Senior Thesis Final Report

8621 Georgia Avenue
Silver Spring, Maryland

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Advisor: Dr. Thomas Boothby

April 8, 2015

8621 Georgia Avenue
Silver Spring, Maryland

General Building Data:

Building Height: 161 feet
Number of Stories: 17 floors
Size: 347,009 ft^2
Cost: $51 million
Occupancy: Mixed Use
-Residential, Parking Garage, Retail
Construction: Beginning in 2015

Architecture:

The façade of the building brings a refreshing modern addition to the skyline of the developing city of Silver Spring. The position of the building takes advantage of two major view corridors in the urban fabric and has an inviting present on the busy Georgia Avenue.

Structural Systems:

This concrete building utilizes mild reinforced cast-in-place two way flat slabs with full drop panels for the parking garage on floors 1-4 and a post-tensioned cast-in-place two way flat slab for the remainder of the apartment level floors. The lateral system is comprised of 14 concrete shear walls located around stair and elevator cores. The column grid is relatively square vary from 16-24' in length.

Construction:

Construction is scheduled to be 24-28 months and will begin in early 2015. Important factors will be coordinating work with the surrounding existing buildings on all sides and impact of the high water table on the foundation construction.

MEP:

Floors 1-4 (parking garage) will be open and designed as an open structure. Each apartment will be conditioned by a conventional split system heat pump with back-up electric heat. Outdoor air is provided by an exterior louver.

Lighting / Electrical:

The building will have 277/480V as the primary power with 480-120/208V transformers. Branch lighting/power panels will be placed in the cellar and every 4^{th} apartment level. These panels serve the local receptacles, lighting, and HVAC units.

Project Sponsor: Holbert Apple Associates

Nick Dastalfo I Structural
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For more information, see my CPEP site:
http://www.engr.edu/ae/thesis/portfolios/2015/njd5133/index.html

Acknowledgments

I would like to thank the following people for helping me through my thesis this year:

- The engineers at Holbert Apple Associates for providing me the opportunity to use the 8621 Georgia Avenue project for my senior thesis. They, especially David Smith and David Holbert, were a tremendous help in supplying me with all of the information and plans that I needed.

- I would also like to thank all of the engineers at Pennoni Associates Inc. for the time effort and tutelage during my internships. The knowledge I gained regarding instruction in RAM and the general engineering process made thesis go a lot smoother.

- I would like to thank the entire AE faculty for their passion for their students and instruction over the past 5 years. Knowledge learned in every AE class contributed to the basis for this project as well as my future career.

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Executive Summary

The building at 8621 Georgia Avenue is proposed to be built on an existing 0.69 acre parking lot located in the downtown business district of Silver Spring, Maryland. The 17 story, 347,000 ft^2 project will create more downtown multi-family housing and parking for the booming region. Construction on the project began at the beginning of 2015 and is anticipated to take 20-24 months.

The originally designed structure of the building begins with a dual system of mat foundation and spread footings. The gravity system on the first four floors of the building, which will be utilized as a parking garage, consists of two way concrete with the use of drop panels. The 12 remaining floors above are post-tensioned concrete slabs. The lateral system of the building consists of 14 shear walls. A structural overview of the existing concrete system is presented in greater detail within the first portion of the report. The remainder of the report with focus on the steel redesign of the building.

The primary structural redesign of the building was accomplished by implementing a composite beam-girder system for the apartment levels atop the existing concrete parking garage. The stringent height restriction in the area controlled a lot of the design decisions. Bay sizes were limited to be cooperate with the architecture of the apartments and parking garage as well as to minimize beam depth. To accommodate a tight height restriction, a level of parking garage was moved below grade which lent itself to a redesign of the foundation system.

The lateral displacements on the building were amplified due to the decreased building mass. The lateral system was redesign to accommodate the new building stiffness and deflections. The existing concrete shear wall system was adjusted to fit in with the steel redesign and multiple moment frames were added to reduce displacements as well as building torsion.

Two breadth topics were investigated as results from the steel redesign. One breadth is related to the parking garage ventilation while the other is a construction cost analysis. Previously, the parking garage levels were designed as open air structures, but with a level being below grade, a ventilation system needed to be designed. Finally, an extensive cost analysis was performed on the building to determine the feasibility of the redesign.

After investigations were completed, it was found that the steel redesign is feasible and relatively cost effective, but it may not be the most efficient system. Due to the minimized bay sizes and beam size requirements to minimize vibrations, the steel members are not as optimized as they could be. The steel system also would increase project schedule and potentially cause problems on what appears to be a very condensed sight in an urban setting. Therefore, the steel redesign option could be a feasible option for a building owner but not the system that I would personally recommend.

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Note: All renderings of the building or diagrams depicting plans, sections, or elevations are provided by Holbert Apple Associates.

General Building Description

The building at 8621 Georgia Avenue is owned by FP Wilco, LLC. in the downtown business district of Silver Spring, Maryland. The new 17 story, 347,000 ft^2 building will provide 4 floors of parking and 13 floors of apartments to the residents and workers in the area. The total height of the building will be 161 ft. The building is designed to reach to the exact allowable height mandated by the zoning height ordinance. As of the 50% permit drawings, the project is anticipated to cost $52 million dollars.

Picture 1: Rendered image from Southwest. Image courtesy of Holbert Apple Associates.

Great efforts were made in the design process to earn a LEED Silver rating for the building. The location of 8621 Georgia Avenue permits sustainable transportation features such as being within a half block of the nearest metro stop and includes parking amenities for bicyclists. Water drainage issues were also strongly considered for this urban, impervious site downtown. The green roof helps reduce the carbon footprint of the building while simultaneously helping to manage a significant portion of the water run-off.

The first floor has a dual function as the space serves both the private residents of the building as well as the public. The program on the first floor includes a Cyber Café, Fitness room, apartment lobby, and parking spaces (including bicycle and ADA parking). All of these areas, except the parking garage, are double height ceilings and are accessible from the street.

The parking garage portion of the structure continues up from the ground to the 4^{th} floor and includes a total of 197 spaces. These first four floors are the only portion of the building that maintains its' rectangular footprint. Starting at the 5^{th} floor, above the parking garage, the form of the building takes on a U-shape with a green roof with box planters in the center of the 'U'.

Floors 5 through 16 are occupied with 292 multi-family apartments of varying sizes with accessible balconies. The upper residential floors are serviced by two stair towers and three elevators. The typical floor plan for the apartments is repeated until the penthouses on the 16^{th} floor. The rooftop of the building is adorned with a pool, bathhouse, club, and rooftop garden terrace.

The façade of the building is comprised of precast concrete panels, a glass curtain wall system, and a masonry veneer. The precast concrete panels only occur at the levels of the parking garages. The apartment levels feature a prefinished aluminum panel curtain wall system as well as a masonry veneer on the west elevation. The details of how these façade elements are tied into the structure will be discussed later in this report.

Structural Overview of Existing Design

Brief Structural Description

Similar to the surrounding structures, 8621 Georgia Avenue is made of primarily concrete. The foundation of the structure is supported by concrete columns and piers along with spread footings, strip footings, and foundation walls. The shear wall cores are located by the stair towers and elevator towers which span the entire height of the building and are responsible for resisting the majority of the lateral loads. The first four floors utilize mild reinforced flat plate concrete slabs for the floors of the parking garage. Four inch drop panels are used throughout and additional beams are only used in situations where they were absolutely necessary to meet the design parameters. The 5th floor and above utilizes post-tensioned flat plate concrete slabs. This design choice to use post-tensioning was made to maximize floor to floor heights amidst the stringent zoning height ordinance.

A brief summary of the structural materials used in the project are given below.

Concrete		
Use	Strength (psi)	Weight (pcf)
Footings	3000	145
Foundation Walls	4000	145
Shear Walls	5000	145
Columns	5000-7000	145
Interior SOG	3500	145
Exterior SOG	4500	145
Reinforced Slabs / Beams	5000	145
Parking Structure	5000	145

Reinforcement	
Use	Grade
Deformed Reinforcing bars	ASTM A615, Grade 60
Weldable deformed reinforcing bars	ASTM A706
WWF	ASTM A185
7-wire Low Relaxation Prestressing	ASTM A416, Grade 270
Full Mechanical Connection	DYWIGAG, Lenton Or equivalent meeting ACI 318-12.14.3

Figure 1: Concrete and reinforcements materials and specifications.

Steel		
Use		Grade
Wide Flange		ASTM A992
Structural Shapes and Plates		ASTM A36
Structural Pipe		ASTM A53, Grade B, Fy = 35ksi
HSS		A500, Grade B, Fy = 46ksi
Cold-Formed Steel		ASTM A653 (G-60 Galv.)
	<43 mils	Fy= 33 ksi
	>54 mils	Fy= 50 ksi
Fasteners		
Use		Grade
High Strength Bolts		ASTM A325
Anchor Rods		ASTM F1554, Grade 36
Threaded Rods		ASTM A36
Shear Studs		ASTM A108

Figure 2: Fasteners and Steel materials and specifications.

Foundation System

A geotechnical study was done on the site by Schnabel Engineering Consultants, Inc. who was able to provide useful recommendations for the foundation to the design team and structural engineer. Spread footings and column footings were advocated as good choices for the foundation system. The column footings were recommended to be designed with an 8,000 psf soil bearing capacity while the wall footings were suggested to be 6,000 psf.

The proximity of the water table to the depth of the foundation was a principal concern in their geotechnical evaluation. The groundwater table will only be approximately 5ft. below the lowest level (electrical cellar) in some locations on the site. This observation of the site called for sub-drainage materials adjacent to the foundation walls which will be bearing soil pressure.

Typical Foundation details are shown below:

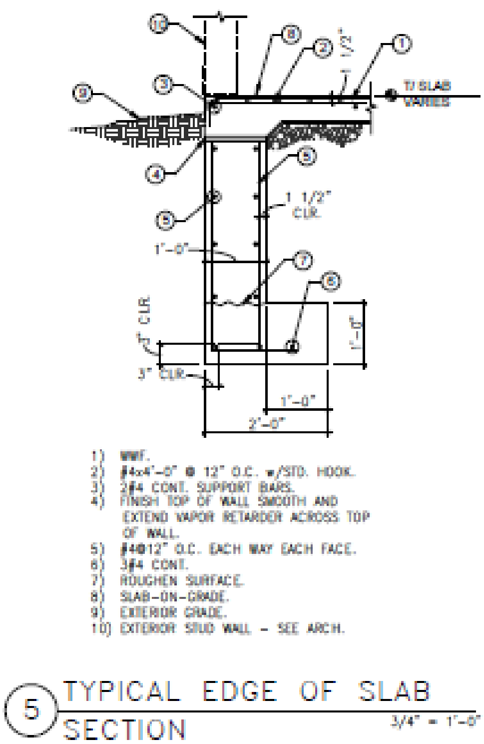


Figure 3: Typical Slab Detail. From S2.01

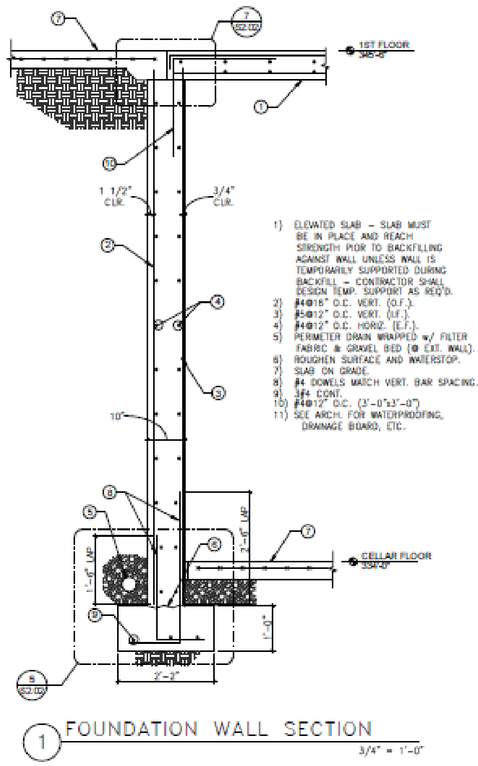


Figure 4: Typical Foundation Wall Detail. From S2.01

Only a small portion of the buildings' total footprint, approximately 4,854 ft^2, goes below grade. This area is strictly for service use with electrical rooms, storage, and mechanical rooms. This level utilizes foundation walls to resist the lateral force of the soil pressures.

The geotechnical report on the soil composition of the site estimated the equivalent fluid pressure on these foundation walls to be 50 psf.

The foundation system also utilizes three mat foundations beneath the three stair towers. Two of the mat foundations are on ground level, while the third is beneath the electrical cellar. Their thicknesses vary from 3 to 4 feet.

The loading for the typical foundation wall is shown below. The loading shown is assuming that there is a surcharge applied above.

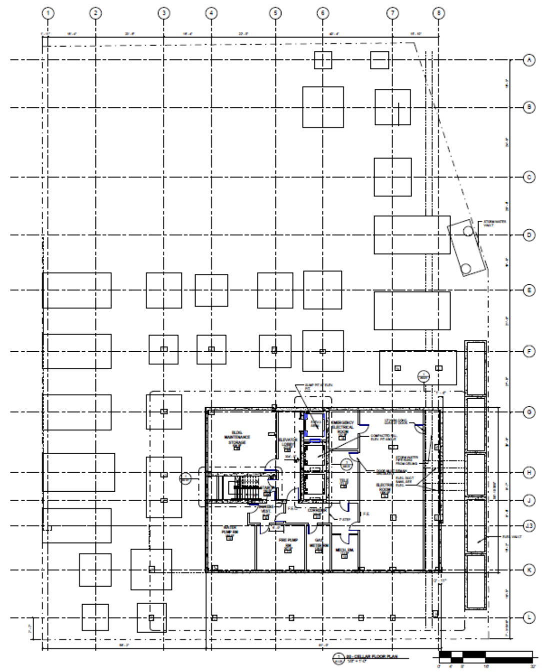


Figure 5: Cellar level floor plan

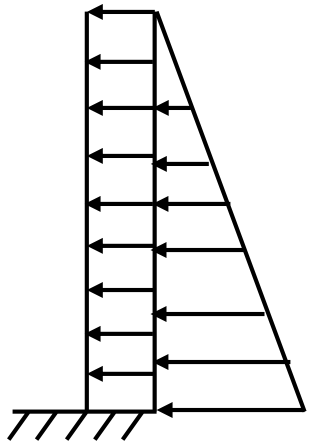


Figure 6: Foundation wall loading due to surcharge and soil pressure.

Gravity System

As previously mentioned, 8621 Georgia Avenue is a concrete structure utilizing flat plate slabs throughout the building for the floor system. Drop panels are used only on the parking levels but are avoided on the apartment floors to maintain a spacious floor to floor height. The slab on grade is 8" thick mildly reinforced concrete slab and has an 18" step in elevation. In the floor system above the sub-grade cellar, a drop in the slab is required.

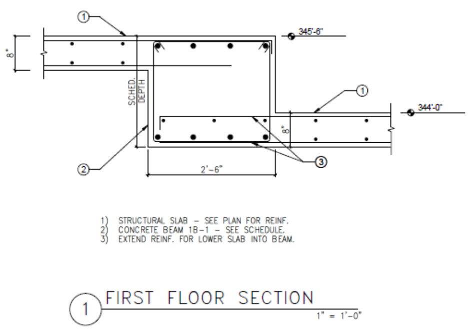


Figure 7: Slab on Grade above Cellar

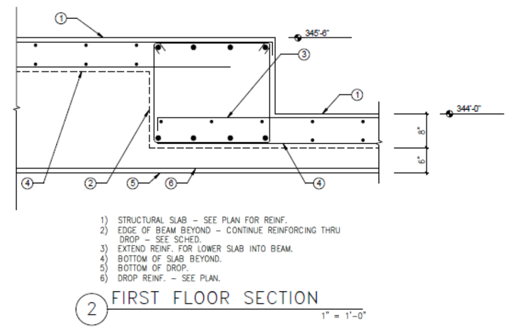


Figure 8: Typical Slab on Grade

Parking Garage

In the first 4 floors, as well as the first apartment level on the 5th floor, the structure will feature an 8" deep mild-reinforced cast-in-place two-way flat plate concrete slab system. The drop panels at each interior column will be 8' x 8' x 4" while the drop panels on the exterior columns will be 4' x 4' x 4".

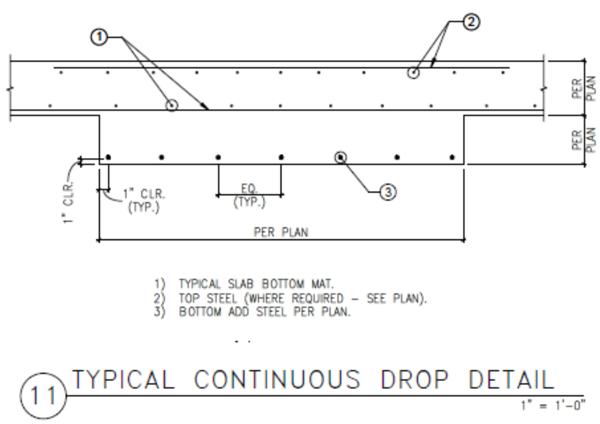


Figure 9: Typical Drop Panel

Apartments

Above the 5th floor and for the remaining floors, the structure consists of a 7.5" deep post-tensioned cast-in-place two-way flat plate concrete slab system. The use of drop panels and beams was minimized but is needed in some locations to control long-term slab deflections for longer spans. The post-tensioning system will be discussed in greater detail later in this report.

Typical Bay

A typical bay size for the project varies with columns' spaces ranging from approximately 16 ft. to 24 ft. in each direction. These bay sizes are consistent throughout the whole building despite the functional transition from parking to residential. The larger bays are located where the drive lane of the parking garage is. Because the same column locations are continued up the entire building height, there was not the need for sloped columns or large transfer girders. The only situations where transfer girders were needed were at the second floor due to the transition from retail/lobby space to the parking structure and also adjacent to the pool at the top of the building.

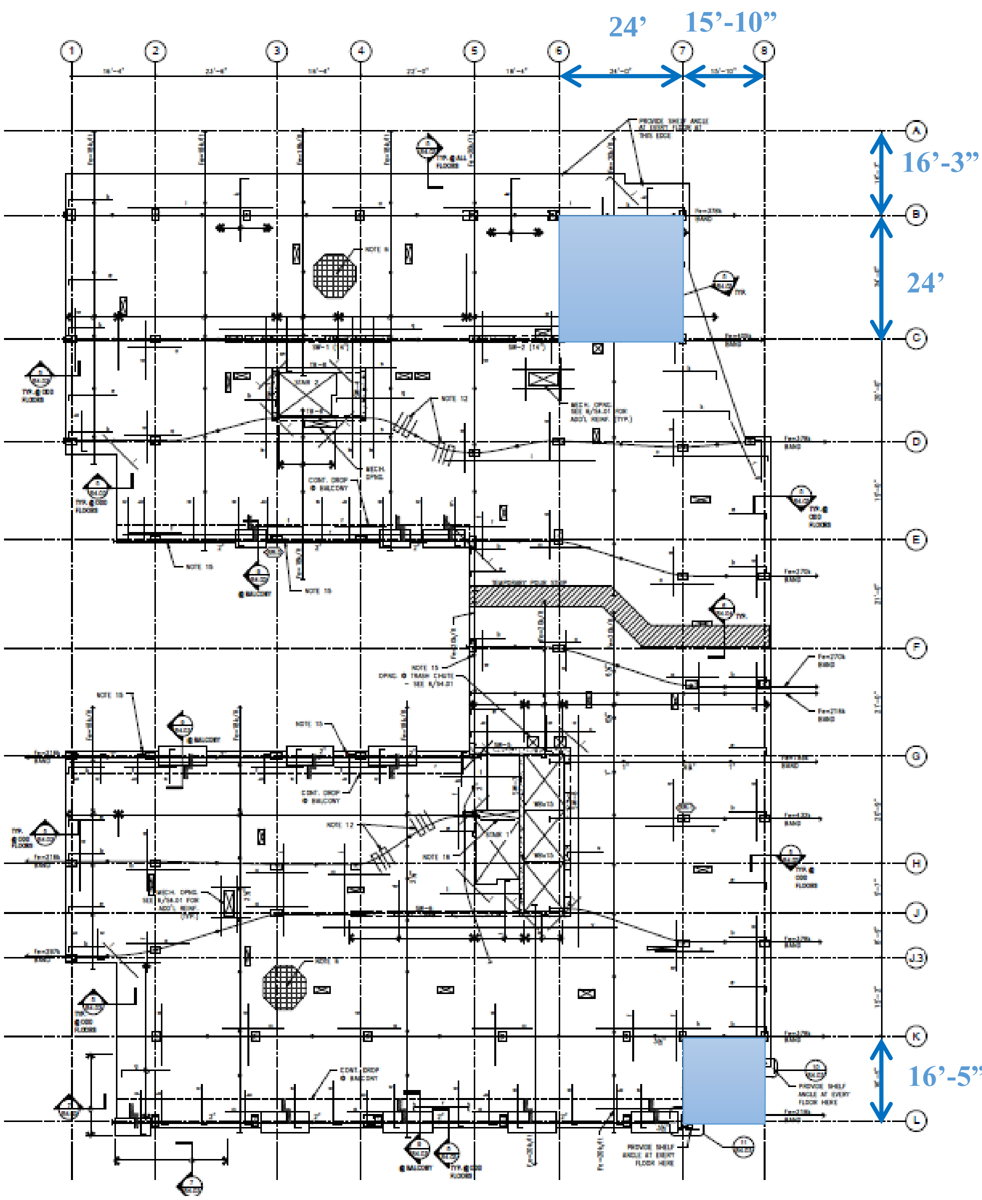


Figure 10: Typical Bays Analyzed in Technical Report 2

Columns

In order to accommodate the accumulated load in the lower floors, the concrete columns change in size and strength throughout the height of the building. Three different strengths of concrete are used in the columns throughout the project. The concrete strength increases in the lower floors to handle the higher axial compression loads without having to make the columns huge. This structural design decision will reap benefits by saving space in the apartment and parking garage floors.

Concrete Columns	
Location	Strength (psi)
Above 8th	5000
4th-8th	6000
Below 4th	7000

Figure 11: Concrete Columns Strengths

The column sizes generally seem to increase slightly by 2”- 4” in each dimension below the 4th floor. Although the column sizes and strengths change, the reinforcing in the columns is uniform throughout the entire height of the building.

Post Tensioning

Floors 5 through 16, which house the multi-family apartments, utilizes post-tensioning in the floor slabs. Both banded tendons and uniformly distributed tendons are used in addition to other mild steel reinforcing. The banded tendons typically run in the plan east-west direction while the uniformly distributed tendons span across the plan north-south direction.

The banded tendons vary in strength from 216 kips to 513 kips while the distributed tendons have a linear strength varying from 18 k/ft to 22 k/ft. The figure below shows the locations of these post-tensioned cables on the typical apartment framing plan.

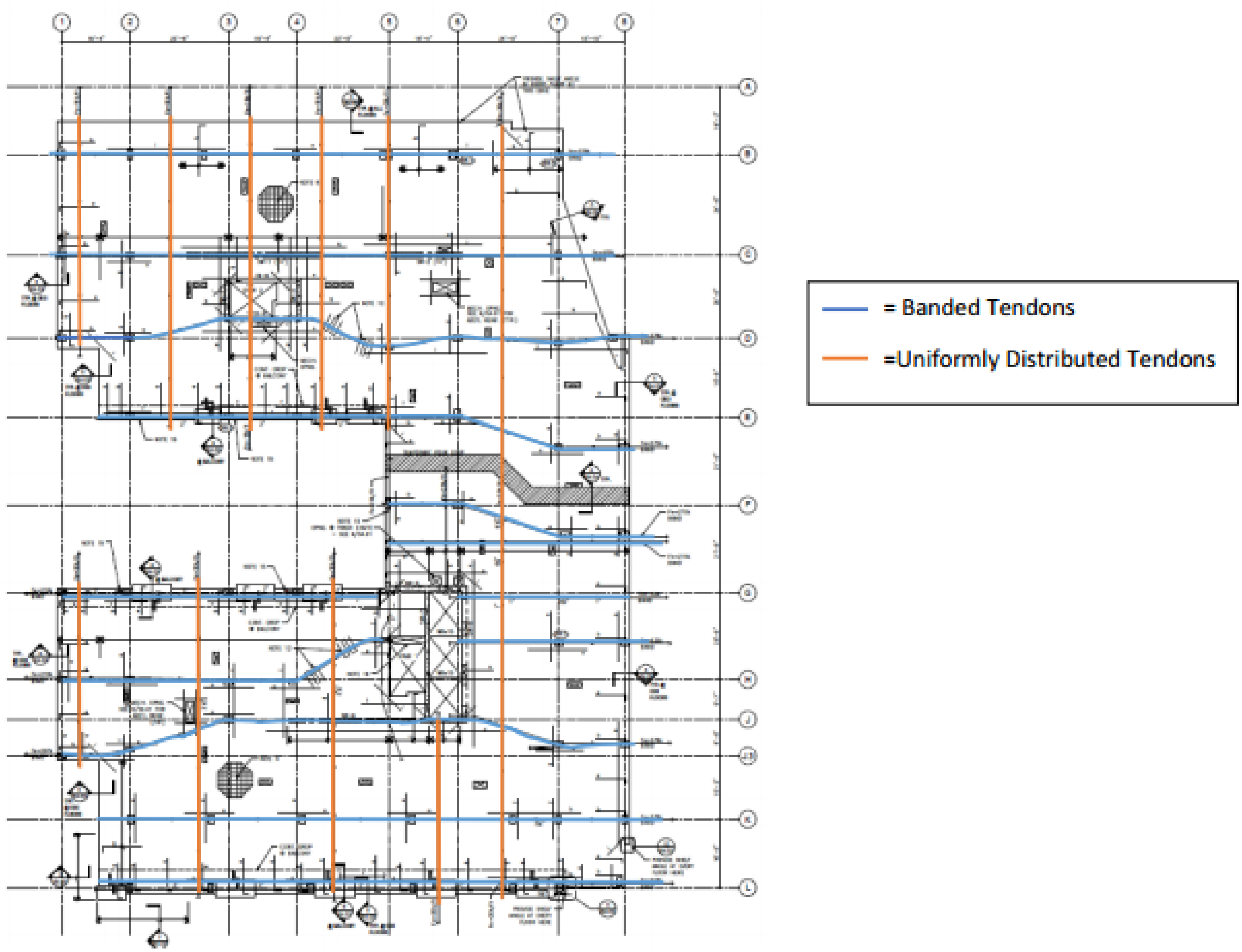


Figure 12: Post Tensioning Arrangement

The post-tensioned strands do not span straight across the building in the center of the slab, but oscillate between the top and bottom of slab depending on its position relative to columns or any openings. The detail below shows the typical band orientation when being placed within a slab.

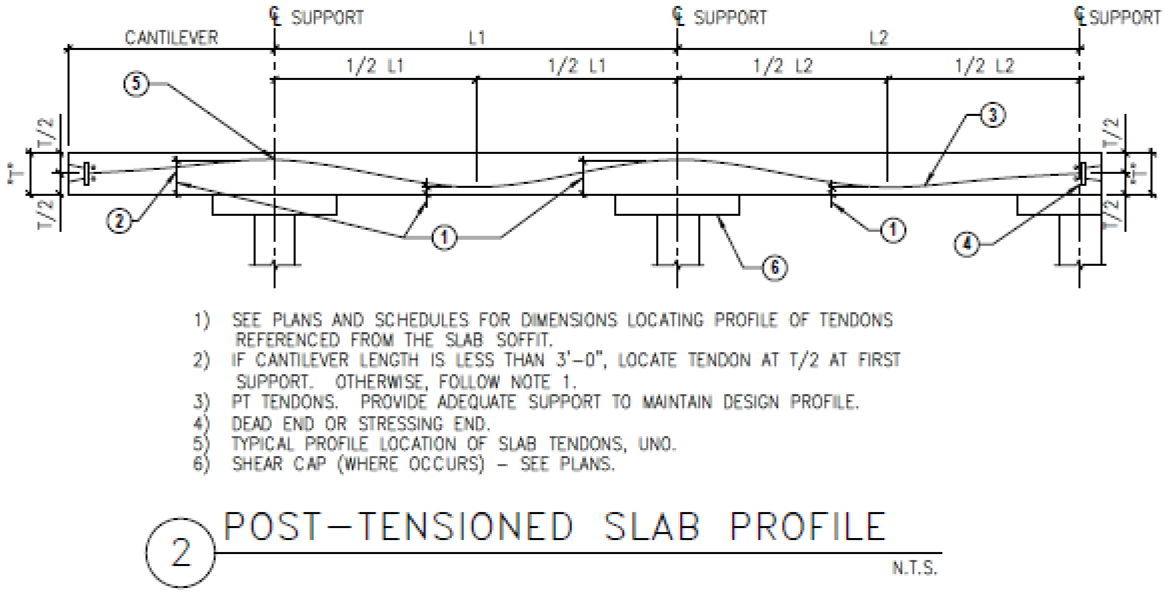


Figure 13: Typical Post-Tensioning Slab layout

Roof System

The roof area of 8621 Georgia Avenue is highlighted by having an 18' x 56' pool. The structure around the pool will consist of a mild-reinforced cast-in-place concrete slab and beam system. The pool will basically be a large concrete box filled with the appropriate waterproofing materials. An isometric view of the 16th floor pool level with a club, locker room, roof terrace, and other apartment suites is shown below.

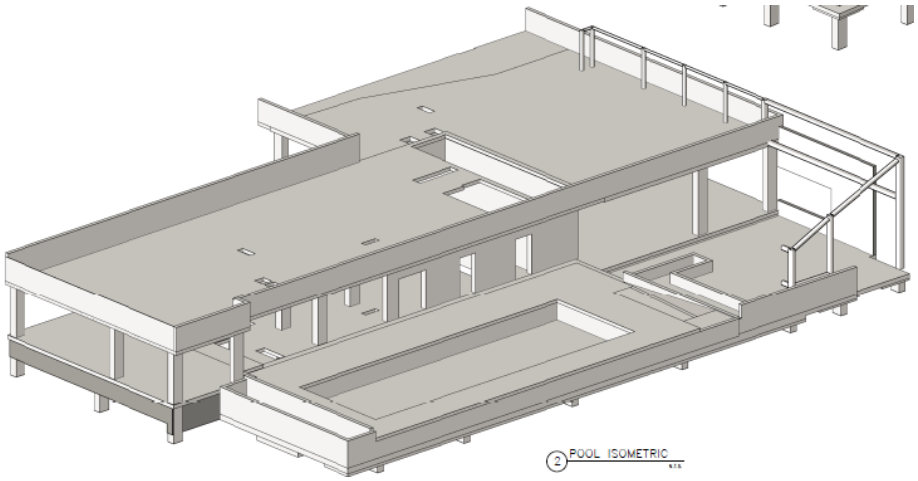


Figure 14: 16th Floor Isometric View

The roof construction is the same post-tensioned concrete two-way slab that is present in the floors below. A 1' layer of concrete topping is added to the slab then completed with a terrace finish.

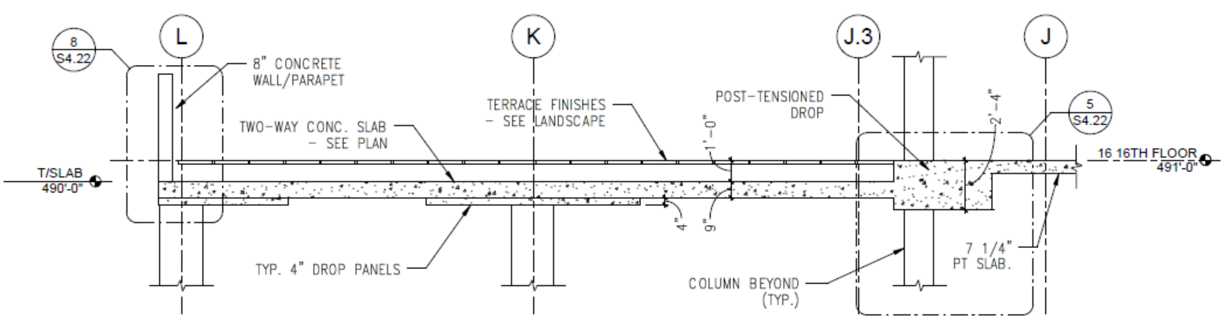


Figure 15: 16th Floor Section

Underneath the pool, the slab is depressed by 16" before additional concrete slabs and walls are built up upon it to house the pool. A section through this condition of the 16th floor slab is shown below.

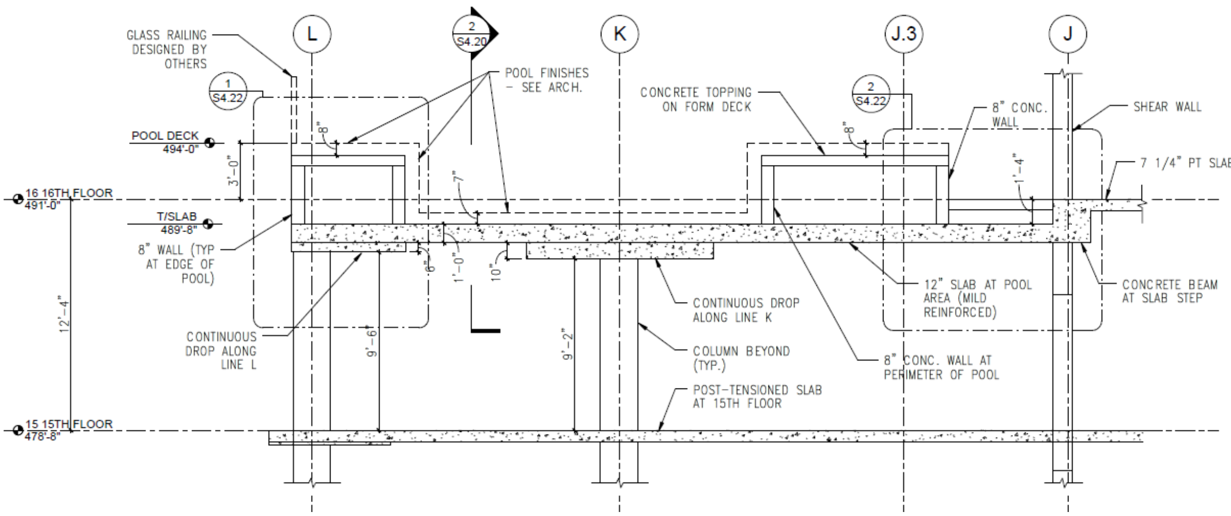


Figure 16: 16th Floor Section through Pool

Bio-Retention Area

On the fifth floor the footprint of the building plan changes and steps back into a ‘U’ shape from a rectangular form. The center of this ‘U’ is home to a bio-retention area and outdoor terraces accessible to the apartment occupants.

To deal with the massive 600 PSF superimposed dead load of the bio-retention area and surrounding planters, the concrete slab is increased to 12” thick in this section of the floor plan. The drop panels on the interior columns run continuous through the 3 columns directly supporting the bio-retention area. In these locations, the total slab thickness will be 20 inches.

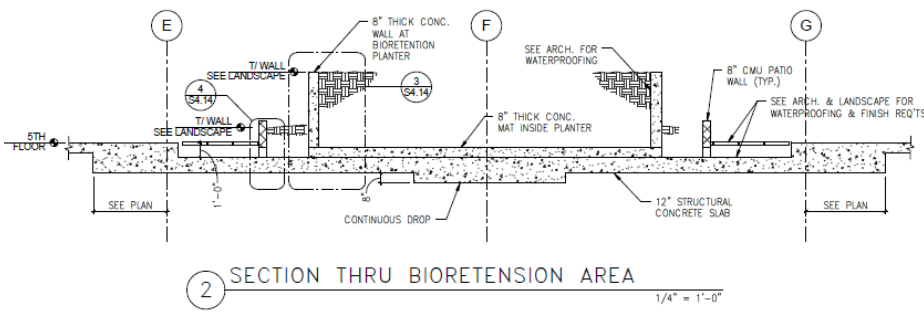


Figure 17: Bio-Retention Cross Section

To accommodate the bio-retention area and planters, small 8” thick concrete walls resist the soil pressure from the potentially saturated beds of soils and foliage.

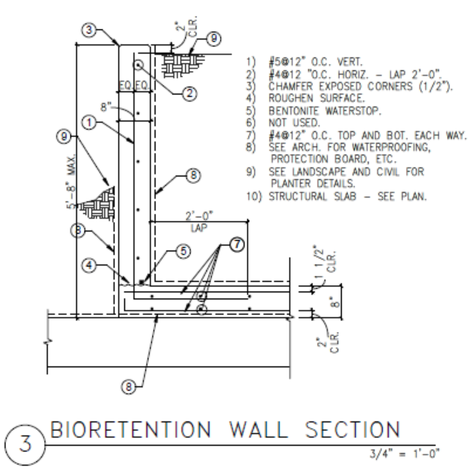


Figure 18: Bio-retention wall

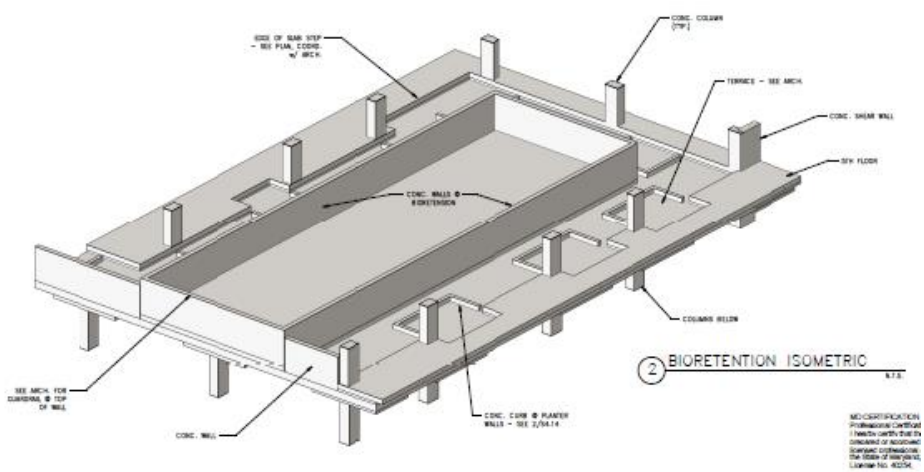


Figure 19: Bio-retention Isometric

Lateral System

The Lateral Force Resisting System (LFRS) of 8621 Georgia Avenue consists of 14 regular concrete shear walls that are 12" thick. These shear walls are concentrated around the stair and elevator towers within the building. A few concrete moment frames exist in various bays but the majority of LFRS elements are the aforementioned shear walls.

The reinforcing in each wall calls for #5's at 12 inches on center, each way, each face. This is a fairly typical rebar arrangement for shear walls and is kept uniform across each shear wall regardless of height or location. The figure below shows the locations of the shear walls.

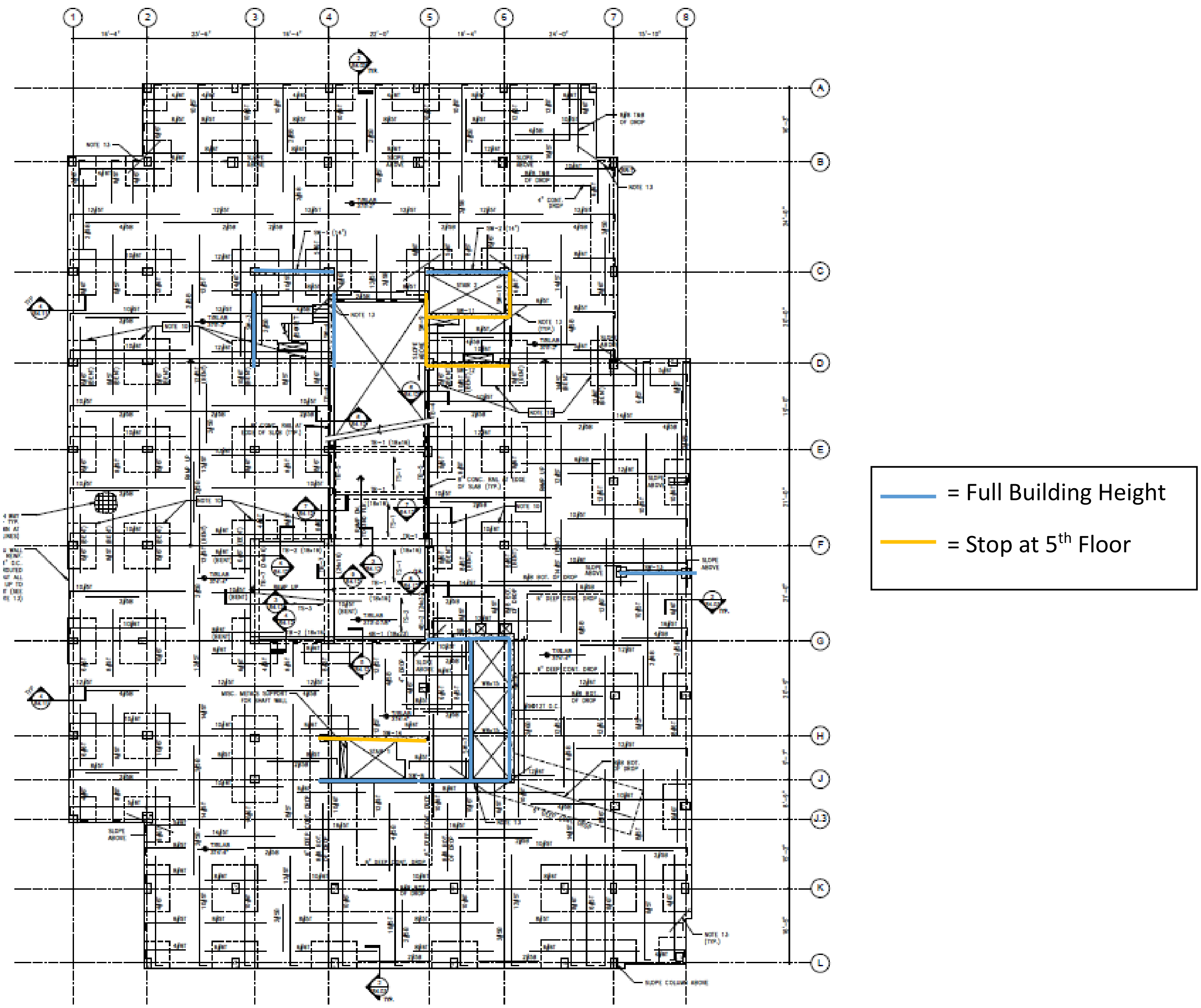


Figure 20: Shear Wall Locations

Load Paths

Gravity

The gravity loads from the building are those caused by the combination loading of the dead and live loads. These loads will be resisted by the concrete floor slabs at each level. The slabs will distribute the load to the nearest columns (or shear walls) by which its' bay is bound by. The columns will then carry the load directly down the building and into the foundations, and eventually into undisturbed, virgin soil.

The figure to the right gives an example of the load path in a section of the building due to gravity loads.

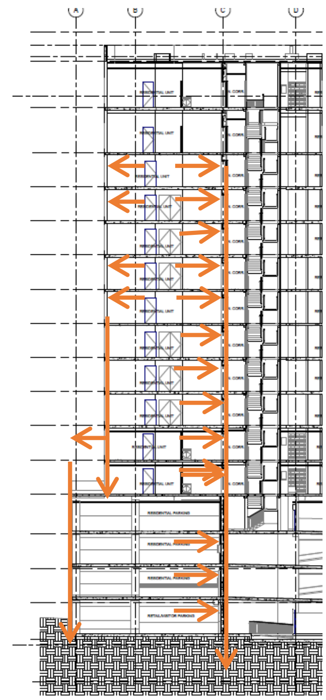


Figure 21: Gravity Load Path

Lateral

The controlling lateral load on 8621 Georgia Avenue is wind. This wind force will exert itself on the façade of the building as a positive or negative pressure distribution. The façade will distribute the force from the wind pressure to the floor slabs via the connection by which the façade is attached to the structure. This creates a horizontal force at each floor level.

This force is distributed amongst the columns and shear walls on that floor by the diaphragmatic action of the concrete slab. Because the diaphragm is comprised of concrete, and consequently can be considered a rigid diaphragm, the loads will distribute to the LFRS elements based on stiffness. The shear walls are inherently stiffer than the columns when oriented parallel to the horizontal force. Because there are multiple shear walls in each direction, they will be the primary means to resisting the lateral load as opposed to the concrete columns.

Once the lateral load has been transferred from the shell, into the diaphragm, and then into the LFRS elements, these elements carry this horizontal load down through the building and into the foundation.

Design Codes and Standards

Below is given a list of all applied codes and reference standards for the structural design of the 8621 Georgia Avenue project:

- International Code Council
  - International Building Code, 2012
  - International Mechanical Code, 2012
- American Society of Civil Engineers
  - ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute
  - ACI 318-11: Building Code Requirements for Structural Concrete
  - ACI Manual of Concrete Practice – Parts 1 through 5
- ASHRAE Handbook
- Concrete Reinforcing Steel Institute
  - Manual of Standard Practice
- Post Tensioning Institute
  - Post Tensioning Manual, 6th Edition
- American Institute of Steel Construction
  - Steel Construction Manual, 14th Edition, 2010
  - AISC 360-10: Specification for Structural Steel Buildings
  - Design Guide 11- Floor Vibrations Due to Human Activity
- Structural Welding Code – Steel ANSI/AWS D1.1-10
- North American Specification for the Design of Cold-Formed Steel Structural Members (S100-07/SI-10)
- Metal Bar Grating Manual – 6th Edition (ANSI/NAAMM MBG 531-09)
- RS Means Construction Cost Data 2015
  - RS Means Mechanical Cost Data 2015

Proposal

Problem Statement

The building at 8621 Georgia Avenue consists of a two way concrete flat plate system, with a lateral system comprised of 14 shear walls. Following previous analysis in the fall semester, through a series of four technical reports, the structure was proven to be acceptable for both strength and serviceability requirements.

A hypothetical scenario is to be explored where the structure of the building is to be redesigned using a composite beam steel system. The redesign must consider the strict height restriction for the area and will undoubtedly need to eliminate a floor level. One level of the parking garage will be moved below grade to allow the same number of apartment levels as originally designed. Switching from steel to concrete should reduce the overall building weight which prompts a foundation design to see if more economical designs exist. A detailed cost assessment of the two design options will be required to determine the feasibility of each system. An additional mechanical system would also need to be designed for the floor of the building to be moved below grade.

Problem Solution

The proposed solution for the design problem is a steel framing system for the apartment levels, with the use of reinforced concrete shear walls for the lateral system. The current shear wall configuration of the building will remain the same because they are needed for the stair/elevator towers and have already been proven to function as an efficient lateral system. The parking garage levels will remain in reinforced concrete while the rest of the superstructure will be redesigned in steel. RAM will be used to analyze the gravity system while ETABS will be used to analyze the lateral system in concurrence with hand spot checks.

The decision to explore a steel system is based on several factors. The primary reason to investigate a steel system is for shear educational gain and to discover the advantages and disadvantages of using steel versus concrete structural systems. Upon a site visit to the area, other surrounding buildings of similar scale were built in both steel and concrete. Therefore, empirically, both systems seem feasible but a more quantitative approach will be used for a more definitive comparison.

A steel system would decrease the building mass and effect of seismic loads on the building. In reducing the amount of formwork and concrete pours could also speed up the schedule of the project. As discovered in Technical Report 3, a steel system would appear to be plausible only if a level was eliminated. In order to compensate for that loss, the addition of a sub-grade parking level will be explored to maintain the original square footage of rentable space.

The removal of one above level of parking garage will still be a challenge and require the total structural depth to be limited to a 18 inch depth. Composite beams will be used to reduce structure depth, as opposed to non-composite beams. The majority of the connections will be pins. Some moment frames along the building perimeter are anticipated and will require moment connections. Examples of each connection type present in the redesign will be designed using knowledge acquired in AE 534.

The bay size for the steel redesign will need to be re-examined. The current bays are square in size and will most likely be combined with adjacent bays to form rectangular bays with a 2:1 ratio, which is geometry more indicative of a steel system. If this condition is not feasible while maintaining the necessary structural depth, the existing bay arrangement be shifted to maximize the efficiency of the steel.

In order to facilitate this design solution, two breadth areas will be covered to create a more well-rounded design and conclusion for the building.

Breadth Topics

Mechanical Breadth: Parking Garage HVAC System

One of the scenarios being investigated involves placing a level of parking garage below grade. The levels of parking garage above ground have half walls which categorize those floors as being ventilated by open air and do not require ventilation. If a floor of the parking garage were to be moved below grade, that floor would not be able to be naturally ventilated by open air. To solve this problem an HVAC system for that floor will be designed.

Construction Management Breadth: Cost Comparison

Within the decision to redesign the building in steel, a level of the parking garage will be moved below grade which will influence the cost of the project. A detailed cost analysis will be performed to compare the cost of the steel and concrete structures as well as consider the excavation cost associated with the steel redesign. The cost of materials and labor will be considered in addition to potential economic benefits from more or less area of rentable apartment space.

MAE Requirements

Throughout the investigation process multiple areas of graduate level coursework will be implemented into the redesign of the building. Computer modelling is one area in which this knowledge will be implemented. RAM will be used to analyze the building's gravity system, while ETABS will be used to analyze the lateral system. These tasks will utilize skills attained in AE530, *Computer Modeling of Building Structures*. Additionally, a few of the typical steel connections in the redesign will be designed using methods learned in AE 534, *Analysis and Design of Steel Connections*.

Structural Depth

Design Decisions

The proposed structural redesign of the building is to convert the structural system from a post-tensioned concrete slab system to a composite beam and girder system. This is to investigate the feasibility of a steel system for 8621 Georgia Avenue. Before any initial designs of the gravity system could be made, specific design constraints and goals needed to be considered:

- Height Restriction – minimize structural depth
- Bay Size/Column Spacing
- Bay/Column layout that is conducive for both the parking garage and apartments.
- Minimize architectural impact
- Fireproofing of Deck/Slab

These design constraints and goals were fundamental in driving the decision making in the redesign of the structural system. In addition to the aforementioned considerations, any major architectural changes will be avoided in order to make a more definitive comparison between the original and redesign options. Any significant changes to the architecture would create a skewed cost comparison and mask the true advantages between the two options. Therefore, in order to perform the most objective investigation of the feasibility of a steel system, the architecture of the building will be preserved.

One of the most important factors to consider in the redesign process is the advantages of the two materials and how they perform most efficiently. As previously discussed in the structural overview of the existing system, the current design is tailored to a concrete system, specifically a post-tensioned arrangement. The proportion of the bay size, shear wall locations, and column locations are all indicative of a concrete system.

As opposed to concrete, steel performs most efficiently in rectangular bays with a bay length to width ratio of $1 \leq l/w \leq 2$. Steel systems are most constructible when the columns are on grid and the bay sizes are relatively regular throughout the building plan. Neither of these rules of thumb are present and need to be addressed.

Due to the size of the parking spaces and required throughway width for the parking garage, the bays alternate between longer and shorter bays. In order to allow the bay width to be modularized, the parking spaces have been converted from 90° to 30° angled spaces. The minimum required throughway width for this parking arrangement is 7' smaller per code requirements and allows all of the bays in the X direction to be the same dimension.

In the Y direction, column and shear wall locations were shifted in order to standardize the bay lengths in that direction. This change in the column locations and grids was able to fit into the existing building form, plus or minus a couple of feet.

The code required dimensions of the parking spaces are listed below and given for 90°, 60°, 30°, and 15° orientations. The newly oriented spaces in the four levels of parking garage meet the requirements for the 60° and 30° spaces.

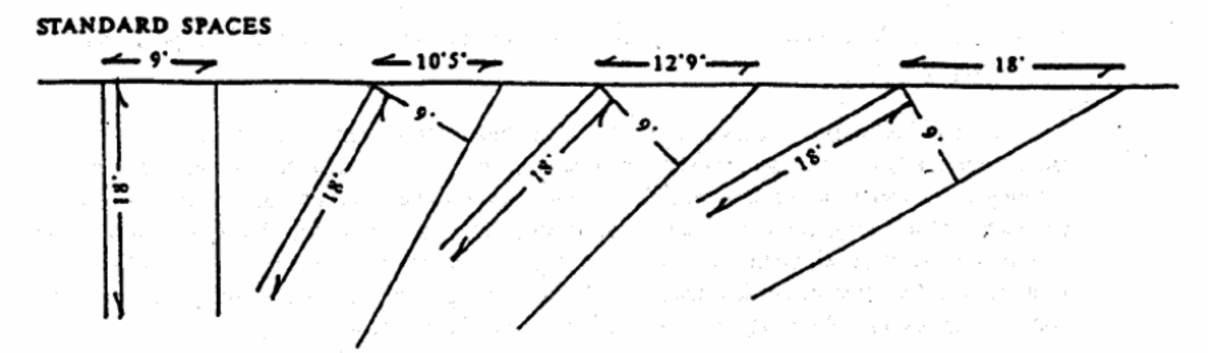


Figure 22: Parking Space Size Requirements

Below is the new parking garage layout implementing the diagonal parking scheme. Throughways around the exterior are 18' while the center ramp is 22 feet wide.

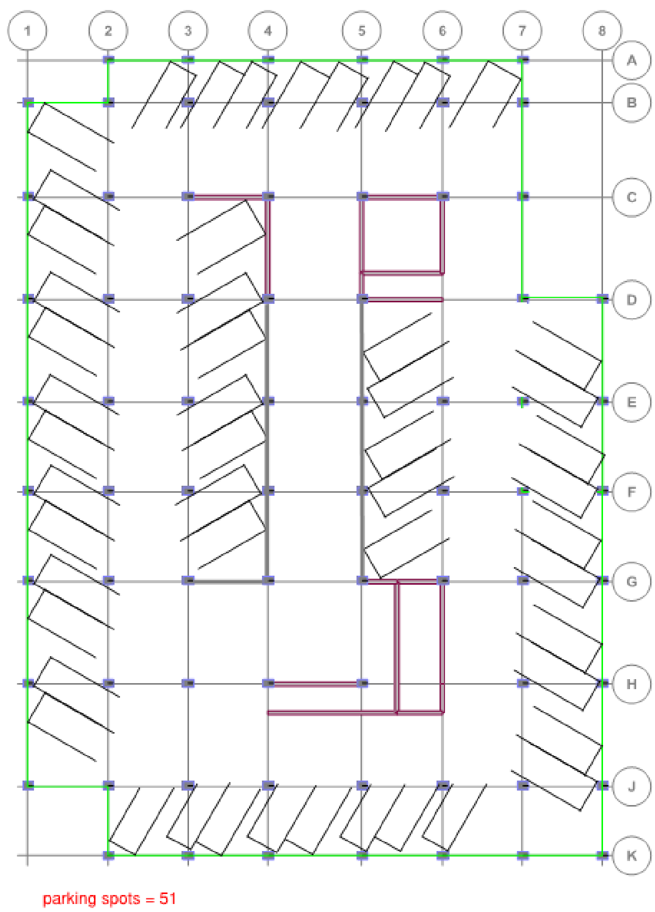


Figure 23: Parking Space Layout

The columns were also moved onto the newly created grid to form a typical bay of 18x24. This creates a bay ratio of 1.333, which is in the acceptable range for a steel system to take advantage of its' structural strengths and characteristics. A regular bay size that is repeated throughout the building will also increase the constructability of the structure.

Currently the existing height of the building is at the height restriction in place by Montgomery County, MD of 162'-4". One characteristic of a steel system is that it typically has a greater structural depth compared to concrete. In order to accommodate this additional building height, one level of the parking garage will be moved below grade. In addition to that, the implemented steel system will be of composite design which will minimize beam/girder depth due to the added strength of the concrete on metal deck.

After being design for strength and serviceability, the beams and girders will also be design for vibrations. In a building with multiple shared occupants vibrations from others can be felt through the floor if vibrations are not considered.

The utilization of the steel members will also be considered. Creating an efficient design for the structure is very important and will have an impact on the cost analysis for the steel system. As a guideline, a utilization of over 80% will be considered satisfactory for the amount of load that is applied compared to its' capacity.

Composite Steel Beam Girder System

The bay sizes and column placements needed to be changed in order to allow for a design that takes advantage of the ideal steel bay proportions. Bay sizes were modularized and columns were relocated onto the column grid. The result was bays that are 18'-0" x 24'-0". The typical bay has one infill beam that spans the long direction.

The beam and girder sizes for the typical bay are a W12x22 and a W14x34. With these selected sizes the span to depth ratios of these members are 24 and 16 respectively. These values are within the industry recommended values of 25-30 for beams and 15-20 for girders.

The steel system will incorporate shear studs welded to the top of the beams in order to engage the concrete above and increase capacity of the beams. This decision enabled the use of beams that are approximately 2 inches less than if a non-composite system was used. The fourth floor has an additional structural depth allowance in order to support the Bio-retention area of the green roof. In order to meet the height restriction, the girders needed to be limited to W14's on the typical level and W18's under the Bio-retention area.

The modular and repetitive nature of the bay layout will help the project be more economical. The economy is found is the repetition of beam shapes and connections and only needing a few different sizes on site. The same members and arrangement from bay to bay and floor to floor leads to more efficient fabrication and installation.

The final designs for these floors are given in the following pages. All material strengths and properties of the designed members are given in the material information provided in the structural description on page 8. All hand calculations and confirmations of these designs can be found in Appendix C.

Plan of Floor 5: Bio-Retention Area

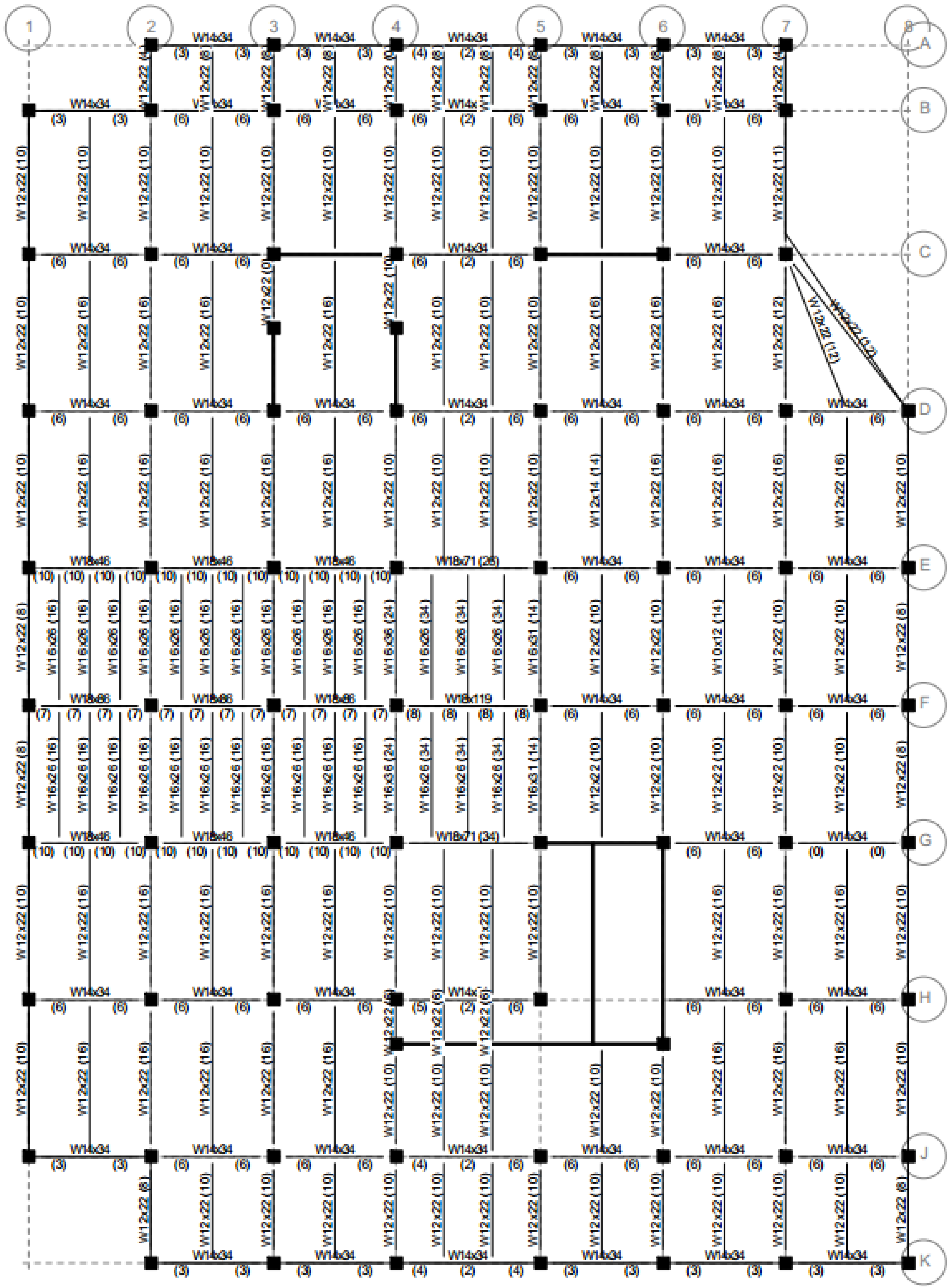


Figure 24: 5th Floor Plan

Plan of Floors 5-15: Typical Apartment

Figure 25: Typical Apartment Framing

Plan of Floor 16: Penthouse

Figure 26: 16th Floor Framing

Steel Utilization

Initially the typical bay was designed to have infill beams at the third points, resulting in a beam spacing of 6 feet. This additional infill beam was originally included in an attempt to reduce the moment on the girders such that a W14 could become a reality. As will be discussed later in the report, the sizes of the beams and girders were governed by the serviceability requirement of vibration control. With the beams sized up to reduce vibrations, their utilization and interaction values were merely around 0.40. This was not an efficient use of the steel.

The design was then changed to its final form in having a single infill beam at mid-span, leaving a 9' spacing of the beams. By increasing the gage of the deck to a 1.5VLR18, this allowed the two span unshored clear span length to reach over 9 feet. Using a stronger deck allowed an additional infill beam to be eliminated from each bay. This also increased the interaction of the beams and girder so that the steel is used much more efficiently.

The interaction of the beams and girders for the redesigned steel floors is given in the following pages.

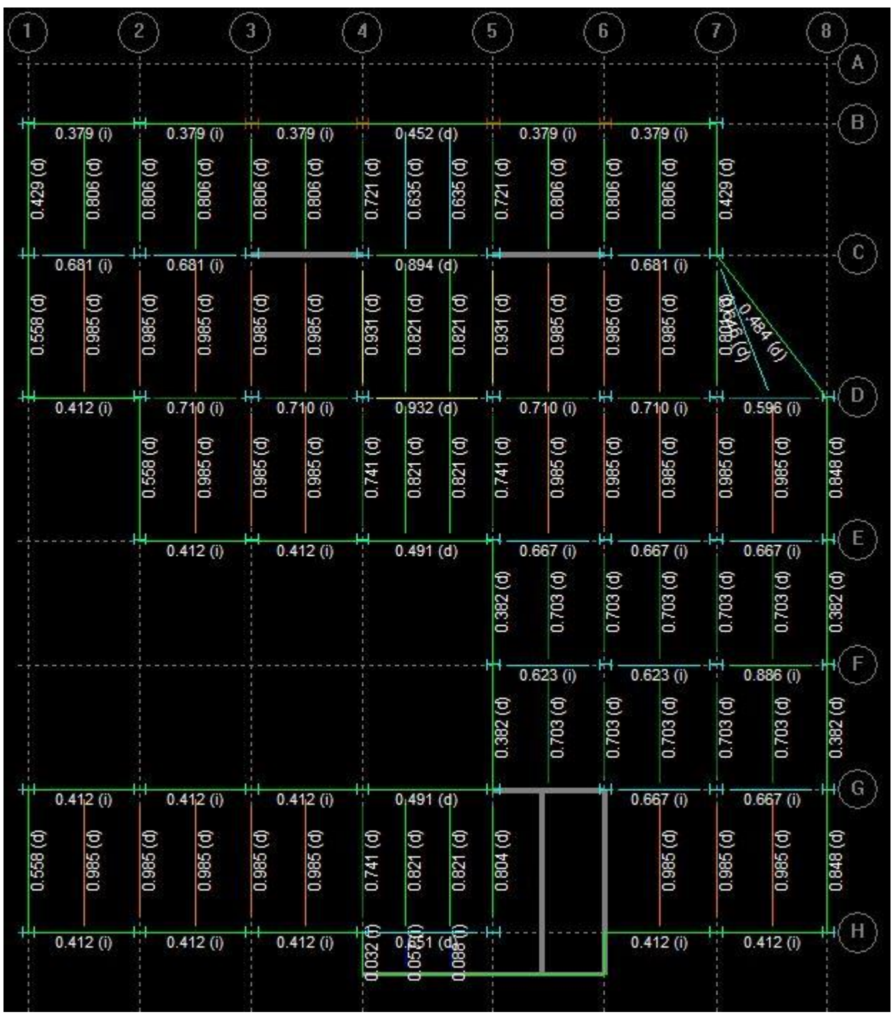


Figure 27: 16th Floor Steel Utilization

Typical Apartment Floor Interactions:

Figure 28: Typical Apartment Floor Utilization

Fourth Floor Steel Interaction:

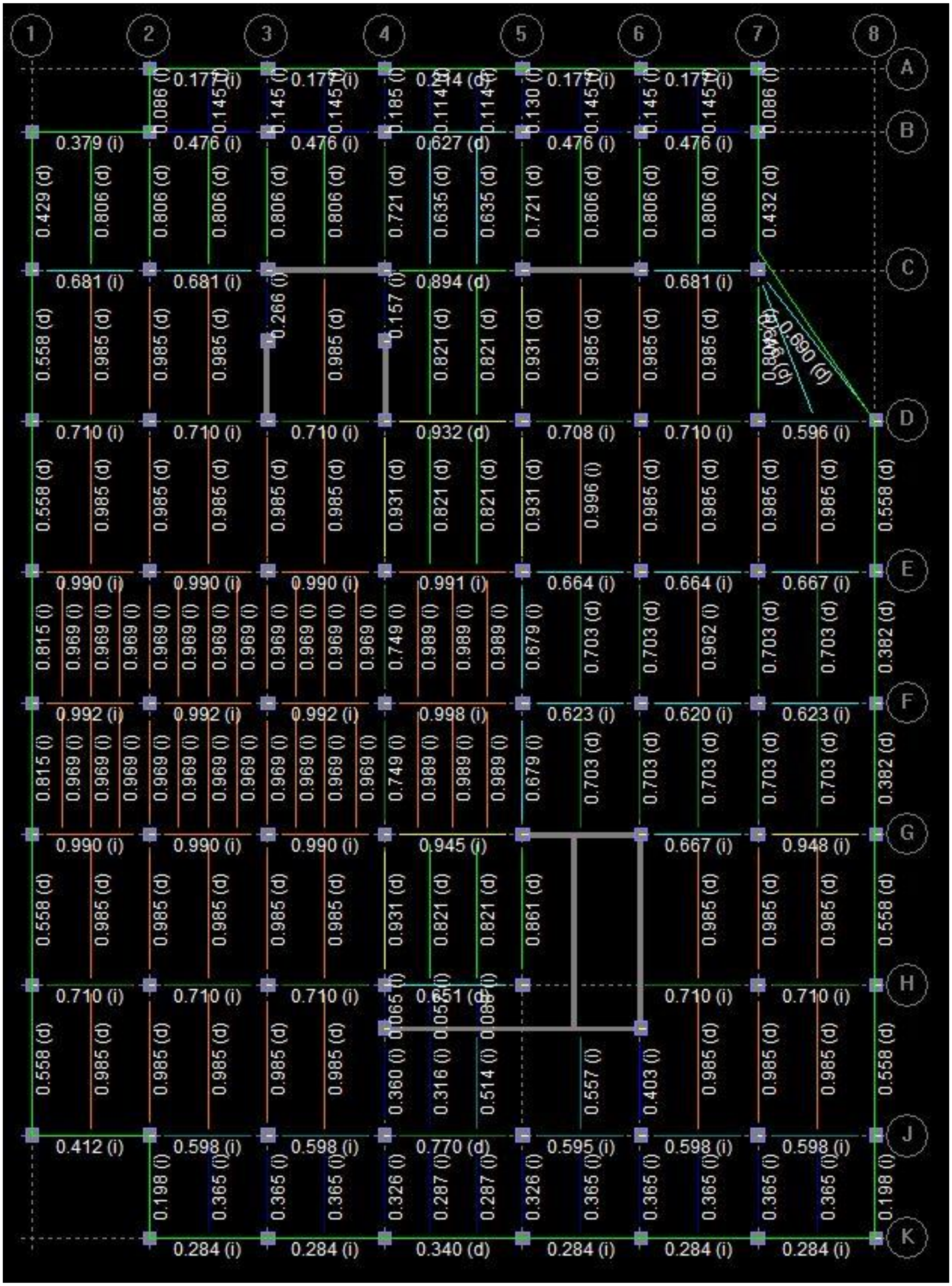


Figure 29: 5th Floor Steel Utilization

Beams Orientation:

Generally speaking, most steel systems function with the beams spanning the long direction of the bay and the girders spanning the short direction. Although this is the preferred method and normally the more efficient, all alternatives need to be considered for optimization of the beam arrangement.

The previous design presented had the beams spanning the 24’ direction of the 18’ x 24’ bays. The alternative layout that will be considered will have the beams span the 18’ direction and the girders span the 24’ direction. Because the girders are now spanning 24’ and the selected deck only has a maximum unshored clear span limit at 10’-5” for a 2 span condition, an additional infill beam needs to be added. Shoring the beams is an option but would be far more costly and time consuming in the projects schedule.

The addition of another member for every bay already makes the option with the beams in the short direction seem less ideal. Nonetheless, this arrangement was modelled and the design compared. The two designs will be compared based on a cost standpoint of how many members are required, the number of shear studs, and the tonnage of steel.

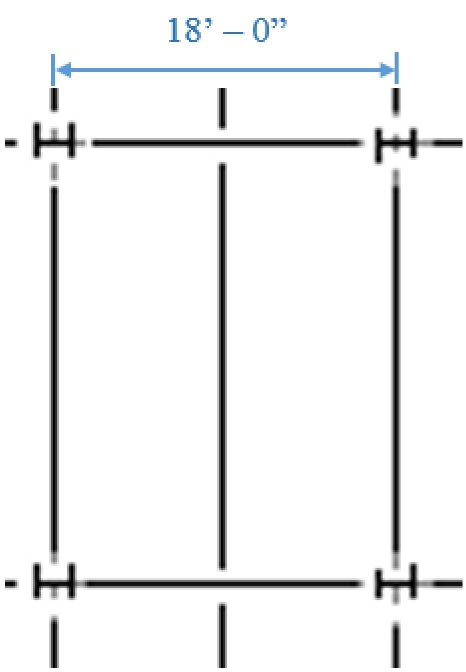


Figure 30: Beams Oriented in the long direction

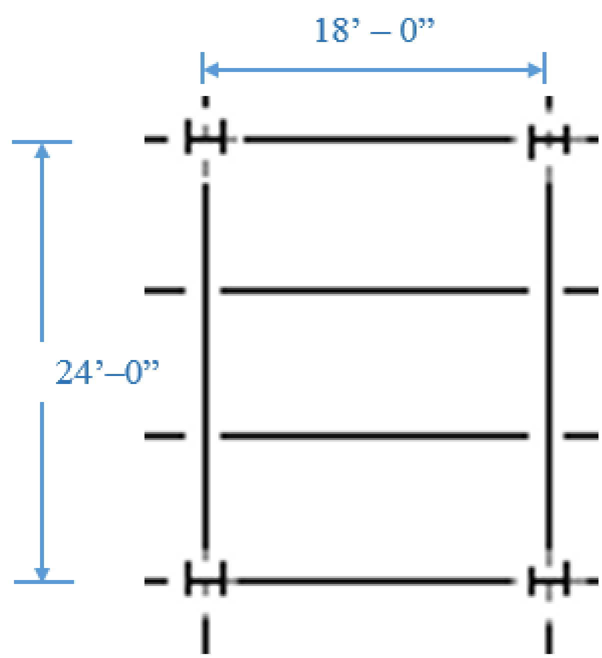


Figure 31: Beams Oriented in the short direction

Beam Orientation			
	Steel Weight (tons)	Number of members	Number of studs
Long Direction	590.9	2,220	25,093
Short Direction	627.3	2,577	28,387

Based on the results above, the arrangement with the beams in the long direction is more cost effective. This orientation uses less steel and requires fewer number of members and studs. These differences will yield a cost advantage in terms of material cost as well as assembly labor. The above comparison does not even consider the added schedule time of framing additional members. Therefore, with those factors also considered, the final design presented already is the most efficient way to orient the beams.

Another consideration is the number of connections and the time and cost associated with fabricating them onsite. Connections account for approximately 10% of the overall steel construction cost and can be easily reduced by laying out the beams efficiently.

Full floor plans of the two beam orientations can be found in Appendix B.

Column Orientation

Based on the lateral system results attained in Technical Report 4, the displacements in the X direction were known to be controlling over the displacements in the Y direction. Therefore, the columns in the steel redesign were oriented with their strong axis in the X direction. This increased the building stiffness in that direction. This decision also lent itself to adding moment frames as well, which work at resisting lateral forces in the X direction.

Column Design

The columns in 8621 Georgia Avenue transition from concrete to steel at the fourth level where the occupancy of the space changes from parking garage to apartments. The columns are designed to be spliced at every two floors and are connected to the gravity system via shear connections.

When designing the columns, W shapes of size 14, 12, and 10 were considered for possible column shapes. The column designs for each of these sizes was compared based on total steel weight. All of the designed columns in the building are either W10's or W12's. Splicing the columns every two floors allowed the sizes to change throughout the building height which avoided using the larger W14 columns. By changing the column sizes over these height intervals through the building, the selected column sizes were able to be designed in order to maximize the utilization of the steel.

The color scale is show below and reveals that the majority of the columns have an interaction of over 0.70. The blue columns designate columns where the interaction is below 0.4 and there is left over capacity. All of these columns are located in the upper floors or around the perimeter where the loads are less.

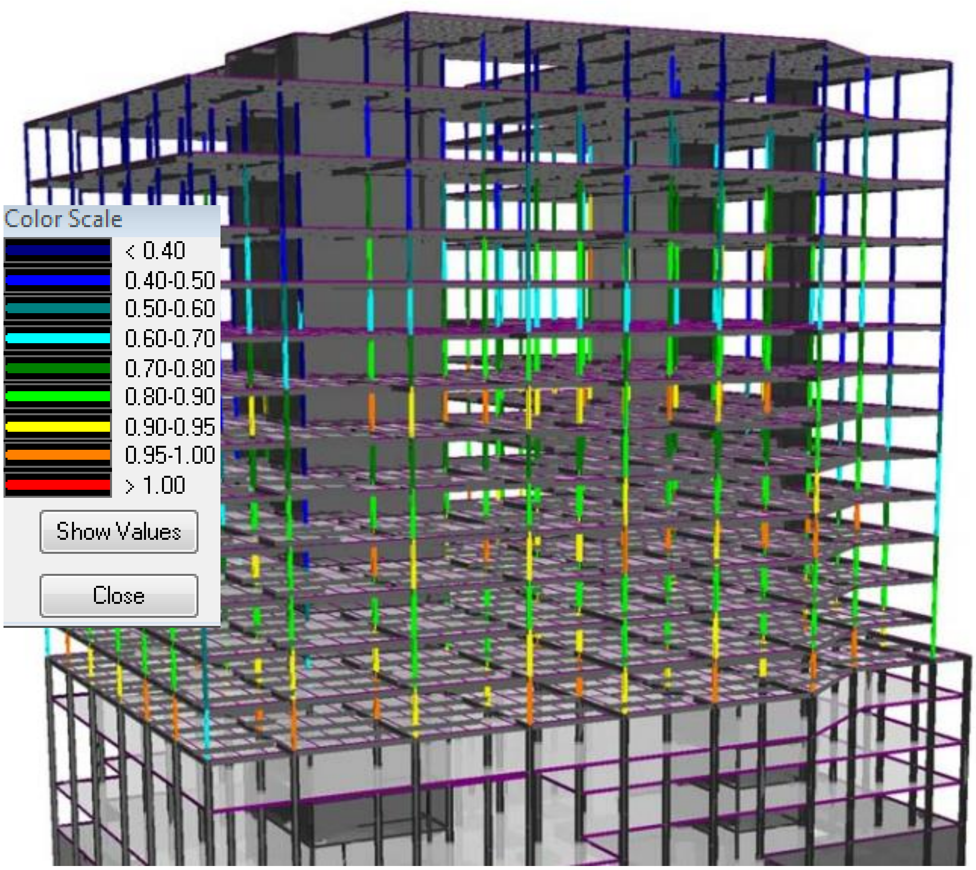


Figure 32: 3D View of Column Interactions

Floor Vibrations Analysis

With the member sizes for the gravity system being limited in size by the maximum desired structural depth, the vibration accelerations of the floor system were of concern. If the member sizes are too shallow or not heavy enough, the inertia of the members and floor system will not be sufficient to resist vibration accelerations.

Although vibrations is only a serviceability condition, annoying vibrations can impact the occupants and their quality of life. The response due to walking and dynamic activity can vary based on the magnitude, frequency, and location of the loads. Effects due to vibrations from elsewhere in the building can be very disruptive and take away from the privacy of an apartment.

A detailed vibration analysis was performed using AISC Design Guide 11- Floor Vibrations Due to Human Activity. This was done to analyze the designs based on strength and deflections that were attained by hand and through RAM Structural Systems. The designs were refined and sizes were adjusted in order to abide by Design Guide 11.

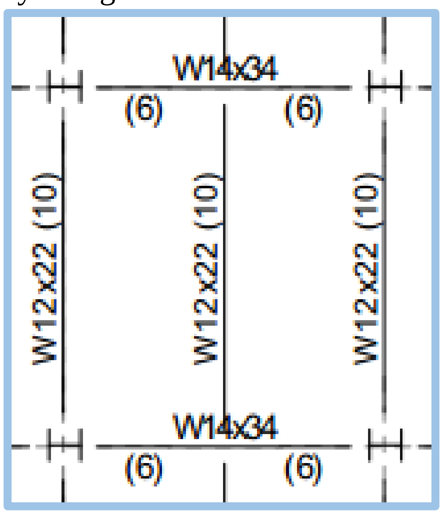


Figure 33: Typical Bay Design

The mode properties and frequencies of the beams and girders were calculated. The equivalent panel weights of the floor system were determined using the given loading calculated in previous sections of this report. These values were found such that the acceleration of the floor system can be determined. The acceleration of the floor system is measured as a ratio to the acceleration of gravity. Other variables were used based on human walking induced vibrations. The acceptable vibration acceleration given in Design Guide 11 is 0.5%.

$$\frac{a_o}{g} \geq \frac{a_p}{g}$$

$$0.5\% \geq 0.48\%$$

Foundation Redesign

As a direct result from the steel redesign of the apartment levels, the building mass will decrease. Therefore, an investigation of the existing foundation system was performed to determine if the size of the footings and mat foundations could be reduced. Alternatives of replacing the mat foundations with spread footings will also be investigated.

The potential associated cost savings from reducing the foundations could prove to be an important advantage in favor of the steel redesign. A disadvantage of this design decision will come with a large cost in the additional excavation required to go 12 feet lower in the soil. Both factors will be considered and accounted for in the final cost comparison.

The existing foundation plan is shown below. The outlines of the footings are highlighted on the plan. The elements of the foundation at the Cellar level are shown on a partial plan on the next page.

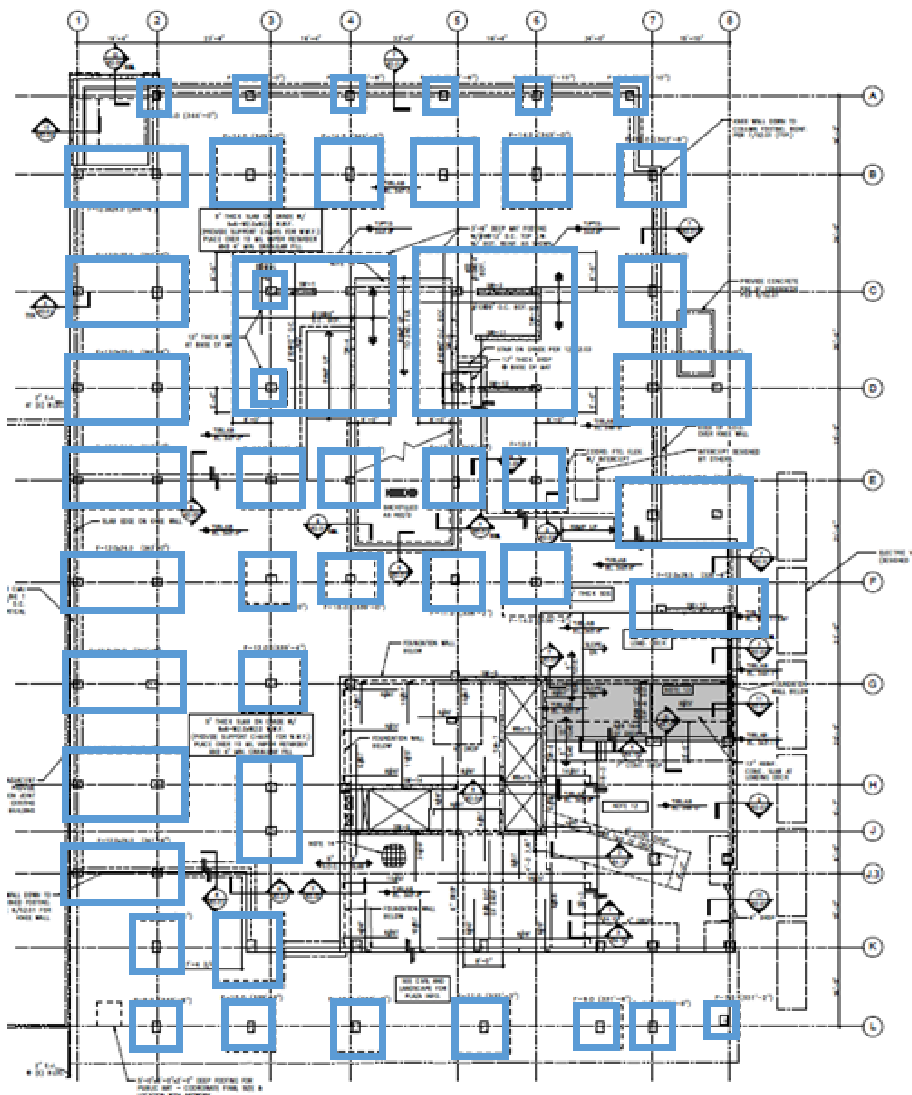


Figure 34: Existing Foundation Design

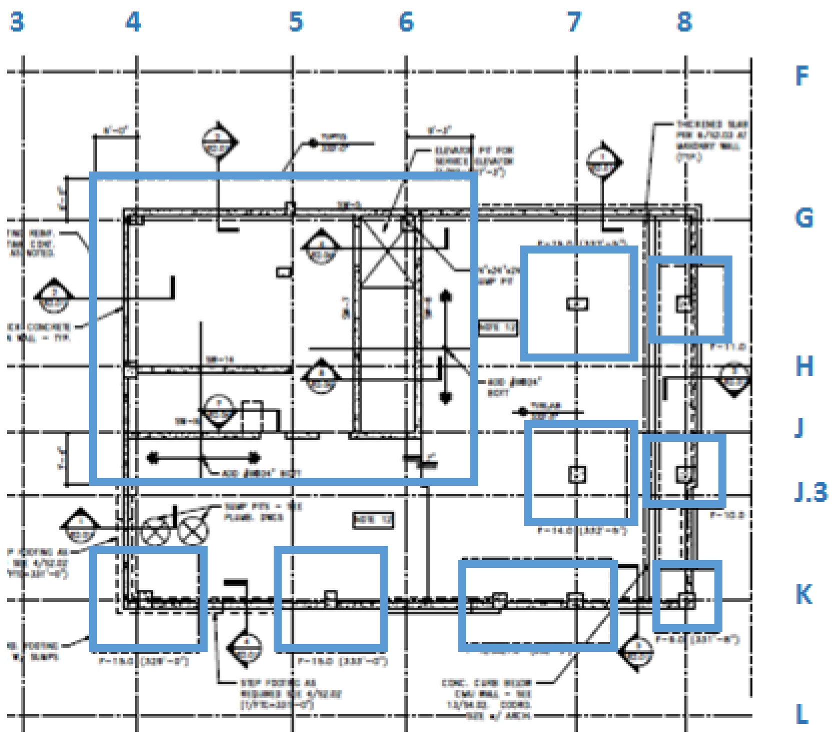


Figure 35: Existing Foundation Design at Cellar Level

With the reduced load on the foundations due to the smaller building weight, many of the footing sizes can be reduced. In the original design, there are multiple conditions where adjacent columns share a footing. This is done because the two individual footings for the columns above overlap or nearly touch at these conditions. When the footing sizes are reduced these dual footings can be broken into individual footings.

The mat foundations were investigated as well to determine whether they can be reduced into multiple elements. After designing the geometry of the spread and wall footings that would replace the mat foundations, the result was an array of differently sized square and rectangular sections will varying depths. Although more materials will be used, the existing mat foundations were kept due to constructability and the associated ease of forming and pouring just one foundation element.
The typical continuous wall footings throughout the building footprint are 5 feet wide and 18 inches deep. The final design for the foundation plan is shown on the next page. Hand calculations, verifications, and spreadsheets associated with the foundation design can be found in Appendix E.

The geometry of the designed footings is shown in the plans below (in feet), by showing the planar dimensions in Figure 36 and the depths in Figure 37.

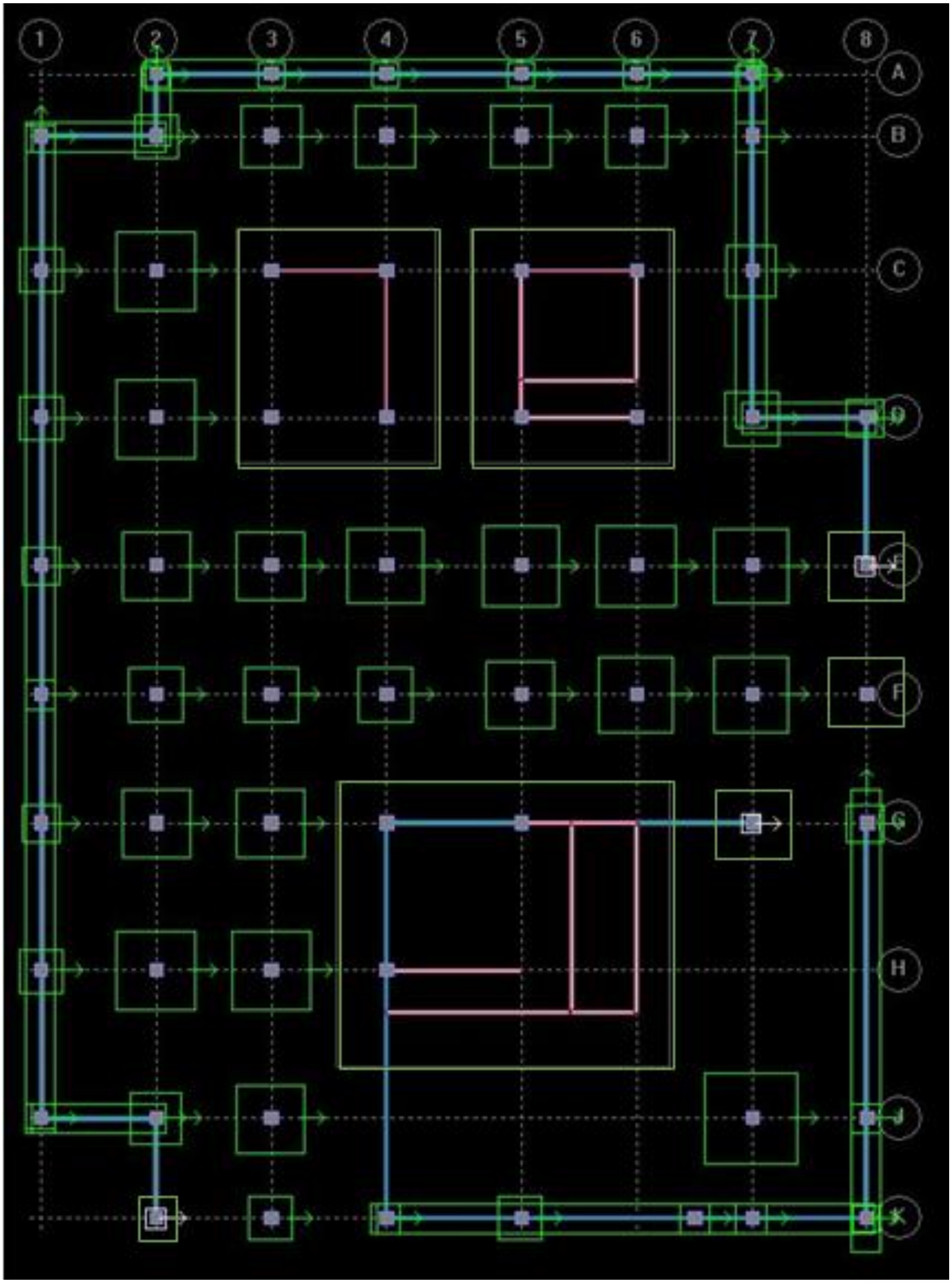


Figure 36: Redesigned Foundations in RAM

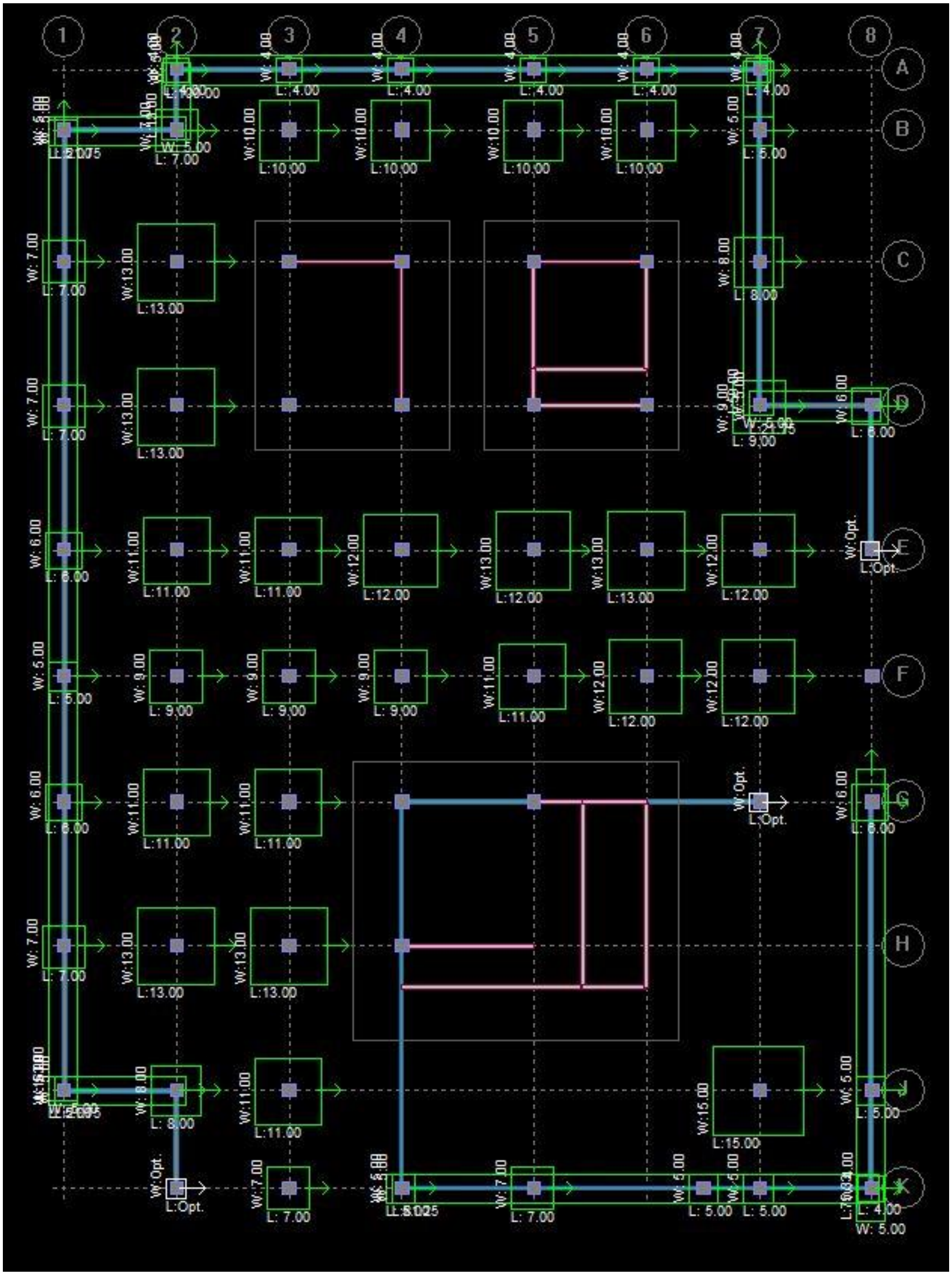


Figure 37: Redesign Foundation Geometry

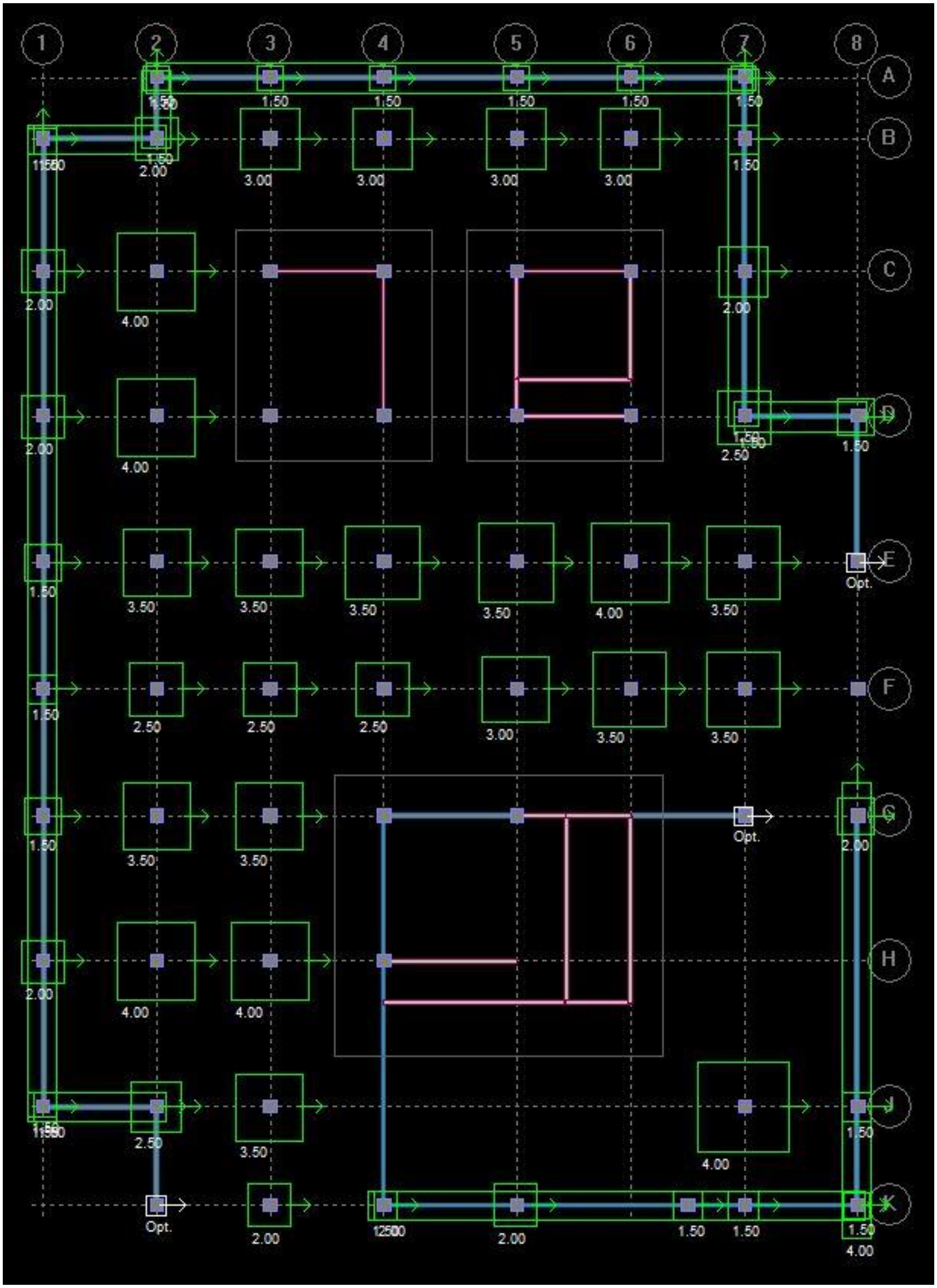


Figure 38: Redesigned Foundation Depth

As predicted, the sizes of the foundations dramatically decreased. The material savings from in concrete and rebar is tabulated below, comparing the existing foundations to the redesigned system supporting the steel superstructure.

Foundation System Comparison			
	Concrete (CY)	Formwork (SFCA)	Steel (tons)
Existing Foundations	1762.4	10599.5	69.48
Re-Designed Foundations	1105.0	7094.7	56.14

With the addition of another below-grade floor, the electric cellar level is now 12 feet deeper than in the original design. At a lower depth the horizontal soil pressure on the foundation walls increases linearly per foot based on the equivalent fluid pressure of the soil. The existing foundation walls needed to be analyzed for the new, greater horizontal forces.

The thickness of the original wall passed under the new loading. The reinforcing of the foundation walls was adjusted to improve constructability. Originally, the walls employed three different sizes of rebar between the inside face, outside face, and stirrups in the wall. The walls reinforcing was redesign to only use #5 bars with the same spacing on each face. This does not yield a large cost advantage but increases constructability of the wall in the field.

A diagram of the foundation wall design is shown below. Additional calculations can be found in Appendix E.

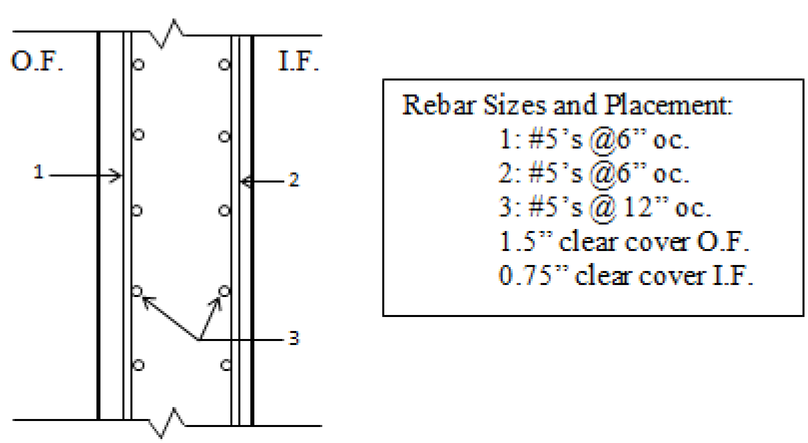


Figure 39: Foundation Wall Design

Overturning and Foundation Impact

The overturning and foundation impacts due to wind and seismic loading were considered. The table below shows the base shear and overturning moment applied due to each load case. The controlling overturning moments, in both direction, were caused by case 1 of the wind load cases. The applied moments were compared to the resisting moment due to the building weight. The safety factor between the resisting and applied moments was calculated. Code dictates that the safety factor is greater than 1.5 but standard industry practice uses a factor between 2 and 3. The factors resulting from this analysis are both in excess of 56. Therefore, the building is more than adequate to handle the overturning moment. This result is not surprising because the building dimensions and base shear did not change much from the original design because wind controls over the seismic lateral case. .

Overturning Moments

Load Cases	Base Shear X Direction (k)	Base Shear Y Direction (k)	Overturning X Direction (' k)	Overturning Y Direction (' k)
Wind Case 1 – X Direction	779.74	-	**52,242.58**	-
Wind Case 1 – Y Direction	-	553.52	-	**53,137.92**
Wind Case 2 – X Direction (+M)	584.81	-	39,182.27	-
Wind Case 2 – X Direction (-M)	584.81	-	39,182.27	-
Wind Case 2 – Y Direction (+M)	-	415.14	-	
Wind Case 2 – Y Direction (-M)	-	415.14	-	39,853.44
Wind Case 3	584.81	415.14	39,182.27	39,853.44
Wind Case 4 (Additive +Moments)	438.99	311.63	29,412.33	39,853.44
Wind Case 4 (Additive –Moments)	438.99	311.63	29,412.33	29,916.48
Wind Case 4 (+M's in Opposite Directions)	438.99	311.63	29,412.33	29,916.48
Wind Case 4 (-M's in Opposite Directions)	438.99	311.63	29,412.33	29,916.48
Seismic X	441.42	-	29,575.14	-
Seismic Y	-	441.42	-	42,376.32

Resisting Moment:

X Direction:

$M_{resisting} = 44{,}142.34^k \text{ x } 67 \text{ ft.} = 2{,}957{,}536.78 \text{ ' k}$

$$\frac{2{,}957{,}536.78}{52{,}242.58} = 56.6 > 1.5$$

Y Direction:

$M_{resisting} = 44{,}142.34^k \text{ x } 96 \text{ ft.} = 4{,}237{,}664.64 \text{ ' k}$

$$\frac{4{,}237{,}664.64}{53{,}137.92} = 79.7 > 1.5$$

Lateral Analysis

The lateral analysis of 8621 Georgia Avenue will evaluate the effectiveness of the building to resist lateral forces due to wind and seismic activity. This will be done utilizing computer 3D modeling and hand calculations which can be found in Appendix G.

With the structural system changing from concrete to steel the weight of the building decreased. This will directly affect the seismic forces on the building. Although, these forces are proportional to the building weight and will decrease as well, the lateral system needs to be checked for the new structural system.

The current lateral system consists of concrete shear walls that are centered around the stair towers. These are typically convenient locations for shear walls but also are a good arrangement for the original post-tensioned slabs to avoid shortening and residual stresses in the slab.

The first design change to the lateral system was to eliminate two shear walls that were located outside of the stair towers. These two shear walls became a conflict with some of the architecture on the apartment levels. These shear walls were also located close to the center of rigidity and did not carry a large impact on reducing lateral displacements.

Upon removing the two shear walls mentioned above, the buildings lateral displacements for wind case 2 became too large. In order to provide the necessary lateral resistance without creating any architectural conflicts, 3 bays of moment frames were provided at the north and south end of the building. These moment frames are positioned at the ends of the building to reduce the torsional effects on the building as well.

As was the case in Technical Report 4, a 3D ETABS was created in order to analyze 8621 Georgia Avenue's lateral system. The lateral system was analyzed under wind and seismic loads calculated using ASCE7-10. Two spot checks of the lateral system were performed to verify the shear wall and moment frame designs, which can be found in Appendix G.

Overview

The lateral system consists of 13 shear walls and 3 bays of moment frames on the north and south side of the building. Some of the shear walls are only in the first three floors of parking garage while the shear walls around the stair towers and the moment frames rise the full height of the building. The typical shear wall is 12" thick with the exception of shear walls 1 and 2 which are 14" thick. The following image shows an overall view of the lateral system with the moment frames on each end of the building.

Although the building is exempt from any of the building irregularity provisions described in ASCE due to the Seismic Design Category A status, torsional behavior was considered in the lateral system layout. The steel moment frames were placed at the end of the building to limit displacements in the weak direction and minimize any torsional effects on the building.

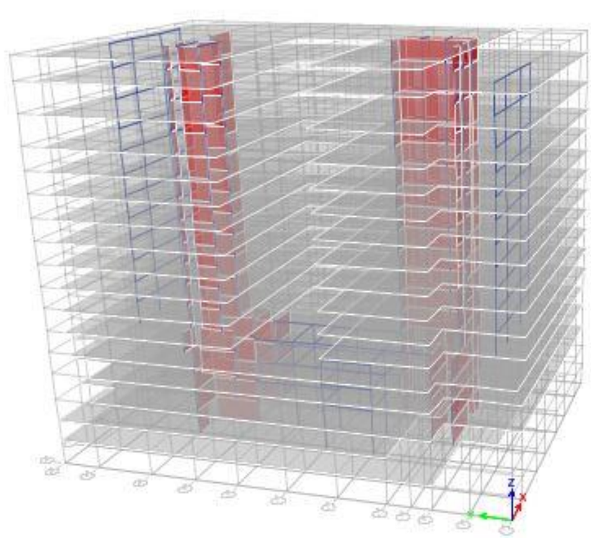


Figure 40: Overall View of Lateral System

Wind Loads

The wind loads on the building were calculated using ASCE 7-10. As per the ASCE procedures, four different wind cases were applied. Wind cases also consider positive and negative moments under the same loading. The four wind cases take quartering winds and torsional effects into consideration. The following tables show the applied wind pressures and resulting forces on each story of the building. Calculated building properties such as the center of pressure, center of rigidity, and center of mass can be found in Appendix G.

Wind Forces

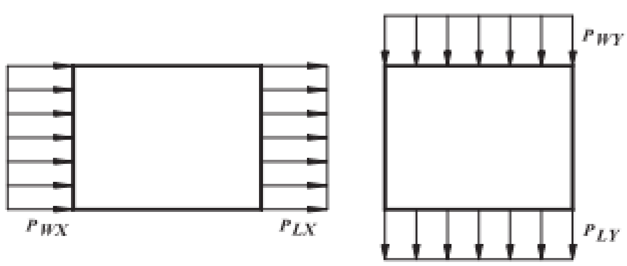
The wind analysis of the building was conducted in accordance with the Main Wind Force Resisting System directional procedure for determining wind loads. This procedure outlines 4 wind load cases to be considered. The various cases consider wind from each of the 4 major faces of the building and incorporate torsional moment of the building due to the wind.

Case 1:

The first case of the wind analysis is simply applying the full load orthogonal to the building in each of the two primary axis. The east/west direction is the long direction of the building, which has a greater surface area for the wind pressure to act over. The base shear values in each direction are also given.

Case 1 N/S Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	Story Force (k)
1	10.167	134.33	13.10	-4.39	1365.73	23.89
2	9.333	134.33	13.80	-4.64	1253.70	23.12
3	9.333	134.33	15.00	-5.06	1253.70	25.15
4	11	134.33	16.10	-5.40	1477.63	31.77
5	9.333	134.33	16.80	-5.63	1253.70	28.13
6	9.333	134.33	17.40	-5.87	1253.70	29.17
7	9.333	134.33	18.00	-6.05	1253.70	30.15
8	9.333	134.33	18.40	-6.21	1253.70	30.85
9	9.333	134.33	18.90	-6.37	1253.70	31.68
10	9.333	134.33	19.30	-6.50	1253.70	32.34
11	9.333	134.33	19.70	-6.62	1253.70	33.00
12	9.333	134.33	20.10	-6.75	1253.70	33.66
13	9.333	134.33	20.40	-6.86	1253.70	34.17
14	9.333	134.33	20.70	-6.97	1253.70	34.68
15	12.333	134.33	21.10	-7.09	1656.69	46.71
16	12.667	134.33	21.50	-7.22	1701.56	48.87
17	9.333	134.33	21.60	-7.25	1253.70	36.17
					Base Shear =	553.52

Case 1 E/W Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	Story Force (k)
1	10.167	175.5	13.1	-4.392	1784.31	31.21
2	9.333	192	13.8	-4.644	1791.94	33.05
3	9.333	192	15	-5.058	1791.94	35.94
4	11	192	16.1	-5.4	2112.00	45.41
5	9.333	192	16.8	-5.634	1791.94	40.20
6	9.333	192	17.4	-5.868	1791.94	41.69
7	9.333	192	18	-6.048	1791.94	43.09
8	9.333	192	18.4	-6.21	1791.94	44.10
9	9.333	192	18.9	-6.372	1791.94	45.29
10	9.333	192	19.3	-6.498	1791.94	46.23
11	9.333	192	19.7	-6.624	1791.94	47.17
12	9.333	192	20.1	-6.75	1791.94	48.11
13	9.333	192	20.4	-6.858	1791.94	48.84
14	9.333	192	20.7	-6.966	1791.94	49.58
15	12.333	192	21.1	-7.092	2367.94	66.76
16	12.667	192	21.5	-7.218	2432.06	69.84
17	9.333	160.5	21.6	-7.254	1497.95	43.22
					Base Shear =	779.74



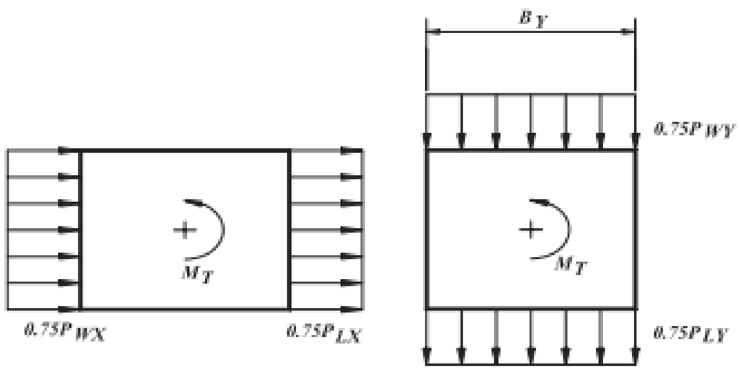
CASE 1

Case 2:

The second case addresses the effects of potential quartering wind conditions and their effects. Three quarters of the design wind pressures are considered in addition to a torsional moment about a vertical axis of the building with an eccentricity equal to 15% of the windward face.

Case 2 N/S Wind Forces									
Floor Number	**Floor to Floor Height (ft.)**	**Wall Length (ft.)**	**Windward Pressure (psf)**	**Leeward Pressure (psf)**	**Tributary Area (sqft.)**	**0.75 * Story Force (k)**	**B (ft.)**	**e (ft.)**	**M (ft.*k)**
1	10.167	134.33	13.10	-4.39	1365.73	17.92	134.33	20.15	361.02
2	9.333	134.33	13.80	-4.64	1253.70	17.34	134.33	20.15	349.44
3	9.333	134.33	15.00	-5.06	1253.70	18.86	134.33	20.15	380.02
4	11	134.33	16.10	-5.40	1477.63	23.83	134.33	20.15	480.10
5	9.333	134.33	16.80	-5.63	1253.70	21.09	134.33	20.15	425.04
6	9.333	134.33	17.40	-5.87	1253.70	21.88	134.33	20.15	440.84
7	9.333	134.33	18.00	-6.05	1253.70	22.61	134.33	20.15	455.62
8	9.333	134.33	18.40	-6.21	1253.70	23.14	134.33	20.15	466.26
9	9.333	134.33	18.90	-6.37	1253.70	23.76	134.33	20.15	478.81
10	9.333	134.33	19.30	-6.50	1253.70	24.26	134.33	20.15	488.77
11	9.333	134.33	19.70	-6.62	1253.70	24.75	134.33	20.15	498.74
12	9.333	134.33	20.10	-6.75	1253.70	25.25	134.33	20.15	508.70
13	9.333	134.33	20.40	-6.86	1253.70	25.63	134.33	20.15	516.43
14	9.333	134.33	20.70	-6.97	1253.70	26.01	134.33	20.15	524.16
15	12.333	134.33	21.10	-7.09	1656.69	35.03	134.33	20.15	705.82
16	12.667	134.33	21.50	-7.22	1701.56	36.65	134.33	20.15	738.46
17	9.333	134.33	21.60	-7.25	1253.70	27.13	134.33	20.15	546.67
					Base Shear=	415.14			

Case 2 E/W Wind Forces									
Floor Number	**Floor to Floor Height (ft.)**	**Wall Length (ft.)**	**Windward Pressure (psf)**	**Leeward Pressure (psf)**	**Tributary Area (sqft.)**	**0.75 * Story Force (k)**	**B (ft.)**	**e (ft.)**	**M (ft.*k)**
1	10.167	134.33	13.10	-4.39	1365.73	23.41	175.50	26.33	616.22
2	9.333	134.33	13.80	-4.64	1253.70	24.79	192.00	28.80	713.89
3	9.333	134.33	15.00	-5.06	1253.70	26.96	192.00	28.80	776.36
4	11	134.33	16.10	-5.40	1477.63	34.06	192.00	28.80	980.81
5	9.333	134.33	16.80	-5.63	1253.70	30.15	192.00	28.80	868.33
6	9.333	134.33	17.40	-5.87	1253.70	31.27	192.00	28.80	900.61
7	9.333	134.33	18.00	-6.05	1253.70	32.32	192.00	28.80	930.80
8	9.333	134.33	18.40	-6.21	1253.70	33.07	192.00	28.80	952.55
9	9.333	134.33	18.90	-6.37	1253.70	33.96	192.00	28.80	978.17
10	9.333	134.33	19.30	-6.50	1253.70	34.67	192.00	28.80	998.53
11	9.333	134.33	19.70	-6.62	1253.70	35.38	192.00	28.80	1018.89
12	9.333	134.33	20.10	-6.75	1253.70	36.09	192.00	28.80	1039.25
13	9.333	134.33	20.40	-6.86	1253.70	36.63	192.00	28.80	1055.04
14	9.333	134.33	20.70	-6.97	1253.70	37.18	192.00	28.80	1070.84
15	12.333	134.33	21.10	-7.09	1656.69	50.07	192.00	28.80	1441.95
16	12.667	134.33	21.50	-7.22	1701.56	52.38	192.00	28.80	1508.63
17	9.333	134.33	21.60	-7.25	1253.70	32.42	160.50	24.08	780.42
					Base Shear=	584.81			

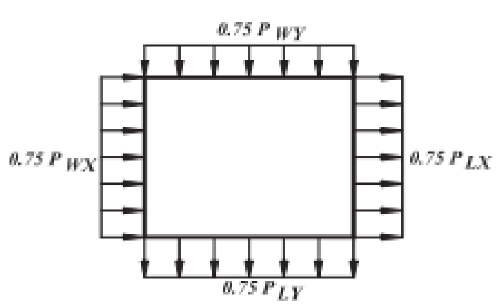


Case 3:

This case is the same described in case 1 but with three quarters of the design wind pressure being applied simultaneously to each side. The forces given in the following tables would be applied concurrently to the building as oppose to individually like in the first two cases.

Case 3 N/S Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.75 * Story Force (k)
1	10.167	134.33	13.10	-4.39	1365.73	17.92
2	9.333	134.33	13.80	-4.64	1253.70	17.34
3	9.333	134.33	15.00	-5.06	1253.70	18.86
4	11	134.33	16.10	-5.40	1477.63	23.83
5	9.333	134.33	16.80	-5.63	1253.70	21.09
6	9.333	134.33	17.40	-5.87	1253.70	21.88
7	9.333	134.33	18.00	-6.05	1253.70	22.61
8	9.333	134.33	18.40	-6.21	1253.70	23.14
9	9.333	134.33	18.90	-6.37	1253.70	23.76
10	9.333	134.33	19.30	-6.50	1253.70	24.26
11	9.333	134.33	19.70	-6.62	1253.70	24.75
12	9.333	134.33	20.10	-6.75	1253.70	25.25
13	9.333	134.33	20.40	-6.86	1253.70	25.63
14	9.333	134.33	20.70	-6.97	1253.70	26.01
15	12.333	134.33	21.10	-7.09	1656.69	35.03
16	12.667	134.33	21.50	-7.22	1701.56	36.65
17	9.333	134.33	21.60	-7.25	1253.70	27.13
					Base Shear =	415.14

Case 3 E/W Wind Forces						
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.75 * Story Force (k)
1	10.167	175.5	13.1	-4.392	1784.31	23.41
2	9.333	192	13.8	-4.644	1791.94	24.79
3	9.333	192	15	-5.058	1791.94	26.96
4	11	192	16.1	-5.4	2112.00	34.06
5	9.333	192	16.8	-5.634	1791.94	30.15
6	9.333	192	17.4	-5.868	1791.94	31.27
7	9.333	192	18	-6.048	1791.94	32.32
8	9.333	192	18.4	-6.21	1791.94	33.07
9	9.333	192	18.9	-6.372	1791.94	33.96
10	9.333	192	19.3	-6.498	1791.94	34.67
11	9.333	192	19.7	-6.624	1791.94	35.38
12	9.333	192	20.1	-6.75	1791.94	36.09
13	9.333	192	20.4	-6.858	1791.94	36.63
14	9.333	192	20.7	-6.966	1791.94	37.18
15	12.333	192	21.1	-7.092	2367.94	50.07
16	12.667	192	21.5	-7.218	2432.06	52.38
17	9.333	160.5	21.6	-7.254	1497.95	32.42
					Base Shear =	584.81



CASE 3

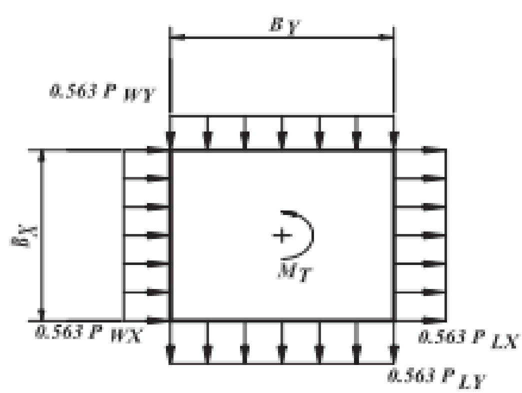
Case 4:

This case is the same described in case 3 but with 56.3% of the full design wind pressure being applied simultaneously to each side.

Case 4 N/S Wind Forces

Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.563 * Story Force (k)	B (ft.)	e (ft.)	M (ft.*k)
1	10.167	134.33	13.10	-4.39	1365.73	13.45	134.33	20.15	361.02
2	9.333	134.33	13.80	-4.64	1253.70	13.02	134.33	20.15	349.44
3	9.333	134.33	15.00	-5.06	1253.70	14.16	134.33	20.15	380.02
4	11	134.33	16.10	-5.40	1477.63	17.89	134.33	20.15	480.10
5	9.333	134.33	16.80	-5.63	1253.70	15.83	134.33	20.15	425.04
6	9.333	134.33	17.40	-5.87	1253.70	16.42	134.33	20.15	440.84
7	9.333	134.33	18.00	-6.05	1253.70	16.97	134.33	20.15	455.62
8	9.333	134.33	18.40	-6.21	1253.70	17.37	134.33	20.15	466.26
9	9.333	134.33	18.90	-6.37	1253.70	17.84	134.33	20.15	478.81
10	9.333	134.33	19.30	-6.50	1253.70	18.21	134.33	20.15	488.77
11	9.333	134.33	19.70	-6.62	1253.70	18.58	134.33	20.15	498.74
12	9.333	134.33	20.10	-6.75	1253.70	18.95	134.33	20.15	508.70
13	9.333	134.33	20.40	-6.86	1253.70	19.24	134.33	20.15	516.43
14	9.333	134.33	20.70	-6.97	1253.70	19.53	134.33	20.15	524.16
15	12.333	134.33	21.10	-7.09	1656.69	26.30	134.33	20.15	705.82
16	12.667	134.33	21.50	-7.22	1701.56	27.51	134.33	20.15	738.46
17	9.333	134.33	21.60	-7.25	1253.70	20.37	134.33	20.15	546.67
					Base Shear=	311.63			

Case 4 E/W Wind Forces									
Floor Number	Floor to Floor Height (ft.)	Wall Length (ft.)	Windward Pressure (psf)	Leeward Pressure (psf)	Tributary Area (sqft.)	0.563 * Story Force (k)	B (ft.)	e (ft.)	M (ft.*k)
1	10.167	134.33	13.10	-4.39	1365.73	17.57	175.50	26.33	616.22
2	9.333	134.33	13.80	-4.64	1253.70	18.61	192.00	28.80	713.89
3	9.333	134.33	15.00	-5.06	1253.70	20.24	192.00	28.80	776.36
4	11	134.33	16.10	-5.40	1477.63	25.56	192.00	28.80	980.81
5	9.333	134.33	16.80	-5.63	1253.70	22.63	192.00	28.80	868.33
6	9.333	134.33	17.40	-5.87	1253.70	23.47	192.00	28.80	900.61
7	9.333	134.33	18.00	-6.05	1253.70	24.26	192.00	28.80	930.80
8	9.333	134.33	18.40	-6.21	1253.70	24.83	192.00	28.80	952.55
9	9.333	134.33	18.90	-6.37	1253.70	25.50	192.00	28.80	978.17
10	9.333	134.33	19.30	-6.50	1253.70	26.03	192.00	28.80	998.53
11	9.333	134.33	19.70	-6.62	1253.70	26.56	192.00	28.80	1018.89
12	9.333	134.33	20.10	-6.75	1253.70	27.09	192.00	28.80	1039.25
13	9.333	134.33	20.40	-6.86	1253.70	27.50	192.00	28.80	1055.04
14	9.333	134.33	20.70	-6.97	1253.70	27.91	192.00	28.80	1070.84
15	12.333	134.33	21.10	-7.09	1656.69	37.58	192.00	28.80	1441.95
16	12.667	134.33	21.50	-7.22	1701.56	39.32	192.00	28.80	1508.63
17	9.333	134.33	21.60	-7.25	1253.70	24.33	160.50	24.08	780.42
					Base Shear=	438.99			



Wind Drift Checks:

The worst case drift conditions for each wind load case were determined and listed below. The maximum drifts experienced were compared to the accepted industry standard limit of H/400 for drift. All cases pass the allowable drift limits under wind loads. For each case, the maximum drift shown was measure at the 17th level of the building.

Drift due to Wind Load Cases			
Load Case	**Maximum Drift (in)**	**Allowable Drift (in)**	**Pass/Fail**
Wind Case 1 – X Direction	4.16	5.025	PASS
Wind Case 1 – Y Direction	4.52	5.025	PASS
Wind Case 2 – X Direction (+M)	2.71	5.025	PASS
Wind Case 2 – X Direction (-M)	4.07	5.025	PASS
Wind Case 2 – Y Direction (+M)	2.80	5.025	PASS
Wind Case 2 – Y Direction (-M)	4.76	5.025	PASS
Wind Case 3	3.00	5.025	PASS
Wind Case 4 (Additive +Moments)	3.78	5.025	PASS
Wind Case 4 (Additive –Moments)	3.59	5.025	PASS
Wind Case 4 (+M's in Opposite Directions)	3.69	5.025	PASS
Wind Case 4 (-M's in Opposite Directions)	4.28	5.025	PASS

The story forces and building loads presented in the above tables were applied to the building. Results of displacement and stresses were compared to serviceability and strength criteria from ASCE 7-10. The wind forces determined on the building act through the center of pressure, while the seismic forces are exerted through the center of mass. All eccentricities are with respect to the center of rigidity of each floor. The floor diaphragms are rigid and distribute the lateral loads based on location of the lateral force resisting elements.

Pictured below are images from ETABS showing the shell stresses in the shear walls. The two shear walls shown in elevation are shear walls 1 and 2 which are each 14" thick and span the entire height of the building. As expected, stresses are greater at the ends of the wall due to the walls behavior in flexure. Shear stresses are greater at the lower floors of the building.

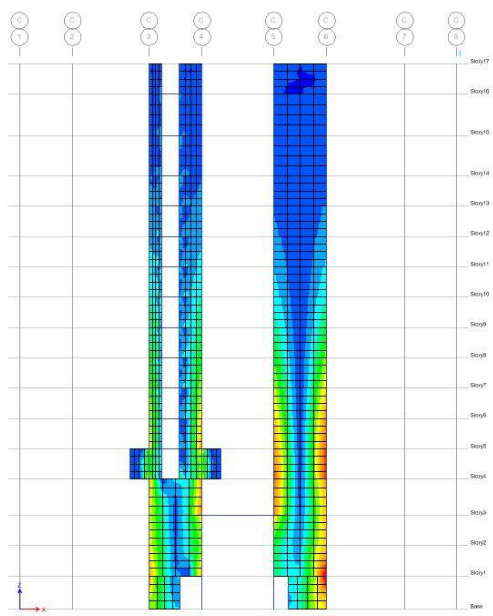


Figure 41: Shell Stresses of Shear Wall 1 and 2

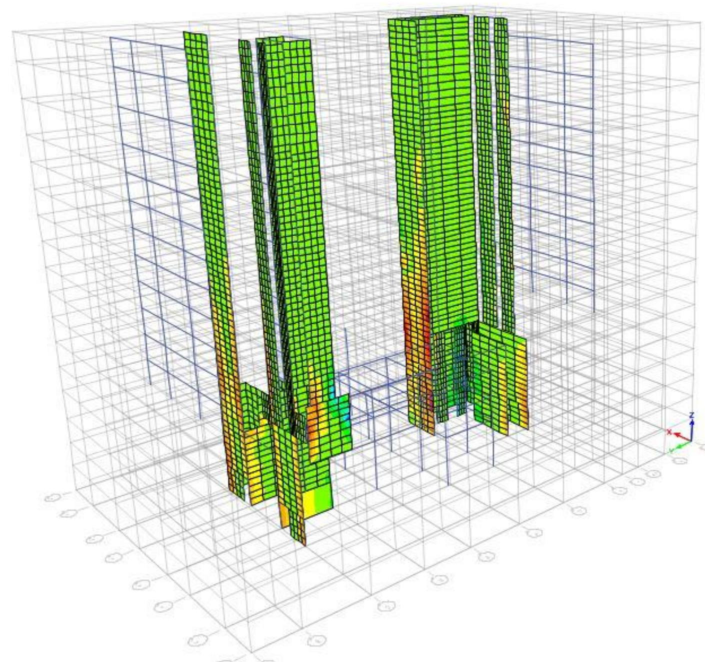


Figure 42: Shell Stresses of Shear Walls

Seismic Forces

As discussed in Technical Report 2, 8621 Georgia Avenue falls into a Seismic Design Category A. Due to this, the building is exempt from the more detailed analysis for seismic loading found in ASCE Ch. 11. The seismic loading for this building is governed by the provisions in Section 1.4 for the general structural integrity of the building.

Therefore, the seismic story forces are given by taking 1/100th of the story weight. A rough approximation of the story weights was performed in Technical Report 2. The following table includes a more detailed summation of the total dead load structural mass on each floor. Because the simplified method for determining seismic story forces is entirely dependent on mass, the story forces are the same in both the X and Y direction.

Tables 12.3-1, 2 were investigated for horizontal and vertical building irregularities. None of the irregularities are applicable for Seismic Design Category A so no additional requirements are necessary. The building maintains a relatively geometric profile throughout its perimeter and height so this is a reasonable conclusion.

Although additional provisions were not required due to structural irregularities, torsional effects were considered when placing elements of the lateral force resisting system.

Seismic Drift Checks

After a seismic analysis of the building was performed using ETABS. The results below document the story displacement and story drift. The allowable drift limit under seismic load was determined using Table 12.12-1 in ASCE 7-10 for allowable Seismic Story Drift. For a building of risk category I, the allowable story drift is 2%. The maximum drift values occurred at the 17th floor and all passed the allowable drift limit.

Displacements due to Seismic Loading

	X Direction				Y Direction			
Floor	**Story Displacement (in.)**	**Story Drift (%)**	**Allowable Drift (%)**	**Pass/Fail**	**Story Displacement (in.)**	**Story Drift (%)**	**Allowable Drift (%)**	**Pass/Fail**
17	2.85	0.144	2%	PASS	1.52	0.076	2%	PASS
16	2.74	0.142	2%	PASS	1.45	0.076	2%	PASS
15	2.41	0.138	2%	PASS	1.28	0.073	2%	PASS
14	2.00	0.125	2%	PASS	1.07	0.067	2%	PASS
13	1.78	0.120	2%	PASS	0.95	0.064	2%	PASS
12	1.55	0.113	2%	PASS	0.83	0.060	2%	PASS
11	1.33	0.105	2%	PASS	0.71	0.056	2%	PASS
10	1.11	0.097	2%	PASS	0.60	0.052	2%	PASS
9	.90	0.087	2%	PASS	0.49	0.047	2%	PASS
8	.71	0.077	2%	PASS	0.38	0.041	2%	PASS
7	.52	0.064	2%	PASS	0.28	0.035	2%	PASS
6	.35	0.050	2%	PASS	0.20	0.028	2%	PASS
5	.20	0.034	2%	PASS	0.12	0.020	2%	PASS
4	.07	0.015	2%	PASS	0.07	0.014	2%	PASS
3	.03	0.008	2%	PASS	0.04	0.012	2%	PASS
2	.04	0.002	2%	PASS	0.03	0.011	2%	PASS

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

	Risk Category		
Structure	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$\boxed{0.020h_{sx}}$	$0.015h_{sx}$	$0.010h_{sx}$

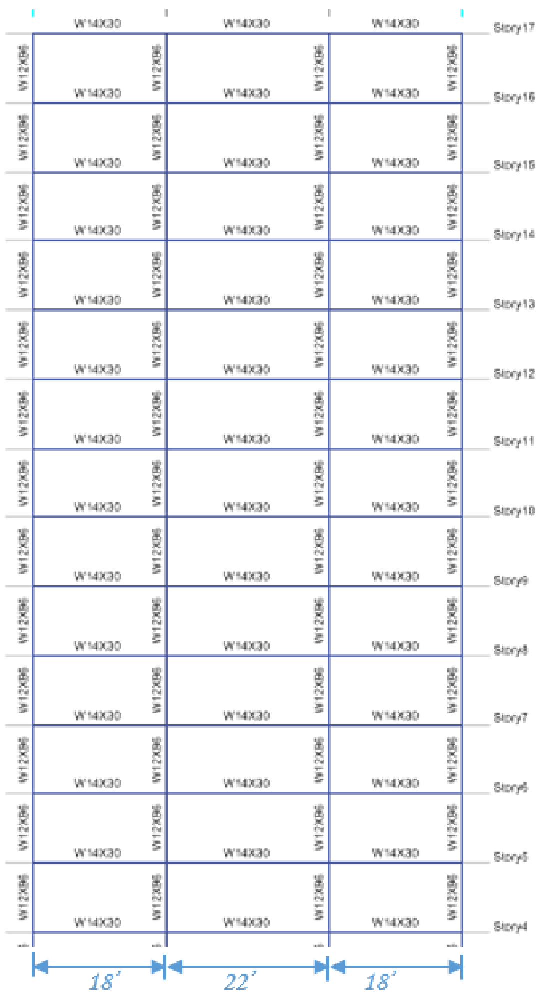
[a] h_{sx} is the story height below Level x.

Steel Moment Frames

As previously mentioned in the overview of the lateral system design, two 3-bay long moment frames were added to the structure. These moment frames are located at the north and south end of the building and rise the full building height. The application and placement of these frames were determined in order to address specific design considerations.

The addition of these moment frames was prompted by a high story drift in the X direction of the building due to wind case 2. This wind case involves a wind load in the east-west direction of the building in addition to a moment. In order to reduce X-direction displacement and building torsion, these frames were oriented at the perimeter of the building in the X direction.

Creating moment frames within the structural also provided the opportunity for a more extensive investigation into designing the typical connections for the structure.



Columns: W12x96

Beams: W14x34

Figure 43: Moment Frames

The previous figure shows an elevation view of one of the frames. The column and beam sizes are typical sizes that appear elsewhere in the structure. As opposed to the gravity columns which reduce in size at upper floors due to a reduction in load, the size of the columns in the moment frames are maintained in order to resist the lateral load as well as the axial load.

Below is an elevation of the same moment frames with the moment diagram superimposed over the frame. The moments do not change with regards to the building height, thus reinforcing the design decision to maintain the same column size throughout the height.

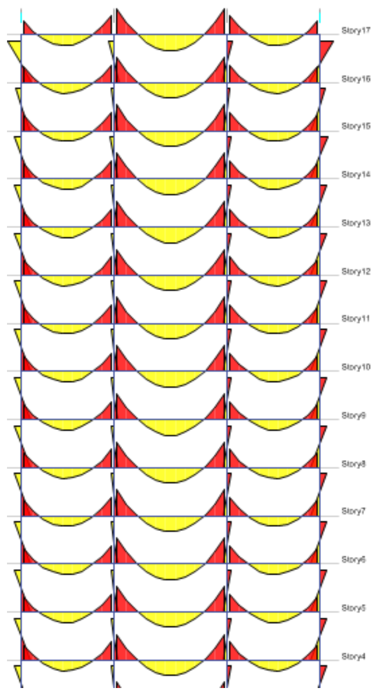


Figure 44: Moment Diagram on Moment Frames

MAE Coursework Integration

Requirements for the Masters of Architectural Engineering degree were met by applying graduate level coursework to multiple parts of my thesis. The knowledge gained in these classes in most represented through the computer modeling and connection design portions of this project. The application of these skills utilized material learned in AE 530, *Computer Modelling of Building Structures*, as well as AE534, *The Design of Steel Connections*.

The gravity system of 8621 Georgia Avenue was designed using RAM Structural System. This was a program that was learned through internships, in-class tutorials, and a self-study of the software. This self-study was performed by completing all of the tutorials offered by Bentley for their software. RAM was useful in modelling the gravity system by providing interactions and deflections for all of the steel members as well as calculating material take-offs based on member designs.

The lateral system was modelled in ETABS which is a computer program that was primarily learned in AE 530. This software was used in both the fall and spring semester. ETABS effectively models and applies seismic and wind forces to the lateral system of the building. The equivalent story forces were verified by hand and by excel spreadsheet. These values were used to design and check the elements of the lateral system.

Another software that was used throughout the duration of the project was Risa 2D. This program was learned through instruction in multiple AE classes over the past years. Risa was used to perform basic calculations to determine shear and moment forces, or deflection values for simple beam or frame arrangements. For the redesign, Risa was specifically used to model the lateral soil forces on the foundation wall and calculate the maximum moment and shear forces.

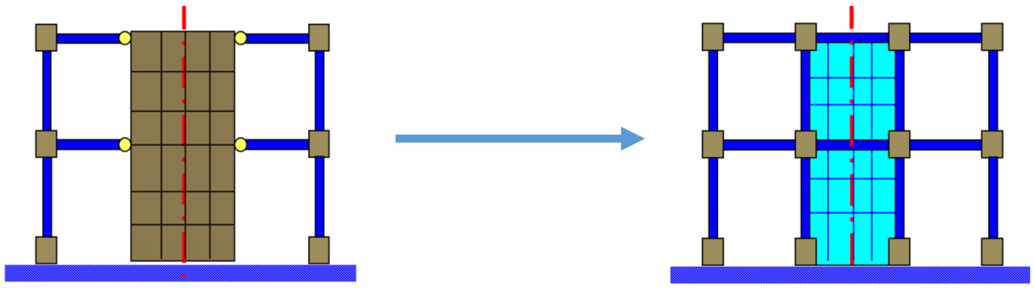
Two typical connections for the building were designed. The majority of the steel gravity system uses a simple shear connection for the beam-girder and girder-column connections such that moment is not transferred. The moment frames in the building require moment connections at the girder-column conditions. The typical shear and moment connection were designed.

Modeling Decisions

The structure considered for this analysis is a 17 story concrete building with shear walls as its primary lateral resisting members. There are some drop beams on the lower 4 levels to accommodate the parking garage. Although all concrete frames transfer some moment and lateral force, only the shear walls, drop beams, and columns directly supporting them will be included in the model. This decision is made both to simply the model but also to conservatively determine the loads on these elements.

The 14 shear walls in the building were all modeled as membrane elements. Membranes do not account for out-of-plane shear forces because they have no out-of-plane stiffness. This is ideal because in our theoretical lateral analysis we assume that shear walls can only resist in-plane loads.

In modeling the shear walls as membranes, extra effort had to be taken to assure the proper shear and moment continuity where beams framed into the shear walls. Additional “fake” beams and columns (the same thickness as the shear wall) had to be added in these circumstances. This was especially the case on some of the coupled shear walls to adequately model the coupling beams.



The diaphragms on every floor were modeled as being rigid. This allowed the lateral forces to transfer and be distributed to the lateral force resisting elements. The forces transferred from the rigid diaphragm are distributed based on the location of the lateral force resisting elements.

The openings in the floor diaphragms were not modeled. Large opening in the shear walls for doors were included but all other smaller openings were not modeled. This was done as a means to avoid unnecessary complexity within the model. The decision to disregard these openings will have negligible results on the model.

Connection Design

Another application of the graduate degree coursework from the AE program was found in the connection design of the two typical connections within the building. Two connections were designed by hand; a shear tab, and web-bolted, flange-bolted moment connection. The shear tab is the most common connection in the redesign steel system for 8621 Georgia Avenue.

A shear tab was used, as opposed to other shear connections, because it is relatively inexpensive and easy to install. The connection accounts for the majority of the connections in the building so a small cost or time savings per connection could become substantial in the scope of the entire project.

A sketch of a shear tab connection is shown below. This is merely an example diagram, the final connection design has been sketched within the hand calculations for the connections.

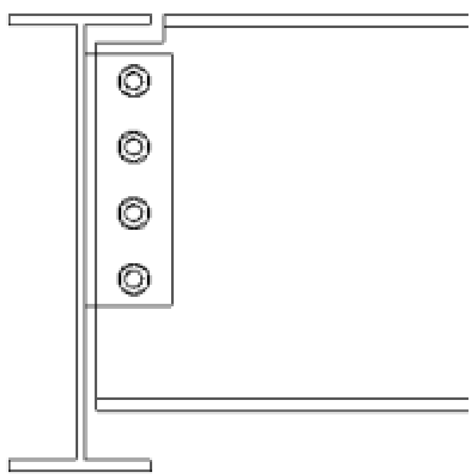


Figure 45: Example of a shear tab connection

A shear tab connection is simply a plate that is welded to the web of a girder or column and then attached to the beam by bolts or welds. The designed shear tab requires four ¾” diameter A325N bolts for the beam-girder or girder-column connection.

A typical moment connection was also designed to be implemented in the moment frames at the north and south side of the building. Because these frames are participating in the lateral system, the connections need to properly transfer moment from the beams to the columns.

An example diagram of a web-bolted, flange-bolted moment connection is shown below. Plates are bolted to the web and flange of the girder and welded to the flange or web or the column. Doubler plates are also shown below in the example connection. Doubler plates are used within the column to stiffen and support the flanges in the shear zone of the girder. Although in the case of the moment frames in 8621 Georgia Avenue, the shear force is not high enough for doubler plates to be needed.

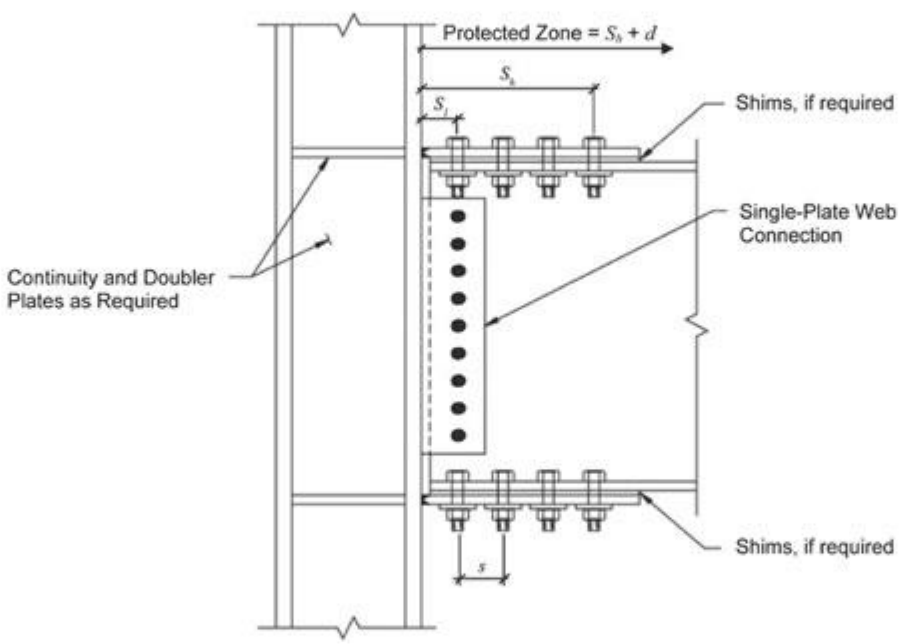


Figure 46: Example of a flange-bolted, web-bolted moment connection

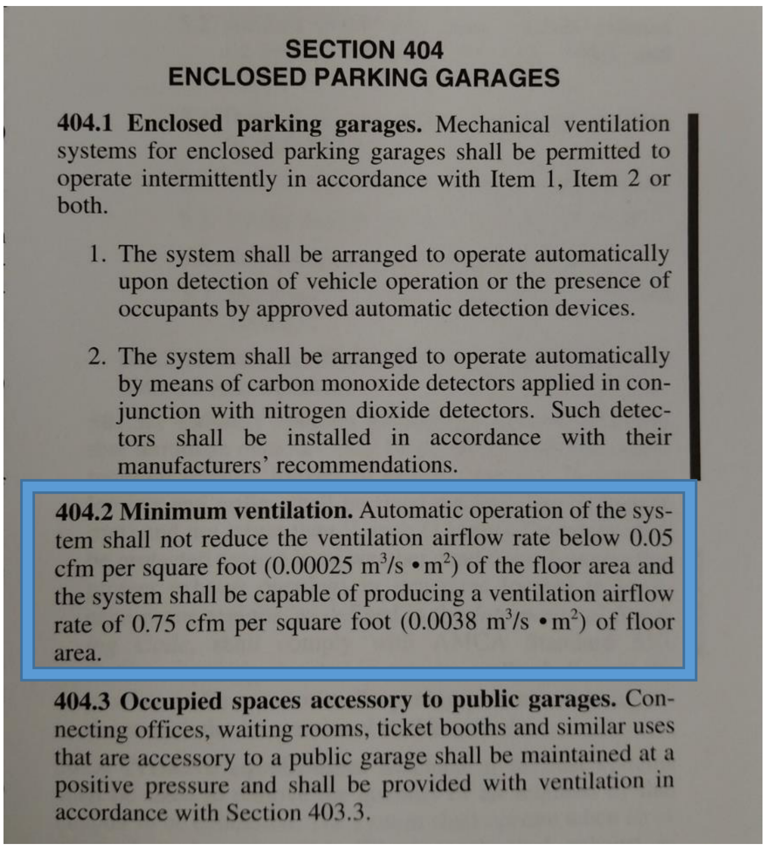
More detailed assumptions, diagrams and limit states of these two connections can be found in the calculations within Appendix H. Although each connection was designed in detail, a 10% structural steel allowance will be factored in for connections as opposed to pricing out each element of the connection. This is an accepted industry rule of thumb and will be acceptable for this cost analysis application.

Breadth #1: Mechanical

In the existing design of the parking garages, they were above grade and design as an open air structure. In order to facilitate the structural redesign, one of these floor needed to be moved below grade. With this design change, a ventilation system needs to be design to exhaust the air from the parking garage.

The International Mechanical Code, as well as the ASHRAE Handbook, give the same minimum ventilation airflow rate of 0.75 CFM per square foot. The design for the parking garage will include 4 exhaust fans to remove air and rely on the created negative pressure to bring in fresh air from the outside and floor above.

The required exhaust load per fan was found using the code specified airflow rate and the square footage of the parking garage. It was assumed that each fan would exhaust fan equally and contribute the same amount to the overall required CFM to be exhausted.



SECTION 404
ENCLOSED PARKING GARAGES

404.1 Enclosed parking garages. Mechanical ventilation systems for enclosed parking garages shall be permitted to operate intermittently in accordance with Item 1, Item 2 or both.

1. The system shall be arranged to operate automatically upon detection of vehicle operation or the presence of occupants by approved automatic detection devices.
2. The system shall be arranged to operate automatically by means of carbon monoxide detectors applied in conjunction with nitrogen dioxide detectors. Such detectors shall be installed in accordance with their manufacturers' recommendations.

404.2 Minimum ventilation. Automatic operation of the system shall not reduce the ventilation airflow rate below 0.05 cfm per square foot (0.00025 $m^3/s \cdot m^2$) of the floor area and the system shall be capable of producing a ventilation airflow rate of 0.75 cfm per square foot (0.0038 $m^3/s \cdot m^2$) of floor area.

404.3 Occupied spaces accessory to public garages. Connecting offices, waiting rooms, ticket booths and similar uses that are accessory to a public garage shall be maintained at a positive pressure and shall be provided with ventilation in accordance with Section 403.3.

Figure 47: Section 404 from the International Mechanical Code

Two of the fans in the southwest corner exhaust the air directly outdoors while the other two fans utilize ducting to achieve an even and distributed air exhaustion across the floor plan. The blue arrows on the floor plan show the flow of air due to the negative air pressure.

Another consideration was the location that the air would be exhausted. The south and east side of the building are heavy pedestrian areas, so the air needed to be exhausted to the west in order to meet the ASHRAE minimum distances between air exhaust vents and building openings. The fan in the southeast corner required additional ducting to carry the exhausted air to the west side of the building.

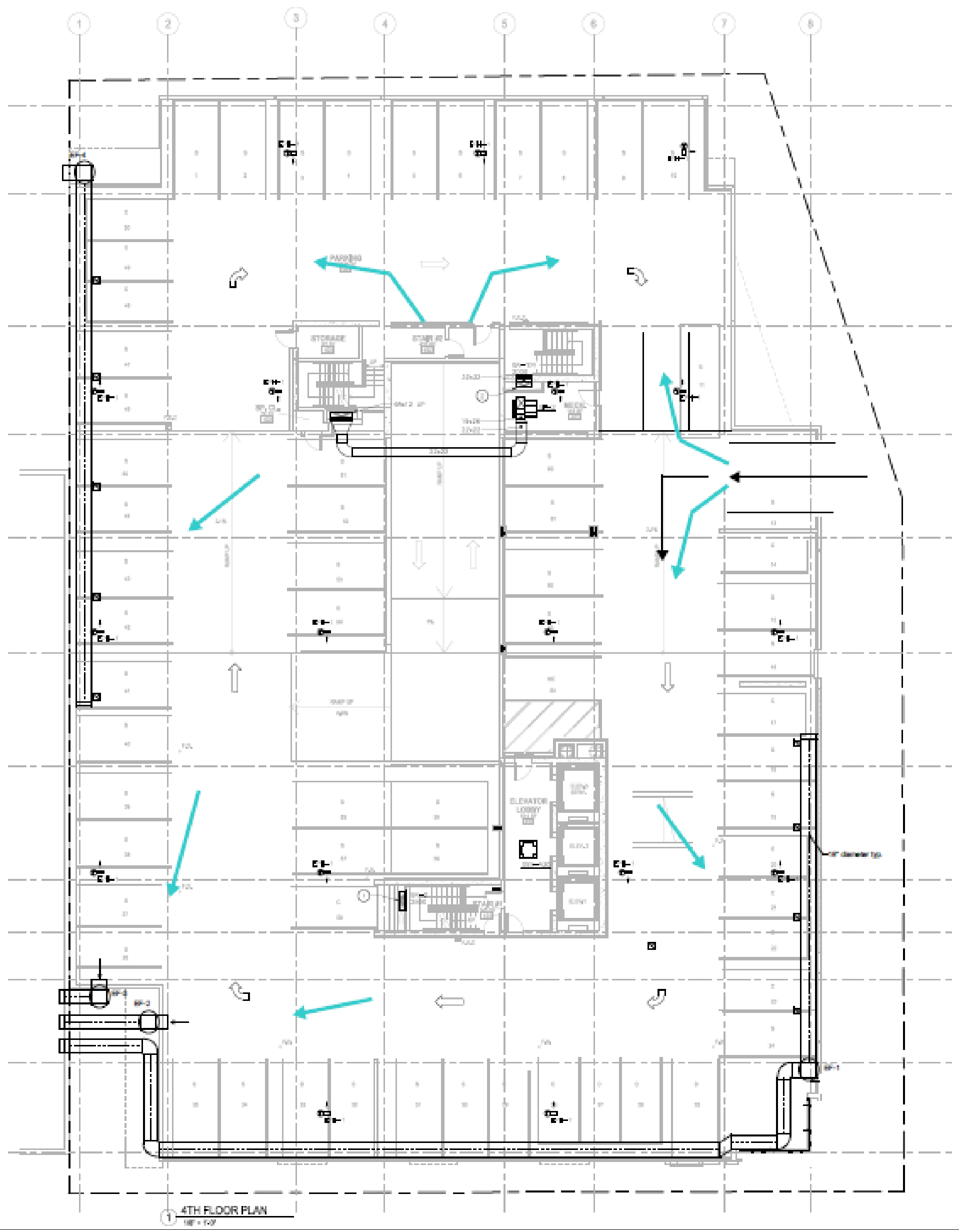


Figure 48: New Mechanical Design superimposed on existing parking garage layout

Another criteria that was considered for the ventilation of the parking garage was the number of air changes that would take place per hour. For parking garages, 4-6 air changes per hour is a recommended value, so the ventilation system was designed for a total fresh air supply load of 5 air changes per hour.

Using two tables from the ASHRAE Handbook, the ducting was sized in order to carry the air exhaust load based on an air speed of 1800 ft/min. These tables can be found in Appendix I. Circular and rectangular duct sizes were found using the tables but circular ducts were selected based on availability of ducts that large.

The additional ventilation something also comes with an incurred cost. In the following pages of calculations, a cost analysis of the ventilation system was performed using RS Means Mechanical Cost Data 2015. The greatest cost of the ventilation system is the additional excavation required at the three corners of the building where the fans are located in. The total cost of the ventilation system will be an estimated $52,300.

In summary, due to the rearrangement of the parking garage floors, air needs to be exhausted from the floor below grade. Four exhaust fans will be provided to exhaust the air and supply fresh air via a negative air pressure. The cost of installing this system would be $52,300.

Breadth #2: Cost Analysis

In order to make a more realistic determination of the feasibility of a steel system, a detailed cost analysis was performed on the building. The scope of the cost analysis focused upon the elements of the building that were changed or redesigned in order to get a final additional cost or savings by switching to a steel system. Cost information used and provided by the general contractor was used for pricing elements of the existing structure. Newly designed elements were priced using RS Means Construction Cost Data 2015.

The major costs considered were the savings from redesigning the foundations, the cost difference between the steel and concrete system, the additional excavation cost, and other miscellaneous incurred costs.

The following cost data was acquired using RS Means Construction Cost Data 2015. The suburbs around DC tend to have cheaper concrete rates than on average due to the popularity of concrete construction in the area. For this reason, the steel system may be more expensive than RS Means may yield. This fact will not be included in the cost analysis but will be considered in the final system comparisons.

Steel System Cost Analysis		
Redesigned System	Cost	Net Difference
Steel Framing	$9,452,438.87	- $3,400,735.87
Foundation	$1,028,110	+ $721,356.23
Ventilation	$52,273.75	- $52,273.75
Additional Excavation	$149,803.90	- $149,803.90
Parking Spaces	$79,380.00	- $79,380.00
Totals	$10,762,006.52	- $2,960,837.29

Detailed item by item cost breakdowns can be found in Appendix J. The net differences of the systems refer to the comparison between the designed system and what was redesigned to accommodate the steel structural system.

In order to appropriately price the cost of eliminating parking spots, the total costs will be expressed in dollars lost per year. The annual cost of a parking place will be estimated based on the monthly cost of a nearby garage at 921 Wayne Avenue (shown below). Because this garage is only .3 miles away from 8621 Georgia Avenue, the same monthly rate for parking will be applied.

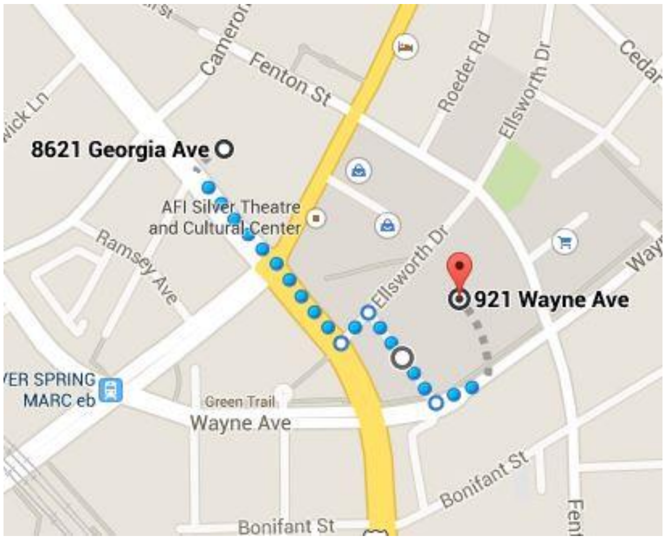


Figure 49: Map showing a nearby parking garage

The cost of a monthly parking space at 921 Wayne Avenue is $189/month. Considering the 35 spots that were eliminated over the 4 floors of parking garage, the total cost of this change is $79,380. A breakdown of the annual losses per floor can be found in Appendix J.

System Comparisons

The original goal in performing this investigation was to determine whether or not a steel system is feasible to implement on 8621 Georgia Avenue. The comparison between the two systems will evaluate all aspects of the systems and not simply just cost. This section of the report will only point out the advantages and disadvantages of each system while concluding with a final recommendation for the building owner.

The first factor that weighs into the comparison is the overall cost of the building. As determined by the cost analysis in the previous section, the steel system and the supplemental design additions caused from that switch, is $2,960,000 more expensive. This price differential would mean a 6% increase in cost from the original $52 million. There were some savings available in reducing the foundations but the steel system on it's on is an additional $3.4 million cost.

Another factor to consider would be the availability of labor and work. In the DC area, concrete is the favored building material. The price of building in concrete is generally cheaper because of the way the building market in DC is. Designing a building in steel could cause some difficulties bringing in materials

The site of 8621 Georgia Avenue is a confined space, surrounded by buildings on all side. This could create a problem for staging and lay down in a steel building. In a concrete building, the concrete can be supplied in smaller amounts via trucks and does not take up as much site space. Therefore, a concrete system has some construction management benefits.

Another aspect worth comparing between the two systems is the floor vibrations. The initial concrete system is resistant to vibrations due to the stiff nature of concrete and thickness of the slab. When optimized for strength and deflection, the steel system did not meet the vibration criteria and need to be increased in size. Both systems meet vibration criteria, but the concrete system is more efficient in doing so.

The floor to ceiling heights in the apartments is the same as originally designed. This was only accomplished by moving a level of parking garage below grade. Nonetheless, the ceiling height per floor is unchanged.

The fire resistance of the ceiling to floor construction in both systems is designed to meet a 2 hour fire rating. The 1.5VLI18 deck for the steel redesign was chosen to achieve this fire rating. The steel columns will need fireproofing applied to them or will need to be encased in a fireproof material.

Therefore, in many categories the two systems are equivalent in meeting certain design criteria. Yet, weighing the advantages and disadvantages it is clear that concrete is the better system. A steel system is still very feasible but comes with a higher cost and some construction difficulties without any staggering advantage over concrete.

Given all of the previously discussed designs and analysis, the steel system does appear to be feasible but would not be recommended. Converting this concrete building into a steel building has many design constraints but this report demonstrates that it is possible to meet these design goals. The steel system is not as efficient or cost effective but works for this application. Therefore it is very feasible for 8621 Georgia Avenue to be redesigned in steel.

A potential advantage to the steel system that has not been investigated yet would be an extensive schedule analysis. If there was a significant time savings in the project construction time, the steel option may begin to be viable. Otherwise, outside of any considerable schedule benefits, the concrete system seems to be the better decision.

Although the redesign steel system may not be the most effective system for the building owner to employ, it may be worth investigating an additional level of parking below grade. The cost of additional excavation was not very excessive because it is only one floor below grade. The additional income from another full floor of 50 parking place could make the initial cost worth it in the long term.

Conclusions

This report consisted of an analysis and redesign of the recently designed 8621 Georgia Avenue building in Silver Spring, MD. During the fall semester, analyses were conducted on the proposed gravity and lateral systems. The original designs were determined to be adequate for strength and serviceability criteria. After successfully concluding that the original building design is deficient, a hypothetical scenario was created to investigate the feasibility of a composite steel system.

The structural redesign utilized a composite steel system which replaced a post tensioned concrete flat slab. The composite action of the steel was chosen over non-composite in order to minimize the structural depth. The bay sizes needed to be rearranged in order to be more suitable for a steel system. The program of the parking garage needed to be slightly shifted in order to accomplish a modular bay size across the floor plan. The gravity system was designed using RAM structural system and verified by hand.

The transition to a steel system decreased the building weight and allowed the foundations to be reduced and redesigned. The foundations simplified and reduced to result in a significant cost savings. The foundations were designed in RAM and checked by hand.

The lateral system of the building was subjected to wind and seismic loads. Wind loads were found to be the controlling lateral load. The existing lateral system was modified and added steel moment frames were created to aid in story drift control and building torsion. ETABS was used to model the lateral system.

The first breadth study included a design of the ventilation system that would need to be implemented to ventilate the underground parking garage. Exhaust fans were sized and provided to remove air from the parking garage based on airflow rates in the ASHRAE Handbook

The second breadth was a cost analysis of new steel system. This cost analysis considered secondary effects from the steel redesign such as reduced foundations and a new ventilation system. The cost analysis was used to not only determine the price of the new steel system but the additional cost more than the concrete system.

The typical connections in the steel system were also design. The shear and moment frame connections were evaluated by hand and factored into the overall cost analysis.

It was determined that a steel option for 8621 Georgia Avenue would be feasible, yet not as efficient or cost-effective as concrete. Both systems meet criteria for vibrations, fire, and strength. The two systems also have the same floor to ceiling height despite the thicker structural depth. The initially design concrete system offers a cost savings compared to the proposed steel design. The only factor that might advocate for a steel system would be an expedited construction time.

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Note: Various course notes and handouts were also used in the completion of this report in addition to those listed above.

Appendices:
Appendix A: Design Loads

The determination of the design loads for the project were found using the codes and references listed in the previous section of this report. The following section will report from where in each particular code that the design values are derived from.

National Codes

The two codes that were used in the design of the building were the IBC 2012 and ASCE 7-10. Chapters 4, 11-30 on live loads and lateral loads were used to generate the loadings for these conditions in 8621 Georgia Avenue. All of the design loads used in the project can be found on sheet S0.01

Gravity

Dead Load

The typical roof, floors, and parking areas were given an additional superimposed dead load in addition to the material self-weights. Other atypical conditions received an additional superimposed dead load based upon experience and specifications with those systems.

Superimposed Dead Loads in addition to the Self-Weight	
Structural Element	Weight (psf)
Typical Roof	30
Typical Floor	15
Parking Areas	10
Unique Conditions	
Intensive Green Roof	60
Bio-Retention Planter	600
Courtyard Planters	240

Figure 50: Superimposed Dead Load Values

Live Load

All live loads were determined using Chapter 4 of ASCE 7-10 and Chapter 16 of IBC 2012 on live loads. In accordance with IBC 2012 section 1607.02, the column, foundation, and beam live loads were able to be reduced.

Snow Load

The ground snow load for Silver Springs, Maryland is recorded as 30PSF according to Chapter 7 of ASCE 7-10. In most cases, the roof snow load can be reduced by a factor of 0.7 (assuming no other factors apply) but the Montgomery County amendments set the minimum roof snow load to 30 PSF, so there is no reduction from the ground to roof snow load

Lateral Loads

The Lateral loads for 8621 Georgia Avenue were determined using chapters 11-13 and 26-30 covering seismic and wind loading. For this project the wind load was the controlling lateral load. Similar to the gravity loads, all design loads are found on sheet S0.01.

Wind

The wind load was specifically found using chapters 26-30 from ASCE 7-10. The building is considered to be Risk Category 2 with a Wind Exposure Category C and basic wind speed of 110 MPH. Net design pressures on various parts of the enclosure are given in the table below:

Net Design Pressures	
Walls (Zone 4)	+20 PSF, -20 PSF
Walls (Zone 5)	+20 PSF, -34 PSF
Roofs (Zone 1)	-27 PSF
Roofs (Zone 2)	-44 PSF
Roofs (Zone 3)	-59 PSF

Figure 51: Net Wind Pressures

Seismic

The seismic design loads were primarily found in Chapters 11 and 12 of ASCE 7-10. Specific components and systems dealing with the architecture, mechanical, electrical, etc. also reference Chapter 13 of ASCE 7-10.

The building is a Risk Category 2 with an importance factor of 1.0 that falls in Seismic Design Category A.

Soil

The lateral soil loads on the building were the same loads recommended by the geotechnical report performed by Schnabel Engineering Consultants, Inc. The soil load was determined to have a sliding resistance of 0.35 and a net pressure of 50 PSF/ft of depth.

Appendix B: Framing Options

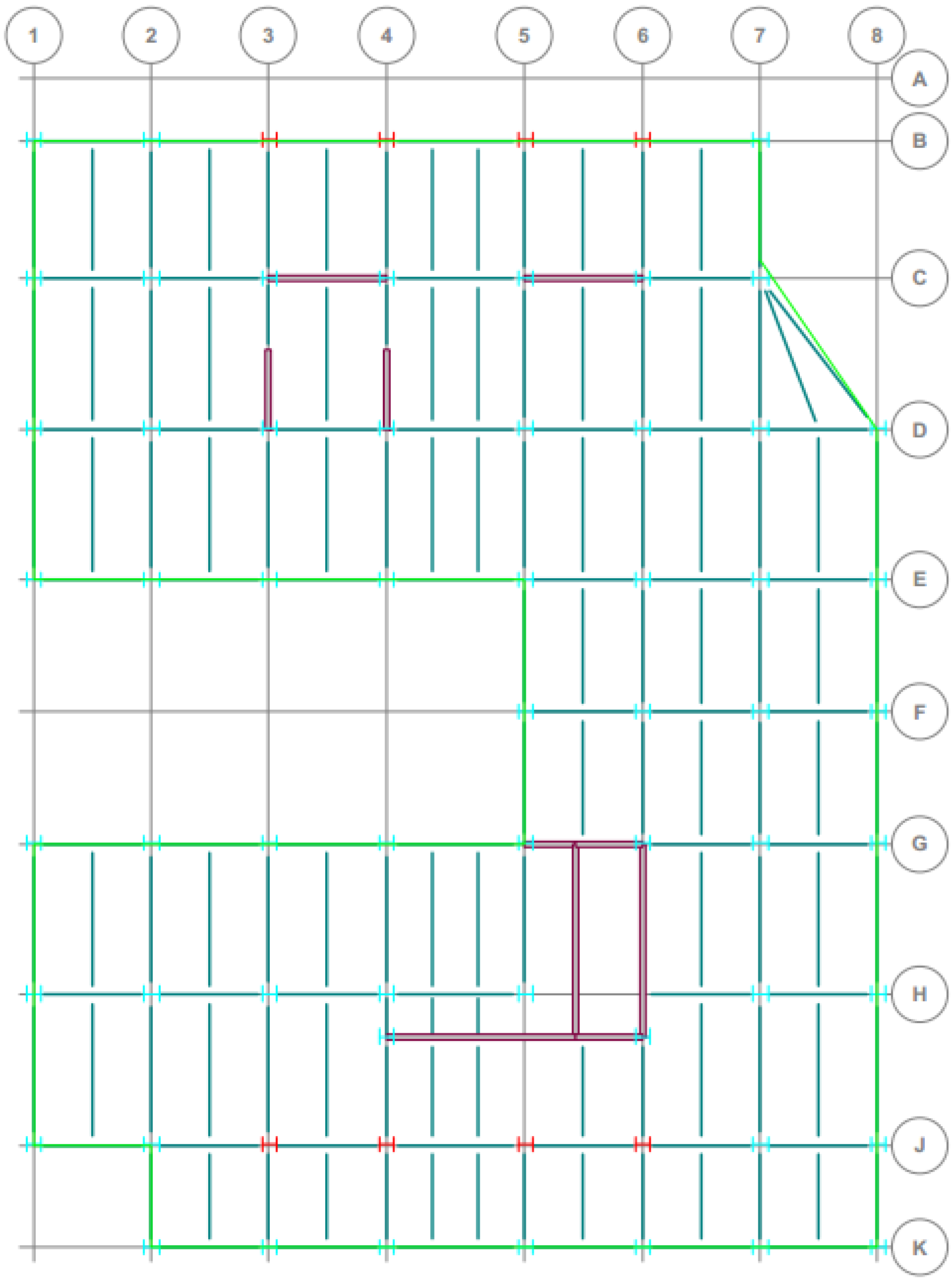


Figure 52: Beams Spanning Longways

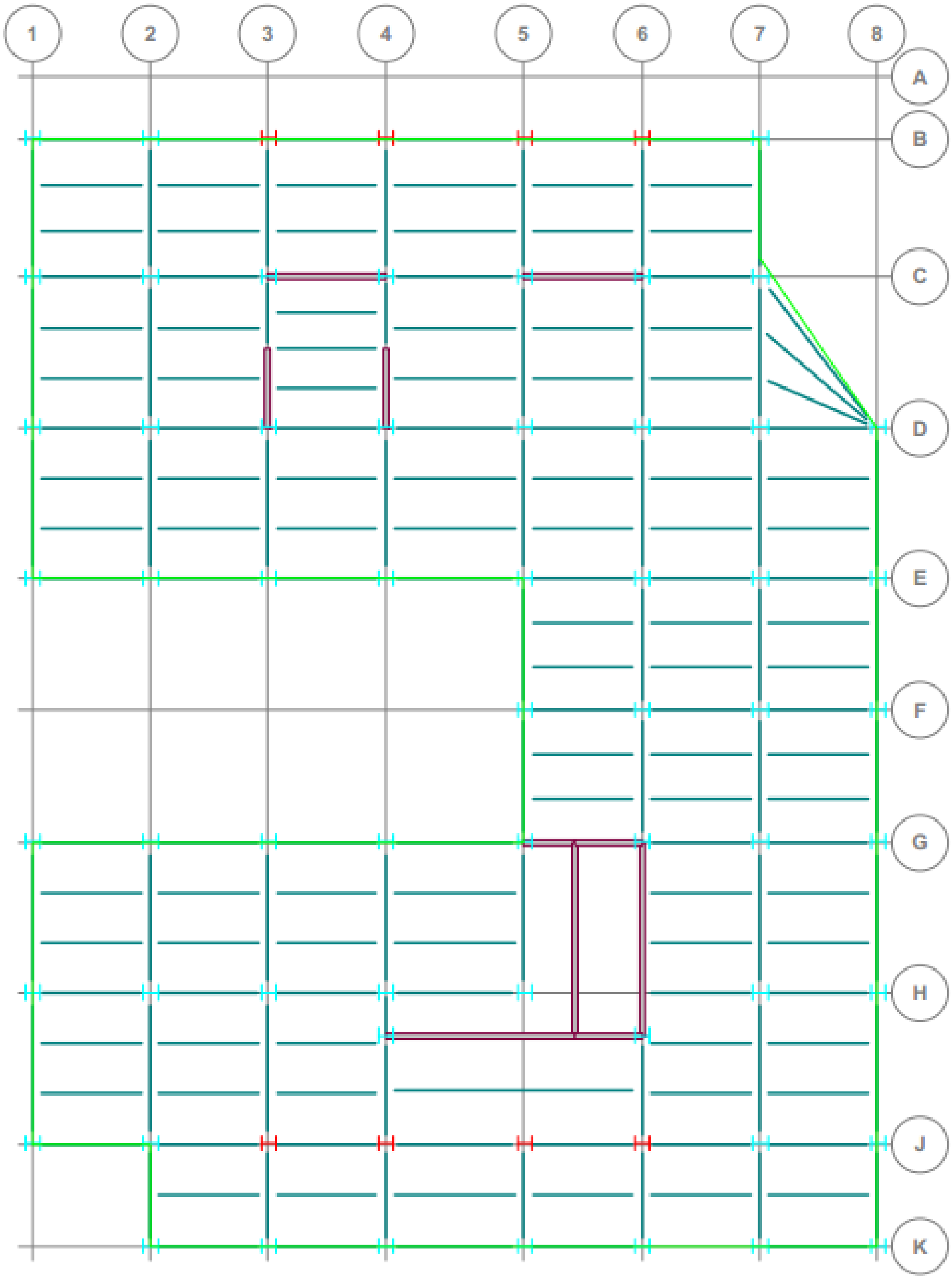


Figure 53: Beams Spanning Shortways

Appendix C: RAM Model Checks

Redesign Scope:
- Move (1) layer of parking garage below grade
- Design foundations and foundation walls
- Design apartment levels in steel
  - keep parking garage in concrete.

Depth of Structure

16 floor to floor heights
−1 level moved below grade
−3 levels of existing reinforced concrete parking garage
12 floors of steel to be redesigned.

$\frac{9'\text{-}8''}{12} = \frac{116}{12} = 9.66\text{ in.}$ ← additional depth gained per floor by moving a level below grade

$9.67'' + 7.25'' = 16.92\text{ in.}$ → max structural depth.
(7.25″) → existing structural depth

- Decrease penthouse ceiling height by 1'-10" so that the new floor to floor height is 10'-6" which is still larger than the typical 9'4" apartment.

$\frac{1'\text{-}10''}{12} = \frac{22}{12} = 1.83'' + 16.92'' = 18.75''$

$18.75'' - 4.75'' = 14''$ → max depth of girder.
(4.75″) → deck depth

Deck

1.5VL118, 4.75" (t = 3.25") 41 psf

2 span max unshored clear span − 10'-1"

Loads

DL = 41 psf (deck) + 15 psf (superimposed) + 5 psf (beam allowance) + 2 psf (girder allowance) + 2 psf (collateral) = 65 psf

LL = 50 psf → residential loading
100 psf → public corridors

Live Load Reduction

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

$$L = 100\left(0.25 + \frac{15}{\sqrt{4 \cdot 432}}\right)$$

$L = 61.1$ psf

Design Loads

1.2 DL + 1.6 LL

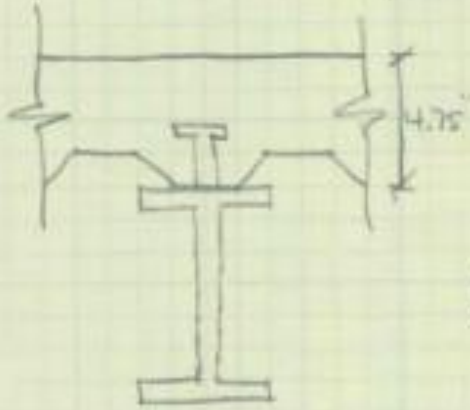
$W_u = 1.2(65) + 1.6(61) = 176$ psf

1.5VLI18 @ 9'-0" o.c.
211 psf > 176 psf

- one infill beam @ midspan; s = 9'

$W_u = 176 \text{ psf} \times 9' = 1.584$ klf

$$M_u = \frac{w\ell^2}{8} = \frac{1.584(24)^2}{8} = 114.0 \text{ ft-k}$$



24 ft. beams, max 1 stud/ft.

1 stud/rib = 17.2^k/stud
2 stud/rib = 14.6^k/stud

Assum $a = 1"$, $Y = 4.75" - \frac{1}{2} = 4.25"$

∴ use 4" to be more conservative

Trial Size for Composite Beams

W12x19, $\Sigma Q_n = 69.6^k \rightarrow \frac{69.6}{17.2} = 4.05 \rightarrow 5 \times 2 = 10$ studs/beam

W12x16, $\Sigma Q_n = 94.3^k \rightarrow \frac{94.3}{17.2} = 5.5 \rightarrow 6 \times 2 = 12$ studs/beam

W12x14, $\Sigma Q_n = 85.3^k \rightarrow \frac{85.3}{17.2} = 4.96 \rightarrow 5 \times 2 = 10$ studs/beam

W10x26, $\Sigma Q_n = 95.1^k \rightarrow \frac{95.1}{17.2} = 5.53 \rightarrow 6 \times 2 = 12$ studs/beam

W10x22, $\Sigma Q_n = 81.1^k \rightarrow \frac{81.1}{17.2} = 4.72 \rightarrow 5 \times 2 = 10$ studs/beam

W10x19, $\Sigma Q_n = 70.3^k \rightarrow \frac{70.3}{17.2} = 4.09 \rightarrow 5 \times 2 = 10$ studs/beam

Check a assumption

$$b_{eff} = \min \begin{cases} \frac{span}{8} \times 2 = 2\left(\frac{24(12)}{8}\right) = \underline{72"} \\ \frac{spacing}{2} \times 2 = 2\left(\frac{9(12)}{2}\right) = 108" \end{cases}$$

$$a = \frac{95.1^k}{0.85(4)(72)} = 0.39 < 1"$$

∴ $Y2 = 4"$ is a conservative value

Check Unbraced Length

$1.4\,DL \rightarrow 1.4(65)(9) + 1.4(26) = \underline{855.4\ plf}$

$1.2DL + 1.6LL \rightarrow 1.2(585+26) + 1.6(61) = 830.8\ plf$

$\therefore$ 1.4 DL load case controls

$$M_u = \frac{0.855\,(24)^2}{8} = 61.56'^k$$

- All options pass, $\Phi M_p > M_u$ ✓

Shape	Steel Weight	Stud Weight	Total Weight	
W12×19	456	100	556	3
W12×16	384	120	504	2
W12×14	336	100	$\boxed{436}$	1
W10×26	624	120	744	
W10×22	528	100	628	
W10×19	456	100	556	4

$\therefore$ Proceed with a W12×14, w/10 studs ; $I_x = 88.6\ in^4$

Check Wet Concrete Deflection

$W_{wc} = 65(9) + 14 = 599 \rightarrow 0.599\ klf$

$$\Delta_{wc} = \frac{5(0.599)(24)^4(1728)}{(384)(29000)(88.6)} = 1.74\ in.$$

$$\Delta_{wc\ max} = \frac{L}{240} = \frac{24(12)}{240} = 1.2'' < 1.74'' \quad \times$$

$\therefore$ Try increasing to a W12×19

Try W12x19;

$$W_{wc} = 65(9) + 19 = 604 \rightarrow 0.604\,klf$$

$$\Delta_{wc} = \frac{5(0.604)(24)^4(1728)}{(384)(29000)(130)} = 1.195''$$

$$\Delta_{wc\,max} = 1.2'' > 1.195'' \quad \checkmark \text{ ok}$$

Live Load Deflection

$$W_{LL} = (61\,psf)(9) = 549 \rightarrow 0.55\,klf$$

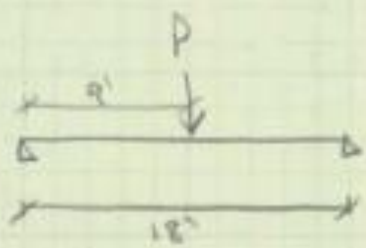
$$\Delta_{LL} = \frac{5(0.55)(24)^4(1728)}{384(29000)(244)} = 0.58''$$

$$\Delta_{LL\,max} = \frac{L}{360} = \frac{(24)(12)}{360} = 0.8'' > 0.58'' \quad \checkmark \text{ ok}$$

∴ Choose W12x19 w/ 10 studs

Girder Design

- span - 18'
- spacing - 24'



$$P_u = 126\,psf(9')(12)(2) = 27.22^k$$

$$M_u = \frac{P\ell}{4} = \frac{27.22(18)}{4} = 122.5'^k$$

If $a = 1''$, $y_2 = 4.25'' \rightarrow \underline{4''}$

Possible Girder Sizes

W14x22, $\Sigma Q_n = 81.1 \rightarrow \frac{81.1}{17.2} = 4.72 \rightarrow 5 \times 2 = 10$ studs

W12x22, $\Sigma Q_n = 81 \rightarrow \frac{81}{17.2} = 4.71 \rightarrow 5 \times 2 = 10$ studs

W12x19, $\Sigma Q_n = 69.6 \rightarrow \frac{69.6}{17.2} = 4.05 \rightarrow 5 \times 2 = 10$ studs

Check a

$$b_{eff} = \min \begin{cases} \frac{18(12)}{8} \times 2 = \underline{54''} \\ \frac{24(12)}{2} \times 2 = 288'' \end{cases}$$

$$a = \frac{81.1}{0.85(4)(54)} = 0.44'' < 1.0''$$

Check Unshored Length

$1.4\,DL \rightarrow 1.4(65)(24) + 1.4(22) = \underline{2214.8\ plf}$

$1.2DL + 1.6LL \rightarrow 1.2[(65)(24) + 22] + 1.6(61) = 1996\ plf$

$$M_u = \frac{2.215(18)^2}{8} = 89.71'k$$

$\therefore$ Try W14x22 w/ 10 studs, $I_x = 199$

Check Wet Concrete Deflection

$w_{wc} = 65(18) + 22 \sim 1192 \rightarrow 1.19$ klf

$$\Delta_{wc} = \frac{5(1.19)(18)^4(1728)}{(384)(29000)(199)} = 0.49\ in$$

$$\Delta_{wc_{max}} = \frac{L}{240} = \frac{18(12)}{240} = 0.9\,in > 0.49\,in \checkmark$$

Check Live Load Deflection

$I_{LB} = 352 \text{ in}^4$

$W_{LL} = 61(18') = 1098 \text{ plf} \rightarrow 1.1 \text{ klf}$

$P_{LL} = 1.1 \text{ klf}(18') = 19.8^k$

$\Delta_{LL} = \dfrac{0.05(19.8)(18)^3(1728)}{29000(352)} = 0.977$

$\Delta_{LL_{max}} = \dfrac{L}{360} = \dfrac{18(12)}{360} = 0.6 < 0.977$ X

$\therefore$ Section fails, $I_{LB} \geq 573.4 \text{ in}^4$

↳ Try W14x34

Check Unbraced Length

$1.4\,DL \rightarrow 1.4(65)(24) + 1.4(34) = \underline{2231.6 \text{ plf}}$

$1.2\,DL + 1.6\,LL \rightarrow 1.2((65)(24)+34) + 1.6(61) = 2010.4 \text{ plf}$

$M_u = \dfrac{2.232(18)^2}{8} = 90.4'^k < 205'^k$ ✓

Check Wet Concrete Deflection

$W_{WC} = 65(18) + 34 = 1204 \rightarrow 1.204 \text{ klf}$

$\Delta_{WC} = \dfrac{5(1.204)(18)^4(1728)}{(384)(29000)(340)} = 0.288$

$\Delta_{WC_{max}} = \dfrac{L}{240} = \dfrac{18(12)}{240} = 0.9 > 0.288 \text{ in}$ ✓

$\Delta_{WC_{max}} > \Delta_{WC}$

Check Live Load Deflection

$I_{LB} = 582\ in^4$

$w_{LL} = 61(18) = 1098\ plf \rightarrow 1.1\ klf$

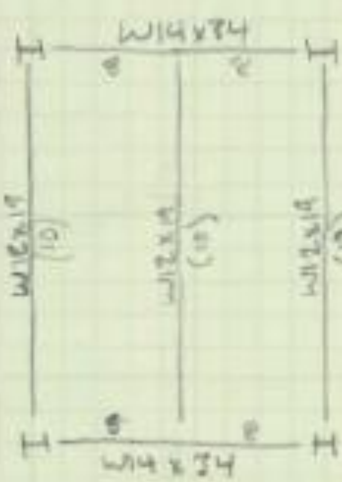
$P_{LL} = (1.1\ klf)(18') = 19.8^k$

$$\Delta_{LL} = \frac{0.05(19.8)(18)^3(1728)}{29000(582)} = 0.591''$$

$$\Delta_{LL_{max}} = \frac{L}{360} = \frac{18(12)}{360} = 0.6'' > 0.591'' \checkmark$$

$$\Delta_{LL_{Max}} > \Delta_{LL}$$

Final Design



Check Gravity Column from RAM:

Interior Column @ D-6 on Story 5

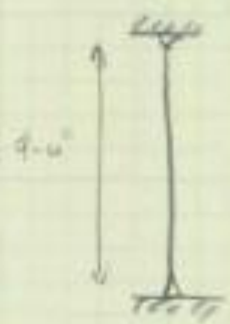
Floor LL = 80 psf
Floor DL = 75 psf
Roof LL = 20 psf

RAM Design: W14x99

$P_D = 390^k$
$P_L = 416^k$
$P_{Lr} = 8^k$

$P_u = 1.2D + 1.6L + 0.5Lr$

$P_u = 1.2(390^k) + 1.6(416^k) + 0.5(8^k) = 1137.6^k$



$l_u = 9.33'$

AISC Specifications Table C-A-7.1...
pinned-pinned => $K = 1.0$

- Both directions are unbraced. $r_x > r_y$ for W-shapes so y-y axis buckling controls

$KL_y = 9.33' \rightarrow$ round to 10'

Table 4-1

$l_u = 10' \longrightarrow \phi P_n = 1210^k > P_u$ ✓

$\frac{P_u}{\phi P_n} = \frac{1137.6}{1210} = 0.94 < 1.0$ - interaction from RAM, 0.95 ✓

Exterior Column @ G-8 on Story 5

RAM design: W12x53

$P_D = 195^k$
$P_L = 208^k$
$P_{LR} = 4^k$

$P_u = 1.2D + 1.6L + 0.5L_R$

$P_u = 1.2(195) + 1.6(208) + 0.5(4) = 568.8^k$

From Table 4-1

$KL_u = 10' \rightarrow \phi P_n = 592^k > P_u$ ✓

$\frac{P_u}{\phi P_n} = \frac{568.8}{590} = 0.964 < 1.0$ ✓ – RAM interaction provided 0.99

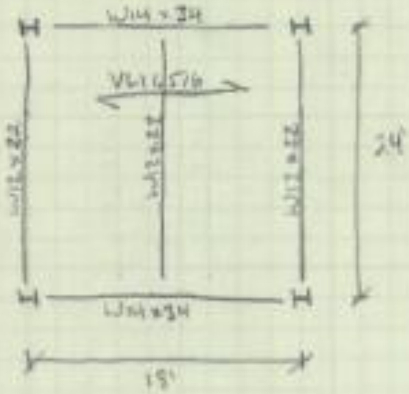
– Both interior and exterior column designs pass axial compression capacity.

Note: These checks are only quick validations that the RAM software is functioning properly and coming up with reasonable designs. RAM's analysis of the columns is much more detailed and considers P-Δ effects. The interaction given from RAM is slightly higher than calculated due to its inclusion of the higher order effects.

Appendix D: Vibrations

- To eliminate any resonate harmonic effects from occupants, a vibrations study will be preformed using AISC Design Guide 11 while using the criteria for walking vibrations.

Typical Interior Bay



Slab:
- $1\frac{1}{2}$" Deck
- $3\frac{1}{4}$" topping, $4\frac{3}{4}$ total
- LW Concrete
- $f'_c = 4000$ psi

- Apartment Occupancy

W14x34	W12x22
$A = 10\ in^2$	$A = 6.48\ in.^2$
$I_x = 340\ in^4$	$I_x = 156\ in.^4$
$d = 14\ in.$	$d = 12.3\ in.$

For W12x26 beams,

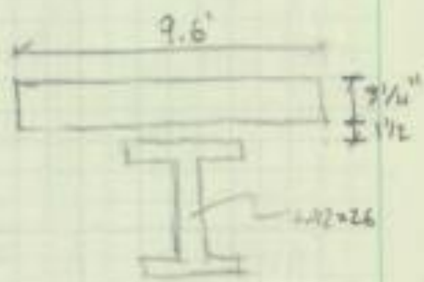
$$b_{eff} = \begin{cases} \text{spacing} = 9' \\ 0.4L_b = 0.4(24) = 9.6' \end{cases} \qquad b_{eff} = 9.6'$$

Modular Ratio:

$$n = \frac{E_s}{1.35\,E_c} = \frac{29{,}000}{1.35(110)^{1.5}\sqrt{4}} = 9.31$$

Find $\bar{y}$ about top of slab

$$\bar{y} = \frac{\left(\frac{115.2}{9.31}\right)(3.25)\left(\frac{3.25}{2}\right) + 6.48(4.75 + 6.15)}{\left(\frac{115.2}{9.31}\right)(3.25) + 6.15}$$



$$\bar{y} = 2.93\ in.$$

Adjusted I,

$$I_{comp} = 156 + \frac{\left(\frac{115.2}{9.31}\right)(3.25)^3}{12} + 6.48\left(4.75 + 6.15 - 2.93\right) + \frac{115.2}{9.31}(3.25)\left(2.93 - \frac{3.25}{2}\right)^2$$

$$I_{comp} = 156 + 35.4 + 51.65 + 68.49$$

$$I_{comp} = 311.54 \text{ in.}^4$$

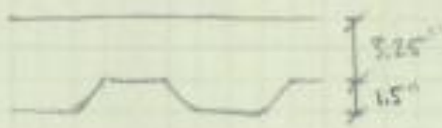
Determine w_{beam}

$$w_{beam} = 9.6(41 + 11 + 4) + 22 = 559.6 \text{ plf}$$

Find Δ_b

$$\Delta_b = \frac{5 w_b L_b^4}{384 E I_b} = \frac{5(0.560)(24)^4(1728)}{384(29000)(311.54)} = 0.463$$

Find effective Depth



$$d_e = 3.25 + \frac{1.5}{2} = 4$$

$$D_s = \frac{12(4)^3}{12(9.31)} = 6.87 \text{ in}^4/\text{ft}$$

$$D_b = \frac{I_b}{S} = \frac{311.54}{9.6} = 32.5 \text{ in}^4/\text{ft}$$

Find participating width

$$B_b = C_b\left(\frac{D_s}{D_b}\right)^{0.25} L_b = 2\left(\frac{6.87}{32.5}\right)^{0.25} \cdot 24 = 32.55'$$

↳ 2.0 for internal panels

$$B_b \leq \frac{2}{3}(3 \times L_g) \longrightarrow 32.6' \leq 2(18) = 36' \checkmark$$

$$W_o = \frac{4}{\pi}\frac{W_s}{S}B_oL_o = \frac{4}{\pi}\left(\frac{560}{9.6}\right)(32.55)(24) = 58{,}022$$

$$W_o = 58.02^k$$

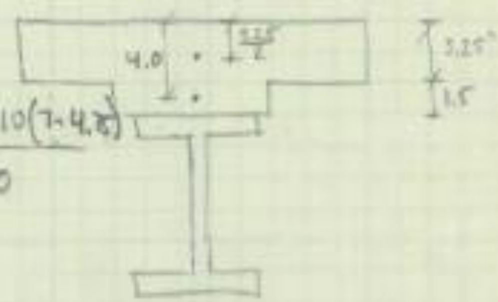
Find beam frequency

$$f_b = 0.18\sqrt{\frac{g}{\Delta_b}} = 0.18\sqrt{\frac{386}{0.463}} = 5.20\text{ Hz.}$$

for W14×34 girder

$$b_{eff} = \begin{cases} 0.4L_g = 0.4(18) = \underline{7.2'} \\ \text{spacing} = 24' \end{cases} \qquad b_{eff} = 7.2'$$

Find $\bar{y}$



$$\bar{y} = \frac{\left(\frac{86.4}{9.31}\right)(3.25)\left(\frac{3.25}{2}\right) + \left(\frac{43.2}{9.31}\right)(1.5)(1.25) + 10(7+4.85)}{\frac{86.4}{9.31}(3.25) + \left(\frac{43.2}{9.31}\right)(1.5) + 10.0}$$

$$\bar{y} = 4.01''$$

Adjusted I

$$I_{comp} = \frac{\left(\frac{86.4}{9.31}\right)(3.25)^3}{12} + \frac{\left(\frac{43.2}{9.31}\right)(1.5)^3}{12} + 340 + \left(\frac{86.4}{9.31}\right)(3.25)(4.01-3.25)^2 +$$
$$\left(\frac{43.2}{9.31}\right)(1.5)(4.01-1.5)^2 + 10\,(7.0+4.75-4.01)^2$$

$$I_{comp} = 26.55 + 1.31 + 340 + 17.42 + 43.85 + 599.08$$

$$I_{comp} = 1028.21\text{ in}^4$$

$$I_g = I_{nc} + (I_{comp} - I_{nc})/4$$
$$= 340 + 0.25(1028 - 340) = 512.1\text{ in}^4$$

Determine Δ_g

$$w_g = \frac{\Sigma w_b}{S} L_b + SW = \frac{560}{9.6}(24) + 34 = 1434 \text{ plf}$$

$$\Delta_g = \frac{5 w_g L_g^4}{384 E I_g} = \frac{5(1434)(18)^4(1728)}{384(29000)(512.1)} = 0.228 \text{ in.}$$

- Because $L_g > B_n$, Δ_g needs no modification

Determine W_g

$$w_g = \frac{1434}{24} = 59.75 \text{ psf}$$

$$D_s = \frac{I_b}{S} = \frac{311.54}{9.6} = 32.5$$

$$D_g = \frac{I_g}{L_b} = \frac{512.1}{24} = 21.34$$

$$B_g = C_g (D_s / D_g)^{.25} L_g = 1.8 \left(\frac{32.5}{21.34}\right)^{.25} \times 18 = 36$$

↳ 1.8 for girder with shear connections

Assume 3 bays → $\frac{2}{3}(66) = 44'$

$$W_g = w_g B_g L_g = (59.75)(44)(18) = 47.32^k$$

Determine W:

$$W = \frac{\Delta_b}{\Delta_b + \Delta_g} W_b + \frac{\Delta_g}{\Delta_b + \Delta_g} W_g$$

$$W = \frac{0.463}{0.463 + 0.228}(58.02) + \frac{0.228}{0.228 + 0.463}(47.32)$$

$$W = 54.5^k$$

Determine f_n

$$f_n = 0.18\sqrt{\frac{g}{\Delta_b + \Delta_g}} = 0.18\sqrt{\frac{386.4}{.228 + .465}}$$

$$f_n = 4.26\ Hz.$$

Evaluation Criteria:

- Assume $P_o = 65$ k.
- Assume $\frac{a_o}{g} = 0.005\,g$
- Assume 5% damping, $\beta = 0.05$

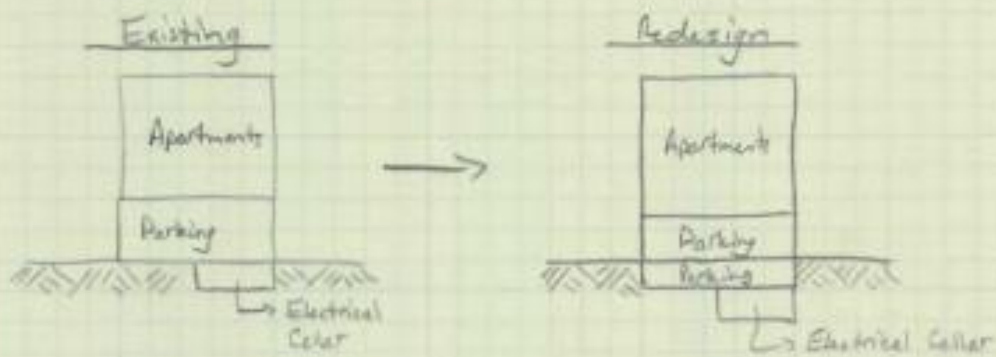
$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \le \frac{a_o}{g} = 0.005$$

$$\frac{65\,e^{(-0.35 \times 4.26)}}{0.05(54500)} = 0.0048 \le 0.005 \quad \text{ok.} \checkmark$$

Appendix E: Foundations

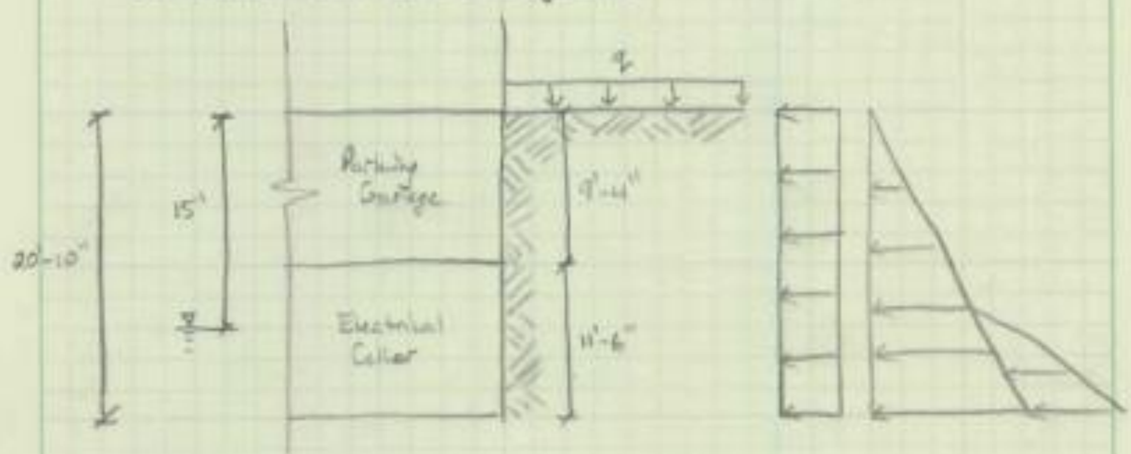
Overturning Moments				
Load Cases	Base Shear X Direction (k)	Base Shear Y Direction (k)	Overturning X Direction (‘ k)	Overturning Y Direction (‘ k)
Wind Case 1 – X Direction	779.74	-	**52,242.58**	-
Wind Case 1 – Y Direction	-	553.52	-	**53,137.92**
Wind Case 2 – X Direction (+M)	584.81	-	39,182.27	-
Wind Case 2 – X Direction (-M)	584.81	-	39,182.27	-
Wind Case 2 – Y Direction (+M)	-	415.14	-	
Wind Case 2 – Y Direction (-M)	-	415.14	-	39,853.44
Wind Case 3	584.81	415.14	39,182.27	39,853.44
Wind Case 4 (Additive +Moments)	438.99	311.63	29,412.33	39,853.44
Wind Case 4 (Additive –Moments)	438.99	311.63	29,412.33	29,916.48
Wind Case 4 (+M’s in Opposite Directions)	438.99	311.63	29,412.33	29,916.48
Wind Case 4 (-M’s in Opposite Directions)	438.99	311.63	29,412.33	29,916.48
Seismic X	441.42	-	29,575.14	-
Seismic Y	-	441.42	-	42,376.32

- The existing design has a small electrical cellar below grade.
- The foundation system consists of spread footings and wall footings with one large area of MAT Foundation.
- The proposed redesign will be to move one layer of parking garage below grade and relocate the electric cellar beneath it.

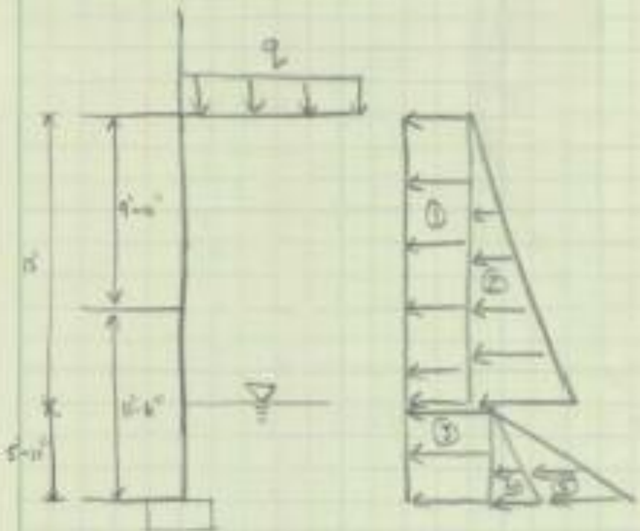


Geotech Report:
- groundwater level is 20 ft. below existing grade.
- recommend equivalent fluid pressure of 50 psf/ft.
- due to fluctuating water levels, assume hydrostatic pressure 15 ft. below surface

Foundation Walls to be designed...



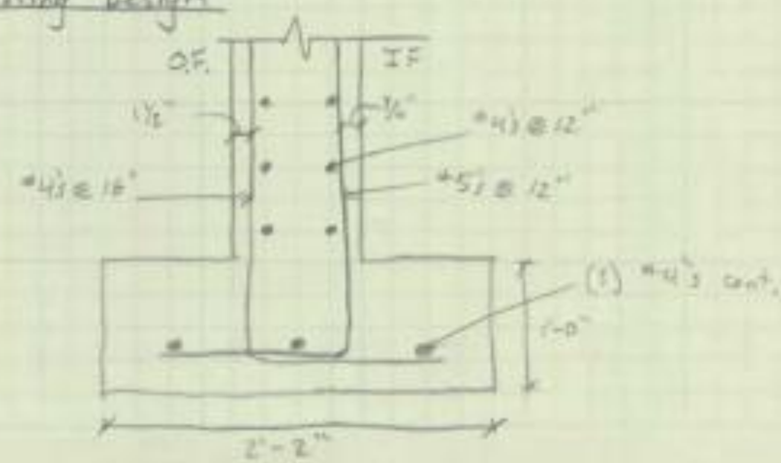
Load Determination



q = 100 psf
EFP = 50 pcf
K_o = 0.5

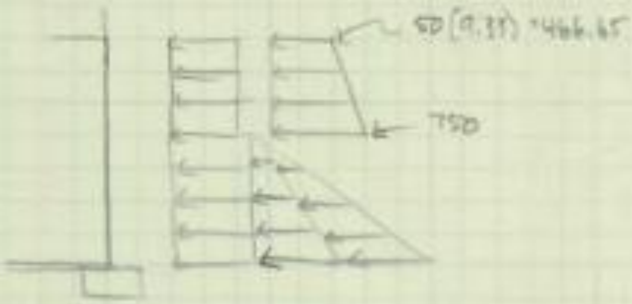
	Max Stress	Force
1	0.5(100psf) = 50psf	50psf (15') = 750 plf @ 13'-4"
2	50(15) = 750psf	$\frac{1}{2}$(50)(15')^2 = 5,625plf @ 10'-10"
3	42 + 750 = 792psf	792(5.83) = 4,617.4plf @ 2'-11"
4	0.5(100 - 62.4)(5.83) = 110	$\frac{1}{2}$(110)(5.83) = 320.65plf @ 1'-11"
5	62.4(5.83) = 364	$\frac{1}{2}$(364)(5.83) = 1061 plf @ 1'-11"

Existing Design

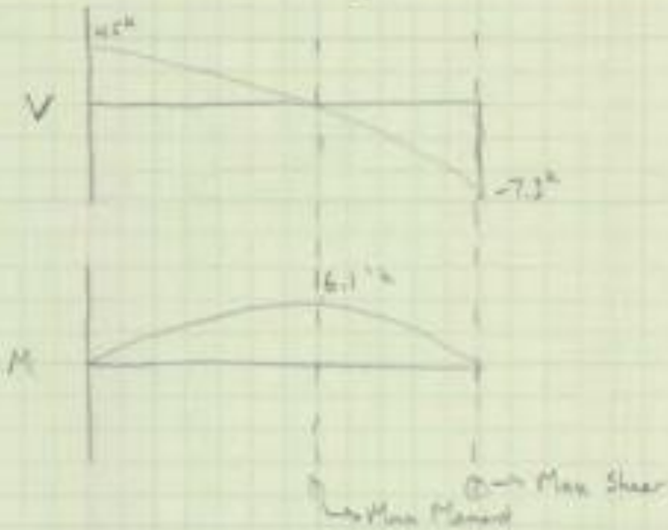


- Check existing design on loading of new foundation wall.

Wall of Electrical Cellar:



- Loads were obtained using Risa 2D, see attached print out



Design Loads:

$V_u = 1.6(7.3) = 11.7^k$

$M_u = 1.6(16.1') = 25.8'^k$

* 1.6 multiplier for soil factored load

Check Foundation Wall

$V_u = 11.7^k$
$M_u = 25.8^k$

$11{,}700 \leq 0.75(2)\sqrt{4000}\,(12)(10 - \tfrac{5}{8} - .75)$

$11{,}700 \not\leq 9820$

∴ No Good → increase to 12" wall, $d = 10.625"$

$11{,}700 \leq 0.75(2)\sqrt{4000}\,(12)(10.625)$

$11{,}700 \leq 12096$ lbs

$V_u \leq \phi V_n$ ✓ ∴ OK, proceed to Flexure

$(25.8^k)(12) = 0.9\,A_s(60)\left(10.625 - \frac{1.96\,A_s}{2}\right)$

$5.733 = 10.625\,A_s - 0.98\,A_s^2$

$A_s \geq 0.569\ in^2/ft$

$A_{s,prov} = 0.31\ in^2$ (#5 @ 12")

$A_s > A_{s,prov}$ ✗ ∴ No good, decrease spacing to #5's @ 6"

∴ Use #5 @ 6" o.c. for I.F. and O.F.

$\rho_{min,vert} = 0.0012$ $\qquad 0.0012 = \frac{A_s}{12(12)}$

$A_s \geq 0.1728\ in^2$

$A_{s,prov} = 0.62\ in^2 > 0.173\ in^2$ ✓

∴ Vert. Reinf. Good

Check Horiz. Reinforcing...

$\rho_{min,horiz} = 0.002$ $\qquad 0.002 = \frac{A_s}{12(12)}$

$A_s \geq 0.288\ in^2$

$A_{s,prov} = 0.2\ in^2$ (#4's @ 12")

$A_s > A_{s,prov}$ X No good, try #5's @ 12"

#5's @ 12" → $A_{s,prov} = 0.31\ in^2 > A_s = 0.288\ in^2$

∴ Use #5's @ 12"

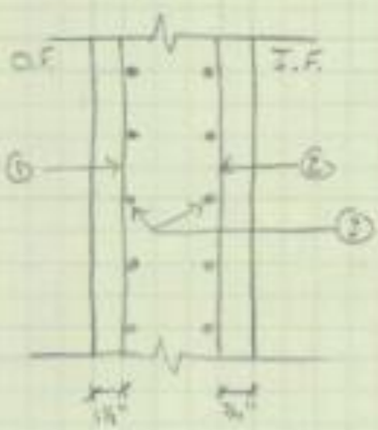
<u>Ductility Check</u>

$a = 1.96(0.31) = 0.608 \longrightarrow c = \frac{0.608}{0.85} = 0.715$

$\varepsilon_t = \frac{0.003}{0.715}(10.625 - 0.715) = 0.0416 \geq 0.005$ ✓

∴ $\phi = 0.9$ ok

<u>Foundation Wall Design</u>



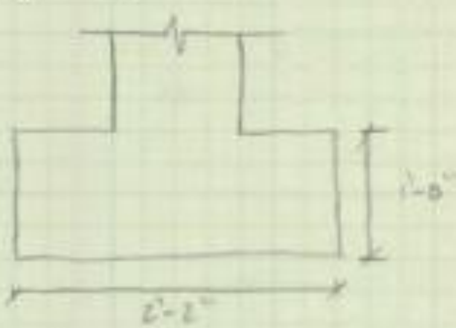
① #5's @ 6" o.c. O.F
② #5's @ 6" o.c. I.F
③ #5's @ 12" o.c. E.F.

Check Wall Footing

- Sub-grade floors are not load bearing, therefore, the footing need only support the wall self weight

- Geotech reports 6,000 psf for allowable bearing pressure for wall footings.

Existing Design



$$q_a \geq q_u = \frac{P}{A}$$

$$q_u = \frac{(150)(1)(1)(20.83)}{1(2.1667)}$$

$q_u = 1{,}442$ psf   assume $FS = 3$

$q_a = 6000\ \text{psf} \geq 3(1442) = 4326\ \text{psf}$ ✓

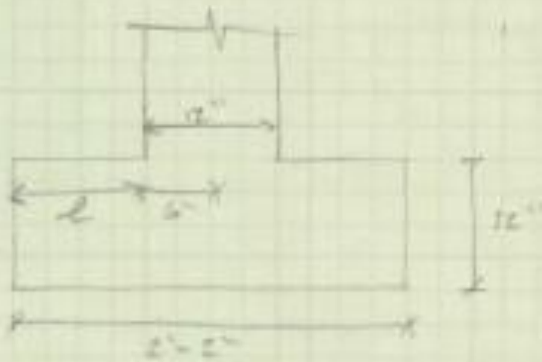
∴ Soil OK for Bearing

Check Concrete

$P = 3.13$ klf

$q_A = 6000$ psf

$FS = 3$



$q_A \geq \frac{3,130}{2.167(1)}$

$6000 \geq 1444$ psf ✓

$\ell = \frac{2.167 - 0.5}{2} = 0.833$ ft.

$P_u = 1.2(3.13) = 3.76^k$ $q = \frac{3.76}{3} = 1.25$

$M_u = \frac{3.76(0.833)^2}{2} = 1.3'^k$

Shear

$\ell_{crit} = \frac{26" - 12"}{2} = 7"$

$\phi V_n \leq \phi\, 4/3 \sqrt{f'_c} \cdot b \cdot h$

$1.25(1)(1000) \leq 0.55(4/3)\sqrt{4000}\,(12)(12)$

$1250 \leq 6,680$ lbs. ✓

Flexure

$12" - 3" - 1.5(0.5) = 8.25"$

$1.3'^k(12) = 0.9 A_s\, 60\left(8.25" - \frac{1.96 A_s}{2}\right)$

$0.289 = 8.25 A_s - 0.98 A_s^2$

$A_s = 0.035\ in^2$

$A_{s,prov} = \#4\text{'s} @ 12"$, $A_{s,prov} = 0.2\ in^2$

$A_{s,prov} > A_s$ ✓

∴ Existing Wall Footing ok.

Column Footing Design

- Due to the reduced structural weight (due to the concrete to steel change), it is anticipated that most footings can be reduced in size.

- Column E-6 will be used for design. A complete column footing design for the extents of the building will be performed in RAM

Footing Loading

Column Tributary Area = $420\ ft^2$

4 Floors of Parking Garage:

Dead: $4\left(\left(420 \times 130\ psf\right) + \left(150 \cdot 3 \cdot 10\right)\right) = 236.4^k$

Live: $4\left(420 \cdot 50\right) = 84^k$

12 Floors Apartments

Dead: $12\left(420 \cdot 70\right) = 352.8^k$

Live: $12\left(420 \cdot 60\right) = 302.4^k$

$P_D = 589.2^k$

$P_L = 386.4$

$P = P_D + P_L = 589.2 + 386.4 = 975.6^k$

- Force loading the column, $P = 975.6^k$

Footing Design

- Geotech reports states an allowable soil bearing pressure of 8000 psf for column footings.

$$q_a \geq \frac{P}{A}$$

$$8\text{ksf} \geq \frac{975.6}{B^2}$$

$$B^2 \geq \frac{975.6}{8} \rightarrow B = 11.04$$

$\therefore$ Try a $12' \times 12'$ footing

$$P_u = 1.2(529.2) + 1.6(386.4)$$
$$P_u = 1325.3^k$$

$$q_u = \frac{P_u}{12^2} = \frac{1325.3}{144} = 9.2\text{ ksf} \rightarrow 63.9\text{psi}$$

$$q_u = 63.9\text{ psi}$$

One Way Shear

$$V_c = \phi 4\sqrt{f'_c}$$
$$= 0.75(4)\sqrt{4000}$$
$$= 190\text{ psi}$$

- Two way shear controls by inspection

Two Way Shear

$$d^2\left(v_c + \frac{q}{4}\right) + d\left(v_c + \frac{q}{2}\right)w = \frac{q}{4}\left(BL - w^2\right)$$

$$d^2\left(190 + \frac{63.9}{4}\right) + d\left(190 + \frac{63.9}{4}\right)18'' = \frac{63.9}{4}\left((144)^2 - (18)^2\right)$$

$$205.98d^2 + 3707.6d = 326{,}081.7$$

$$d \geq 31.8''$$

$$h = 31.8 + 3 + 0.625$$

$$h = 35.425'' \longrightarrow \text{Use } h = 36''$$

Proceed w/ h = 36 in.

$$d = 36 - 3 - 0.625 = 32.375$$

$$\ell = \frac{12' - 1.5'}{2} = 5.25'$$

$$M_u = \frac{9.2\text{ ksf}\,(5.25)^2}{2}$$

$$= 126.8'^{k} \longrightarrow \text{Find } a \ldots$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{60 A_s}{0.85(4)(12)} = 1.47 A_s$$

$$M_u = \phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right)$$

$$126.8'^{k}(12) = 0.9 A_s 60\left(32.375 - \frac{1.47 A_s}{2}\right)$$

$$28.178 = 32.375 A_s - 0.735 A_s^2$$

$$A_s = 0.89 \text{ in}^2$$

$\therefore$ Use #9 @ 12" o.c. $A_s = 1.0 \text{ in}^2$

$$\rho = \frac{A_s}{bh} = \frac{1.0}{12(36)} = 0.0023 \geq 0.0018 \quad \checkmark$$

$$a = 1.47 A_s = 1.47(1) = 1.47''$$

$$c = \frac{a}{0.85} = \frac{1.47}{0.85} = 1.73''$$

$$\varepsilon_s = \frac{0.003}{1.73}(32.375 - 1.73) = 0.053 \geq 0.005 \text{ in/in} \quad \checkmark$$

$$\therefore \phi = 0.9 \quad \text{ok}$$

<u>Final Design:</u>

12' × 12' × 3' Footing w/ (12) #9's each way

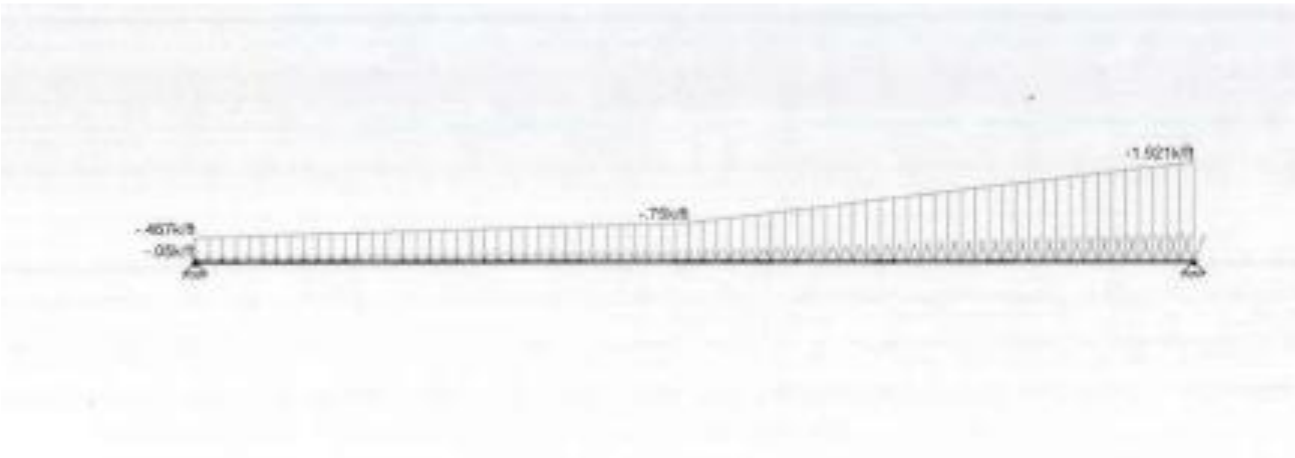
<u>Design Comparison</u>

	Existing	Redesign
Area	13' × 13'	12' × 12'
Depth	3'-8"	3'-0"
Reinf.	(12) #10's	(11) #9's
Vol. of Concrete	620 ft³	432 ft³
Weight of Steel	1348.3 lb.	898.3 lb.

<u>Comments:</u>

- 30.3% decrease in concrete used
- 33.4% decrease in weight of reinforcing

Risa 2D was used to model the loading on the foundation walls due to surcharge as well as soil and water lateral load. These forces were verified by hand and used in the design of the foundation walls.



Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	0	4.507	0
	2	0	2.815	10.623
	3	0	.709	15.788
	4	0	-2.469	13.655
	5	0	-7.306	0

Figure 52: Foundation Wall Loading

Appendix F:Wind and Seismic Loads
Building Properties

When buildings are exposed to lateral loads the act through different point of the building depending on the nature of the load. Wind and seismic forces interact with the building differently because wind is a pressure force whereas seismic force is a function of mass. The tables below will located the point at which these forces act through.

Center of Mass:

The center of mass represents the mean position of the mass located in a building or on a floor. The center of mass is the location in which external loads and moments on a building act through. The seismic forces on a building act through the center of mass.

Center of Mass by Floor						
	ETABS		Calculated by Hand		Error	
Floor	X Direction	Y Direction	X Direction	Y Direction	X	Y
17	70.02	107.89	78.15	90.32	11.60%	16.29%
16	67.67	88.36	78.15	90.32	15.49%	2.21%
15	71.25	90.95	78.15	90.32	9.67%	0.66%
14	71.23	90.95	78.15	90.32	9.67%	0.66%
13	71.23	90.95	78.15	90.32	9.67%	0.66%
12	71.23	90.95	78.15	90.32	9.67%	0.66%
11	71.23	90.95	78.15	90.32	9.67%	0.66%
10	71.23	90.95	78.15	90.32	9.67%	0.66%
9	71.23	90.95	78.15	90.32	9.67%	0.66%
8	71.23	90.95	78.15	90.32	9.67%	0.66%
7	71.23	90.95	78.15	90.32	9.67%	0.66%
6	71.23	90.95	78.15	90.32	9.67%	0.66%
5	65.92	94.71	78.15	90.32	18.55%	4.64%
4	66.79	93.74	84.90	96.24	27.11%	2.67%
3	66.85	94.18	79.46	96.24	14.38%	2.19%
2	57.68	120.03	79.46	96.24	37.76%	19.83%
1	64.71	103.18	79.46	96.24	22.79%	6.73%

The detailed spreadsheet containing the calculated values are provided in the appendix. One discrepancy in the results is that ETABS included the slab in the COM calculation whereas the hand spot checks just included the shear walls. The footprint of the floor plan changes on the bottom 4 floors and the top two floors while the shear wall configurations do not change. Therefore larger error is expected on those floors due to that. The slab in the X direction steps back a bay above floor 4 which accounts for some of the variability in that direction. The Y direction mass distribution is fairly consistent throughout the building height, which is reflected by the low margin of error for those calculations.

Center of Rigidity:

The center of rigidity is the centroid of the stiffness for a building or individual floor. The stiffness elements considered for the center of rigidity are the shear walls and drop beams previously mentioned in this report. Forces that act through any point other than the COR cause an incidental torsion on the building because the load is applied eccentrically to the centroid of stiffness. Because 8621 Georgia Avenue is a rectangular building with relatively well distributed lateral force resisting elements, it is expected that the COR and COM points will not differ greatly. Therefore, the accidental torsion on the building should be minimal.

Center of Rigidity by Floor						
	ETABS		Calculated by Hand		Error	
Floor	X Direction	Y Direction	X Direction	Y Direction	X	Y
17	90.867	98.442	82.368	93.04	9.35%	5.49%
16	90.192	98.960	83.369	92.481	7.56%	6.55%
15	89.562	99.562	83.271	92.543	7.02%	7.05%
14	88.907	100.04	82.368	93.04	7.35%	6.99%
13	88.344	100.331	82.368	93.04	6.76%	7.27%
12	87.095	100.590	82.368	93.04	5.43%	7.51%
11	87.742	100.525	82.368	93.04	6.12%	7.45%
10	86.410	100.47	82.368	93.04	4.68%	7.40%
9	85.706	100.072	82.368	93.04	3.89%	7.03%
8	85.030	99.239	82.368	93.04	3.13%	6.25%
7	84.486	97.701	82.368	93.04	2.51%	4.77%
6	84.320	95.004	82.368	93.04	2.31%	2.07%
5	85.046	90.579	82.368	93.04	3.15%	2.72%
4	85.864	86.273	78.807	85.959	8.22%	0.36%
3	85.385	86.316	78.504	86.468	8.06%	0.18%
2	84.938	90.165	78.504	86.468	7.57%	4.10%
1	90.227	89.499	78.654	86.222	12.83%	3.66%

The detailed spreadsheet and calculations associated with this table is located in the appendix.

Center of Pressure:

The lateral wind forces applied to a building are pressure loads on the façade that we simplify to story forces based on the exposed surface area the pressure is acting on. Because the wind force is dependent on the geometric exposure of the building, the resultant force acts through the centroid of that area. Therefore the wind forces will act through these points, which are called the Center of Pressure.

Center of Pressure by Floor		
Floor	**X Direction**	**Y Direction**
17	67.16	108.35
16	67.16	91.90
15	67.16	91.90
14	67.16	91.90
13	67.16	91.90
12	67.16	91.90
11	67.16	91.90
10	67.16	91.90
9	67.16	91.90
8	67.16	91.90
7	67.16	91.90
6	67.16	91.90
5	67.16	95.96
4	67.16	95.96
3	67.16	95.96
2	59.23	116.12
1	59.23	104.17

The Table below is an example of the hand checks performed to verify the lateral forces applied to the model by ETABS. The building mass was approximated based on the structural weight of the building. The seismic story forces on the building are directly proportional to the weight of the building calculated below.

		Columns						Shear Walls				Floors					
		16"x24"			18"x24"			12		14		8		7.25			
	Floor ht.	#	Area (ft2)	Force (k)	#	Area (ft2)	Force (k)	Length (ft.)	Force (k)	Length (ft.)	Force (k)	Area (ft2)	Force (k)	Area (ft2)	Force (k)	Total Story Weight (k)	Lateral Seismic Story Force (k)
Floor 17	9.33	48	2.67	179.20	0	3	0	262.39	367.34	46.66	76.21	0	0	784.00	71.05	693.81	6.94
Floor 16	12.67	48	2.67	243.20	0	3	0	262.39	498.54	46.66	103.43	0	0	17008.00	1541.35	2386.52	23.87
Floor 15	12.33	61	2.67	300.93	0	3	0	262.39	485.42	46.66	100.71	0	0	21479.00	1946.53	2833.59	28.34
Floor 14	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 13	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 12	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 11	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 10	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 9	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 8	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 7	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 6	9.33	61	2.67	227.73	0	3	0	262.39	367.34	46.66	76.21	0	0	21479.00	1946.53	2617.82	26.18
Floor 5	9.33	60	2.67	224.00	0	3	0	262.39	367.34	46.66	76.21	21479.00	2147.90	0	0	2815.45	28.15
Floor 4	11.00	0	2.67	0	74	3	366.30	274.57	453.04	36.66	70.57	25136.00	2513.60	0	0	3403.51	34.04
Floor 3	9.33	0	2.67	0	52	3	218.40	274.57	384.40	36.66	59.88	25136.00	2513.60	0	0	3176.27	31.76
Floor 2	9.33	0	2.67	0	62	3	260.40	274.57	384.40	36.66	59.88	16746.00	1674.60	0	0	2379.27	23.79
Floor 1	10.17	0	2.67	0	66	3	301.95	274.57	418.72	36.66	65.22	21076.00	2107.60	0	0	2893.50	28.93
																Base Shear	441.42

Appendix G: Lateral Checks

Floor Height	Floor to Floor	Wall Length	Windward Pressure	Leeward Pressure	Trib Area	Force	.563*F	0.75*F	B	e	M
10	10.167	175.5	13.1	-4.392	1784.31	31.21	17.57	23.41	175.50	26.33	616.22
19.5	9.333	192	13.8	-4.644	1791.94	33.05	18.61	24.79	192.00	28.80	713.89
29	9.333	192	15	-5.058	1791.94	35.94	20.24	26.96	192.00	28.80	776.36
40	11	192	16.1	-5.4	2112.00	45.41	25.56	34.06	192.00	28.80	980.81
49	9.333	192	16.8	-5.634	1791.94	40.20	22.63	30.15	192.00	28.80	868.33
59	9.333	192	17.4	-5.868	1791.94	41.69	23.47	31.27	192.00	28.80	900.61
68	9.333	192	18	-6.048	1791.94	43.09	24.26	32.32	192.00	28.80	930.80
77	9.333	192	18.4	-6.21	1791.94	44.10	24.83	33.07	192.00	28.80	952.55
87	9.333	192	18.9	-6.372	1791.94	45.29	25.50	33.96	192.00	28.80	978.17
96	9.333	192	19.3	-6.498	1791.94	46.23	26.03	34.67	192.00	28.80	998.53
105	9.333	192	19.7	-6.624	1791.94	47.17	26.56	35.38	192.00	28.80	1018.89
115	9.333	192	20.1	-6.75	1791.94	48.11	27.09	36.09	192.00	28.80	1039.25
124	9.333	192	20.4	-6.858	1791.94	48.84	27.50	36.63	192.00	28.80	1055.04
133	9.333	192	20.7	-6.966	1791.94	49.58	27.91	37.18	192.00	28.80	1070.84
146	12.333	192	21.1	-7.092	2367.94	66.76	37.58	50.07	192.00	28.80	1441.95
158	12.667	192	21.5	-7.218	2432.06	69.84	39.32	52.38	192.00	28.80	1508.63
161	9.333	160.5	21.6	-7.254	1497.95	43.22	24.33	32.42	160.50	24.08	780.42
					Base Shear	779.74	438.99	584.81			

Floor Height	Floor to Floor	Wall Length	Windward Pressure	Leeward Pressure	Trib Area	Force	.563*F	0.75*F	B	e	M
10	10.167	134.33	13.10	-4.39	1365.73	23.89	13.45	17.92	134.33	20.15	361.02
19.5	9.333	134.33	13.80	-4.64	1253.70	23.12	13.02	17.34	134.33	20.15	349.44
29	9.333	134.33	15.00	-5.06	1253.70	25.15	14.16	18.86	134.33	20.15	380.02
40	11	134.33	16.10	-5.40	1477.63	31.77	17.89	23.83	134.33	20.15	480.10
49	9.333	134.33	16.80	-5.63	1253.70	28.13	15.83	21.09	134.33	20.15	425.04
59	9.333	134.33	17.40	-5.87	1253.70	29.17	16.42	21.88	134.33	20.15	440.84
68	9.333	134.33	18.00	-6.05	1253.70	30.15	16.97	22.61	134.33	20.15	455.62
77	9.333	134.33	18.40	-6.21	1253.70	30.85	17.37	23.14	134.33	20.15	466.26
87	9.333	134.33	18.90	-6.37	1253.70	31.68	17.84	23.76	134.33	20.15	478.81
96	9.333	134.33	19.30	-6.50	1253.70	32.34	18.21	24.26	134.33	20.15	488.77
105	9.333	134.33	19.70	-6.62	1253.70	33.00	18.58	24.75	134.33	20.15	498.74
115	9.333	134.33	20.10	-6.75	1253.70	33.66	18.95	25.25	134.33	20.15	508.70
124	9.333	134.33	20.40	-6.86	1253.70	34.17	19.24	25.63	134.33	20.15	516.43
133	9.333	134.33	20.70	-6.97	1253.70	34.68	19.53	26.01	134.33	20.15	524.16
146	12.333	134.33	21.10	-7.09	1656.69	46.71	26.30	35.03	134.33	20.15	705.82
158	12.667	134.33	21.50	-7.22	1701.56	48.87	27.51	36.65	134.33	20.15	738.46
161	9.333	134.33	21.60	-7.25	1253.70	36.17	20.37	27.13	134.33	20.15	546.67
					Base Shear	553.52	311.63	415.14			

Shear Wall Strength Checks

① Find Controlling Load Case (ASCE 7-10, Section 2.3.2)

② 1.2D + 1.6L + 0.5Lr

④ 1.2D + 1.0W + L + 0.5S

⑥ 0.9D + 1.0W

⑦ 0.9D + 1.0E

→ Because Wind controls over seismic and creates greater story forces. Therefore case 4 and 6 will produce the critical horizontal load on a shear wall. Between the two of them, case 4 will create the greatest vertical force. Case 6 is primarily there to check against uplift. Due to the weight and nature of the structure I do not think uplift is a concern.

That being said, Case 4 will be used to check the strength of the shear walls.

The Shear Walls to be evaluated will be SW2 and SW8. Both walls will be analyzed at the base.

Shear Wall #2

Location = C5-C6
Height = 9'-4"
Length = 12'-0"
thickness = 14"

Shear Wall #8

Location = G6-J6
Height = 9'-4"
Length = 24'-0"
thickness = 12"

Shear Wall #2

Determine Dead Load

Level 17 = $[(98\,psf)(299.75\,ft^2) + (12)(14/12)(9.33)(150)]/1000 = 48.97^k$
Level 16 =
Level 15 =
Level 14 =
Level 13 = Levels 5-16 are the same as 17 $= 48.97^k$
Level 12 =
Level 11 =
Level 10 =
Level 9 =
Level 8 =
Level 7 =
Level 6 =
Level 5 = $[(130\,psf)(299.75\,ft^2) + (12)(14/12)(9.33)(150)]/1000 = 58.56^k$
Level 4 = $[(130\,psf)(228.62\,ft^2) + (12)(14/12)(9.33)(150)]/1000 = 49.31^k$
Level 3 = $[(130\,psf)(190.75\,ft^2) + (12)(14/12)(9.33)(150)]/1000 = 44.39^k$
Level 2 = Level 2 is the same as Level 3 $= 44.39^k$

Total $P = 12(48.97) + 58.56 + 49.31 + 2(44.39)$
$P = 784.29^k$

Determine Wind Load

- Wind Load found in Case 1 of Wind Loading Analysis

$W = 779.74^k$

Determine Live Load

Level 5-17 = $(50\,psf)(299.75\,ft^2)/1000 = 14.99^k$
Level 4 = $(50\,psf)(228.62\,ft^2)/1000 = 11.43^k$
Level 2-3 = $(50\,psf)(190.75\,ft^2)/1000 = 9.58^k$

$L = 13(14.99^k) + 11.43 + 2(9.58)$
$L = 225.46^k$

Determine Snow Load

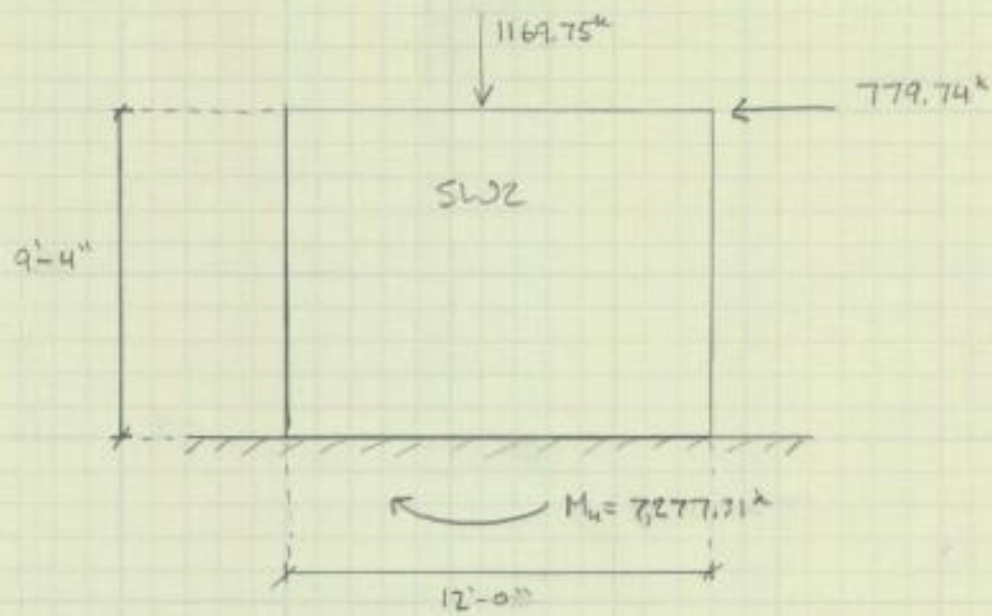
$S = (21\,psf)(299.75\,ft^2)/1000 = 6.29^k$

$S = 6.29^k$

Loading

$$P_u = 1.2(784.29) + 225.46 + 0.5(6.29) = 1169.75^k$$

$$V_u = 1.0(779.74) = 779.74^k$$



Check Strength of SW2

$$\phi V_n \geq V_u = 779.74^k$$

Determine V_c

Properties:

$\lambda = 1.0$

$f'_c = 5000$ psi

$h = 14"$

$d = 0.8\,l_w = 0.8(12) = 9.6' \times 12 = 115.2"$

$N_u = 7277.31^k$

$$V_c = 2\sqrt{5000}\,(14)(115.2)\left(\tfrac{1}{1000}\right)$$
$$= 228.1^k$$

$$V_c = \left(3.3\sqrt{5000}\,(14)(115.2) + \frac{7277.31(115.2)}{4 \cdot 144}\right)\left(\frac{1}{1000}\right)$$
$$= 337.8^k$$

$$V_c = \left[0.6\sqrt{5000} + \frac{144\left(1.25\sqrt{5000} + \frac{0.2(7277.31)}{14(144)}\right)}{\frac{12 \times 9.53\,V_u}{V_u} - \frac{144}{2}}\right]\frac{(14)(115.2)}{1000}$$

$$= \left[42.43 + \frac{12831.9}{39.96}\right] \times 1.613 = 586.4$$

$$V_c = \min\begin{cases} 377.8 \\ 586.4 \end{cases} \rightarrow V_c = 337.8^k > 228.1$$

$\therefore$ Use $V_c = 337.8^k$

$$\phi V_c = 0.75(337.8) = 253.4^k < 779.74^k$$

$\therefore$ Wall w/o reinforcement is no good... find V_s.

Determine V_s

$$V_s = \frac{A_v f_y d}{s}$$

Reinforcement: #5 @ 12" each way, each face
$f_y = 60^{ksi}$
$d = 115.2"$
$s = 12"$

$A_v = 2(2)(.31) = 1.24\,in^2/12\,in$

$\underbrace{2(2)}_{\text{2 rows, each face}}$

$$V_s = \frac{1.24\,in^2 (60\,ksi)(115.2)}{12"}$$

$= 714.24^K$

$V_n = V_c + V_s = 337.8 + 714.24 = 1052.0^k$

$\Phi V_n = 0.75(1052) = 789.0^k > V_u = 779.74^k$ ✓

∴ SW2 is adequate for Strength

Shear Wall #8

Determine Dead Load

Level 17 $= [(120\ psf)(936\ ft^2) + (28)(\frac{12}{12})(9.33)(150)]/1000 = 160.87^k$
Level 15-16 $= [(98\ psf)(576\ ft^2) + (28)(\frac{12}{12})(12.5)(150)]/1000 = 108.95^k$
Level 5-14 $= [(98\ psf)(576\ ft^2) + (28)(\frac{12}{12})(9.33)(150)]/1000 = 95.63^k$
Level 4 $= [(98\ psf)(576\ ft^2) + (28)(\frac{12}{12})(11)(150)]/1000 = 102.65^k$
Level 3 $= [(98\ psf)(576\ ft^2) + (28)(\frac{12}{12})(9.33)(150)]/1000 = 95.63^k$
Level 2 $= [(28)(\frac{12}{12})(9.33)(150)]/1000 = 39.2^k$

Total $P = 160.87 + 2(108.95) + 10(95.63) + 102.65 + 95.67 + 39.2$

$P = 1572.55^k$

Determine Wind Load

- Wind load found in Case 1 of Wind loading analysis

$W = 553.52^k$

Determine Live Load

Level 17 $= (50\ psf)(936\ ft^2)/1000 = 46.8^k$
Level 3-16 $= (50\ psf)(576\ ft^2)/1000 = 28.8^k$

$L = 46.8^k + 15(28.8)$

$L = 478.8^k$

Determine Snow Load

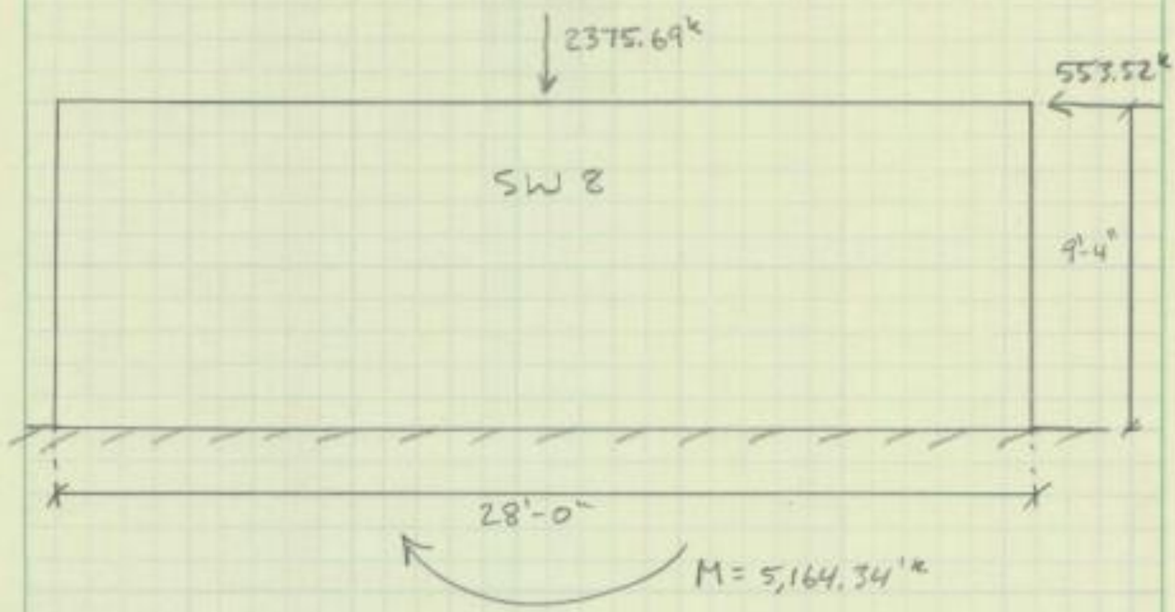
$S = (21\ psf)(936\ ft^2)/1000 = 19.66^k$

$S = 19.66^k$

Loading

$P_u = 1.2(1572.55) + 478.8 + 0.5(19.66) = 2375.69^k$

$V_u = 1.0(553.52) = 553.52^k$



Check Strength of SW8

$\phi V_n \geq V_u = 553.52^k$

Determine V_c

Properties:

$\lambda = 1.0$

$f'_c = 5000$ psi

$h = 12''$

$d = 0.8 l_w = 0.8(28)(12) = 268.8''$

$N_u = 5164.34^k$

$$V_c = 2\sqrt{5000}(12)(268.8)\left(\frac{1}{1000}\right)$$
$$= 456.17^k$$

$$V_c = \left[3.3\sqrt{5000}(12)(268.8) + \frac{5764.34(268.8)}{4(12)(28)}\right]\left(\frac{1}{1000}\right)$$
$$= 753.7^k$$

$$\phi V_c = \phi V_n = 0.75(753.7) = 565.28^k > V_u = 553.52^k$$

∴ SW8 is adequate for strength. The strength check shows that the shear wall is adequate w/o reinforcing. But reinforcing is most likely required for flexure. A shear wall spot check for flexure is beyond the scope of this report but would be needed to determine if the wall really didn't need reinforcing.

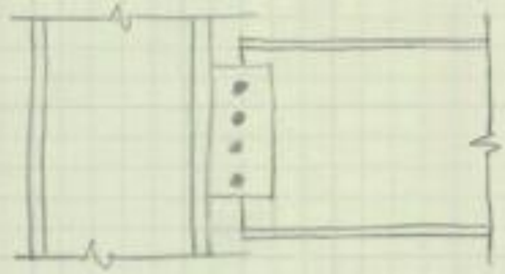
Appendix H: Connection Design

Shear and Moment Connection Design

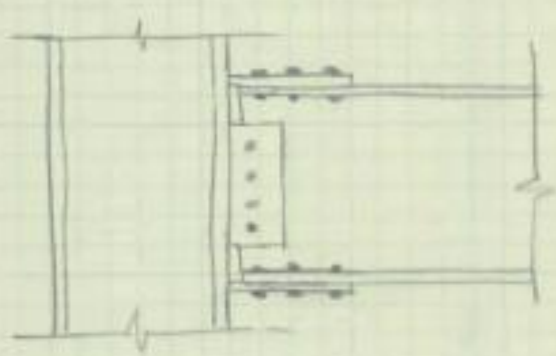
- Upon redesign of the building from concrete to steel, the beam to column connections require much more detail and design.

- The two most common connections for the building will be designed. The shear connections will utilize a shear tab (single plate) connection due to the cost efficiency and ease of construction. The moment connections will be designed as Flange Bolted/web bolted.

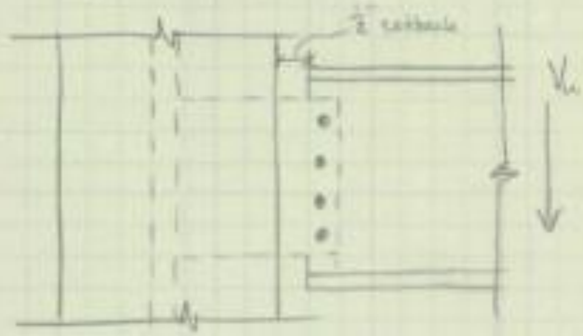
Shear Tab



Flange Bolted/Web Bolted



Shear Tab Design



Design Parameters

- $V_u = 15^k$ ← max end reaction of all beams w/ shear connections
- Beam: W14x34 A992 Steel
- Column: W14x90, A992 Steel
- Plate A36
- Bolts: 3/4" ϕ, A325N

Properties

W14x34
$d = 14.0$ in.
$t_w = 0.285$ in.
$b_f = 6.75$ in.
$t_f = 0.455$ in.

W14x90
$d = 14.0$ in.
$t_w = 0.440$ in.
$b_f = 14.5$ in.
$t_f = 0.71$ in.

$$e = \left(\frac{14}{2} - \frac{0.71}{2}\right) + \frac{1}{2} + 1\tfrac{1}{2} = 8.65\text{ in.}$$

of Bolts

$$C_{req} = \frac{15^k}{17.9^k} = 0.838$$

Table 7-6

$s = 3$ in
$e_x = 9$ in
} in order for $C > C_{req}$, there must be 4 bolts

∴ Use 4 bolts w/ 5/16" plate, $C = 1.21$

Bolt Group Strength

$\phi r_{n_{bolt}} = 17.9^k$

$\phi r_n \left(\begin{smallmatrix}\text{bearing}\\ \text{web}\end{smallmatrix}\right) = 0.75(2.4)(65)(3/4)(0.455) = 39.93^k > 15^k$ ✓

$\phi r_n \left(\begin{smallmatrix}\text{bearing}\\ \text{plate}\end{smallmatrix}\right) = 0.75(2.4)(58)(3/4)(5/16) = 24.5^k > 15^k$ ✓

$\phi R_n = 1.51(17.9) = 27.0^k > 15^k$ ✓

Max Plate Thickness

$A_{bearing} = \pi\left(\frac{0.75}{2}\right)^2 = 0.442\ in^2$

$C' = 11.3\ in$

$M_{max} = 1.25 \cdot F_{nv} \cdot A_b \cdot C'$

$= 1.25(48)(0.442)(11.3) = 299.7^{"k}$

$t_{max} = \frac{6 M_{max}}{F_y d^2} = \frac{6(299.7)}{36(12)^2} = 0.347 > 5/16"$

$\therefore$ 5/16" plate Ok

Plate Shear Yielding

$\phi R_n = 1.0(0.6)(36)(12)(5/16) = 81^k > 15^k$ ✓ $\therefore$ Ok

Plate Shear Rupture

$\phi R_n = 0.75(0.6)(58)(12 - 4(3/4 + 1/8)) \cdot 5/16 = 69.3^k > 15^k$ ✓ Ok

Plate Block Shear

$L_{ev} = 1.5"$
$L_{eh} = 1.5"$

Table 9-3a = 46.2^k
Table 9-3b = 170^k
Table 9-3c = 194^k

$\phi R_n = 5/16(46.2 + 170) = 67.6^k > 15^k$

Flexure Shear Interaction

$V_u = 15^k$
$V_c = 1.0(0.6)(36)(12 \cdot 5/16) = 81^k$

$M_u = 15^k \cdot 8.65\ in = 129.75^{\text{"}k}$
$\phi M_n = 0.9 F_y \cdot Z = 0.9(36) \cdot \frac{5/16 (12)^2}{4} = 364.5^{\text{"}k}$

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 \le 1.0$$

$$\left(\frac{15}{81}\right)^2 + \left(\frac{129.75}{364.5}\right)^2 = 0.161 \le 1.0 \quad \therefore ok$$

Plate Buckling

$$\lambda = \frac{h_o\sqrt{F_y}}{10 t_w\sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}} = \frac{12\sqrt{36}}{10(5/16)\sqrt{475+280\left(\frac{12}{8.65}\right)^2}}$$

$\lambda = 0.69$
$\lambda = 0.69 \le 0.7 \quad \therefore \quad Q = 1$

$F_{cr} = Q F_y \longrightarrow F_{cr} = F_y$

$\phi M_n = 0.9(36) \cdot \frac{5/16 (12)^2}{6} = 243^{\text{"}k} > M_u = 129.75^{\text{"}k}$

Weld Strength

$t_{weld\ min} = \frac{5}{16} \cdot \frac{5}{8} = 0.195 \longrightarrow 1/4\text{"}$ weld

$\ell = 12 - 2(1/4) = 11.5 \qquad e = 8.65\ in.$

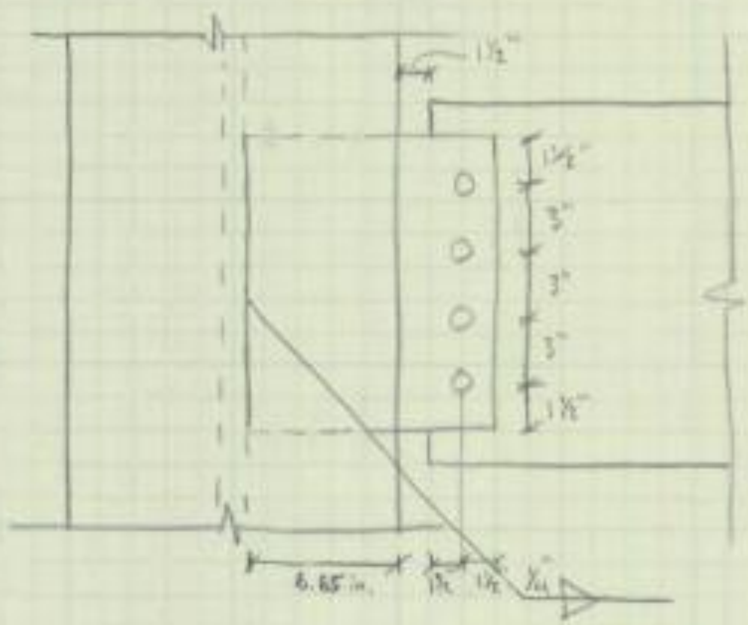
$a = \frac{e_x}{\ell} = \frac{8.65}{11.5} = 0.752 \rightarrow 0.75$

Table 8-4
$k = 0$, $a = 0.75$ } $C = 1.67$ $\quad \phi R_n = 0.75(2)(1)(1.67)(11.5) = 28.8^k > 15^k$
$\therefore ok$

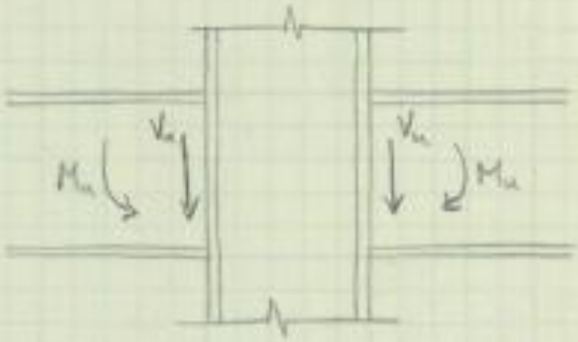
Shear Tab Design

(4) - 3/4" Φ A325N Bolts

5/16" A36 Plate



Moment Connection Design
- Flange Plate Bolted / Web Bolted



Column: W12x53

Beams: W14x34
$V_u = 18^k$
$M_u = 90'^k$

- assume two rows of bolts in flange plates
- assume one row of bolts in web plates

Properties:

W12x53	W14x34
$d = 12.1$ in.	$d = 14.0$ in^2
$b_f = 10$ in.	$b_f = 6.75$ in.^{2}
$t_f = 0.575$ in.	$t_f = 0.455$ in.
$t_w = 0.345$ in.	$t_w = 0.285$ in.
$S_x = 70.6$ in^3	$S_x = 48.6$ in^3
$k_{des} = 1.18$	

- Use 3/4" ϕ A325-N bolts

Girder Web to Column Flange

$V_u =$

$$\#\text{ bolts} = \frac{18}{17.9} = 1 \longrightarrow \text{Use 3 bolts}$$

$$\text{Plate Length} = 2(3) + 2(1.25) = 8.5"$$

Trial Plate Thickness for Shear Rupture

$$t_{plate} = \frac{V_u}{\phi\, 0.6\, F_u\, A_{nv}} = \frac{18}{0.75(0.6)(58)(8.5 - 3(3/4 + 1/8))} = 0.12$$

$\therefore$ Use 1/4" PL

<u>Gross Shear</u>

$$\phi V_n = \phi\, 0.6\, F_y A_g = 0.75(0.6)(36)(8.5)(1/4) = 34.43^k >$$

<u>Block Shear</u>

Table 9.3 a = 35.3 k/in
9.3 b = 117 k/in
9.3 c = 132 k/in

$$\phi V_n = 1/4(35.3 + 117) = 38.1^k >$$

<u>Bolt Shear / Bearing / TO</u>

Bolt Shear : $\phi r_n = 17.9^k$

Bearing : $\phi r_n = \phi\, 2.4\, d_b F_u t$
$= 0.75(2.4)(3/4)(58)(1/4) = 19.58^k$

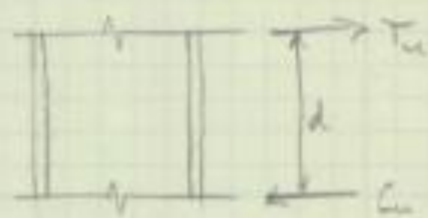
<u>TO to Edge</u> : $\phi r_n = \phi\, 1.2\, L_c F_u t$
$= 0.75(1.2)\left(1.25 - \left(\frac{3/4 - 1/16}{2}\right)\right)(58)(1/4) = 11.8^k$

<u>TO Other</u> : $\phi r_n = \phi\, 1.2\, L_c F_u t$
$= 0.75(1.2)(3/4 - 1/16)(58)(1/4) = 10.6^k$

$$\phi V_n = 11.8 + 17.9 + 10.6 = 40.3^k >$$

<u>Tension Flange Plate</u>

$$T_u = \frac{M_u}{d + t_p}$$



$$T_u = \frac{90 \times 12}{12.1 + .25} = 87.45^k$$

$b_f = 6.75$ in. → Use 6" plate , 2 rows of bolts

Tension Yielding

$\phi T_n = \phi F_y A_g = 0.9(36)(6)(1/4) = 48.6^k < 87.45^k$ X

∴ No Good, increase PL to 1/2" thick

$\phi T_n = 0.9(36)(6)(1/2) = 97.2$

$97.2 > T_n = \frac{90 \times 12}{11.9 + .5} = 87.1^k$ ✓ ok.

Tension Rupture

$A_e = \min \begin{cases} 0.85(6)(1/2) = 2.55 \text{ in}^2 \\ (6 - 2(.875))(\frac{1}{2}) = \underline{2.13 \text{ in}^2} \end{cases}$

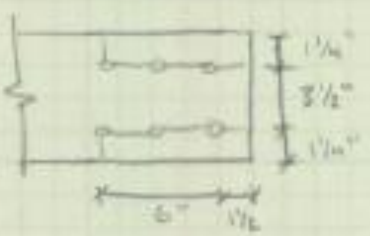
$\phi T_n = \phi F_u A_e = 0.75(58)(2.13) = 92.66^k > 87.1^k$ ✓ ok

Block Shear

gage = 3.5"

$\#\text{ of bolts} = \frac{87.1}{17.9} = 4.87 \rightarrow$ Use 6 bolts

Try 2 rows of 3 bolts



$A_{gv} = 2(7.5)(1/2) = 7.5 \text{ in}^2$

$A_{nv} = 7.5 - [2(2.5)(1/4 + 1/8)(1/2)] = 5.31 \text{ in}^2$

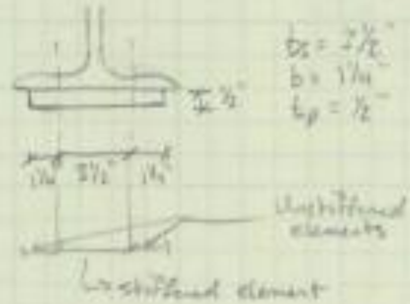
$A_{nt} = 2(1.25 - 1/2(0.875))(1/2) = 0.81 \text{ in}^2$

$U_{bs} F_u A_{nt} = 1.0(58)(0.81) = 47^k$
$0.6 F_u A_{nv} = 0.6(58)(5.31) = 184.8^k$
$0.6 F_y A_{gv} = 0.6(36)(7.5) = 162^k$

$\phi R_n = 0.75(47 + 162) = 156.8^k > 87.1^k$ ✓ ok

∴ Tension Flange Plate ok

Compression Flange Plate



$b_s = 3\frac{1}{2}"$
$b = 1\frac{1}{4}"$
$t_p = \frac{1}{2}"$

Stiffened Element

$$\frac{3.5}{.5} \le \frac{253}{\sqrt{36}}$$

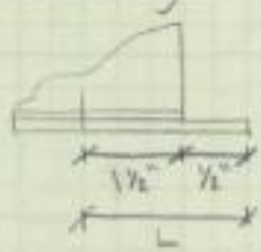
$$7 \le 42.17 \checkmark$$

Unstiffened

$$\frac{1.25}{.5} \le \frac{95}{\sqrt{36}}$$

$$2.5 \le 15.83 \checkmark$$

Flexural Buckling



$K = 0.65$
$L = 1.5 + .5 = 2"$
$r = 0.289\, t_p = 0.145$

$$\frac{KL}{r} = \frac{0.65(2)}{0.145} = 8.97 < 25$$

$\therefore F_{cr} = F_y$

$$\phi T_n = 0.9\, F_{cr} A_g$$
$$= 0.9\, F_y A_g$$
$$= 0.9(36)(7.5)$$

$\phi T_n = 243^k > 87.1^k$ ✓ ok

$\therefore$ Compression Flange Plate ok

Girder Limit States

Reduced Flexural Strength

$A_{fg} = b_f t_f = (6.49)(0.38) = 2.47 \text{ in}^2$

$A_{fn} = A_{fg} - 2(d_b + 1/8) t_f = 2.47 - 2(.875)(0.38) = 1.81 \text{ in}^2$

$\frac{F_y}{F_u} = \frac{50}{65} = 0.77 \leq 0.8 \quad \therefore Y_t = 1.0$

$F_u A_{fn} > Y_t F_y A_{fg}$
$65(1.81) > 50(2.47)$
$117.7 > 123.5$ ✓ ok

$\phi M_n = \phi F_y S_x = \frac{0.9(50)(48.6)}{12} = 182.3^k > 90'^k$ ✓

Beam Flange Block States

$A_{gv} = 2(7.5)(1/2) = 7.5 \text{ in}^2$

$A_{nv} = 7.65 - 2(2.5)(.875)(0.38) = 5.99 \text{ in}^2$

$A_{nt} = 2(1.25 - 1/2(.875))(.38) = 0.618 \text{ in}^2$

$U_{bs} F_u A_{nt} = 1.0(65)(0.618) = 40.2^k$
$0.6 F_u A_{nv} = 0.6(65)(5.99) = 233.6^k$
$0.6 F_y A_{gv} = 0.6(50)(7.5) = 225^k$

$\phi R_n = 0.75(40.2 + 225) = 198.9^k > 87.1^k$ ✓ ok.

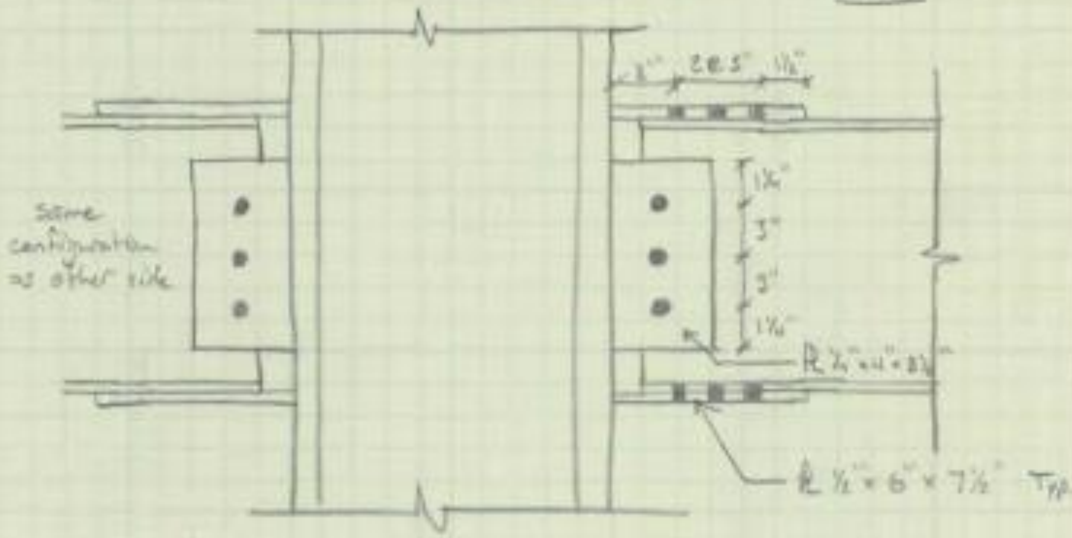
$\therefore$ Girder Limit States Ok

Moment Connection Design

Typ. Flang Plate : ½" × 6" × 7½"

Typ. Web Plate : ¼" × 4" × 8½"

NTS.



Appendix I: Mechanical Breadth

The following tables were used to size the mechanical ducts needed for the ventilation system. Given the required airflow of 0.075 CFM, the parking garage square footage, and the number of exhaust fans, the air quantity per fan was found to be 4750CFM.

Entering the Table with 4750 CFM and an air velocity of 1800fpm, the size of the duct was found to be 24” diameter. The table on the following page shows equivalent sizes for rectangular ducts. A circular duct was selected to do availability of large rectangular ducts and cost information availability.

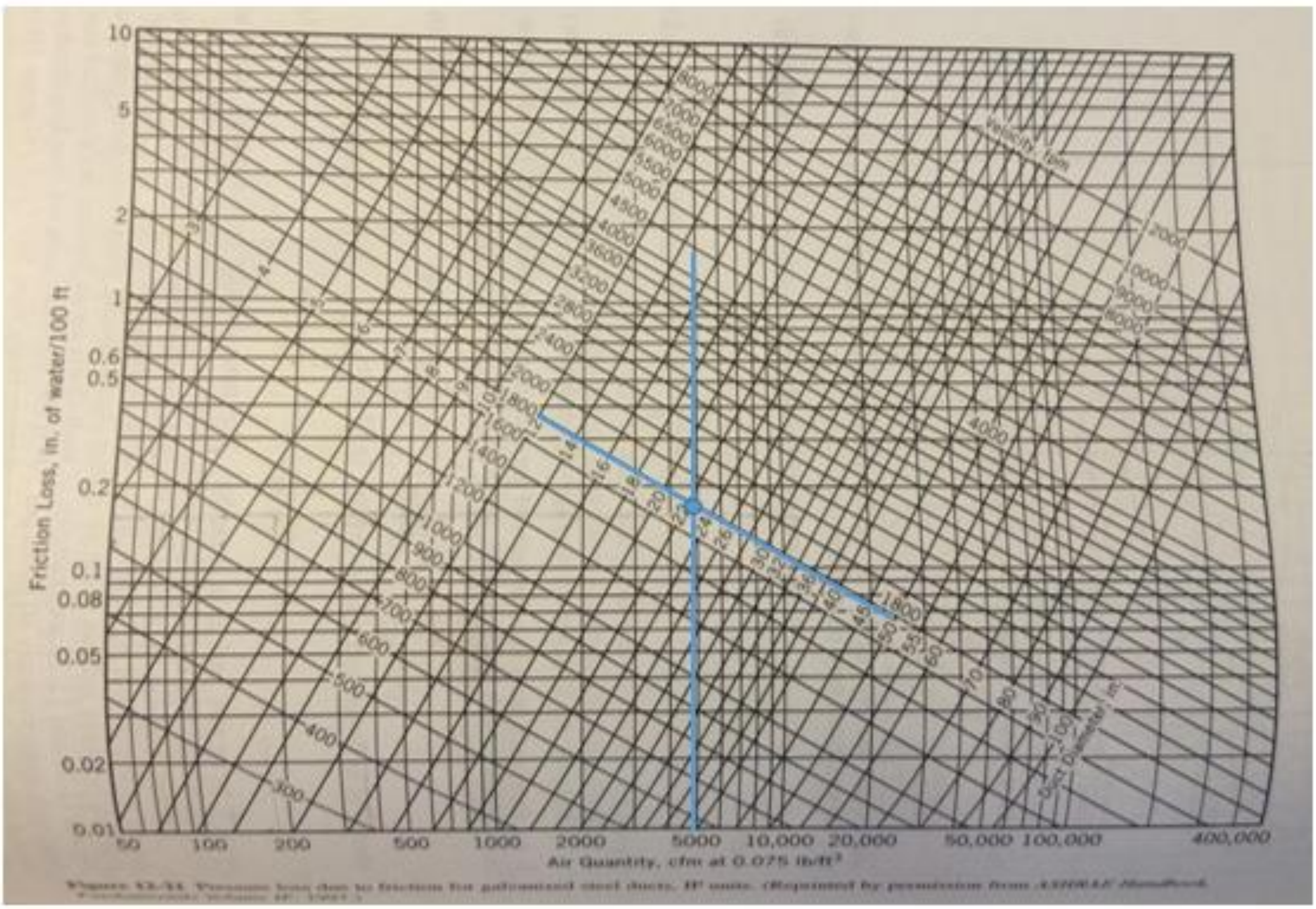


Figure 53: Figure 12-21 from the ASHRAE Handbook

Any size combination that lies to the right of the stepped line yields an acceptable duct size for the ventilation system.

Table 12-7 Circular Equivalents of Rectangular Ducts for Equal Friction and Capacity—Dimensions in Inches, Feet, or Meters

Side a of Rectangular Duct	Diameter D_c of Circular Duct																
	b = 6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	22	24
6	6.6																
7	7.1	7.7															
8	7.5	8.2	8.8														
9	8.0	8.6	9.3	9.9													
10	8.4	9.1	9.8	10.4	10.9												
11	8.8	9.5	10.2	10.8	11.4	12.0											
12	9.1	9.9	10.7	11.3	11.9	12.5	13.1										
13	9.5	10.3	11.1	11.8	12.4	13.0	13.6	14.2									
14	9.8	10.7	11.5	12.2	12.9	13.5	14.2	14.7	15.3								
15	10.1	11.0	11.8	12.6	13.3	14.0	14.6	15.3	15.8	16.4							
16	10.4	11.4	12.2	13.0	13.7	14.4	15.1	15.7	16.3	16.9	17.5						
17	10.7	11.7	12.5	13.4	14.1	14.9	15.5	16.1	16.8	17.4	18.0	18.6					
18	11.0	11.9	12.9	13.7	14.5	15.3	16.0	16.6	17.3	17.9	18.5	19.1	19.7				
19	11.2	12.2	13.2	14.1	14.9	15.6	16.4	17.1	17.8	18.4	19.0	19.6	20.2	20.8			
20	11.5	12.5	13.5	14.4	15.2	15.9	16.8	17.5	18.2	18.8	19.5	20.1	20.7	21.3	21.9		
22	12.0	13.1	14.1	15.0	15.9	16.7	17.6	18.3	19.1	19.7	20.4	21.0	21.7	22.3	22.9	24.1	
24	12.4	13.6	14.6	15.6	16.6	17.5	18.3	19.1	19.8	20.6	21.3	21.9	22.6	23.2	23.9	25.1	26.2
26	12.8	14.1	15.2	16.2	17.2	18.1	19.0	19.8	20.6	21.4	22.1	22.8	23.5	24.1	24.8	26.1	27.2
28	13.2	14.5	15.6	16.7	17.7	18.7	19.6	20.5	21.3	22.1	22.9	23.6	24.4	25.0	25.7	27.1	28.2
30	13.6	14.9	16.1	17.2	18.3	19.3	20.2	21.1	22.0	22.9	23.7	24.4	25.2	25.9	26.7	28.0	29.3
32	14.0	15.3	16.5	17.7	18.8	19.8	20.8	21.8	22.7	23.6	24.4	25.2	26.0	26.7	27.5	28.9	30.1
34	14.4	15.7	17.0	18.2	19.3	20.4	21.4	22.4	23.3	24.2	25.1	25.9	26.7	27.5	28.3	20.7	31.0
36	14.7	16.1	17.4	18.6	19.8	20.9	21.9	23.0	23.9	24.8	25.8	26.6	27.4	28.3	29.0	30.5	32.0
38	15.0	16.4	17.8	19.0	20.3	21.4	22.5	23.5	24.5	25.4	26.4	27.3	28.1	29.0	29.8	31.4	32.8
40	15.3	16.8	18.2	19.4	20.7	21.9	23.0	24.0	25.1	26.0	27.0	27.9	28.8	29.7	30.5	32.1	33.6

Source: Reprinted by permission from *ASHRAE Handbook, Fundamentals Volume*, 1989.

Ventilation Design for Sub-Grade Parking Garage

- Initially, all levels of parking garage were above grade and designed as an open air structure.
- To accomodate the structural redesign, one level of parking garage will be moved below grade. Therefore a ventilation system needs to be designed to exhaust the air.

- The 2012 International Mechanical Code and the ASHRAE handbook set the following minimum ventilation airflow rates for enclosed parking garages:

  1) Automatic operation of the system shall not reduce the ventilation airflow rate below 0.05 CFM/ft.² when space is considered unoccupied.

  2). System shall be capable of producing a ventilation airflow rate of 0.75 CFM/ft.²

Design Decision: Design system for required ventilation rate of 0.75 CFM/ft.²
- Install motion detectors and carbon monoxide detectors to allow automatic shut on/off of system to optimize performance at non-peak hours of ventilation needs

Parking Garage Floor Area = 25,176 SF → Design to 26,000 SF

$$0.75\ \frac{CFM}{SF} \times 26{,}000\ SF = 19{,}500\ CFM$$

Required Exhaust = 19,500 CFM

∴ Provide 4 Exhaust Fans, each requiring at least 4,875 CFM

- 4 units will be situated in the corners of the building footprint
- Exhaust fans will exhaust air to outside at street level.
- Exhaust ducting will run along the perimeter foundation walls.

Ventilation Requirements Check:

$$Q = nV$$

Q = total fresh air supply (CFM/min)
n = required air change / hour (h^{-1})
V = volume of garage

- it is recommend to have 4-6 air changes per hour for a parking garage. Assume 5

$$Q = 5\left(\frac{1}{hr}\right)\left(\frac{1hr}{60min}\right) \times \left[26000ft^2 \times 9ft\right]$$

$$Q = 19{,}500 \text{ CFM} \checkmark$$

Assume 1800 ft/min air...

$$\frac{4875 \text{ CFM}}{1800 \text{ ft/min}} = 2.71 ft^2 \leftarrow$$ required airflow area needed for ducts

Provide 24" Φ ducts

$$A = \pi(1')^2 = 3.14 ft^2 > 2.71 ft^2 \checkmark \text{ ok.}$$

	Cost		units	CY	ft	Cost	
Exhaust Fans	$	2,600.00	4			$	10,400.00
Belt Drive 1/8" SP							
24", 6430 CFM, 1HP							
Ducts	$	44.50			380	$	16,910.00
Spiral, galv. steel							
24" dia. 24 gauge							
Connectors	$	53.00	38			$	2,014.00
24" diameter							
End Cap	$	86.00	2			$	172.00
24" diameter							
Excavating	$	9.05		455		$	4,117.75
Common earth							
10-14' deep							
3/4 CY Excavator							
Hauling	$	10.40		455		$	4,732.00
30 mph, cylce 30 min							
Additional Concrete							
Slab						$	6,375.00
Forms						$	1,957.00
Walls						$	5,560.00
				Total Cost		**$52,237.75**	

Appendix J: Cost Analysis Breadth

Existing Concrete Structural System				
Building Element	Quantity	Unit	Unit Cost	Cost
Slab on Grade	21,195	SF	$ 5.00	$ 105,975.00
Elevated Slab - 8" Mild Reinforced	26,307	SF	$ 17.00	$ 447,219.00
Elevated Slab - 7.5" PT	242,919	SF	$ 17.00	$ 4,129,623.00
Elevated Slab - 12" Mild Reinforced	8,379	SF	$ 20.00	$ 167,580.00
Column/Slab Drop Heads	23,129	SF	$ 12.00	$ 277,548.00
Wall Columns	818	CY	$ 750.00	$ 613,500.00
Miscellaneous Concrete Items	28	Floor	$ 10,000.00	$ 280,000.00
			Total Cost	$ 6,021,445.00

Ventilation System					
	Cost	units	CY	ft	Cost
Exhaust Fans	$ 2,600.00	4			$ 10,400.00
Belt Drive 1/8" SP					
24", 6430 CFM, 1HP					
Ducts	$ 44.50			380	$ 16,910.00
Spiral, galv. steel					
24" dia. 24 gauge					
Connectors	$ 53.00	38			$ 2,014.00
24" diameter					
End Cap	$ 86.00	2			$ 172.00
24" diameter					
Excavating	$ 9.05		455		$ 4,117.75
Common earth					
10-14' deep					
3/4 CY Excavator					
Hauling	$ 10.40		455		$ 4,732.00
30 mph, cylce 30 min					
Additional Concrete					
Slab					$ 6,375.00
Forms					$ 1,957.00
Walls					$ 5,560.00
			Total Cost		**$52,237.75**

Steel Cost:

Beam Size	#	lbs	tons	LF	Cost/LF	Cost per ton	Cost
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]

Columns							
size	lb/ft	#	LF	lbs	tons	$/LF	$
W10	33	128	3297.9	108830.7	54.41535	78.5	$ 258,885.15
W10	39	21	626	24414	12.207	78.5	$ 49,141.00
W12	40	17	410.3	16412	8.206	86.5	$ 35,490.95
W10	45	15	426.3	19183.5	9.59175	78.5	$ 33,464.55
W10	49	18	504	24696	12.348	78.5	$ 39,564.00
W12	50	2	56	2800	1.4	86.5	$ 4,844.00
W12	53	3	84	4452	2.226	86.5	$ 7,266.00
W10	54	16	448	24192	12.096	78.5	$ 35,168.00
W12	58	4	112	6496	3.248	86.5	$ 9,688.00
W10	60	8	224	13440	6.72	116	$ 25,984.00
W12	65	2	56	3640	1.82	146	$ 8,176.00
W10	68	6	168	11424	5.712	116	$ 19,488.00
W12	72	4	112	8064	4.032	146	$ 16,352.00
W10	77	9	252	19404	9.702	116	$ 29,232.00
W12	79	5	140	11060	5.53	146	$ 20,440.00
W12	87	2	56	4872	2.436	146	$ 8,176.00
W10	88	4	112	9856	4.928	116	$ 12,992.00
W12	96	4	112	10752	5.376	146	$ 16,352.00
W10	100	6	168	16800	8.4	187	$ 31,416.00
W12	106	1	28	2968	1.484	146	$ 4,088.00
W10	112	1	28	3136	1.568	187	$ 5,236.00
					173.4461		$ 671,443.65
						Cost/ton	$4000/ton
					Column	Total	$ 1,365,228.05

Total Steel Cost		
Beams	$	5,318,803.40
Columns	$	1,365,228.05
Connections	$	668,403.15
	$	7,352,434.60

Foundations Cost Comparison

Existing Foundations														
		B	L	D		Volume	SFCA		# of bars	# of bars	bar	total length	Volume	Weight(tons)
Cellar MAT		53.583	43.667	4		346.64	778.00		157	128	1	13714.34	95.238	23.333
4-K	F-15	15	15	4.1667		34.72	250.00		14	14	1.27	420	3.704	0.908
5.5-K	F-15	15	15	4.1667		34.72	250.00		14	14	1.27	420	3.704	0.908
6.5-7-K	F-12x21	12	21	3.5		32.67	231.00		14	long	1	294	2.042	0.500
									14	short	0.6	168	0.700	0.172
7-G-H	F-15	15	15	4.1667		34.72	250.00		14	14	1.27	420	3.704	0.908
7-J	F-14	14	14	4		29.04	224.00		13	13	1.27	364	3.210	0.787
8-G-H	F-11	11	11	3.1667		14.19	139.33		11	11	1	242	1.681	0.412
8-J	F-10	10	10	3		11.11	120.00		11	11	0.79	220	1.207	0.296
8-K	F-9	9	9	2.6667		8.00	96.00		9	9	0.79	162	0.889	0.218
2-A	F-6	6	6	1.8333		2.44	44.00		10	10	0.31	120	0.258	0.063
3-A	F-6	6	6	1.8333		2.44	44.00		10	10	0.31	120	0.258	0.063
4-A	F-6	6	6	1.8333		2.44	44.00		10	10	0.31	120	0.258	0.063
5-A	F-6	6	6	1.8333		2.44	44.00		10	10	0.31	120	0.258	0.063
6-A	F-6	6	6	1.8333		2.44	44.00		10	10	0.31	120	0.258	0.063
7-A	F-6	6	6	1.8333		2.44	44.00		10	10	0.31	120	0.258	0.063
8-A	F-6	6	6	1.8333		2.44	44.00		10	10	0.31	120	0.258	0.063
1-2-B	F-12x24	12	24	3.5		37.33	252.00		11	long	0.79	264	1.448	0.355
									11	long	0.6	264	1.100	0.270
									11	short	0.6	132	0.550	0.135
									6	short	0.44	72	0.220	0.054
3-B	F-14	14	14	4		29.04	224.00		13	13	1.27	364	3.210	0.787
4-B	F-14	14	14	4		29.04	224.00		13	13	1.27	364	3.210	0.787
5-B	F-13	13	13	3.6667		22.95	190.67		12	12	1.27	312	2.752	0.674
6-B	F-14	14	14	4		29.04	224.00		13	13	1.27	364	3.210	0.787
7-B	F-12	12	12	3.5		18.67	168.00		10	10	1.27	240	2.117	0.519
1-2-C	F-13X25	13	25	4		48.15	304.00		20	long	1	500	3.472	0.851
									10	short	1	130	0.903	0.221
									8	short	0.6	104	0.433	0.106
MAT LEFT TOWER		32.333	33	3.5		138.31	457.33		33	33	1	2156.00	14.972	3.668
									33	33	1.27	2156.00	19.015	4.659
MAT RIGHT TOWER		32.333	33	3.5		138.31	457.33		33	33	1	2156.00	14.972	3.668
									33	33	1.27	2156.00	19.015	4.659
7-C	F-13	13	13	3.6667		22.95	190.67		12	12	1.27	312	2.752	0.674
1-2-D	F-13X25	13	25	4		48.15	304.00		20	long	1	500	3.472	0.851
									10	short	1	130	0.903	0.221
									8	short	0.6	104	0.433	0.106
7-8-D	F-13X26.5	13	26.5	4		51.04	316.00		10	long	1.27	265	2.337	0.573
									10	long	1	265	1.840	0.451
									10	short	1	130	0.903	0.221
									8	short	0.6	104	0.433	0.106
1-2-E	F-12X24	12	24	3.5		37.33	252.00		11	long	0.79	264	1.448	0.355
									11	long	0.6	264	1.100	0.270
									11	short	0.6	132	0.550	0.135
									6	short	0.44	72	0.220	0.054
3-E	F-12	12	12	3.5		18.67	168.00		10	10	1.27	240	2.117	0.519
4-E	F-11	11	11	3.1667		14.19	139.33		11	11	1	242	1.681	0.412
5-E	F-12	12	12	3.5		18.67	168.00		10	10	1.27	240	2.117	0.519
6-E	F-13	13	13	3.6667		22.95	190.67		12	12	1.27	312	2.752	0.674
7-8-E	F-13X26.5	13	26.5	4		51.04	316.00		10	long	1.27	265	2.337	0.573
									10	long	1	265	1.840	0.451
									10	short	1	130	0.903	0.221
									8	short	0.6	104	0.433	0.106
1-2-F	F-12X24	12	24	3.5		37.33	252.00		11	long	0.79	264	1.448	0.355
									11	long	0.6	264	1.100	0.270
									11	short	0.6	132	0.550	0.135
									6	short	0.44	72	0.220	0.054

3-F	F-10	10	10	3		11.11	120.00		11	11	0.79	220	1.207	0.296
4-F	F-10	10	10	3		11.11	120.00		11	11	0.79	220	1.207	0.296
5-F	F-11	11	11	3.1667		14.19	139.33		11	11	1	242	1.681	0.412
6-F	F-14	14	14	4		29.04	224.00		13	13	1.27	364	3.210	0.787
7-8-F	F-12X26.5	12	26.5	3.5		41.22	269.50		24	long	1	636	4.417	1.082
									14	short	1	168	1.167	0.286
									8	short	0.6	96	0.400	0.098
1-2-G	F-12X24	12	24	3.5		37.33	252.00		11	long	0.79	264	1.448	0.355
									11	long	0.6	264	1.100	0.270
									11	short	0.6	132	0.550	0.135
									6	short	0.44	72	0.220	0.054
3-G	F-12	12	12	3.5		18.67	168.00		10	10	1.27	240	2.117	0.519
1-2-H	F-13X25	13	25	4		48.15	304.00		20	long	1	500	3.472	0.851
									10	short	1	104	0.722	0.177
									8	short	0.6	104	0.433	0.106
3-H-J	F-12X21	12	21	3.5		32.67	231.00		14	long	1	294	2.042	0.500
									14	short	0.6	168	0.700	0.172
1-2-J.3	F-12X24	12	24	3.5		37.33	252.00		11	long	0.79	264	1.448	0.355
									11	long	0.6	264	1.100	0.270
									11	short	0.6	132	0.550	0.135
									6	short	0.44	72	0.220	0.054
2-K	F-11	11	11	3.1667		14.19	139.33		11	11	1	242	1.681	0.412
3-K	F-14	14	14	4		29.04	224.00		13	13	1.27	364	3.210	0.787
2-L	F-9	9	9	2.6667		8.00	96.00		9	9	0.79	162	0.889	0.218
3-L	F-10	10	10	3		11.11	120.00		11	11	0.79	220	1.207	0.296
4-L	F-10	10	10	3		11.11	120.00		11	11	0.79	220	1.207	0.296
5-L	F-11	11	11	3.1667		14.19	139.33		11	11	1	242	1.681	0.412
6.5-L	F-9	9	9	2.6667		8.00	96.00		9	9	0.79	162	0.889	0.218
7-L	F-8	8	8	2.5		5.93	80.00		7	7	0.79	112	0.614	0.151
8-L	F-7	7	7	2.1667		3.93	60.67		7	7	0.6	98	0.408	0.100
					Total	1762.40	10599.50					Total		69.475
					Mat Foundations	623.27	1692.67					Mat Foundations		39.987
					Without Mats	1139.13	8906.84					Without Mats		29.488

Foundation Comparison			
	Concrete (yds)	Concrete (SFCA)	Steel (tons)
Existing	1139.13	8906.84	29.488
Redesing	604	5402	16.5
	535.13	3504.84	12.988
Forming /SFCA		145	
Material /CY	310		
Placing-pumped /CY	32.5		
Rebar /tons			2300
	$ 183,282.03	$ 508,201.80	$ 29,872.40
	$ 691,483.83		$ 29,872.40
	Total Foundation Savings	$ 1,229,558.03	

Parking Spot Costs			
Parking Garage Levels	Spots Eliminated	Cost per Month	Annual Losses
Basement	11	189	$ 24,948.00
1st Floor	6	189	$ 13,608.00
2nd Floor	7	189	$ 15,876.00
3rd Floor	11	189	$ 24,948.00
		Total Cost	$ 79,380.00

Additional Excavation			
	CY	$/CY	Cost
Excavating	7702	9.05	$ 69,703.10
Hauling	7702	10.4	$ 80,100.80
			$ 149,803.90