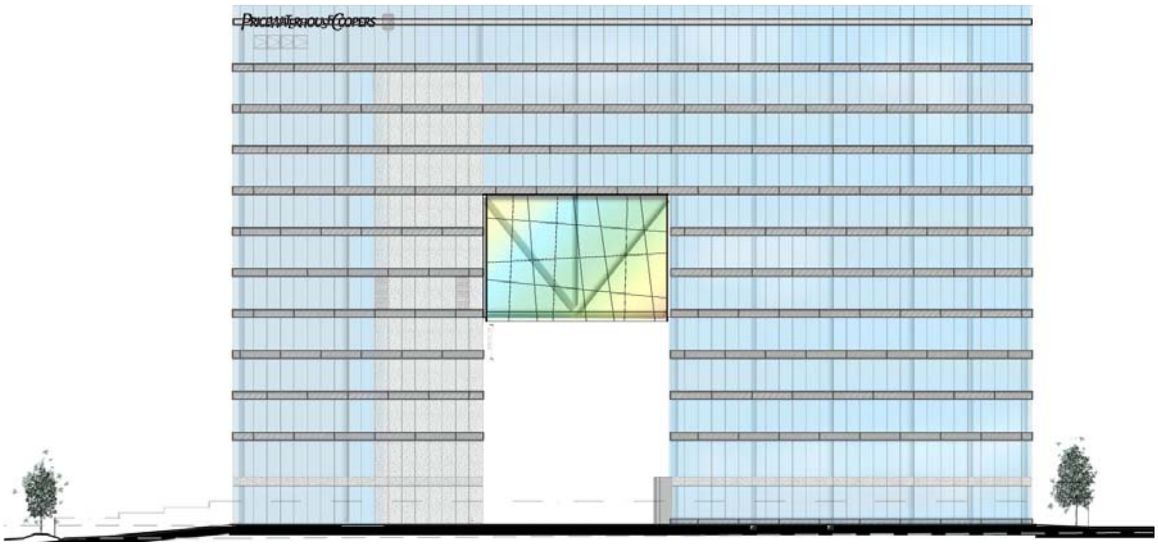
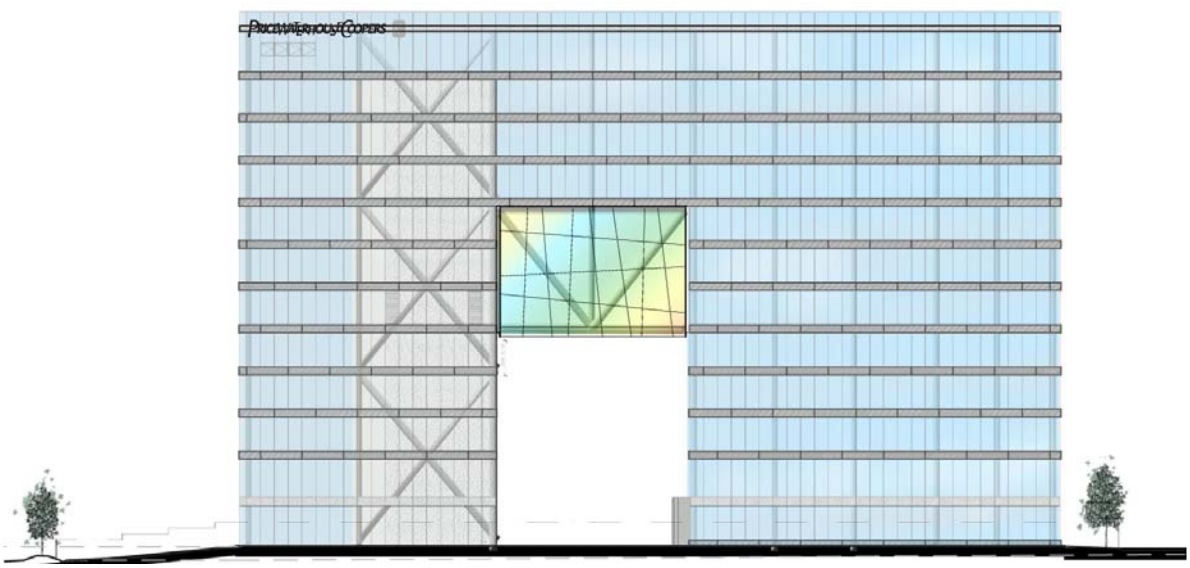


Figure74: East Façade - Existing Design

Proposed East Façade



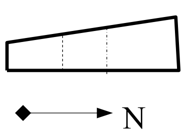


Figure75: East Façade – Proposed Design

5.4 Braced Frames at the Core – Architectural Redesign

The final design of the lateral system resulted in braced frames at the core, as opposed at the perimeter. Figure 76 and 77 below summarize the architectural changes made in order to accommodate the steel braces of the lateral system redesign. The major differences are the relocation of the elevator and duct shaft. This was required in order to place a steel brace where the elevator shaft was originally located. The duct shaft was allowed to be relocated because it only contained a standpipe and no mechanical or electrical equipment. Another architectural impact was an elongation of the stair case due to an increase in wall thickness where braces were located.

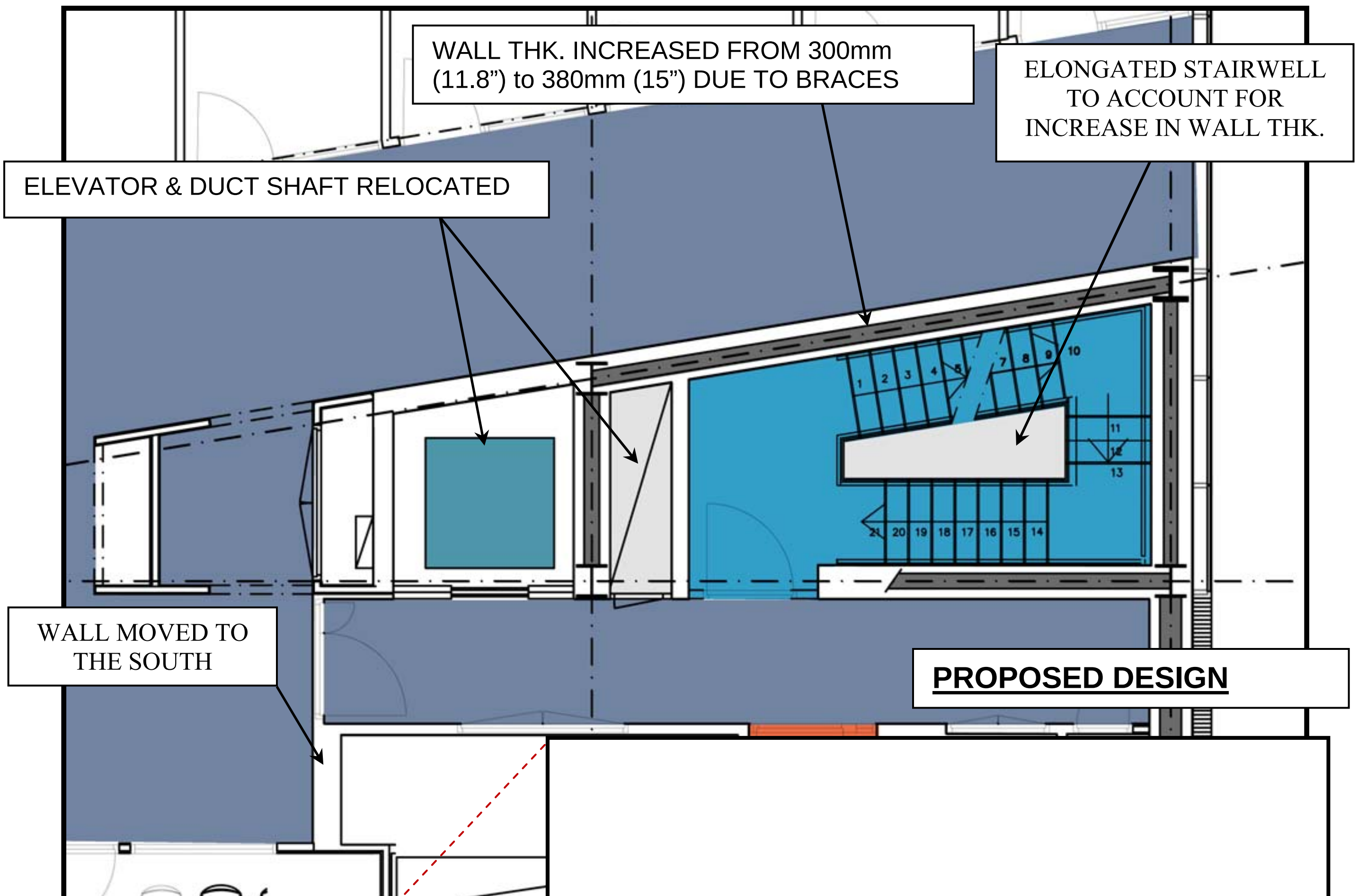


Figure76: Proposed design

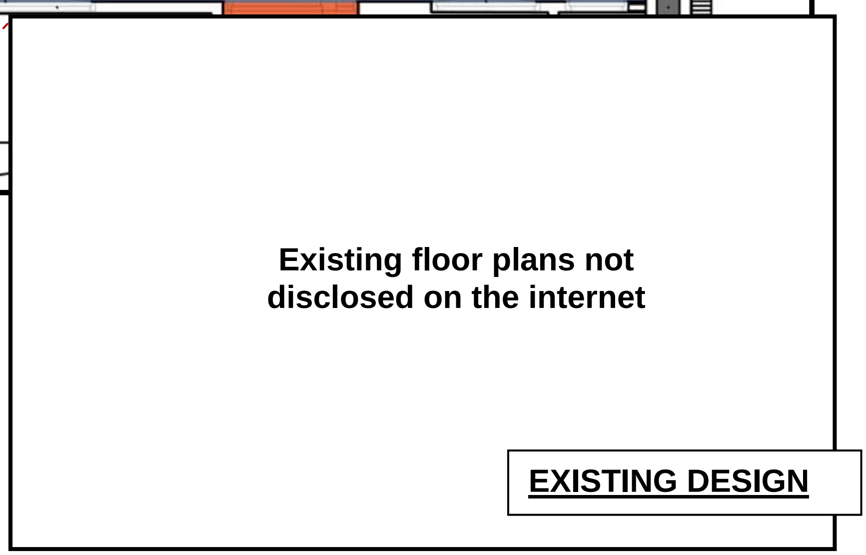
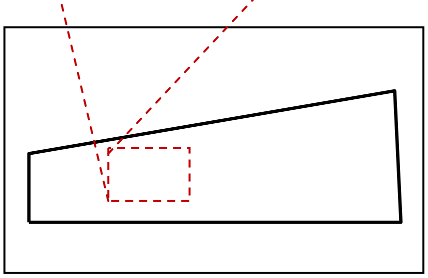


Figure77: Existing design



Orientation plan

6 Construction Management – Cost and Scheduling

6.1 Introduction

To further determine which floor system would be most viable for the Boston area, a cost and schedule comparison was conducted between composite concrete deck and precast concrete plank. The results obtained from this analysis indicate that the construction cost of the two systems are comparable, however the precast concrete plank system has potential to trim 23% off of the construction schedule. Therefore the prefabrication aspect of the plank system can potentially yield a more economical solution. This may also indicate that composite concrete deck is preferred in for Boston for reasons of structural performance rather than economy.

6.2 Cost Comparison

A simplified cost comparison was conducted using values obtained in the RS Means Construction Cost Data 2009. As the estimate was conducted for comparison purposes, items that were the same in both systems were omitted. In addition there were a number simplifying assumptions made:

- Both floor systems were considered to be on a steel frame with similar gravity columns, girders and bracing members.
- All girders supporting precast plank were approximated to weigh 22lb/ft.
- There is one shear stud / 36” of girder length for attachment of girder to composite slab.
- Additional cost due incurred by angled ends of precast concrete plank members was ignored.

Cost Comparison Summary:

Composite Steel Deck on Composite Steel Frame						
Quantity	**Description**	**Extended Cost ($)**			
		Material	Labor	Equipment	Total
150000 S.F.	Metal Decking	279,000	69,000	6,000	354,000
660 Ton	Structural Steel	1,518,000	250,800	87,120	1,855,920
1500 C.S.F	WWF 6 x 6	23,475	33,000	-	56,475
1960 C.Y.	Light Weight Concrete	286,160	-	-	286,160
14871 Ea.	Studs - 3/4"	8,030	11,153	5,651	24,835
150000 S.F.	Concrete Finish	-	73,500	3,000	76,500

Total = $ 2,653,889.57

Precast Plank on Steel Frame					
Quantity	**Description**	**Extended Cost ($)**			
		Material	Labor	Equipment	Total
430 Ton	Structural Steel	989,000	163,400	56,760	1,209,160
150000 S.F.	Precast Plank, 10" thick	1,147,500	126,000	78,000	1,351,500
923 C.Y.	2" Concrete Topping	97,838	-	-	97,838
150000 S.F.	Concrete Finish	-	52,500	6,000	58,500
2758 Ea.	Shear Stud - 3/4"	1,489	2,069	1,048	4,606

Total = $ 2,721,603.86

6.3 Schedule Comparison

A simplified construction schedule was created for both floor systems using Microsoft Project 2009. Again a number of assumptions were made in the schedule estimate:

- 40 pieces of steel erected per day
- 2 day schedule increase for 5 story opening in façade

From the schedules it was determined that it took the following number of days to erect the structure:

+ Composite Steel Deck = 52 days
+ Precast Concrete Plank = 40 days

The results from this study indicate that precast plank provides 23% reduction in construction time due less steel framing members required during erection of the structure.

6.4 Square Foot Cost Estimate

A square foot estimate of the redesign was conducted for comparison purposes with that of Oslo. This simplified estimate was assembled aided by an online RSMeans resource, from which only a few building parameters were input. Not included in this estimate were added cost for the opening in the façade, auditorium, and high end mechanical equipment. It is therefore likely that cost will be higher than that estimated. However, it does provide an indication as to what the PwC building would cost if hypothetically built in Boson.

Cost - Boston Redesign = \$33mil + *additional cost for 5 opening center of façade*
+ *additional cost for premium MEP equipment*
+ *additional cost for auditorium*

→ *After additional costs are considered the estimate may be more similar to the existing design.*

Cost - Existing Design = \$45mil + *assumes an exchange rate of \$6.67 = 100kr*

Summary output of square foot estimate provided by the RS Means online resource:

Estimate Name:	**PwC** Dor, Boston, MA, 02125
Building Type:	**Office,11-20 Story with Double Glazed Heat Absorbing Tinted Plate Glass Panels / Steel Frame**
Location:	**BOSTON, MA**
Stories:	**12**
Story Height (L.F.):	**12**
Floor Area (S.F.):	**185900**
Labor Type:	**Union**
Basement Included:	**Yes**
Data Release:	**Year 2008 Quarter 1**
Cost Per Square Foot:	**\$177.38**
Building Cost:	**\$32,974,500**

Costs are derived from a building model with basic components. Scope differences and market conditions can cause costs to vary significantly.

7 Conclusion

There were many factors that dictated the choice of structural system, and the ultimate solution became one that balanced structural performance, economy and architectural expression.

A lot of attention was directed towards the comparison of composite concrete deck and precast concrete plank. Precast concrete plank is more commonly used Norway, however composite concrete deck is preferred in Boston for office buildings. Through discussion with design professionals, it was found that composite concrete deck provides more flexibility for future alterations, because it is not limited by the cutting of pre-stressed strands found in precast concrete plank. A study on the economic aspect, however, revealed that precast concrete plank can be favorable also in Boston, due to cost saving incurred by reduction of construction schedule of up to 23%.

In the redesign, composite concrete deck on composite steel beams was determined to be the most viable floor system for Boston. The framing plan conducted in the redesign conformed easily to the existing architectural layout, and with the use of composite action in beams and girders, structural depth was minimized. The proposed design resulted in a structural depth of 19.25", which is 5" deeper than the existing design. Composite action also yielded a more economic solution due the allowance of smaller steel members.

The redesign of the lateral force resisting system was performed using steel as the choice of material. Amongst other reasons, steel was selected because of its compatibility with the steel framing chosen in the redesign of the floor system. Much effort was devoted towards determining a structure that met design criteria. The resulting structure uses concentrically braced chevron frames at the core with moment frames acting as outriggers to perimeter columns. Despite efforts, it was concluded that the design was an uneconomic solution because of the large axial forces in the columns, induced by the narrow aspect ratio of the core. Given more time to explore the use of braced frames in combination with moment frames, a more economic steel structure could likely be determined. If not, the most viable structural system for the PwC building, if hypothetically located in Boston, would be concrete shear walls at the core in combination with the proposed floor system.

Prefabrication and standardization can produce very cost effective structures. The elegant design of the existing structure is an excellent example of this. Potential advantages of the proposed design are; structurally, a more flexible deck for future alterations and reduction of construction schedule with an all steel lateral and floor system.

Appendix

A.1 Wind Loads

A.1.1 Gust Factor

Gust Factor		
	E/W	N/S
L	82	234
B	234	82
h	147	147
n_1	0.6	0.6
$n_1 > 1$	Flex	Flex
g_Q, g_v	3.4	3.4
g_r	4.05	4.05
zhat	87.6	87.6
I_z	0.25	0.25
L_z	441.8	441.8
Q	0.72	0.84
V	105	105
V_z	88.45	88.45
N_1	3	3
R_n	0.07	0.07
n_h	4.6	4.6
R_h	0.19	0.19
n_b	7.3	2.6
R_B	0.13	0.31
nl	8.6	24.4
R_L	0.11	0.04
R	0.015	0.015
G	**0.772967**	**0.838464**

A.1.2 Velocity Pressure

Velocity Pressure	
V - Basic Wind speed	105
Occupancy Category	III
K_d	0.85
Importance Factor	1
Exposure Category B	B
K_{zt}	1

A.1.3 Pressure Coefficients

Pressure Coefficients	
C_p Windward wall	0.8
C_p Leeward wall E / W	-0.5
C_p Leeward wall N / S	-0.5
C_p Side wall	-0.7
G_{cpi} Internal Pressure	

A.1.4 Wind Pressures – Wind from North/South

North / South L = 234ft B =82ft

Floor	height (ft)	K_z	q_z	Pressure (psf)						
				N/S Windward			N/S Leeward			Total
Roof	146	1.102	26.439	17.73	+/-	4.76	-10.97	+/-	4.76	28.70
12	133	1.072	25.712	17.24	+/-	4.76	-10.97	+/-	4.76	28.22
11	121	1.043	25.027	16.78	+/-	4.76	-10.97	+/-	4.76	27.76
10	109	1.013	24.292	16.29	+/-	4.76	-10.97	+/-	4.76	27.26
9	97	0.979	23.486	15.75	+/-	4.76	-10.97	+/-	4.76	26.72
8	85	0.943	22.629	15.18	+/-	4.76	-10.97	+/-	4.76	26.15
7	73	0.903	21.668	14.53	+/-	4.76	-10.97	+/-	4.76	25.50
6	61	0.858	20.586	13.81	+/-	4.76	-10.97	+/-	4.76	24.78
5	49	0.806	19.341	12.97	+/-	4.76	-10.97	+/-	4.76	23.94
4	37	0.744	17.854	11.97	+/-	4.76	-10.97	+/-	4.76	22.95
3	25	0.666	15.971	10.71	+/-	4.76	-10.97	+/-	4.76	21.68
2	13	0.553	13.270	8.90	+/-	4.76	-10.97	+/-	4.76	19.87
1	0		0.000							0.00

A.1.5 Wind Pressures – Wind from East/West

East / West L = 82ft B = 234ft

Floor	h_x	K_z	q_z	Pressure						
				N/S Windward			N/S Leeward			Total
Roof	146	1.102	26.44	16.34	+/-	4.76	-10.21	+/-	4.76	26.56
12	133	1.072	25.71	15.89	+/-	4.76	-10.21	+/-	4.76	26.11
11	121	1.043	25.03	15.47	+/-	4.76	-10.21	+/-	4.76	25.69
10	109	1.013	24.29	15.02	+/-	4.76	-10.21	+/-	4.76	25.23
9	97	0.979	23.49	14.52	+/-	4.76	-10.21	+/-	4.76	24.73
8	85	0.943	22.63	13.99	+/-	4.76	-10.21	+/-	4.76	24.20
7	73	0.903	21.67	13.39	+/-	4.76	-10.21	+/-	4.76	23.61
6	61	0.858	20.59	12.73	+/-	4.76	-10.21	+/-	4.76	22.94
5	49	0.806	19.34	11.96	+/-	4.76	-10.21	+/-	4.76	22.17
4	37	0.744	17.85	11.04	+/-	4.76	-10.21	+/-	4.76	21.25
3	25	0.666	15.97	9.87	+/-	4.76	-10.21	+/-	4.76	20.09
2	13	0.553	13.27	8.20	+/-	4.76	-10.21	+/-	4.76	18.42
1	0		0.00							0.00

A.2 Seismic Loads

A.2.1 Code Values

Location	Boston, Mass	
Latitude	42.35	
Longitude	-71.06	
Site Class	E	Table 20.3 - 1
S_s	0.28	USGA Java Motion Parameter:
S_1	0.068	USGA Java Motion Parameter:
F_a	2.41	Table 11.4-1
F_v	3.5	Table 11.4-2
S_{MS}	0.6748	Eq 11.4-1
S_{M1}	0.238	Eq 11.4-2
S_{DS}	0.450	
S_{D1}	0.159	
Occupancy Category	II	IBC Table 1604.5
SDC	B	Table 11.6-1
Imprtance Factor	1	
TL	6	Figure 22-15

A.2.2 Base shear R = 3.25

N/W and E/W Direction		
R	3.25	Table 12.2 -1, Ordinary steel concentrically braced frames
Cd	3.25	Table 12.2 -1, Ordinary steel concentrically braced frames
Ct	0.02	Table 12.8-2, all other structural systems
hn	147	Building height
x	0.75	Table 12.8-2, all other structural systems
Cu	1.6	Table 12.8-1

Ta = ct*hn^x	0.84
T = Cu*Ta	1.35
Cs = Min:	
SDS / (R/I)	0.138
SD1 / (T(R/I))	**0.036**
(SD1*TL)/((T^2)*(R/I))	0.161

Weight	11176
Vb = Cs*W	404

A.2.3 Base Shear R = 6

N/W and E/W Direction		
R	6	Table 12.2 -1, Special steel concentrically braced frames
Cd	5	Table 12.2 -1, Special steel concentrically braced frames
Ct	0.02	Table 12.8-2, all other structural systems
hn	147	Building height
x	0.75	Table 12.8-2, all other structural systems
Cu	1.6	Table 12.8-1

Ta = ct*hn^x	0.84
T = Cu*Ta	1.35
Cs = Min:	
SDS / (R/I)	0.075
SD1 / (T(R/I))	**0.020**
(SD1*TL)/((T^2)*(R/I))	0.087

Weight	11176
Vb = Cs*W	219

A.2.4 Story Force Distribution R = 3.25

Story forces n/s and e/w direction

T= 1.350 s
k= 1.425
V_b= 404 kips

Split forces on diaphragm according to mass
% Mass North Leg = 0.32
% Mass South Leg = 0.68

Seismic Loads in North / South Direction - Ordinary braced frames

i	h_i (ft)	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)	V_i (kips)	By (ft)	5%By (ft)	Ax	M_z (k-ft)
Roof	12	144	1035	1231974	0.187	75	75	234	12	1.06	936
12	12	132	1035	1088311	0.165	67	142	234	12	1.06	827
11	12	120	1035	950098	0.144	58	200	234	12	1.00	681
10	12	108	1035	817643	0.124	50	250	234	12	1.00	586
9	12	96	1035	691308	0.105	42	293	234	12	1.00	495
8	12	84	1035	571522	0.087	35	328	234	12	1.13	463
7	12	72	1035	458811	0.070	28	356	234	12	1.39	457
6	12	60	1035	353835	0.054	22	377	234	12	1.50	380

Split Diaphragm

i	h_i (ft)	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i S	f_i N	V_i (kips)	By S	By N	5%By S	5%By N	Ax S	Ax N	M_z S	M_z N
5	12	48	800	199060	0.030	3.9	8.3	390	74	106	4	5.3	1.0	1.15	15	50
4	12	36	800	132113	0.020	2.6	5.5	398	74	106	4	5.3	1.1	1.41	10	41
3	12	24	800	74134	0.011	1.5	3.1	402	74	106	4	5.3	1.0	1.46	5	24
2	12	12	800	27609	0.004	0.5	1.1	404	74	106	4	5.3	1.0	0.89	2	5

Σ	11481	6596416		404

A2.5 Amplification Factor, R = 3.25, Loading in East- West Direction

Amplification Factor in the East-West Direction (Y Dir)														
i	h_i	h	δA		δB		δavg.		δmax		(δmax /		Ax	
	(ft)	(ft)	(in.)		(in.)		(in.)		(in.)		(1.2*δavg))^2		(1< Ax < 3)	
Roof	12	144	1.68		2.70		2.19		2.70		1.06		1.06	
12	12	132	1.50		2.42		1.96		2.42		1.06		1.06	
11	12	120	1.31		1.80		1.55		1.80		0.93		1.00	
10	12	108	1.10		1.50		1.30		1.50		0.92		1.00	
9	12	96	0.90		1.20		1.05		1.20		0.91		1.00	
8	12	84	0.70		1.24		0.97		1.24		1.13		1.13	
7	12	72	0.50		1.21		0.86		1.21		1.39		1.39	
6	12	60	0.34		0.94		0.64		0.94		1.50		1.50	
Diaphragm Splits			S	N	S	N	S	N	S	N	S	N	S	N
5	12	48	0.24	0.39	0.37	0.70	0.30	0.55	0.37	0.70	1.03	1.15	1.03	1.15
4	12	36	0.15	0.19	0.24	0.47	0.20	0.33	0.24	0.47	1.05	1.41	1.05	1.41
3	12	24	0.08	0.12	0.12	0.31	0.10	0.22	0.12	0.31	1.01	1.46	1.46	1.46
2	12	12	0.03	0.05	0.04	0.07	0.03	0.06	0.04	0.07	1.02	0.89	1.02	1.00

A2.6 Amplification Factor, R = 3.25, Loading in North- South Direction

Amplification Factor in the North-Soth Direction (X Dir)									
i	h_i	h	δA	δB	δavg.	δmax	(δmax /	Ax
	(ft)	(ft)	(in.)	(in.)	(in.)	(in.)	(1.2*δavg))^2	(1< Ax < 3)
Roof	12	144	1.83	2.07	1.95	2.07	0.78	1.00
12	12	132	1.67	1.88	1.78	1.88	0.78	1.00
11	12	120	1.49	1.67	1.58	1.67	0.78	1.00
10	12	108	1.30	1.45	1.38	1.45	0.77	1.00
9	12	96	1.11	1.23	1.17	1.23	0.77	1.00
8	12	84	0.93	1.01	0.97	1.01	0.75	1.00
7	12	72	0.95	0.81	0.88	0.95	0.81	1.00
6	12	60	0.79	0.63	0.71	0.79	0.86	1.00
5	12	48	0.59	0.47	0.53	0.59	0.86	1.00
4	12	36	0.44	0.32	0.38	0.44	0.93	1.00
3	12	24	0.18	0.19	0.18	0.19	0.74	1.00
2	12	12	0.07	0.08	0.08	0.08	0.74	1.00

A.2.7 Total Building Weight Calculations

Decking + SIMP

Level	Trib. Area (ft^2)	Loads			Weight (kip)
		Steel Deck	Conc. Dk. (psf)	SIMP (psf)	
Roof	14391	1.8	41	15	832
12	14391	1.8	41	15	832
11	14391	1.8	41	15	832
10	14391	1.8	41	15	832
9	14391	1.8	41	15	832
8	14391	1.8	41	15	832
7	14391	1.8	41	15	832
6	14391	1.8	41	15	832
5	11009	1.8	41	15	636
4	11009	1.8	41	15	636
3	11009	1.8	41	15	636
2	11009	1.8	41	15	636
					9200

Façade

Story	Perimiter (ft)	Trib Height (ft)	Wall Load (psf)	Weight (kip)
Roof	581	10	15	87.15
12	581	12	15	104.58
11	581	12	15	104.58
10	581	12	15	104.58
9	581	12	15	104.58
8	581	12	15	104.58
7	581	12	15	104.58
6	581	12	15	104.58
5	587	12	15	105.66
4	587	12	15	105.66
3	587	12	15	105.66
2	587	12	15	105.66
			Total :	**1241.85**

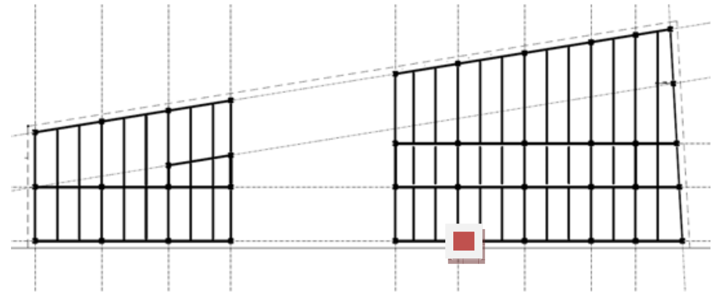
Steel Shapes - From RAM output
Distribute weight of steel shapes evenly across floors for calc of seismic loads

Steel Shapes	Weight (kip)
Beams	313
Columns	223
Braces	163
Total :	**854**

Total Building Weight

Level	Actual Wt. (kip)	Actual Wt. (psf)	Wt. used in Design (psf)	Wt. used in Design (kip)
Roof	977	76	80	1035
12	995	77	80	1035
11	995	77	80	1035
10	995	77	80	1035
9	995	77	80	1035
8	995	77	80	1035
7	995	77	80	1035
6	995	77	80	1035
5	800	82	85	800
4	800	82	85	800
3	800	82	85	800
2	800	82	85	800
Total:	**11140.5292**			**11481**

A.3 Column Spot Check



Exterior Column E-6

Loads:

Dead:	Deck + Slab+ ST. Mem =	50	psf
	SIMP =	15	psf
	Façade =	15	psf
Live:	Office =	80	psf
	Roof (+snow) =	100	psf

Tributary Area

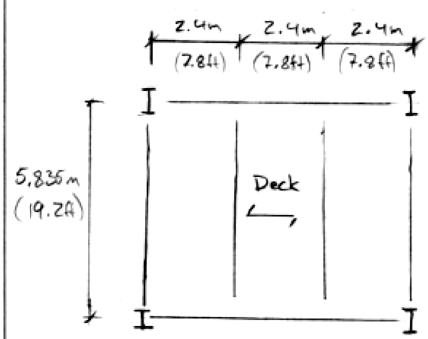
L_{east} =	22.2	ft
L_{west} =	23.6	ft
L_{north} =	19.2	ft
L_{south} =	1	ft
AT =	232	ft2
AI =	926	ft2

Story	Height	D (psf)	D_{facade} (plf)	L (psf)	A_T (ft2)	A_I (ft2)	A_I Total (ft)	LL Red.
Roof	144	65	90	100	232	926	926	0.74
12	132	65	180	80	232	926	1852	0.60
11	120	65	180	80	232	926	2778	0.53
10	108	65	180	80	232	926	3704	0.50
9	96	65	180	80	232	926	4630	0.47
8	84	65	180	80	232	926	5556	0.45
7	72	65	180	80	232	926	6482	0.44
6	60	65	180	80	232	926	7408	0.42
5	48	65	180	80	232	926	8334	0.41
4	36	65	180	80	232	926	9260	0.41
3	24	65	180	80	232	926	10186	0.40
2	12	65	180	80	232	926	11112	0.40

Story	Total Load (kip)		Factored Load, Pu (kip)		
	D	L	1.2D +1.6L	1.4D	
Roof	17	17	48	24	
12	36	28	89	51	
11	56	38	128	78	
10	75	47	**166**	105	Splice
9	94	56	203	132	
8	113	65	239	158	
7	132	73	275	185	
6	152	81	**311**	212	Splice
5	171	88	346	239	
4	190	96	381	266	
3	209	103	416	293	
2	228	111	451	320	

Column E-6 Spot Check					
	Floor	Pu	KL (ft)	Least Wt. Mem.	PhiPn
Hand Calc.	Floor 1-4	166	12	W12x40	328
	Floor 5-8	311	12	W12x40	328
	Floor 9-roof	451	12	W12x53	547
RAM	Floor 1-4	155	12	W12x40	328
	Floor 5-8	287	12	W12x40	328
	Floor 9-roof	429	12	W12x53	547

A.4 Beam and Girder - Spot Check



Deck & Slab

Design Loads

SIMP$_{DL}$ = 15 psf

Facade = 180 plf

Deck$_{DL}$ + Slab$_{DL}$ = 41 psf

Live Load = 80 psf

Lightweight Concrete 115 pcf, f'_c = 3ksi

2" LOK-FLOOR, 20 gage Deck w/ 3.25" Slab

Max unshored length = 9.48'

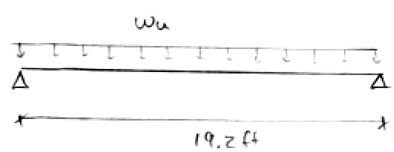
Beam Design

Trib Area = (7.8ft)(19.2ft) = 149.76ft ≈ 150 ft^2

Influence Area: $2A_T = (2)(150 ft^2) = 300 ft^2$

$LL = L_o\left(.25 + \frac{15}{\sqrt{300}}\right) = L_o\, 1.11 \Rightarrow$ No Reduction

LL = 80psf



Wu = Dead = (15 psf + 41 psf)(7.8ft) = 436.8 plf

Live = (80 psf)(7.8ft) = 624 plf

Strength: 1.2D + 1.6L = 1.53 k/ft

$M_u = \frac{w_u \ell^2}{8} = \frac{(1.53)(19.2^2)}{8} = 70.5$ ft·k

Deflection:

$$\Delta_{LL} \leq \frac{(19.2)(12)}{360} = .64'' = \frac{(5)(.624/12)(19.2\times12)^4}{384(29000)(I_{req})}$$

$$\Rightarrow I_{req} = 102.8 \, in^4$$

$$\Delta_T \leq \frac{(19.2)(12)}{240} = .96'' = \frac{(5)(1.06/12)(19.2\times12)^4}{384(29000)(I_{req})}$$

$$\Rightarrow I_{req} = 116.4 \, in^4$$

$$\Delta_{PC} \leq \frac{(19.2)(12)}{360} = .64'' = \frac{(5)(.41/12)(19.2\times12)^4}{(384)(29000)(I_{req})}$$

$$\Rightarrow I_{req} = 67.5 \, in^4 \; * \text{controls}$$

Size Comparison

Composite — For $Y_2 = 5 - \frac{a}{2} = 4.5$ (assume $a = 1''$) — $Q_n = 17.2$ {2" deck, deck perp.

Mem	I_x	ϕM_p	ΣQ_n	#	Equiv lb	
W10×12	53.8	74.8	44.2	(6)	230 + 60 = 290	× $I_x < I_{req}$
W10×15	68.9	94.4	55.1	(8)	288 + 80 = 368	
W12×14	88.6	101	51.9	(8)	268.8 + 80 = 348.8	⟸ OK

Non Composite ⇒ ϕM_p for span of 20ft

Mem	I_x	ϕM_p	ΣQ_n	#	Equiv lb	
W12×16	103	75.4	—	—	307.2	× $I_x < I_{req}$
W10×19	96.3	81.0	—	—	365	× $I_x < I_{req}$
W12×19	130	92.6	—	—	365	⟸ OK

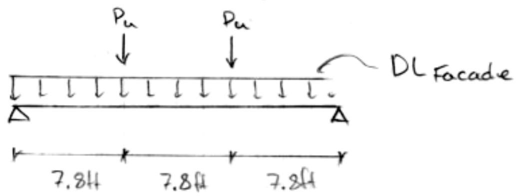
Member Selection is consistent with RAM

Exterior Girder Design

Trib Area: $\left(\frac{19.2ft}{2}\right)(23.4ft) = 224.64 \approx 225ft^2$

Influence Area: $2A_T = 450ft^2$

$LL = L_o\left(.25 + \frac{15}{\sqrt{450}}\right) = L_o(.96) \Rightarrow LL = 77\,psf$



Pu: Dead: $(41psf + 15psf)\left(\frac{19.2ft}{2}\right)(7.8ft) = 4.2^k$

Live: $(77psf)\left(\frac{19.2ft}{2}\right)(7.8ft) = 5.8^k$

Wu: Dead: $.18^{k}/ft$

- Strength: $1.2D + 1.6L = 14.3^k = P_u$

$1.2D + 1.6L = .216^{k}/ft = w_u$

$M_u = (14.3^k)(7.8ft) + \frac{(.216^{k}/ft)(23.4ft)^2}{8}$

$= 126.3\,kft$

- Deflection:

$\Delta_{LL} \leq \frac{(23.4)(12)}{360} = .78'' = \frac{(5.8)(23.4 \times 12)^3}{(28)(29\,000)\,I_{req}}$

$\Rightarrow I_{req} = 202.7\,in^4$

$\Delta_T \leq \frac{(23.4)(12)}{240} = 1.17 = \frac{(10)(23.4 \times 12)^3}{(28)(29\,000)I_{req}} + \frac{(5)(.18/12)(23.4 \times 12)^3}{(384)(29000)(I_{req})}$

$$1.17 = \frac{272.6}{I_{req}} + \frac{41.87}{I_{req}}$$

$$\Rightarrow I_{req} = 268.8\,in^4 \Leftarrow$$

$$\left[\text{Pre composite DL} = (41\,psf)(7.8\,ft)\left(\frac{19.2\,ft}{2}\right) = 3.1^k = P_u\right]$$

$$\Delta_{PC} \leq \frac{(23.4)(12)}{360} = .78'' = \frac{(3.1^k)(23.4 \times 12)^3}{28(29\,000)}$$

$$\Rightarrow I_{req} = 108.36\,in^4$$

Member Selection Assume $a = 1''$ $\Rightarrow Y_2 = 4.5''$ $Q_n = 18.3$

Composite Limit depth 14"

Mem	I_x	I_{LB}	ϕM_p	ΣQ_n	#	Equiv(16)	
W12×16	103	207	134	94.4	12	497.9	× $I_x, I_{LB} < I_{req}$
W12×19	130	255	140	69.7	8	579.7	× $I_{LB} < I_{req}$
W14×22	199	424	183	81.2	10	619.2	⇐ ok

Non Composite

W12×26	204	-	140	–	–	613.6	× $I_x < I_{req}$
W14×26	245		151	–	–	613.6	× $I_x < I_{req}$
W16×26	301		166	–	–	613.6	16" > Depth Limit
W14×30	291		177	–	–	708	⇐ ok

Member Selection is consistent with RAM

A.5 Trial Designs of Steel Bracing Configurations

A.5.1 Trial 1

Members:
W14x132 Columns
W18 x 86 Beams
HSS10x10x0.5 Bracing

Period:
Mode 1: 4.50s - torsion about south leg
Mode 2: 2.43 - torsion about north leg

Max Deflections:
Seismic loaded in X direction = 1.67in
Seismic loaded in Y direction = 4.95in
Wind Loaded in Y direction = 9.11in
Wind loaded in X direction = 4.95i

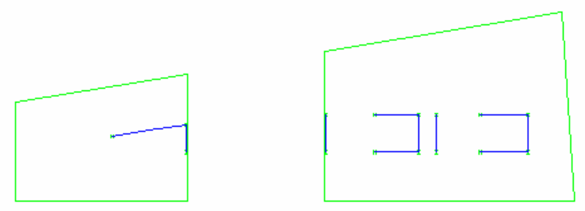


Figure A1: Trial 1 bracing plan

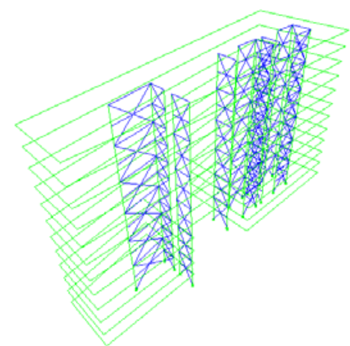


Figure A.2: Trial 1 bracing 3D view

A.5.2 Trial 2

Members:
W14x257 Columns
W18 x 143 Beams
HSS10x10x0.5 Bracing

Period:
Mode 1: 1.79s - torsion about south leg
Mode 2: 1.45s - Short direction (Y)

Max Deflections:
Seismic loaded in X direction = 0.78in
Seismic loaded in Y direction = 0.67in
Wind Loaded in Y direction = 1.38in
Wind loaded in X direction = 0.45in

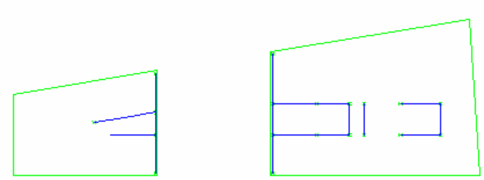


Figure A.3: Trial 2 bracing plan

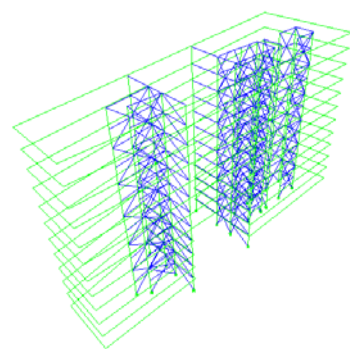


Figure A.4: Trial 2 bracing 3D view

A.5.3 Trial 3

Members:

W14x257 Columns
W18 x 143 Beams
W14 x 120 Bracing

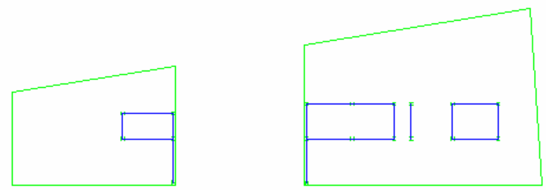


Figure A.5: Trial 3 bracing plan

Period:

Mode 1: 1.45s - torsion about south leg
Mode 2: 1.37s - Short direction (Y)

Max Deflections:

Seismic loaded in X direction = 0.37in
Seismic loaded in Y direction = 0.64in
Wind Loaded in Y direction = 1.13in
Wind loaded in X direction = 0.21in

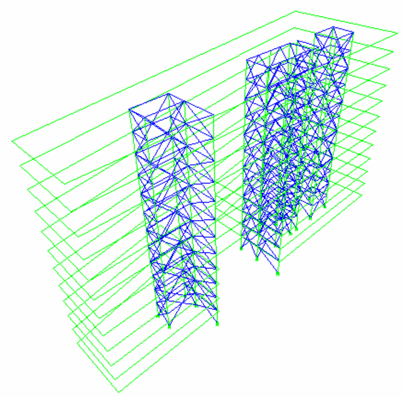


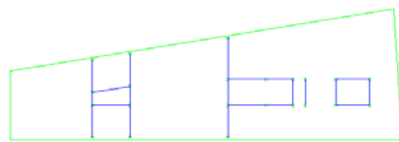
Figure A.6: Trial 3 bracing 3D view

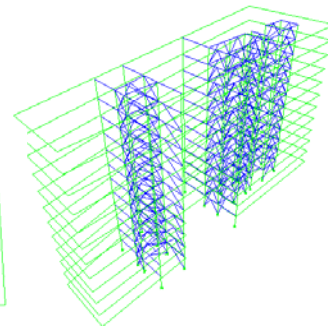
A.6 Effect of Various Parameters on Fundamental Natural Period of Braced Frames

- *Base model from which members were changed:*

  Columns: W14x283
  Beams: W24x68
  Braces: HSS10x10x.5

  T_{base} = 1.63s





Comparisons were made against the fundamental period of mode 1 only

- *Which members have greater effect on period when dramatically increased in size, columns, beams or braces?*

Member	**T (s)**	**% of base value**
Change all columns from W14x257 to W14x730	1.26	77
Change all beams from W22x86 to W27x539	1.53	94
Change all braces from HSS10x10x.5 to W14x132	1.44	88

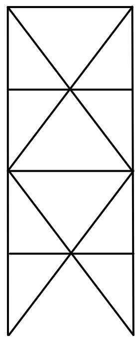
- *Which columns have greater effect on the period, upper or lower? Change columns along grid line 3 from W14x257 to W14x730:*

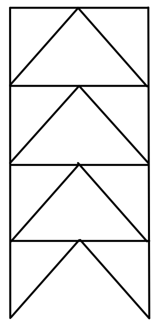
Loaciton	**T - Fundemental Period (s)**	**% of base value**
Bottom four columns	1.60	98
Middle four coulumns	1.62	99
Top four columns	1.64	101

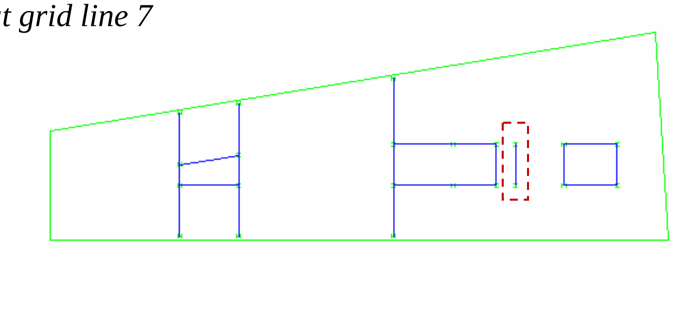
- *Hypothetically see what happens if the 5 story opening in the center of the façade were not there.*

Diaphragm configuration	**T (s)**	**% of base**
Single diaphragm at base	1.63	100

- *How does the fundamental period change with brace configuration? Compare inverted chevron bracing vs. cross bracing at grid line 7*







Brace Configuration	T (s)	% of base value
Cross	1.63	100
Chevron	1.64	101

- *Which column locations have greater effect on the period*? Change bottom four columns from W14x257 to W14x730 at each grid line.

Grid line Location (Center two column rows)	T - Fundemental Period (s)	% of base value
3	1.6043	98.42
4	1.6046	98.44
5	1.6250	99.69
6	1.6153	99.10
6.8	1.6073	98.61
7	1.5909	97.60
7.7	1.5462	94.85
8.8	1.5460	94.85

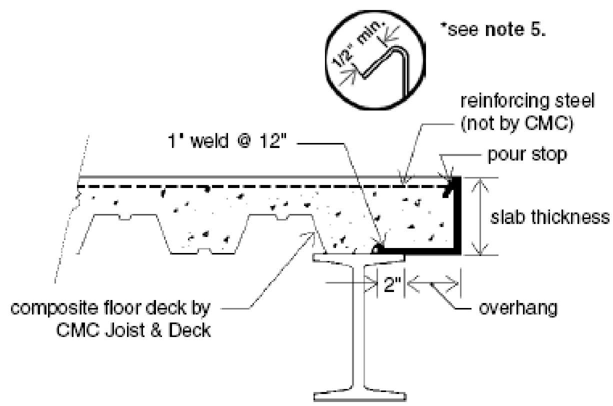
- *How much do the moment frames contribute to lowering the fundamental period? How does this performance weigh against the additional cost of the moment frames?*

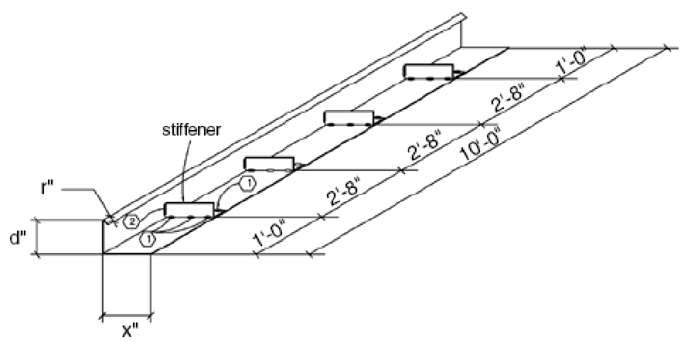
Frame Configuration	T (s)	% of base
Without Moment frames	1.81	111

A.7 Deck Details

Typical details of pour stop. The steel angle would also need to support curtainwall façade loads.

Images Provided by CMC Joist an d Deck





A.8 RS Means 2009 Cost Estimate

Composite Deck on Steel Frame

Quantity	Description	Crew	Daily Output	Labor Hours	Unit	Per Unit Cost				Extended Cost				Overhead and Profit / Per Unit				Extended Overhead and Profit			
						Material	Labor	Equipment	Total	Material	Labor	Equipment	Total	Mat.	Labor	Equip.	Total	Mat.	Labor	Equip.	Total
150000	Metal Decking	E4	3000	0.011	S.F.	2	0	0	2	279,000	69,000	6,000	354,000	2	1	0	3	306,000	126,000	7,500	439,500
660	Stuctural Steel	E6	14.2	9.014	Ton	2,300	380	132	2,812	1,518,000	250,800	87,120	1,855,920	2,525	670	145	3,340	1,666,500	442,200	95,700	2,204,400
1500	WWF 6 x 6	2 Rodm	31	0.516	C.S.F.	16	22	-	38	23,475	33,000	-	56,475	17	37	-	54	25,800	54,750	-	80,550
1960	Light Weight Concrete				C.Y.	146	-	-	146	286,160	-	-	286,160	161	-	-	161	315,560	-	-	315,560
14871	Studs - 3/4"	E10	935	0.017	Ea.	1	1	0	2	8,030	11,153	5,651	24,835	1	1	0	2	8,774	20,225	6,246	35,244
150000	Concrete Finish	C10C	1715	0.014	S.F.	-	0	0	1	-	73,500	3,000	76,500	-	1	0	1	-	109,500	4,500	114,000
											Total =	$	2,653,889.57						Total =	$	3,189,254

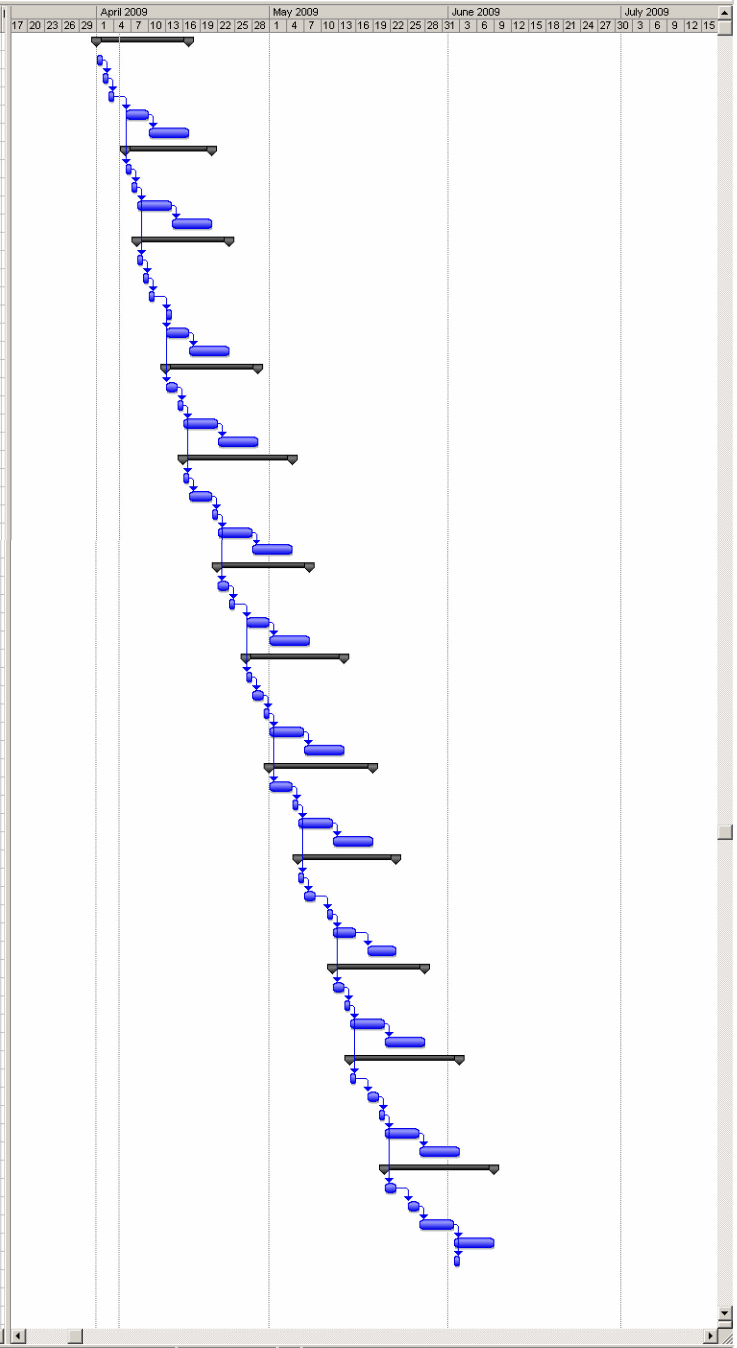
Precast Concrete Plank on Steel Frame

Quantity	Description	Crew	Daily Output	Labor Hours	Unit	Per Unit Cost				Extended Cost				Overhead and Profit / Per Unit				Extended Overhead and Profit			
						Material	Labor	Equipment	Total	Material	Labor	Equipment	Total	Mat.	Labor	Equip.	Total	Mat.	Labor	Equip.	Total
430	Stuctural Steel	E6	14.2	9.014	Ton	2,300	380	132	2,812	989,000	163,400	56,760	1,209,160	2,525	670	145	3,340	1,085,750	288,100	62,350	1,436,200
150000	Precast slab, 10" thick	C11	3600	0.02	S.F.	8	1	1	9	1,147,500	126,000	78,000	1,351,500	8	1	1	11	1,267,500	220,500	87,000	1,575,000
923	2" Concrete Topping				C.Y.	106	-	-	106	97,838	-	-	97,838	117	-	-	117	107,991	-	-	107,991
150000	Concrete Finish	C10D	2400	0.01	S.F.	-	0	0	0	-	52,500	6,000	58,500	-	1	0	1	-	78,000	7,500	85,500
2758	Shear Stud - 3/4"	E10	935	0.017	Ea.	1	1	0	2	1,489	2,069	1,048	4,606	1	1	0	2	1,627	3,751	1,158	6,536
											Total =	$	2,721,603.86						Total =	$	3,211,227

A.9 Precast Plank Construction Schedule

Microsoft Project - Schedule Precast

	Task Name	Duration	Start	Finish
1	**Level 2**	**12 days**	**Wed 4/1/09**	**Thu 4/16/09**
2	Erect Columns - Level 1-2	1 day	Wed 4/1/09	Wed 4/1/09
3	Erect Braces - Level 1	1 day	Thu 4/2/09	Thu 4/2/09
4	Erect Girders - Level 2	1 day	Fri 4/3/09	Fri 4/3/09
5	Erect/Grout Precast Plank - Level 2	4 days	Mon 4/6/09	Thu 4/9/09
6	2" Concrete Topping - Level 2	5 days	Fri 4/10/09	Thu 4/16/09
7	**Level 3**	**11 days**	**Mon 4/6/09**	**Mon 4/20/09**
8	Erect Braces - Level 2	1 day	Mon 4/6/09	Mon 4/6/09
9	Erect Girders - Level 3	1 day	Tue 4/7/09	Tue 4/7/09
10	Erect/Grout Precast Plank - Level 3	4 days	Wed 4/8/09	Mon 4/13/09
11	2" Concrete Topping - Level 3	5 days	Tue 4/14/09	Mon 4/20/09
12	**Level 4**	**12 days**	**Wed 4/8/09**	**Thu 4/23/09**
13	Erect Columns - Level 3-4	1 day	Wed 4/8/09	Wed 4/8/09
14	Erect Braces - Level 3	1 day	Thu 4/9/09	Thu 4/9/09
15	Erect Girders - Level 4	1 day	Fri 4/10/09	Fri 4/10/09
16	Erect Temporary Column	1 day	Mon 4/13/09	Mon 4/13/09
17	Erect/Grout Precast Plank - Level 4	4 days	Mon 4/13/09	Thu 4/16/09
18	2" Concrete Topping - Level 4	5 days	Fri 4/17/09	Thu 4/23/09
19	**Level 5**	**12 days**	**Mon 4/13/09**	**Tue 4/28/09**
20	Erect Braces - Level 4	2 days	Mon 4/13/09	Tue 4/14/09
21	Erect Girders - Level 5	1 day	Wed 4/15/09	Wed 4/15/09
22	Erect/Grout Precast Plank - Level 5	4 days	Thu 4/16/09	Tue 4/21/09
23	2" Concrete Topping - Level 5	5 days	Wed 4/22/09	Tue 4/28/09
24	**Level 6**	**13 days**	**Thu 4/16/09**	**Mon 5/4/09**
25	Columns Level 5-6	1 day	Thu 4/16/09	Thu 4/16/09
26	Erect Braces - Level 5	2 days	Fri 4/17/09	Mon 4/20/09
27	Erect Girders - Level 6	1 day	Tue 4/21/09	Tue 4/21/09
28	Erect/Grout Precast Plank - Level 6	4 days	Wed 4/22/09	Mon 4/27/09
29	2" Concrete Topping - Level 6	5 days	Tue 4/28/09	Mon 5/4/09
30	**Level 7**	**12 days**	**Wed 4/22/09**	**Thu 5/7/09**
31	Erect Braces - Level 6	2 days	Wed 4/22/09	Thu 4/23/09
32	Erect Girders - Level 7	1 day	Fri 4/24/09	Fri 4/24/09
33	Erect/Grout Precast Plank - Level 7	4 days	Mon 4/27/09	Thu 4/30/09
34	2" Concrete Topping - Level 7	5 days	Fri 5/1/09	Thu 5/7/09
35	**Level 8**	**13 days**	**Mon 4/27/09**	**Wed 5/13/09**
36	Columns Level 7-8	1 day	Mon 4/27/09	Mon 4/27/09
37	Erect Braces - Level 7	2 days	Tue 4/28/09	Wed 4/29/09
38	Erect Girders - Level 8	1 day	Thu 4/30/09	Thu 4/30/09
39	Erect/Grout Precast Plank - Level 8	4 days	Fri 5/1/09	Wed 5/6/09
40	2" Concrete Topping - Level 8	5 days	Thu 5/7/09	Wed 5/13/09
41	**Level 9**	**12 days**	**Fri 5/1/09**	**Mon 5/18/09**
42	Erect Braces - Level 8	2 days	Fri 5/1/09	Mon 5/4/09
43	Erect Girders - Level 9	1 day	Tue 5/5/09	Tue 5/5/09
44	Erect/Grout Precast Plank - Level 9	4 days	Wed 5/6/09	Mon 5/11/09
45	2" Concrete Topping - Level 9	5 days	Tue 5/12/09	Mon 5/18/09
46	**Level 10**	**13 days**	**Wed 5/6/09**	**Fri 5/22/09**
47	Columns Level 9-10	1 day	Wed 5/6/09	Wed 5/6/09
48	Erect Braces - Level 9	2 days	Thu 5/7/09	Fri 5/8/09
49	Erect Girders - Level 10	1 day	Mon 5/11/09	Mon 5/11/09
50	Erect/Grout Precast Plank - Level 10	4 days	Tue 5/12/09	Fri 5/15/09
51	2" Concrete Topping - Level 10	5 days	Mon 5/18/09	Fri 5/22/09
52	**Level 11**	**12 days**	**Tue 5/12/09**	**Wed 5/27/09**
53	Erect Braces - Level 10	2 days	Tue 5/12/09	Wed 5/13/09
54	Erect Girders - Level 11	1 day	Thu 5/14/09	Thu 5/14/09
55	Erect/Grout Precast Plank - Level 11	4 days	Fri 5/15/09	Wed 5/20/09
56	2" Concrete Topping - Level 11	5 days	Thu 5/21/09	Wed 5/27/09
57	**Level 12**	**13 days**	**Fri 5/15/09**	**Tue 6/2/09**
58	Columns Level 11-Roof	1 day	Fri 5/15/09	Fri 5/15/09
59	Erect Braces - Level 11	2 days	Mon 5/18/09	Tue 5/19/09
60	Erect Girders - Level 12	1 day	Wed 5/20/09	Wed 5/20/09
61	Erect/Grout Precast Plank - Level 12	4 days	Thu 5/21/09	Tue 5/26/09
62	2" Concrete Topping - Level 12	5 days	Wed 5/27/09	Tue 6/2/09
63	**Roof**	**13 days**	**Thu 5/21/09**	**Mon 6/8/09**
64	Erect Braces - 11	2 days	Thu 5/21/09	Fri 5/22/09
65	Erect Girders - Roof	2 days	Mon 5/25/09	Tue 5/26/09
66	Erect/Grout Precast Plank - Roof	4 days	Wed 5/27/09	Mon 6/1/09
67	2" Concrete Topping - Level 7	5 days	Tue 6/2/09	Mon 6/8/09
68	Remove Temporary Column	1 day	Tue 6/2/09	Tue 6/2/09



A.10 Composite Deck Construction Schedule

Microsoft Project - Schedule

	Task Name	Duration	Start	Finish	Predecessors	Resource
1	**Level 2**	**16 days**	**Wed 4/1/09**	**Wed 4/22/09**		
2	Erect Columns - Level 1-2	1 day	Wed 4/1/09	Wed 4/1/09		
3	Erect Braces - Level 1	1 day	Thu 4/2/09	Thu 4/2/09	2	
4	Erect Beams/Girders - Level 2	2 days	Fri 4/3/09	Mon 4/6/09	3	
5	Deck/Shear Studs - Level 2	5 days	Tue 4/7/09	Mon 4/13/09	4	
6	Pour/Finish Concrete - Level 2	7 days	Tue 4/14/09	Wed 4/22/09	5	
7	**Level 3**	**15 days**	**Tue 4/7/09**	**Mon 4/27/09**		
8	Erect Braces - Level 2	1 day	Tue 4/7/09	Tue 4/7/09	4	
9	Erect Beams/Girders - Level 3	2 days	Wed 4/8/09	Thu 4/9/09	8	
10	Deck/Shear Studs - Level 3	5 days	Fri 4/10/09	Thu 4/16/09	9	
11	Pour/Finish Concrete - Level 3	7 days	Fri 4/17/09	Mon 4/27/09	10	
12	**Level 4**	**16 days**	**Fri 4/10/09**	**Fri 5/1/09**		
13	Erect Columns - Level 3-4	1 day	Fri 4/10/09	Fri 4/10/09	9	
14	Erect Braces - Level 3	1 day	Mon 4/13/09	Mon 4/13/09	13	
15	Erect Beams/Girders - Level 4	2 days	Tue 4/14/09	Wed 4/15/09	14	
16	Erect Temporary Column	1 day	Thu 4/16/09	Thu 4/16/09	15	
17	Deck/Shear Studs - Level 4	5 days	Thu 4/16/09	Wed 4/22/09	15	
18	Pour/Finish Concrete - Level 4	7 days	Thu 4/23/09	Fri 5/1/09	17	
19	**Level 5**	**15 days**	**Thu 4/16/09**	**Wed 5/6/09**		
20	Erect Braces - Level 4	1 day	Thu 4/16/09	Thu 4/16/09	15	
21	Erect Beams/Girders - Level 5	2 days	Fri 4/17/09	Mon 4/20/09	20	
22	Deck/Shear Studs - Level 5	5 days	Tue 4/21/09	Mon 4/27/09	21	
23	Pour/Finish Concrete - Level 5	7 days	Tue 4/28/09	Wed 5/6/09	22	
24	**Level 6**	**17 days**	**Tue 4/21/09**	**Wed 5/13/09**		
25	Columns Level 5-6	1 day	Tue 4/21/09	Tue 4/21/09	21	
26	Erect Braces - Level 5	1 day	Wed 4/22/09	Wed 4/22/09	25	
27	Erect Beams/Girders - Level 6	3 days	Thu 4/23/09	Mon 4/27/09	26	
28	Deck/Shear Studs - Level 6	5 days	Tue 4/28/09	Mon 5/4/09	27	
29	Pour/Finish Concrete - Level 6	7 days	Tue 5/5/09	Wed 5/13/09	28	
30	**Level 7**	**16 days**	**Tue 4/28/09**	**Tue 5/19/09**		
31	Erect Braces - Level 6	1 day	Tue 4/28/09	Tue 4/28/09	27	
32	Erect Beams/Girders - Level 7	3 days	Wed 4/29/09	Fri 5/1/09	31	
33	Deck/Shear Studs - Level 7	5 days	Mon 5/4/09	Fri 5/8/09	32	
34	Pour/Finish Concrete - Level 7	7 days	Mon 5/11/09	Tue 5/19/09	33	
35	**Level 8**	**17 days**	**Mon 5/4/09**	**Tue 5/26/09**		
36	Columns Level 7-8	1 day	Mon 5/4/09	Mon 5/4/09	32	
37	Erect Braces - Level 7	1 day	Tue 5/5/09	Tue 5/5/09	36	
38	Erect Beams/Girders - Level 8	3 days	Wed 5/6/09	Fri 5/8/09	37	
39	Deck/Shear Studs - Level 8	5 days	Mon 5/11/09	Fri 5/15/09	38	
40	Pour/Finish Concrete - Level 8	7 days	Mon 5/18/09	Tue 5/26/09	39	
41	**Level 9**	**16 days**	**Mon 5/11/09**	**Mon 6/1/09**		
42	Erect Braces - Level 8	1 day	Mon 5/11/09	Mon 5/11/09	38	
43	Erect Beams/Girders - Level 9	3 days	Tue 5/12/09	Thu 5/14/09	42	
44	Deck/Shear Studs - Level 9	5 days	Fri 5/15/09	Thu 5/21/09	43	
45	Pour/Finish Concrete - Level 9	7 days	Fri 5/22/09	Mon 6/1/09	44	
46	**Level 10**	**17 days**	**Fri 5/15/09**	**Mon 6/8/09**		
47	Columns Level 9-10	1 day	Fri 5/15/09	Fri 5/15/09	43	
48	Erect Braces - Level 9	1 day	Mon 5/18/09	Mon 5/18/09	47	
49	Erect Beams/Girders - Level 10	3 days	Tue 5/19/09	Thu 5/21/09	48	
50	Deck/Shear Studs - Level 10	5 days	Fri 5/22/09	Thu 5/28/09	49	
51	Pour/Finish Concrete - Level 10	7 days	Fri 5/29/09	Mon 6/8/09	50	
52	**Level 11**	**16 days**	**Fri 5/22/09**	**Fri 6/12/09**		
53	Erect Braces - Level 10	1 day	Fri 5/22/09	Fri 5/22/09	49	
54	Erect Beams/Girders - Level 11	3 days	Mon 5/25/09	Wed 5/27/09	53	
55	Deck/Shear Studs - Level 11	5 days	Thu 5/28/09	Wed 6/3/09	54	
56	Pour/Finish Concrete - Level 11	7 days	Thu 6/4/09	Fri 6/12/09	55	
57	**Level 12**	**17 days**	**Thu 5/28/09**	**Fri 6/19/09**		
58	Columns Level 11-Roof	1 day	Thu 5/28/09	Thu 5/28/09	54	
59	Erect Braces - Level 11	1 day	Fri 5/29/09	Fri 5/29/09	58	
60	Erect Beams/Girders - Level 12	3 days	Mon 6/1/09	Wed 6/3/09	59	
61	Deck/Shear Studs - Level 10	5 days	Thu 6/4/09	Wed 6/10/09	60	
62	Pour/Finish Concrete - Level 10	7 days	Thu 6/11/09	Fri 6/19/09	61	
63	**Roof**	**16 days?**	**Thu 6/4/09**	**Thu 6/25/09**		
64	Erect Braces - 11	1 day	Thu 6/4/09	Thu 6/4/09	60	
65	Erect Beams/Girders -Roof	3 days	Fri 6/5/09	Tue 6/9/09	64	
66	Deck/Shear Studs -Roof	5 days	Wed 6/10/09	Tue 6/16/09	65	
67	Pour/Finish Concrete -Roof	7 days	Wed 6/17/09	Thu 6/25/09	66	
68	Remove Temporary Column	1 day?	Wed 6/17/09	Wed 6/17/09	66