800 NORTH GLEBE

Arlington, VA

Final Report

Ryan Johnson

Structural Option

Advisor: Dr. Linda Hanagan

Spring 2010

800

NORTH GLEBE

GENERAL BUILDING DATA

Location: Arlington, VA
Occupant: Up for Lease
Occupancy Type: Mixed-Use Office
Size: 316,000 SF
10 Stories + 1 Penthouse and
3 Below Grade Parking Levels
Completion Date: 2011
Delivery Method: Design-Bid-Build
Contract Type: At Risk
Project Cost: $62 Million

PROJECT TEAM

Owner: The JBG Companies
Architect: Cooper Carry
Landscape Architect: Bowman Construction
Structural Engineer: Structura
Civil Engineer: Bowman Construction
MEP: Girard Engineers
Contractor: Clark Construction Group LLC

MEP SYSTEMS

-9 VAV 30,000-37,000 cfm airhanding units: 1 per level
-3 800 gpm cooling towers with one chilled water free cooling heat exchanger
- 480/277 V electrical distribution system carried by a 3000 A busway
-Surface mounted and recessed fluorescents, and wall mounted CFLs fixtures throughout the building
-Occupancy sensors installed on all office levels
-450 KW emergency generator powering lights, fire pumps, sump pumps and stairwell pressurization fans

ARCHITECTURE

-Three sail-like sweeping glass curtain walls
-Precast concrete panels, decorative stone and metal cladding adorn the fascade
-Vertical and horizontal precast bands create sight lines
-Building geometry defined by radial lines and circles
-Ground level reatail offering 14'-6" ceiling heights with floor-to-ceiling glazing
-Diamond expression decorative composite metal canopy with backlit glass over main retail entrance
-Building setbacks located at levels 4, 6 and 8
-Numerous amentities located on-site: Retail, restaurant, cafe, fitness center.
-Designed for LEED Gold Certification

STRUCTURAL SYSTEMS

-Three levels of substructure used for parking
-Post-tensioned girders with 9" thick one-way slab allowing for 30' x 46' open bay design on levels 2 - 10
-10.5" two-way slabs used for building stepouts below level 6
-Lateral resistance provided by two 12" think "C" shaped shear walls at the buildings core
-Column size variation from 12" x 24" to 30" x 30" with a singular round 36" dia. column at entrance
-Foundations range in size from 4'-0" square up to 14'-0" square with caissons supporting where aplicable
-Lowest level parking garage has a 4" thick SOG



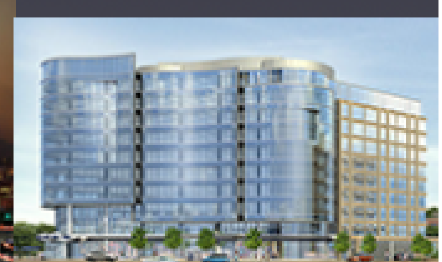

Table of Contents

Executive Summary

The thesis study performed for 800 North Glebe examined the design implications of changing the current slab system to a two-way post-tensioned system and the affects it will have on the lateral force resisting system. To employ an alternative slab system in the building, the bay sizes were reduced from a 30’-0”x46’-0” grid to a 30’-0”x23’-0” grid, thus increasing the total column quantity. The column sizes were reduced on the upper seven levels from 30”x30” to 24”x24”. Column grids aligned one another in the superstructure levels, but the interface to the below level parking garage required sloping a row of columns on the first level and adding a corbel-transfer beam system on the first subgrade floor.

The existing system consists of a 9” mildly reinforced one-way slab cast over wide-shallow post-tensioned girders with two “C” shaped core shear walls resisted the lateral load imposed on the structure. The structural depth redesign implemented two-way post-tensioning of an 8” flat slab with banded tendons in the east-west direction and distributed tendons in the north-south direction. Tendons banded over the column strip were analyzed to act as beams within a concrete moment frame. This permitted the lateral force resisting system in the east-west direction to be analyzed as a duel system; a concrete moment frame along with the shear wall core. However, since the code does not specifically address post-tensioned systems as lateral resisting, doctoral research papers were consulted for analysis and recommendations.

Changing the column grid unquestionably affected the architectural floor plans of the building. The existing layout was studied to determine the proper size of rentable offices and workstations and great effort was made to keep the same ratios. So as to not diminish the number of offices available, interior partition walls being moved around were kept to a minimum. Final floor plan redesigns maintained the same quantity of workstations and offices, while meeting all applicable egress codes.

A large part of the buildings appeal is the glass curtain wall sail which spans the entire building, from the ground level retail space to the tenth level offices. Calculated wind pressures from the depth study were used in analyses and it was determined that the 7’-7 ¼” x 5’-0” glass plys and the aluminum mullions were adequately sized to meet all deflection criteria.

The second breadth topic conducted was construction management sequencing and cost analysis of the structural system for both the existing design and the thesis redesign. Because 800 North Glebe is a spec office building, immediate revenue upon completion is a primary concern. The original system was concluded to be more time and cost effective; taking 43 days as compared to 94 days, and costing nearly $684,000 less.

Acknowledgements

The author wishes to extend his thanks to the following professionals, AE faculty and individuals for their guided assistance and generosity throughout the year with this thesis project.

Structura

Mark Erdman
Ryan Sarazen
Caitlyn Mueller
The entire Structura staff

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Adam Peters

Cooper Carry

Katie Peterschmidt

The Pennsylvania State University

Dr. Linda Hanagan
Professor M. Kevin Parfitt
Professor Robert Holland
Professor Paul Bowers
The entire AE faculty and staff

Holbert Apple Associates, Inc.

Mr. Richard Apple, P.E.

He would also like to say a special thank you to his parents, Eric and Margy Johnson, and his family and friends. Everything he has accomplished over the past five years could not have been possible without your love and support.

Introduction

Located in the Ballston district of Arlington, VA, 800 North Glebe offers class-A mixed-use office space and one level of public space. Three levels of below grade parking are shared between 800 N. Glebe and 900 N. Glebe: Virginia Tech's new research facility. Vertical transportation of stairways and elevators bring you from the garage to the large open retail and gathering space. Levels two through ten provide open plan office space. Column spacing of 30' x 46' allows for 30,000 square foot floor plates with 9'-0" floor-to-ceiling heights. Building setbacks are located at levels four, six, and eight to aesthetically vary the building and offer different office layouts as seen in Figures 1 through 4.

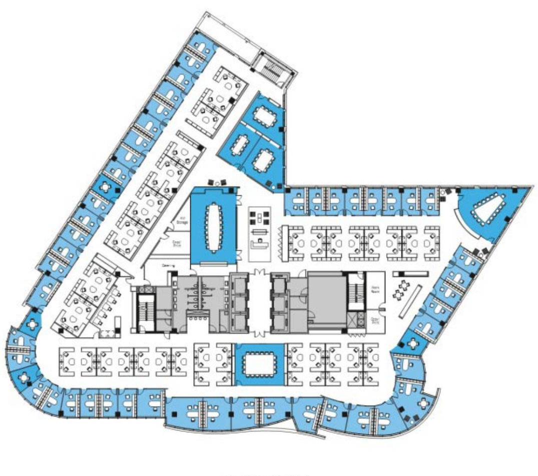

Figure 1: Floor Level 3

Figure 2: Floor level 5

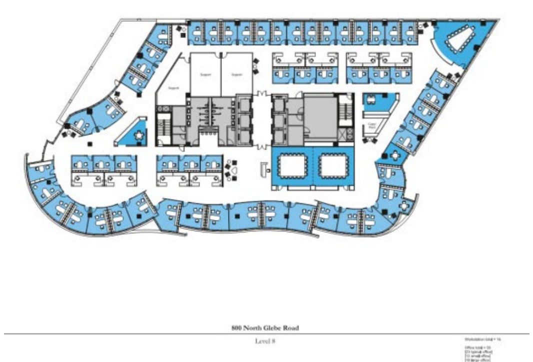

Figure 3: Floor level 8

Figure 4: Floor level 10

Architectural Overview

800 North Glebe is a 10-story 316,000 square-foot iconic commercial building. Retail and public gathering spaces are located at street level in the 2-story lobby of the building. The remaining nine levels will provide class-A mixed-use offices. 800 North Glebe was designed for LEED Gold Certification by utilizing numerous strategies to minimize its carbon footprint.

Figure 3: South-East Facade

Innovative sustainable and responsible design practices are one of the designer's primary goals. Integration of sustainability and every day design by minimizing the carbon footprint, balancing energy, resources and feasibility all went into design on 800 North Glebe. In accordance with the U.S. Green Building Council's Leadership in Energy and Environmental Design, the owner has a goal to achieve LEED Gold Certification, which the designers fulfilled. LEED Gold Certification requires the design to attain at least 34 out of 61 possible points.

The 10-story façade, created by three sail-like sweeping glass curtain walls, accentuate the sight lines of the building. Radial lines and circles were widely used to define the crown and drum feature of level one and the sail feature of the remaining levels. Refer to Figure 5, 6 and 7 for visual representation of façade features.

Figure 4: Sail Feature

Retail and community spaces on the ground level offer 14'-6" ceiling heights with floor-to-ceiling glazing. Over the main building entrance, there is a diamond expression decorative composite metal canopy with a plaster soffit and sunguard ultrawhite laminated backlit glass as shown in Figures 5 and 6. Offices on the remaining levels offer 9'-0" floor-to-ceiling heights.

Figure 5: Front View

Three types of Architectural precast panels, metal cladding and glazing will adorn 800 North Glebe's façade. The large sail-like curtain wall consists of Viracon VRE 1-46 on insulated heat strengthened vision and spandrel glass with PVD finished custom color composite metal mullions. Along the street level, one will find a variety of stone, metal and glazing. These include Oconee granite with a polished finish at the base, insulated spandrel glass, precast concrete panels with a light sandblast finish and PVDF finished aluminum louvers.

Figure 6: Canopy Over Main Entrance

Vertical bands rising up the building are made of precast concrete panels with a medium sandblast finish while horizontal bands consist of exposed aggregate finished panels. Other glazing found on the building is sunguard supernatural-68 on ultrawhite insulated glass and Viracon VRE 1-46 on insulated punch vision glass.

Protection from the elements on the roof is provided by the composite roof membrane. The composite consists of R-19 high density rigid insulation, protection board, and fully adhered 60 mil TPO membrane on top of a structural concrete slab. Where the roof system terminates at a curtain wall, fluid applied waterproofing is placed atop drainage board.

Existing Structural System Overview

Foundation

Geotechnical studies performed by ATC Associated Inc., reported site and subsurface conditions encountered and the following information details their geotechnical recommendations for the project. Three levels of parking make up the substructure of 800 N. Glebe, at roughly thirty feet below existing grade. Groundwater levels were encountered at depths ranging from approximately 22' to 37' below the existing ground surface.

Gravel, sand, silt and clay comprise the underlain site between existing elevation and bedrock, located 35.7' to 58.8' below existing ground surfaces. The analysis indicated that spread footing foundations bearing on the dense residual soil would be feasible for a majority of the structure. However, under interior wall, the foundation shall be designed with minimum widths of 18" to 24", where many are designed to be 12'x12'x6'. Below the ground level lobby area, caissons needed to be a minimum diameter of 60" and a mat foundation would be sufficient when designed for a maximum allowable bearing pressure of 3.5 ksf.

3 ksi normal-weight concrete (NWC) is used for the foundations and interior slab on grade, the garage slab-on-grade (SOG) uses 4.5 ksi NWC and the cellar columns are composed of 4 ksi and 8 ksi. Reinforcing varies in size throughout the footings and caissons, depending on thickness. A large mat foundation is located below the shearwalls at a thickness of 6'-0".

Superstructure

A 4" thick SOG is located near the main entrance of the retail lobby. A 24" wide x 30" deep turndown, reinforced with #5s, surrounds the perimeter of the SOG. The ground level retail includes a 10" thick one-way slab with 10'-0"x10'-0"x5.5" drop panels support around the columns for punching shear resistance. Plaza slabs are 12" thick with 10'-0"x10'-0"x12" drop panels. Concrete strengths for the ground level include 3 ksi (SOG), 5 ksi (plaza slabs and framed interior slabs) and 4, 6 & 8 ksi (superstructure columns). Reinforcement for the SOG includes 6x6-10/10 welded-wire-fabric, while the one-way slab is reinforced with #5, #6 and #7s.

The remaining levels of the superstructure employ a one-way slab over post tensioned girders for the majority of the slab area which is represented as yellow in Figure 7. Girders range in size from 48" wide x 18" thick to 72" wide x 20" deep. Post tension tendons are ½" diameter with .153 square in. area low-relaxation strands with an ultimate strength of 270 ksi. A minimum of two post tension cables pass through the column reinforcement in the direction of the girder. This allows for continuous force distribution from one span to another, spanning the East/West directions. For levels two through six, two-way mildly reinforced slabs, colored cyan in Figure 7. Two-way slabs are 10.5" thick and are generally reinforced with #5 @ 10" in both directions. Drop panels in these areas are typically 10'-0"x10'-0"x7.5" to alleviate punching shear at the columns. Slabs over the 36" diameter column are 12" thick with #5 @ 12" parallel to the girder and #6 @10" perpendicular to the girders, due to the cantilever action.

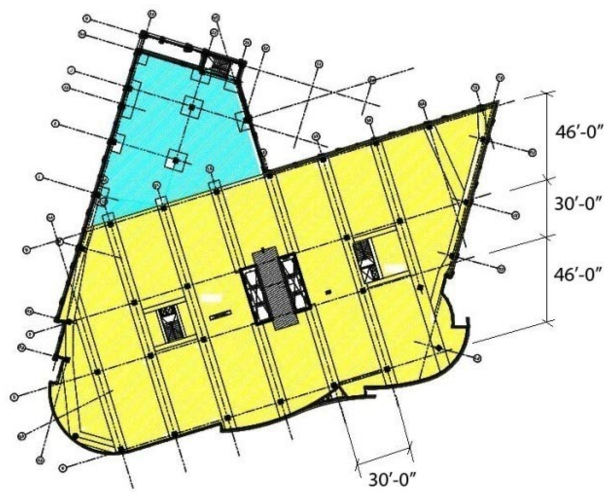

Figure 7: Slab Type Layout

Though the primary supporting material is concrete, steel shapes are used throughout the building for additional support. Elevator openings are supported by S8x18.4. HSS 6x3x1/4 were used as beams for additions support of shaft walls and W12x16s were used as elevator safety beams below the slabs. Steel allows for easy attachment of elevator rails and differential shaft openings.

Lateral System

Shear walls in the core of the building provide the entire lateral support, as designed by the engineer, as seen in Figure 8. However, since the building primarily consists of reinforced concrete columns and post-tensioned concrete beams, part of the lateral forces could be distributed through these members, as seen in Figure 9 where columns are red and beams cyan.

Two 12"thick "C" shaped walls, 31.83' long East/West and 9.58' long North/South per each "C", encase the elevator banks and are reinforced with #4 horizontally and #5 vertically. From the sixth floor down, walls running North/South are specially reinforced three feet from each end with #7 and #8 rebar. All of the shear walls use concrete with a compressive strength of f'_c= 6 ksi. Building drift criteria for wind loads is L/400 or 3/8" inter-story drift at typical floors (12'-9" floor-to-floor) and for seismic loads is L/76 or 2" inter-story drift at typical floors (12'-9" floor-to-floor).

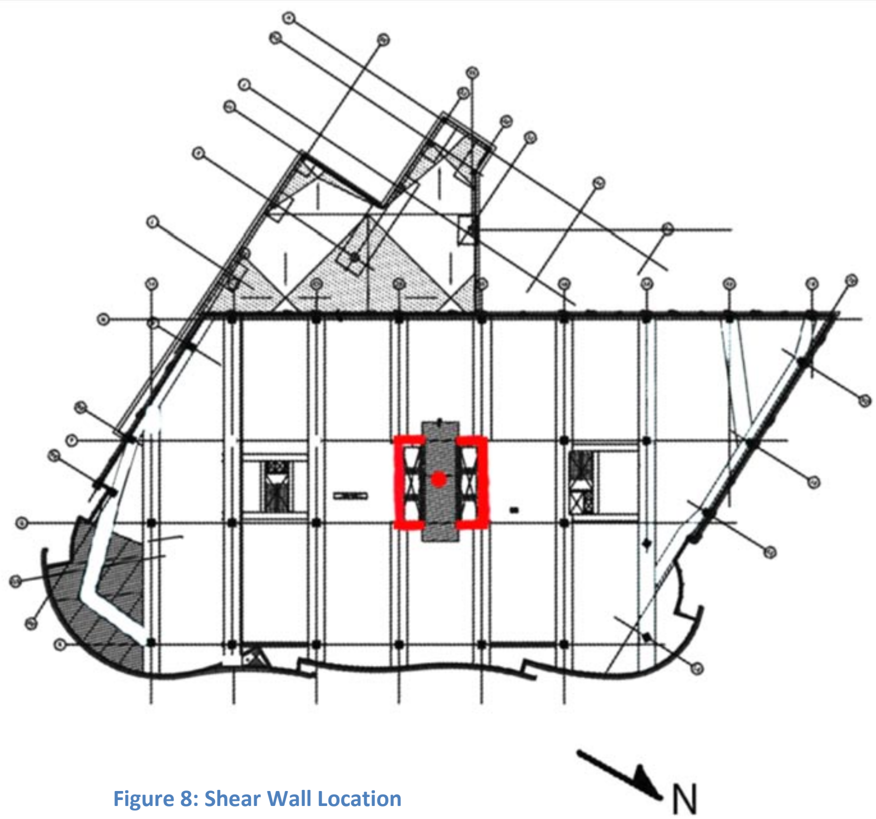

Figure 8: Shear Wall Location

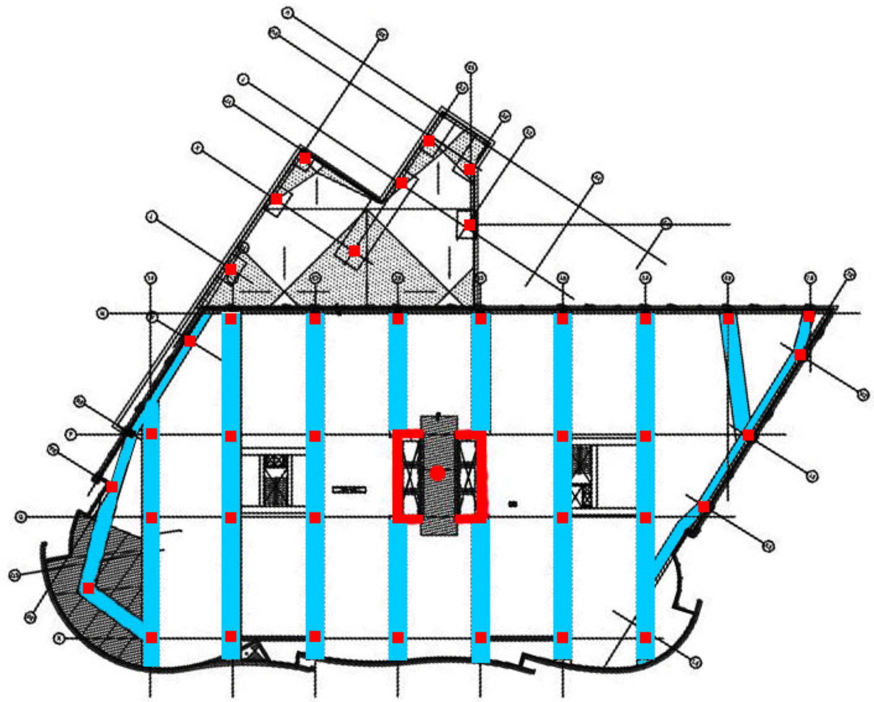

Figure 9: Lateral System Alternative

The columns throughout the building are primarily 30”x30” with 72” wide by 18” deep post-tensioned beams tying into them. Though these members were not designed to take the primary lateral force, they will transfer loads through themselves, and therefore have some affect on the lateral system. A 9” normally reinforced concrete slab transfers loads to the post-tensioned beams and act as a rigid diaphragm for the structure. Also, post-tensioned tendons surround the building slab edges to reduce slab deflection, but will also help transfer lateral forces. These are not marked above but are around the entire one-way slab perimeter.

Existing Structural System Analysis

Deflection Criteria

Horizontal Framing Deflections:

- **Live Load**
  - **< L/600 or ½”**

- **Total Load Excluding Self Weight**
  - **< L/480 or ¾”**

Lateral Drift:

- **Wind Loads**
  - **< L/400 or 3/8”**

- **Seismic Loads**
  - **< L/76 or 2”**

Main Structural Elements Supporting Components and Cladding:

- **At Screenwalls**
  - **< L/240 or ¾”**

- **At Floors Supporting Curtainwalls**
  - **< L/600 or ½”**

- **At Roof Parapet Supporting Curtainwalls**
  - **< L/600 or ½”**

- **At Non-Brittle Finishes**
  - **< L/240**

Materials

Steel:

Wide Flange	**50 ksi (A992)**
Plates, Channels, Angles and Bars	**36 ksi (A36)**
Round Pipes	**42 ksi (A53 Grade B)**
HSS Rectangular or Square Tubing	**46 ksi (A500 Grade B)**
HSS Round Tubing	**42 ksi (A500 Grade B)**
Bolts	**36/45 ksi (A325 or A490)**
Anchor Rods	**(F1554 Grade 55)**
Weld Strength	**70 ksi (E70XX)**

Concrete:

Foundations, Int. Slab on Grade	**f'c = 3000 psi**
Interior Walls	**f'c = 5000 psi**
Ext. Slab of Grade, Pads, Garage SOG	**f'c = 4,500 psi**
Garage and Plaza Slabs, Framed Int. Slabs	**f'c = 5000 psi**
Ext. Walls, Beams, Basement Walls	**f'c = 4000 & 5000 psi**
Deck Supported Slabs	**f'c = 3500 psi**
Cellar Columns	**f'c = 4000 & 8000 psi**
Superstructure Columns	**f'c = 4000, 8000 & 6000 psi**
Shear Walls	**f'c = 6000 psi**
Masonry	**f'm = 1500 psi**

Reinforcement:

Longitudinal Bars	**60 ksi (A615)**
Deformed Bars (Ties)	**60 ksi (A615)**
Welded Wire Mesh	**(A185)**

Post Tensioning:

Tendons	**270 ksi (A416)**

Cold Formed Steel:

20 Gage	**33 ksi (A653)**
18 Gage	**33 ksi (A653)**
16 Gage	**50 ksi (A653)**

Note: Material strengths are based on American Society for Testing and Materials (ASTM) standard rating.

Design Process – Gravity System

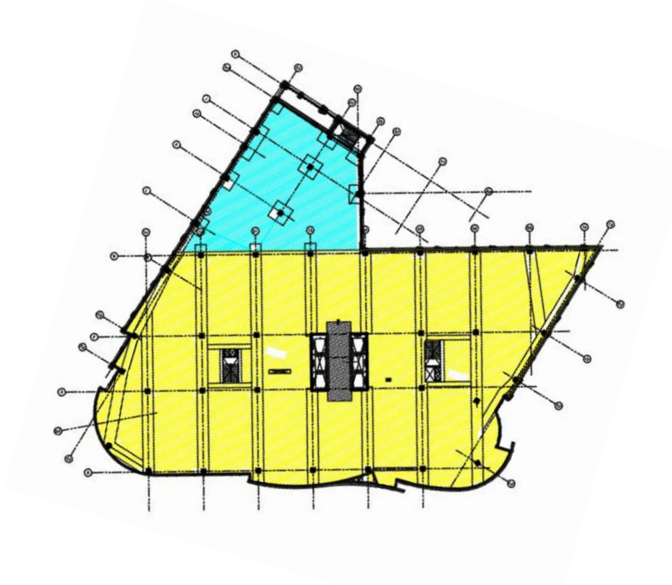

Figure 10: Slab Systems

All of the levels of the superstructure employ a one-way slab system over post-tensioned girders, colored yellow in Figure 10. Also, levels two through six have an extended area where a two-way mildly reinforced slab is implemented, colored cyan in Figure 10. Both slab analysis calculations may be referenced in prior technical report's appendices.

Slab thickness is 9" with concrete compressive strength of f'_c= 5000 psi. ACI 318-8, *Approximate Method of Frame Analysis*, was the design method utilized because the slab had met all of the provisions. Construction of the slab and girders was determined to be cast monolithically. Because of these finding, the strip, colored cyan in Figure 11, was analyzed as a solid slab with both ends continuous. The amount of steel reinforcement in the slab was found to be #6 @10" top reinforcing and #5 @10" bottom reinforcing.

A post-tensioned girder was examined using the simplified method of load balancing provided by Mr. Richard Apple of Holbert Apple Associates. The girder being analyzed is shaded cyan in Figure 12, which spans between 4 columns. The two outer spans, from column face to column face, are of equal length (46'-0") while the interior span is 14' shorter (30'-0"). Preliminary span-depth ratios and tendon stress were performed. More in-depth calculations of the existing gravity system may be found in the tech reports of semester one.

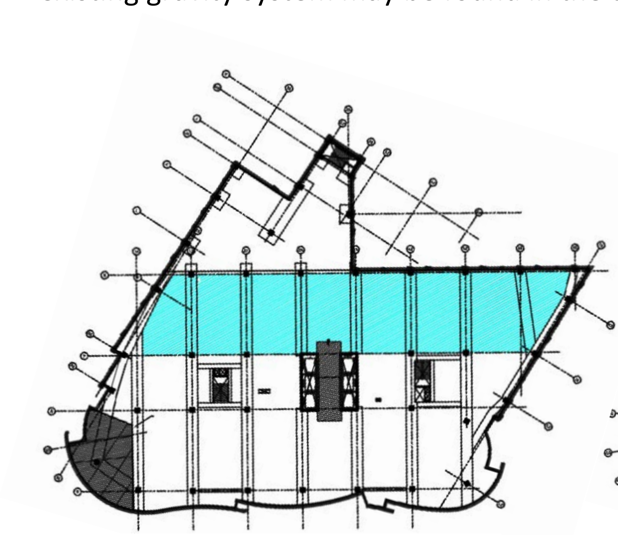

Figure 12: One-way Slab Strip

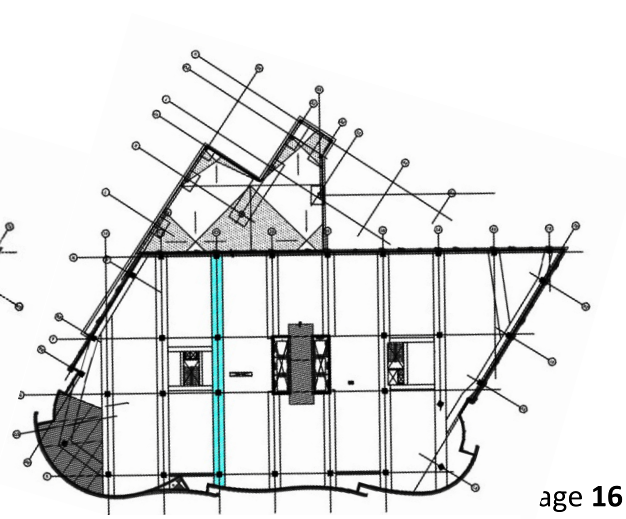

Figure 11: Post-tensioned Girder

Design Process – Lateral System

Lateral analysis was performed with both wind and seismic loading in mind. Determination of which lateral loads will control the design, how the lateral loads are distributed among load resisting elements in a logical load path, and verify the lateral load resisting system have been sufficiently designed for strength and serviceability. Preliminary hand calculations were performed to investigate and determine the relative stiffness of each lateral load resisting shear wall. It was concluded that each shearwall distributed the forces uniformly in each respective direction. Shearwall relative stiffness was then used to calculate the structural center-of-rigidity (COR). Two computer models were created in ETABS to compare and verify hand calculations, one with only the shearwalls and one with the entire structure modeled. Wind and seismic loads were applied to the building and due to the nonuniform and unique smooth curved shape of 800 North Glebe, it was found that when looking at strength design, wind created greater loads. When looking at serviceability issues, seismic created greater concerns. However, thesis calculations were performed with the assumption that wind loading would play a greater role in lateral system design because of the significant surface area of the façade. This led to ASCE 7-05 load case 6 (0.9D + 1.6W) being used for analysis.

Wind

A box was drawn around the building shape, along the principle lateral system axis, as seen in Figure 13. The size of the box was approximated to enclose a majority of the building and to determine the center-of-pressure. It can be seen that the lower side of the building is perpendicular to the applied wind load. Because of this, the wind forces in this direction are larger than the wind forces acting on the left side of the building, but both faces experience significantly large wind pressures. Lateral load calculations discussed later will determine the extent of the forces increase.

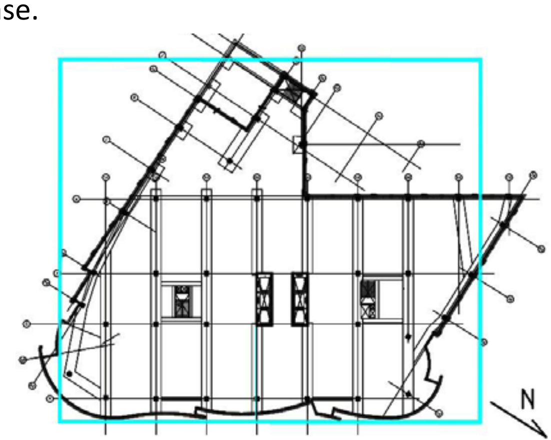

Figure 13: Initial Wind Load Determination

Seismic

Seismic calculations of 800 North Glebe were based upon ASCE 7-05 for thesis design. The engineering firm had used ASCE 7-02 / IBC 2003 and the 2003 Virginia Uniform Statewide Building Code to calculate the base shear from the equivalent lateral force analysis procedure. Design criteria variables were used to determine story forces at each level, story shear at each level, and base shear. The model output for maximum modal period of vibration was found to be 5.6079 seconds. However, this value was not used as the fundamental period because it means the structure is more flexible than what value the code permits for fundamental period of vibration, T_aC_u = 1.868s. A lower period of vibration being used for design assumes the lateral resisting structural elements are more rigid and therefore, must be designed for the larger forces. When only the shear walls are analyzed compared to the entire structure, as seen in Figure 14 respectively, a larger period was found, meaning the structure is less stiff. The largest difference can be found in the building rotation (torsion). Since the lateral shearwall core is centrally located with the majority of the building spread over a large slab area causing the building to significantly rotate. The columns and beams are spread throughout the structure, increasing the stiffness and reducing the torsional effects.

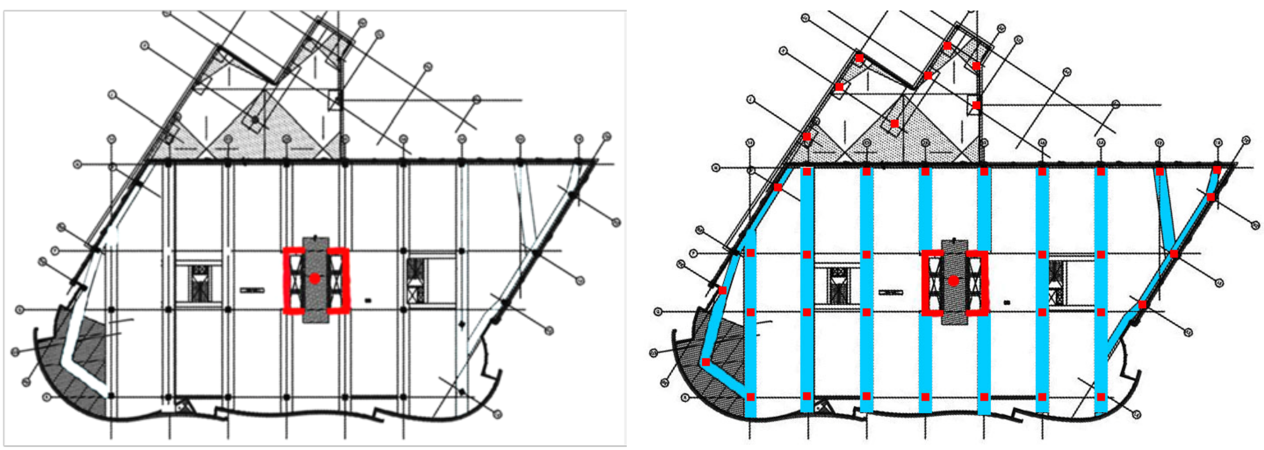

Figure 14: Shear Wall vs. Entire Structure

Problem Statement

The first three technical assignments had found that current slab system and lateral structural system are capable of resisting applied loads to the building. However, because of the building shape and setbacks, two different slab systems are used throughout 800 North Glebe, where there are a total of four different slab thicknesses, using a variety of concrete strengths. Also, because of the large bay sizes, 30'x 46' typical, perimeter post-tensioned beams were added to help reduce slab edge deflections where the glass curtain wall system is attached. With this information in mind, the proposed goal is to reduce bay sizes and implement a uniform slab thickness. A new column layout required a column-beam transfer system on the first parking level to distribute loads so that the parking levels were not heavily disturbed with columns lying in the driving path.

Problem Solution

Based on the analysis performed in technical report II, to allow for a uniform slab thickness with the new column grid layout, a two-way post-tensioned floor slab system would be optimal. Since post-tensioned slabs are cast-in-place, it is possible to implement the system into 800 North Glebe with its unique curved slab edges.

The increase of columns will help to reduce the building torsion, but will require transfer girders in the garage level to distribute the forces around the garage thruways and to the foundations. The current foundations will then need to be redesigned to support the new loading pattern. Along with the increased number of columns to help reduce building torsion, a post-tensioned floor slab is more rigid and therefore will contribute to the lateral load carry capacity of the structure. The original structural model, assuming only the shear walls participating in lateral load carry will be compared to a model of the entire structure participating in the lateral system.

Since part of the building is already a post-tensioned floor system, it can be deduced that the Arlington area has the proper contractors to complete the structure. Many large cities do not have experienced post-tensioned laborers, but this is not the case for Arlington. Standardizing the slabs would help to reduce the variety of concrete trades on the project.

Design Goals

The overall design goal of this project is to redesign the slab system of 800 North Glebe as a two-way post-tensioned slab. This will allow for a uniform slab type and thickness throughout the superstructure and have the entire structure participate in the lateral force resisting system. Additional goals to be met throughout this project include:

- Have the entire structure participate in the lateral force resisting system and reduce the overall lateral load carried by the central core shear walls
- Reduce the impact of alterations to the architectural floor plans as laid out by the architect
- Reduce column sizes where applicable
- Not reduce the floor-to-ceiling height
- Determine affects of structural changes will have on architectural floor plans
- Compare sequencing and cost differences between systems
- Use computer programs such as RAM Concept and ETABS to perform an in-depth lateral and gravity analysis to create a more efficient structure

MAE Course Related Study

To fulfill MAE requirements for senior thesis, the knowledge learned through master's level courses will be implemented. AE 538, *Earthquake Resistant Design of Buildings* was used in conjunction with information taught in AE 597A, *Computer Modeling*, to critically analyze the structural system of 800 North Glebe. Even though RAM Concept was not specifically taught, the concepts of meshing, diaphragms and property modifications will be used. Along with AE 597A, the information taught in AE 542, *Building Enclosures,* will be utilized through the determination of curtain wall systems. More information into how these courses were used in the analysis and design for thesis will be discussed throughout the report.

Structural Depth Study

The structural depth study includes a design and analysis of a new gravity and lateral system for 800 North Glebe as defined in the problem statement. For this to be accomplished, all interior and exterior columns, along with the slab were designed and their integration into the lateral system was performed. Final conclusions and recommendations are based on all the impacts on the structure, which include but are not limited to; performance, architectural impact, and constructability.

Design Codes and Standards

Thesis design had been performed with the most up to date codes and standard available. These may differ from the original design, resulting in possible calculation variations.

Original Design:

- International Building Code, 2003
- Virginia Uniform Building Code, 2003
- American Society of Civil Engineers (ASCE)
    - ASCE 7-02, Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
    - Building Code Commentary 318-08
    - Structural Concrete for Buildings, ACI 301
- America Institute of Steel Construction (AISC)
    - Manual of Steel Construction, Thirteenth Edition, 2005

Thesis Design with Additional References:

- International Building Code, 2006
- Virginia Uniform Building Code, 2003
- American Society of Civil Engineers (ASCE)
    - ASCE 7-05, Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
    - Building Code Commentary 318-05
- America Institute of Steel Construction (AISC)
    - Manual of Steel Construction, Thirteenth Edition, 2005

Material Properties

Steel:

Wide Flange	**50 ksi (A992)**
Plates, Channels, Angles and Bars	**36 ksi (A36)**
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HSS Rectangular or Square Tubing	**46 ksi (A500 Grade B)**
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Anchor Rods	**(F1554 Grade 55)**
Weld Strength	**70 ksi (E70XX)**

Concrete:

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Interior Walls	**f'c = 6000 psi**
Ext. Slab of Grade, Pads, Garage SOG	**f'c = 4500 psi**
Garage and Plaza Slabs, Framed Int. Slabs	**f'c = 8000 psi**
Ext. Walls, Beams, Basement Walls	**f'c = 4000 & 5000 psi**
Deck Supported Slabs	**f'c = 3500 psi**
Cellar Columns	**f'c = 4000 & 8000 psi**
Superstructure Columns	**f'c = 4000, 6000 & 8000 psi**
Shear Walls	**f'c = 6000 psi**
Masonry	**f'm = 1500 psi**

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Cold Formed Steel:

20 Gage	**33 ksi (A653)**
18 Gage	**33 ksi (A653)**
16 Gage	**50 ksi (A653)**

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Slab Design

Design Loads

Gravity - Live Loads

ASCE 7-05, *Minimum Design Loads for Buildings and other Structures,* was the main reference for determination of loads in this project for 800 North Glebe. These loads were compared to the loads specified by the designer per IBC 2003 and the 2003 Virginia Uniform State Building Code which references ASCE 7-02. A few loadings used by the designer were seen to be greater, i.e. garage entry, and therefore the larger value was used for thesis because of the significant increase. These values are outlined in Table 1 below.

Live Loads				
Description	Location	Designer Loads	(ASCE 7-05)	Thesis Loads
Parking	P3	40	40	40
Stairs	P3	100	100	100
Parking	P2	40	40	40
Stairs	P2	100	100	100
Parking	P1	40	40	40
Stairs	P1	100	100	100
Garage Entry	Level 1	250	50	250
Main Retail/Assembly	Level 1	100 125 250	100	100
Elevator Lobby	Level 1	100	100	100
Entry	Level 1	100	100	100
Loading Dock	Level 1	350		350
Yards and Terraces	Level 1	100	100	100
Marquees and Canopies	Level 2	75	75	75
Corridors Above First Floor	Level 2-10	100	80	80
Walkways and Elevated Platforms		60	60	60
Mechanical	Penthouse	150	125	125
Roof	Roof	30	20	20

Table 1: Live Loads

Gravity - Dead Loads

Building dead loads and their general description are laid out in Table 2 below. Slab areas were taken from CAD floor plans provided by the designer and varied by floor because of the curves and the major setback at levels four, six and eight. For the original design, slab thicknesses of 7 ½”, 9”, 10 ½” and 12” are used per floor depending on the location and area usage. Two-way mildly reinforced slabs located on levels two though six have slab thicknesses of 10 ½” with 7” thick drop panels to reduce the punching shear around the columns. Across the post-tensioned girders is the 9” one-way slab. Located at the main entrance is a 36” diameter column rising from the ground to the top of the building with a 12” cantilevered slab. The 12” slab was needed because of the increased moment the cantilevered section caused over the beam. However, the thesis design implemented a uniform 8” two-way slab with 4” shear caps around the columns for all elevated slabs.

Dead Loads				
Description	Location	Designer	Superimposed Dead Load	Thesis Loads
Concrete	All Levels	150 pcf		150 pcf
Partitions, Finishes	All Levels		20 psf	20 psf
MEP	All Levels		5 psf	5 psf
Precast Panels	Curtain Wall		35 psf	20 psf*
Curtain Glass	Curtain Wall		15 psf	

*Assume the façade is composed of 20% precast and 80% glazing.

Table 2: Dead loads

Design Process – Initial Layout

To allow for the implementation of a two-way slab system, the column layout needed to be redesigned. Bay sizes were reduced from 46’-0” X 30’-0” down to 30’-0” X 23’-0” to allow for a uniform slab type and thickness, as seen in Figures 15 and 16 respectively. Other systems were capable of being implemented onto the new column layout, which include steel frame system, but to avoid floor-to-ceiling height reduction the two-way concrete slab was the preferred system. Design issues that were of concern include: which direction for banded tendons, how to deal with openings, and use of shear caps for punching shear.

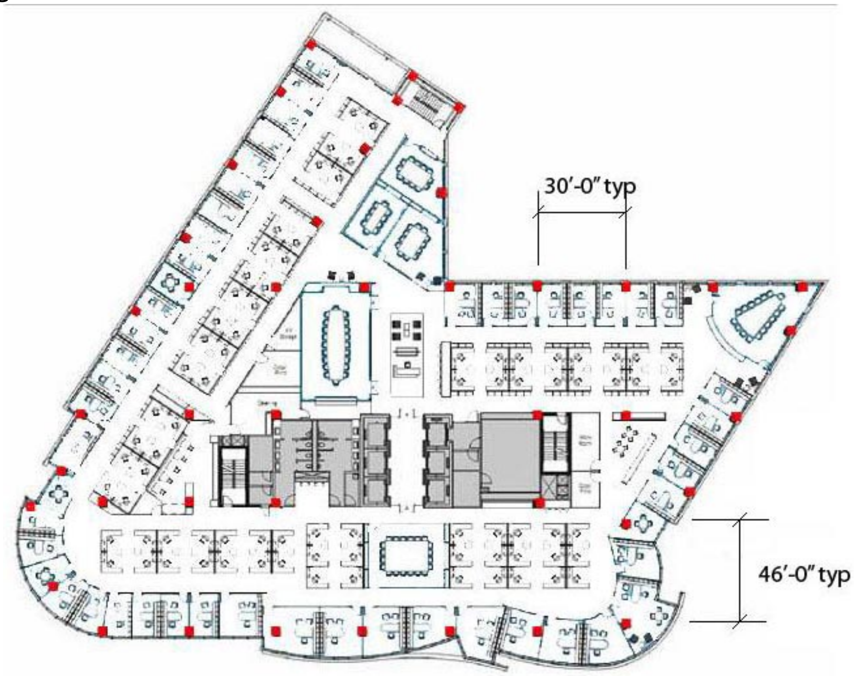


Figure 15: Current Column Layout

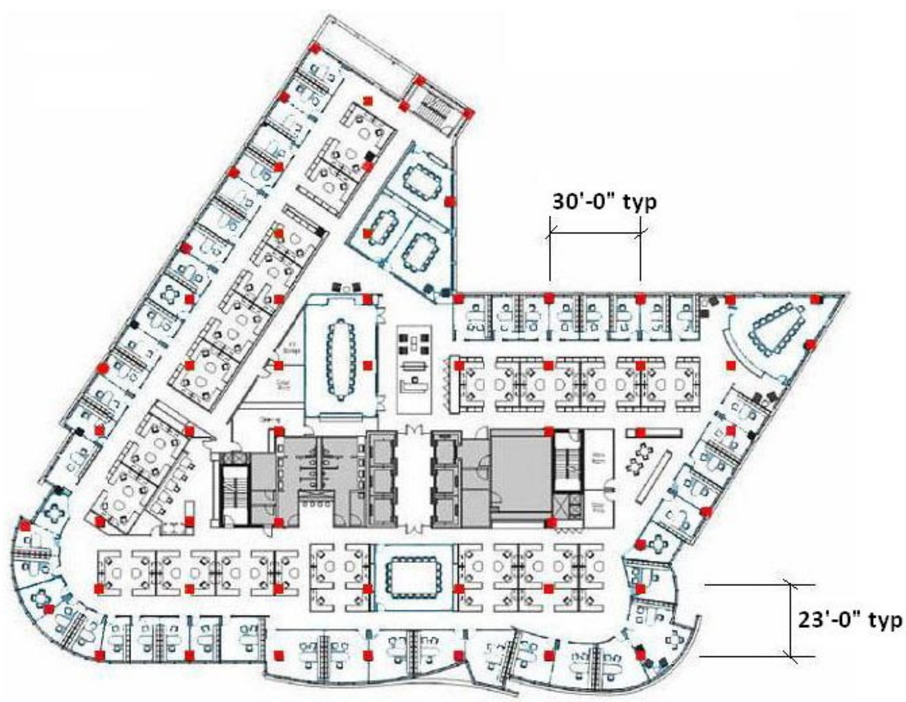


Figure 16: New Column Layout

An initial slab thickness of 8" was calculated based on the span-to-depth ratio of *Holbert Apple Associates" Post-Tensioned Concrete Practical Applications"*. Shear capitals were needed at all column locations to reduce shear failure and allow for increased slab-to-column reinforcing. Descriptions of these are discussed in the analysis and detailing section. Uniformly distributed tendons span the 30'-0" long direction while bonded tendons would span the shorter 23'-0" direction. Shortening concerns were addressed to avoid negative shear wall affect. However, since the shear wall core was centrally located, which is the preferred method to avoid shortening problems; this was presumed to not be an issue.

Design Process – Computer Model

RAM Concept models were created for the two primary slab layouts; the lower six levels and the upper four levels. This program was chosen because of the finite element analysis capabilities for a two-way post-tensioned slab system. An initial slab strip was modeled in Ram Concept's *Slab Wizard* to determine the initial amount of tendons needed to balance 70% of the construction dead load and their respective profile depth at ends and midpoints. Hand calculations were performed to determine the initial effective stress, but the percent difference between those values was around 35%. It was concluded that this was far too large for hand calculations to be a viable design method. Once an initial tendon scheme was calculated, the layout was implemented over the entire slab. Due to slab nonuniforminties, such as openings, overhangs and nonprismatic slab edges, tendons were altered to meet ACI precompression minimums of 250psi accordingly. However, for two-way slab design, the typical precompression is 150psi-250psi, and up to 300psi in end spans.

Uniformly distributed tendons have four strands per tendon. Spacing of these tendons was based on ACI 318-08, section 18.12.4, where spacing shall provided a minimum average effective prestressing force of 125 psi. Where slab opening less than 4'-0" in width are located, the tendons swept around each side, but where openings were larger, the tendons were anchored off and resumed on the opposite side. The profile depths at the ends are 1" below the top of slab, while profile depths at the midspan are 5.6" below. The distributed on tendons in the north-south direction is laid out in Figures 17 and 18.

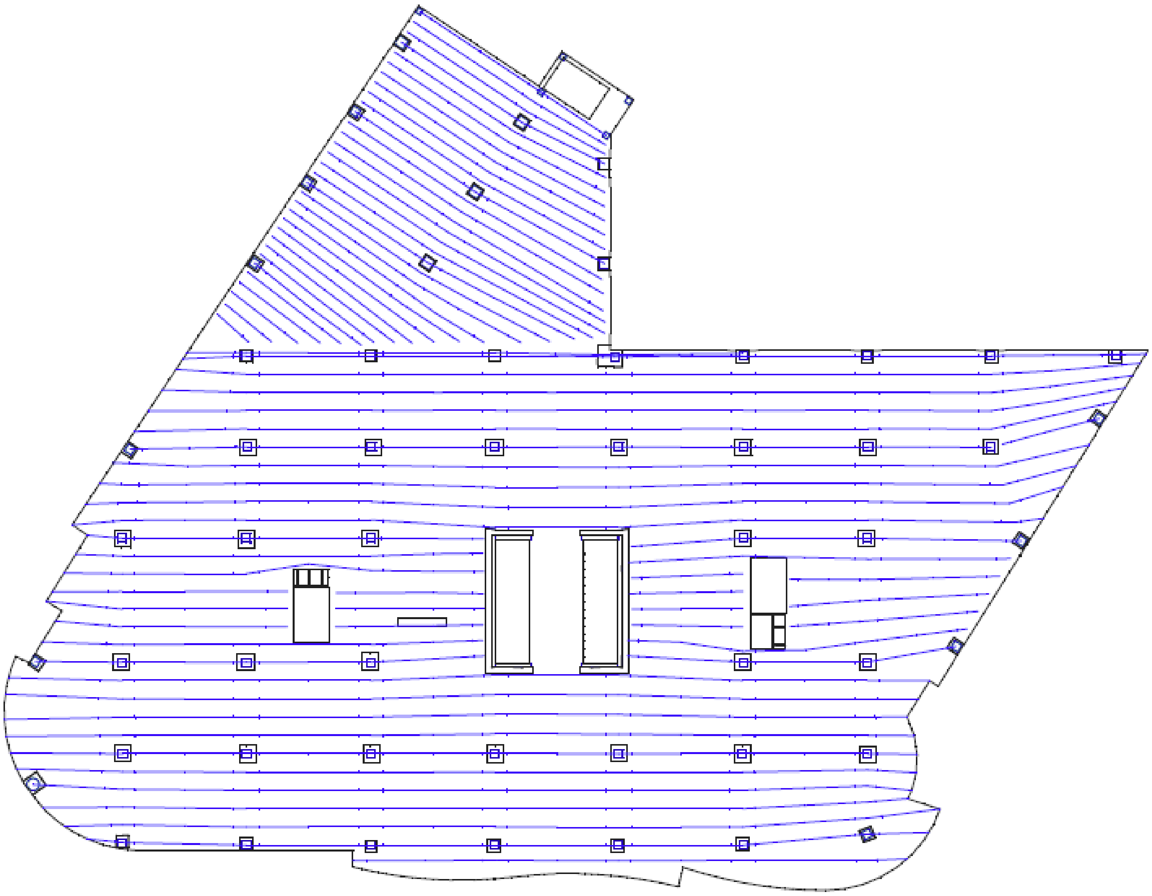


Figure 17: Lower 6 Levels Distributed Tendon Layout

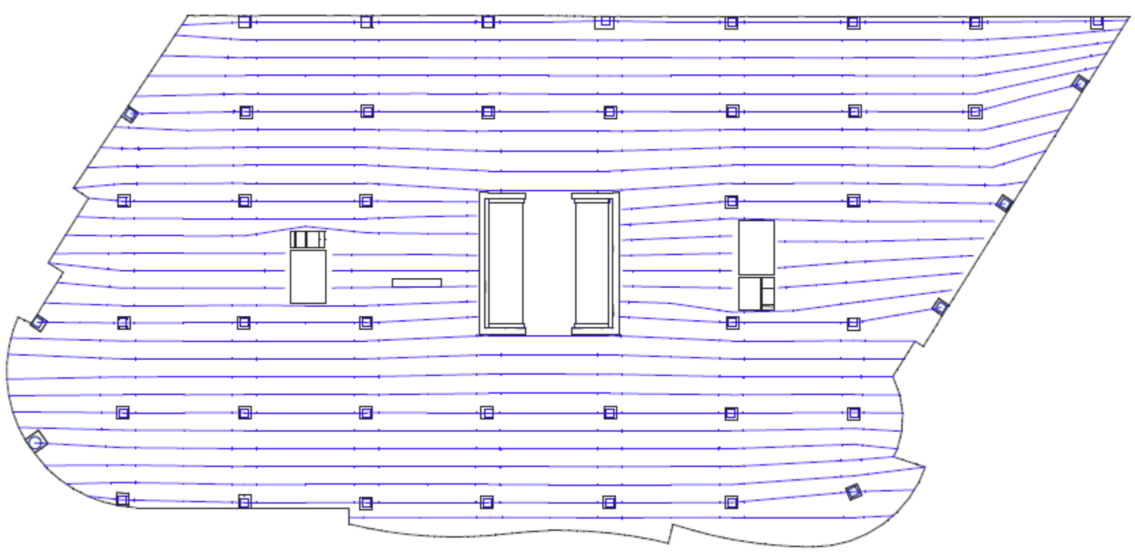


Figure 18: Upper 4 Levels Distributed Tendon Layout

Tendons banded along the short direction were grouped in the columns strip, as seen in Figures 19 and 20. This allowed for tendon forces of 650kips, on average. The banded tendon profiles in this direction were designed with the same depth as the uniformly distributed tendon layout. These banded tendon regions act as beams in the lateral system concrete moment frames.

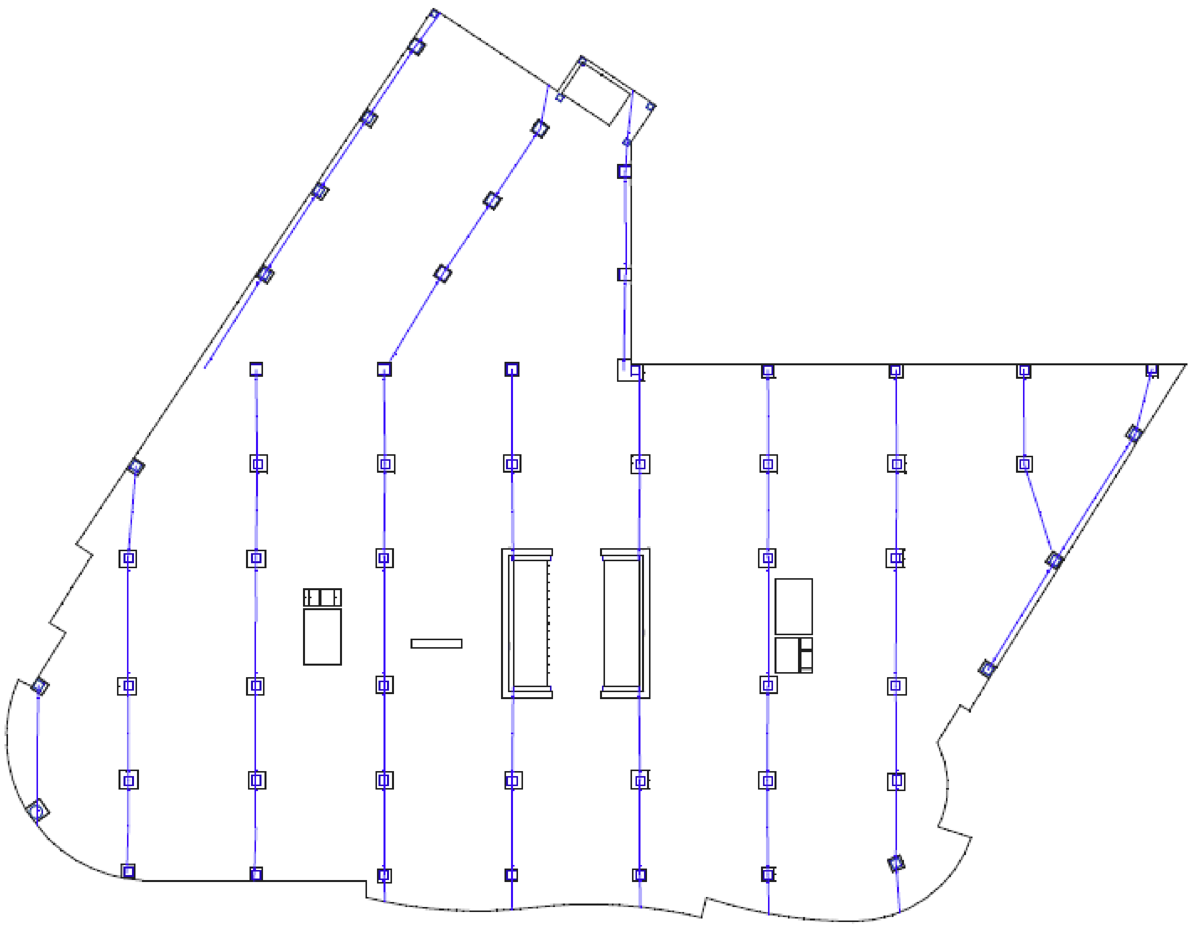


Figure 19: Lower 6 Levels Banded Tendon Layout

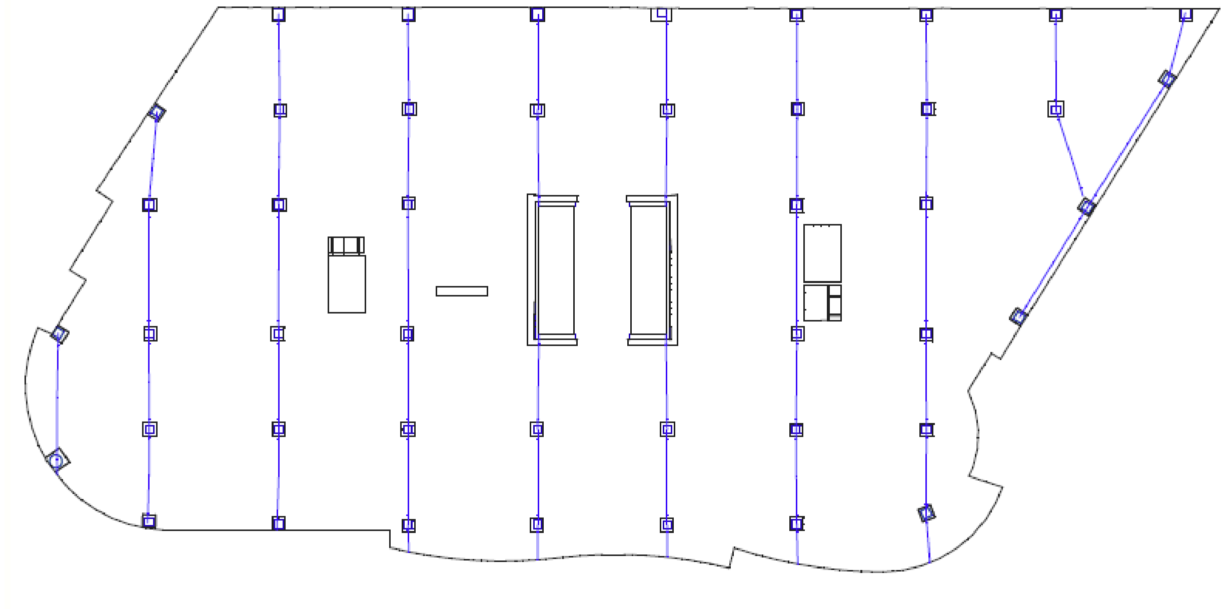


Figure 20: Upper 4 Levels Banded Tendon Layout

Design Process – Analysis and Detailing

Strength checks were performed on the initial tendon layout. Analysis verification was based on chapter 18 and Appendix B of ACI 318-05, and a detailed list of sections used is available in Appendix C. Design included the most significant load case from ASCE 7-05. Factored moments and shear were to meet the Equivalent Frame method of ACI section 13.7, but it was permitted to use a more detailed method including elastic theory, which I determined to be a RAM Concept model.

Although this flat plate system was used in the lateral force resisting system (LFRS), the column strip area was analyzed with section 18.2: Slab System, and not by section 18.7: Flexural Members. Classification of the two-way slab met criteria for Class-U design, in which stresses at service are permitted to be performed with uncracked sections, along with $f_t \leq 7.5\sqrt{f_c'}$. Slab concrete strength was designed with 8,000 psi concrete. This allowed for a maximum precompression tensile zone extreme fiber stress at service conditions to be 671 psi. The steel tendons permissible tensile stresses were designed not to exceed $0.7 * f_{pu}$, where f_{pu} was taken to be 26, 000 ksi.

Anchoring and stressing the tendons will be done in sections due to constructability restrictions. The spans are so great that tendons will need to be done in stages. Anchorage zones are composed of two sections; the local and general zone, as seen in Figure 21. Both local and general zone design is based upon factored prestressing force P_{pu}, and is strongly influenced by the specific characteristics of the anchorage device and its respective reinforcing. Reinforcing for the local zone is done to allow for proper function of the anchoring device. General zone reinforcing is designed to resist bursting, spalling and longitudinal edge tension forces.

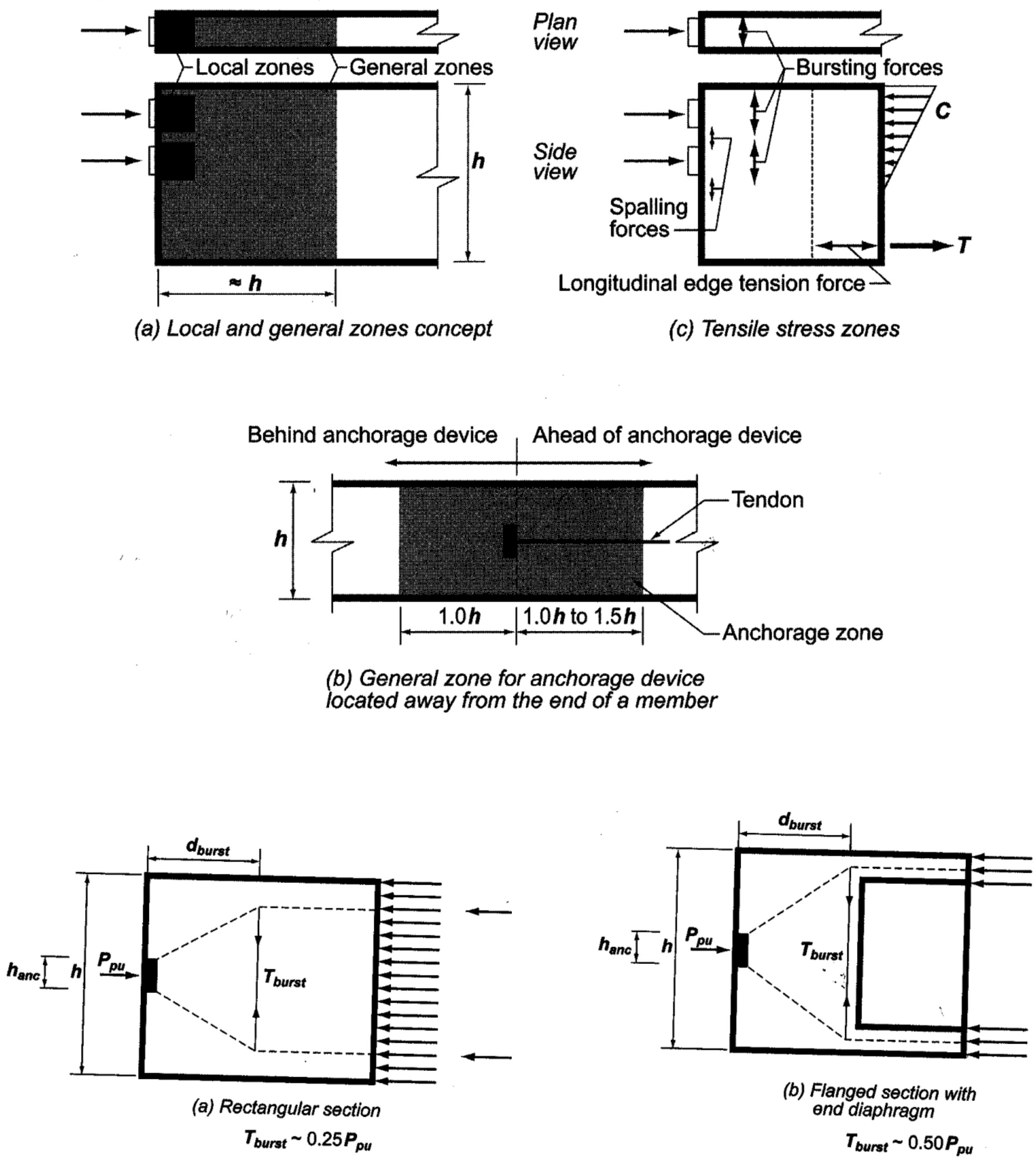


Figure 21: Anchorage Zones

Determination of anchorage specifics are primarily done at the shop drawing stage of design with test information from the manufacturer. Therefore, since the reinforcing and design of this region is heavily influenced by the anchored selection, explicit requirements for reinforcing cannot be completed until specific devices are chosen and are not in the scope of thesis. Schematic layouts of reinforcing can be seen in Figure 22 below.

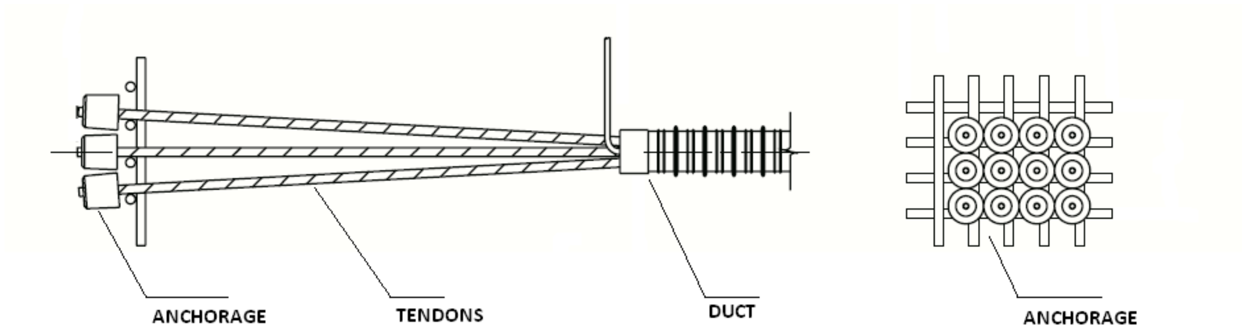


Figure 22: Anchorage Zone Plan and Elevation View

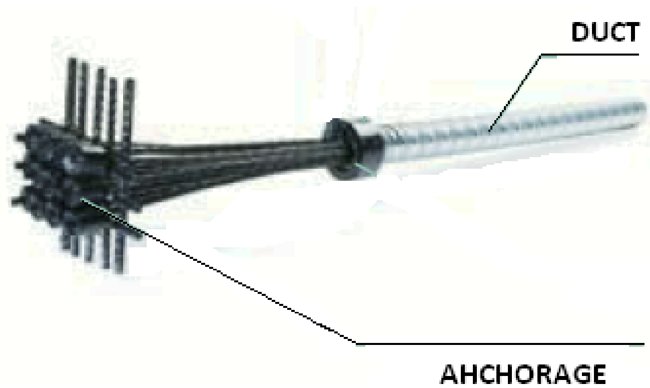


Figure 23: Actual Anchorage Image

A slab system without beams has major punching shear concerns around the columns. ACI states “slabs with unbounded tendons, a minimum of two ½” diameter, seven-wire post-tensioned strands shall be provided in each direction at columns, either passing through or anchored within the region bounded by the longitudinal reinforcement of the columns.” These two tendons must pass under orthogonal tendons in adjacent spans to aid in suspending the span following a punching shear failure. The aforementioned requirement for uniformly distributed tendons in one direction and banded tendons in the other can be satisfied by first placing the banded tendons and then placing the distributed tendons. However, since the slab system is also part of the LFRS, positive and negative moment may be induced at slab-column connects and increased reinforcing in these areas. The increased concrete area around the slab-column interface allowed for more reinforcement space. A detail of the tendon layout around a column can be seen in Figure 24 below.

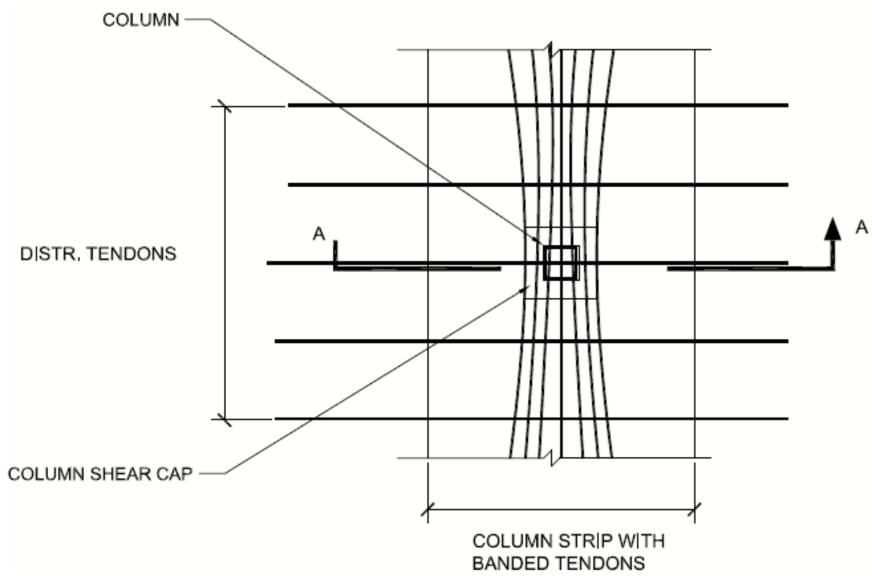


Figure 24: Tendon Layout Plan Around Column

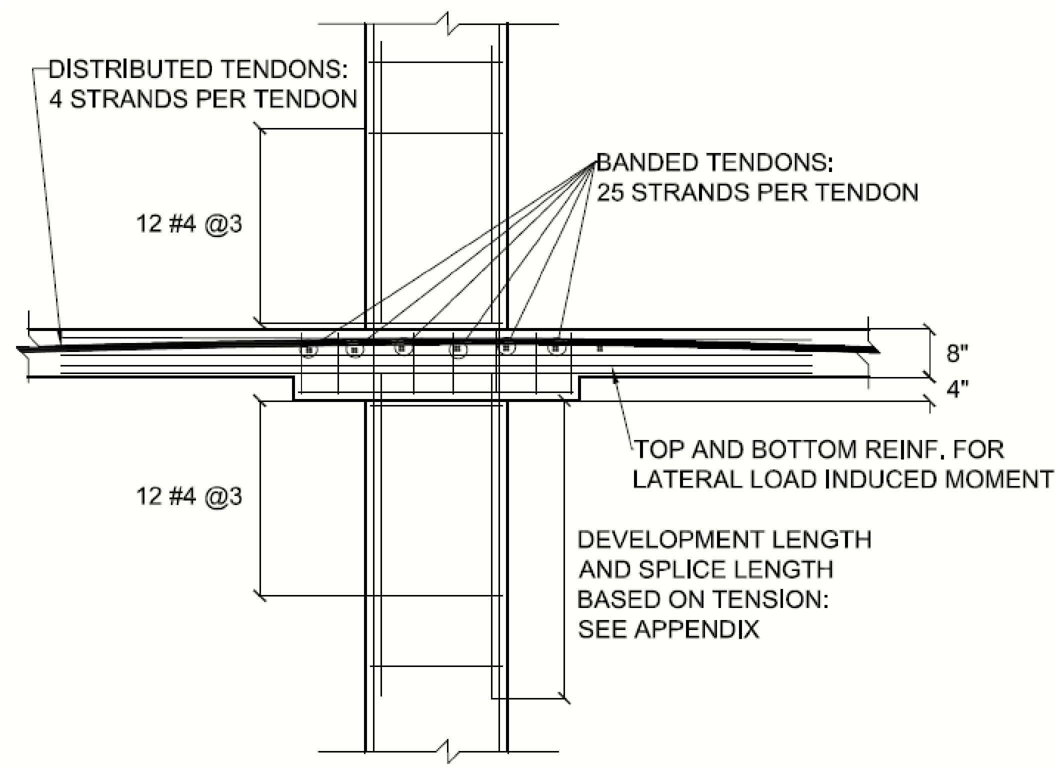


Figure 25: Section A-A

Two-way slabs deflection determination dealt with deflection in both directions in an additive process, where Figure 26 shows how a bay of a two-way slab deflects. Slab deflection calculations were based on live load criteria.

- Service LC –(Dead Load + Balanced Load) = Immediate Load Deflection
- Long Term LC – (Dead Load + Balanced Load) = Time Dependent Deflection

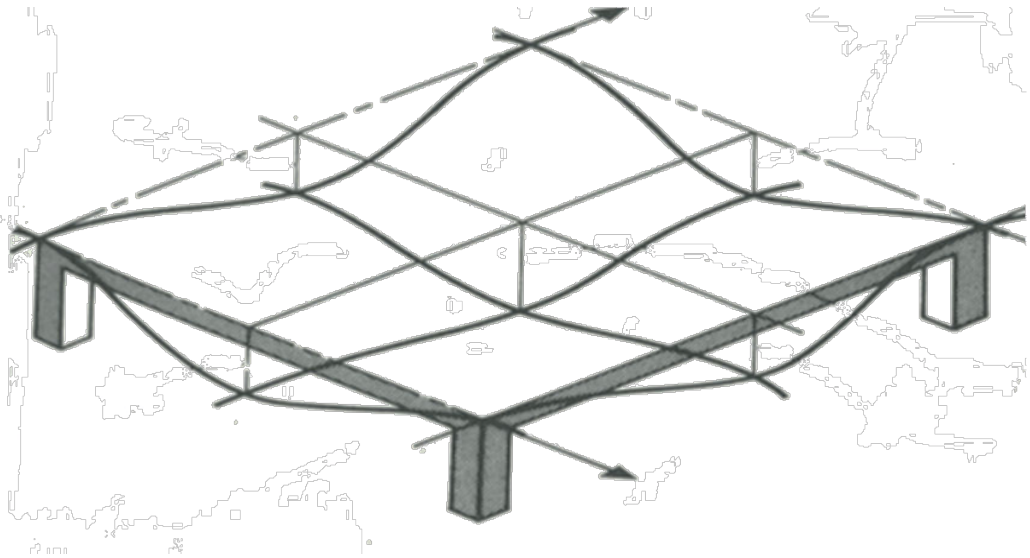


Figure 26: Two-way Slab Deflection

Deflection values were taken from RAM Concept and compared to the values of a two-way slab deflection calculation performed by hand. Figure 27 displays a contour deflection plan of the entire third floor slab and depicts what areas were concerned with deflection verification.

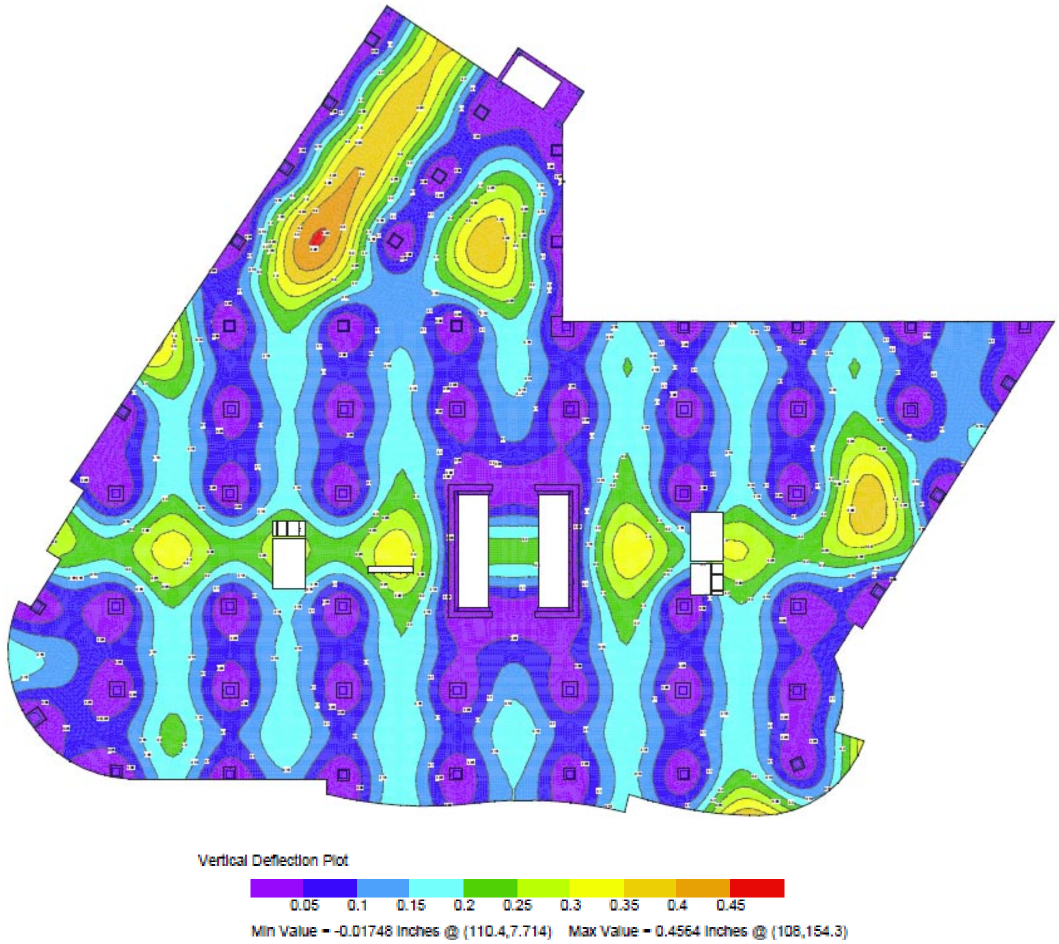


Figure 27: Lower Level Deflection Contour Plan

Based on ACI 318-08 section 18.3.5, immediate live load deflection shall not exceed $\frac{l}{360}$ and shall not exceed $\frac{l}{240}$ for time-deflection characteristics. Time related deflections concerns itself with creep, shrinkage and loss of tension in the tendons over time. As seen in Table 3, the deflection criteria of the slab are adequately reinforced to meet code.

Deflection					
	Code Maximum	Hand Calculation		RAM Concept	
Live Load	0.74"	0.362"	MEETS CODE	0.31"	MEETS CODE
Time Related	1.1"	NA	NA	1.06"	MEETS CODE

Table 3: Deflection Verification

Lateral System Optimization Design

Load Combinations

AISC 7-05 section 2.3, strength design load combinations were considered for factoring gravity and lateral loads in analysis. When only gravity load cases are considered, load case 2 usually governs. However, when lateral loads are involved in analysis, load cases 4, 5, 6 or 7 may govern depending on lateral load magnitudes and whether overturning is addressed. The load combinations considered for thesis analysis are listed below. For the thesis building being analyzed, these combinations were entered into an ETABS model.

1. 1.4(D+F)
2. 1.2(D+F+T) + 1.6(L+H) + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(L_r 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

Once the controlling wind and earthquake cases were found, it was determined by shears at the base level, the load cases including 1.6W were larger in the north-south (X) direction and the east-west (Y) direction, which can be seen in Table 4 below. This is primarily due to the large surface areas of the façade, which produce larger wind pressures, and therefore larger story forces on the structure. The wind loads in east-west directions had a much more significant increase compared to the north-south direction.

X-Direction Section Cut @ Base Level			Y-Direction Section Cut @ Base Level	
Load combination	Direct Shear force		Load combination	Direct Shear force
Wind: Combinations 4 and 6	**848.87**		Wind: Combinations 4 and 6	**964.67**
Earthquake: Combination 5 and 7	722.2		Earthquake: Combination 5 and 7	868.2

Table 4: Load Combination Check

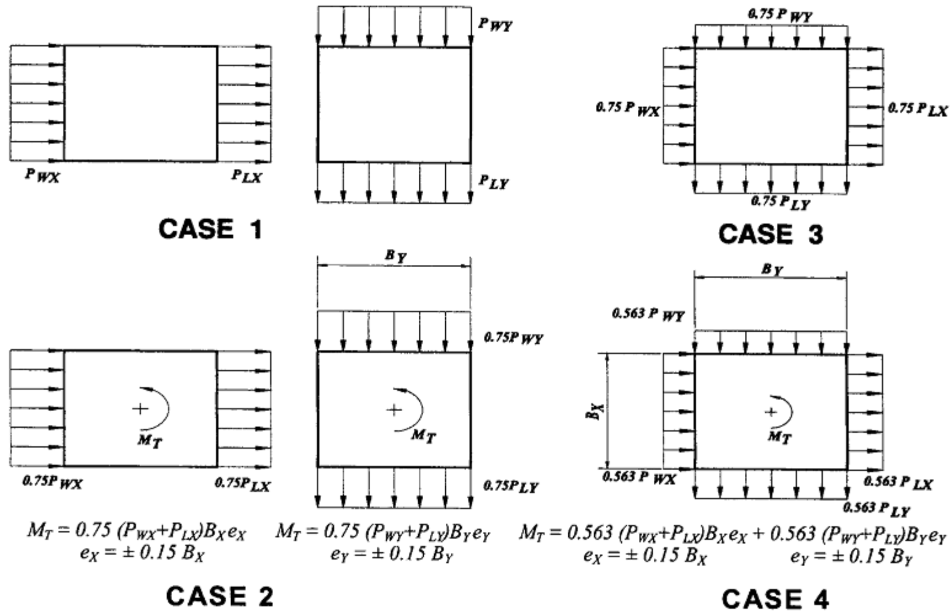
Design Loads

Wind Loads

ASCE 7-05 was the governing resource for wind load calculations. Section 6.5 describes *Method 2 – Analytical Procedure* for main wind-force resisting systems (MWRS) of enclosed buildings. Exposure, height, topographic effects, wind direction and wind velocity all played a part in determining velocity pressures. In conjunction with gust effect factors, external and internal pressure coefficients, and force coefficients it was eventually determine the base shear for the building. Section four outlines four cases in which wind loads should be applied to determine the greatest story forces. These cases were entered into a computer model and it was found that case one, full wind loads applied to the primary axis without eccentricity effects, produced the greatest forces on the structure. Table 5 details the values of all wind cases used in determination and visual representations of the cases are in Figure XX.

X-Direction Wind Case Determination			Y-Direction Wind Case Determination		
Wind Case	X-Force	Y-Force	Wind Case	Y-Force	X-Force
Case 1	**-848.87**	0.77	Case 1	**-964.67**	0.96
Case 2: Positive Pressure and Torsion	-641.12	8.05	Case 2: Positive Pressure and Torsion	-712.97	-5.59
Case 2: Negative Pressure and Torsion	-632.1	-6.91	Case 2: Negative Pressure and Torsion	-734.05	7.1
Case 3: Positive X Pressure, Positive Y Pressure	-636.01	-722.9	Case 3: Positive X Pressure, Positive Y Pressure	-722.9	-636.01
Case 3: Positive X Pressure, Negative Y Pressure	-637.22	724.04	Case 3: Positive X Pressure, Negative Y Pressure	724.04	-637.22
Case 4: Postive X Pressure and Torsion, Postive Y Pressure and Torsion	-485.11	-528.67	Case 4: Postive X Pressure and Torsion, Postive Y Pressure and Torsion	-528.67	-485.11
Case 4: Postive X Pressure and Negative Torsion, Postive Y Pressure and Torsion	-486.02	556.53	Case 4: Postive X Pressure and Negative Torsion, Postive Y Pressure and Torsion	556.53	-486.02
Case 4: Postive X Pressure and Torsion, Postive Y Pressure and Negative Torsion	-475.59	-544.48	Case 4: Postive X Pressure and Torsion, Postive Y Pressure and Negative Torsion	-544.48	-475.59
Case 4: Postive X Pressure and Torsion, Negative Y Pressure and Postive Torsion	-478.34	-539.89	Case 4: Postive X Pressure and Torsion, Negative Y Pressure and Postive Torsion	-539.89	-478.34
Case 4: Negative X Pressure and Postive Torsion, Postive Y Pressure and Torsion	470.03	-529.59	Case 4: Negative X Pressure and Postive Torsion, Postive Y Pressure and Torsion	-529.59	470.03

Table 5: Wind Case Determination



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Figure 28: Wind Case Visual Representation

Variables used in analysis are outline in Table 6 below, and the calculations are shown in Appendix E. Tables7 shows how the forces act on the building in the north-south (X) direction while Figure 29 depicts how the forces act on the structure. The figures and tables are based on the MWRS calculations and are the forces used in the computer model.

Table 8 shows the forces acting in the east-west (Y) Directions, and Figures 30 depiction how these pressures act on the building at each level. Values in the east-west direction were found to be greater than those in the north-south direction.

Wind Loads			
Category			Reference
Basic Wind Speed (mph)	V_{3s}	90	Figure 6-1
Importance Factor	I	1.0	Tabke 6-1
Exposure Category	-	B	6.5.6.3
Directionality Factor	K_d	0.85	Tabke 6-4
Topographic Factor	K_{zt}	1.00	6.5.7.1
Intensity of Turbulance	I_z	Varies	Eq. 6-5
Integral Length Scale of Turbulence	L_z	Varies	Eq. 6-7
Background Response Factor (North/South)	Q	0.780	Eq. 6-6
Background Response Factor (East/West)	Q	0.778	Eq. 6-6
Gust Effect Factor (N/S)	G_f	0.8191	6.5.8.1
Gust Effect Factor (E/W)	G_f	0.8175	6.5.8.1
	GC_{pi}	0.18	Figure 6-5
	GC_{pi}	-0.18	Figure 6-5
Windward Pressure	C_p	0.8	Figure 6-6
Leeward Pressure (E/W)	C_p	-0.5	Figure 6-6
Leeward Pressure (N/S)	C_p	-0.45	Figure 6-6 (interpolated)
Velocity Pressure Exposure Coefficient Evaluated at Height z	K_z	Varies	Table 6-3
Velocity Pressure at Height z	q_z	Varies	Eq. 6-15
Velocity Pressure at Mean Roof Height	q_h	19.70	Eq. 6-15

Table 6: Wind Load Variables

Wind Loads N/S (Short Walls Resisting)									
Floor	Story Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)		Force of Windward Pressure (k)	Story Shear Windward (k)	
					Windward	Leeward			
PH Roof	0	153.75	1.12	19.70	12.91	-7.26	24.75	0.00	
PH	18.5	135.25	1.08	18.99	12.44	-7.26	42.58	24.75	
10	13.75	121.5	1.04	18.41	12.07	-7.26	35.07	67.33	
9	13.75	107.75	1.01	17.79	11.66	-7.26	32.65	102.40	
8	12.75	95	0.97	17.16	11.25	-7.26	30.21	135.05	
7	12.75	82.25	0.93	16.47	10.79	-7.26	28.89	165.26	
6	12.75	69.5	0.89	15.70	10.29	-7.26	27.41	194.16	
5	12.75	56.75	0.84	14.82	9.71	-7.26	25.68	221.56	
4	12.75	44	0.78	13.78	9.03	-7.26	23.59	247.24	
3	12.75	31.25	0.71	12.49	8.19	-7.26	20.88	270.84	
2	12.75	18.5	0.61	10.76	7.05	-7.26	9.66	291.72	
1	18.5	0	0.00	0.00	0.00	0.00	0.00	301.38	
				Σ Factored Windward Story Shear (k)=					482.21
				Σ Factored Windward Overturning Moment (ft-k)=					44515

Wind Loads N/S (Short Walls Resisting)						
Floor	Story Height (ft)	Height Above Ground (ft)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)	
PH Roof	0	153.75	20.17	39.18	0.00	
PH	18.5	135.25	19.70	67.75	39.18	
10	13.75	121.5	19.33	56.53	106.94	
9	13.75	107.75	18.92	53.33	163.47	
8	12.75	95	18.51	50.11	216.80	
7	12.75	82.25	18.05	48.80	266.92	
6	12.75	69.5	17.55	47.31	315.71	
5	12.75	56.75	16.97	45.58	363.02	
4	12.75	44	16.29	43.50	408.60	
3	12.75	31.25	15.45	40.78	452.10	
2	12.75	18.5	14.31	19.61	492.88	
1	18.5	0	0.00	0.00	512.49	
		Σ Factored Windward Story Shear (k)=				819.99
		Σ Factored Windward Overturning Moment (ft-k)=				73606

Table 7: North-South Wind Load Forces

Wind Loads E/W (Long Walls/Frame Resisting)									
Floor	Story Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf) Windward	Wind Pressure (psf) Leeward	Force of Windward Pressure (k)	Story Shear Windward (k)	
PH Roof	0	153.75	1.12	19.70	12.88	-8.05	28.72	0.00	
PH	18.5	135.25	1.08	18.99	12.42	-8.05	49.41	28.72	
10	13.75	121.5	1.04	18.41	12.04	-8.05	40.70	78.13	
9	13.75	107.75	1.01	17.79	11.64	-8.05	37.89	118.83	
8	12.75	95	0.97	17.16	11.23	-8.05	35.06	156.72	
7	12.75	82.25	0.93	16.47	10.77	-8.05	33.53	191.78	
6	12.75	69.5	0.89	15.70	10.27	-8.05	31.80	225.31	
5	12.75	56.75	0.84	14.82	9.69	-8.05	29.80	257.12	
4	12.75	44	0.78	13.78	9.01	-8.05	27.38	286.92	
3	12.75	31.25	0.71	12.49	8.17	-8.05	24.23	314.30	
2	12.75	18.5	0.61	10.76	7.03	-8.05	11.21	338.53	
1	18.5	0	0.00	0.00	0.00	0.00	0.00	349.74	
				Σ Factored Windward Story Shear (k)=					559.58
				Σ Factored Windward Overturning Moment (ft-k)=					51657

Wind Loads E/W (Long Walls/Frame Resisting)						
Floor	Story Height (ft)	Height Above Ground (ft)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)	
PH Roof	0	153.75	20.93	47.33	0.00	
PH	18.5	135.25	20.47	81.87	47.33	
10	13.75	121.5	20.09	68.37	129.20	
9	13.75	107.75	19.69	64.56	197.57	
8	12.75	95	19.28	60.72	262.13	
7	12.75	82.25	18.82	59.19	322.85	
6	12.75	69.5	18.32	57.46	382.05	
5	12.75	56.75	17.74	55.46	439.51	
4	12.75	44	17.06	53.04	494.97	
3	12.75	31.25	16.22	49.89	548.01	
2	12.75	18.5	15.08	24.04	597.91	
1	18.5	0	0.00	0.00	621.95	
		Σ Factored Windward Story Shear (k)=				995.12
		Σ Factored Windward Overturning Moment (ft-k)=				89167

Table 8: East-West Wind Load forces

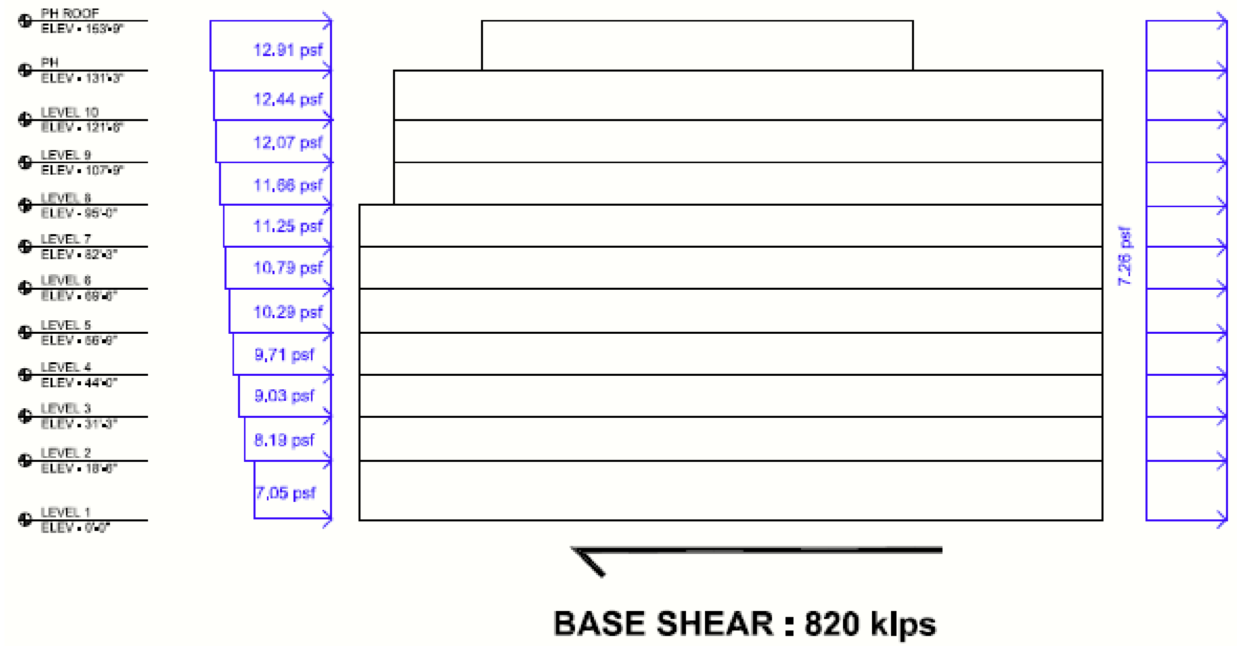


Figure 29: North-South Wind Loads

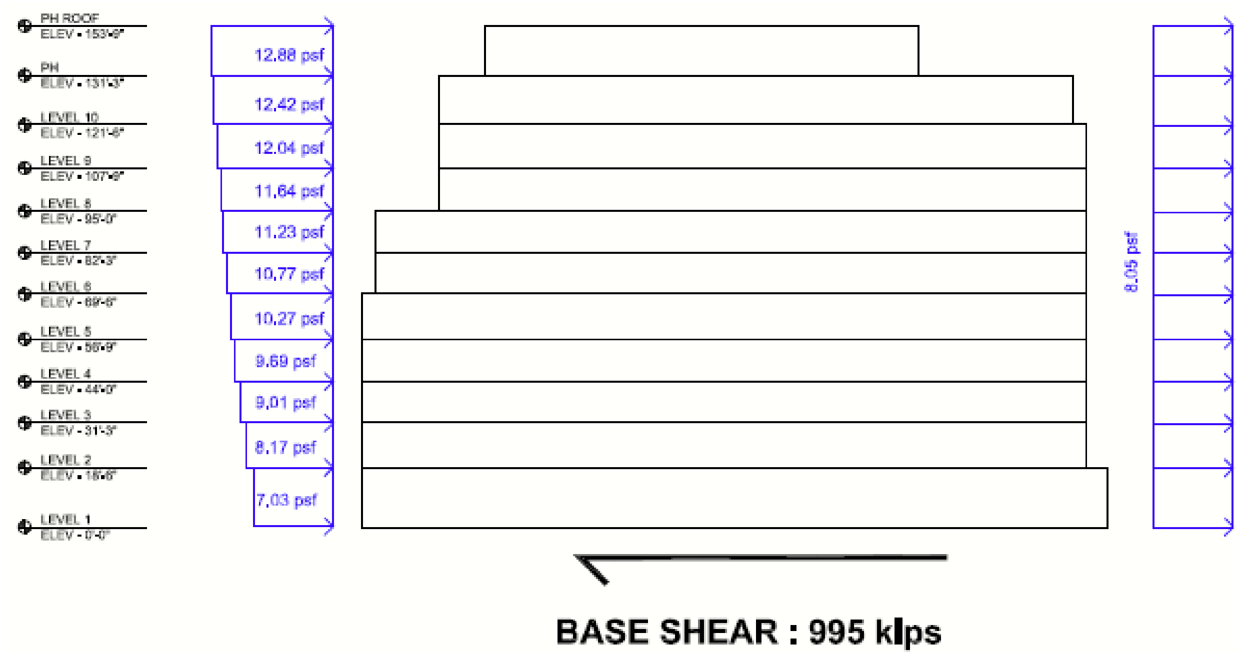


Figure 30: East-West Wind Loads

Seismic Loads

Seismic calculations of 800 North Glebe were based upon ASCE 7-05 section 12.8 *Equivalent Lateral Force Procedure* for thesis design. For this method to be used, checks to determine if there were any horizontal or vertical irregularities were conducted. Tables 9 and 10 outline the status checks used.

Horizontal Irregularities			
Type	**Description**	**Status**	**Reference Section**
Torsional	Exists when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story the two ends of the structure drifts at the two ends of the structure.	X	12.7.3 16.2.2
Extreme Torsional	Exists when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure.	OK	NA
Reentrant Corner	Exists where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	OK	NA
Diaphragm Discontinuity	Exists where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	OK	NA
Oout-of-Plane Offsets	Exists where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	OK	NA
Nonparallel System	Exists where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	OK	NA

Table 9: Horizontal Irregularity Check

Vertical Irregularities			
Type	**Description**	**Status**	**Reference Section**
Stiffness-Soft Story	exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	OK	NA
Stiffness Extreme Soft Story	exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	OK	NA
Weight (Mass)	exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	OK	NA
Vertical Geometry	exist where the horizontal dimension of the seismic force–resisting system in any story is more than 130% of that in an adjacent story.	OK	NA
In-Plane Discontinuity in Vertical Lateral Force-Resisting Elements	exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	OK	NA
Discontinuity in Lateral Strength-Weak Story	exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	OK	NA
Discontinuity in Lateral Strength-Extreme Weak Story	exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	OK	NA

Table 10: Vertical Irregularity Check

The model output for maximum modal period of vibration was found to be 3.404 seconds. However, this value was not used as the fundamental period because it means the structure is more flexible than what value the code permits for fundamental period of vibration, T_aC_u = 1.4845 s. A lower period of vibration being used for design assumes the lateral resisting structural elements are more rigid and therefore, must be designed for the larger forces. The modal periods of vibration for the structure are found in Table11 below. Since the lateral shearwall core is centrally located with the majority of the building spread over a large slab area causing the building to significantly rotate. The columns are spread throughout the structure, increasing the stiffness and reducing the torsional effects. Based upon knowledge from research and relevant courses, it was determined for the east-west direction the lateral system could be classified as a duel system with at least twenty-five percent of the forces going to the moment frames. The north-south direction was classified as ordinary reinforced shear walls, where coupling beams were utilized.

	ETABS Output		
	X - Translation	Y - Translation	Z - Rotation
Thesis Redesign	3.404	1.101	1.9734
Existing Structure (Shearwalls Only)	5.6079	1.494	6.1358

Table 11: Modal Periods of Vibration

Design criteria variables used for thesis analysis can be found below in Table 12. Design criteria variables were used to determine story forces at each level, story shear at each level, and base shear, where the output is located in Table 13 for the X-direction and Table 14 for the Y-direction. Figures 31 and 32 were constructed to display how these forces acted on the building in their respective directions, while calculations to support the excel graph below are located in Appendix E.

Design Criteria Variables			
Seismic Use Group		Group II	
Site Class		D	Geotech Report
Seismic Design Category		B	Table 11.6-1
Importance Factor	I_e	1.00	Table 11.5-1
Spectral Response Acceleration, Short	Ss	0.179	USGS
Spectral Response Acceleration, 1s	S_1	0.063	USGS
Site Coefficient	Fa	1.6	Table 11.4-1
Site Coefficient	F_v	2.4	Table 11.4-2
Soil Modified Acceleration	SMS	0.2864	
Soil Modified Acceleration	S_{M1}	0.1512	
Design Spectral Response, short	Sds	0.191	USGS
Design Spectral Response, 1s	S_{d1}	0.101	USGS
Response Modification Coefficient	RX	5	Table 12.2-1
	RY	5.5	
Approx. Period Parameter	C_t	0.02	Table 12.8-2
Building height (above grade)	hn	153.75	
Approx. Period Parameter	x	0.75	Table 12.8-2
Approx. Fundamental Period	Ta	0.873	Eq. 12.8-8
Calculated Period Upper Limit Coefficient	C_u	1.7	Table 12.8-1
	TaCu	1.483	12.8.2
Long Period Transition Period	T_L	8	Figure 22-15

		X-Dir	Y-Dir	
	Ta	0.873	0.873	C_t*h^x
	T_b	3.404	1.1	ETABS
Seismic Respoonse Coefficient	Cs=min	0.0382	0.0347	SDS/(R/I)
		0.0136	0.0166	SD1/(T(R/I))
		0.0732	0.1209	SD1*TL/(T^2(R/I))
Building Weight (kips)	W	49641.84	49641.84	
Base Shear	V_b	674.1369	826.0391	Cs x W
Structural Period Exponent	k	1.492267	1.492267	12.8.3

Table 12: Seismic Design Variables

X-Direction							
Level (i)	Story Height (h_i)	Height Above Ground (h)	Story Weight (w (kips))	$w*h_k$	C_{vx}	Lateral Story force (f_i)	Story Shear (V_i)
PHR	18.5	153.75	524.51	961755.3	0.03	19.72	19.72
Roof	13.75	135.25	4181.11	6331661	0.19	129.81	149.53
10	13.75	121.5	4355.72	5620890	0.17	115.24	264.77
9	12.75	107.75	4342.48	4684330	0.14	96.04	360.81
8	12.75	95	4350.33	3888786	0.12	79.73	440.54
7	12.75	82.25	4546.41	3277650	0.10	67.20	507.74
6	12.75	69.5	5344.45	2996653	0.09	61.44	569.18
5	12.75	56.75	5321.16	2204916	0.07	45.21	614.38
4	12.75	44	5459.97	1547608	0.05	31.73	646.11
3	12.75	31.25	5354.67	910853.5	0.03	18.67	664.79
2	18.5	18.5	5861.03	455966.4	0.01	9.35	674.14
			49641.8373	32881069			
Level (i)	Story Height (h_i)	Height Above Ground (h)	B_y	$5\%B_y$	A_x	M_z(ft-k)	M_z(in-k)
PHR	18.5	153.75	215	10.75	1.0	212	2543.6
Roof	13.75	135.25	215	10.75	1.0	1395	16745.9
10	13.75	121.5	215	10.75	1.0	1239	14866.1
9	12.75	107.75	215	10.75	1.0	1032	12389.1
8	12.75	95	215	10.75	1.0	857	10285.0
7	12.75	82.25	215	10.75	1.0	722	8668.7
6	12.75	69.5	215	10.75	1.0	660	7925.5
5	12.75	56.75	215	10.75	1.0	486	5831.5
4	12.75	44	215	10.75	1.0	341	4093.1
3	12.75	31.25	215	10.75	1.0	201	2409.0
2	18.5	18.5	215	10.75	1.0	100	1205.9
							86963.7

Table 13: North-South Seismic Forces

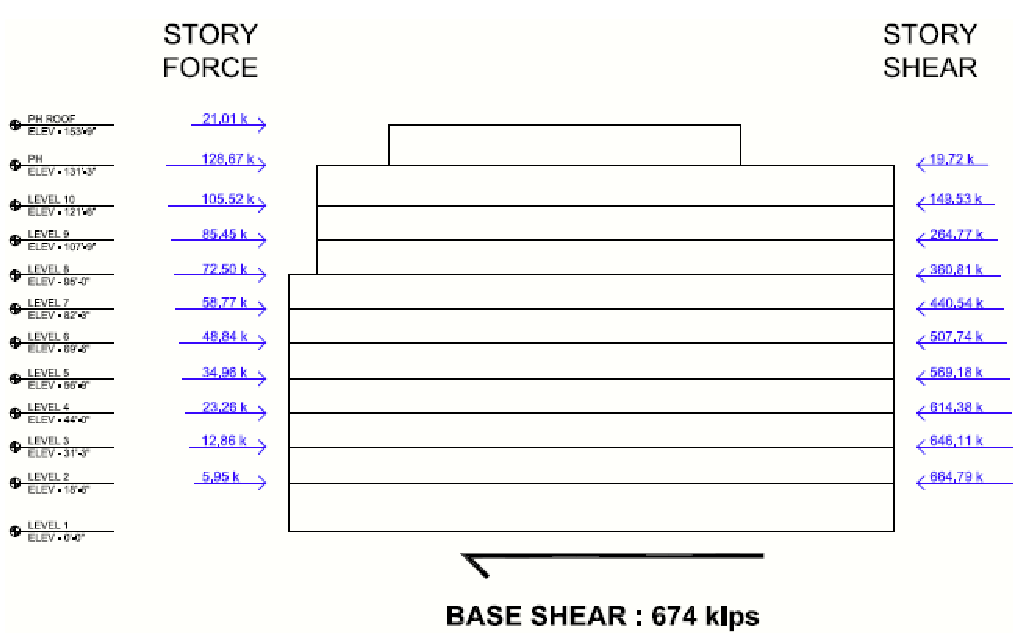


Figure 31: North-South Building Loads

Y-Direction							
Level (i)	Story Height (h_i)	Height Above Ground (h)	Story Weight (w (kips))	w^*h_k	C_{vx}	Lateral Story force (f_i)	Story Shear (V_i)
PHR	18.5	153.75	524.51	961755.3	0.03	24.16	24.16
Roof	13.75	135.25	4181.11	6331661	0.19	159.06	183.23
10	13.75	121.5	4355.72	5620890	0.17	141.21	324.43
9	12.75	107.75	4342.48	4684330	0.14	117.68	442.11
8	12.75	95	4350.33	3888786	0.12	97.69	539.81
7	12.75	82.25	4546.41	3277650	0.10	82.34	622.15
6	12.75	69.5	5344.45	2996653	0.09	75.28	697.43
5	12.75	56.75	5321.16	2204916	0.07	55.39	752.82
4	12.75	44	5459.97	1547608	0.05	38.88	791.70
3	12.75	31.25	5354.67	910853.5	0.03	22.88	814.58
2	18.5	18.5	5861.03	455966.4	0.01	11.45	826.04
			49641.84	32881069			
Level (i)	Story Height (h_i)	Height Above Ground (h)	B_y	$5\%B_y$	A_x	M_z(ft-k)	M_z(in-k)
PHR	18.5	153.75	250	12.5	1.0	302	3624.2
Roof	13.75	135.25	250	12.5	1.0	1988	23859.6
10	13.75	121.5	250	12.5	1.0	1765	21181.2
9	12.75	107.75	250	12.5	1.0	1471	17652.0
8	12.75	95	250	12.5	1.0	1221	14654.1
7	12.75	82.25	250	12.5	1.0	1029	12351.2
6	12.75	69.5	250	12.5	1.0	941	11292.3
5	12.75	56.75	250	12.5	1.0	692	8308.8
4	12.75	44	250	12.5	1.0	486	5831.9
3	12.75	31.25	250	12.5	1.0	286	3432.4
2	18.5	18.5	250	12.5	1.0	143	1718.2
							123905.9

Table 14: East-West Seismic Forces

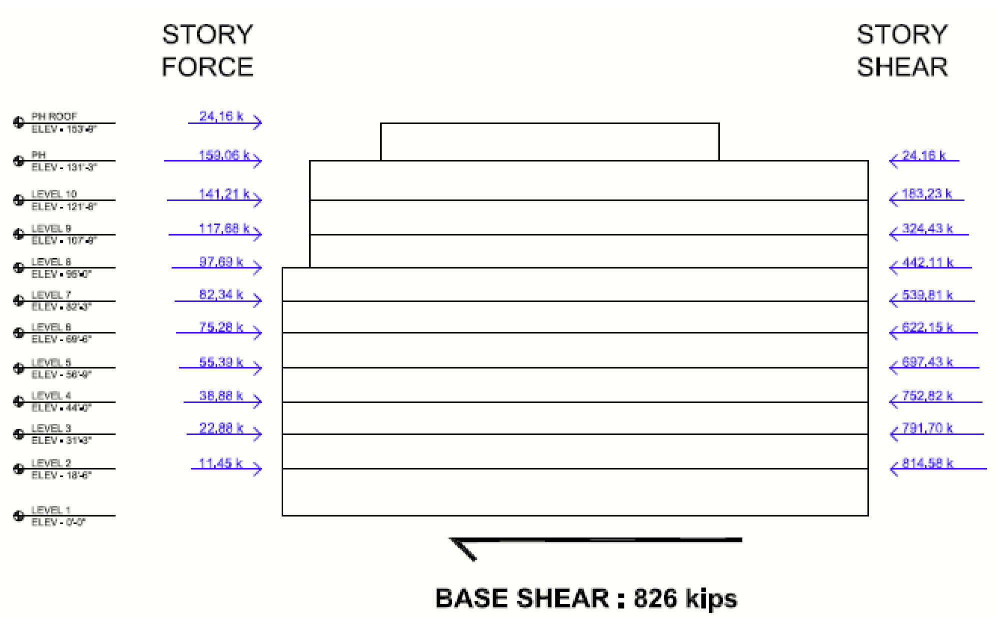


Figure 32: East-West Seismic Building Loads

Design Process – Load Path and Distribution

Loads travel throughout a building's structure laterally and vertically until they reach the ground. The path which loads are distributed is based on member relative stiffness. The members with a higher relative stiffness have larger forces induced into them. Concrete moment frames are incorporated into the east-west lateral system, while shear walls with coupling beams participate in the north-south direction. Coupled shear walls act as a unit when resisting lateral loads. The coupling beams present are composed of the floor slab with increased reinforcement. Their stiffness was based upon slab width and wall thickness.

The code is not very specific about how a two-way post tensioned slab can be accounted for in a lateral resisting system. It is not explicitly stated what calculations are used for determining force distribution, but there have been published engineering research journals on the subject matter.

Pushover analysis research performed by Virote Boonyapinyo, Pennung Warnitchai and Nuttawuk Intaboot on the seismic capacity of post-tensioned slab-column frame building determined the seismic capacity for these systems. Their model was of a 9-story lat-plate building: with and without shearwalls. They had determined that a slab-column system combined with drop panels and shear walls had significant increases in strength and stiffness. Gross member properties are used for slab-beams and columns, along with effects of shear caps being included which increase the flexural stiffness of the slab-column connection. Their findings determined the failure mechanism of slab-beam flexural yielding resulted in considerable building stiffness decrease and the building behaved as a strong column-weak beam mechanism. Drop panels increased the lateral capacity by almost 18% and shear walls increased it by nearly 40%.

Given that the banded tendons along the east-west direction in the column strip were modeled as extremely wide-shallow beams in a moment resisting frame, the lateral loads were transferred through the rigid diaphragm to the beams and finally to either the supporting columns or the shear walls. Shear walls were assumed to not take any out-of-plane forces, but in reality the walls orthogonal to the applied loads would participate by acting similar to the flanges of a steel W-shape.

Research paper 258 for the 8th Nations Conference on Earthquake Engineering was also referenced in thesis redesign. Results from their research determined positive moment developed on one side of the slab-column connection and bottom reinforcement should be provided to reduce the possibility of large crack formation. Some of the authors had also performed similar lateral loading experiments and discussed their findings in "Hysteretic behavior of exterior post-tensioned flat plate connections". They had observed that tendon layout greatly influenced the lateral drift capacity, dissipated energy, failure mechanism, and ductility of the structure. Flexural capacity was met by the post-tensioned tendons prior to punching shear failure, unlike mild reinforcement. Banded tendon layouts had inferior lateral drift capacity, energy dissipation capacity, and punching shear resistance. Use on bonded bottom reinforcement was recommended to provide resistance to moment reversal in high seismic regions.

Design Process – ETABS Computer Model

A computer model was created using ETABS, Computer and Structures Inc. structural modeling and analysis program. The model included the entire structural system of columns, beams and shear walls because their stiffness would participate in transferring lateral forces. Figure 33 depicts columns and shear walls in red, column strips in yellow and the concrete slab in green. Results from the model determined the center-of-rigidity and elements' stiffness and story displacements. Load combinations were entered manually into ETABS based on AISC 7-05. Analysis assumptions that were included in the ETABS model include, but are not limited to:

- Rigid diaphragms modeled at each floor level.
- P-Delta effects taken into account.
- All restraints on the bottom level were modeled as fixed.
- Structural members were modeled without their material properties mass per unit area.
- Shear walls modeled as shell elements meshed into areas with a maximum dimension of 24"x24" to allow for the walls to act as a rigid unit.
    - Shell element resistance properties were manual reduced to minimize the walls capabilities of taking out-of-plane bending.
- Beams and columns were modeled as line elements.
- The moment of inertias of columns and portions of the shears walls were reduced to $0.7I_g$. This is done to account for inelastic response of members and the decrease in effective stiffness.
- Beam elements included a 0.5 rigid end offset multiplier that assume each end to be 50% rigid for bending and shear deformation.
- Seismic loads were applied to the center-of-mass of each floor diaphragm.
- Wind Loads were applied at the center-of-pressure.
- Coupling beams act between the shear wall returns.
    - Coupling beams are sized to be the thickness of the slab, width of the shear wall and material properties of the slab.

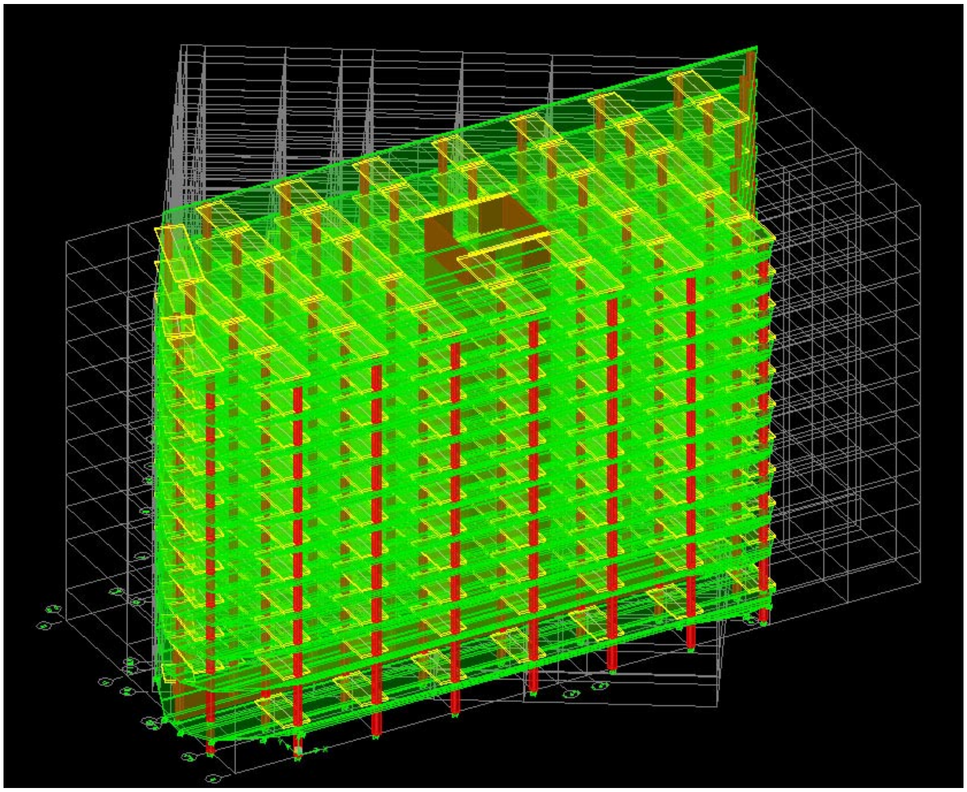


Figure 33: ETABS Model

Design Process – Rigidity and Relative Stiffness

Eccentricities resulting from lateral loads not being applied at the center-of-rigidity (COR) cause torsion on the building. Wind loads are applied at the center-of-pressure (COP), while seismic forces are applied at the center-of-mass (COM). In the case of 800 North Glebe, neither of these two centers coincides with the COR. Refer to Table 15 and Figure 34 to view the difference of the COM to the COR. The floor plan displayed shows all the members participating in the LFRS, which include shearwalls, columns and idealized banded tendon column strips.

	Center of Mass (in)		Center of Rigidity (in)		Eccentricity (in)	
Story	XCM	YCM	XCR	YCR	ex	ey
MAIN ROOF	1128.1	690.3	1202.9	875.5	-74.8	-185.2
10TH	1128.1	690.3	1207.5	872.1	-79.4	-181.8
9TH	1128.1	690.3	1212.1	869.1	-84.0	-178.8
8TH	1128.1	690.3	1216.5	865.4	-88.4	-175.1
7TH	1128.1	690.3	1220.7	860.1	-92.6	-169.8
6TH	1089.1	867.6	1225.1	849.4	-136.0	18.2
5TH	1089.1	867.6	1231.5	827.5	-142.4	40.2
4TH	1087.9	887.2	1237.4	801.9	-149.6	85.4
3RD	1087.9	887.2	1242.4	776.7	-154.6	110.5
2ND	1089.1	886.6	1246.9	757.4	-157.9	129.2

Table 15: Eccentricity Determination

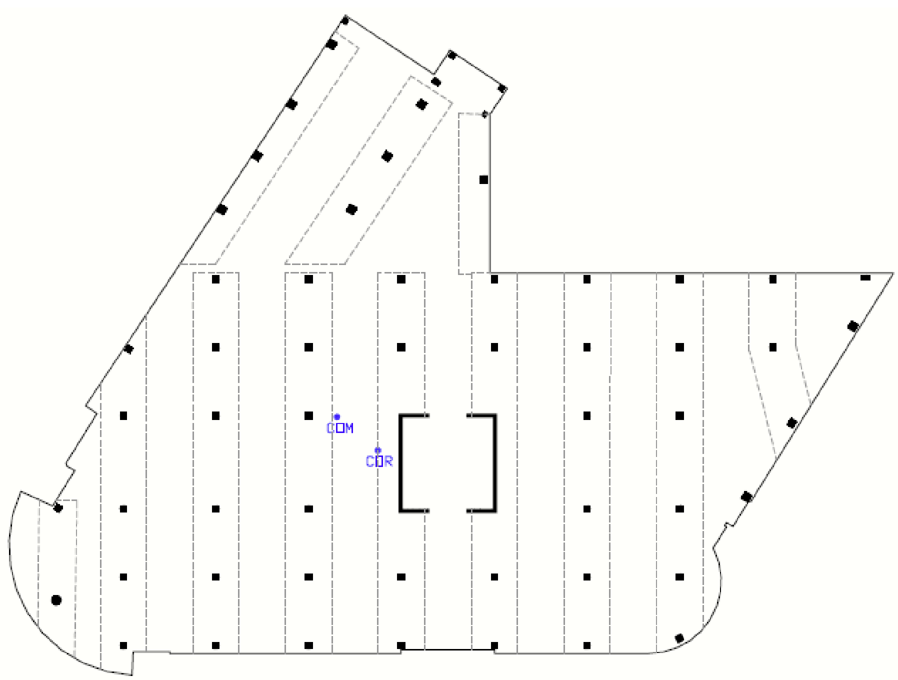


Figure 34: COM vs. COR

Design Process – Torsion

The eccentricity of the COM to the COR causes a torsional moment on the building. AISC 7-05 section 12.8.4 was used to determine this total moment produced by inherent torsion and accidental torsion. Inherent torsion is, as stated by section 12.8.4.1, "For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, resulting from eccentricity between the locations of the center-of-mass and the center of rigidity." Accidental torsion is, as specified by section 12.8.4.2, "The accidental torsional moments, M_{ta}, (kip) caused by assumed displacement of the center-of-mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces." To obtain the overall building moment, M_{ta} was added to M_t, creating the largest torsional moment, shown in Table 16.

	Torsional Moment							
	North-South Direction (Short Wall Resisting)				East-West Direction (Long Wall Resisting)			
Floor	Floor Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	M_t total (ft-k)	Floor Lateral Force (k)	M_t (ft-k)	M_{ta} (ft-k)	M_t total (ft-k)
Main Roof	108.4	-20078.6	1165.3	21244.0	131.0	-9799.2	1637.4	11436.5
10	90.5	-16447.9	972.4	17420.3	109.4	-8682.1	1367.5	10049.6
9	85.3	-15255.9	917.4	16173.3	103.3	-8673.7	1291.2	9964.9
8	80.2	-14041.7	862.0	14903.6	97.2	-8584.4	1214.4	9798.8
7	78.1	-13255.8	839.3	14095.1	94.7	-8766.3	1183.8	9950.2
6	75.7	1381.3	813.7	-567.6	91.9	-12500.8	1149.3	13650.1
5	72.9	2928.5	784.0	-2144.5	88.7	-12637.0	1109.2	13746.2
4	69.6	5942.4	748.1	-5194.2	84.9	-12693.1	1060.8	13753.9
3	65.3	7211.1	701.5	-6509.7	79.8	-12338.5	997.9	13336.4
2	31.4	4053.5	337.3	-3716.2	38.5	-6072.6	480.8	6553.4
			Total	65704.0			Total	112240.0

Table 16: Torsional Moment

It was found that the torsional moment in the east-west direction was larger. This is primarily due to the fact that the building shape does not step back on the perpendicular face, and therefore, the eccentricity stays the same the entire height of the building. The torsional moment in the other direction changes signs on the sixth floor, where the major building set back occurs, switching the eccentricity from negative to positive.

Design Process – Shear

A building experiences a direct shear and possibly a torsional shear when a lateral loads are applied. Direct shear is the force acting on the floor diaphragms applied directly to the lateral resisting members. To determine the direct shear, the story shear was multiplied by the relative stiffness of each participating member.

Torsional shear is the force cause by eccentricity. The torsional shear is similar to torsional moment, as it takes into account the difference in distance from the COM to the COR. The following equation was used to determine the torsional shear.

$$V_i = \frac{V_{tot} e d_i k_i}{J}$$

V_i = torsional shear of element i
V_{tot} = story shear
e = distance from COM to COR
d_i = distance from element I to COR
k_i = relative stiffness of element i
$J = \Sigma k_i \times d_i^2$

A strength check must be performed to verify that each member is capable of transferring both direct and torsional shear. ACI 381-08 section 21.9.4.1, *Special Structural Walls and Coupling Beams Shear Strength* was used for the central core shear walls, and it states:

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

This equation recognizes the higher shear strength of walls with high shear-to-moment ratios. Where chord reinforcement is provided near wall edges in concentrated amounts for resisting bending moments, reinforcement should not be include in calculating ρ_t. However, the extra steel provided in the short shear walls is included for resisting shear forces and therefore shall be accounted for in thesis calculations.

The shear walls included in the duel system were analyzed and the flexural reinforcement was calculated. Maximum moment values from ETABS were used in the calculations and it was concluded 57in^2 would be needed in the outer 10' on either side. Reinforcement could be reduced if axial loads were included in the ETABS model, but for computer analysis programming purposes this was not included.

Design Process – Drift and Displacement

Story drift and lateral displacements are not considered strength design concerns but are regarded as serviceability issues. Seismic drift is addressed in AISC 7-05 while wind drift is not addressed in the code, but, is normally limited to L/400, based on standard engineering practice over the years. In the case of 800 North Glebe:

Wind: Δ_{max} = (153.75' x 12"/1') / 400 = 4.61"

The max wind displacement in the east-west direction (i.e. long shearwalls resisting), from ETABS was calculated to be 0.6326". The calculated displacement at the main roof level is well below the allowable wind displacement of 4.61". When looking at the north-south direction (i.e. short walls resisting), the displacement at the main roof level was found to be 3.2046" from ETABS. Table 17 summarizes the max point displacement per floor.

	North-South (X) Direction				East-West (Y) Direction			
	Wind Load Combo		Seismic Load Combo		Seismic Load Combo		Wind Load Combo	
Story	UX	UY	UX	UY	UY	UX	UY	UX
MAIN ROOF	3.2046	0.0575	3.1034	0.0917	0.6326	-0.1356	0.4349	-0.066
10TH	2.9021	0.0522	2.8194	0.0869	0.5918	-0.1265	0.3908	-0.0577
9TH	2.5783	0.0453	2.5121	0.0807	0.5395	-0.1158	0.3417	-0.0491
8TH	2.2303	0.0373	2.1776	0.0733	0.4783	-0.1032	0.2919	-0.0408
7TH	1.8608	0.0282	1.8196	0.0647	0.3822	-0.1236	0.242	-0.0329
6TH	1.4797	0.0174	1.4489	0.0539	0.338	-0.0734	0.1921	-0.025
5TH	1.0975	0.004	1.0761	0.0384	0.2675	-0.0577	0.1441	-0.0172
4TH	0.7358	-0.0058	0.723	0.024	0.1964	-0.0417	0.1046	-0.0124
3RD	0.4404	0.0003	0.411	0.0121	0.1279	-0.0265	0.0675	-0.0078
2ND	0.1938	0.0035	0.1663	0.0041	0.0653	-0.0129	0.0344	-0.0037

Table 17: Max Point Displacement

Interstory drift was calculated by ETABS for both load cases and can be found in the Tables 18 below. These values do not represent the corrected drift calculation which includes C_d. Corrected values are summarized in Table 19. The limits for interstory drifts at typical floors (12'-9") are 0.375" for wind and the equations for seismic as seen below.

Design: $$\delta_x = \frac{C_d\,\delta_{xe}}{I}$$

Code: $$\Delta_a = 0.02h_{sx}$$

Interstory displacements from the ETABS model is significantly less than the allowable limits for both load cases. The values from floor-to-floor do not deviate from one another by any significant value, with the exception of the 2nd level. Complete tables of drift and displacement values may be found in Appendix E while summaries are found below in Tables 21. Table 22 represents the corrected drift, which takes into account C_d/I.

	Max Drift with Respect to h_n			
	Seismic Load Combo		Wind Load Combo	
Story	Drift Y	Drift X	Drift Y	Drift X
MAIN ROOF	0.000266	0.001856	0.000288	0.001977
10TH	0.000342	0.002009	0.000321	0.002117
9TH	0.0004	0.002186	0.000325	0.002275
8TH	0.000445	0.00234	0.000327	0.002415
7TH	0.000471	0.002423	0.000326	0.002491
6TH	0.000461	0.002437	0.000329	0.002498
5TH	0.000464	0.002308	0.000306	0.002364
4TH	0.000448	0.002039	0.000263	0.002089
3RD	0.000409	0.001599	0.000216	0.001635
2ND	0.00029	0.000739	0.000084	0.000861

Table 18: Uncorrected Interstory Drifts

	Max Drift with Respect to h_n			
	Seismic Load Combo		Wind Load Combo	
Story	Drift Y	Drift X	Drift Y	Drift X
MAIN ROOF	0.001197	0.008352	0.001296	0.0088965
10TH	0.001539	0.0090405	0.0014445	0.0095265
9TH	0.0018	0.009837	0.0014625	0.0102375
8TH	0.0020025	0.01053	0.0014715	0.0108675
7TH	0.0021195	0.0109035	0.001467	0.0112095
6TH	0.0020745	0.0109665	0.0014805	0.011241
5TH	0.002088	0.010386	0.001377	0.010638
4TH	0.002016	0.0091755	0.0011835	0.0094005
3RD	0.0018405	0.0071955	0.000972	0.0073575
2ND	0.001305	0.0033255	0.000378	0.0038745

Table 19: Corrected Interstory Drifts

Column Design

Design Loads

Original column layout for 800 North Glebe required 30”x30” reinforced concrete columns. The new column layout increased the total number of columns and removed the need for post-tensioned girders while reducing the tributary area per column. A standard interior column, as seen in Figure 35 was designed to resist axial loads and moments induced by the slab gravity and lateral loads. A table of the axial gravity loads for the selected column is shown below in Table 20. Moment forces used in design were taken from RAM Concept and ETABS models.

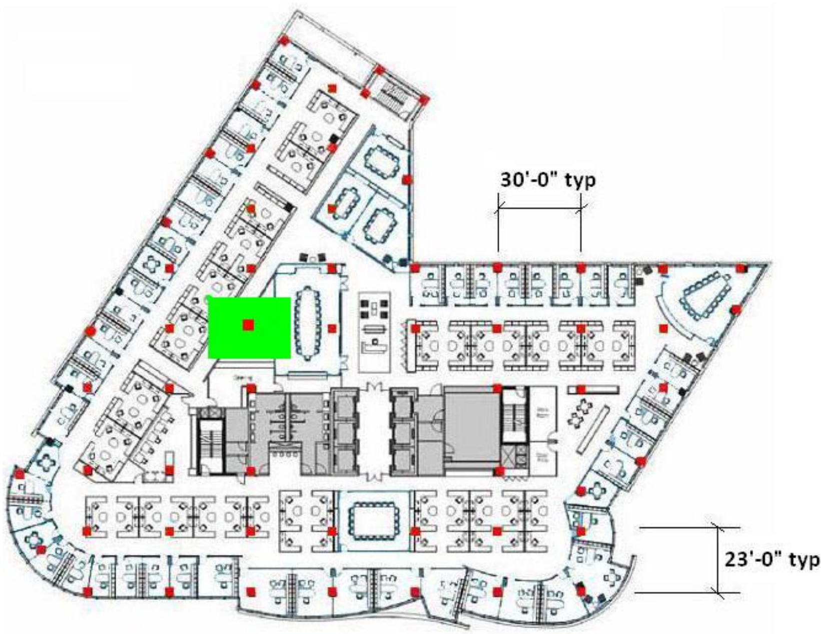


Figure 35: Standard Interior Column

Floor	Tributary Width (ft^2)	Dead Load (lb)	Live Load (lb)	Superimposed Dead Load (lb)	Total Dead Load (lb)	Snow Load (lb)	1.4DL (kips)	1.2DL + 1.6LL (kips)	1.2DL + 1.6S + L (kips)	Total Load (kips)
Roof	660	66000	66000	16500	82500	16500	115.5	204.6	191.4	204.6
10	660	66000	66000	16500	82500	0	115.5	204.6	165.0	409.2
9	660	66000	66000	16500	82500	0	115.5	204.6	165.0	613.8
8	660	66000	66000	16500	82500	0	115.5	204.6	165.0	818.4
7	660	66000	66000	16500	82500	0	115.5	204.6	165.0	1023.0
6	660	66000	66000	16500	82500	0	115.5	204.6	165.0	1227.6
5	660	66000	66000	16500	82500	0	115.5	204.6	165.0	1432.2
4	660	66000	66000	16500	82500	0	115.5	204.6	165.0	1636.8
3	660	66000	66000	16500	82500	0	115.5	204.6	165.0	1841.4
2	660	66000	66000	16500	82500	0	115.5	204.6	165.0	2046.0
1	660	66000	66000	16500	82500	0	115.5	204.6	165.0	2250.6

Table 20: Column Axial Loads

Slab Gravity Moment (RAM Concept)					Lateral Load Moment (ETABS)				
Floor	Interior Column		Exterior Column		Floor	Interior Column		Exterior Column	
	Axial (k)	Moment ft-k)	Axial (k)	Moment ft-k)		Axial (k)	Moment ft-k)	Axial (k)	Moment ft-k)
Roof	204.6	320	122.8	206	Roof	204.6	NA	122.8	NA
10	409.2	320	245.5	206	10	409.2	224	245.5	NA
9	613.8	320	368.3	206	9	613.8	321	368.3	NA
8	818.4	320	491.0	206	8	818.4	368	491.0	NA
7	1023.0	428	613.8	280	7	1023.0	460	613.8	NA
6	1227.6	428	736.6	280	6	1227.6	540	736.6	NA
5	1432.2	428	859.3	280	5	1432.2	580	859.3	NA
4	1636.8	428	982.1	280	4	1636.8	630	982.1	NA
3	1841.4	428	1104.8	280	3	1841.4	643	1104.8	NA
2	2046.0	428	1227.6	280	2	2046.0	716	1227.6	NA
1	2250.6	428	1350.4	280	1	2250.6	623	1350.4	NA

Table 21: Column Axial and Moment Loads

Design Process – Strength Calculation and Detailing

Exterior columns usually carry smaller gravity loads but have larger tensile forces induced by lateral loads. This predominantly results in higher reinforcement percentages. The aforementioned loads were used to design initial column sizes and were then entered into PCA Column to be analyzed. Reinforcing for the columns concerned itself with both gravity and lateral design, where a general reinforcing can be found in Table 22.

Floor	Tributary Width (ft^2)	Column Size	Quantity	Rebar	
Roof	660	16x16	24	#5	8
10	660	24x24	41	#8	12
9	660	24x24	41	#8	12
8	660	24x24	41	#8	12
7	660	24x24	44	#8	12
6	660	24x24	44	#8	12
5	660	24x24	44	#8	12
		30x30	7	#9	16
4	660	24x24	44	#8	12
		30x30	7	#9	16
3	660	24x24	44	#8	12
		30x30	8	#9	16
2	660	30x30	52	#9	16
1	660	30x30	52	#9	16

Table 22: Column Sizing

Detailed calculations for typical columns may be found in Appendix D. Typical column reinforcing details can be seen below in the images of Figure 36.

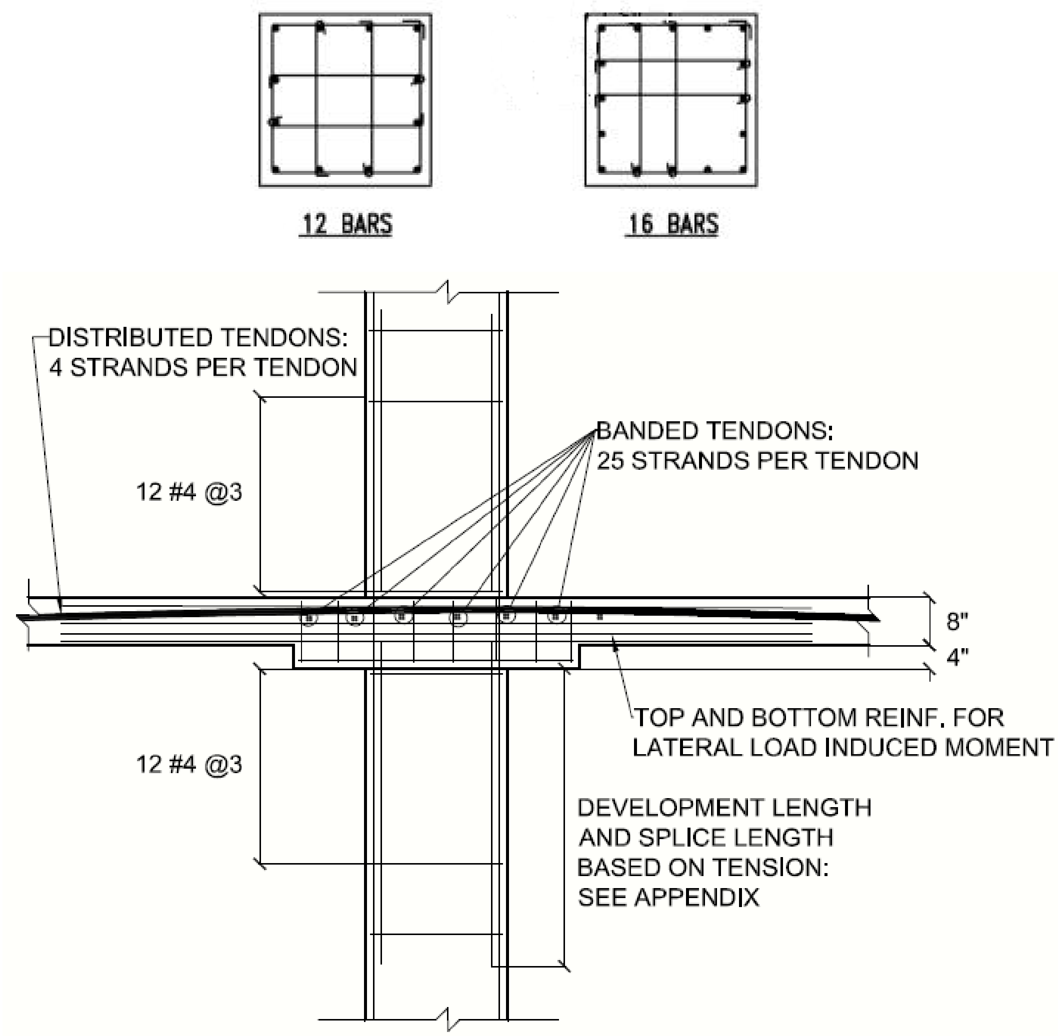


Figure 36: Typical Column Reinforcing Plan and Section

Based on ACI 318-08, rebar development lengths and splicing requirements shall conform to Cass B provisions. Lap Splices are a multiple of the tensile development length l_d, which is calculated in accordance with section 12.2:

$$l_d = \frac{f_y \varphi_t \varphi_e}{20\tau\sqrt{f_c'}} d_b$$

The modified column grid had only been implemented into the superstructure layout and below grade levels were not able to be drastically modified because of parking requirements. A nondirect load path was caused, as seen in Figure 37, and required a row of columns to be designed with a slope.

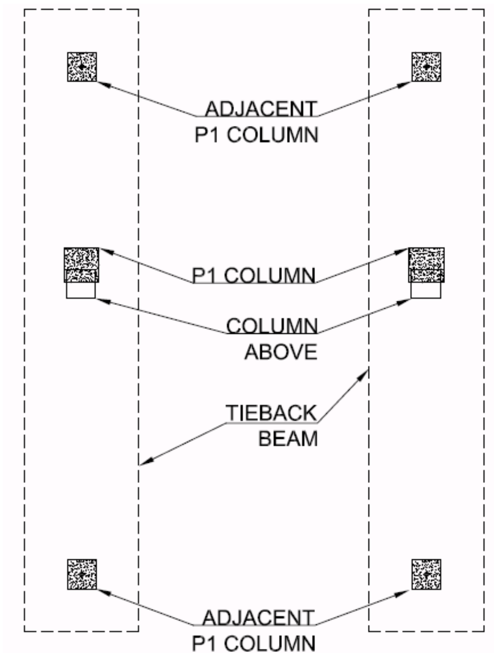


Figure 37: Sloped Column Plan

The columns below the first level do no lie directly under the offset sloped column base, leading to eccentricity and large shear forces on first sublevel columns. Tension was also introduced into the slab around these ground level columns. To resist the high forces in those areas, column corbels and transfer beams were required to be designed using ACI 318 section 11.8. Detailing for the sloped column and corbel are seen in Figure 38.

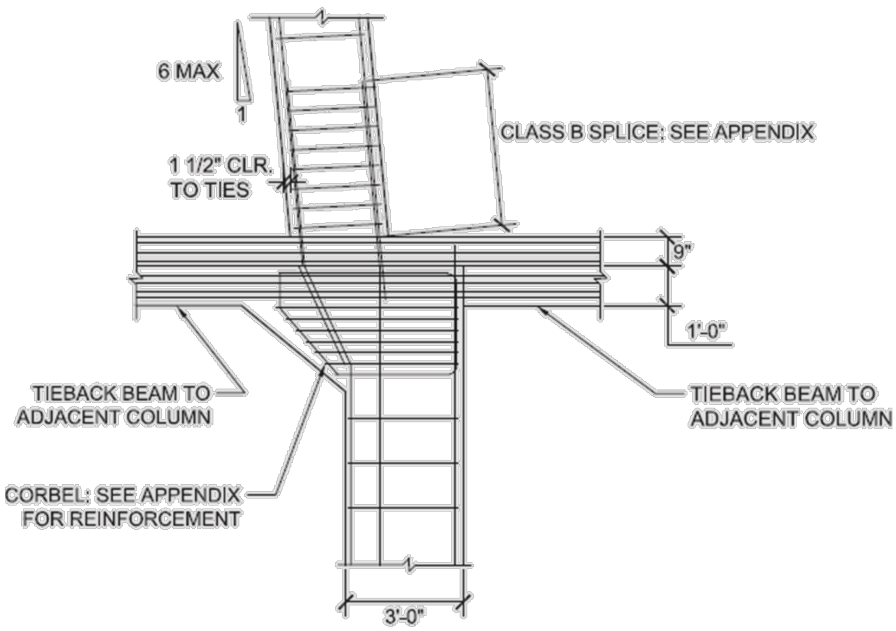


Figure 38: Sloped Column Reinforcing

Foundation Impacts

Overturning and Addition Building Weight

Overturning moments are an important effect to consider because they affect various parts of the building, primarily the foundations. 800 North Glebe includes three levels of below grade parking supported by 30"x30" and 36"x36" reinforced concrete columns. The outer columns along the east face of the building are tied into 72" diameter concrete caissons. The shear walls are supported by a 6'-0" thick concrete mat foundation 58'-6" wide by 45'-4" long.

The size of the supporting foundations was analyzed because of the column layout alteration. Most of the foundation sizes were reduced, but the required number of foundations was increased. The overturning moment, similar to the other calculations, were preformed with wind loads being controlled by case 1. The overturning values for both wind directions are in Table 23 below. Using load case 6, there was an upward reaction on the supporting structure because from the ETABS output. However, gravity loads were not taken into account for the lateral force computer model. The maximum upward reaction was 332 kips and the maximum gravity force reaction on this region is ten times greater.

		North/South Wind		East/West Wind	
Floor	Building Height (ft)	story force (k)	Overturing Moment (ft-k)	story force (k)	Overturing Moment (ft-k)
PH Roof	153.8	62.7	9639.2	75.7	11643.8
PH	135.3	108.4	14661.5	131.0	17716.3
10	121.5	90.5	10990.4	109.4	13291.8
9	107.8	85.3	9194.8	103.3	11129.9
8	95.0	80.2	7617.3	97.2	9229.5
7	82.3	78.1	6421.6	94.7	7789.7
6	69.5	75.7	5260.7	91.9	6390.1
5	56.8	72.9	4138.8	88.7	5035.9
4	44.0	69.6	3062.1	84.9	3734.1
3	31.3	65.3	2039.1	79.8	2494.6
2	18.5	31.4	580.5	38.5	711.6
1	0.0	0.0	0.0	0.0	0.0
Σ Total Overturning Moment (ft-k)=			73606		**89167**

Table 23: Overturning Moment

System Comparison and Conclusion

Modifying the structural slab system from a one-way mildly reinforced slab over post-tensioned girders to a two-way post-tensioned flat plate slab added some inherent benefits to the structural behavior. Reducing the slab thickness from 9" to 8" and removing the need for beams minimized building weight. Deflection for the redesigned slab was determined to meet all ACI 318-08 criteria limits.

The reduction in weight helped to reduce the modal response for both translation directions and for the rotation. Implementing the post-tensioned slab into the lateral force resisting system allowed for the concrete moment frame to take part of the forces and not rely solely on the shear wall core resisting lateral loads. Based upon the impact on the gravity and lateral force resisting systems, the new post-tensioned redesign is a viable alternative.

Master level courses used during the structural depth study include 538: *Earthquake Resistant Design of Buildings* and AE 597A: *Computer Modeling.* The theory of seismic structural behavior learned from these courses gave a broader understanding of how 800 North Glebe should react under such loading. The use of a computer model was vital to the analysis. This allowed for an integrated system design, in which structural members participate with one another.

Architectural Breadth

To accommodate for the slab system redesign, two rows of columns were added. The increase in columns meant the architectural floor plans needed to be altered. The existing layout was studied to determine the proper size of rentable offices and cubicles and great effort was made to keep the same ratios. So as to not diminish the number of offices available, interior partition walls being moved around were kept to as minimal as possible. The owner of 800 North Glebe would like to keep the mix-use office building as a class-A space, and not reduce what they can offer their tenants.

Floor Plan Redesign

Design Process

Floor plans for the current architectural floor plans were available for levels three, five, eight and ten. An office breakdown of the available floors can be seen in Table 24. These levels were assumed to be the same for adjacent floor and therefore not all of the levels were analyzed. Figures 39, 41, 43 and 45 represent the existing office plans created by the architect. Thesis redesign of floor plans can be found in Figures 40, 42, 44, and 46.

Level	Typical Office	Small Office	Large office	Exec. Office	Total Offices	Work stations
3	35	0	11	1	47	79
5	34	0	0	0	34	83
8	23	12	19	1	55	16
10	12	0	5	1	18	65

Table 24: Office Layout Count

LEVEL 3

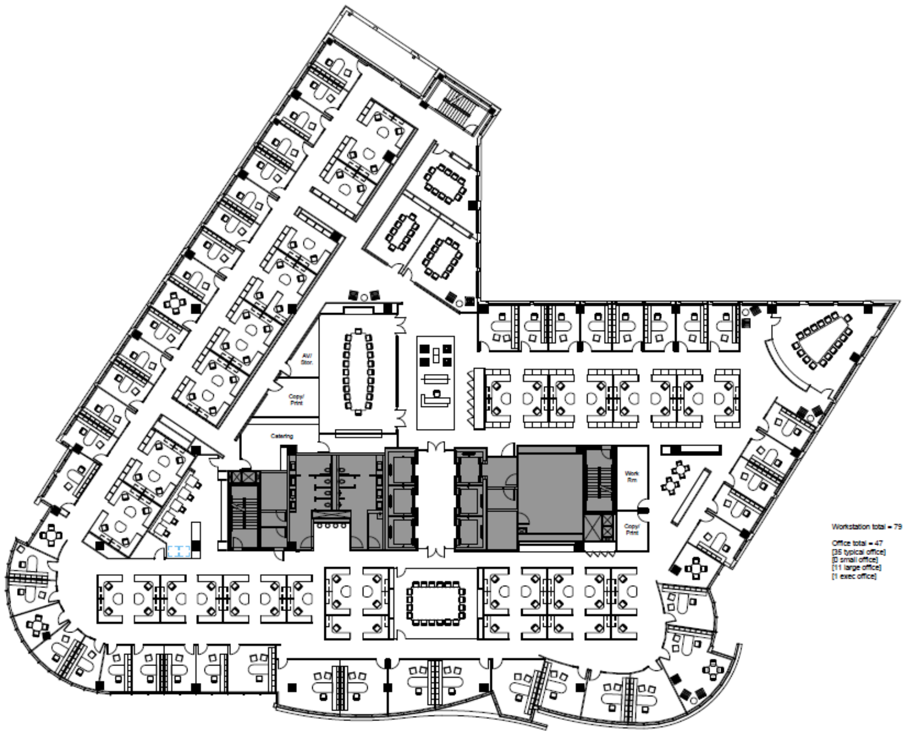


Figure 39: Existing Level 3 Plan

Figure 40: Thesis Level 3 Redesign Plan

LEVEL 5

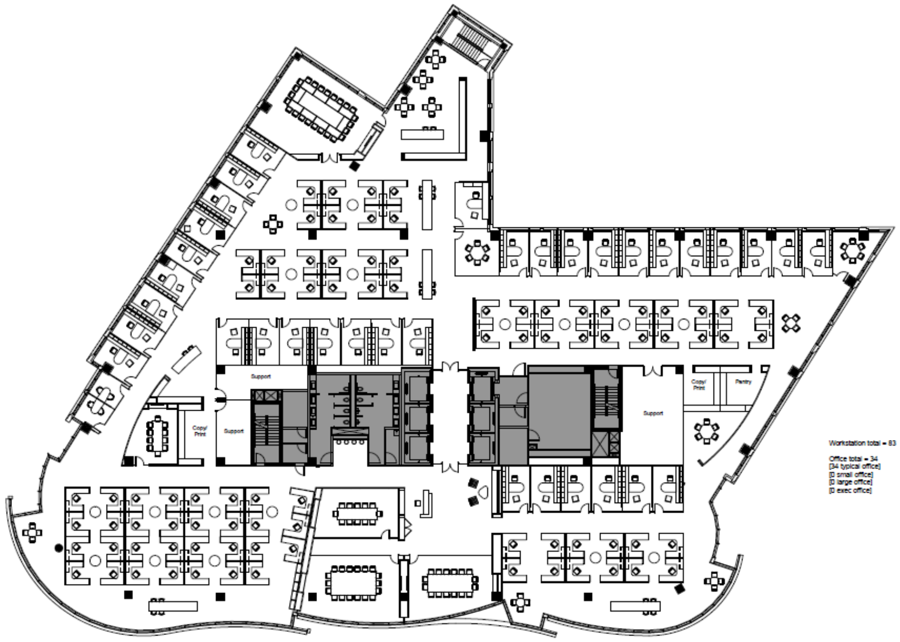


Figure 41: Existing Level 5 Plan

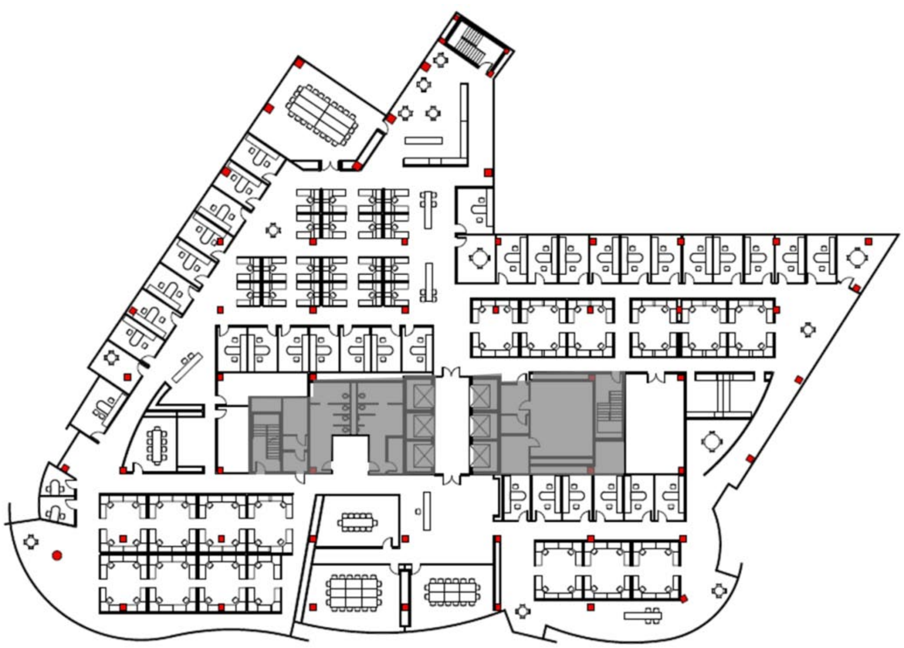


Figure 42: Thesis Level 5 Redesign Plan

LEVEL 8

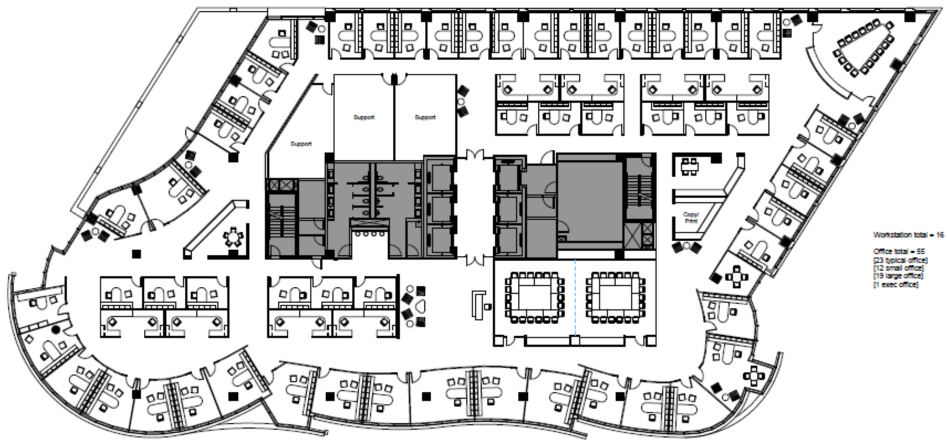


Figure 43: Existing Level 8 Plan

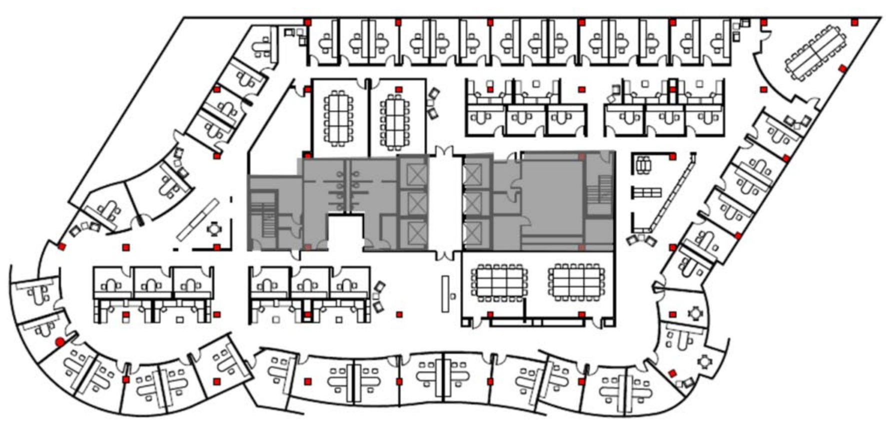


Figure 44: Thesis Level 8 Redesign Plan

LEVEL 10

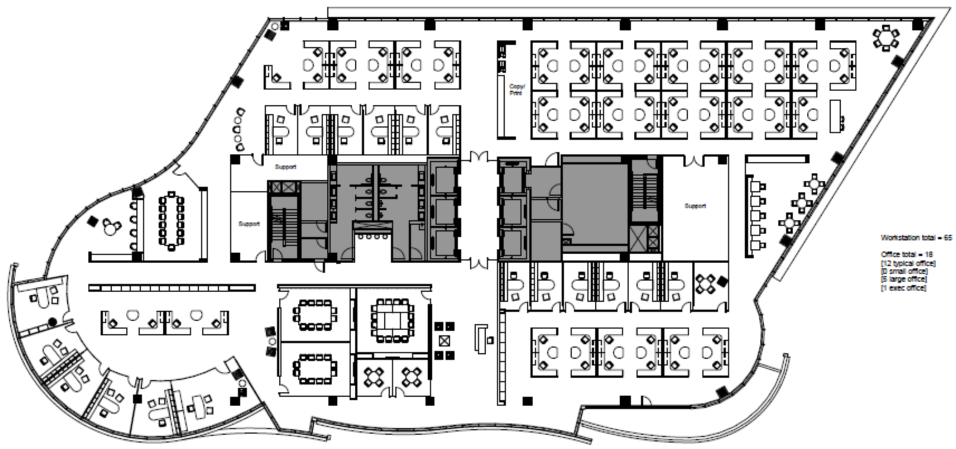


Figure 45: Existing Level 10 Plan

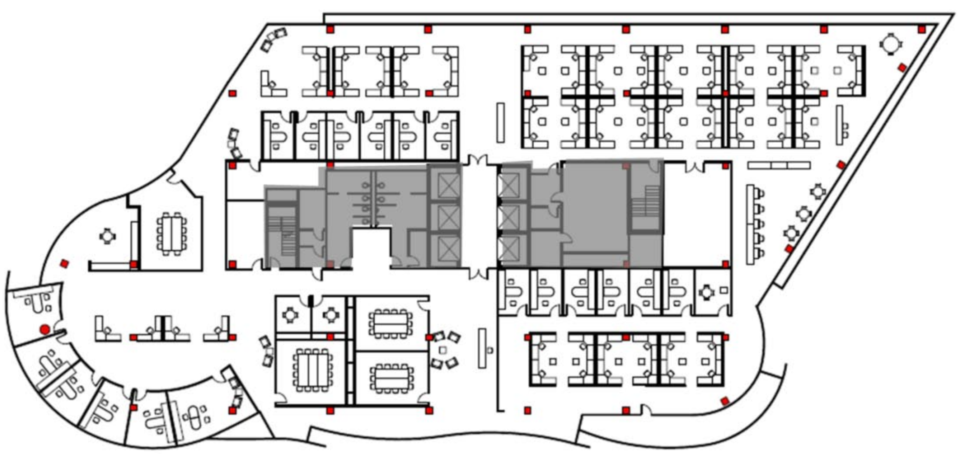


Figure 46: Thesis Level 10 Redesign Plan

Building Envelope Study

Existing Glass Façade Study

The façade system on 800 North Glebe is primarily composed of a glass curtain wall. Since the function of the building is an office, increased day lighting is important for worker productivity. The existing curtain wall system is a stick built system of insulating Viracon VRE glazing with aluminum mullions attached with anchors at each level. The largest insulating glass unit for vision glass on the building is 7’-7 ¼” x 5’-0” composed of ¼” clear Heat Strengthened exterior and interior ply with Low-E coating on the #2 surface, and a ½” air space. Figure 47 is a brief diagram of an insulating glass unit. VRE glazing offers a thin layer of Low-E coating, which offers a good balance between solar energy control and light control, seen in Figure 48.

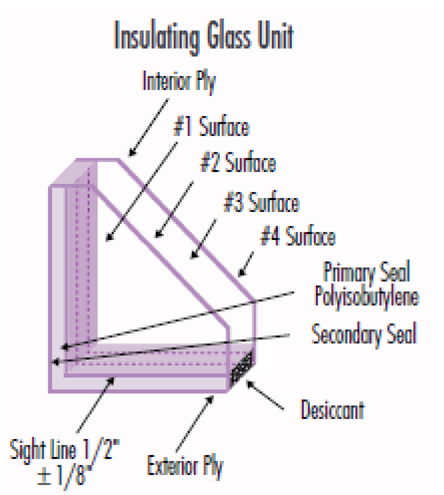


Figure 47: IGU Illustration

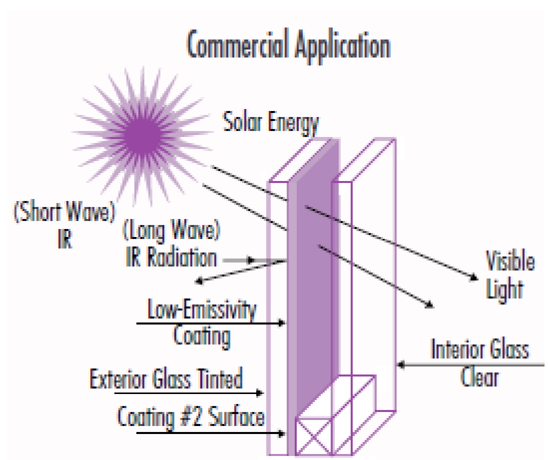


Figure 48: Low-E Affects

Load resistance and deflection were calculated for the glazing based upon previous wind forces calculations. The maximum wind load was 12.44 psf, found at the tenth level. These values were then compared to the allowable values from the manufacturer to determine if the glazing was satisfactory. The aluminum mullions were analyzed to determine their maximum deflection and the load that would be applied to an anchor.

Modification Recommendations and System Detailing

Current glazing panels for 800 North Glebe was determined to be satisfactory for load resistance and deflection. The connections of the glazing to the mullions were then detailed based on manufacturer specification and ASTM E1300, as seen in Figure 49 below. Anchors were then chosen to support the given load and their movement flexibility. The anchor chosen was a Halfen Anchoring System HCWL1/HCWR1 adjustable curtain wall clip. Detailed information and calculations on the materials of the curtain wall system can be found in Appendix F.

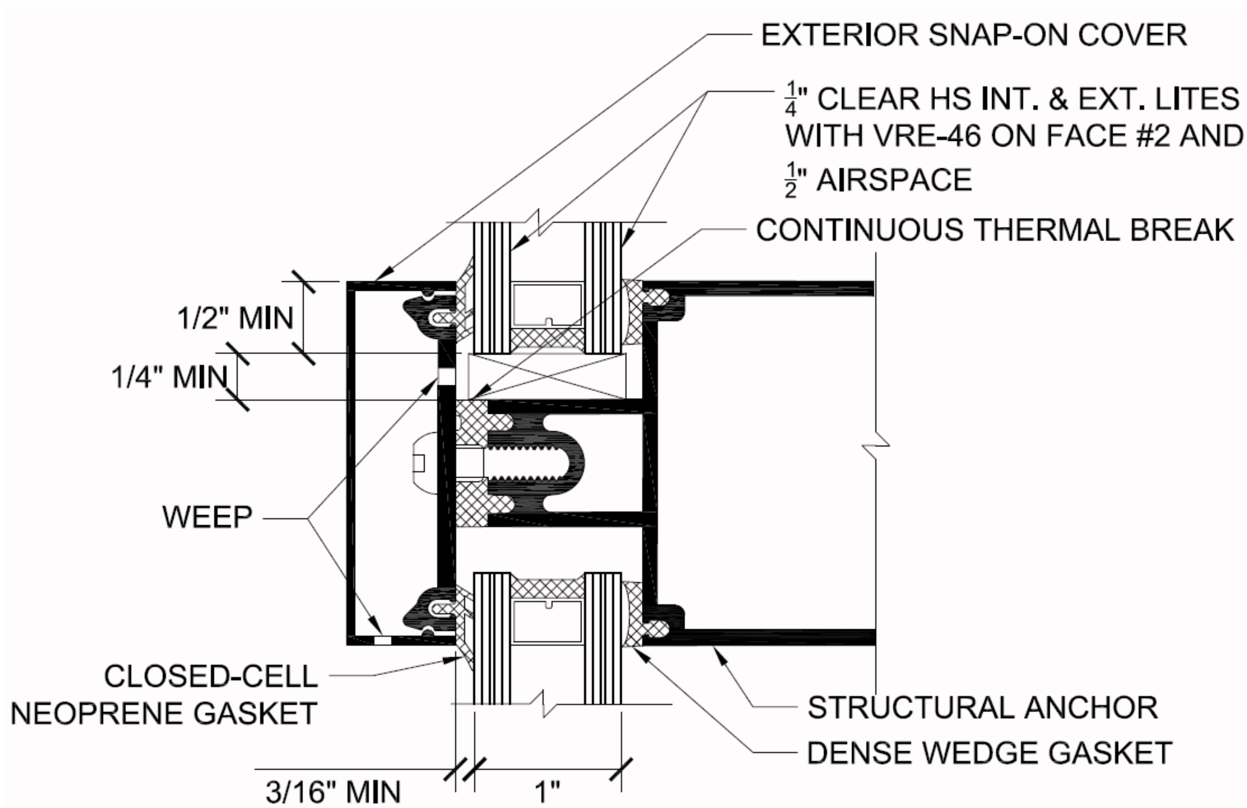


Figure 49: Window Mullion Detail

Construction Management Breadth

The objective of the construction management breadth is to compare a sequencing study and cost analysis the existing design and thesis design for 800 North Glebe. A well designed structure is concerned with resisting all applicable loads while having a constructible building that is cost effective. Changing the slab system to a building wide uniform system will have significant changes on the construction process. The management of the construction site would require a new schedule to allow for more post-tensioning equipment and the timing of the contractors and inspections. Since there was no detailed construction information on the current building system, both the existing and thesis design's construction studies were created.

Only the reinforced concrete structure system was used in the analysis and comparison. Concrete take-offs were performed for both systems, which included concrete, reinforcing and forming. The rest of the construction sequencing and cost, including façade and interior systems, was assumed to remain the same for both designs, and therefore did not contribute to the construction sequencing and cost studies.

Sequencing Study

It had been anticipated that changing the slab-beam system construction from partial post-tensioned design to full post-tensioned design would greatly impact the superstructure schedule. Microsoft Project was used to create schedules for direct comparison of both designs. An arbitrary start date of March 8, 2010 was assumed, because there has yet to be one specified by the owner. Assumptions for the sequencing study include:

- 300 Cubic Yards (CY) daily maximum concrete pours
- 40 hour work week, Monday through Friday
- Portions of the sublevel parking garage structure, shared between 800 and 900 North Glebe was previously constructed due to economical and logistical reasons
- Tendon may not be stressed until 2 days after concrete was poured

The remainder of the building construction for 800 North Glebe will remain similar for both systems. The only difference would be when the subcontractors would be able to begin, which is dependent on their trade. This was not included in construction management breadth.

Existing Building Schedule

A schedule was created for the existing post-tensioned girder/mildly-reinforced slab structural system. It was determined, based upon the previously stated sequencing assumptions that each of the lower six levels would be constructed in three sequences taking 4 days and each of the upper four levels would be two sequences of 4 days per floor as well. Based on the criteria used for sequencing, the total amount of days to construct the existing structure was determined to take 43 days. A summary of the existing structural construction schedule is displayed in Figure 50, while a complete schedule can be found in Appendix G.

	Task Name	Duration	Start	Finish
1	1st Floor Slab	3 days	Mon 3/8/10	Wed 3/10/10
2	1st Floor Column	1 day	Thu 3/11/10	Thu 3/11/10
3	2nd Floor Slab	3 days	Thu 3/11/10	Mon 3/15/10
4	2nd Floor Column	1 day	Tue 3/16/10	Tue 3/16/10
5	3rd Floor Slab	3 days	Tue 3/16/10	Thu 3/18/10
6	3rd Floor Column	1 day	Fri 3/19/10	Fri 3/19/10
7	4th Floor Slab	3 days	Fri 3/19/10	Tue 3/23/10
8	4th Floor Column	1 day?	Wed 3/24/10	Wed 3/24/10
9	5th Floor Slab	3 days	Wed 3/24/10	Fri 3/26/10
10	5th Floor Column	1 day?	Mon 3/29/10	Mon 3/29/10
11	6th Floor Slab	3 days	Mon 3/29/10	Wed 3/31/10
12	6th Floor Column	1 day?	Thu 4/1/10	Thu 4/1/10
13	7th Floor Slab	3 days	Thu 4/1/10	Mon 4/5/10
14	7th Floor Column	1 day?	Tue 4/6/10	Tue 4/6/10
15	8th Floor Slab	3 days	Tue 4/6/10	Thu 4/8/10
16	8th Floor Column	1 day?	Fri 4/9/10	Fri 4/9/10
17	9th Floor Slab	3 days	Fri 4/9/10	Tue 4/13/10
18	9th Floor Column	1 day?	Wed 4/14/10	Wed 4/14/10
19	10th Floor Slab	3 days	Wed 4/14/10	Fri 4/16/10
20	10th Floor Column	1 day?	Thu 4/15/10	Thu 4/15/10
21	Roof Slab	3 days	Mon 4/19/10	Wed 4/21/10

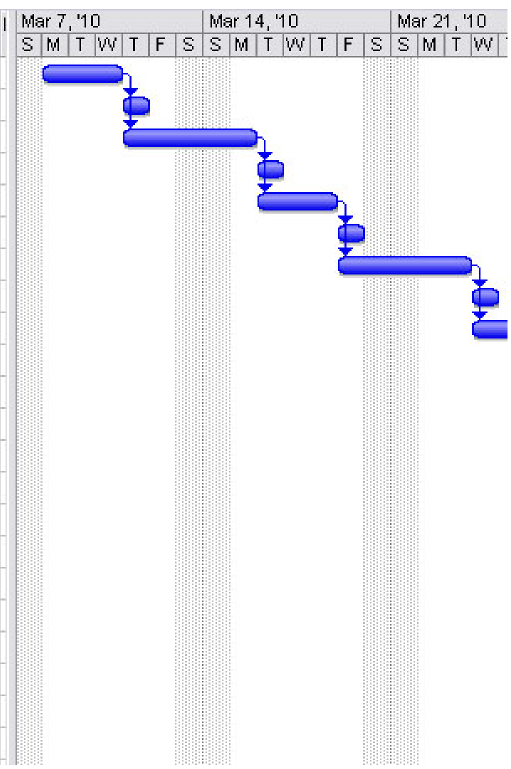
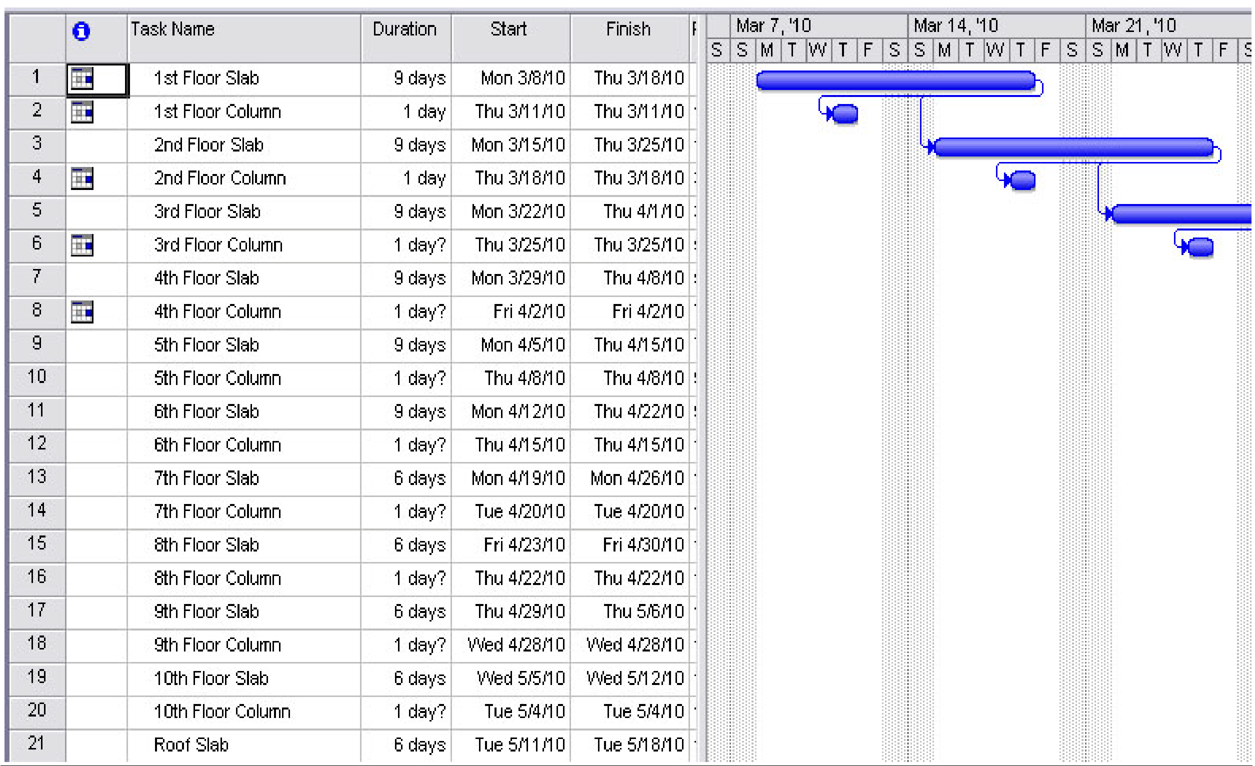


Figure 50: Existing Building Schedule

Thesis Design Schedule

The same assumptions for the existing structure schedule were used on the thesis redesign schedule. Construction sequencing for the thesis design followed the same plan as the existing building sequencing. Total construction time for the thesis redesign was found to be 94 days, which is an increase of 51 days from the existing structure schedule. Due to the post-tensioned system being included in the lateral system, seismic detailing inspection would be needed. This is one of the reasons for the drastic increase in construction time. Refer to Appendix G for a complete structure construction schedule and Figure 51 for a summary.



	Task Name	Duration	Start	Finish
1	1st Floor Slab	9 days	Mon 3/8/10	Thu 3/18/10
2	1st Floor Column	1 day	Thu 3/11/10	Thu 3/11/10
3	2nd Floor Slab	9 days	Mon 3/15/10	Thu 3/25/10
4	2nd Floor Column	1 day	Thu 3/18/10	Thu 3/18/10
5	3rd Floor Slab	9 days	Mon 3/22/10	Thu 4/1/10
6	3rd Floor Column	1 day?	Thu 3/25/10	Thu 3/25/10
7	4th Floor Slab	9 days	Mon 3/29/10	Thu 4/8/10
8	4th Floor Column	1 day?	Fri 4/2/10	Fri 4/2/10
9	5th Floor Slab	9 days	Mon 4/5/10	Thu 4/15/10
10	5th Floor Column	1 day?	Thu 4/8/10	Thu 4/8/10
11	6th Floor Slab	9 days	Mon 4/12/10	Thu 4/22/10
12	6th Floor Column	1 day?	Thu 4/15/10	Thu 4/15/10
13	7th Floor Slab	6 days	Mon 4/19/10	Mon 4/26/10
14	7th Floor Column	1 day?	Tue 4/20/10	Tue 4/20/10
15	8th Floor Slab	6 days	Fri 4/23/10	Fri 4/30/10
16	8th Floor Column	1 day?	Thu 4/22/10	Thu 4/22/10
17	9th Floor Slab	6 days	Thu 4/29/10	Thu 5/6/10
18	9th Floor Column	1 day?	Wed 4/28/10	Wed 4/28/10
19	10th Floor Slab	6 days	Wed 5/5/10	Wed 5/12/10
20	10th Floor Column	1 day?	Tue 5/4/10	Tue 5/4/10
21	Roof Slab	6 days	Tue 5/11/10	Tue 5/18/10

Figure 51: Thesis Redesign Schedule

Cost Study

Cost is as important measure of the success for a project. A design that keeps the cost relatively low is preferred by the owner, which may lead to repeat client work. Cost comparisons were created for both the existing and thesis structure design. These cost analysis were performed using *RS Means 2009 Construction Cost Date* based upon material unit quantity take-offs. Tables 25 display a summary of the material take-offs for 800 North Glebe's structure, while a more detailed list can be found in Appendix H.

System	Material	Units	Existing Building	Thesis Building
Slab	Structural Concrete	Cubic Yards	8600	7400
	Rebar	Tons	138	165
	Tendons	Pounds	NA	332800
Beams	Structural Concrete	Cubic Yards	1730	NA
	Rebar	Tons	123	NA
	Tendons	Pounds	130600	NA
Columns	Structural Concrete	Cubic Yards	2060	2550
	Rebar	Tons	107	131
Foundations	Structural Concrete	Cubic Yards	860	880
	Rebar	Tons	32	31

Table 25: Material Take-off Summary

In order to perform an accurate cost analysis comparison, a few assumptions needed to be made and they include:

- 10% overhead and profit used for analysis
- 2009 labor costs used in study
- Both systems to be built with the sublevel already constructed
- Concrete would be placed by pump

Existing Building Analysis

The total estimated cost of the existing structural design was $10.3 million. A detailed cost breakdown for the existing structure may be found in Appendix H, while Table 26 shows a summary.

	Ext. Mat.	Ext. Labor	Ext. Equip.	Ext. Total	Ext. Mat. O&P	Ext. Labor O&P	Ext. Equip. O&P	Ext. Total O&P	Total
Concrete	$ 1,535,010.49	$ -	$ -	$ 1,535,010.49	$ 1,686,292.28	$ -	$ -	$ 1,686,292.28	**$ 3,221,302.77**
Placing	$ -	$ 159,626.46	$73,546.10	$ 233,172.56	$ -	$ 246,264.96	$ 81,030.26	$ 327,295.22	**$ 560,467.78**
Finishing	$ -	$ 106,267.00	$ -	$ 106,267.00	$ -	$ 154,846.20	$ -	$ 154,846.20	**$ 261,113.20**
Forms	$ 611,789.60	$ 1,508,804.20	$ -	$ 2,120,593.80	$ 674,879.10	$ 2,338,092.70	$ -	$ 3,012,971.80	**$ 5,133,565.60**
Reinforcement	$ 351,817.06	$ 1,185.56	$ -	$ 353,002.62	$ 386,391.24	$ 1,937.46	$ -	$ 388,328.70	**$ 741,331.32**
Post-tensioning	$ 82,260.36	$ 87,483.24	$ 2,611.44	$ 172,355.04	$ 90,094.68	$ 142,323.48	$ 2,611.44	$ 235,029.60	**$ 407,384.64**
	$2,580,877.51	$1,863,366.46	$76,157.54	**$ 4,520,401.51**	$ 2,837,657.30	$2,883,464.80	$ 83,641.70	**$ 5,804,763.80**	**$ 10,325,165.31**

Table 26: Existing Building Cost Summary

Thesis Design Analysis

Total cost for the thesis redesign structure was estimated to be $11.0 million. The increase in cost for the new system is heavily based upon the amount of labor need for post-tensioning. The detailed thesis redesign cost breakdown is located in Appendix H and Table 27 is a brief summary.

	Ext. Mat.	Ext. Labor	Ext. Equip.	Ext. Total	Ext. Mat. O&P	Ext. Labor O&P	Ext. Equip. O&P	Ext. Total O&P	Total
Concrete	$ 2,057,650.71	$ -	$ -	$ 2,057,650.71	$ 2,265,231.69	$ -	$ -	$ 2,265,231.69	**$ 4,322,882.40**
Placing	$ -	$ 140,595.26	$64,728.48	$ 205,323.74	$ -	$ 216,727.77	$ 71,320.78	$ 288,048.55	**$ 493,372.29**
Finishing	$ -	$ 106,267.00	$ -	$ 106,267.00	$ -	$ 154,846.20	$ -	$ 154,846.20	**$ 261,113.20**
Forms	$ 528,996.27	$ 1,299,878.79	$ -	$ 1,828,875.06	$ 583,930.53	$ 2,012,747.52	$ -	$ 2,596,678.05	**$ 4,425,553.11**
Reinforcement	$ 221,492.70	$ 1,185.56	$ -	$ 222,678.26	$ 243,260.44	$ 1,937.46	$ -	$ 245,197.90	**$ 467,876.16**
Post-tensioning	$ 209,664.00	$ 222,976.00	$ 6,656.00	$ 439,296.00	$ 229,632.00	$ 362,752.00	$ 6,656.00	$ 599,040.00	**$ 1,038,336.00**
	$ 3,017,803.68	$1,770,902.61	$71,384.48	**$4,860,090.77**	$ 3,322,054.66	$2,749,010.95	$ 77,976.78	**$6,149,042.39**	**$ 11,009,133.16**

Table 27: Thesis Redesign Cost Summary

Construction Management Conclusion

Construction time was increased by approximately 120% and the cost was 7% higher for the post-tensioned system. Table 28 was created to show the comparison values for the existing and thesis designs. Post-tensioning increases construction time by double the original design. The primary reason for this increase was because of the wait time between pouring a slab and stressing the tendons. During this wait time, the contractor is able to beginning forming and continues with other duties that were not taken into account in the construction sequence. Based upon the aforementioned sequencing and cost analysis comparison, the thesis structural redesign is not the optimal system.

	Existing Structural System	Thesis Structural System	% Incease
Cost	$10.3 Million	$11.0 Million	7%
Time	43 Day	94 Days	118%

Table 28: Construction Comparison

Final Conclusion and Recommendations

Post-tensioning is one of many different slabs systems that could have been implemented into the new column redesign. A post-tensioning system was chosen for various reasons, which include:

- If a significant part of the load is resisted by post-tensioning, non-prestressed reinforcement can be simplified and standardized to a larger degree
- Material handling is reduced since the total tonnage of steel (non-prestressed + prestressed) and concrete is less than for a R.C. floor
- Allows earlier stripping of formwork
- Most of the permanent loads are balanced by post-tensioning, thus considerably reduces tensile forces and deflection which cause increased cracked
- Reduction in slab thickness, and in turn floor weight and material consumption

The bay sizes were reduced from a 30'-0"x46'-0" grid to a 30'-0"x23'-0" grid, thus increasing the total column quantity. The column sizes were reduced on the upper seven levels from 30"x30" to 24"x24". Column grids matched one another on the superstructure levels, but the interface to the below level parking garage required sloping a row of columns on the first level and adding a corbel-transfer beam system on the first sublevel. Also, changing the column grid unquestionably affected the architectural floor plans of the building. The existing layout was studied to determine the proper size of rentable offices and cubicles and great effort was made to keep the same ratios. So as to not diminish the number of offices available, interior partition walls being moved around were kept to as minimal as possible.

The construction management sequencing and cost analysis of the structural system for both the existing design and the thesis redesign concluded that the original design was more cost effective. Because 800 North Glebe is a spec office building, immediate revenue upon completion is a primary concern. The thesis system was concluded to take more time and be more costly; 51 more days to complete and costing nearly 6% greater.

Based upon all of the information from the thesis work, it has been concluded that the original design is a more efficient system. The minimal gains that accompany the structural behavior do not outweigh the time and cost implications of the proposed system.

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