TECHNICAL REPORT ONE

1776 Wilson Boulevard

Arlington Virginia

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Structural Option

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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.

Fig. 1 Lobby Rendering

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.


Fig. 2 Green Roof Rendering

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 foot print 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.

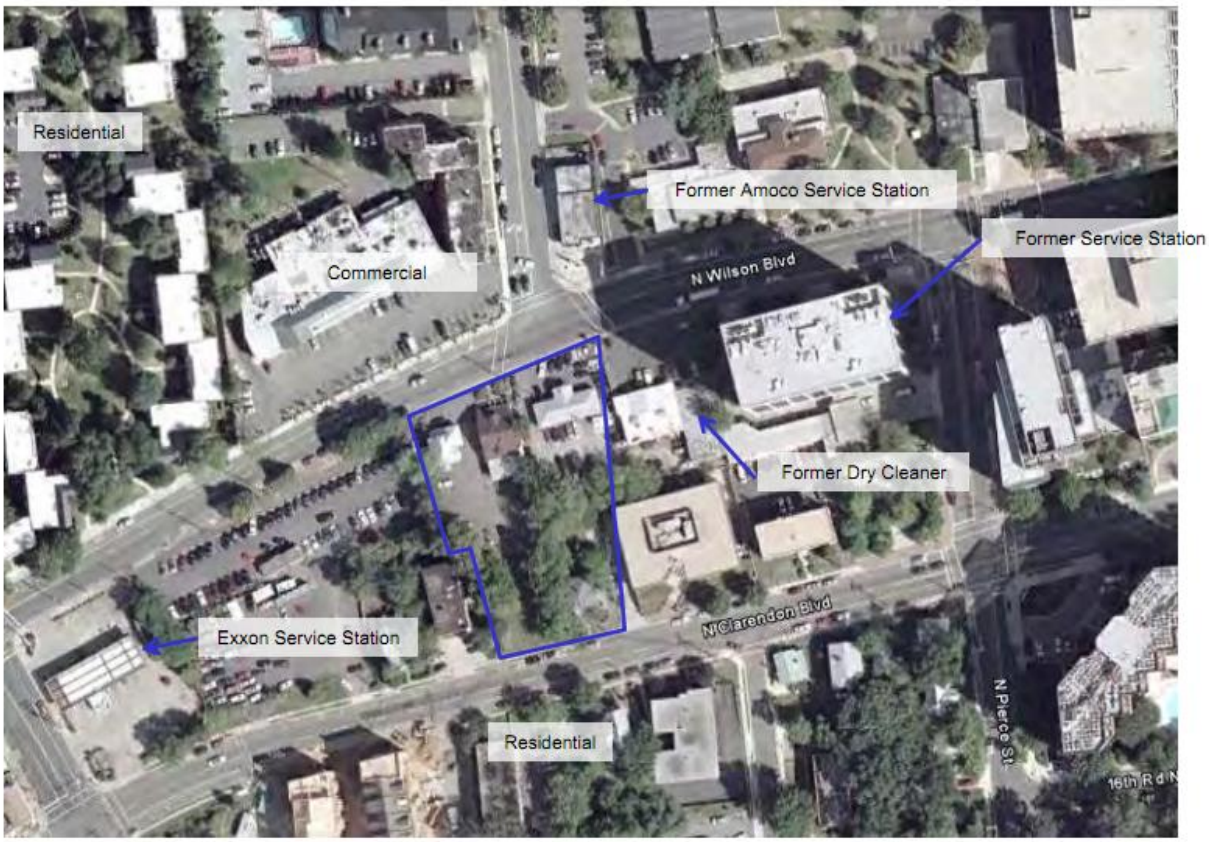


Fig. 3 Aerial View of Site

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 stratums:

Stratum	Name	Description
I	Fill/Possible Fill	17-36 feet below site grades consisting of various amounts of sand, gravel, and clay
II	Natural Alluvial/Marine Solids	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
III	Residual Soils/Weathered Rock	Below stratum 2 and consists of Micaceous silty sand with rock fragments.

Table 1 Soil Stratums

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.

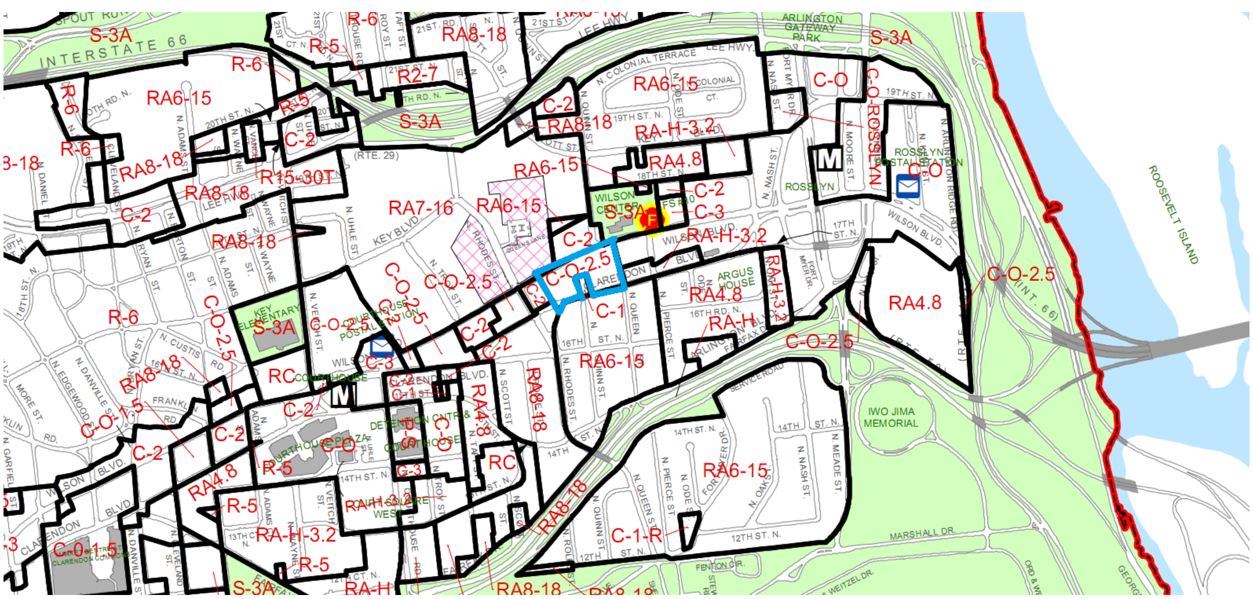


Fig. 4 Zoning Map for Arlington - The blue outline marks the district where 1776 Wilson is to be located

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.

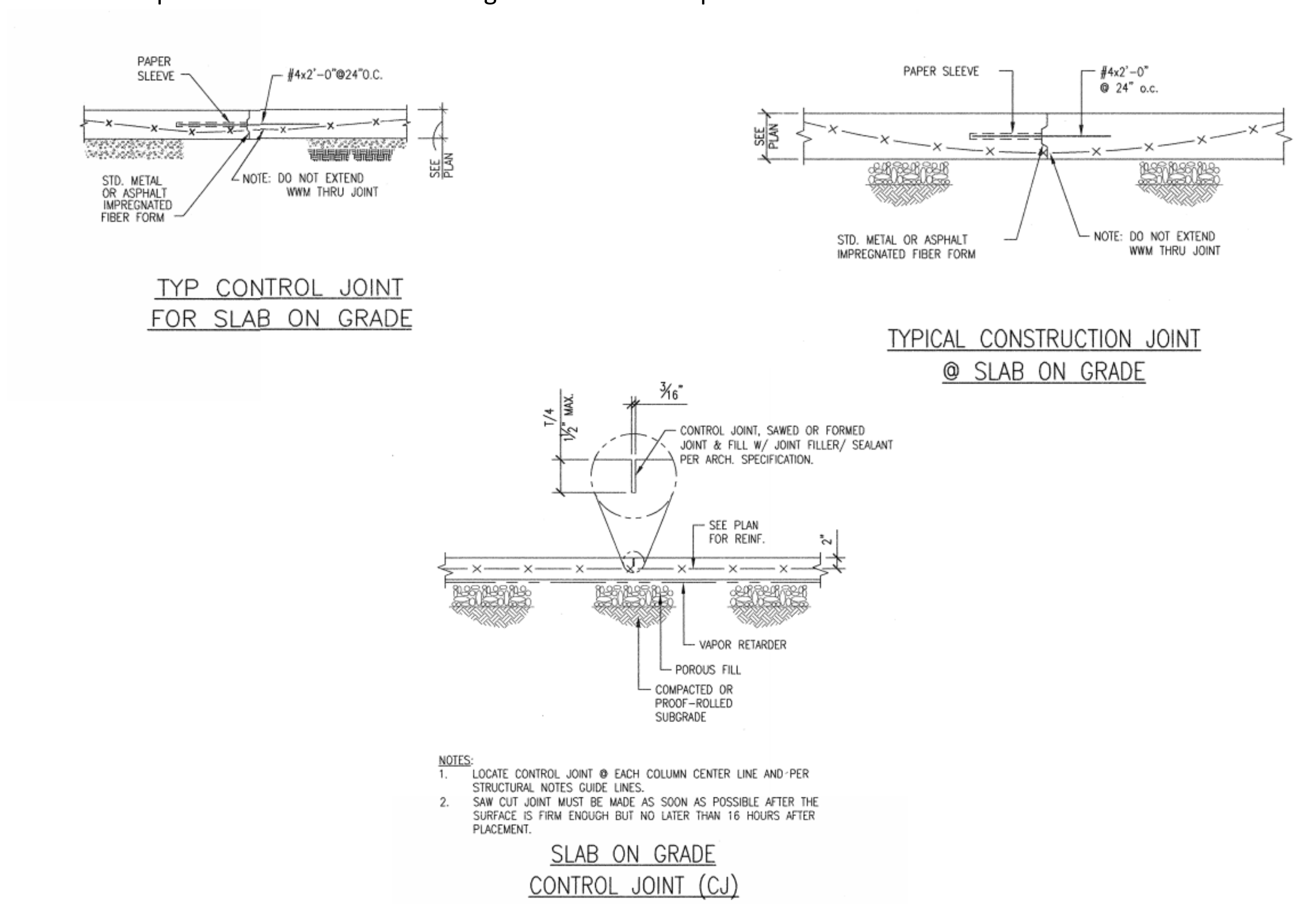


Fig. 5 Slab on Grade Control Joint

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.

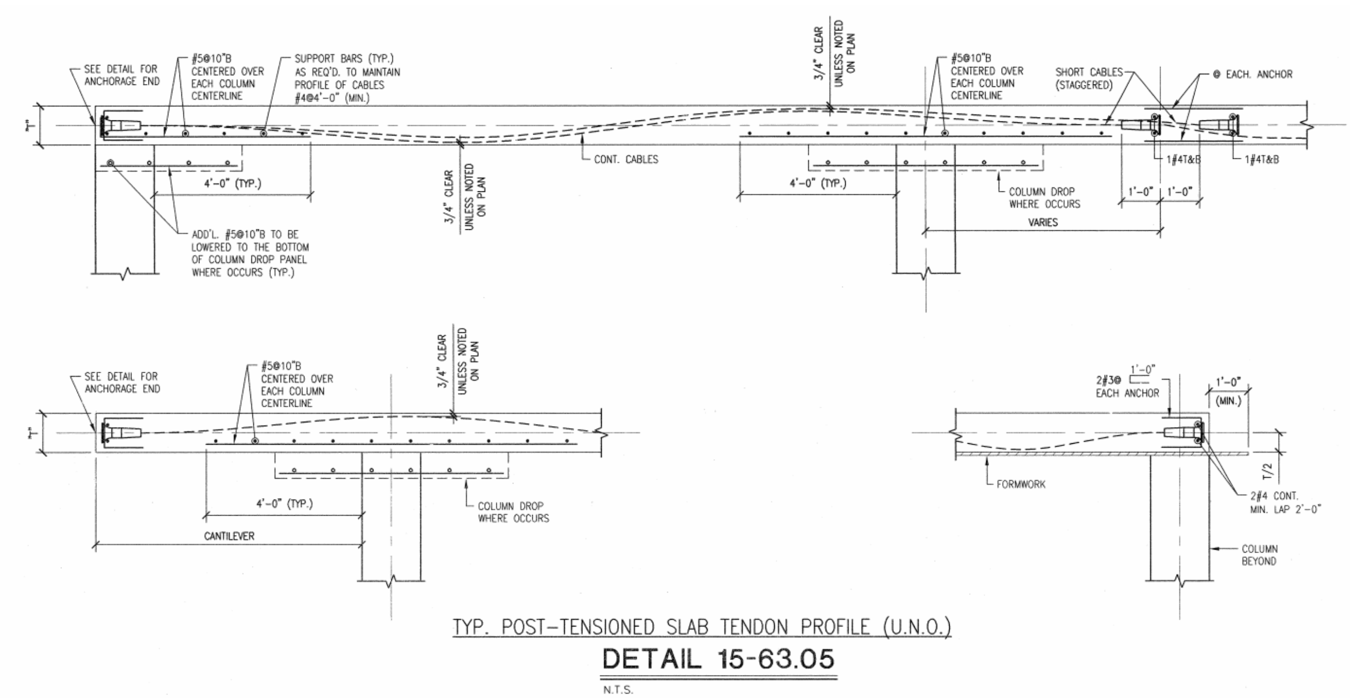


Fig. 6 Typical Post Tensioned Slab Tendon Profile

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.

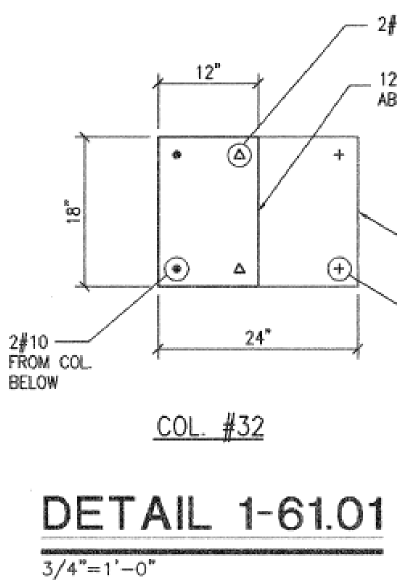
Columns

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.

Floor	Sizes	Reinforcement	Compressive Strength (ksi)
5th	22x22, 12x30	4#10, 8#11, 4#9	Typically 5, some columns are 6
4th	22x22, 12x30	4#10, 8#10, 4#9	Typically 5, some columns are 6
3rd	22x22, 12x30	4#9, 4#10, 4#11, 8#10, 8#11	Typically 5, some columns are 6 and 8
2nd	22x22, 12x30	4#10, 4#11, 8#10, 12#11, 6#9	Typically 5, some columns are 6
1st	24x24, 12x30, 24x29 ¾*	4#11, 8#9, 8#10, 8#11 12#11,	Typically 8, some columns are 10
Basement Levels	24x24, 12x30, 32x18, 24x18, 12x18*	4#11, 12#11, 8#11, 4#10, 6#9, 8#9	Typically 8 at the B1 level, 6 below, some columns are 10

Table 2 Column Schedule Summary

*see following details



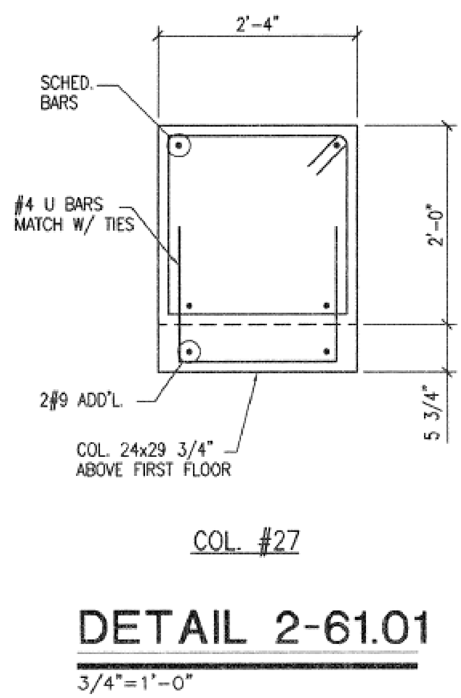


Fig. 7 Column Details

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
- IBC 2006 International Building Code published by the International Code Council, Inc

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.

Structural Element	Strength
Footings, walls, and grade beams	F'c = 4ksi
Framed floors, precast concrete units, and slab on grade	F'c=5ksi
Columns	F'c=5,6,8, and 10ksi
Light weight concrete	F'c=3ksi
Reinforcement steel	ASTM-A615, Grade 60
Welded wire mesh	ASTM-A185

Table 3 Materials

Post Tensioned Concrete – tendons consist of steel strands that conform to ASTM A-416, Fpu=270,000 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 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.

Occupancy	Design	ASCE 7-10
Office lobbies 1st floor corridors	100 psf	100 psf
Offices	50 psf	50 psf
Corridors above first floor	80 + 20 psf for partitions	80 psf
Roof	30 psf	20 psf
Stairs and exit ways	100 psf	100 psf
Storage	125 psf	125 psf
Fitness center	100 psf	100 psf

Table 4 Live Load Summary

Floor	Design Load
Normal weight concrete	150 pcf
MEP/ceiling	15 psf
Drop panels	Same as normal weight concrete

Table 5 Floor Dead Loads

Roof	Design Load
Normal Weight Concrete	150 pcf
Solar panels and racking system	6.6-8 psf
Roof paver, insulation, and waterproofing	24 psf

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.

Snow Load Criteria	Value
Exposure Factor	Ce = 0.9
Thermal Factor	Ct = 1.0
Importance Factor	Is = 1.0
Ground Snow Load	Pg = 25 psf
Flat Roof Snow Load	Pf = 15.75 psf
Snow Density	17.25 lb/ft^3

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.

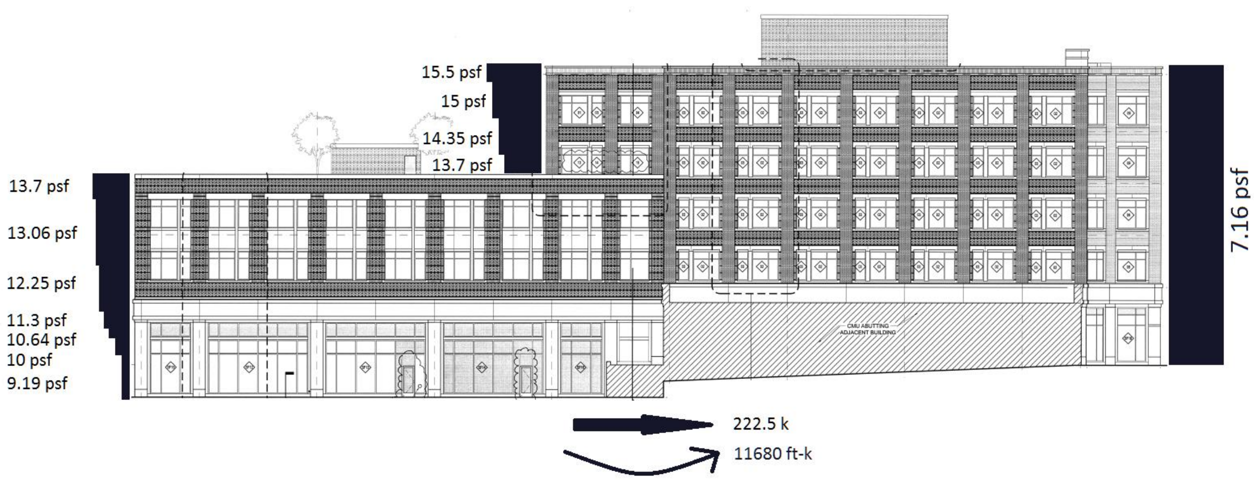


Fig. 8 N-S Elevation With Wind Data

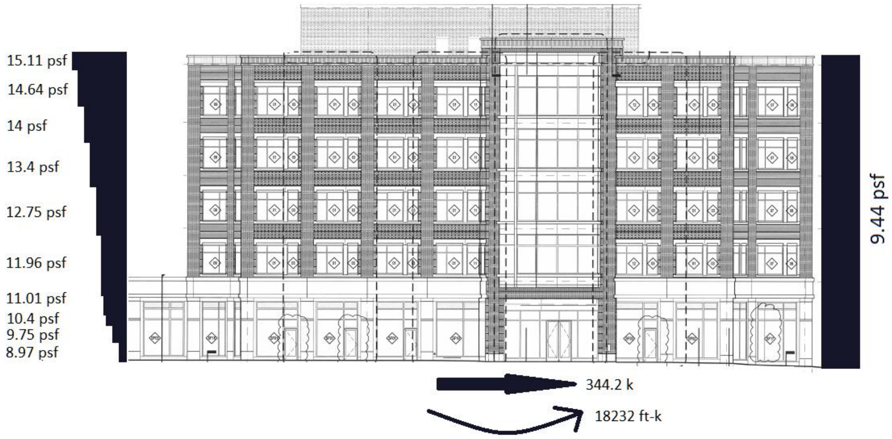


Fig. 9 E-W Elevation With Wind Data

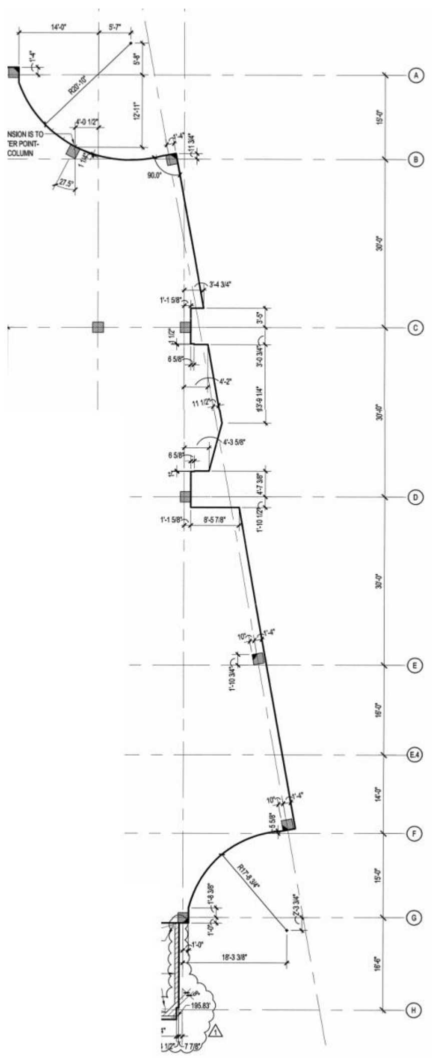


Fig. 10 North Facade In Plan View

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.

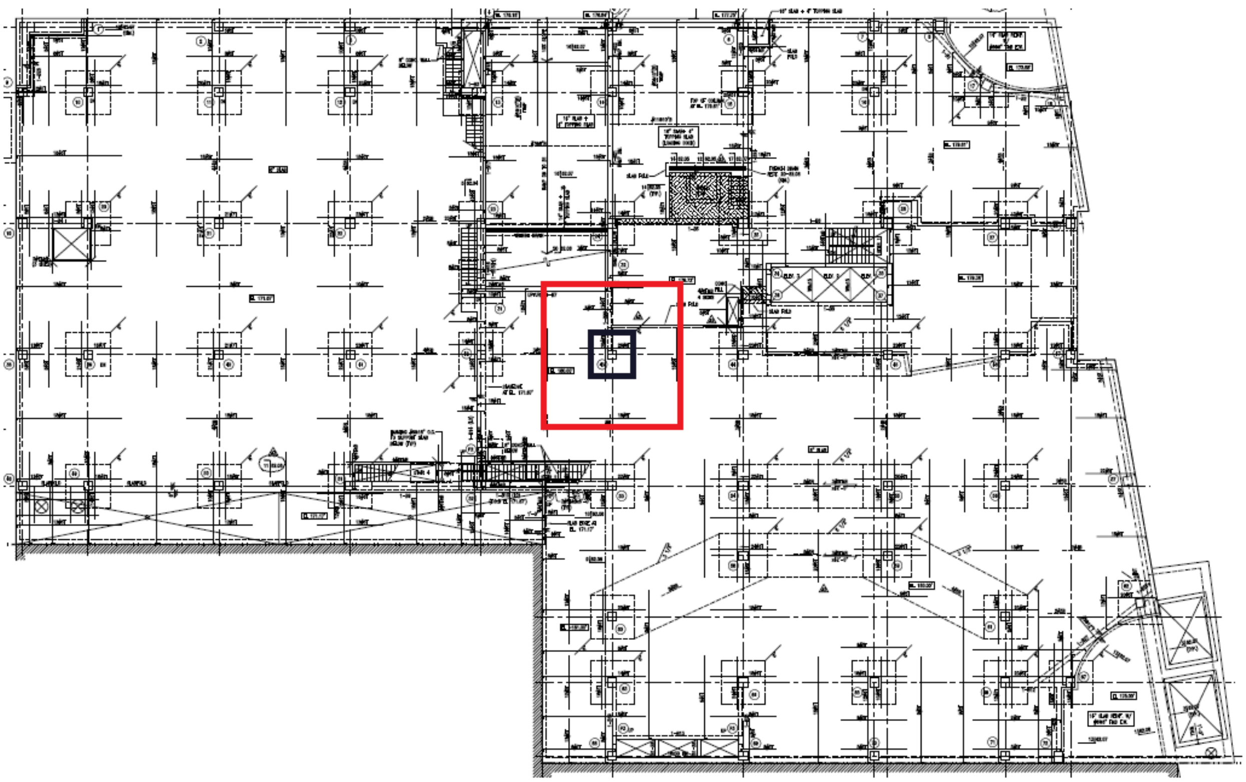


Fig. 12 First Floor Framing Plan - column d-6 is called out as well as the tributary area

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.

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

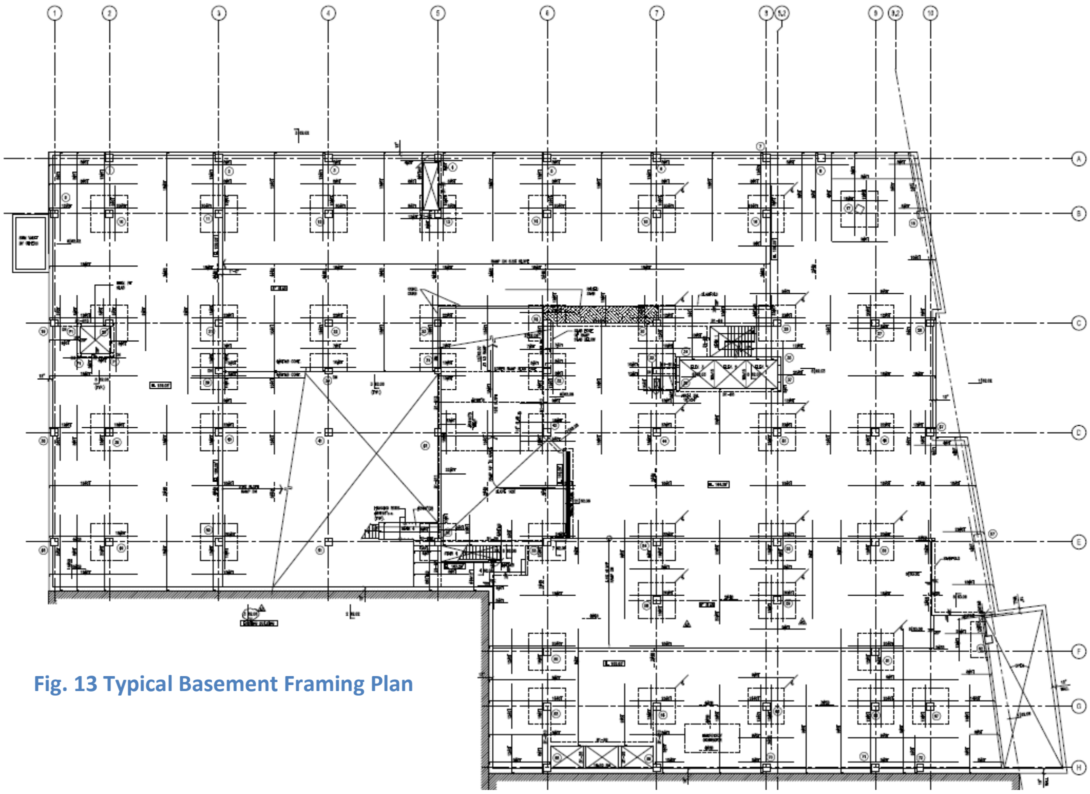


Fig. 13 Typical Basement Framing Plan

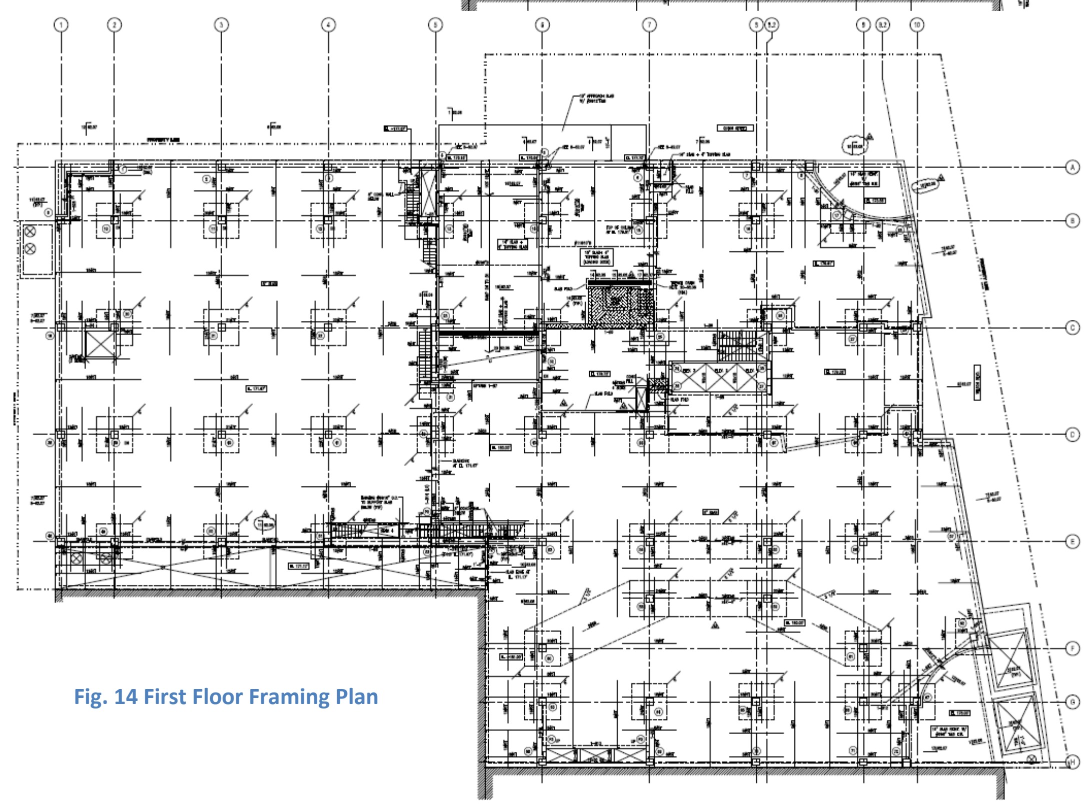


Fig. 14 First Floor Framing Plan

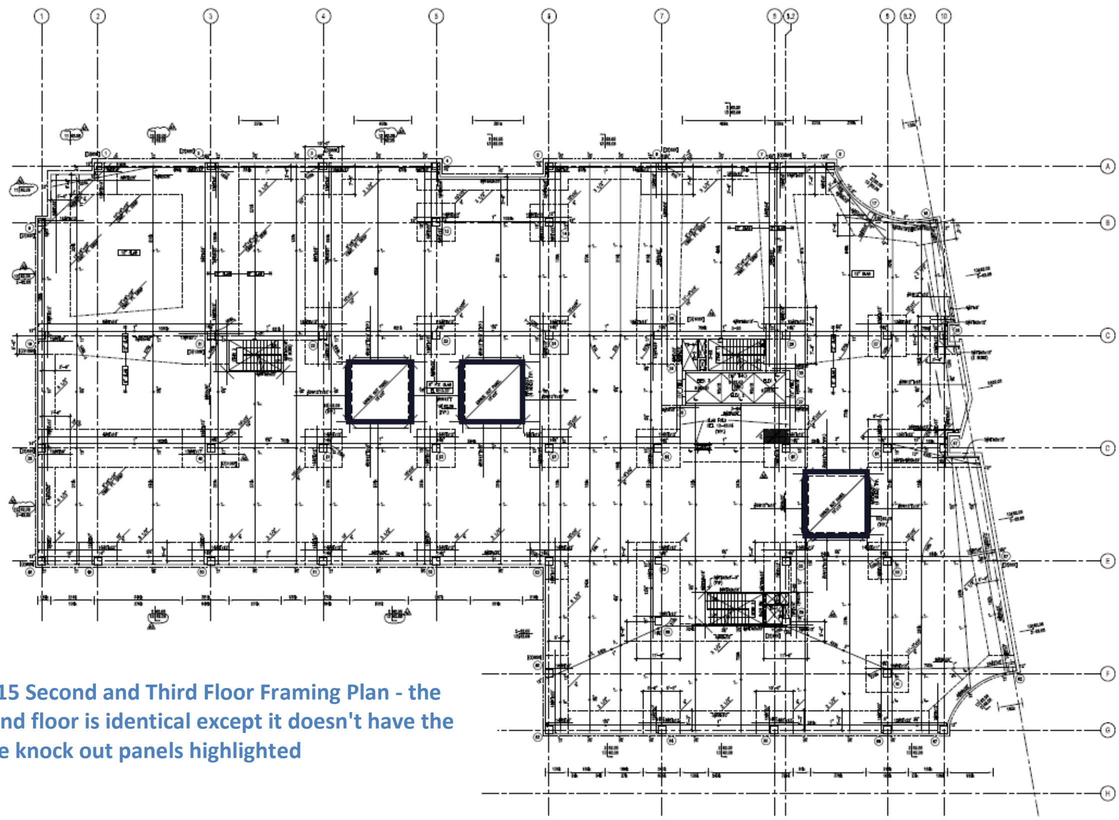


Fig. 15 Second and Third Floor Framing Plan - the second floor is identical except it doesn't have the three knock out panels highlighted

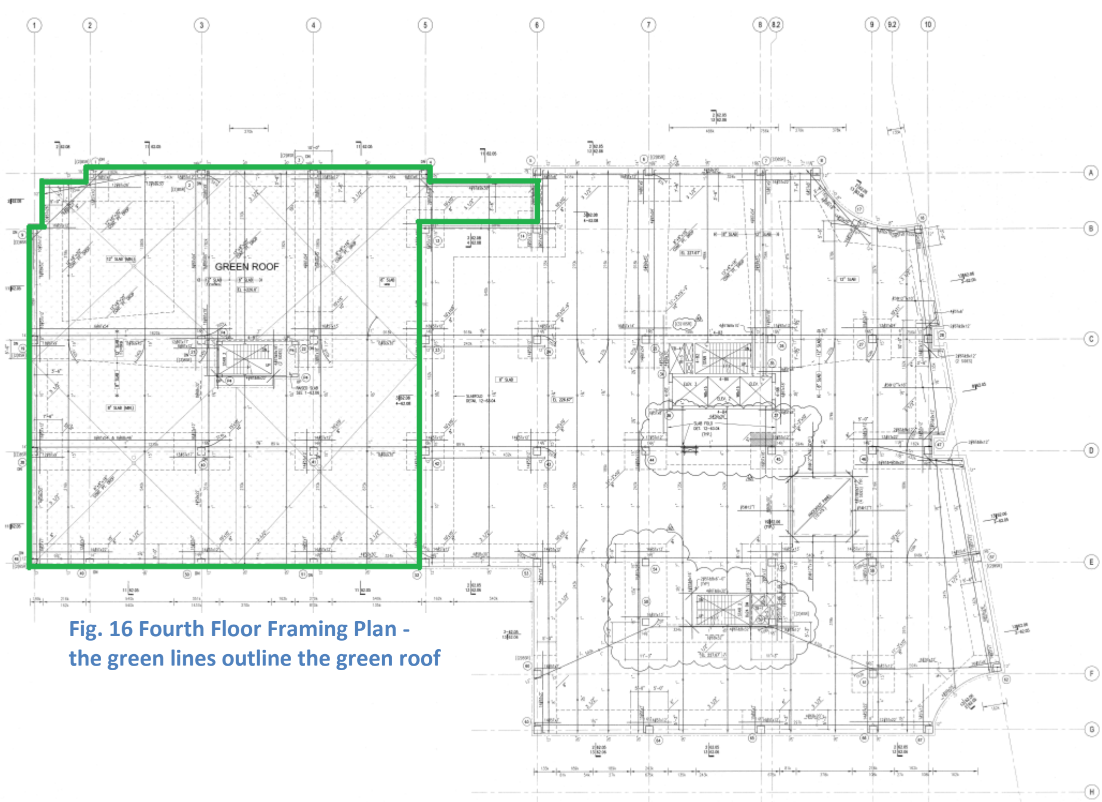


Fig. 16 Fourth Floor Framing Plan - the green lines outline the green roof

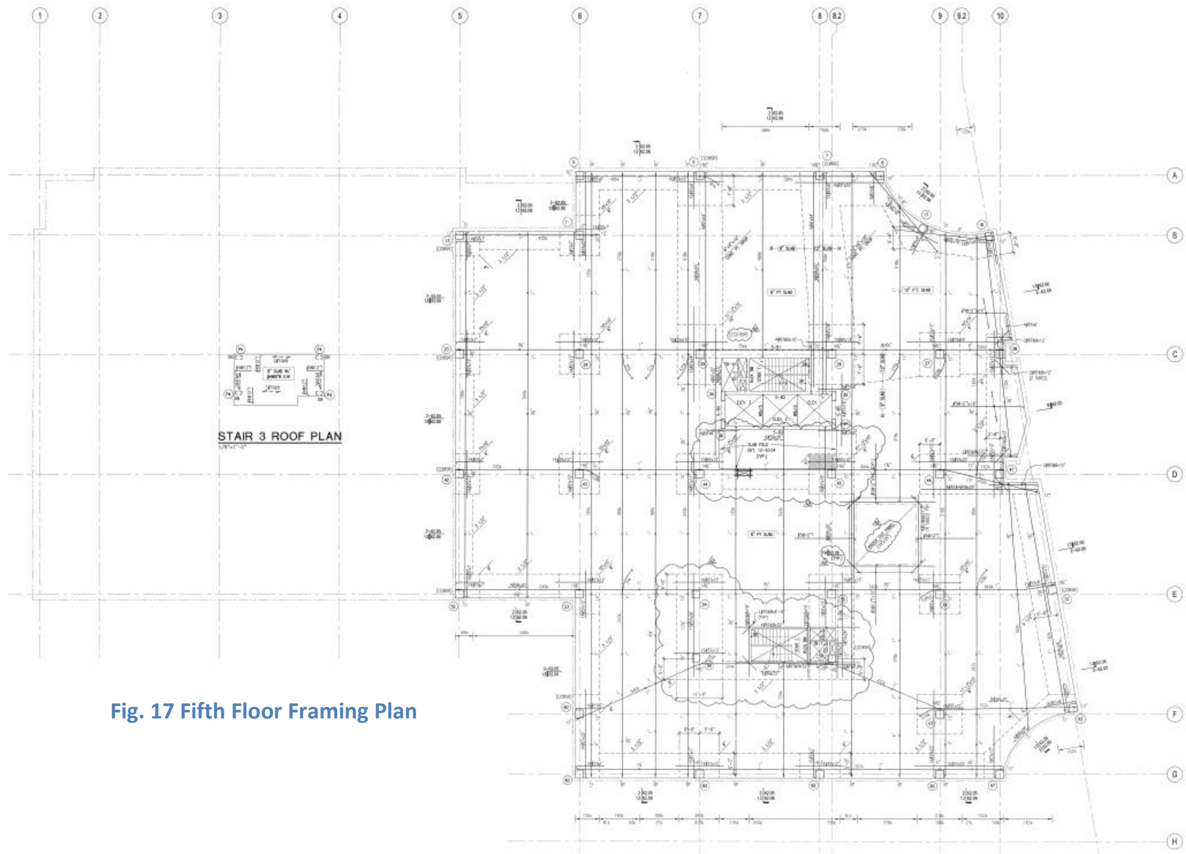


Fig. 17 Fifth Floor Framing Plan

*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

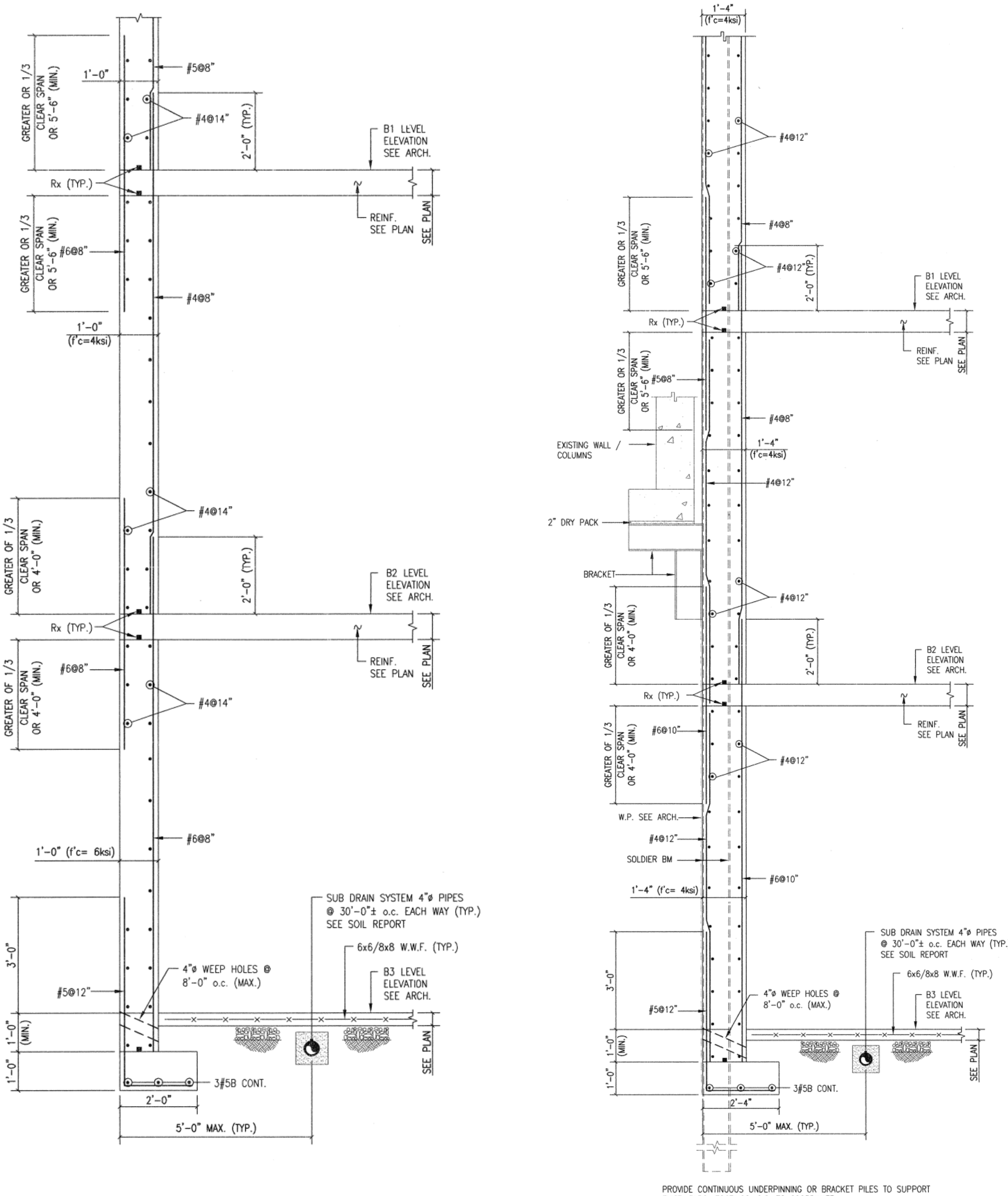


Fig. 18 Wall Sections

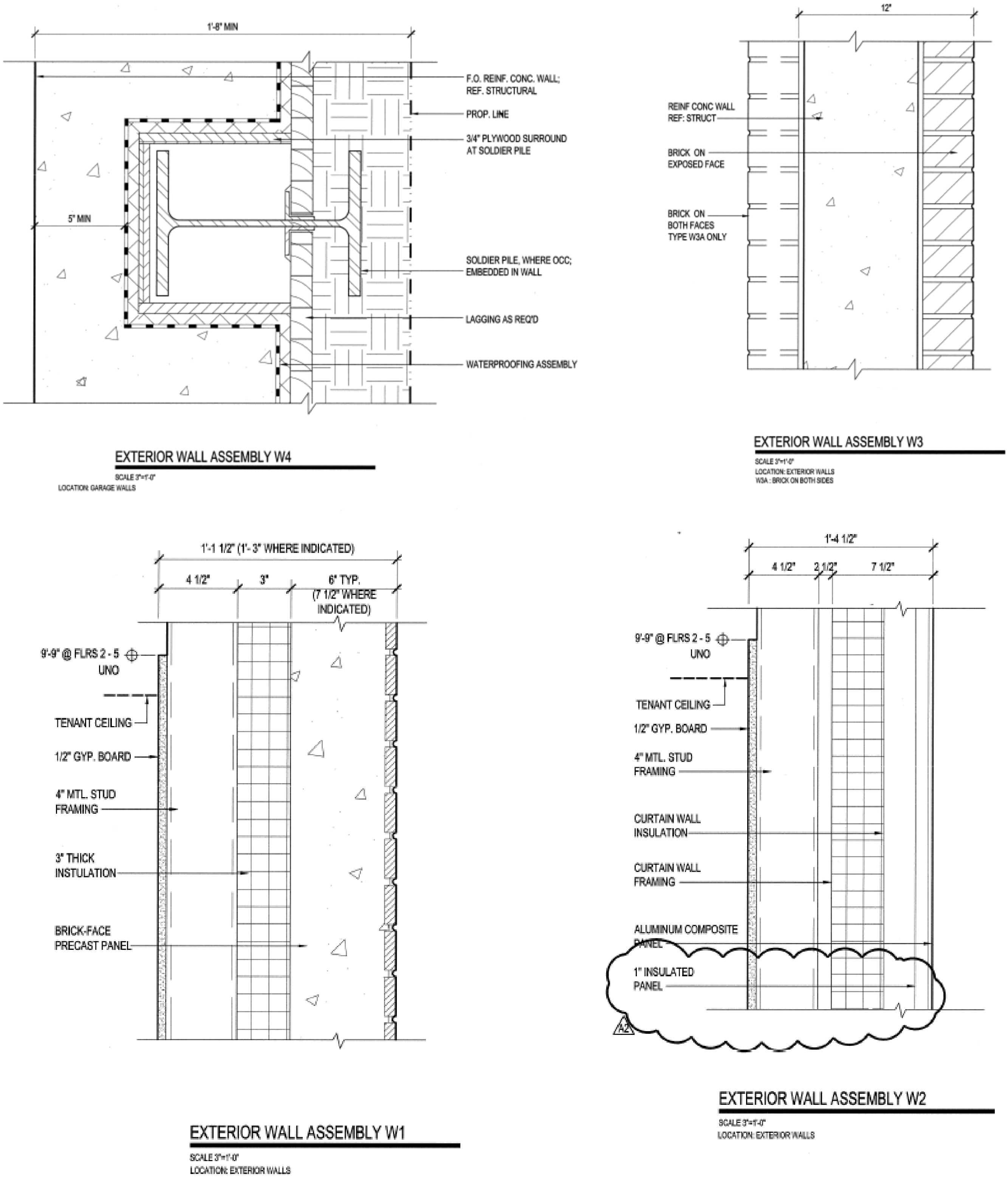


Fig. 19 Exterior Wall Assemblies

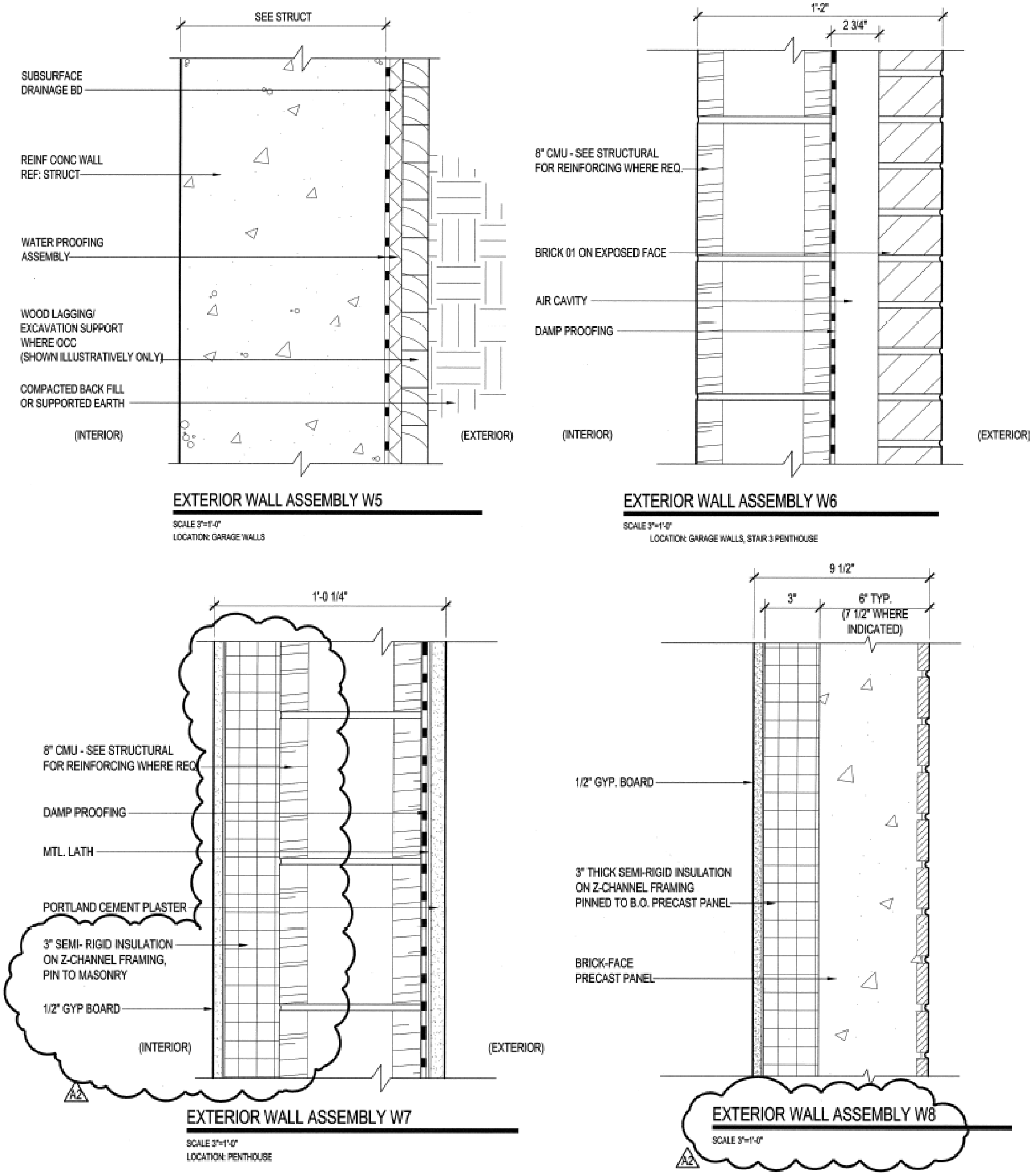


Fig. 20 Exterior Wall Assemblies (continued)

APPENDIX C

Wind Loads

Summary Tables

Risk Category	II
Basic Wind Speed	105 mph
Exposure	B
Kd	0.85
Kzt	1
Gf (N-S)	0.84
Gf (E-W)	0.82

	N-S	E-W
Windward Cp	0.8	0.8
Leeward Cp	-0.37	-0.5
Sidewall Cp	-0.7	-0.7
Roof Cp		
First Value		
(0-h/2)	-0.9	-0.9
(h/2-h)	-0.9	-0.9
(h-2h)	-0.5	-0.5
(>2h)	-0.3	-0.3
Second Value	-0.18	-0.18

z (ft)	Kz	qz (psf)
0-15	0.57	13.67
20	0.62	14.87
25	0.66	15.83
30	0.7	16.79
40	0.76	18.23
50	0.81	19.43
60	0.85	20.39
70	0.89	21.35
80	0.93	22.31
90	0.96	23.03

Pressures (psf)	N-S	E-W
Leeward	-7.16	-9.44
Windward		
0-15ft	9.19	8.97
20	10	9.75
25	10.64	10.4
30	11.3	11.01
40	12.25	11.96
50	13.06	12.75
60	13.7	13.4
70	14.35	14
80	15	14.64
90	15.5	15.11
Penthouse	26.1	14.65

Story Forces (k)	N-S	E-W
Roof	27	37.9
5th	49.4	70
4th	45.5	67.6
3rd	43.1	79.3
2nd	62.4	116
Base Shear	222.5	344.2

	N-S	E-W
Overturning Moment (ft-k)	11680	18232

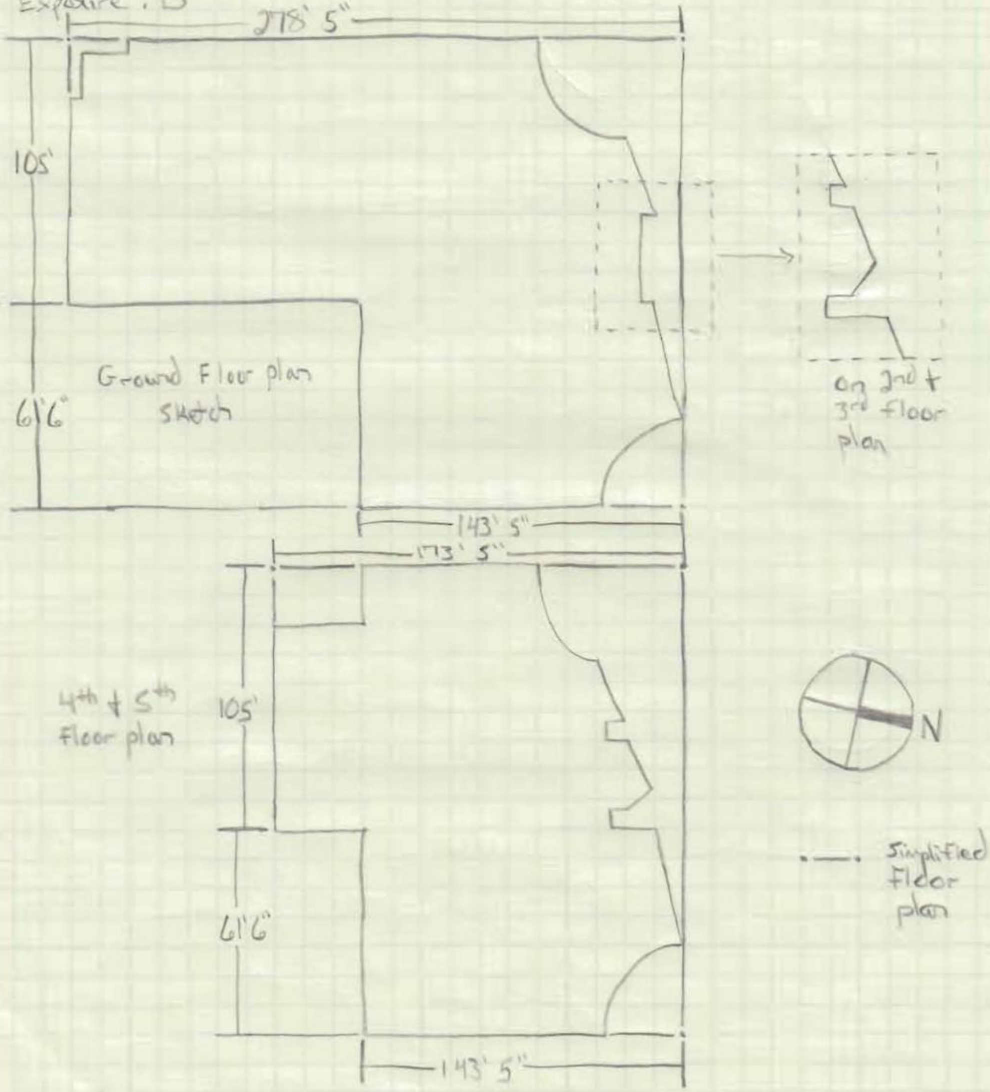
Josh Urban | AE Senior Thesis | Wind

Location : Arlington, Virginia

Risk Category : II

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

Exposure : B



Wind

Wind Directionality Factor: $K_d = 0.85$ (Table 26.6-1)

Topographic Factor: $K_{zt} = 1.0$ (refer to sect. 26.8)

Gust Effect Factor: Building max height = 83' < 300'

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i}$$

N-S

$$L_{eff} = \frac{28\tfrac{1}{3}(278\tfrac{5}{12}) + 41\tfrac{2}{3}(278\tfrac{5}{12}) + 55(278\tfrac{5}{12}) + 68\tfrac{1}{3}(173\tfrac{2}{12}) + 83(173\tfrac{1}{12})}{28\tfrac{1}{3} + 41\tfrac{2}{3} + 55 + 68\tfrac{1}{3} + 83}$$

$$= \frac{61046}{276.333} = 221' \Rightarrow 83' < 4(221) = 884'$$

E-W

$$L_{eff} = \frac{28\tfrac{1}{3}(166.5) + 41\tfrac{2}{3}(166.5) + 55(166.5) + 68\tfrac{1}{3}(166.5) + 83(166.5)}{28\tfrac{1}{3} + 41\tfrac{2}{3} + 55 + 68\tfrac{1}{3} + 83}$$

$$= \frac{46010}{276.333} = 166.5' \Rightarrow 83' < 4(166.5) = 666'$$

Can use 26.9.3
for n_a calc.

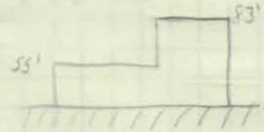
Wind

$n_a \Rightarrow$ hybrid system use Shear walls + Moment Frames

use $n_a = \frac{75}{h}$

$n_a = \frac{75}{83} = 0.9 < 1$

FLEXIBLE



Gust Factor : G_f

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}$$

$Q_{EW} = 0.8$

$Q_{NS} = 0.83$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$$

Table 26.9-1

$$I_z = 0.3\left(\frac{33}{83}\right)^{1/6} = 0.257$$

$\beta = 0.05$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$

$= 4.16$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_z}$$

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V$$

$$= 0.45\left(\frac{83}{33}\right)^{\frac{1}{4}}\left(\frac{88}{60}\right)115$$

$= 87.3$ ft/s

$\bar{b}$ + $\bar{\alpha}$ from table 26.9-1

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$$= 320\left(\frac{83}{33}\right)^{1/3}$$

$= 435$ ft

$$N_1 = \frac{0.9(435)}{87.3} = 4.48$$

$$R_n = \frac{7.47(4.48)}{(1+10.3(4.48))^{5/3}}$$

$= 0.0544$

$R_{EW} = 0.098$

$R_{NS} = 0.124$

$R_h = R_n$

$$R_n = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$$

$$\eta = \frac{4.6(0.9)(83)}{87.3} = 3.94$$

$= 0.25 - 0.032(0.9996)$

$= 0.22$

$R_B = R_B$

$$\eta_{EW} = \frac{4.6(0.9)(278.42)}{87.3} = 13.2$$

$$\eta_{NS} = \frac{4.6(0.9)(166.5)}{87.3} = 7.9$$

$R_{B_{EW}} = 0.073$

$R_{B_{NS}} = 0.119$

$R_L = R_L$

$$\eta_{EW} = \frac{15.4(0.9)(166.5)}{87.3} = 26.4$$

$$\eta_{NS} = \frac{15.4(0.9)(278.42)}{87.3} = 44.2$$

$R_{L_{EW}} = 0.037$

$R_{L_{NS}} = 0.0223$

Wind

$$G_{f_{EW}} = 0.925\left(\frac{1+1.7(0.257)\sqrt{(3.4^2)(0.8)^2+(4.16)^2(0.095)^2}}{1+1.7(3.4)(0.257)}\right)$$

$$= \left(\frac{2.2}{2.485}\right)0.925$$

$$= 0.82$$

$$G_{f_{NS}} = 0.925\left(\frac{1+1.7(0.257)\sqrt{(3.4)^2(.83)^2+(4.16)^2(0.124)^2}}{1+1.7(3.4)(0.257)}\right)$$

$$= \left(\frac{2.253}{2.485}\right)0.925$$

$$= 0.84$$

$GC_{pi} = \pm 0.18$ Enclosed (Table 26.11-1)

$K_z \Rightarrow$ Refer to table 27.3-1 for values, Exposure B

Velocity Pressures

Sample calc. $\Rightarrow q_z = 0.00256\, K_z K_{zt} K_d V^2$

$= 0.00256(0.57)(1.0)(0.85)(105)^2$

$= 13.67$ psf (0-15 ft)

The rest done in excel

= 14.87 psf	(20 ft)
= 15.83 psf	(25 ft)
= 16.79 psf	(30 ft)
= 18.23 psf	(40 ft)
= 19.43 psf	(50 ft)
= 20.39 psf	(60 ft)
= 21.35 psf	(70 ft)
= 22.31 psf	(80 ft)
= 23.03 psf	(90 ft)

Wind

Design Wind Pressure $\Rightarrow p = qG_fC_p - q_i(GC_{pi})$

$q = q_z$ (Windward)
$q = q_h$ (Leeward)
$q_i = q_h$ (enclosed)

N-S

Windward $C_p = 0.8$
Leeward $C_p = -0.37$
$\frac{L}{B} = \frac{278.42}{166.5} = 1.67$
Sidewall $C_p = -0.7$

Roof $C_p \Rightarrow$ Roof 1 (55')
$\frac{h}{L} = 0.198$ Slope < 10°

1st Value: -0.9 $(0 - h/2)$
-0.9 $(h/2 - h)$
-0.5 $(h - 2h)$
-0.3 $(> 2h)$
2nd Value: -0.18

Roof 2 (83')
Same Values

E-W

Windward $C_p = 0.8$
Leeward $C_p = -0.5$
$\frac{L}{B} = \frac{166.5}{278.42} = 0.598$
Sidewall $C_p = -0.7$

Roof $C_p \Rightarrow$ Roof 2
$\frac{h}{L} = \frac{83}{166.5} = 0.498 < 5$

Same values as N-S

Sample Calculations

N-S Windward

$p = q_z(0.84)(0.8) - 20.37(\pm 0.18)$
$= 0.672q_z + 3.67$ psf
(up to roof 1)

$p = 0.672q_z - 23.03(\pm 0.18)$
$= 0.672q_z + 4.15$ psf
(up to roof 2)

Leeward

$p = 23.03(0.84)(-0.37)$
$= -7.16$ psf (roof 2)

E-W Windward

$p = q_z(0.82)(0.8) - 23.03(\pm 0.18)$
$= 0.656q_z + 4.15$

Leeward

$p = 23.03(0.82)(-0.5)$
$= -9.44$ psf

Internal pressures usually cancel, ignore right side of p equation.

Wind

Windward

	N-S	E-W
0-15 ft	9.19 psf	8.99 psf
20	10	9.75
25	10.64	10.4
30	11.3	11.01
40	12.25	11.56
50	13.06	12.75
60	13.7	13.4
70	14.35	14
80	15	14.64
90	15.5	15.11

Story Forces (N-S)

2^{nd} (28.3') $\Rightarrow 10.64\left(\frac{28.3}{2}\right)(166.5) + 11.3\left(\frac{17.3}{2}\right)(166.5) = 37.6\text{ K}$

3^{rd} (41.67') $\Rightarrow 12.25\left(\frac{17.3}{2}\right)(166.5) + 12.25\left(\frac{17.3}{2}\right)(166.5) = 27.2\text{ K}$

4^{th} (55') $\Rightarrow 13.06\left(\frac{17.3}{2}\right)(166.5) + 13.7\left(\frac{17.3}{2}\right)(166.5) = 29.6\text{ K}$

5^{th} (68.3') $\Rightarrow 13.7\left(\frac{17.3}{2}\right)(166.5) + 14.35\left(\frac{14.67}{2}\right)(166.5) = 32.7\text{ K}$

Roof (83') $\Rightarrow 15\left(\frac{14.67}{2}\right)(166.5) = 18.3\text{ K}$

(E-W)

$2^{nd} \Rightarrow 10.4\left(\frac{28.3}{2}\right)(278.42) + 11.01\left(\frac{17.3}{2}\right)(278.42) = 61.4\text{ K}$

$3^{rd} \Rightarrow 11.96\left(\frac{17.3}{2}\right)(278.42) + 11.96\left(\frac{17.3}{2}\right)(278.42) = 44.3\text{ K}$

$4^{th} \Rightarrow 12.75\left(\frac{17.3}{2}\right)(278.42) + 13.4\left(\frac{17.3}{2}\right)(173.42) = 39.2\text{ K}$

$5^{th} \Rightarrow 13.4\left(\frac{17.3}{2}\right)(173.42) + 14\left(\frac{14.67}{2}\right)(173.42) = 33.3\text{ K}$

Roof $\Rightarrow 14.64\left(\frac{14.67}{2}\right)(173.42) = 18.6\text{ K}$

Wind

Add Leeward

N-S

$2^{nd} \Rightarrow 37.6 + \frac{7.16(20.83)(166.5)}{1000} = 62.4K$

$3^{rd} \Rightarrow 43.1K$

$4^{th} \Rightarrow 45.5K$

$5^{th} \Rightarrow 49.4K$

$Roof \Rightarrow 27K$

E-W

$2^{nd} \Rightarrow 61.4 + \frac{9.11(20.83)(278.0)}{1000} = 116K$

$3^{rd} \Rightarrow 79.3K$

$4^{th} \Rightarrow 67.6K$

$5^{th} \Rightarrow 70K$

$Roof \Rightarrow 37.9K$

N-S Base Shear = 222.5K

E-W Base shear = 344.2K

to lowest point of elevation

N-S overturning moment $= (62.4 \times 28.3) + (43.1 \times 41.67) + (45.5 \times 55) + (49.4 \times 68.3) + (27 \times 83) = 11680$ ft·K

E-W overturning moment $= (116 \times 28.3) + (79.3 \times 41.67) + (67.6 \times 55) + (70 \times 68.3) + (37.9 \times 83) = 18232$ ft·K

Wind Loads for Penthouse

Sect. 29.5-1

$F = q_z G C_f A_f$

$q_z \Rightarrow K_{zt} = 0.99 \quad (100')$

Penthouse roof is at 95'

$q_z = 23.75$ psf

$G = 0.82 \quad (E\text{-}W)$

$G = 0.84 \quad (N\text{-}S)$

$C_f \Rightarrow \frac{h}{D} = \frac{95}{47.5} = 2$

Interpolate

$\frac{2-1}{7-1} = \frac{x-1.3}{1.4-1.3}$

$0.1 = 6x - 7.8$

$x = 1.32$

$C_f = 1.32$

$A_f = 992 \text{ ft}^2 \quad (N\text{-}S)$

$A_f = 570 \text{ ft}^2 \quad (E\text{-}W)$

$F_{N-S} = 23.75(.84)(1.32)(992) = 26.1$ psf

$F_{E-W} = 23.75(0.82)(1.32)(570) = 14.65$ psf

APPENDIX D

Seismic Loads

Summary Tables

Site Class	D
Ss	12.50%
S1	6%
Fa	1.6
Fv	2.4
Sms	0.2
Sm1	0.144
Sds	0.133
Sd1	0.096
Category	B
PGA	6%
Site Coefficient	1.6

R	3
Ω	3
Cd	2.5
Cs	0.032

Story	Weight (k)
Roof	4296
5th	3611
4th	3723
3rd	5219
2nd	5419
Total	22268
Base Shear	712.6

Story	Force (k)
Roof	235
5th	155.3
4th	122.4
3rd	121.5
2nd	77.7

Overturning Moment (ft-k)	44110

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Site Class: D (Table 20.3-1)

$S_S = 12.5\%$ (Fig. 22-1)
$S_1 = 6\%$ (Fig. 22-2)

$F_a = 1.6$ (Table 11.4-1, site class D $S_S \le 0.25$)
$F_v = 2.4$ (Table 11.4-2, site class D $S_1 \le 0.1$)

$S_{MS} = F_a S_S = 0.2$ $S_{DS} = 2/3\, S_{MS} = 0.133$
$S_{M1} = F_v S_1 = 0.144$ $S_{D1} = 2/3\, S_{M1} = 0.096$

Table 11.6-1 $\Rightarrow S_{DS} < 0.167 \Rightarrow$ risk category II $\Rightarrow$ Category A
Table 11.6-2 $\Rightarrow 0.067 < S_{D1} < 0.133 \Rightarrow$ risk category II $\Rightarrow$ Category $\boxed{B}$

PGA = 6% (Fig. 22-7)

Site Coefficient: $F_{PGA} = 1.6$ (Table 11.8-1 $\Rightarrow$ PGA ≤ 0.1 Site Class D)

Response Modification Coefficient: R (Table 12.2-1)

Floors 1-3 $\Rightarrow$ ordinary reinf. conc. Shear walls $\Rightarrow R=5$
Floors 4-5 $\Rightarrow$ Ordinary reinf. conc. moment Frame $\Rightarrow R=3$

Overstrength Factor: Ω_0 (Table 12.2-1)

Floors 1-3 $\Rightarrow \Omega_0 = 2\frac{1}{2}$
Floors 4-5 $\Rightarrow \Omega_0 = 3$

Deflection Amplification Factor: C_d (Table 12.2-1)

Floors 1-3 $\Rightarrow C_d = 4\frac{1}{2}$
Floors 4-5 $\Rightarrow C_d = 2\frac{1}{2}$

Controlling Values

$R = 3$
$\Omega_0 = 3$
$C_d = 2.5$

Sect. 12.2.3.1 $\Rightarrow$ upper system R is lower than the lower system R, use upper system values for both systems.

Seismic

Use Equivalent Lateral Force Method

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

Seismic Response Coefficient : C_s (Refer to Sec. 12.8.1)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.133}{\left(\frac{3}{1.0}\right)} = 0.0443 \text{ (Moment Frame)}$$

$\hookrightarrow$ Seismic Importance Factor (Table 1.5-2 $\Rightarrow$ risk category II)

$T_L = 8s$

$T_a = C_t h_n^x$

$= 0.016(97.33)^{.9}$

$= 0.985s$

$h_n = 97.33'$ (from lowest point to penthouse)

$C_t = 0.016$, $x = 0.9$ } Table 12.8-2

T cannot exceed $C_u T_a = 1.7(0.985) = 1.67s$

$C_u = 1.7$ (Table 12.8-1 $\Rightarrow S_{D1} \leq 0.1$)

$T \leq T_L \Rightarrow C_s$ should not exceed : $C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.096}{0.985\left(\frac{3}{1}\right)} = 0.032 < C_s$ ✗

$\boxed{\text{Use } C_s = 0.032}$

W = effective seismic weight

$$\text{Roof DL} = \underbrace{\left(\frac{12}{12} \times 150\right)}_{\text{12" NW conc. slab}} + \underbrace{24\text{psf}}_{\text{roof pavers insulation water proofing}} + \underbrace{8.0\text{psf}}_{\text{Solar PV Panel load}} = \boxed{182\text{psf}}$$

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

Roof Snow Load $< L_r \Rightarrow$ use $L_r = 30$psf

Floor Dead Loads $\Rightarrow$ 8", 12", 14", & 16"

$\left(\frac{8}{12} \times 150\right) + 15 = 115$ psf

12" $\Rightarrow$ 165 psf

14" $\Rightarrow$ 190 psf

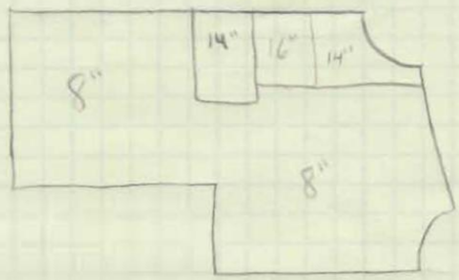
16" $\Rightarrow$ 215 psf

15 psf is extra assumed allowance

Seismic

Break Down of approximate areas for slabs by floor

1st Floor



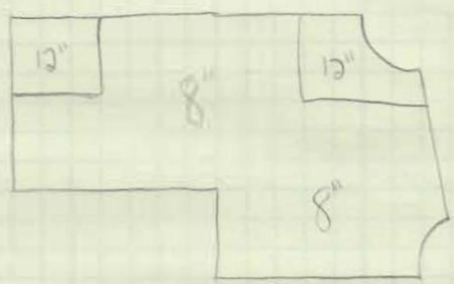
14" => 4674 ft²

16" => 1380 ft²

8" => 30645 ft²

* 1st Floor isn't * included

2nd and 3rd Floor



12" => 4975 ft²

8" => 31978 ft²

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

Non Green Roof

12" = 2212 ft²

8" = 23349 ft²

5th Floor => Same as non green roof portion of 4th Floor

Seismic

Wall DL ⇒ assume 30 psf for thin brick veneer or cast in place concrete panels and use 30 psf only to be conservative and to simplify calculations

Roof Load

$$W_{rf} = 19984(182) + 889.8\left(\frac{13.33}{2}\right)(30) + \underbrace{166(2350) + 257.67(12)(30)}_{\text{Penthouse}}$$
$$= 4296K$$

Floor Load

$$5^{th} = 2212(165) + 23349(115) + 679.8(13.4)(30) + 288$$
$$= 3611K$$

$$4^{th} = 3585K + \frac{12(11475)}{1000} = 3723K$$

$$3^{rd} = 4975(165) + 31978(115) + 889.8(13.4)(30) + 362.8$$
$$= 5219K$$

$$2^{nd} = 4975(165) + 31978(115) + 889.8(20.9)(30) + 362.8$$
$$= 5419K$$

Total DL = 22268K

$$V = C_s W$$
$$= 0.032(22268)$$
$$= 712.6K \text{ Base Shear}$$

Additional Structural weight

Columns ⇒ 2nd ⇒ 22"x22" = 3.36 SF x 51 columns = 171 SF ⇒ 2285 cubic ft $\boxed{342.8K}$

12"x30" = 2.5 SF x 4 columns = 10 SF ⇒ 133.3 ft³ $\boxed{20K}$

3rd ⇒ 342.8 + 20 = $\boxed{362.8K}$

4th ⇒ 3.36 SF x 36 columns ⇒ 121 SF ⇒ 1613 ft³ 242K + 20K $\boxed{262K}$

5th ⇒ 121 SF x 14.67 = 1775 ft³

Non typical column height

2.5 x 4 x 14.67 = 147 ft³ $\boxed{288K}$

Seismic

Distribute Forces

$$F_x = C_{vx}V \Rightarrow C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

K ⇒ interpolate between 1 & 2

$$\frac{0.983-0.5}{2.5-0.5} = \frac{x-1}{2-1}$$

$$2x-2 = 0.483$$

$$x = 1.24 = K$$

Sample calc

$$\text{Roof} = \frac{4296(83)^{1.24}}{5419(28.33)^{1.24}+5219(41.67)^{1.24}+3723(55)^{1.24}+3611(68.33)^{1.24}+4296(83)^{1.24}}$$

↳ roof level

$$= \frac{1.03e^6}{3.4e^5+5.32e^5+5.36e^5+6.8e^5+1.03e^6} = 0.33$$

Rest of the calcs. done in excel

$5^{th} \Rightarrow 0.218$
$4^{th} \Rightarrow 0.172$
$3^{rd} \Rightarrow 0.17$
$2^{nd} \Rightarrow 0.109$

$\Sigma = 1.0$ ✓

Story Forces

$F_R = 0.33(712.6) = 235\text{ K}$

$F_5 = 155.3\text{ K}$

$F_4 = 122.4\text{ K}$

$F_3 = 121.5$

$F_2 = 77.7\text{ K}$

Overturning Moment

$$(235\times83)+(155.3\times68.33)+(122.4\times55)+(121.5\times41.67)+(77.7\times28.3) = \boxed{44110\text{ ft}\cdot\text{K}}$$

APPENDIX E

Snow Loads

Josh Urban	AE Senior Thesis	Snow

Flat Roof Snow Load: $p_f = 0.7 C_e C_t I_s p_g$

$C_e \Rightarrow$ Category B Fully Exposed $\Rightarrow 0.9$ (Table 7.2)

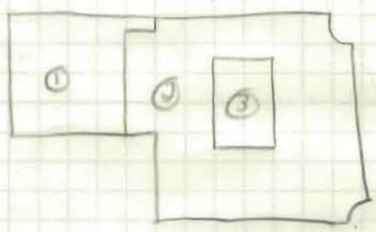
$C_t \Rightarrow 1.0$ (Table 7.3)

$I_s \Rightarrow 1.0$ Risk Category II (Table 1.5-2)

$p_g \Rightarrow$ Ground Snow Load $\Rightarrow 25$ psf (Figure 7.1)

$p_f = 0.7(0.9)(1.0)(1.0)(25)$
$= 15.75$ psf

Snow Drift



Roof 1 ⇒ 55' (From lowest elevation)

Roof 2 ⇒ 83'

Roof 3 ⇒ 97.33' (Penthouse)

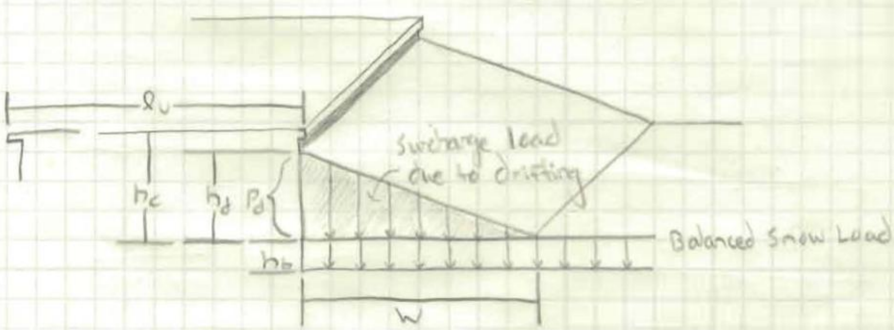
$\gamma = 0.13 p_g + 14 = 17.25$ $\quad \frac{p_s}{\gamma} = 0.913$

$p_s = C_s p_f = 15.75$

	Roof 1-2	Roof 2-3
$\frac{h_c}{h_b}$	$\frac{27}{0.913} > 0.2$	$\frac{13.33}{0.913} > 0.2$

$C_t = 1.0$ (Table 7-3)
Warm Roof
$C_s = 1.0$ (Fig. 7-2)

Need to apply drift loads



Roof 3 to roof 2

N-S

$l_u = 47.5'$

$h_d = 0.43\sqrt[3]{47.5}\sqrt[4]{25+10} - 1.5$
$= 2.29$

$h_d < h_c$

$w = 4h_d$
$= 4(2.29)$
$= 9.15'$

$p_d = h_d \gamma$
$= 2.29(17.25)$
$= 39.5$ psf

E-W

$l_u = 84.5'$

$h_d = 0.43\sqrt[3]{84.5}\sqrt[4]{25+10} - 1.5$
$= 3.09$

$h_d < h_c$

$w = 4h_d$
$= 4(3.09)$
$= 12.36'$

$p_d = h_d \gamma$
$= 3.09(17.25)$
$= 53.3$ psf

Roof 2 to roof 1

Only N-S windward drift

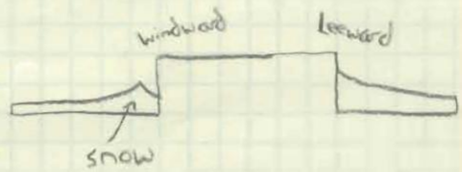
$l_u = 173.5'$

$h_d = 0.43\sqrt[3]{173.5}\sqrt[4]{25+10} - 1.5$
$= 4.33$

$h_d < h_c$

$w = 4h_d$
$= 4(4.33)$
$= 17.32'$

$p_d = h_d \gamma$
$= 4.33(17.25)$
$= 74.7$ psf



h_d equation comes from Fig. 7-9

APPENDIX F

Spot Checks

Josh Urban | AE Senior Thesis | Spot checks

Column at D-6 : Design

Tributary Area = $30 \times 30 = 900\ ft^2$ ($K_{LL}A_T = 4(900) = 3600 > 400$) can be reduced

1st Floor: LL Reduction = $0.25 + \frac{15}{\sqrt{3600 \times 4}} = 0.375 \Rightarrow$ Use 0.4

$\hookrightarrow$ 4 floors

$P_L = 0.4(80+20)(4)(900) = 144\ k$

$P_S = 30(900) = 27\ k$

$\hookrightarrow L_R >$ Snow Load so use L_R

$P_D = \left[\left(\frac{10}{12} \times 150\right) + \underbrace{24+8}_{\text{Roofing}}\right](900) + \left[\left(\frac{8}{12} \times 150\right) + 15\right](4)(900) = 555.3\ k$

15 → Extra Allowance; 4 → # of Floors

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

$P_u = 1.2(555.3) + 1.6(144) + 0.5(27)$

$\boxed{P_u = 910\ k}$

$\boxed{M_u = 503\ ft \cdot k}$

$w_u = 1.2D + 1.6L$
$= 1.2(115 \times 30) + 1.6(100 \times 30)$
$= 8.94\ klf$

$M_u = \frac{w_u l_n^2}{16} = \frac{8.94(30)^2}{16} = 503\ ft \cdot k$

don't know l_n so use l to be conservative

$f'_c = 5$ ksi
$f_y = 60$ ksi
$\rho_g = 0.015 \Rightarrow$ assumed value

Spot checks

$$A_g(trial) \geq \frac{P_u}{0.4(f'_c + f_y \rho_g)} = \frac{910}{0.4(5+60 \times .015)} = 386 \text{ in}^2$$

Try 20×20

$$e = \frac{M_u}{P_u} = \frac{563}{910} = 0.55 \text{ ft} = 6.6 \text{ in}$$

$$\frac{e}{h} = \frac{6.6}{20} = 0.33$$

$$\gamma = \frac{20 - 2(2.5)}{20} = 0.75$$

• Determine ρ_g from interaction diagrams

$$\frac{\phi P_n}{A_g} = \frac{P_u}{A_g} = \frac{910}{20 \times 20} = 2.28 \text{ ksi}$$

$$\frac{\phi M_n}{A_g h} = \frac{M_u}{A_g h} = \frac{563 \times 12}{20 \times 20 \times 20} = 0.75$$

$\rho_g \approx 0.043 \Rightarrow$ economical values for ρ_g range from 1-2%

choose larger cross section

Try 22×22

$$\gamma = \frac{22 - 2(2.5)}{22} = 0.77$$

$$\frac{P_u}{A_g} = \frac{910}{22 \times 22} = 1.88$$

$$\frac{M_u}{A_g h} = \frac{563 \times 12}{22 \times 22 \times 20} = 0.62$$

Interpolate

$\gamma = 0.75 \Rightarrow \rho_g = 0.026$

$\gamma = 0.9 \Rightarrow \rho_g = 0.022$

Try larger size

Spot checks

Try 24 x 24

$\gamma = \frac{24 - 2(2.5)}{24} = 0.79$ $\frac{P_u}{A_g} = \frac{910}{24 \times 24} = 1.58$ $\frac{M_u}{A_g h} = \frac{503 \times 12}{24 \times 24 \times 20} = 0.52$

Interpolate:

$\gamma = 0.75 \Rightarrow \rho_g \approx 0.015$
$\gamma = 0.9 \Rightarrow \rho_g \approx 0.0125$

$\rho_g = 0.015 - (0.015 - 0.0125)\left(\frac{.04}{.15}\right)$
$\rho_g = 0.014$ ✓

• Select reinforcement

$A_{st} = \rho_g A_g$
$= 0.014\,(24 \times 24)$
$= 8.06\ in^2$

$6\#11 = 6(1.56) = 9.36\ in^2$
$8\#10 = 8(1.27) = 10.16\ in^2$

$\phi P_{n\,max} = 0.8\,\phi\left[0.85 f'_c (A_g - A_{st}) + f_y (A_{st})\right]$
$= 0.8(0.65)\left[0.85(5)(576 - 9.36) + 60(9.36)\right]$

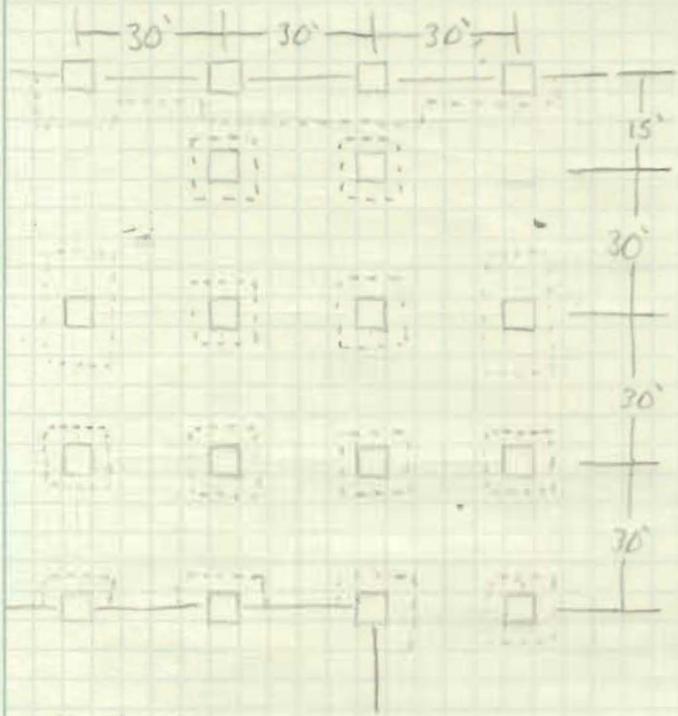
compression controlled

$= 1544\,k > 910\,k$ ✓

$\phi P_n > P_u$ ✓

$\boxed{\begin{array}{l} 24 \times 24 \text{ tied Column} \\ (6)\ \#11 \text{ Bars} \\ F_y = 60\,ksi \\ f'_c = 5\,ksi \end{array}}$

Spot checks :



- - → indicates drop panels

Dead Loads

slab ⇒ $\left(\frac{8}{12}\right) \times 150 = 100$ psf

Superimposed = 15 psf

Drop panels ⇒ $\underbrace{16' \times 16' \times 8''}_{\text{typical}}$

All drop panels will be treated as equivalent to simplify calculations

$\frac{8}{12} \times 150 = 100$ psf

Live Loads

100 psf

Materials

$F'_c = 6000$ psi

$F_y = 60000$ psi

$F_{pu} = 270{,}000$ psi

7 wire strand $\frac{1}{2}''$ diameter

$A = 0.153\ in^2$

Estimated prestress losses = 15 ksi (assumed)

$F_{se} = 0.7(270) - 15 = 174$ ksi

$P_{eff} = AF_{se} = 26.6\ ^K/\text{tendon}$

Slab Thickness

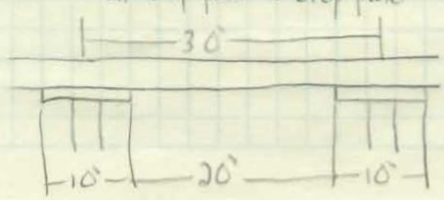
ACI table 9.5 (c)

Ext. Panel ⇒ $l_n/33 \Rightarrow 30 \times 12/33 = 7.27''$

Int. Panel ⇒ $l_n/36 \Rightarrow 20 \times 12/36 = 6.67''$ ⇒ $8''$ ✓

Designed slab is 8"

l_n = drop panel to drop panel



Spot Checks

Class U (ACI 18.3.3)

$A = bh$
$= (30\times12)(8)$
$= 2880 \text{ in}^2$

$S = \frac{bh^2}{6}$
$= \frac{(30\times12)(8)^2}{6}$
$= 3840 \text{ in}^3$

allowable Stresses

at time of jacking (ACI 18.4.1)

$f'_{ci} = 3000$ psi

Compression $= 0.6 f'_{ci}$
$= 0.6(3000)$
$= 1800$ psi

Tension $= 3\sqrt{f'_{ci}}$
$= 3\sqrt{3000}$
$= 164$ psi

at service loads (ACI 18.4.2 a)

$f'_c = 6000$ psi

Compression $= 0.45 f'_c$
$= 0.45(6000)$
$= 2700$ psi

Tension $= 6\sqrt{f'_c}$
$= 6\sqrt{6000}$
$= 465$ psi

assume 60% target load balance of slab self weight
$0.6(100) = 60$ psf

Tendon locations

Ext. Support ⇒ 4"
Int. Support top ⇒ 7"
Int. Span bottom ⇒ 1"
End Span bottom ⇒ 1.75"

$a_{INT} = 7 - 1 = 6"$
$a_{EXT} = \frac{(4+7)}{2} - 1.75 = 3.75"$

assume end span governs

Spot checks

$w_b = .6w_{DL} = 60\times30 = 1800\ plf$

$$P = \frac{w_b L^2}{8a_{end}} = \frac{1800(30)^2}{8(\frac{2.75}{12})} = 648K$$

$$\therefore \#\ of\ tendons = \frac{648}{26.6} = 24.4 \Rightarrow 24\ tendons$$

actual Force $= 24(26.6) = 638.4\ K$

Adjust w_b for actual force

$$w_b = \frac{638.4}{648} = 0.985(1800) = 1773\ plf$$

Determine Precompression stress

$$\frac{P_{actual}}{A} = \frac{638.4(1000)}{2880} = 221.7\ psi \quad > 125_{psi}\ (min) \quad < 300psi\ (max) \quad \checkmark$$

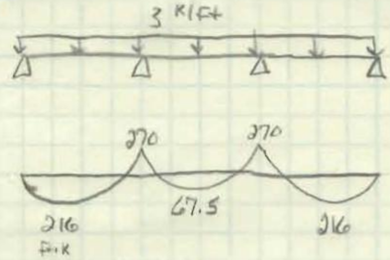
$$w_b = \frac{638.4(8)(6/12)}{30^2} = 2.84\ K/ft$$

$$\frac{w_b}{w_{DL}} < 100\% \Rightarrow \frac{2.84}{\left(\frac{30(100)}{1000}\right)} = \frac{2.84}{3} = 0.95 \checkmark$$

Spot checks

Dead Load Moments

$w_{DL} = \frac{30(100)}{1000} = 3 \text{ k/ft}$

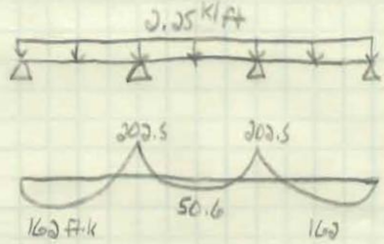


Live Load Moments

Reduction => $.25 + \frac{.15}{\sqrt{900}} = 0.75$

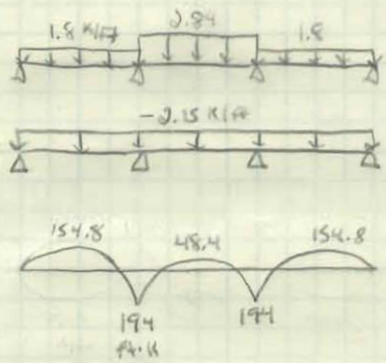
$LL = 100 \text{ psf} \times .75 = 75 \text{ psf}$

$w_{LL} = \frac{30(75)}{1000} = 2.25 \text{ K/ft}$



Total Balancing Moments

$\frac{1.8 + 2.84 + 1.8}{3} = 2.15 \text{ K/ft}$



Spot Checks

Check Stresses

Stage 1: immediately after jacking (ACI 18.4.1)

Int: $f_{top} = \frac{(-67.5+48.4)(12)(1000)}{3840} - 222 = -282$ psi < 1800 psi ✓ (comp)

$f_{bot} = \frac{(67.5-48.4)(12)(1000)}{3840} - 222 = -162$ psi < 1800 psi ✓ (comp)

End: $f_{top} = \frac{(-216+154.8)(12)(1000)}{3840} - 222 = -413$ psi < 1800 psi ✓ (comp)

$f_{bot} = \frac{(216-154.8)(12)(1000)}{3840} - 222 = -30.75$ psi < 1800 psi ✓ (comp)

Support: $f_{top} = \frac{(270-194)(12)(1000)}{3840} - 222 = 15.5$ psi < 164 psi ✓ (tens)

$f_{bot} = \frac{(-270+194)(12)(1000)}{3840} - 222 = -459.5$ psi < 1800 psi ✓ (comp)

Stage 2: at service load (18.3.3 + 18.4.2)

Int: $f_{top} = \frac{(-67.5-50.6+48.4)(1000)(12)}{3840} - 221 = -439$ psi < 2700 psi ✓ (comp)

$f_{bot} = \frac{(67.5+50.6-48.4)(1000)(12)}{3840} - 221 = -3.1$ psi < 2700 psi ✓ (comp)

End: $f_{top} = \frac{(-216-162+154.8)(1000)(12)}{3840} - 221 = -918$ psi < 2700 psi ✓ (comp)

$f_{bot} = \frac{(216+162-154.8)(1000)(12)}{3840} - 221 = 476$ psi > 465 psi X (tens)

Support: $f_{top} = \frac{(270+202.5-194)(12)(1000)}{3840} - 221 = 649$ psi > 465 psi X (tens)

Stresses are <u>not</u> within permissible code limits.