TECHNICAL REPORT THREE

1776 Wilson Boulevard

Arlington Virginia

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Structural Option

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Executive Summary

Technical report three serves as an in depth analysis of the lateral system for 1776 Wilson Boulevard, located in Arlington Virginia. 1776 Wilson is a five story office building with retail at the ground level as well as three and a half levels of below grade parking. This building is currently under construction and will be approximately 249,000 SF when complete with a lump sum construction contract value of 63.5 million dollars.

The structural system consists of two different lateral force resisting systems. The first three stories have a dual system of ordinarily reinforced concrete moment frames with concrete shear walls. The fourth and fifth stories contain only moment frames. The post tensioned floor slabs serve as the beams in the moment frames. A computer model of the structure was created using Bentley's RAM software. All of the columns were modeled identically and the structure has rigid diaphragms due to being all concrete. Because of this, there are moment frames acting in both directions. Using results from technical report one and new calculations, a study on shear forces, distribution of shear forces, effects of torsion, overturning moments, and drift was performed.

Using load combinations from ASCE 7-10 showed that the following combinations control when taking into account lateral loads:

- 1.2D + 1.6W + L + 0.5(Lr or S or R)
- 1.2D + 1.0E + L + 0.2S
- 0.9D + 1.6W
- 0.9D + 1.0E

Seismic controls in almost every aspect except for story force at the 2nd level where the wind load controls. Because of this, all four of these load combinations should be used and compared. The seismic combinations were only used in this report, however, due to the fact that it controls in most all cases. Four wind load cases from ASCE 7-05 were also used in analyzing the wind loads on the structure. The third case controls. This case takes into account torsional effects and eccentricity along with the wind loadings in the north-south and east-west directions.

Spot checks were also performed on the shear walls to make sure they were adequate to carry the loads distributed to them. After all the analysis was complete, a better understanding of how the lateral loads are distributed through this system was gained and the current design is adequate to resist these loads.

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 be finished in August of 2012, 1776 Wilson will contain approximately 249,000 SF. The lump sum construction contract is valued at 63.5 million dollars.

Fig. 1 Lobby Rendering

Designed by RTKL Associates, 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 is currently up for lease and the future tenants are currently unknown. 1776 Wilson will also include a three and half level parking garage which will be able to accommodate over 200 cars The retail space will have a high ceiling making tenant mezzanines possible. Most of the mechanical equipment will be located in a penthouse on top of the building. Besides the flexible office space, one of the most important 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, located within walking distance of two Metro stations; several retail outlets and restaurants are within close proximity of the site.

1776 Wilson Boulevard also goes beyond the norm for 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 includes the site of the finished building; this area generally designates 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 in accordance with building code. 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 façades along with aluminum framed curtain walls. The rest of the façade features precast 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.

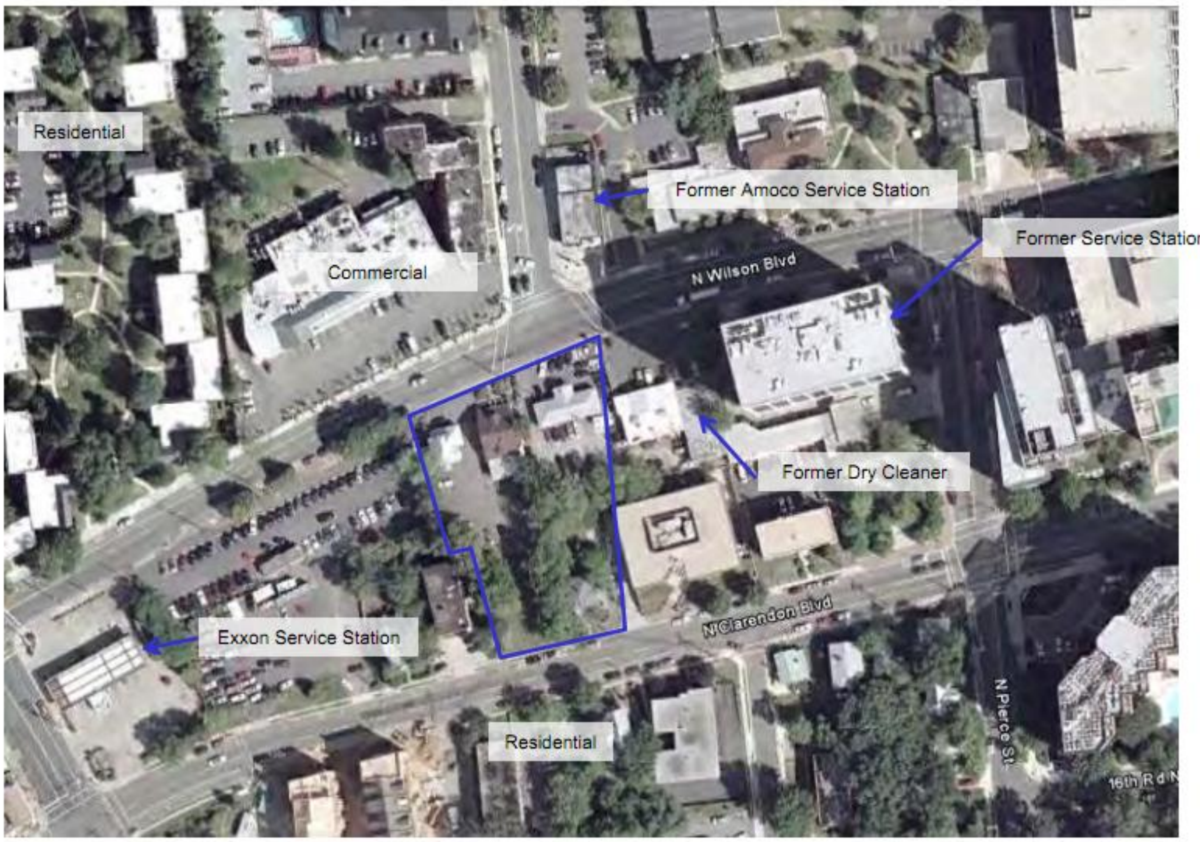


Fig. 3 Aerial View of Site

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 divided the soil into three strata:

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.

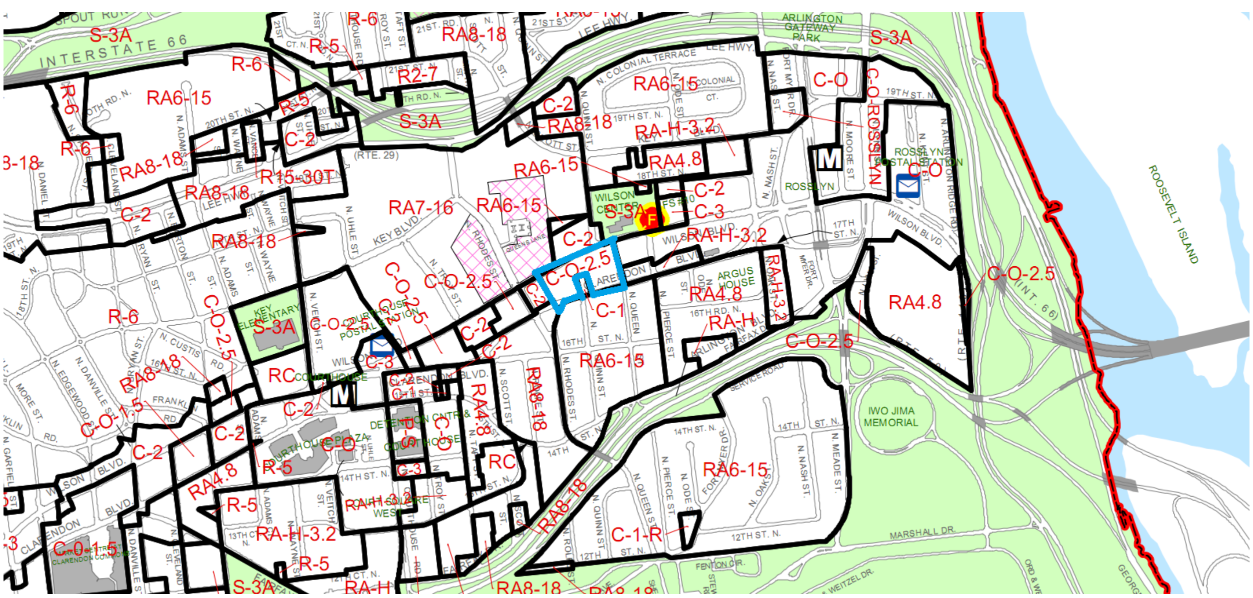


Fig. 4 Zoning Map for Arlington - The blue outline marks the district where 1776 Wilson is to be located

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 placed over 10 mil polyethylene and 6 inches of washed gravel. Control joints are located at 20 feet on center for all exterior slabs. Interior slabs are to be placed 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 is placed 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 are to be mud matted after excavation so that equipment won’t get stuck in the soft and sensitive soils.

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.

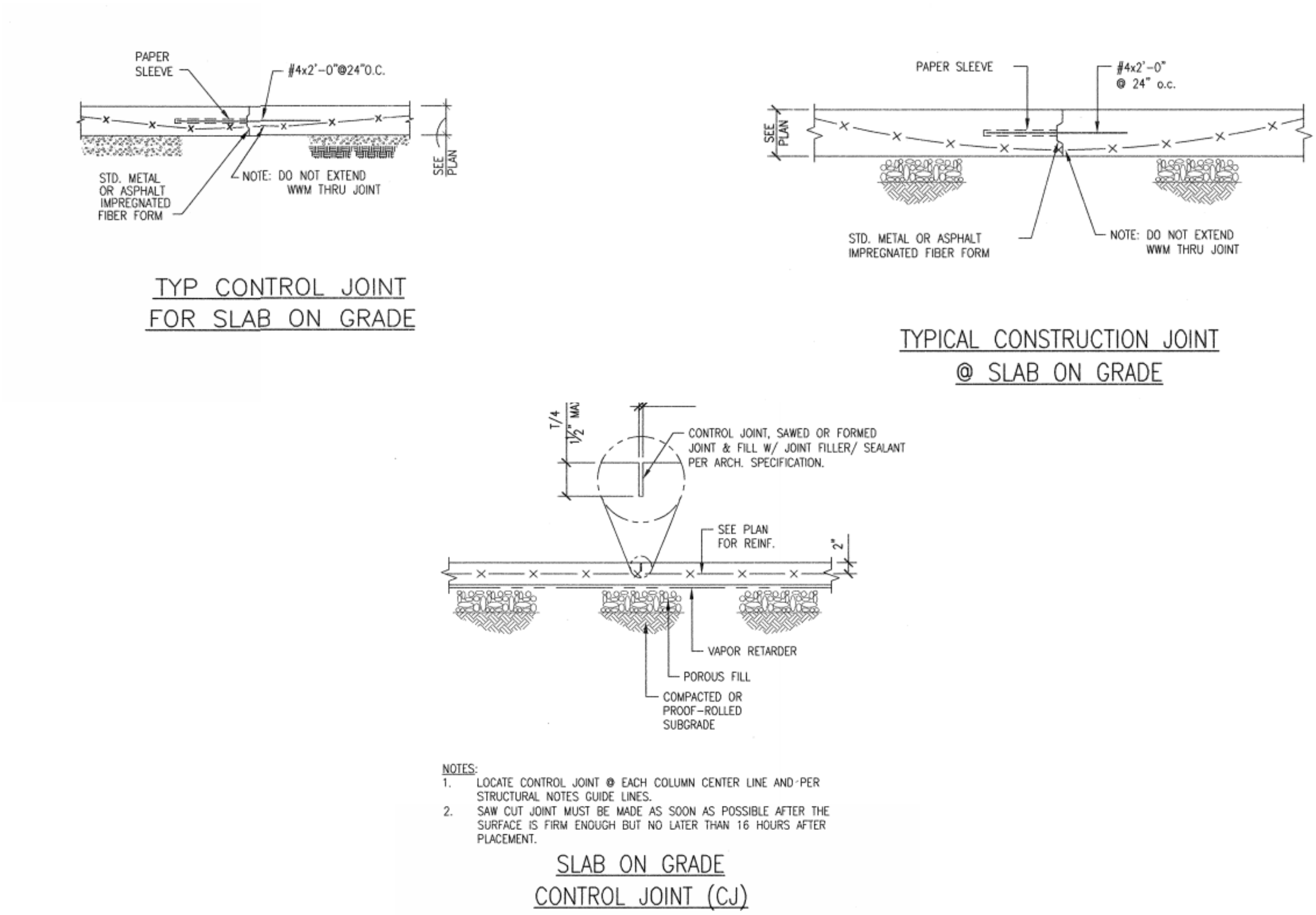


Fig. 5 Slab on Grade Control Joint

Floor System

This project uses a high strength post tensioned concrete structure. Each floor consists of flat slabs 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. Each slab has drop panels at the column locations that are typically 8"thick, much larger than typical drop panels. Select locations have drop panels even thicker with 10" being the largest thickness found in the building. 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.

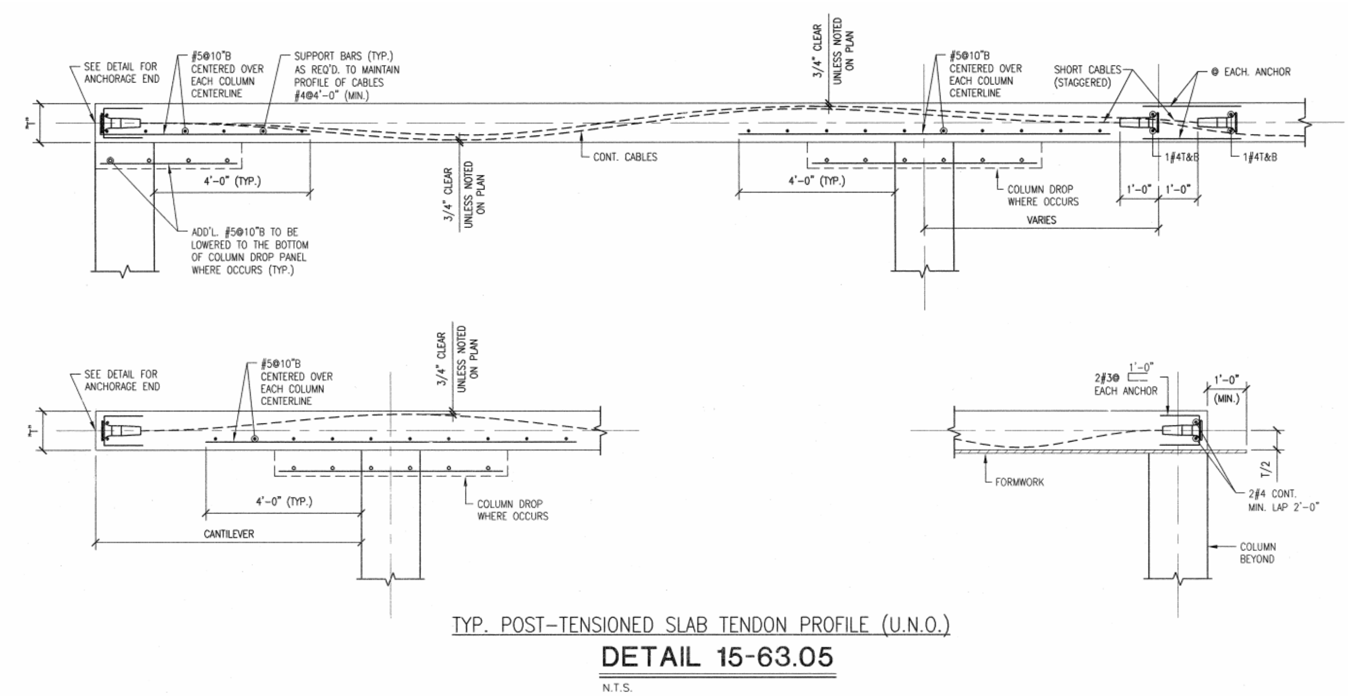


Fig. 6 Typical Post Tensioned Slab Tendon Profile

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.

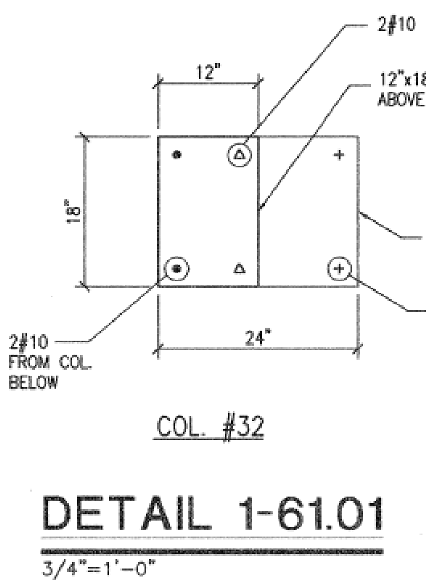
Columns

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. The reinforced concrete columns on the upper floors are typically 22x22 inch columns and 12x30 inch columns; the lower levels are typically 24x24 inch columns. 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



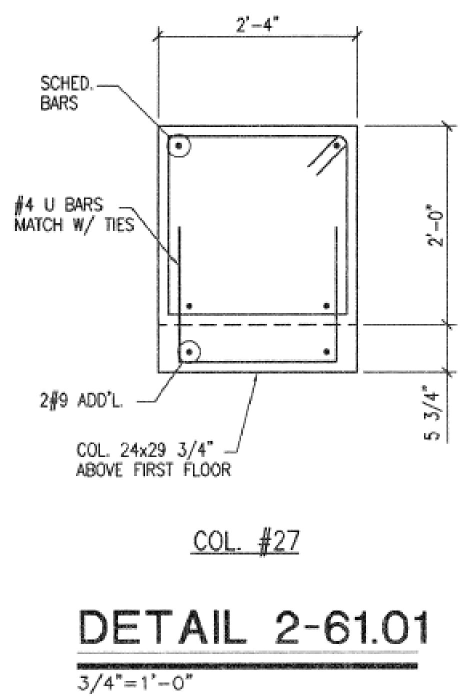


Fig. 7 Column Details

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 surround the elevator shafts of the bottom three stories providing aid to the moment frames. This creates a dual system on the bottom three stories that share the lateral loads. Simplifications were made for the wind analysis done and ASCE 7-10 offers a way to calculate seismic loads for buildings with different vertical lateral force resisting systems. More information on those calculations can be found in the wind and seismic sections of this report.

The lateral loads will be distributed by relative stiffness. Starting at the roof diaphragm and then travelling through the columns that help make up the reinforced concrete moment frames to the floor diaphragm. The floor slabs themselves serve as the beams in the moment frames and transfer the loads to the columns on the floor below, eventually reaching the shear walls on the bottom three stories. Once the lateral loads reach the shear walls, 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.

The shear walls highlighted in red in the figure below are located in the same spot on each of the first three floors and maintain a 12” thickness. The longer shear wall (noted as shear wall 2 in appendix C) is much more rigid compared to the other two shear walls. Together, these three shear walls help lower the lateral forces that need to be resisted by the moment frames. Due to torsion, the building will want to rotate around the taller right half of the building so the shear walls are able to help decrease the movement of the left half of the structure.

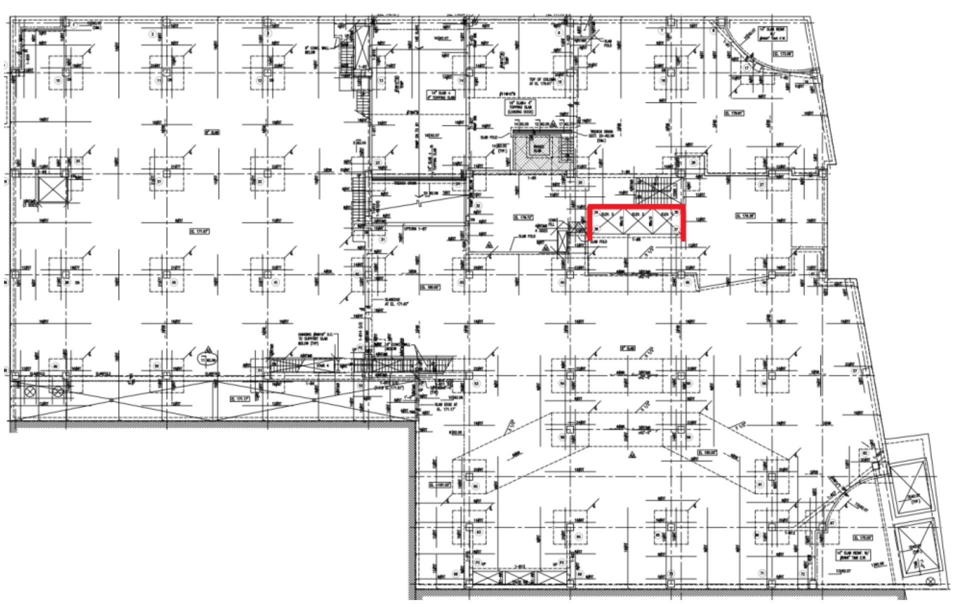


Fig. 8 Shear Wall Locations

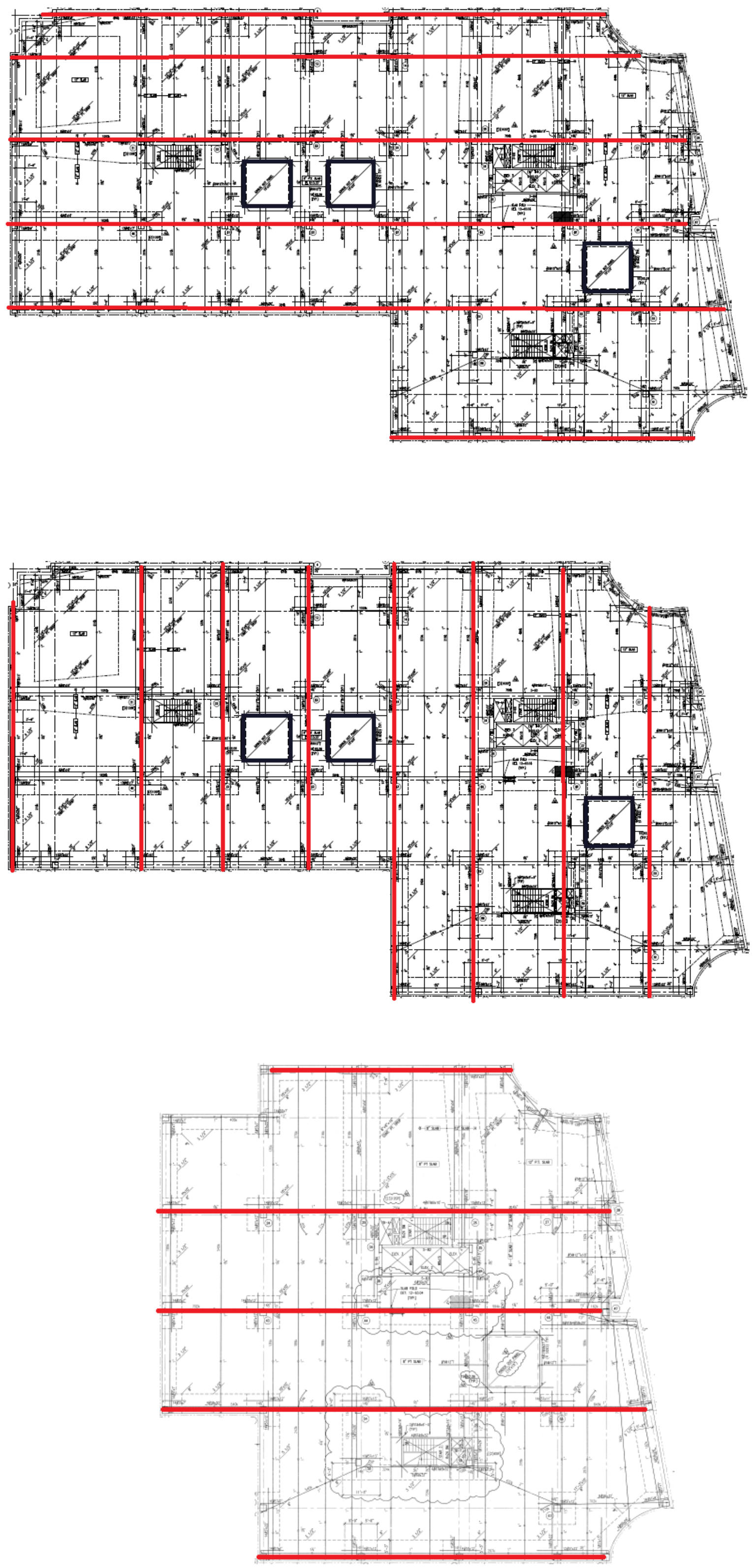


Fig. 9 Moment Frame General Layout

Fig. 10 Moment Frame General Layout Continued

The concrete moment frames in the building were modeled to act in both directions general layouts for the frames in both the NS and EW direction are shown in figures 9 and 10.

Table 3 summarizes the shear forces that need to be distributed amongst the shear walls after taking into account how much of that force is drawn to the concrete shear walls. There are no shear walls on the 4th and 5th floors so all of the lateral loads are resisted by the moment frames.

Story	EW %	NS %	EW Direct (k)	NS Direct (k)
2nd	55.2	56.6	42.9	44
3rd	70.8	77.4	86	94
4th	100	100	122.4	122.4
5th	100	100	155.3	155.3

Table 3 Distribution to Moment Frames

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
- ASCE 7-05 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.

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 4 Materials

Post Tensioned Concrete – tendons consist of steel strands that conform to ASTM A-416, Fpu=270,000 psi. Tendons are stressed once the concrete reaches 75% of design strength.

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 mortar for 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 + 15 psf for partitions	50 psf + 15 psf for partitions
Corridors above first floor	80	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 5 Live Load Summary

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

Table 6 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
Vegetation	1-2 psf

Table 7 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 8 Snow Load Information

Deflection Criteria

- Live Loads: L/360
- Total Deflection: L/240
- Lateral Drift: H/400

Load Combinations

The following load combinations from ASCE 7-10 section 2.3.2 (using strength design) were utilized in combining factored loads for the gravity and lateral load analysis. Since this tech report focuses on the lateral loads, live loads, construction loads, and hydrostatic loads were not considered. 1776 Wilson Boulevard doesn't contain any composite steel members which are where construction loads would come into play for the computer model done in RAM Modeler.

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

In reference to figure 6-9 of ASCE 7-05, there are four design wind load cases that need to be taken into account. These four cases were taken into account when creating the RAM model for the structure which led to 143 different load combinations when combined with seismic, dead, and notional combinations. Case one is full design wind pressure acting along each principal axis, these occur separately in contrast to case three which uses the same loading as case one but it acts simultaneously and at 75% of the specified value. Case two and four are similar to case one and three respectively with the addition of a torsional moment.

When considering lateral loads, load cases 4 through 7 generally control. The controlling load combination from that group depends on if 1.6W is larger than 1.0E. In most instances, the seismic load is still larger than 1.6W, however the 2nd story shear force does control. Because of this, it is advisable to take into consideration all four applicable load cases. This report will focus solely on load cases 5 and 7 since seismic controls in most cases.

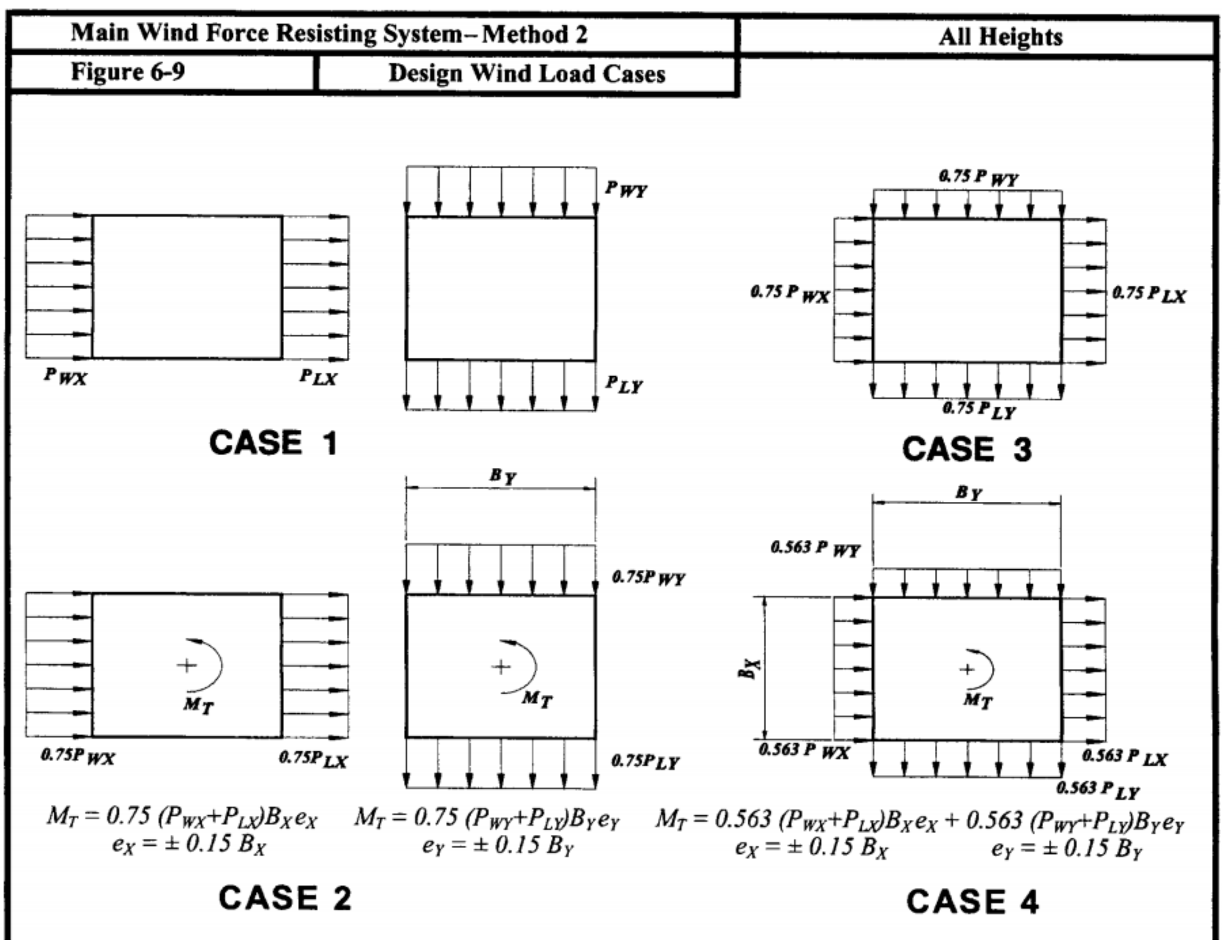


Fig. 10 Design Wind Cases from ASCE 7-05

Computer Model

A computer model was created for this report using Bentley's RAM Structural System. The building's shape was simplified for this model and live loads were not taken into account since the main purpose of this model was to gain a better understanding of how the lateral loads are distributed throughout the resisting systems. The diaphragms are modeled as rigid since the structure is fully concrete. Data gathered from this model was compared to hand calculations for wind and seismic loads. The wind values calculated by hand were in agreement with the values found in the program. The seismic values differed but this is because the computer model has simplified floor systems that don't take into account variations in thicknesses as accurately as the hand calculations do.

The concrete moment frames in the structure consist of the columns and the floor slabs acting as beams. Since all the columns were modeled in the same fashion, the structure consists of moment frames in both directions. The shear walls on the bottom three stories ease the loads by drawing lateral forces to them which means the left half of the structure sees a significant reduction in lateral loads compared to a system absent of shear walls. This makes it easier to resist rotation about the taller half of the structure.

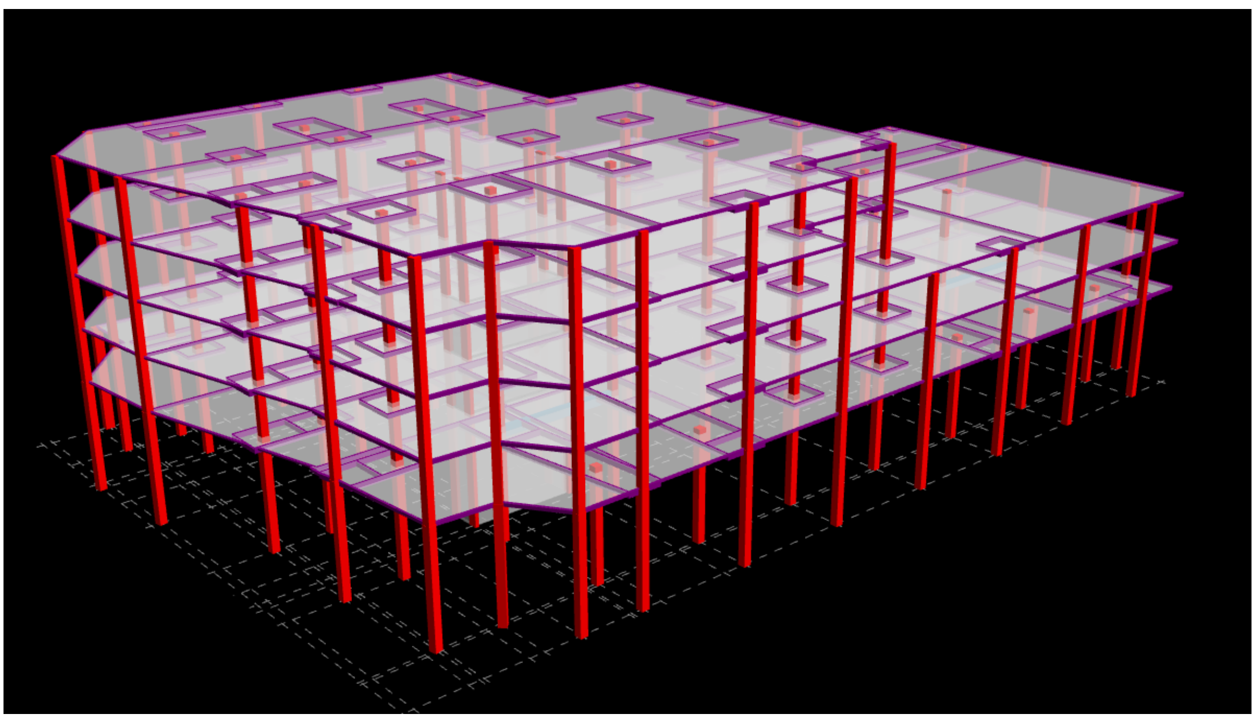


Fig. 11 3D Structural Model in RAM

Wind Loads

Wind loads for 1776 Wilson were calculated with accordance to ASCE 7-10 for technical report one 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. Due to the complicated nature of ASCE 7-10, the wind loads were recalculated using ASCE 7-05 in order to get more accurate values for this technical report. The new calculations resulted in a velocity pressure of 16.9 psf. This value is in agreement with the velocity pressure of 17 psf given in the structural notes for 1776 Wilson. The wind pressures in the E-W direction controlled over those in the N-S direction due to a larger leeward pressure. The values ranged from 7 psf to 11.5 psf with a leeward pressure of 7.2 psf.

After modeling 1776 Wilson in RAM, the analysis showed that the model is rigid in the X direction and flexible in the Y direction. This would lead to a different gust factor for the upper stories of the building since the building was assumed to be rigid in both directions for the initial calculations. The modified gust factor of 0.86 wasn't a significant increase and the modified values only increased by about 0.1 psf so the initial calculations were kept despite the initial assumption of a rigid structure in both directions being incorrect.

There is still a discrepancy between the calculated pressures and the actual pressures listed in the structural notes. This is due to similar assumptions and simplifications made for both hand calculations and for the creation of a computer model. The floor plans and facades were simplified to get quicker but generally accurate results for wind loads applied to the building. The north façade in particular would need a more detailed and in depth analysis for wind loads due to its complex nature as shown in figure 15. The irregular shape and face of the façade falls under the limitations of chapter 6 in ASCE 7-05 so wind tunnel testing could be done for more specific wind loading information.

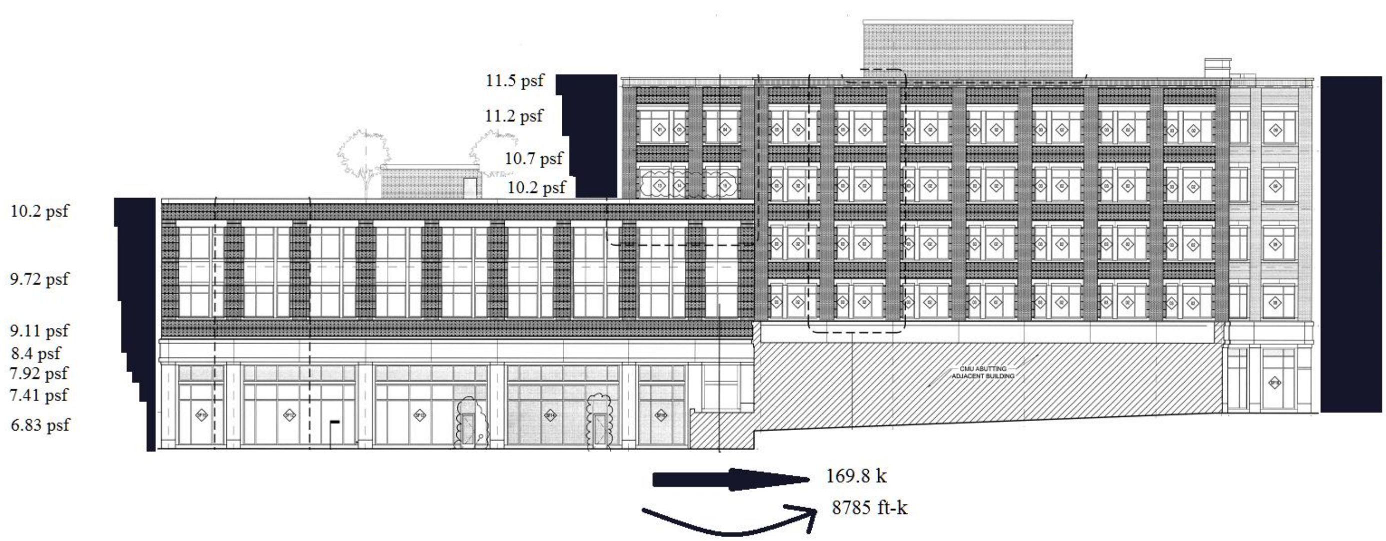


Fig. 12 Wind Loads NS Direction

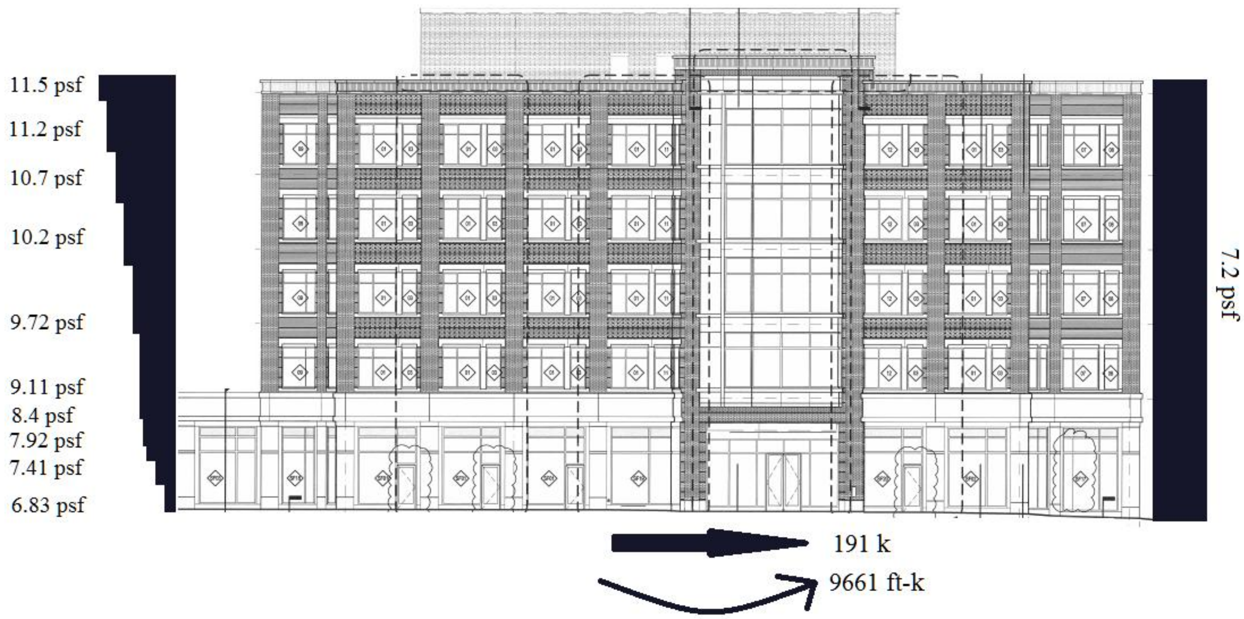


Fig. 13 Wind Loads EW Direction

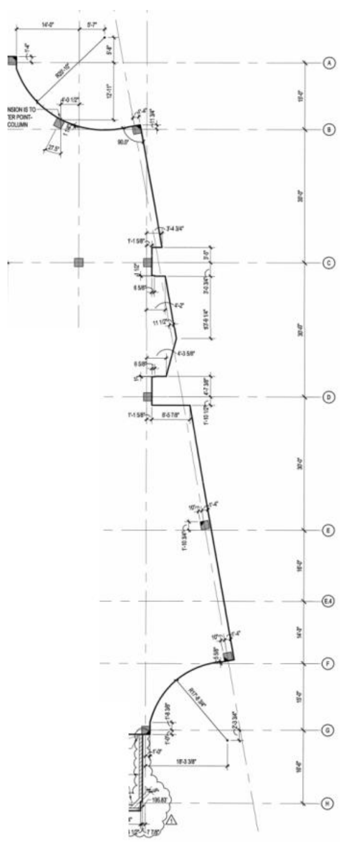


Fig. 14 North Facade Plan View

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. The floor slabs were simplified for the computer model which resulted in a lower building weight, therefore a lower base shear was found through RAM. My hand calculations were more specific to the variations in slab thicknesses so the hand calculated values are represented in this report due to the weights of each floor being more accurate than the computer model.

The base shear was calculated to be 712 kips which is within 5% of the base shear listed in the structural notes for the building (684 kips). The overturning moment was calculated to be 44110 ft-k which is much larger than the controlling wind load overturning moment so it is safe to say that seismic loads control the lateral design for this building.

Fig. 15 Seismic Loads

Center of Mass and Rigidity

Locations for the center of mass and the center of rigidity were taken straight from RAM in order to locate the resultant story forces. Since the center of mass and the center of rigidity do not correspond there will be a torsional moment induced on the building. The eccentricity of the structure is rather large which could be the result of a couple of different factors. Not all continuous drop panels were modeled due to time concerns and the shape of the floor slabs were simplified.

	Xr	Yr	Xm	Ym
Roof	158.65	98.59	169.24	90.28
5th	152.45	104.58	169.31	90.20
4th	147.9	109.45	117.40	101.08
3rd	151.32	109.72	121.72	99.96
2nd	154.64	109.78	122.58	97.32

Table 9 Center of Mass and Rigidity

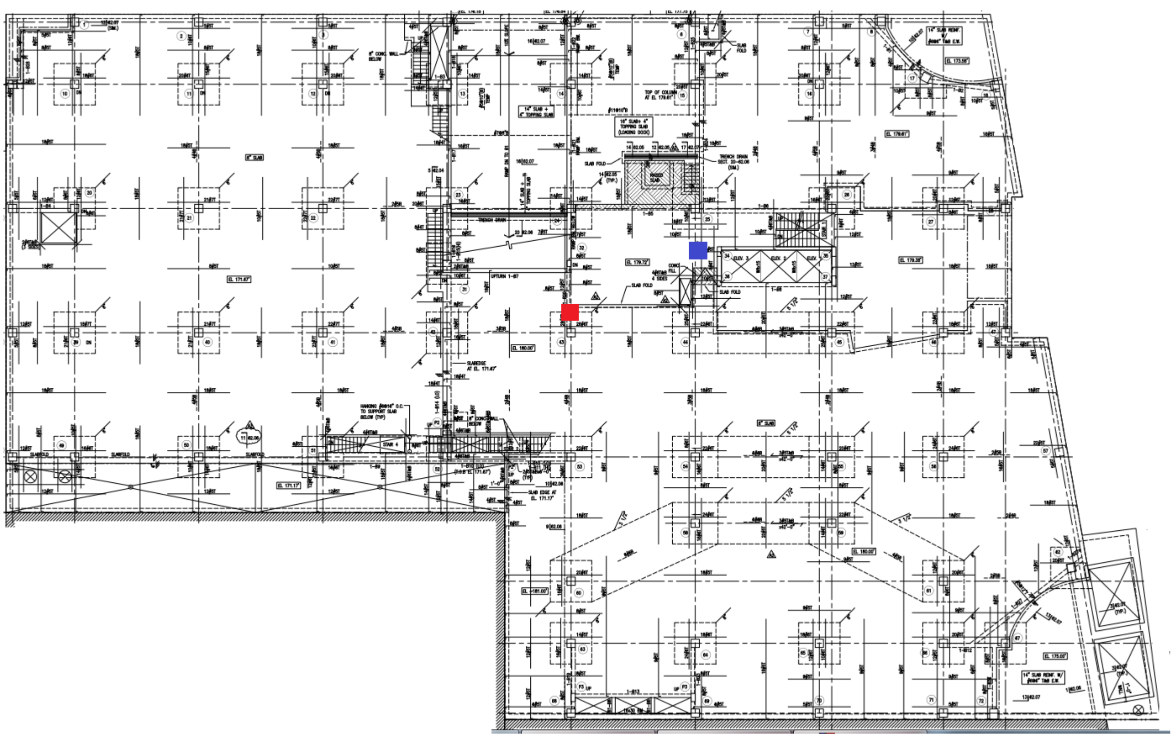


Fig. 16 Center of mass is represented by the red box, center of rigidity is represented by the blue box.

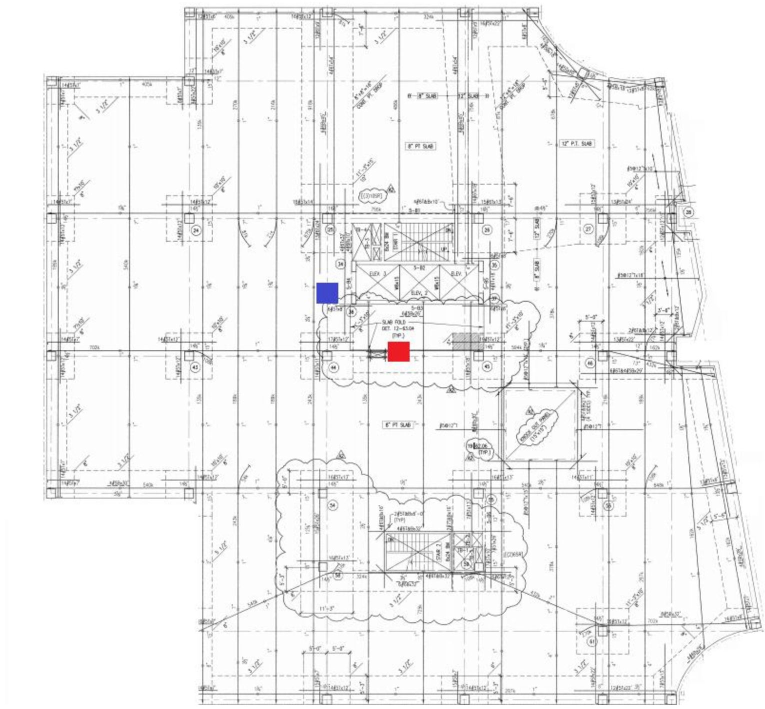


Fig. 17 Center of mass is represented by the red box, center of rigidity is represented by the blue box.

Shear Forces

The lateral loads will create direct shear forces in the shear walls and moment resisting frames. There will also be a shear forces due to torsion since the center of rigidity does not coincide with the center of mass. Hand calculations were done to determine what percentage of lateral loads is distributed to each of the shear walls. The remaining percentage is distributed amongst the moment frames. Table 10 and 11 summarize the data found from these calculations. The stiffness of the shear walls was calculated using the following equation for fixed at top and bottom:

$$k\ fixed = \frac{Ec}{\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)}$$

Level	Story Lateral Stiffness Kx (k/ft)	Story Lateral Stiffness Ky (k/ft)
Roof	16674.64	17201.56
5th	25675.32	15158.92
4th	95258.3	32243.73
3rd	117893.1	41784.25
2nd	61320.44	27260.75

Table 10 Story Lateral Stiffnes

Shear Wall	Level	K (k/ft)	%
1	2nd	6104.5	22.4
	3rd	6104.5	14.6
2	2nd	26622	43.4
	3rd	26622	22.6
3	2nd	6104.5	22.4
	3rd	6104.5	14.6

Table 11 Shear Wall Distribution

These results show that most of the lateral forces are resisted by the moment frames. Once the percentages were determined for each shear wall, the direct shear could be distributed amongst the shear walls. Table 12 summarizes the direct shears and shears caused by torsion for each wall. The value of shear used in the calculations was taken from the seismic load hand calculations which provided more accurate numbers. The following equation was used to calculate direct shear:

$$Vi^d = \frac{Ki}{\Sigma Ki} V$$

Shear due to torsion was calculated next using the equation:

$$Vi^t = \frac{Ve\ di\ Ki}{J}$$

$$(J = \Sigma\ Ki\ Di^2)$$

Shear Wall	Level	Direct Shear (k)	e_x (ft)	e_y	J	V^t (k)
1	2^{nd}	17.4	32	12.5	1.89e7	13.8
	3^{rd}	17.7	29.6	9.8	2.15e7	20.9
2	2^{nd}	33.7	32	12.5	1.89e7	18.4
	3^{rd}	27.5	29.6	9.8	2.15e7	19.76
3	2^{nd}	17.4	32	12.5	1.89e7	36.1
	3^{rd}	17.7	29.6	9.8	2.15e7	49.2

Table 12 Direct Shear and Shear due to Torsion

Direction of Shear Forces

The following two figures show the directions of the shear forces, both direct shear and shear due to torsion, in the shear walls of the first three floors. Since there is essentially moment resisting concrete frames in both directions, there are frames that directly resist the application of lateral loads in both the N-S and E-W direction.

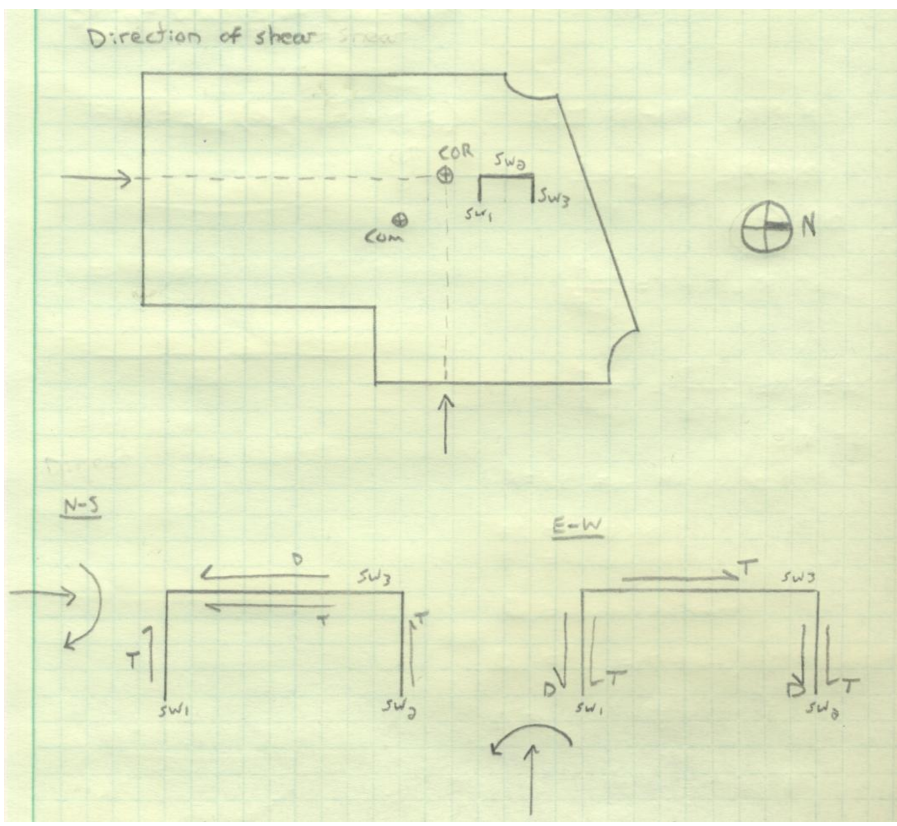


Fig. 18 Direction of Shear

Shear Wall Spot Checks

Spot checks were performed on the shear walls to make sure they could resist the direct shear forces and those caused by torsion. The hand calculations for these spot checks can be found in appendix D. The equation used for these spot checks was taken from ACI 318-08, section 21.9.4. That equation is:

$$\emptyset Vn = \emptyset Acv\ (\alpha c\ \lambda \sqrt{f'c} + \rho t\ fy)$$

Where Acv is area of concrete found by length x thickness
αc depends on the ratio of wall height to length, for each shear wall this value is 3.0
λ is taken as 1.0 for normal weight concrete
Pt is vertical reinforcement area divided by spacing times thickness of wall
F'c = 4000 psi and fy = 60000 psi for the concrete shear walls

Drift

Drift values for each story were taken from the RAM analysis done on the structure and compared to industry accepted values of h/400 and the more conservative h/600. It was important to check h/600 due to the brick veneer on the building which will crack easier and is difficult and expensive to repair. Wind loads are considered to be a serviceability issue so no load factors need to be applied when checking drift. Seismic, however, is a strength issue and needs to be factored using 1.0. The load case that controlled drift was seismic loading in the E-W direction with +/- eccentricity.

Story	Height (ft)	h/400	h/600	RAM Value Seismic
Roof	83	2.49	1.66	0.78524
5th	68.3	2.049	1.37	0.67939
4th	55	1.65	1.1	0.53202
3rd	41.67	1.25	0.83	0.39593
2nd	28.3	0.859	0.566	0.24909

Table 13 Drift

Conclusion

The lateral force resisting systems of 1776 Wilson Boulevard were analyzed for this report. The wind and seismic loads calculated in technical report one (with revisions done to the wind loads for this report) were applied to the structure as it was modeled in RAM in order to perform this analysis. The loads had to be factored using load combinations found in ASCE 7-10, seismic controlled in most all cases but it is important to note that load combinations involving wind loads should still be checked due to one instance where the wind load controlled. Four wind loading cases from ASCE 7-05 were also checked to make sure all the proper loading cases and combinations were being taken into account. In all, there were 143 different load combinations used by RAM to analyze the structure.

Since simplifications to the floor slabs were used in the RAM model, hand calculated values were used. This is because the hand calculated values took into account variations in floor thicknesses more accurately therefore provided more accurate numbers. The building has a large eccentricity in the X direction which could be due to simplifications made in the computer model. The shape of the floor slabs were simplified and not all drop panels were modeled in the interest of time. Slab openings were also not modeled. The structural system did pass all checks performed on drift. The drift values were compared to h/400 and the more conservative h/600 due to the brick veneer on the building's façade. The shear walls were also found to be adequate to carry the loads that are distributed to them.

After all the analysis was performed, the structure utilized for 1776 Wilson is adequate to resist the lateral loads applied as well as the distribution throughout. Because of this assignment, a much better understanding of the lateral system was gained. A more in depth model and analysis will be necessary if a lateral resisting system change is included in the proposal for this senior thesis project.

Appendix A

Wind Load Calculations

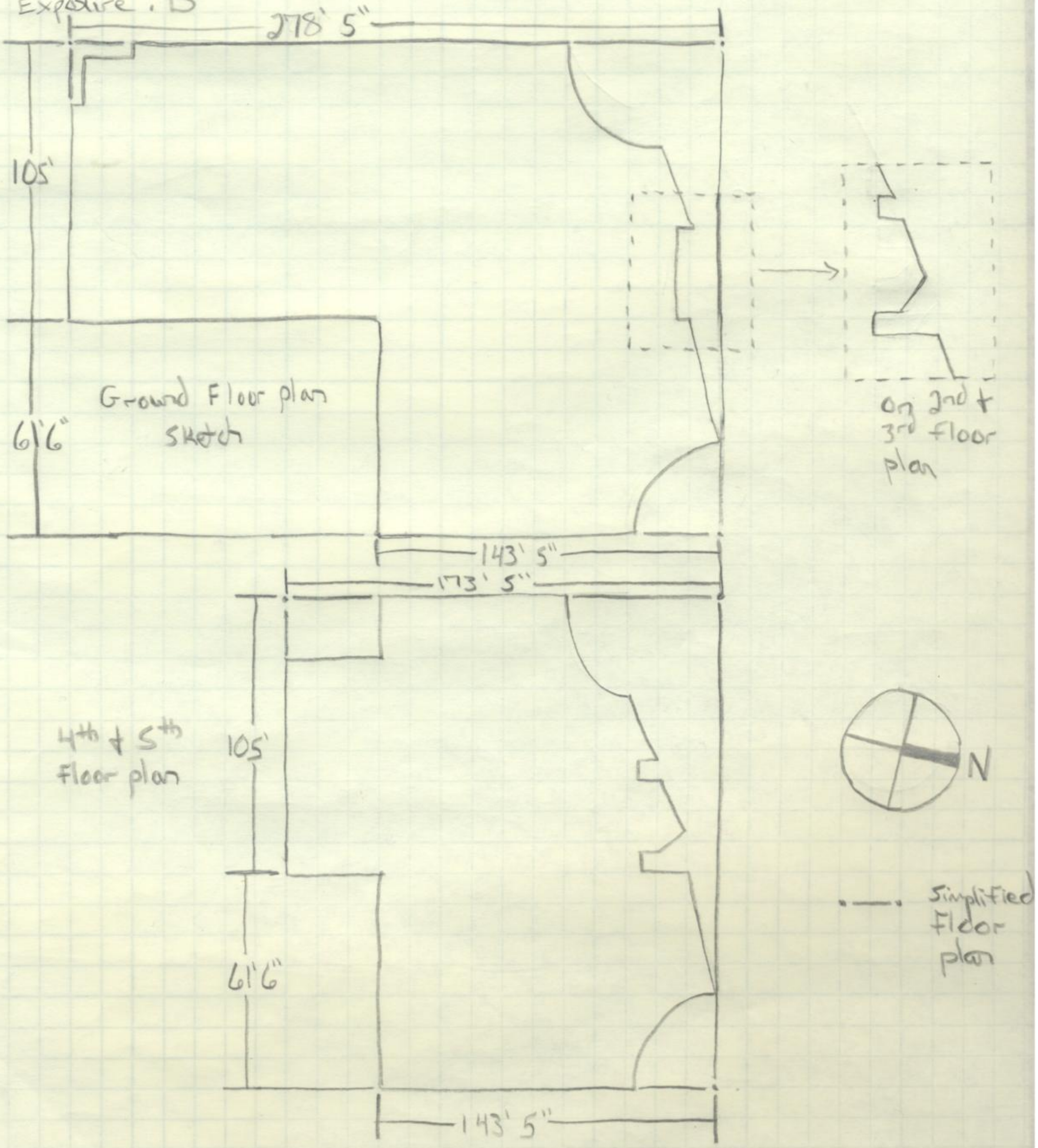
Josh Urban | AE Senior Thesis | Wind L...

Location : Arlington, Virginia

Risk Category : II

Basic Wind Speed : 90 mph (Fig. 6-1)

Exposure : B



Wind

Wind Directionality Factory: $K_d = 0.85$ (table 6.4)

Topographic Factor: $K_{zt} = 1.0$ (refer to sect. 6.5.7)

Gust Effect Factor ⇒ Rigid ⇒ 0.85 (refer to sect. 6.5.8.1)

$GC_{pi} = \pm 0.18$ Enclosed (Fig. 6-5)

K_z ⇒ Refer to table 6-3, Exposure B

Velocity Pressures

Sample Calculation ⇒ $q_z = 0.00256\, K_z K_{zt} K_d V^2$ (I = 1.0)

$= 0.00256(0.57)(1.0)(0.85)(90)^2$

$= 10.05$ psf (0-15 ft)

$= 10.9$ (20 ft)
$= 11.64$ (25 ft)
$= 12.34$ (30 ft)
$= 13.4$ (40 ft)
$= 14.3$ (50 ft)
$= 15$ (60 ft)
$= 15.7$ (70 ft)
$= 16.4$ (80 ft)
$= 16.9$ (90 ft)

Design Wind Pressure

$$P = q G_f C_p - \underbrace{q_i (GC_{pi})}_{}$$

Internal pressures usually cancel ⇒ neglect

$q = q_z$ windward
$q = q_h$ Leeward
$q_i = q_h$ Enclosed

N - S

windward $C_p = 0.8$
Leeward $C_p = -0.37$
$\frac{L}{B} = \frac{278}{167} = 1.66$
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 $C_p = -0.5$
$\frac{L}{B} = \frac{167}{278} = 0.6$
Sidewall $C_p = -0.7$

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

Same values as N-S

windward ⇒ Sample calculation

N - S

$p = q_z(0.85)(0.8)$
$= 10.05(.85)(0.8)$
$= 6.83$ (0-15 ft) in psf

= 7.41 (20 ft)
= 7.92 (25)
= 8.4 (30)
= 9.11 (40)
= 9.72 (50)
= 10.2 (60)
= 10.7 (70)
= 11.2 (80)
= 11.5 (90)

E-W

$P = q_z(.85)(.8)$

Same values as N-S

Leeward ⇒ sample calculations

N-S $p = 16.9(0.85)(-0.37)$
$= -5.32$ psf

E-W $p = 16.9(0.85)(-0.5)$
$= -7.2$ psf

Story Forces (N-S)

2^{nd} (28.3') => $8.4\left(\frac{28.3}{2}\right)(166.5) + 8.4(1.7)(166.5) + 9.11(4.95)(166.5) + 5.32(20.8)(166.5) = 48.1K$

3^{rd} (41.67') = 32.8 K

4^{th} (55') = 34.5 K

5^{th} (68.3') = 36 K

Roof (83') = 18.4 K

(E-W)

2^{nd} = 54.6K

3^{rd} = 36.96K

4^{th} = 38.7 K

5^{th} = 40.2 K

Roof = 20.5 K

N-S Base Shear = 169.8 K
E-W Base Shear = 191 K

E-W overturning moment will be greater than N-S

$E\text{-}W\ M_o = (54.6 \times 28.3) + (36.96 \times 41.67) + (38.7 \times 55) + (40.2 \times 68.3) + (20.5 \times 83) = 9661$ ft·K

Wind Loads for Penthouse

$F = q_z G C_f A_f$

$q_z \Rightarrow K_{z+} = 0.99$ (100') Penthouse roof is at 95'

$q_z = 20.53$ psf

$C_f = \frac{h}{D} = \frac{95}{47.5} = 2 \Rightarrow$ interpolate $\Rightarrow C_f = 1.32$

$A_f = 992\ ft^2$ (N-S)

$A_f = 570\ ft^2$ (E-W)

$F_{N-S} = 20.53(.85)(1.32)(992) = 22.9\ K$

$F_{E-W} = 20.53(.85)(1.32)(570) = 13.13\ K$

Appendix B

Seismic Load Calculations

Josh Urban | AE Senior Thesis | Seismic

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 $S_S \leq 0.25$)
$F_v = 2.4$ (Table 11.4-2, site class D $S_1 \leq 0.1$)

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

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

PGA = 6% (Fig. 22-7)

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

Response Modification Coefficient: R (Table 12.2-1)

Floors 1-3 ⇒ ordinary reinf. conc. shear walls ⇒ R=5
Floors 4-5 ⇒ ordinary reinf. conc. moment frame ⇒ R=3

Overstrength Factor: Ω_0 (Table 12.2-1)

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

Deflection Amplification Factor: C_d (Table 12.2-1)

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

Controlling Values

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

Sect. 12.2.3.1 ⇒ 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 ⇒ $V = C_s W$

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

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

↳ Seismic Importance Factor (Table 1.5-2 ⇒ 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 ⇒ $S_{D1} \leq 0.1$)

$T \leq T_L$ ⇒ 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 $< L_r$ ⇒ use $L_r = 30$ psf

Floor Dead Loads ⇒ 8", 12", 14", & 16"

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

12" ⇒ 165 psf

14" ⇒ 190 psf

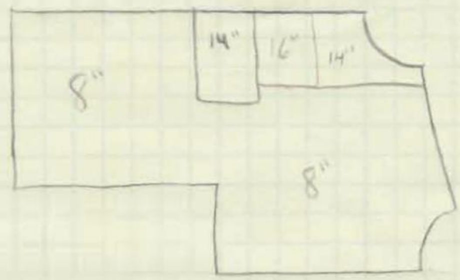
16" ⇒ 215 psf

15 psf is extra assumed allowance

Seismic

Break Down of approximate areas for slabs by floor

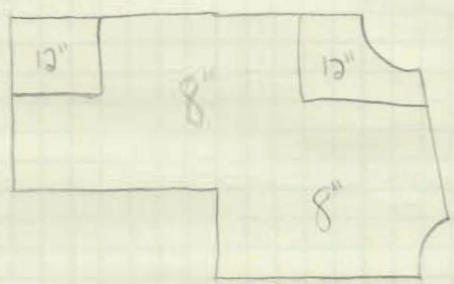
1st Floor



14" ⇒ 4674 ft^2
16" ⇒ 1380 ft^2
8" ⇒ 30645 ft^2

* 1st Floor isn't *
included

2nd and 3rd Floor



12" ⇒ 4975 ft^2
8" ⇒ 31978 ft^2

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

Non Green Roof
12" = 2212 ft^2
8" = 23349 ft^2

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

$$W_{rf} = 19984(182) + 889.8\left(\frac{13.33}{2}\right)(30) + \underbrace{166(3550) + 257.67(12)(30)}_{\text{Penthouse}}$$
$$= 4296\text{K}$$

Floor Load

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

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

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

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

Total DL = 22268 K

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

Additional Structural weight

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

12" x 30" = 2.5 SF x 4 columns = 10 SF ⇒ 133.3 ft³ $\boxed{20\text{ K}}$

3rd ⇒ 342.8 + 20 = $\boxed{362.8\text{ K}}$

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

5th ⇒ 121 SF x 14.67 = 1775 ft³

Non typical column height

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

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

$$\frac{0.985 - 0.5}{2.5 - 0.5} = \frac{x-1}{2-1}$$

$2x - 2 = 0.485$

$x = 1.24 = K$

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

5th ⇒ 0.218
4th ⇒ 0.172
3rd ⇒ 0.17
2nd ⇒ 0.109

$\Sigma = 1.0$ ✓

Story Forces

$F_R = 0.33(712.6) = 235\,K$

$F_5 = 155.3\,K$

$F_4 = 122.4\,K$

$F_3 = 121.5$

$F_2 = 77.7\,K$

Overturning Moment

$$(235 \times 83) + (155.3 \times 68.33) + (122.4 \times 55) + (121.5 \times 41.67) + (77.7 \times 28.3) = \boxed{44110\ \text{ft·K}}$$

Appendix C

Shear Distribution

$$\text{Rigidity}_y = \frac{E}{\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)}$$

for fixed

$E = 57000\sqrt{f'c} = 57000\sqrt{4000}$
$= 3605000$

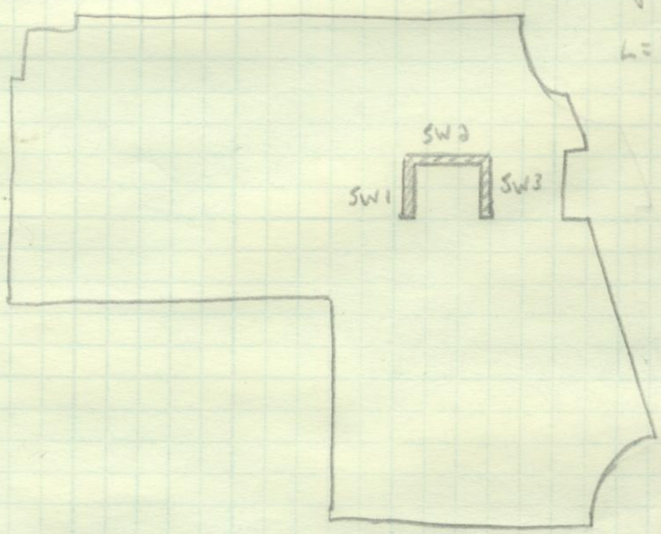
12" thickness

h = height (28.3' ground floor
13.3' 2nd + 3rd)

L = length

$L_1 = L_3 = 9'5''$

$L_2 = 26'8''$



assume → N
so that SW2 is in
N-S direction only
SW1 + 3 are in E-W
direction only

Ground floor

$$R_1 = R_3 = \frac{3605000(12)}{\left(\frac{340}{113}\right)^3 + 3\left(\frac{340}{113}\right)} = 1239.2$$

$$R_2 = \frac{3605000(12)}{\left(\frac{340}{320}\right)^3 + 3\left(\frac{340}{320}\right)} = 9861$$

2nd + 3rd Floors

$$R_1 = R_3 = \frac{3605000(12)}{\left(\frac{160}{113}\right)^3 + 3\left(\frac{160}{113}\right)} = 6104.5$$

$$R_2 = \frac{3605000(12)}{\left(\frac{160}{320}\right)^3 + 3\left(\frac{160}{320}\right)} = 26622$$

SW1

$2^{nd} \Rightarrow \frac{6104.5}{27260.75} \times 100 = 22.4\%$

$3^{rd} \Rightarrow \frac{6104.5}{41784.25} \times 100 = 14.6\%$

SW2

$\frac{26622}{61320.44} \times 100 = 43.4\%$

$\frac{26622}{117893.1} \times 100 = 22.6\%$

SW3

Values = SW1

4th & 5th

all forces resisted by moment frames

Total story lateral stiffness values taken from RAM

Direct Shear

$$V_i^d = \frac{R_i}{\Sigma R_i} \times V$$

N-S: 2nd Floor $V_2^d \Rightarrow 0.434 \times 77.7$

$= 33.7K$

E-W: 2nd Floor $V_1^d \Rightarrow V_3^d \Rightarrow .224 \times 77.7 = 17.4K$

N-S: 3rd Floor $V_2^d \Rightarrow 0.226 \times 121.5$

$= 27.5K$

E-W: 3rd Floor $V_1^d \Rightarrow V_3^d \Rightarrow 0.146 \times 121.5$

$= 17.7K$

Torsional Shear

$$V_i^t = \frac{V e d_i R_i}{J}$$

2nd story: $e_x = 32'$, $e_y = 12.5'$

3rd story: $e_x = 29.6'$, $e_y = 9.8'$

2nd

$$V_1^t = \frac{77.7(32)(17.2)(6104.5)}{1.89e^7}$$

$= 13.8K$

$$V_3^t = \frac{77.7(32)(44.9)(6104.5)}{1.89e^7}$$

$= 36.1K$

$$V_2^t = \frac{77.7(12.5)(13.45)(26622)}{1.89e^7}$$

$= 18.4K$

$J = \Sigma R_i d_i^2 = (6104.5)(17.2)^2 + (6104.5)(44.9)^2$
$+ (26622)(13.45)^2$
$= 1.89e^7$ (2nd Floor)

$J = (6104.5)(20.5)^2 + (6104.5)(48.2)^2$
$+ (26622)(13.4)^2$
$= 2.15e^7$ (3rd floor)

3rd

$$V_1^+ = \frac{121.5(29.6)(205)(6104.5)}{2.15e^7}$$

$$= 20.9K$$

$$V_3^+ = \frac{121.5(29.6)(48.2)(6104.5)}{2.15e^7}$$

$$= 49.2K$$

$$V_2^+ = \frac{121.5(9.8)(13.4)(26622)}{2.15e^7}$$

$$= 19.76K$$

Appendix D

Shear Wall Spot Checks

Shear Wall Spot Checks

$$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'c} + \rho_t f_y) \qquad \text{ACI 318-08} \quad 21.9.4$$

α_c : $\frac{h_{w_1}}{l_{w_1}} = \frac{13.3}{9.42} = 1.4 \leq 1.5$

use 3.0

$\frac{h_{w_3}}{l_{w_3}} = 1.4 \Rightarrow$ use 3.0

$\frac{h_{w_2}}{l_{w_2}} = \frac{13.3}{26.67} = 0.5 \leq 1.5$

use 3.0

Shear Wall 1 + 3

$V_n = \overset{0.75}{\phi}(1356)[(3.0)(1.0)\sqrt{4000} + .0061(60000)]$

$\phi V_n = 565.2$ K

$\lambda = 1.0$ for NW conc.

$f'c = 4000$ psi

$f_y = 60000$ psi

A_{cv} = length × thickness

$A_{cv_1} = A_{cv_3} = (9.42 \times 12)(12) = 1356 \text{ in}^2$

$A_{cv_2} = (26.67)(12)(12) = 3840 \text{ in}^2$

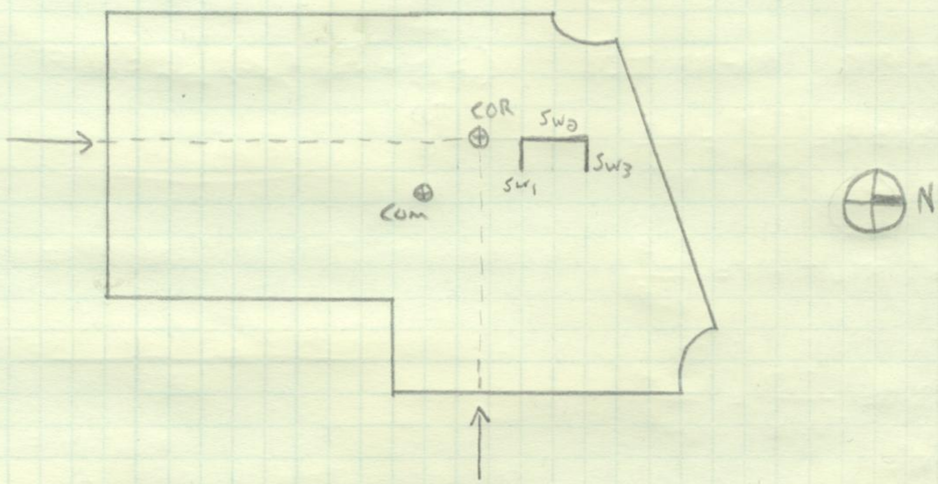
$\rho_{t_1} = \rho_{t_3} \Rightarrow (2)\#6 @ 12"$ O.C.

$\frac{2 \times 0.44}{12 \times 12} = 0.0061$

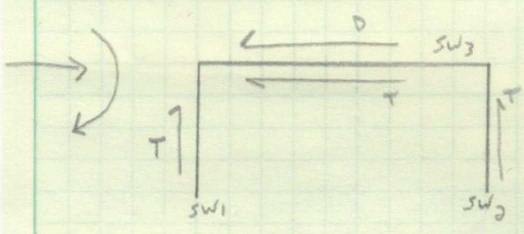
spacing, thickness

$\rho_{t_2} \Rightarrow (2)\#6 @ 12"$ O.C. $\Rightarrow 0.0061$

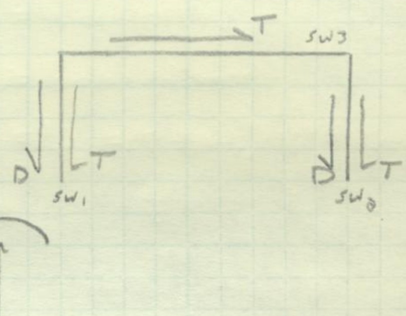
Direction of shear



N-S



E-W



Floor	2	2	2	3	3	3
Shear Wall	1	3	2	1	3	2
(K) V_u (N-S)	13.8	36.1	35.1	20.9	49.2	45.9
(K) V_u (E-W)	28.5	50.8	18.4	43.9	72.2	19.76
$\leq \phi V_n$?	Yes	Yes	Yes	Yes	Yes	Yes