TECHNICAL REPORT ONE

1776 Wilson Boulevard

Arlington Virginia

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Structural Option

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Table of Contents

Executive Summary ............................................................................................................................................. 2
Introduction ......................................................................................................................................................... 3
Site Conditions .................................................................................................................................................. 4
Structural System Overview ............................................................................................................................... 5
  Foundation ....................................................................................................................................................... 6
  Floor System .................................................................................................................................................... 7
  Roof System .................................................................................................................................................. 7
  Columns ......................................................................................................................................................... 8
  Lateral System ............................................................................................................................................... 9
Design Codes .................................................................................................................................................... 10
Material Properties ......................................................................................................................................... 10
Design Loads ................................................................................................................................................... 11
Design Analysis ............................................................................................................................................... 12
  Wind Loads .................................................................................................................................................. 13
  Seismic Loads ............................................................................................................................................. 15
  Spot Checks ................................................................................................................................................ 16
Conclusions ..................................................................................................................................................... 17
Appendices ..................................................................................................................................................... 18
  Appendix A: Framing Plans ............................................................................................................................ 19
  Appendix B: Wall Sections and Assemblies ................................................................................................. 22
  Appendix C: Wind Loads ............................................................................................................................... 26
  Appendix D: Seismic Loads ........................................................................................................................... 35
  Appendix E: Snow Loads ............................................................................................................................... 42
  Appendix F: Spot Checks .............................................................................................................................. 45
Executive Summary

Technical report one serves as a structural concepts and existing conditions report centered on the 1776 Wilson Boulevard project located in Arlington Virginia. Research was done on the location and structural system used for this project and the latest codes and standards were used to perform analysis and checks.

1776 Wilson utilizes a post tensioned concrete structure with two lateral force resisting systems that work together to resist and transfer the lateral loads. The geotechnical report done on the site led to the choice of a shallow four inch slab on grade foundation. The floors are flat slab reinforced concrete slabs with drop panels at the column locations. Post tensioning begins at the second story where the office spaces begin and continues to the top floor, including the penthouse. High strength concrete is utilized in order to create open spaces and high ceiling heights, especially on the ground floor where there is enough space to have tenant mezzanines. The two lateral systems used are reinforced concrete shear walls on the bottom three stories and ordinary reinforced concrete moment frames on the upper stories.

The lateral loads were calculated using ASCE 7-10. Wind loads were found using the Main Wind Force Resisting System (MWFRS) directional procedure and the penthouse was treated separately using chapter 29. Some simplification was done to the floor plan and the treatment of the two lateral force resisting systems. A more detailed analysis will follow once a better understanding of how the two systems work together is gained. The Equivalent Lateral Force Method was used for seismic loads. After the analysis, it was determined that the seismic base shear controls the over turning moment.

Spot checks were performed on an interior column at the ground level and a portion of a two way post tensioned slab. Only gravity loads were taken into account for the column check so my designed column which matched the size of the actual column used will end up being larger once lateral loads are considered which could explain the use of 8000 psi concrete compared to the 5000 psi I used for the check. The slab check included thickness, precompression stress, stresses immediately after jacking, and stresses at service loads in accordance with ACI 318-08. Two stresses at service loads were not within permissible limits by code. Assumptions made due to lack of information at this time could be the cause of this. It is important to note that more information was gained concerning the tendons used that will change the check done. This information was not gained in time to include in this report but will be used in a more detailed analysis of the floor slabs for a future technical report.
Introduction

Located in the Rosslyn/Ballston corridor of Arlington Virginia, 1776 Wilson Boulevard will be a Class A office building with retail space and three and a half levels of below grade parking. Currently under construction, the building is to be built on a previously contaminated Brownfield site that has been redeveloped. Scheduled to finish in August of 2012, 1776 Wilson will be approximately 249,000 SF and the lump sum contract is valued at 63.5 million dollars.

Designed by RTKL Associates, the three and half level parking garage will be able to hold over 200 cars, all 26,000 SF of retail space will be located on the ground floor, and the upper four floors will contain 108,000 SF of flexible office space perfect for a building that currently searching for future tenants. The retail space will have a high ceiling making it possible for tenant mezzanines. Most of the mechanical equipment will be located in a penthouse on top of the building. Besides the flexible office space, one of the biggest interior aspects of the building is the luminous lobby that complements the generous amount of day lighting the building will receive. 1776 Wilson will also provide downtown convenience, it is to be located within walking distance of two metro stations and several retail outlets and restaurants are within close proximity of the site.

1776 Wilson Boulevard also goes above and beyond the norm when it comes to sustainability; the project is designed to be LEED Platinum. The numerous green features include a 17,000 SF green roof, photovoltaic solar panels on the roof, and an incentive program aimed at educating tenants on the sustainability features of the building.

Arlington County’s C-0-2.5 zoning district will house the finished building; this area generally serves commercial office buildings, hotels, and apartments. The upper floors will be considered separate mixed use occupancy while the parking levels are non separated mixed use. A generous amount of glazing helps create a well and naturally lit interior. Typical one inch thick windows with a U value ranging from 0.26 to 0.28 decorate the facades along with aluminum framed curtain walls. The rest of the façade features pre cast concrete and masonry panels. The roof consists of a combination of 10 and 12 inch thick post-tensioned slabs with roof pavers. The PV solar panels will add 6.6 to 6.8 psf to the roof dead load. In addition to the roof pavers, the roof will be insulated and covered by garden covering. Where roof pavers and garden covering aren’t present, elastomeric cementitious topped insulation is used.
Site Conditions

The site is essentially rectangular with approximate dimensions of 275 feet in the North to South direction and 125 to 200 feet in the East to West direction. This provides a total footprint area of approximately 45,500 SF. The existing site grades slope slightly from the North to the South. The surrounding area includes both residential and commercial buildings; the site itself was occupied by one to two story buildings before the project began.

The results found in the geotechnical report for the project were based on nine soil borings. Ground cover at the site was variable and consisted of one of the following:

- 1-3 inches of asphalt with 1-21 inches of gravel below
- 2 inches of gravel
- 4 to 6 inches of top soil
Below the ground cover, a geotechnical report provided by ECS Mid-Atlantic done on the site broke the soil down into three stratams:

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Fill/Possible Fill</td>
<td>17-36 feet below site grades consisting of various amounts of sand, gravel, and clay</td>
</tr>
<tr>
<td>II</td>
<td>Natural Alluvial/Marine Solids</td>
<td>28-52 feet below site grades and under stratum 1, this stratum consists of poorly graded sand, clayey sand, and low plasticity clay with varying gravel content</td>
</tr>
<tr>
<td>III</td>
<td>Residual Soils/Weathered Rock</td>
<td>Below stratum 2 and consists of Micaceous silty sand with rock fragments.</td>
</tr>
</tbody>
</table>

Table 1 Soil Stratams

It was also known that this particular area has high groundwater flow. The ground water is to be controlled by a dewatering system that will need to be put in place during below grade construction.

1776 Wilson falls into Arlington’s C-0-2.5 zoning district. This district is used for office buildings, commercial uses including retail, as well as hotels and apartments. The ratio of maximum office and/or commercial floor area to site area is 2.5:1. No office building is to exceed 12 stories, excluding penthouse spaces, by site plan approval. All penthouses are limited to one floor. Each plot is to have a minimum average width of 100 feet and a minimum area of 20,000 square feet.

![Fig. 4 Zoning Map for Arlington - The blue outline marks the district where 1776 Wilson is to be located](image)
Structural System Overview

Foundation

The geotechnical report called for a shallow foundation system on the stratum one and two soils with a designed bearing capacity of 10,000 psf. The shallow system will consist of a 4 inch thick slab on grade with 6”x6”-8/8′′ W.W.F. lap mesh 6 inches in all directions and concrete footings. The slab is poured over 10 mil polyethylene and 6 inches of washed gravel. Control joints are placed at 20 feet on center for all exterior slabs. Interior slabs are to be poured in 600 SF panels with control joints placed 30 feet on center. The interior slabs are also to be laid over a layer of vapor barrier which sits on top of 6 inches of washed gravel. Groundwater on the site must be at least two feet below the foundation subgrade level, all of these levels should be mud matted after excavation.

All footings are to penetrate at least one foot into undisturbed soil or compacted fill. All exterior footings must be at least 2’6” below the finished grade, this also holds true for footings in unheated spaces such as garages. The typical wall footing will be 12 inches deep and extend 6 inches past the face of the wall. Disturbed earth under footings will be replaced with 2000 psi concrete. The footings will be 4000 psi concrete and the slab on grade will be 5000 psi.

[Fig. 5 Slab on Grade Control Joint]
Floor System

This project uses a post tensioned concrete structure. Each floor consists of flat slabs with drop panels at the column locations ranging in thickness from 4” slab on grades to 12” thick reinforced concrete slabs. Some portions of the building have thicker slabs but 8-12” is the typical size. The drop panels are mostly 8 to 10” thick. Post tensioning is put to use starting on the second floor and the column layouts create typical 30’ by 30’ bays with 30’ by 45’ bays also present. The high strength concrete used for the framing system of the building allows for these bays as well as reducing the total weight of the building, the typical strength is 6000 psi.

![Typical Post Tensioned Slab Tendon Profile](image)

Fig. 6 Typical Post Tensioned Slab Tendon Profile

Roof System

The roof system of 1776 Wilson consists of 8 and 10 inch thick post tensioned two way slabs. The roof area is covered by either vegetation from the green roof, roof pavers, or a concrete wearing slab. Below the roof surface consists of filter fabric which is accompanied by a deck drainage mat where there is vegetation. Four inches of roof insulation is used as well as hot rubberized asphalt for the waterproofing assembly. The roof areas will see added load due to the solar panels and racking system, these will add 6.6 to 8 psf to the roof dead load.
Columns

The column layouts of 1776 Wilson are uniform and create typical 30 feet by 30 feet bays, with some 30 feet by 45 feet bays as mentioned earlier. The reinforced concrete columns on the upper floors are typically 22x22 inches and 12x30 inches; the lower levels are typically 24x24 inches. Reinforcement ranges from #8 to #11 bars. High strength concrete is used to keep column sizes down and to help maintain the 9’ 3” ceiling heights called for in the plans and drawings, as well as a tall ground floor that provides enough room for tenant mezzanines.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Sizes</th>
<th>Reinforcement</th>
<th>Compressive Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5th</td>
<td>22x22, 12x30</td>
<td>4#10, 8#11, 4#9</td>
<td>Typically 5, some columns are 6</td>
</tr>
<tr>
<td>4th</td>
<td>22x22, 12x30</td>
<td>4#10, 8#10, 4#9</td>
<td>Typically 5, some columns are 6</td>
</tr>
<tr>
<td>3rd</td>
<td>22x22, 12x30</td>
<td>4#9, 4#10, 4#11, 8#10, 8#11</td>
<td>Typically 5, some columns are 6 and 8</td>
</tr>
<tr>
<td>2nd</td>
<td>22x22, 12x30</td>
<td>4#10, 4#11, 8#10, 12#11, 6#9</td>
<td>Typically 5, some columns are 6</td>
</tr>
<tr>
<td>1st</td>
<td>24x24, 12x30, 24x29 3/4*</td>
<td>4#11, 8#9, 8#10, 8#11 12#11,</td>
<td>Typically 8, some columns are 10</td>
</tr>
<tr>
<td>Basement Levels</td>
<td>24x24, 12x30, 32x18, 24x18, 12x18*</td>
<td>4#11, 12#11, 8#11, 4#10, 6#9, 8#9</td>
<td>Typically 8 at the B1 level, 6 below, some columns are 10</td>
</tr>
</tbody>
</table>

Table 2 Column Schedule Summary

*see following details

Fig. 7 Column Details
Lateral System

1776 Wilson Boulevard incorporates a combination of ductile reinforced concrete moment frames and reinforced concrete shear walls. The top two stories hold the ordinary moment frames while the shear walls populate the bottom three stories. Simplifications were made for the wind analysis done and ASCE 7-10 offers a way to calculate seismic loads for buildings with different lateral force resisting systems. More information on those calculations can be found in the wind and seismic sections of this report.

The lateral loads start at the roof diaphragm and travel through the columns that help make up the reinforced concrete moment frames to the floor diaphragm. Once the lateral loads reach the shear walls of the lower stories, the walls resist lateral loads and moments about their strong axis. They can also resist transferred gravity loads from tributary members of the structure. The lateral loads will be transferred through the walls to the floor diaphragm where eventually they will be dispersed into the soil once they reach the foundation.

An important note concerning the lateral force resisting systems of 1776 Wilson is that a better understanding of how the two systems work together needs to be gained before a more detailed analysis of the systems can take place. This will be addressed in a future technical report that focuses on a more in depth lateral system analysis and confirmation.
Design Codes

The following documents were used and referenced in the making of this technical report:

- ACI 318-08 Building Code Requirements for Concrete Buildings published by the American Concrete Institute
- ASCE 7-10 Minimum Design Loads for Buildings and Other Structures published by the American Society of Civil Engineers

Other reference notes:
Some information in this report was gathered from a geotechnical report done by ECS Mid-Atlantic, LLC. This report also is the source for the aerial site image used (fig. 3). A structural report done by Innovative Engineering, Inc. was referenced for information on additional loads added to the structure due to the solar panels. Finally, all images used for figures were provided graciously by Skanska USA.

Materials

The following table summarizes the materials and their strengths that are used in the current design for 1776 Wilson.

<table>
<thead>
<tr>
<th>Structural Element</th>
<th>Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footings, walls, and grade beams</td>
<td>( F'c = 4 \text{ksi} )</td>
</tr>
<tr>
<td>Framed floors, precast concrete units, and slab on grade</td>
<td>( F'c = 5 \text{ksi} )</td>
</tr>
<tr>
<td>Columns</td>
<td>( F'c = 5, 6, 8, \text{ and } 10 \text{ksi} )</td>
</tr>
<tr>
<td>Light weight concrete</td>
<td>( F'c = 3 \text{ksi} )</td>
</tr>
<tr>
<td>Reinforcement steel</td>
<td>ASTM-A615, Grade 60</td>
</tr>
<tr>
<td>Welded wire mesh</td>
<td>ASTM-A185</td>
</tr>
</tbody>
</table>

Post Tensioned Concrete – tendons consist of steel strands that conform to ASTM A-416, \( Fpu=270,000 \text{ psi} \). Tendons are stressed after reaching 75% design strength of concrete.

Masonry – concrete masonry units conform to ASTM C 90 Grade 1, minimum \( f'm=1500 \text{ psi} \). Above grade mortar will be type S conforming to ASTM C 270, below grade will be type M, and veneer face brick will be type N.
Design Loads

The live and dead loads used for the designed building were listed on the drawings; ASCE 7-05 and IBC 2006 were mainly used in the design to arrive at these loads. For the analysis done in this technical report, loads were taken from ASCE 7-10 or assumed. Due to lack of certain information, some assumptions may have been off leading to discrepancies in the calculations. This is true mostly for the slab spot check, which will be addressed in the spot checks section of this report. A more detailed analysis will be done once certain loads are verified.

<table>
<thead>
<tr>
<th>Occupancy</th>
<th>Design Load</th>
<th>ASCE 7-10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Office lobbies 1st floor corridors</td>
<td>100 psf</td>
<td>100 psf</td>
</tr>
<tr>
<td>Offices</td>
<td>50 psf</td>
<td>50 psf</td>
</tr>
<tr>
<td>Corridors above first floor</td>
<td>80 + 20 psf for partitions</td>
<td>80 psf</td>
</tr>
<tr>
<td>Roof</td>
<td>30 psf</td>
<td>20 psf</td>
</tr>
<tr>
<td>Stairs and exit ways</td>
<td>100 psf</td>
<td>100 psf</td>
</tr>
<tr>
<td>Storage</td>
<td>125 psf</td>
<td>125 psf</td>
</tr>
<tr>
<td>Fitness center</td>
<td>100 psf</td>
<td>100 psf</td>
</tr>
</tbody>
</table>

Table 4 Live Load Summary

<table>
<thead>
<tr>
<th>Floor</th>
<th>Design Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal weight concrete</td>
<td>150 pcf</td>
</tr>
<tr>
<td>MEP/ceiling</td>
<td>15 psf</td>
</tr>
<tr>
<td>Drop panels</td>
<td>Same as normal weight concrete</td>
</tr>
</tbody>
</table>

Table 5 Floor Dead Loads

<table>
<thead>
<tr>
<th>Floor</th>
<th>Design Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Weight Concrete</td>
<td>150 pcf</td>
</tr>
<tr>
<td>Solar panels and racking system</td>
<td>6.6-8 psf</td>
</tr>
<tr>
<td>Roof paver, insulation, and waterproofing</td>
<td>24 psf</td>
</tr>
</tbody>
</table>

Table 6 Roof Dead Loads
The snow loads for this analysis were taken from ASCE 7-10 chapter 7. Table 5 summarizes the snow load factors used. The ground snow load was decreased for the Arlington area in the transition from ASCE 7-05 to ASCE 7-10, it dropped from 30 psf to 25 psf. Snow drift calculations were done but were not taken into account for other calculations. My calculations for the snow loads and snow drift loads can be found in Appendix E.

<table>
<thead>
<tr>
<th>Snow Load Criteria</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure Factor</td>
<td>Ce = 0.9</td>
</tr>
<tr>
<td>Thermal Factor</td>
<td>Ct = 1.0</td>
</tr>
<tr>
<td>Importance Factor</td>
<td>Is = 1.0</td>
</tr>
<tr>
<td>Ground Snow Load</td>
<td>Pg = 25 psf</td>
</tr>
<tr>
<td>Flat Roof Snow Load</td>
<td>Pf = 15.75 psf</td>
</tr>
<tr>
<td>Snow Density</td>
<td>17.25 lb/ft^3</td>
</tr>
</tbody>
</table>

Table 7 Snow Load Information
Wind Loads

Wind loads for 1776 Wilson were calculated with accordance to ASCE 7-10 using the main wind force resisting system (MWFRS) directional procedure. This allowed for the determination of wind loads in both the north-south and east-west directions. The velocity pressure was found to be 23.03 psf which is larger than the 17 psf called out in the structural notes for the building. When 1776 Wilson was designed, ASCE 7-05 was used and the basic wind speed for Arlington Virginia was 90 miles per hour but the latest edition of ASCE 7 increased the basic wind speeds and Arlington now has 105 miles per hour. A quick velocity pressure check was done using 90 mph and the result was 17 psf.

For the purposes of tech report one, the floor plan of the building was simplified as well as the facades in order to get a general idea of the wind loads. The method used does not take into account nearby structures and the north façade in particular will need a more detailed and in depth analysis due to the irregularity of the façade and the impact that will have on wind loads. These will be taken into consideration for a future technical report. My calculations for the wind loads can be found in Appendix C.
Fig. 9 E-W Elevation With Wind Data

Fig. 10 North Facade In Plan View
Seismic Loads

The seismic loads for the building were calculated in accordance with ASCE 7-10 chapters 11 and 12 and the equivalent lateral force method was used. There were two sets of numbers for each lateral force resisting system, the shear walls and the moment frames. These sets consisted of the response modification coefficient (R), the over strength factor (Ω), and the deflection amplification factor (Cd). Only the R value was involved in the calculations at this point and the set chosen depended on which R value was lower. According to section 12.2.3.1, if the upper system’s R value is lower than the lower system’s R value, you are to use the values for the upper system, in this case the reinforced concrete moment frames.

The various thicknesses in slabs were taken into account for total building seismic weight. The slabs (which range from 8 inches to 16 inches thick) were broken down and an area was calculated for each so as to make sure my numbers weren’t too conservative. More detailed information on dead loads for the building is still to be determined and will be included in a more detailed seismic analysis for a future tech report. My calculations for the seismic loads can be found in Appendix D.

Fig. 11 E-W Elevation With Seismic Data
Spot Checks

As mentioned in the executive summary, two spot checks were carried out for this tech report. The first was a spot check of column D-6 at the ground floor. Only compressive axial forces were taken into consideration for this spot check, the inclusion of lateral loads will increase the column size. Based on the results of the spot check, the column is more than adequate to carry the loads. I designed the column with a strength of 5000 psi but the actual design uses 8000 psi concrete. My spot check led to a 24”x24” column which is the same size as the designed column. The lateral loads will increase the size and reinforcement which could explain the decision to use high strength concrete. This allows for the column size to be minimized which fits in with the building’s theme of wanting to reduce self-weight.

The slab spot check was done on the second story where the post tensioned slabs start. The first check was to determine a thickness; the result was an 8” thick slab which agreed with the designed slab. Next, precompression stress was checked against ACI maximum and minimum limits, the stress of 222 psi met the criteria and was acceptable. The final checks on the slab were stress checks immediately after jacking and at service load. There were two instances at service load where the stress was not within permissible code limits. Some assumptions were made concerning the tendons that could account for this. Information regarding the tendons was gained but not in time to include in this report, a more detailed analysis will be done taking this information into account for a future tech report. Spot check calculations can be found in Appendix F.

Fig. 12 First Floor Framing Plan - column d-6 is called out as well as the tributary area
Conclusions

The first technical report serves as an investigation into the structural system chosen for the 1776 Wilson project as well as the existing conditions. The goal was to gain a better understanding of the system and how it works. A breakdown of different elements in the system has been detailed in this report and calculations were performed to verify the design. The spot check for the column resulted in the same size but with a lower strength. Only gravity loads were taken into account for that spot check, once lateral loads are considered the size should increase. Choosing a higher concrete strength at that point will help decrease the column size.

The spot check done on the two way post tensioned flat slab was done with assumptions made on tendon information not known at the time of preparing this report. These assumptions could have thrown the design off resulting in two stresses at service loads not being within permissible code limits. As previously mentioned, more detailed information on the tendons became available but not in time to be included in this report. Another analysis with the verified numbers will be done and a more thorough check of the existing slab will be completed.

Another main portion of this technical report was an analysis of lateral loads on the lateral force resisting systems of the building. The seismic loads were determined to control the base shear of the building and the over turning moment as well. A more detailed lateral load analysis will be performed for a future tech report and will carry a better understanding of how the two different lateral force resisting systems work together. For this report, simplifications were made in the wind analysis to get a basic idea of the pressures until more knowledge on the system was gained, at which point more accurate calculations can be performed.

Future Considerations: lateral soil loads, wind loads for north façade, roof uplift, and considering lateral loads for gravity members are considerations that will be fulfilled in future technical reports.
APPENDIX A

Framing Plans
Fig. 13 Typical Basement Framing Plan

Fig. 14 First Floor Framing Plan
Fig. 15 Second and Third Floor Framing Plan - the second floor is identical except it doesn’t have the three knock out panels highlighted.

Fig. 16 Fourth Floor Framing Plan - the green lines outline the green roof.
*Note – these framing plans will be difficult to see as a hard copy, especially the 4th and 5th floors. An electronic version of these plans will be put on the CPEP page for this project that will be easier to view.
APPENDIX B

Wall Sections and Assemblies
Fig. 18 Wall Sections
Fig. 19 Exterior Wall Assemblies
Fig. 20 Exterior Wall Assemblies (continued)
APPENDIX C

Wind Loads
Summary Tables

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Basic Wind Speed</th>
<th>Exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>105 mph</td>
<td>B</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Kd</th>
<th>Kzt</th>
<th>Gf (N-S)</th>
<th>Gf (E-W)</th>
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<tbody>
<tr>
<td>0.85</td>
<td>1</td>
<td>0.84</td>
<td>0.82</td>
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</table>

<table>
<thead>
<tr>
<th>Windward Cp</th>
<th>Leeward Cp</th>
<th>Sidewall Cp</th>
<th>Roof Cp</th>
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<tbody>
<tr>
<td>0.8</td>
<td>-0.37</td>
<td>-0.7</td>
<td></td>
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<table>
<thead>
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<th>Pressures (psf)</th>
<th>N-S</th>
<th>E-W</th>
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<tbody>
<tr>
<td>Leeward Windward</td>
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<tr>
<td>0-15ft</td>
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<td>9.19</td>
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<td>10</td>
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<td>10.64</td>
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<td>30</td>
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<table>
<thead>
<tr>
<th>Story Forces (k)</th>
<th>N-S</th>
<th>E-W</th>
</tr>
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<tbody>
<tr>
<td>Roof</td>
<td></td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>37.9</td>
<td></td>
</tr>
<tr>
<td>5th</td>
<td>49.4</td>
<td>70</td>
</tr>
<tr>
<td>4th</td>
<td>45.5</td>
<td>67.6</td>
</tr>
<tr>
<td>3rd</td>
<td>43.1</td>
<td>79.3</td>
</tr>
<tr>
<td>2nd</td>
<td>62.4</td>
<td>116</td>
</tr>
<tr>
<td>Base Shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>222.5</td>
<td>344.2</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Overturning Moment (ft-k)</th>
<th>N-S</th>
<th>E-W</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>11680</td>
<td>18232</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>2 (ft)</th>
<th>Kz</th>
<th>qz (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td>0.57</td>
<td>13.67</td>
</tr>
<tr>
<td>20</td>
<td>0.62</td>
<td>14.87</td>
</tr>
<tr>
<td>25</td>
<td>0.66</td>
<td>15.83</td>
</tr>
<tr>
<td>30</td>
<td>0.7</td>
<td>16.79</td>
</tr>
<tr>
<td>40</td>
<td>0.76</td>
<td>18.23</td>
</tr>
<tr>
<td>50</td>
<td>0.81</td>
<td>19.43</td>
</tr>
<tr>
<td>60</td>
<td>0.85</td>
<td>20.39</td>
</tr>
<tr>
<td>70</td>
<td>0.89</td>
<td>21.35</td>
</tr>
<tr>
<td>80</td>
<td>0.93</td>
<td>22.31</td>
</tr>
<tr>
<td>90</td>
<td>0.96</td>
<td>23.03</td>
</tr>
</tbody>
</table>
Location: Arlington, Virginia

Risk Category: II

Basic Wind Speed: 105 mph (Fig. 26.5-1c)

Exposure: B

---

143.5”

113.5”

6’6”

105’

4th & 5th floor plan

---

Simplified floor plan

---

N
Wind Directionality Factor: \( K_d = 0.85 \) (Table 36.6-1)
Topographic Factor: \( K_z = 1.0 \) (refer to sect. 36.8)
Gust Effect Factor: Building max height = 83' < 300'

\[
\frac{2}{h_1}\frac{2}{h_1}
\]

\[
L_{eff} = \frac{38.5(0.5\times2)+11.3(0.5\times1)+55(115.5)+68(173)+83(173)}{28.5+41.5+55+68.5+83}
\]

\[
= \frac{4046}{76.3} = 53.1 \Rightarrow 83' < 4(53.1) = 88.4'
\]

E-W

\[
L_{eff} = \frac{38.5(166.5)+41.5(166.5)+55(166.5)+68.5(166.5)+83(166.5)}{28.5+41.5+55+68.5+83}
\]

\[
= \frac{46010}{2766.333} = 166.5' < 53' < 4(166.5) = 666'
\]

(For use of Eq. 9.3)

(For no calc.)
\[ G_{EN} = 0.925 \left( \frac{1 + 1.7 (0.257) \sqrt{2} \times 0.572 \times (0.162 \times 0.065)}{1 + 1.7 (3.8) (0.257)} \right) \]

\[ = \left( \frac{0.8}{2.1985} \right) 0.925 \]

\[ = 0.82 \]

\[ G_{NS} = 0.925 \left( \frac{1 + 1.7 (0.257) \sqrt{2} \times 0.572 \times (0.162 \times 0.129)}{1 + 1.7 (3.8) (0.257)} \right) \]

\[ = \left( \frac{0.653}{3.485} \right) 0.925 \]

\[ = 0.84 \]

\[ G_{CR} = \pm 0.18 \quad \text{Enclosed (Table 36.11-1)} \]

\[ k_z \Rightarrow \text{Refer to table 37.3-1 for values, Exposure B} \]

**Velocity Pressures**

Sample calc. \( q_1 = 0.0002 \times h \times k \times 0.6 \times 10^{-2} \)

\[ = 0.0001 \times 0.5 \times 0.8 \times (0.05 \times 10^{-2}) \]

\[ \approx 13.67 \text{ psf} \]

The rest done in Excel

- 14.87 \text{ psf} \quad (30 \pm)
- 15.83 \text{ psf} \quad (35 \pm)
- 16.79 \text{ psf} \quad (30 \pm)
- 18.25 \text{ psf} \quad (40 \pm)
- 19.14 \text{ psf} \quad (50 \pm)
- 20.39 \text{ psf} \quad (60 \pm)
- 21.35 \text{ psf} \quad (70 \pm)
- 22.31 \text{ psf} \quad (80 \pm)
- 23.03 \text{ psf} \quad (90 \pm)
Design Wind Pressure: \( p = q_x C_p \)  

- Windward: \( C_p = 0.8 \)  
  \[ p = 0.67q_x \]  
  (up to roof 1)

- Leeward: \( C_p = 0.5 \)  
  \[ p = 0.67q_x \]  
  (up to roof 1)

**Sample Calculations:**

- Windward:
  \[ p = 0.67q_x \]  
  (up to roof 1)

- Leeward:
  \[ p = 0.67q_x \]  
  (up to roof 1)

---

\( q_x = q_x (\text{windward}) \)
\( q = q_m (\text{cheekward}) \)
\( q = q_n (\text{enclad}) \)

---

\( N = 5 \)  

\( N_s \)  

- E-W:
  \[ C_p = 0.8 \]  
  \[ C_p = 0.5 \]  
  \[ C_p = 0.7 \]  
  \[ C_p = 0.18 \]

---

1st Value: \( -0.9 \)  
2nd Value: \( -0.18 \)

---

Internal pressures usually cancel: ignore right side of \( p \) equation.
1776 Wilson Boulevard  
Arlington Virginia

Dr. Thomas Boothby  
September 23rd, 2011

---

Windward

<table>
<thead>
<tr>
<th>Wind</th>
<th>N-S</th>
<th>E-W</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15 F+</td>
<td>9.81 psf</td>
<td>8.97 psf</td>
</tr>
<tr>
<td>20</td>
<td>16</td>
<td>12.75</td>
</tr>
<tr>
<td>25</td>
<td>10.64</td>
<td>10.4</td>
</tr>
<tr>
<td>30</td>
<td>14.3</td>
<td>13.64</td>
</tr>
<tr>
<td>40</td>
<td>12.65</td>
<td>11.96</td>
</tr>
<tr>
<td>50</td>
<td>13.04</td>
<td>12.75</td>
</tr>
<tr>
<td>60</td>
<td>17.7</td>
<td>13.4</td>
</tr>
<tr>
<td>70</td>
<td>14.25</td>
<td>14</td>
</tr>
<tr>
<td>80</td>
<td>15</td>
<td>14.64</td>
</tr>
<tr>
<td>90</td>
<td>15.5</td>
<td>14.11</td>
</tr>
</tbody>
</table>

---

Story Forces (N-S)

\[ 3^{rd} (58.3^\circ) \implies 10.64 \left( \frac{25.3}{3} \right) (166.5) + 11.7 \left( \frac{12.2}{3} \right) (166.5) = 37.6 \text{ K} \]

\[ 3^{nd} (41.0^\circ) \implies 11.85 \left( \frac{25.3}{3} \right) (166.5) + 12.35 \left( \frac{12.2}{3} \right) (166.5) = 37.3 \text{ K} \]

\[ 4^{th} (53.3^\circ) \implies 13.66 \left( \frac{25.3}{3} \right) (166.5) + 13.77 \left( \frac{12.2}{3} \right) (166.5) = 39.4 \text{ K} \]

\[ 5^{th} (66.3^\circ) \implies 13.7 \left( \frac{25.3}{3} \right) (166.5) + 14.73 \left( \frac{12.2}{3} \right) (166.5) = 32.17 \text{ K} \]

\[ 10^{th} (88.3^\circ) \implies 15 \left( \frac{25.3}{3} \right) (166.5) = 18.3 \text{ K} \]

\[ (E-W) \]

\[ 2^{nd} \implies 10.4 \left( \frac{25.3}{3} \right) (373.42) + 11.01 \left( \frac{12.2}{3} \right) (373.42) = 61.4 \text{ K} \]

\[ 3^{rd} \implies 11.96 \left( \frac{25.3}{3} \right) (373.42) + 11.96 \left( \frac{12.2}{3} \right) (373.42) = 44.3 \text{ K} \]

\[ 4^{th} \implies 13.75 \left( \frac{25.3}{3} \right) (373.42) + 13.75 \left( \frac{12.2}{3} \right) (373.42) = 39.3 \text{ K} \]

\[ 5^{th} \implies 13.4 \left( \frac{25.3}{3} \right) (373.42) + 14 \left( \frac{12.2}{3} \right) (373.42) = 33.3 \text{ K} \]

\[ 10^{th} \implies 14.64 \left( \frac{25.3}{3} \right) (373.42) = 18.6 \text{ K} \]
Odd Leeward

N-S

3rd: 43.1 K
4th: 45.5 K
5th: 49.4 K
Reef: 97 K

E-W

3rd: 79.3 K
4th: 67.6 K
5th: 70 K
Reef: 97 K

N-S Base Shear: 322.5 K

E-W Base Shear: 344.2 K

N-S overturning moment = (64.3 x 26.7) + (43.1 x 41.67) + (45.5 x 55) + (49.4 x 6.3)

+ (97 x 83) = 11680 ft-lb

E-W overturning moment = (116 x 26.7) + (74.7 x 41.67) + (67.6 x 55)

+ (90 x 63.3) + (37.9 x 83) = 18370 ft-lb

Wind Loads for Penthouse

Sect. 29.5-1

\[ F = \frac{q_z}{g_z} \times C_L \times A_z \]

\[ q_z = x_w = 0.99 \ (100') \]

\[ F_{w3} = 23.75 (0.99) (993) \ (psf) = 261 \ psf \]

\[ F_{w3} = 22.75 (0.63) (65.70) \ (psf) = 14.05 \ psf \]

\[ C_z = \frac{D}{n} = \frac{95}{47.5} = 2 \]

\[ C_z = 1.93 \]

\[ A_z = 99 \ ft^2 \ (N-S) \]

\[ A_z = 570 \ ft^2 \ (E-W) \]
APPENDIX D

Seismic Loads
### Summary Tables

<table>
<thead>
<tr>
<th>Site Class</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ss</td>
<td>12.50%</td>
</tr>
<tr>
<td>S1</td>
<td>6%</td>
</tr>
<tr>
<td>Fa</td>
<td>1.6</td>
</tr>
<tr>
<td>Fv</td>
<td>2.4</td>
</tr>
<tr>
<td>Sms</td>
<td>0.2</td>
</tr>
<tr>
<td>Sm1</td>
<td>0.144</td>
</tr>
<tr>
<td>Sds</td>
<td>0.133</td>
</tr>
<tr>
<td>Sd1</td>
<td>0.096</td>
</tr>
<tr>
<td>Category</td>
<td>B</td>
</tr>
<tr>
<td>PGA</td>
<td>6%</td>
</tr>
<tr>
<td>Site Coefficient</td>
<td>1.6</td>
</tr>
</tbody>
</table>

| R  | 3 |
| Ω  | 3 |
| Cd | 2.5 |
| Cs | 0.032 |

<table>
<thead>
<tr>
<th>Story</th>
<th>Weight (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>4296</td>
</tr>
<tr>
<td>5th</td>
<td>3611</td>
</tr>
<tr>
<td>4th</td>
<td>3723</td>
</tr>
<tr>
<td>3rd</td>
<td>5219</td>
</tr>
<tr>
<td>2nd</td>
<td>5419</td>
</tr>
<tr>
<td>Total</td>
<td>22268</td>
</tr>
<tr>
<td>Base Shear</td>
<td>712.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Story</th>
<th>Force (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>235</td>
</tr>
<tr>
<td>5th</td>
<td>155.3</td>
</tr>
<tr>
<td>4th</td>
<td>122.4</td>
</tr>
<tr>
<td>3rd</td>
<td>121.5</td>
</tr>
<tr>
<td>2nd</td>
<td>77.7</td>
</tr>
</tbody>
</table>

<p>| Overturning Moment (ft-k) | 44110 |</p>
<table>
<thead>
<tr>
<th>Site Class: D</th>
<th>Seismic</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Site Class: D</th>
<th>(Table 9.3-1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_2 = 12.5%$</td>
<td>(Fig. 9.2-1)</td>
</tr>
<tr>
<td>$S_1 = 6%$</td>
<td>(Fig. 9.2-2)</td>
</tr>
</tbody>
</table>

$F_a = 1.6$  
$F_r = 2.4$  
(Table 11.4-1, Site class D $S_2 \leq 0.25$)  
(Table 11.4-2, Site class D $S_1 \leq 0.1$)

$S_{15} = F_a S_3 = 0.2$  
$S_{05} = 0.133$  
$S_{11} = F_r S_1 = 0.144$  
$S_{01} = 0.096$

Table 11.6-1: $S_{05} < 0.167$ => Risk Category $I$ => Category $A$  
Table 11.6-2: $0.067 < S_{11} < 0.133$ => Risk Category $II$ => Category $B$

$PGA = 6\%$ (Fig. 12.7-7)  
Site Coefficient: $F_{p0} = 1.6$ (Table 11.8-1, $PGA \leq 0.11$, Site Class D)

Response Modification Coefficient: $R$ (Table 10.3-1)  
Floors 1-3 => Ordinary Rein. Concrete Shear Walls $R = 5$  
Floors 4-5 => Ordinary Rein. Concrete Moment Frame $R = 3$

Over-strength Factor: $\xi_a$ (Table 12.9-1)  
Floors 1-3 $\Rightarrow \xi_a = 0.5$  
Floors 4-5 $\Rightarrow \xi_a = 3$

Description Amplification Factor: $C_0$ (Table 12.9-1)  
Floors 1-3 $\Rightarrow C_0 = 4.1$  
Floors 4-5 $\Rightarrow C_0 = 2.9$

<table>
<thead>
<tr>
<th>Controlling Values</th>
<th>Seq. 10.2.3.1 =&gt; Upper system R is lower than lower system R, use upper system values for both systems.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R = 3$</td>
<td></td>
</tr>
<tr>
<td>$\xi_a = 3$</td>
<td></td>
</tr>
<tr>
<td>$C_0 = 2.5$</td>
<td></td>
</tr>
</tbody>
</table>
Use Equivalent Lateral Force Method

Eqn. 12.8-1 $\Rightarrow V = C_s W$

Seismic Response Coefficient: $C_s$ (Ref to Sec 12.8.1)

\[
C_s = \frac{505}{(1.10)^2} = 0.133 \times \frac{1}{1.10} = 0.1443 \quad \text{(Moment Frames)}
\]

$T_e = \frac{5}{4} \quad C_e = 0.985$

$T_e = C_e h_n$

$h_n = 97.33$ (From lowest point to penthouse)

\[
C_e = 0.985 \quad \text{Table 10.8-2}
\]

$x = 0.93$

$T$ cannot exceed $C_u T_a = 1.7 (0.985) = 1.675$

$C_u = 1.7$ (Table 10.8-1: $S_{Dh} \leq 0.1$)

$T \leq T_e \Rightarrow C_s$ should not exceed: $C_s = \frac{505}{T(\frac{3}{4})} \times 0.985(0.93)$

\[
\text{Use } C_s = 0.032
\]

$W = \text{effective seismic weight}$

Roof $D_L = \left(\frac{12 \times 150}{12}\right) + 24 \text{ psf} + 8.0 \text{ psf} = 180 \text{ psf}$

Roof consists of 10" to 12" thick slabs, use 12" to be conservative.

Roof snow load $< L \Rightarrow$ use $L_r = 30 \text{ psf}$

Floor dead loads $\geq 8\"$, 10", 12", 14", 16"

\[
\left(\frac{8\" \times 150}{12}\right) + 15 = 115 \text{ psf} \quad 12\" = 165 \text{ psf} \quad 14\" = 215 \text{ psf}
\]

18 psf is extra assumed allowance.
Break Down of approximate areas for slabs by floor

1st Floor

14' x 16' x 7'
14' x 8' x 7'
16' x 7' x 7'
16' x 8' x 7'

1st floor includes:
14' x 16' x 7' = 1467.4 ft^3
16' x 7' x 7' = 1380 ft^3
16' x 8' x 7' = 3096.4 ft^3
8' x 7' x 7' = 309.64 ft^3

2nd and 3rd floor:

13' x 13' x 7'
13' x 8' x 7'
13' x 8' x 7'

2nd and 3rd floor includes:
13' x 13' x 7' = 487.5 ft^3
13' x 8' x 7' = 310.74 ft^3
8' x 7' x 7' = 309.64 ft^3

4th Floor = Same as 2nd & 3rd but there is a green roof on left half

Non Green Roof

13' x 13' x 7' = 2212 ft^3
8' x 7' x 7' = 309.64 ft^3

5th Floor = Same as non green roof portion of 4th floor
Wall DL => assume 30 psf for thin brick veneer or cast in place concrete panels and use 20 psf only to be conservative and to simplify calculations

Roof Load

\[ W_{RF} = 19994(180) + 887.8(0.33)(180) + 16(180) + 25.617(10)(180) \]
\[ = 42966 \text{ K} \]

Floor Load

1st: 23712(165) + 53349(115) + 6791.8(10.5)(20) + 338
\[ = 36111 \text{ K} \]

2nd: 49975(165) + 31978(115) + 889.8(10.9)(30) + 362.8
\[ = 5219 \text{ K} \]

3rd: 49756(165) + 31978(115) + 889.8(10.9)(30) + 362.8
\[ = 5419 \text{ K} \]

Total DL = 392546 K

\[ V= c_s w \]
\[ = 0.033 (392546) \]
\[ = 7124.6 \text{ K base shear} \]

Additional Structural Weight

Columns 2nd => 20 x 20: 3.365 ft x 51 columns = 17155 => 2235 cubic ft
\[ 13' x 30' = 3.5 \text{ ft} \times 4 \text{ columns} = 10 ft \Rightarrow 133.3 \text{ ft}^2 \]
\[ = 343.8 \text{ K} \]

3rd => 343.8 + 20 = 363.8 K

4th: 3.365 x 36 columns => 1215 => 1613 ft^2
\[ 243 \times 30 \]
\[ = 363 \text{ K} \]

5th => 1015 x 14.672 = 1775 ft^2
\[ 1.5 x 4 \times 14.672 = 47 \text{ ft}^2 \]
\[ = 388 \text{ K} \]
Distribute Forces

\[ F_x = C_{Vx} V = \frac{w_h x^2}{h^2} \]

Sample Calc

\[ K = \text{interpolate between } 1.0 \]

\[ \frac{C_{Vx} = 0.55}{0.5} \]

\[ 2x - 1 = 0.485 \]

\[ x = 1.2 \]

Roots: \[ 4096(53)^{1.24} \]

\[ +36.11(65.33)^{1.24} + 0.028(83)^{1.24} \]

\[ \rightarrow \text{roof level} \]

\[ = \frac{1.03e^6}{3.9e^6 + 5.32e^6 + 5.36e^6 + 6.1e^6 + 1.03e^6} = 0.33 \]

Rest of the calc. done in Excel.

5th \[ \approx 0.318 \]

4th \[ \approx 0.179 \]

3rd \[ \approx 0.171 \]

2nd \[ \approx 0.109 \]

\[ \varepsilon = 1.0 \]

Story Forces

\[ F_8 = 0.33(712.6) = 235 \text{ kN} \]

\[ F_6 = 155.3 \text{ kN} \]

\[ F_7 = 132.4 \text{ kN} \]

\[ F_5 = 131.5 \text{ kN} \]

\[ F_4 = 111.7 \text{ kN} \]

Overturning Moment

\[ (335 \times 53) + (156.7 \times 63.3) + (130.4 \times 55) + (131.5 \times 91.6) + (71.7 \times 28.3) = 44110 \text{ ft-lb} \]
APPENDIX E

Snow Loads
Josh Urban | Ae Senior Thesis | Snow

Flat Roof Snow Load: $p_f = 0.7C_eC_pI_p$  
$C_e = \text{Category B, Fully Exposed} = 0.9$ (Table 7.2)  
$I_p = 1.0$ (Table 7.3)  
$T = 1.0$ Risk Category II (Table 15.2)  
$p_g = \text{Ground Snow Load} = 0.25 \text{psf}$ (Figure 7.1)

$p_f = 0.7(0.9)(1.0)(1.0)(0.25) = 0.75 \text{psf}$

Snow Drift

Roof 1 = 55° (From level elevation)  
Roof 2 = 53°  
Roof 3 = 97.33° (Parapet)  
$h_e = 0.13p + 14 = 17.35 \frac{p_f}{\gamma} = 0.913 \frac{h_e}{h_b} = 0.913 \begin{array}{c} 0.913 \end{array}

Roof 2-3

Roof 3-2

Need to apply drift loads

Supplementary Load Due to Drifting

Balanced Snow Load
Roof 3 to roof 2

\[ \theta_c = 49.5 \]

\[ h_d = 0.43 \sqrt{17.5 \sqrt{35+10} - 1.5} \]

\[ W = 40 \]

\[ u = (2.39) \]

\[ = 9.15 \]

\[ p_d = h_d \gamma \]

\[ = 2.99 \times (17.35) \]

\[ = 53.3 \text{ psf} \]

Roof 2 to roof 1

Only N-S windward drift

\[ \theta_c = 173.15 \]

\[ h_d = 0.43 \sqrt{173.5 \sqrt{35+10} - 1.5} \]

\[ = 4.23 \]

\[ h_d < h_c \]

\[ W = 40 \]

\[ u = (4.23) \]

\[ = (4.23) \times (17.35) \]

\[ = 74.17 \text{ psf} \]
APPENDIX F

Spot Checks
Column at D-6 : Design  

Tributary Area = 30 x 30 = 900 ft\(^2\) \(\text{Net Area} = 4(900) = 3600 > 400\) 

\(\text{Can be reduced}\)

1st Floor: LL Reduction = 0.35 \(\frac{15}{\sqrt{3600 \times 1.5}}\) = 0.375 \(\Rightarrow\) Use 0.4

\(P_a = 0.4(80 + 20)(4)(900) = 1440 \text{ kN}\)

\(P_a = 30(900) = 27 \text{ kN}\)

(\(L_2 > \text{Snow Load so use } L_R\))

\(P_{LR} = \left[\left(\frac{16}{15}\right)(150) + \frac{1}{8}(900) + \left(\frac{8}{5}(150) + 15\right)(900)\right] = 555.3 \text{ kN}\)

\(P_a = 1.2D + 1.6L + 0.5s\)

\(P_a = 1.2(555.3) + 1.6(144) + 0.5(27)\)

\(P_a = 910 \text{ kN}\)

\(M_u = 503 \text{ kN}\)

\(W_u = 1.2D + 1.6L\)

\(= 1.2(118 + 30) + 1.6(100x30)\)

\(= 394 \text{ kN}\)

\(M_u = \frac{W_u \times 30^2}{16} = 503 \text{ kN} \times 30 \times 30\)

\(= 894 \times 30\)

\(= 503 \text{ kN} \times 30\)

\(\Rightarrow\) don't know in 30 use 40 to be conservative

\(f'c = 5.85\)

\(f_u = 60 \text{ kN} \times \text{m}^2\)

\(\phi = 0.015 \Rightarrow \text{assumed value}\)
\[ A_q \ (\text{in}^2) = \frac{P_u}{0.4 (G_2 + \rho g)} = \frac{910}{0.4 (5 + 60 \times 0.05)} = 384 \text{ in}^2 \]

Try 20 \times 20

\[ e = \frac{M_u}{P_u} = \frac{563}{910} = 0.62 \text{ ft} = 7.46 \text{ in} \]

\[ e = \frac{6.6}{20} = 0.33 \]

\[ Y' = \frac{20 - 2(0.5)}{30} = 0.75 \]

Determine \( q_g \) from interaction dia-graphs

\[ \phi = \frac{P_u}{A_q} = \frac{910}{20 \times 20} = 2.28 \text{ ksi} \]

\[ \phi = \frac{M_u}{A_q h} = \frac{563 \times 10}{20 \times 20 \times 0.5} = 0.75 \]

\( q = 0.043 \) = economical values for \( q_g \) range from 1-9%

Choose larger cross section

Try 30 \times 30

\[ Y' = \frac{30 - 2(0.5)}{30} = 0.77 \]

Interpolate

\[ Y = 0.75 \Rightarrow q = 0.026 \]

\[ Y = 0.9 \Rightarrow q = 0.033 \]

\[ P_u = \frac{910}{20 \times 20} = 1.88 \]

\[ M_u = \frac{563 \times 10}{20 \times 20 \times 0.5} = 0.63 \]

Try larger size
Try $g/\phi y = 0.79$

$\frac{P_n}{A_g} = \frac{910}{2624} = 1.88$

$M_y = \frac{503 \times 12}{4 \times 10^3} = 0.53$

Interpolate:

$\phi = 0.79 \Rightarrow \frac{P_n}{A_g} = 0.015$

$\phi = 0.9 \Rightarrow \frac{P_n}{A_g} \approx 0.0025$

$P_n = 0.015 - (0.015 - 0.0025) \cdot \frac{0.04}{0.15}$

$P_n = 0.014 \checkmark$

Select reinforcement:

$A_{st} = \frac{P_n A_g}{\phi}$

$= 0.014 (34 \times 34)$

$= 8.06 \text{ in}^2$

$A \geq 11 = 6 (1.56) = 9.36 \text{ in}^2 \checkmark$

$8 \times 10 = 8 (1.07) = 8.56 \text{ in}^2$

$\phi P_{n,\text{max}} = 0.8 \left[ \phi (0.85 + \beta (A_g - A_{st})) + \gamma (A_{st}) \right]$

$= 0.8 (0.85) \left[ 0.85 (5/12 - 1.36) + 60 (0.36) \right]$

Contraction

Controlled

$= 1544 \text{ k} > 910 \text{ k} \checkmark$

$\phi P_n > P_n$
Spot Checks:

- 30° + 30° + 30° = 90°

Dead Loads:
- Slab = (2/3) x 150 = 100 psf
- Superimposed = 15 psf
- Drop panel = 10' x 15' x 8'

Live Loads:
- 100 psf

Materials:
- F'c = 6000 psi
- F'f = 6000 psi
- F'p = 370,000 psi
- 7 wire strand 3/8" diameter

A = 0.153 in²

Estimated prestress losses = 15 ksi (assumed)

F'c = 0.7(370) - 15 = 174 ksi

P'cut = A/F'c = 36.6 ksf per tendon

Slab Thickness:

ACI table 9.6 (c)

Ext. Panel: 30 x 12/73 = 7.57'
Int. Panel: 20 x 12/76 = 6.67'

8" x 8" = drop panel to drop panel

8" = drop panel to drop panel

10° - 30° - 10°
Class U (ACI 18.3.3)
\[ A = bh = (30 \times 10) \times (8) = 2400 \text{ in}^2 \]
\[ \frac{L}{c} = \frac{bh^3}{6} = \frac{(30 \times 10 \times 8)^3}{6} = 3840 \text{ in}^3 \]

Available Stresses

At time of jacking (ACI 18.4.1)
\[ f'_{c1} = 3000 \text{ psi} \]
Compression = \( 0.6 f'_{c1} \)
\[ = 0.6 \times 3000 = 1800 \text{ psi} \]
Tension = \( 0.5 f'_{c1} \)
\[ = 0.5 \times 3000 = 1500 \text{ psi} \]

At service loads (ACI 18.4.1.6)
\[ f'_{c} = 6000 \text{ psi} \]
Compression = \( 0.45 f'_{c} \)
\[ = 0.45 \times 6000 = 3750 \text{ psi} \]
Tension = \( 0.5 f'_{c} \)
\[ = 0.5 \times 6000 = 4650 \text{ psi} \]

Assume 60% target load balance & slab self weight
\[ 0.6 \times (100) = 60 \text{ psf} \]

Tendon locations

Ext. Support = 45°
Iron Support top = 7°
Iron Span bottom = 1°
End Span bottom = 1.75°

\[ a_{int} = 7/1 = 6'' \]
\[ a_{ext} = \left( \frac{45}{3} \right) = 1.75 = 3.75'' \]
Assume end span governs.
\[ L_b = 0.6 \times 60 = 36 \text{ in} \]

\[ P = \frac{L_b L}{8 \pi r_d^3} = \frac{1800 \times 36}{8 \left(\frac{1}{3}\right)} = 648 \text{ kN} \]

\[ n = \frac{648}{26} = 24.92 \Rightarrow 24 \text{ tendons} \]

Actual Force = 24 \times 26.4 \approx 633.4 \text{ kN}

Adjust \( L_b \) for actual force

\[ L_b' = \frac{633.4}{648} = 0.985 \times (1800) = 1770 \text{ kN} \]

Determine Precr Emerson Stress

\[ \sigma_{\text{Emerson}} = \frac{633.4 \times 1800}{8 \times 30} = 331.7 \text{ psi} > 105 \text{ psi (min)} \]

\[ L_b = \frac{633.4 \times 60}{30} = 1064 \text{ kN/m} \]

\[ \frac{L_b'}{L_b} < 100 \% \Rightarrow \frac{24.92}{1064} = 0.0235 \]

\[ \frac{L_b'}{L_b} < 100 \% \Rightarrow \frac{24.92}{1064} = 0.0235 \]
**Dead Load Moments**

\[ w_{DL} = 30 \left( \frac{160}{1600} \right) = 3 \text{ kips} \]

**Live Load Moments**

- Reduction: \( \frac{3.5}{18} = 0.75 \)
- \( L_1 = 100 \times 0.75 = 75 \text{ psf} \)
- \( w_{HL} = 32 \left( \frac{19}{1000} \right) = 0.35 \text{ kips} \)

**Total Balancing Moments**

\[ 1.8 + 0.35 + 1.8 = 3.95 \text{ kips} \]
Check Stresses

Stage 1: Immediately after pouring

\[ f_{\text{si}} = \frac{(-67.5 + 45.5 \times 1000)}{3840} \]
\[ f_{\text{si}} = -28 \text{ psi} < 1800 \text{ psi} \checkmark \]
\[ f_{\text{si}} = \frac{(-28 + 191.3 \times 1000)}{3840} \]
\[ f_{\text{si}} = -16 \text{ psi} < 1800 \text{ psi} \checkmark \]

Stage 2: After service load

\[ f_{\text{si}} = \frac{(-67.5 - 50.6 + 45.5 \times 1000 \times 0.3)}{3840} \]
\[ f_{\text{si}} = -43.9 \text{ psi} < 3700 \text{ psi} \checkmark \]
\[ f_{\text{si}} = \frac{(-67.5 + 50.6 - 45.5 \times 1000 \times 0.3)}{3840} \]
\[ f_{\text{si}} = -3.18 \text{ psi} < 3700 \text{ psi} \checkmark \]

Support: \[ f_{\text{si}} = \frac{(316 - 191.3 \times 1000 \times 0.3)}{3840} \]
\[ f_{\text{si}} = -9.16 \text{ psi} < 3700 \text{ psi} \checkmark \]
\[ f_{\text{si}} = \frac{(316 + 191.3 \times 1000 \times 0.3)}{3840} \]
\[ f_{\text{si}} = 476 \text{ psi} \]

Stresses are not within permissible code limits.