800 NORTH GLEBE

Arlington, VA

Technical Report 1

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Structural Option

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Executive Summary

The structural concepts and existing structural conditions report describes the physical conditions for the structure and relative design concepts of 800 North Glebe. 800 North Glebe is a ten story mixed-used office building that will redefine the Ballston, Virginia skyline. Smooth curves created by three sail-like curtain walls and stainless-steel accents adorn the elegant façade. An iconic architectural design had tremendous impact on the structural system. Foundations, slabs, framing and lateral force resisting systems were designed and analyzed with great care to not alter the architectural design. All of the structural components were examined so that an overview could be presented on how they work together with one another.

Existing drawings, specifications and geotechnical conditions were provided by Structura, the engineer of record on the project. These conditions were compared to the loads and design calculations based on the most recent applicable codes and standards. Spot check calculations of framing and typical slabs are included to clarify the thesis design analysis that was performed. Whenever direct design information was not present, an educated assumption was made based on information from courses and consultant clarification.

ASCE 7-05 was the primary code used for thesis analysis calculations, and the finding were compared to the engineers values that were found using IBC 2003. Wind loads were analyzed on a simplified building shape, because the original irregular shape. This was do so simplified code procedures could be followed and a more rigorous analysis would not need to be performed. Wind loads in the East/West direction had proven to control over the North/South direction, producing a greater base shear. Seismic calculations had a controlling base shear value for the building, which was 25% greater than the base shear caused by wind loads. However, the seismic base shear was also found to be more than 50% greater than the value found by Structura, the engineer of record on the project. The variation among codes is a primary reason for the value discrepancies. Another reason for the difference may be that for thesis calculations only the shear walls were considered to take the entire seismic load, while in reality some of the forces would be distributed through the large columns.

Spot checks of gravity loads were performed on a typical one-way slab strip, a two-way flat-slab, a post-tensioned girder and an interior column. The values calculated were compared to those calculated by Structura for validity of thesis findings. Calculations for tech one used simplified analysis procedures found in ASCE 7-05, while the engineer had used a finite element computer model. The simplified methods only analyzed each member as they act in isolation, however the computer model analyzes how members work with one another in the entire system.

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Introduction

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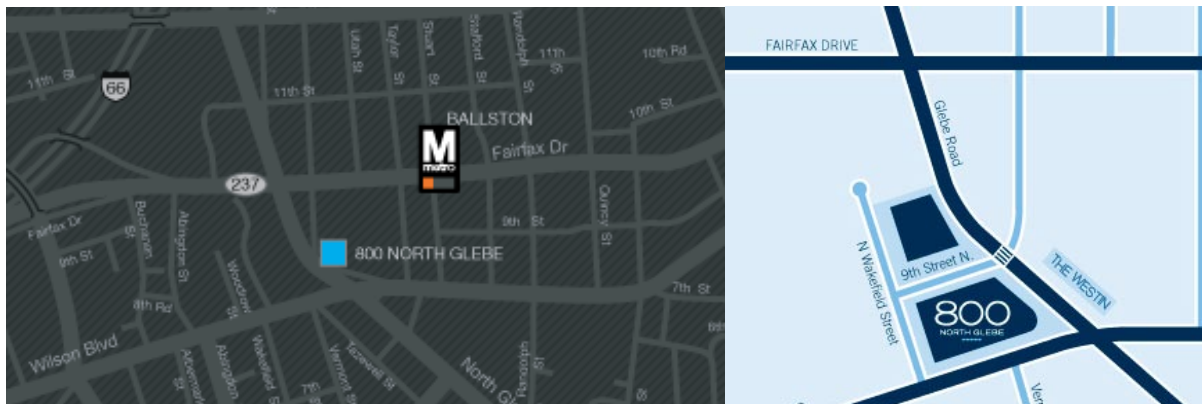
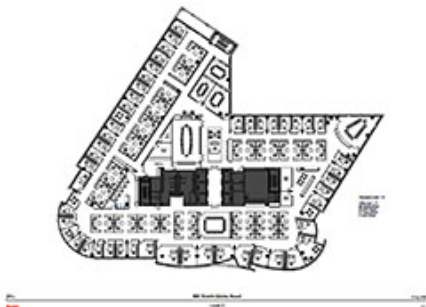**Figures 1: 800 North Glebe Site Location**

Figure 3: Level 5 Plan

Figure 2: Level 3 Plan

Figure 4: Level 8 Plan

Figure 5: Level 10 Plan

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Architectural Overview

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

Figure 6: South East Face

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

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

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

Figure 7: Sail Feature

(Figure 7 and 8) Offices on levels two through eight have 9'-0" ceiling heights, with the remaining two levels will have 10'-0" ceiling heights.

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

Figure 8: Front View

Vertical bands rising up the building are made up of precast concrete panels with a medium sandblast finish while horizontal bands consist of exposed

Figure 9: Canopy Over Main Entrance

aggregate finished panels. Other glazing found on the building is sunguard supernatural-68 on ultrawhite insulated glass and Viracon VRE 1-46 on insulated punch vision glass.

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

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System Overview

Foundation

Geotechnical studies performed by ATC Associated Inc. reported site and subsurface conditions encountered details their geotechnical recommendations for the project. Three levels of parking make up the substructure of 800 N. Glebe at roughly thirty feet below existing grade. Groundwater levels were encountered at depths ranging from approximately 22' to 37' below existing ground surface, and therefore increased uplift pressures were not a concern.

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

3 ksi normal-weight concrete (NWC) is used for the foundations and interior slab on grade, the garage SOG uses 4.5 ksi NWC and the cellar columns are composed of 4 ksi and 8 ksi. Reinforcing varies in size throughout the footings and caissons, depending on thickness.

Superstructure

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

Post-tension design allowed for greater bay sizes of 30'x46'. Greater bay sizes offer more efficient floor plates (30,000 SF) for the tenant. Girders range in size from 48" wide x 18" thick to 72" wide x 20" thick. Post tension tendons are ½" diameter with .153 square in. area low-relaxation strands with an ultimate strength of 270 ksi. A minimum of two post tension cables pass through the column reinforcement in the direction of the girder. This allows for continuous force distribution from one span to another, spanning the East/West directions.

For levels two through six, two-way mildly reinforced slabs (cyan) and one-way slabs over post-tension girders (yellow) are implemented. (Figure 10)Above level six, the superstructure consists of only one-way slabs over post-tension
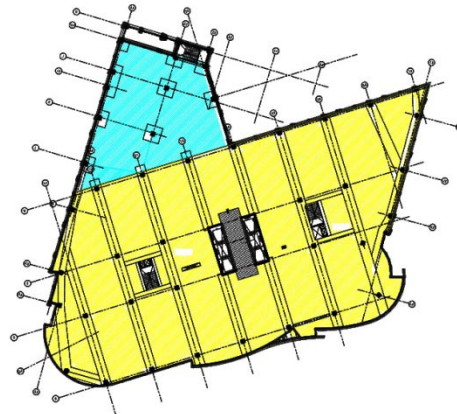
Figure 10: Slab Type Layout

girders. Two-way slabs are 10.5" thick and generally reinforced with #5 @ 10" in both directions. Drop panels in these areas are typically 10'-0"x10'-0"x7.5" to alleviate punching shear at the columns. Slabs over the 36" diameter column are 12" thick with #5 @ 12" parallel to the girder and #6 @10" perpendicular to the girders, due to the cantilever action.

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

Lateral System

Shear walls in the core of the building provide the entire lateral support as designed (Figure 11). Two 12"thick "C" shaped walls, 31.83' long East/West and 9.58' long North/South per each "C", encase the elevator banks and are reinforced with #4 horizontally and #5 vertically. From the sixth floor down, walls running North/South are specially reinforced three feet from each end with #7 and #8 rebar. All of the shear walls use concrete with a compressive strength of f'_c= 8 ksi. Building drift criteria for wind loads is L/400 or 3/8" inter-story drift at typical floors (12'-9" floor-to-floor) and for seismic loads is L/76 or 2" inter-story drift at typical floors (12'-9" floor-to-floor).
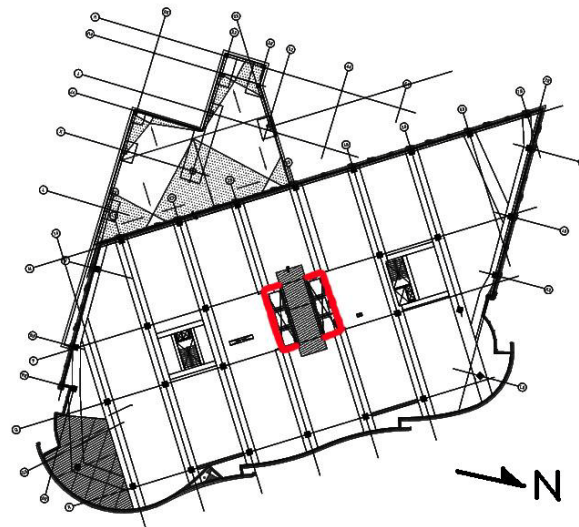
Figure 11: Shear Wall Location

Great care was given to limit the size and number of shear walls so as to not modify the floor layouts. However, since the building primarily consists of reinforced concrete, part of the lateral forces could be distributed through these members. RAM Frame was used by Structura to calculate the lateral forces acting on the building. The use of the program took all load combinations into account and analyzed the applied diaphragm and story forces. Future calculations will show how the overall structural system reacts to the lateral forces caused by wind and seismic.

Design Codes and Standards

Original Design:

- International Building Code, 2003

- Virginia Uniform Building Code, 2003

- American Society of Civil Engineers (ASCE)
    - ASCE 7-02, Minimum Design Loads for Buildings and Other Structures

- American Concrete Institute (ACI)
    - Building Code Commentary 318-02
    - Structural Concrete for Buildings, ACI 301

- America Institute of Steel Construction (AISC)
    - Manual of Steel Construction, Thirteenth Edition, 2005

Thesis Design with Additional References:

- International Building Code, 2006

- Virginia Uniform Building Code, 2003

- American Society of Civil Engineers (ASCE)
    - ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

- American Concrete Institute (ACI)
    - Building Code Commentary 318-08

- America Institute of Steel Construction (AISC)
    - Manual of Steel Construction, Thirteenth Edition, 2005

Deflection Criteria

Horizontal Framing Deflections:

- **Live Load**
  - **< L/600 or ½"**

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

Lateral Drift:

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

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

Main Structural Elements Supporting Components and Cladding:

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

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

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

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

Materials
Steel:

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

Concrete:

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

Reinforcement:

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

Post Tensioning:

Tendons	**270 ksi (A416)**

Cold Formed Steel:

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

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

Building Loads

Live Loads

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

Live Loads				
Description	Location	Designer Loads	(ASCE 7-05)	Thesis Loads
Parking	P3	40	40	40
Stairs	P3	100	100	100
Parking	P2	40	40	40
Stairs	P2	100	100	100
Parking	P1	40	40	40
Stairs	P1	100	100	100
Garage Entry	Level 1	250	50	250
Main Retail/Assembly	Level 1	100 125 250	100	100
Elevator Lobby	Level 1	100	100	100
Entry	Level 1	100	100	100
Loading Dock	Level 1	350		350
Yards and Terraces	Level 1	100	100	100
Marquees and Canopies	Level 2	75	75	75
Corridors Above First Floor	Level 2-10	100	80	80
Walkways and Elevated Platforms		60	60	60
Mechanical	Penthouse	150	125	125
Roof	Roof	30	20	20
Live loads are not permitted to be reduced at garage floor levels				

Table1: Building Live Loads

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Dead Loads

Building dead loads and their general description are laid out in table 2 below. A more detailed description of how the dead loads were calculated can be found in the Appendix. Slab areas were taken from CAD floor plans provided by the designer and varied by floor because of the curves and the major setback at levels four, six and eight. Four slab thicknesses of 7 ½", 9", 10 ½" and 12" are used per floor depending on the location and usage. The 7 ½" slab thickness is located between the elevator banks, primarily because the area is minimal. Two-way mildly reinforced slabs located on levels two though six have slab thicknesses of 10 ½" with 7" thick drop panels to reduce the punching shear around the columns. Across the Post tensioned (PT) girders is the 9" one-way slab. Located at the main entrance is a 36" diameter column rising from the ground to the top of the building with a 12" cantilevered slab. The 12" slab was needed because of the increased moment the cantilevered section caused over the beam.

Dead Loads				
Description	Location	Designer	Superimposed Dead Load	Thesis Loads
Concrete	All Levels	150 pcf		150 pcf
Partitions,Finishes	All Levels		20 psf	20 psf
MEP	All Levels		5 psf	5 psf
Precast Panels	Curtain Wall		35 psf (20%)	20 psf
Curtain Glass	Curtain Wall		15 psf (80%)	

Table 2: Building Dead Loads

Snow Loads

Snow loading was based on tables and charts found in ASCE 7-05. A summary of design criteria values used in calculations are shown below in table 3, while full calculations are found in the Appendix. Snow calculation values were only considered to act on the penthouse roof. Since the penthouse is surrounded by a screen wall, drifting loads were addressed and also used in analysis of the roof structure.

Snow Loads		
Description	Designer Loads	Calculated Loads
P_g (psf)	25 psf	25
I_s	1.0	1.0
C_e	1.0	1.0
C_t	1.0	1.0
P_f (psf)	20 psf	17.5

Table 3: Building Snow Loads

Design Analysis and Conclusion

Wind Analysis

ASCE 7-05 was the governing resource for wind load calculations. Section 6.5 describes *Method 2 – Analytical Procedure* for main wind-force resisting systems (MWRS) for enclosed buildings. Exposure, height, topographic effects, wind direction and wind velocity all played a part in determining velocity pressures. In conjunction with gust effect factors, external and internal pressure coefficients, and force coefficients I was able to eventually determine the base shear for the building. Table 4 outlines the variables used in analysis, and the calculations are shown in the Appendix.

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

Table 4: Building Wind Load Variables

Basic assumptions about the building needed to be made so the analytic procedure could be performed. These include 1) the structure is regularly shaped having no geometric irregularities, 2) the structure responds to wind primarily in the same direction as that of the wind, and 3) the structure is located in a site where there are no channeling effects or buffeting in the wake of upwind obstructions. Curves of the façade were simplified into a generalized rectangle around the building footprint as shown in figure 12. The blue line surrounding the building is the rectangular shape used in the calculations. Because of this simplification, the only conditions analyzed are windward and leeward. Future wind calculations will include a more in-depth analysis of pressures acting along different axis.
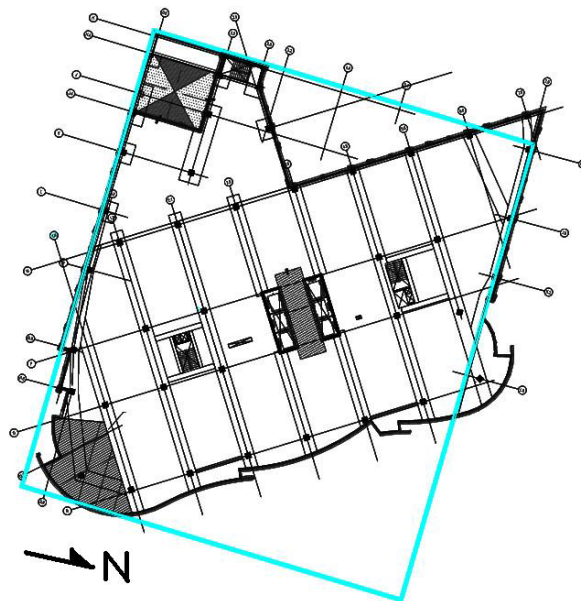

Figure 12: Building Shape adjustment for wind calculations

Tables 5 and 6 show how the forces act on the building in the North/South direction while tables 7 and 8 show the forces acting in the East/West directions respectively. Figures 13 and 14 depiction how these pressures act on the building at each level.

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Floor	Story Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)		Force of Windward Pressure (k)	Story Shear Windward (k)	Moment Windward (ft-k)
					Windward	Leeward			
PH Roof	0	153.75	1.12	19.70	16.45	-10.81	29.87	0.00	4593.03
PH	18.5	135.25	1.08	18.99	15.99	-10.81	51.56	29.87	6972.85
10	13.75	121.5	1.04	18.41	15.61	-10.81	42.80	81.43	5200.02
9	13.75	107.75	1.01	17.79	15.21	-10.81	40.17	124.23	4327.97
8	12.75	95	0.97	17.16	14.79	-10.81	37.52	164.39	3564.02
7	12.75	82.25	0.93	16.47	14.34	-10.81	36.28	201.91	2983.92
6	12.75	69.5	0.89	15.70	13.83	-10.81	34.88	238.19	2424.17
5	12.75	56.75	0.84	14.82	13.25	-10.81	33.26	273.07	1887.40
4	12.75	44	0.78	13.78	12.57	-10.81	31.30	306.33	1377.14
3	12.75	31.25	0.71	12.49	11.73	-10.81	28.75	337.63	898.42
2	12.75	18.5	0.61	10.76	10.59	-10.81	13.64	366.38	252.37
1	18.5	0	0.00	0.00	0.00		0.00	380.02	0.00

Table 5: N/S Windward Pressures

Σ Windward Story Shear (kips)=	380.02
Σ Windward Moment(ft-kips)=	34481.31

Wind Loads (N-S)

Floor	Story Height (ft)	Height Above Ground (ft)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)	Moment Total (ft-k)
PH Roof	0	153.75	27.26	50.06	0.00	7697.22
PH	18.5	135.25	26.79	86.75	50.06	11733.08
10	13.75	121.5	26.42	72.81	136.81	8846.46
9	13.75	107.75	26.01	69.09	209.62	7444.16
8	12.75	95	25.60	65.35	278.71	6207.80
7	12.75	82.25	25.15	64.11	344.06	5272.87
6	12.75	69.5	24.64	62.71	408.17	4358.30
5	12.75	56.75	24.06	61.09	470.87	3466.71
4	12.75	44	23.38	59.13	531.96	2601.63
3	12.75	31.25	22.54	56.58	591.09	1768.08
2	12.75	18.5	21.40	27.56	647.67	509.79
1	18.5	0	0.00	0.00	675.22	0.00

Table 6: N/S Total Pressures

Σ Total Story Shear (kips)=	675.22
Σ Total Moment (ft-kips)=	59006.11

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BASE SHEAR : 675.22 kips
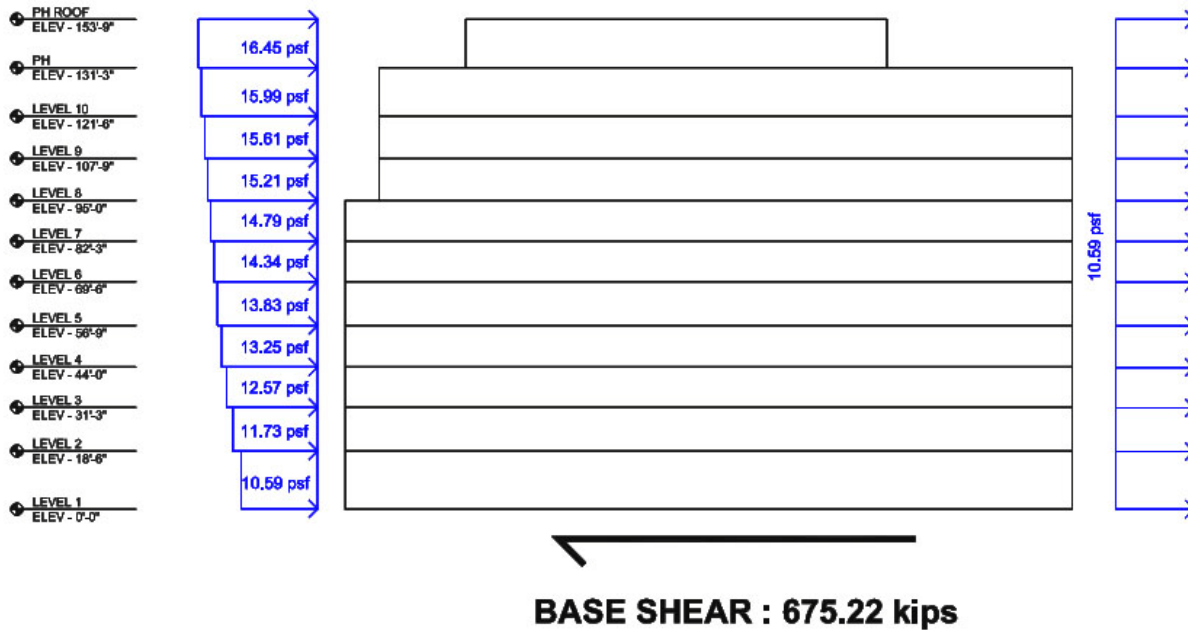
Figure 13: N/S Wind Load Diagram

Wind Loads (E-W)									
Floor	Story Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)		Force of Windward Pressure (k)	Story Shear Windward (k)	Moment Windward (ft-k)
					Windward	Leeward			
PH Roof	0	153.75	1.12	19.70	16.43	-11.60	31.01	0.00	4767.46
PH	18.5	135.25	1.08	18.99	15.96	-11.60	53.51	31.01	7237.69
10	13.75	121.5	1.04	18.41	15.59	-11.60	44.42	84.52	5397.59
9	13.75	107.75	1.01	17.79	15.18	-11.60	41.69	128.95	4492.46
8	12.75	95	0.97	17.16	14.77	-11.60	38.94	170.64	3699.54
7	12.75	82.25	0.93	16.47	14.32	-11.60	37.66	209.58	3097.42
6	12.75	69.5	0.89	15.70	13.81	-11.60	36.21	247.24	2516.44
5	12.75	56.75	0.84	14.82	13.23	-11.60	34.52	283.45	1959.29
4	12.75	44	0.78	13.78	12.55	-11.60	32.49	317.97	1429.65
3	12.75	31.25	0.71	12.49	11.72	-11.60	29.85	350.47	932.72
2	12.75	18.5	0.61	10.76	10.58	-11.60	14.16	380.31	262.01
1	18.5	0	0.00	0.00	0.00	0.00	0.00	394.47	0.00

Table 7: E/W Windward Pressures

Σ Windward Story Shear (kips=	394.47
Σ Windward Moment (ft-kips)=	35792.26

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Wind Loads (E-W)						
Floor	Story Height (ft)	Height Above Ground (ft)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)	Moment Total (ft-k)
PH Roof	0	153.75	28.02	53.53	0.00	8230.66
PH	18.5	135.25	27.56	92.78	53.53	12548.46
10	13.75	121.5	27.18	77.91	146.31	9465.77
9	13.75	107.75	26.78	73.96	224.22	7969.06
8	12.75	95	26.37	69.99	298.18	6649.08
7	12.75	82.25	25.91	68.71	368.17	5651.11
6	12.75	69.5	25.41	67.26	436.88	4674.26
5	12.75	56.75	24.83	65.57	504.13	3721.25
4	12.75	44	24.15	63.54	569.70	2795.75
3	12.75	31.25	23.31	60.89	633.24	1902.96
2	12.75	18.5	22.17	29.69	694.14	549.20
1	18.5	0	0.00	0.00	723.83	0.00

Table 8: E/W Total Pressures

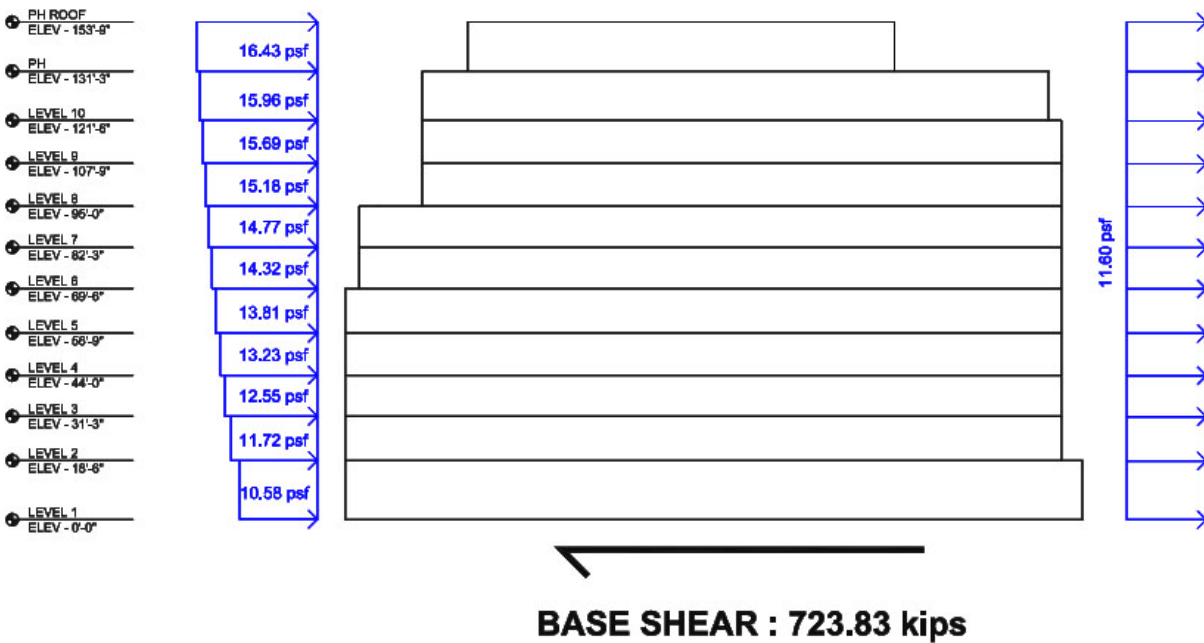| Σ Total Story Shear (kips)=	723.83
Σ Total Moment(ft-kips)=	64157.56

Figure 14: E/W Wind Load Diagram

Values found from thesis calculations are slightly different than those calculated by Structura. Structura had used RAM International to find the loads and applied forces acting on the building. RAM uses IBC 2003 load cases when calculating, while thesis calculations were performed with ASCE 7-05. Variations among equations are minimal, but may be a cause for discrepancies between findings.

Seismic Analysis

Seismic calculations of 800 North Glebe were based upon ASCE 7-05 for thesis design. The engineering firm had used ASCE 7-02 / IBC 2003 and the 2003 Virginia Uniform Statewide Building Code to calculate the base shear from the equivalent lateral force analysis procedure. A large difference among the calculated base shears was found and discussed with Structura and my consultant. It was determined that variations in base shear may be observed when referencing separate codes and standards. Another variation may be because of building weight assumptions used for thesis calculations while Structura had used exact weights determine by their RAM model. Floor weight calculation used for thesis can be found in the Appendix.

One example of these differences is the value used for the spectral response acceleration (S_s and S_1). My values of $S_s=0.154$ and $S_1=0.051$ were found directly from the USGS Ground Motion Parameter Calculator compared to their values of $S_s=0.179$ and $S_1=0.063$. Therefore, since spectral response values are used to determine site modification acceleration values, a variation such as the ones observed can lead to major differences in the final base shear.

Design criteria variables used for thesis analysis can be found below in table 9. Variables used by Structura are located on the left column of the table for comparison. Design criteria variables were then used to determine story forces at each level, story shear at each level, base shear and building overturning moment, located in table 10. Figure 15 was constructed to display how these forces acted on the building, while calculations to support the excel graph below are located in the Appendix.

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Design Criteria Variables				Structura
Seismic Use Group		Group II		Group II
Site Class		D	Geotech Report	D
Importance Factor	I_e	1.00	Table 11.5-1	1.0
Spectral Response Acceleration, Short	S_s	0.154	USGS	0.179
Spectral Response Acceleration, 1s	S_1	0.051	USGS	0.063
Site Coefficient	F_a	1.6	Table 11.4-1	
Site Coefficient	F_v	2.4	Table 11.4-2	
Soil Modified Acceleration	S_{MS}	0.2464		
Soil Modified Acceleration	S_{M1}	0.1224		
Design Spectral Response, short	S_{ds}	0.164	USGS	0.191
Design Spectral Response, 1s	S_{d1}	0.082	USGS	0.101
Response Modification Coefficient	R	5.5	Table 12.2-1	5.5
Seismic Design Category		B	Table 11.6-1	B
Approx. Period Parameter	C_t	0.02	Table 12.8-2	
Building height (above grade)	h_n	153.75		
Approx. Period Parameter	x	0.75	Table 12.8-2	
Approx. Fundamental Period	T_a	0.873	Eq. 12.8-7	1.1
	T_s	0.500	11.4.5	
Calculated Period Upper Limit Coefficient	C_u	1.7	Table 12.8-1	1.698
Seismic Response Coefficient	C_s	0.0298	Eq. 12.8-2	
	C_{smax}	0.0171		
Structural Period Exponent	k	1.45	12.8.3	1.684
Long Period Transition Period	T_L	6	Figure 22-15	
Building Weight (k)	W	59784		73.181.57
Base Shear: Cs x W=	V	1020.70		578
Basic Seismic-Force Resisting System		Ordinary Reinforced Concrete Shear Walls	Structural Plans	
Effects from the below grade parking garages were not taken into account for calculations				

Table 9: Seismic Design Variables

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Seismic Design Loads									
Floor	Story Height (ft)	Height Above Ground h_x (ft)	Story Weight w_x (kips)	$h_x^{\ k}$	$w_x h_x^{\ k}$	C_{vx}	Story Force F_x (kips)	Story Shear V_x (kips)	Moment M_x (ft-k)
PH Roof	0	153.75	698.05	1482.12	1034596.31	0.03	32.03	0.00	4924.05
PH	18.5	135.25	5304.52	1230.69	6528219.75	0.20	202.08	32.03	27331.83
10	13.75	121.5	5210.83	1053.50	5489622.66	0.17	169.93	234.11	20646.93
9	13.75	107.75	5165.42	885.13	4572049.58	0.14	141.53	404.04	15249.82
8	12.75	95	5417.99	737.39	3995192.56	0.12	123.67	545.57	11748.92
7	12.75	82.25	5597.77	598.34	3349359.22	0.10	103.68	669.25	8527.75
6	12.75	69.5	6177.61	468.68	2895333.73	0.09	89.63	772.93	6229.03
5	12.75	56.75	6221.57	349.34	2173454.88	0.07	67.28	862.55	3818.15
4	12.75	44	6353.88	241.55	1534777.91	0.05	47.51	929.83	2090.43
3	12.75	31.25	6250.91	147.07	919339.34	0.03	28.46	977.34	889.33
2	12.75	18.5	6996.27	68.77	481133.48	0.01	14.89	1005.80	275.53
1	18.5	0	389.36	0.00	0.00	0.00	0.00	1020.70	0.00
				Total	32973079.4		1020.7		101731.76

Table 10: Seismic Design Loads

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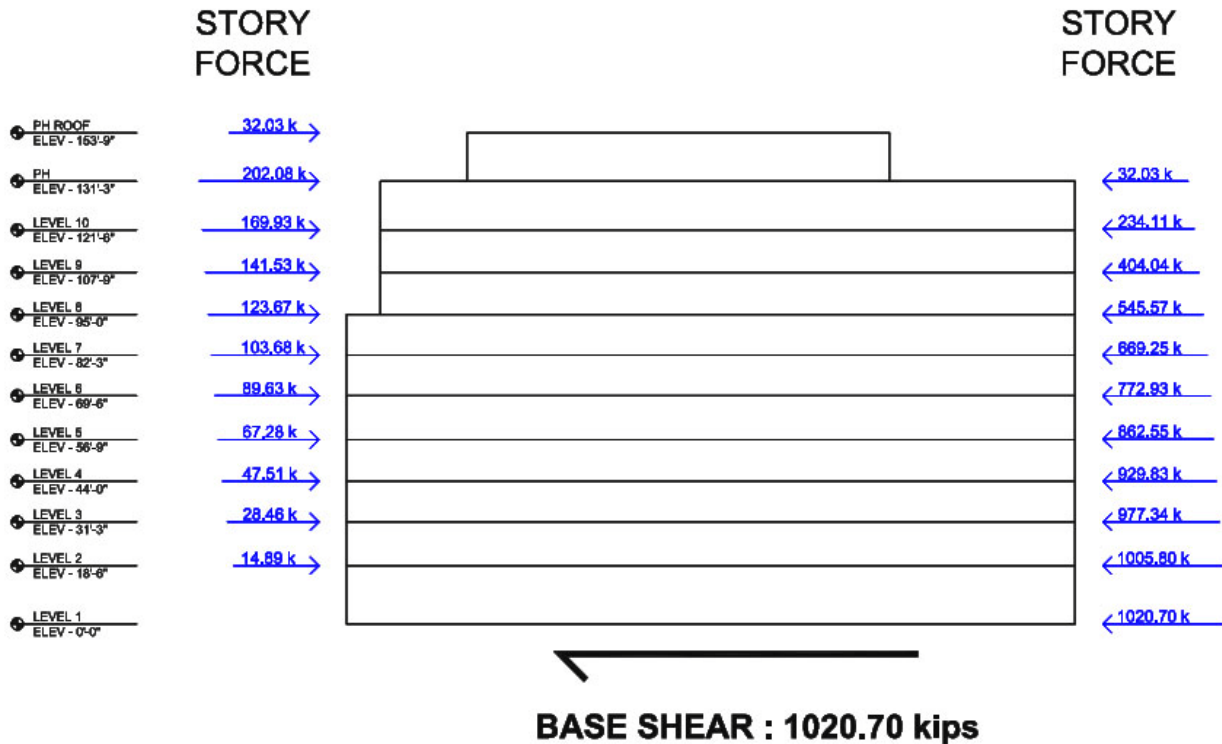
Figure 15: Seismic Force Diagram

Spot Checks

Spot checks of typical framing areas were performed to verify that loading conditions were adequately designed. Only gravity loading was used for spot check analysis, and therefore variations are to be expected between thesis analysis and actual design. Full analysis calculations to support the conclusions can be found in the Appendix.

Slab Analysis

Two different slab spot checks were performed, one for the one-way slab and another for the two-way flat-slab system. Figure 16 illustrates the typical one-way slab strip that was analyzed while figure 16 illustrates the two-way flat-plate slab system that was analyzed.

Figure 16: 1-way slab strip

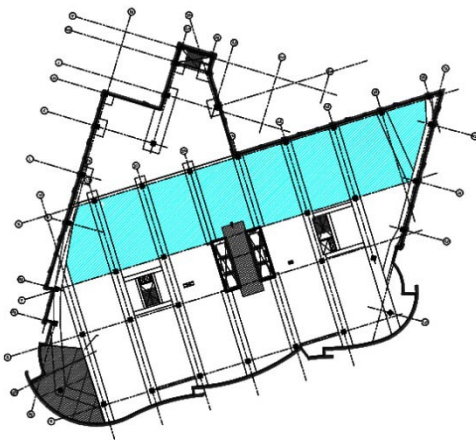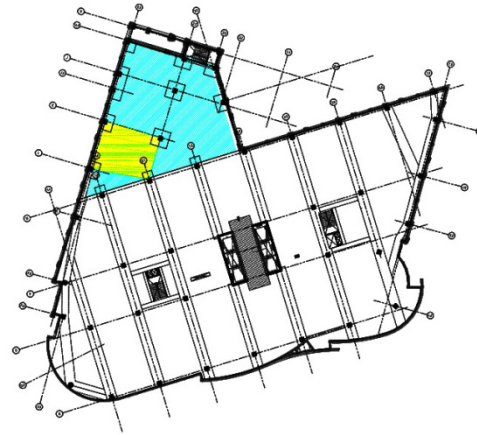Figure 17: 2-way flat-plate system

 All of the levels of the superstructure employ a one-way slab system over post-tensioned girders. Slab thickness is 9" with concrete compressive strength of f'$_c$= 5000 psi. Construction of the slab and girders appeared to be monolithically cast as a single piece, but further investigation determined the girder's concrete compressive strength was f'$_c$= 4000 psi. Because of these finding, the strip was analyzed as a solid slab with both ends continuous.

The slab met all provisions of ACI 318-8 *approximate method of frame analysis*, therefore this method was utilized. Structura had used RAM Concept, which employs three dimensional finite element analysis. Finite element analyzes how the each element works together with entire system, and therefore variations were expected.

Upon completion of the approximate frame analysis method, thesis finding were compared to those calculated by the engineer. Thesis calculations had determined that the slab thickness to be 10.5" compared to the 9" as designed. The amount of steel reinforcement in the slab was found to be less for top reinforcement while bottom reinforcement was found to be greater than Structura design.

ACI 318-08, *Direct Design Method* was used to analyze the two-way slab system. The slab area did not meet all the provisions for this method, but assumptions needed to be made for preliminary analysis. It was assumed that there were three continuous spans in either direction and panels are rectilinear with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2. The yellow colored bay in figure 17 met these provisions the best and therefore was used for calculations. Comparisons of reinforcement in the column and middle strips are found in the Appendix. Variations among the reinforcement are primarily due to my general assumption of the slab meeting all the criteria for the *Direct Design Method*. Future checks involving computer modeling will utilize more accurate methods of analysis. Drop Panels were not taken into account for the slab calculations because of their irregularities.

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Beam Analysis

A post-tensioned girder was examined using the simplified method of load balancing provided by Mr. Richard Apple of Holbert Apple Associates. The girder being analyzed is shaded cyan in figure 18, which spans between 4 columns. The two outer spans are of equal length (44'-0") while the interior span is 14' shorter (30'-0").   Preliminary span-depth ratios were performed and found to be equal to the thickness designed by the engineer. The force acting in the tendons was also found to be very close to the value as designed. From these

Figure 18: Post-tensioned Girder

finding is was concluded that a the load balancing method of one span provided moderately accurate values, but an analysis of the entire span will with a more complete computer program will be provided in the future.

Column Analysis

An interior column of the ninth floor was used for the column spot check (figure 19). Only gravity loads were applied to the column and compared to the maximum allowable loads axial and moment loads that were calculated. It was found that the column could handle a moment of 900 ft-kips if acting along with the 839 kip axial load. Calculations to support the column analysis conclusion can be found in the Appendix.

Figure 19: Ninth Floor Column

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Appendix A: Gravity Loads

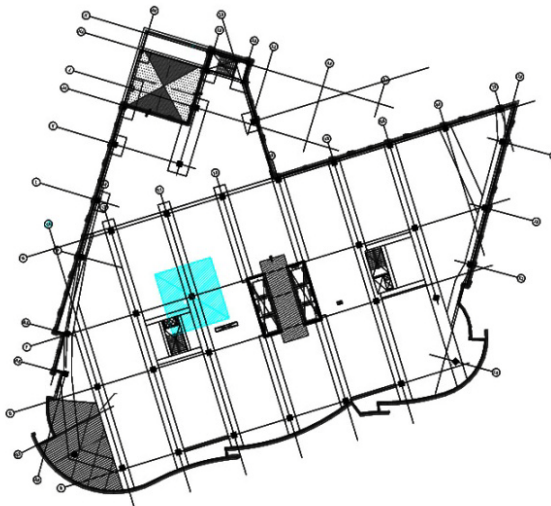| Floor Perimeter (ft)	Floor Area	Floor to Floor Height	Superimposed Dead Load (#)	Floor Weight (#)	Column Weight Above (#)	Column Weight Below (#)	Beam Weight (#)	Curtain Wall Weight (#/ft)	Curtain Wall Weight (#)
250	3208.0	0.0	132550.0	363476.3	0.0	155592.7	0.0	185	46250
717	25572.5	18.5	639313.2	3118897.1	155592.7	188187.9	687975.0	322.5	231232.5
755	26640.5	13.8	666012.9	3036057.4	188187.9	188187.9	714150.0	275	207625
755	26559.5	13.8	663987.9	3028407.4	188187.9	175312.9	714150.0	265	200075
760	26607.5	12.8	665187.9	3033132.4	175312.9	182868.7	972400.0	255	193800
766	27806.8	12.8	695169.1	3167335.5	182868.7	182868.7	972400.0	255	195330
892	32687.8	12.8	817194.1	3803996.8	182868.7	230150.0	714150.0	255	227460
898	32545.3	12.8	813633.0	3787743.5	230150.0	230150.0	729112.5	255	228990
898	33394.3	12.8	834858.0	3901671.0	230150.0	238782.8	717637.5	255	228990
869	32750.3	12.8	818757.3	3784200.1	238782.8	238782.8	747000.0	255	221595
880	35847.3	12.8	896182.3	4248281.4	238782.8	389173.3	747000.0	312.5	275000
		18.5	0.0	0.0	389173.3	0.0	0.0	185	0
			7642845.7	35273198.8	2400057.6	2400057.6	7715975.0	3075.0	2256347.5

Shear Wall Weight (#)	Total Building Weight (#)	Total Building Weight (k)
0	698054.0	698
282994.50	5304515.4	5305
210333.75	5210829.8	5211
195036.75	5165422.8	5165
195036.75	5417993.6	5418
201537.98	5597765.0	5598
201537.98	6177612.5	6178
201537.98	6221571.9	6222
201537.98	6353882.2	6354
201537.98	6250911.0	6251
201537.98	6996270.2	6996
0	389358.3	389
2092629.6	59784186.7	= 59784

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		Total Area	Opening	7.5" slab	10.5" slab	12" slab	Column Area	9" slab	Perimeter
PH Roof	0	3228					20	3208	
Roof	19	26484	800			6453	111	19120	717
10	14	27623	800	570		1325	182	24746	755
9	14	27542	800	570		1364	182	24626	755
8	13	27590	800	570		1346	182	24692	760
7	13	28798	800	570		1327	191	25910	766
6	13	33679	800	570	4363	1309	191	26446	892
5	13	33586	800	570	4385	1307	241	26283	898
4	13	34435	800	570	5415	1273	241	26136	898
3	13	33800	800	570	5550	0	250	26630	869
2	13	37897	1800	570	5550	1503	250	28224	880
1	19	88510	800	570	5550	0	280	81310	

Example of how each floor was calculated:

	Slabs (8)				Total	Drop Panels
Thickness (in)	12	9	7.5	10.5		0
Area (ft^2)	1346	24691.51	570	0	26607.51	0
Unit Weight	150 pcf	150 pcf	150 pcf	150 pcf		150 pcf
Total Weight (#)	201900	2777794.875	53437.5	0	3033132.375	0

Columns							
Height	12.75	Height x Unit Weight	1912.5	W_T	$W_T/2$	Area	
	Columns (8)						
	Size	Quantity	Weight				
	30 x 30	26	310781.25			162.50	
	30 x 18	1	7171.88			3.75	
	18 x 30	1	7734.38			3.75	
	30 x 26	1	10359.38			5.42	
	36" dia.	1	14578.95			7.07	
				350625.8	175312.9		182.49

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Beams														
Size	Depth (in)	Width (ft)	Length											Weight (#)
72 x 18	9	6	40	129	129	50	45	50	45	120	115	50	60	562275
36 x 18	9	3	36											12150
30 x 18	9	2.5	24	24	24	24								27000
48 x 18	9	4	117											52650
72 x 34	25	6	50	40										168750
18 x 18	9	1.5	24	12	24	18	18	20						19575
48 x 34	25	4	80	24										130000

972400

Shear Wall	Length (ft)	Thickness (ft)	Unit Weight (pcf)	Height (ft)	Wall Weight (#)
1	31.83	1	150	12.8	60874.88
2	9.58	1	150	12.8	18321.75
3	9.58	1	150	12.8	18321.75
4	9.58	1	150	12.8	18321.75
5	9.58	1	150	12.8	18321.75
6	31.83	1	150	12.8	60874.88

Total 195036.75

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Snow

GROUND SNOW LOAD: $p_g = 25$ psf
FLAT ROOF SNOW LOAD: $p_f = 20$ psf $\Big\}$ FROM ASCE 7-05

Snow Drift

PH raises 18'-6" above roof line allow for drifting
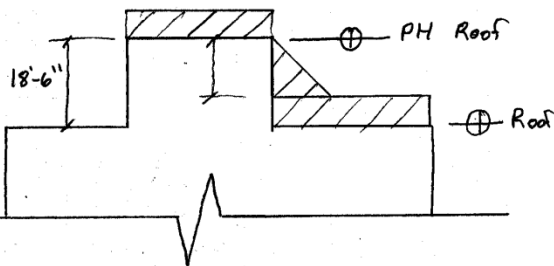
$\gamma = .13 p_g + 14 = (.13)(25) + 14 = 17.25$ pcf

$p_{s_{min}} = .7 C_e C_t I p_g = (.7)(1.0)(1.0)(1.0)(25) = 17.5$ psf
$p_{s_{min}} = 17.5 < 20 \quad \therefore \boxed{p_f = 20 \text{ psf}}$

$p_s = C_s p_f = (1.0)(20) = \boxed{20 \text{ psf}}$

Balanced Snow Ht:
$h_b = \dfrac{p_s}{\gamma} = \dfrac{20}{17.25} = \boxed{1.16 \text{ ft}}$

Clear Ht:
$h_c = 18.5 - 1.16 = 17.34'$

$h_c / h_b = \dfrac{17.34}{1.16} = 14.95' > .2 \quad \therefore$ DESIGN FOR DRIFT

$h_d = 2.64$ ft
$W = 10.56$ ft

$\therefore \quad p_g = \gamma h_d = (17.3)(2.64) = \boxed{45.5 \text{ psf}}$

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JOB TITLE Thesis		

Penn State
State College, PA

JOB NO. Tech 1		**SHEET NO.**	
CALCULATED BY RGJ		**DATE**	10/3/09
CHECKED BY		**DATE**	

VII. Snow Loads :

Roof slope	=	0.0 deg	
Horiz. eave to ridge dist (W) =		202.0 ft	
Roof length parallel to ridge (L) =		210.0 ft	

Type of Roof		Monoslope	
Ground Snow Load	Pg =	25.0 psf	
Importance Category	=	II	
Importance Factor	I =	1.0	
Thermal Factor	Ct =	1.00	
Exposure Factor	Ce =	1.0	

Pf = 0.7*Ce*Ct*I*Pg	=	17.5 psf
Pf min	=	20.0 psf

Flat Roof Snow Load	Pf =	20.0 psf	
Rain on Snow Surcharge Angle =		4.04 deg	
Code Maximum Rain Surcharge		5.0 psf	
Rain on Snow Surcharge	=	0.0 psf	
Unobstructed Slippery			
Surface (per Section 7.4) =		no	
Sloped-roof Factor	Cs =	1.00	

Design Roof Snow Load (Ps)	=	**20.0 psf**	("balanced" snow load)
Building Official Minimum	=	20.0 psf	

Exposure Factor, Ce				
		Exposure of roof		
Terrain		Fully	Partially	Sheltered
	A	n/a	1.1	1.3
	B	0.9	1.0	1.2
	C	0.9	1.0	1.1
	D	0.8	0.9	1.0
Above treeline		0.7	0.8	n/a
Alaska-no trees		0.7	0.8	n/a

NOTE: Alternate spans of continuous beams
and other areas shall be loaded with half the
design roof snow load so as to produce the
greatest possible effect - see code.

Leeward Snow Drifts - from adjacent higher roof

Upper roof length	lu =	62.0 ft	
Projection height	h =	18.5 ft	
Building separation	s =	0.0 ft	
Adjacent structure factor		1.00	
Snow density	γ =	17.3 pcf	
Balanced snow height	hb =	1.16 ft	
	hc =	17.34 ft	
hc/hb >0.2 = 15.0		**Therefore, design for drift**	
Drift height	hd =	2.64 ft	
Drift width	w =	10.56 ft	
Surcharge load:	pd = g*hd =	**45.5 psf**	
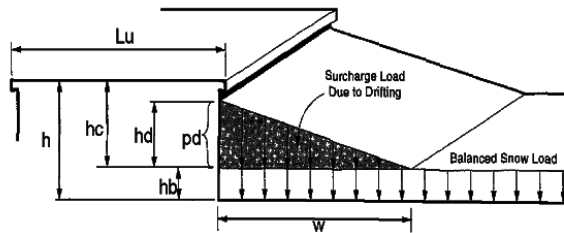
Windward Snow Drifts - Against walls, parapets, etc more than 15' long

Building roof length	lu =	212.0 ft	
Projection height	h =	3.0 ft	
Snow density	γ =	17.3 pcf	
Balanced snow height	hb =	1.16 ft	
	hc =	1.84 ft	
hc/hb >0.2 = 1.6		**Therefore, design for drift**	
Drift height	hd =	1.84 ft	
Drift width	w =	14.72 ft	
Surcharge load:	pd = g*hd =	**31.8 psf**	

Appendix B: Wind Calculations

WIND

METHOD 2 ANALYTICAL PROCEDURE OF CHAP 6

SAMPLE CALC @ LEVEL 9 IN N/S DIRECTION

$$K_z = 2.01 \cdot \left(\frac{z}{z_g}\right)^{2/\alpha}$$

$$K_{z_9} = 2.01 \left(\frac{107.75}{1200}\right)^{2/7} = 1.01$$

$$q_z = 0.00256 \, K_z \, K_{zt} \, K_d \, V^2 \, I$$
$$= (0.00256)(1.01)(1.00)(0.85)(90)^2(1.0) = 17.80$$

$$q_h = 0.00256 \, K_h \, K_{zt} \, K_d \, V^2 \, I$$
$$= (0.00256)(1.12)(1.0)(0.85)(90)^2(1.0) = 19.7$$

[C17]

$$n_1 = 100/H = 100/153.75 = .65 \leq 1 \, Hz \quad \therefore \, \underline{Flexible}$$

$$g_Q = g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$
$$= \sqrt{2\ln(2341.5)} + \frac{0.577}{\sqrt{2\ln(2341.5)}} = 4.086$$

$$\bar{z} = 0.6h = (0.6)(153.75) = 92.25 \geq z_{min} \quad \checkmark \quad OK$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = (0.3)\left(\frac{33}{92.25}\right)^{1/6} = 0.253$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = (320)\left(\frac{92.25}{33}\right)^{1/3} = 450.78$$

1.

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$$Q = \sqrt{\frac{1}{\left(1 + 0.63\left(\frac{B+H}{L_{\bar{z}}}\right)^{.63}\right)}}$$

$$= \sqrt{\frac{1}{1 + 0.63\left(\frac{202+153.75}{450.78}\right)^{.63}}} = 0.780$$

$V = 90\,mph$ from Fig 6-1

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = (0.45)\left(\frac{92.25}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 76.81$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.65)(450.78)}{76.81} = 3.82$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{(7.47)(3.82)}{(1 + 10.3(3.82))^{5/3}} = 0.0602$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{where} \quad \eta = 4.6 n_1 h / \bar{V}_{\bar{z}}$$

$$R_h = 0.153$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{where} \quad \eta = 4.6 n_1 B / \bar{V}_{\bar{z}}$$

$$R_B = 0.119$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{where} \quad \eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$$

$$R_L = 0.0358$$

z

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$$R = \sqrt{\frac{1}{\beta_n} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.17303$$

$\beta_n .02$ for concrete

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

$$\boxed{G_f = 0.81912}$$

$GC_{pi} = \pm .18$ for enclosed buildings

PRESSURE COEFF: C_p FIGURE 6-6

→ WINDWARD $C_p = 0.8$

→ LEEWARD (N/S) $C_p = -0.45$ w/ $L/B = 1.04$

(E/W) $C_p = -0.5$ w/ $L/B = 0.96$

VELOCITY PRESSURE EXPOSURE COEFF AT HT z: k_z Table 6-3

$$k_z = 2.01 \left[\frac{(h_z)}{1200} \right]^{2/7} = 0.26511 (h_z)^{2/7}$$

@ level 9: $k_z = 0.26511 (107.75)^{2/7} = \boxed{1.01}$

VELOCITY PRESSURE AT HT z: q_z Eq. 6-15

$$q_z = 0.00256 (k_z)(k_{zt})(k_d) V^2 I$$

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

$$= 17.6256 \, k_z$$

@ level 9: $q_z = 17.6256 (1.01) = \boxed{17.7936 \; \#/ft^2}$

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@ MEAN ROOF HT

$$q_h = 0.00256 \, k_h \, k_{zt} \, k_d \, V^2 I$$
$$= (17.6256)(1.11747) = \boxed{19.6961}$$

PRESSURES: p_z & p_h (Eq. 6-19)

$$p_z = q_z \, G C_p - q_h \, (GC_{pi}) \qquad \text{WINDWARD}$$
$$p_h = q_h \, G C_p - q_h \, (GC_{pi}) \qquad \text{LEEWARD}$$

N/S: W $\Rightarrow p_z = q_z(0.8191)(0.8) - (19.7)(-0.18) \Rightarrow \boxed{0.6553 q_z + 3.546}$

L $\Rightarrow p_h = (19.7)(0.8191)(-0.5) - (19.7)(0.18) \Rightarrow \boxed{-10.8073}$

E/W: W $\Rightarrow p_z = q_z(0.8175)(0.8) - (19.7)(-0.18) \Rightarrow \boxed{0.654 q_z + 3.546}$

L $\Rightarrow p_h = (19.7)(0.8175)(-.5) - (19.7)(0.18) \Rightarrow \boxed{-11.598}$

FORCE OF WINDWARD PRESSURE IN KIPS:

@ level 9: $\frac{202}{1000}\left[(15.21 psf)\left(\frac{13.75'}{2}\right) + (14.79 psf)\left(\frac{12.75'}{2}\right)\right]$
IN N/S DIR

$$\Rightarrow 0.202 \left[104.5688 + 94.2863\right] = \boxed{40.17^k}$$

STORY SHEAR (kips):

@ level 9: Sum level forces above
IN N/S DIR
$$\Rightarrow (29.87) + (51.56) + (42.8) = \boxed{124.23^k}$$

MOMENT @ Level 9:

$$\Rightarrow \text{level force} \times ht$$
$$\Rightarrow (40.17)(107.75) = \boxed{4328. \quad ft-k}$$

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Appendix C: Seismic Calculations

SEISMIC	ASCE 7-05	

SEISMIC GROUP II
SITE CLASS D
IMPORTANCE 1.0

S_S & S_1 found from USGS GROUND MOTION
Parameter calculator

→ $S_S = 0.154$ (@ .2 sec)

→ $S_1 = 0.051$ (@ 1.0 sec)

SITE COEFF:

$F_A = 1.6$ $F_V = 2.4$ FROM Tables 11.4-1 & 11.4-2 respectively
w/ SITE CLASS D

SITE MODIFIED ACCELERATION:

$S_{MS} = F_A S_S = (1.6)(0.154) = \boxed{0.2464}$

$S_{M1} = F_V S_{M1} = (2.4)(0.051) = \boxed{0.1224}$

DESIGN ACCELERATION:

$S_{DS} = \frac{2}{3} S_{MS} = \boxed{0.164}$ $\Bigg\}$ ∴DESIGN CATEGORY B

$S_{D1} = \frac{2}{3} S_{M1} = \boxed{0.082}$ FROM TABLE 11.6-2

SINCE BLDG IS LATERALLY SUPPORTED BY CONCRETE SHEAR
WALL: $C_t = 0.02$
         $x = 0.75$

$T_a = C_t h_n{}^x = (0.02)(153.75)^{.75} = \boxed{0.873s}$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.164}{0.082} = \boxed{0.50}$ ⇒ $0.8 T_s < 0.4$

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$$C_s = \text{MIN} \begin{vmatrix} S_{ds}/(R/I) & = & \dfrac{0.164}{\left(\frac{5.5}{1.0}\right)} & = & \boxed{0.30} \\ \\ \dfrac{S_{d1}}{T(R/I)} & = & \dfrac{0.082}{\left(\frac{(0.873)(5.5)}{1.0}\right)} & = & \boxed{0.17} \quad \ast \text{ USE} \end{vmatrix}$$

But Not Less Than 0.10

$$V = C_s W \qquad \text{where} \quad W \text{ is shown in Excel}$$
$$= (0.17)(59784^t) \qquad \text{of bldg wt}$$
$$\boxed{= 1020.7}$$

SHOWN IN EXCEL

h_x^k varies per floor   @ level 9: $(107.75)^{1.45} = \boxed{885.13}$

$W_x h_x^k$ varies per floor   @ level 9: $(5165.42)(885.13) = \boxed{4572049}$

$C_{vx} = \dfrac{W_x h_x^k}{\sum W_i h_i^k}$ varies per floor   @ level 9: $\dfrac{4572049}{32973079} = \boxed{0.14}$

$F_x = C_{vx} V$ @ level 9: $(0.14)(1020.7) = \boxed{141.53}$

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		JOB TITLE	Thesis		
Penn State					
State College, PA		JOB NO.	Tech 1	SHEET NO.	
		CALCULATED BY	RGJ	DATE	10/3/09
		CHECKED BY		DATE	

VI. Seismic Loads: ASCE 7- 05

Occupancy Category:	II				
Importance Factor (I) :	1.00				
Site Class :	D				

Ss (0.2 sec) =	15.40 %g							
S1 (1.0 sec) =	5.10 %g							

Fa =	1.600		Sms =	0.246	Sds =	0.164	Design Category =	A
Fv =	2.400		Sm1 =	0.122	Sd1 =	0.082	Design Category =	B

Seismic Design Category =	**B**
Number of Stories:	10
Structure Type:	Not applicable
Horizontal Struct Irregularities:	No plan Irregularity
Vertical Structural Irregularities:	No vertical Irregularity

Flexible Diaphragms:	Yes
Building System:	**Bearing Wall Systems**
Seismic resisting system:	**Ordinary reinforced concrete shear walls**
System Building Height Limit:	**Height not limited**
Actual Building Height (hn) =	153.8 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Factor (R) =	5.5
System Over-Strength Factor (Ωo) =	2
Deflection Amplification Factor (Cd) =	4
Sds =	0.164
Sd1 =	0.082

ρ = redundancy coefficient

Seismic Load Effect (E) = $\rho\, Q_E +/- 0.2 S_{DS} D$ = $\rho\, Q_E$ +/- 0.033D Q_E = horizontal seismic force

Special Seismic Load Effect (E) = $\Omega o\, Q_E +/- 0.2 S_{DS} D$ = 2.0 Q_E +/- 0.033D   D = dead load

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis (Seismic Category A only)   Method Not Permitted

Simplified Analysis   Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis	- Permitted			
Building period coef. (C_T) =	0.020			Cu = 1.70
Approx fundamental period (Ta) =	$C_T h_n{}^x$ =	0.873 sec	x= 0.75	Tmax = CuTa = 1.485
User calculated fundamental period (T) =		0 sec		Use T = 0.873
Long Period Transition Period (TL) =	ASCE7 map =	6		
Seismic response coef. (Cs) =	SdsI/R =	0.030		
need not exceed Cs =	Sd1 I /RT =	0.017		
but not less than Cs =		0.010		
USE Cs =		0.017		
		Design Base Shear V = 0.017W		

Model & Seismic Response Analysis   - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type:	All other structures

Allowable story drift = 0.020hsx   where hsx is the story height below level x

Appendix D: Slab Spot Checks Calculations

9" SLAB
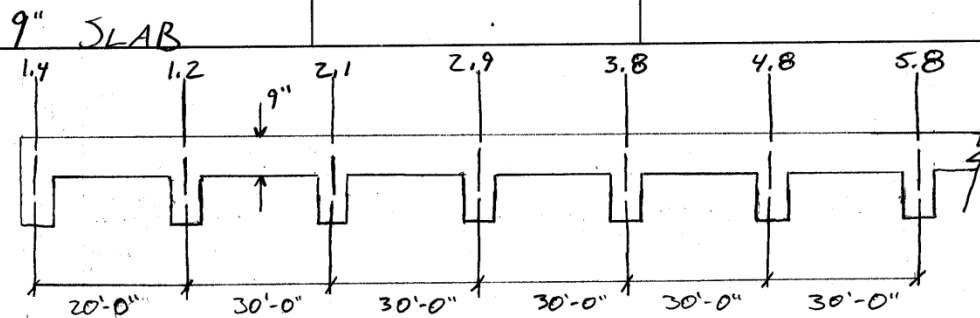
DIRECT DESIGN METHOD

→ Meets all provisions per ACI 318-08 8.3.3 of 1-way slabs

→ $f'_c = 5$ ksi   NWC   w/ $f_y = 60$ ksi

→ #5 @ 10" bottom } perpendicular to PT GIRDERS
   #6 @ 10" top

Slab thickness

• Assuming slab is solid 1-way with both ends continuous
• L = 30'-0" - 6"(for bms) = 24'

$$h_{min} = \frac{\ell}{28} = \frac{24(12)}{28} = 10.28" \Rightarrow \boxed{10.5" > 9" \text{ design}}$$

$$d = 10.5" - 1.0" = 9.5"$$

FACTORED LOADS
$W_L = 100$ psf

$W_D = \left(\frac{10.5}{12}\right)(150 psf) = 131.25$ psf

$W_{SD} = 25$ psf

$$1.2(156.25) + 1.6(100) = \boxed{347.5 \text{ psf} = W_u}$$

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Panel A: End Span w/ discontinuous end integral w/ support

$$M_o^+ = \frac{W_u \ell_n^2}{14} = \frac{(.3475)(20)^2}{14} = 9.91 \text{ ft-k}$$

$$M_{o_\ell}^- = \frac{W_u \ell_n^2}{16} = \frac{(.3475)(20)^2}{16} = 8.675 \text{ ft-k}$$

$$M_{o_\ell}^- = \frac{W_u \ell_n^2}{10} = \frac{(.3475)(20)^2}{10} = 13.88 \text{ ft-k}$$

PANELS B, C, D, E, F, G

$$M_o^+ = \frac{W_u \ell_n^2}{16} = \frac{(.3475)(24)^2}{16} = 12.49 \text{ ft-k}$$

$$M_{o_{\ell,R}}^- = \frac{W_u \ell_n^2}{11} = \frac{(.3475)(24)^2}{11} = 18.17 \text{ ft-k}$$

PANEL H

$$M_o^+ = \frac{W_u \ell_n^2}{14} = \frac{(.3475)(16)^2}{14} = 6.35 \text{ ft-k}$$

$$M_{o_R}^- = \frac{W_u \ell_n^2}{16} = \frac{(.3475)(16)^2}{16} = 5.55 \text{ ft-k}$$

$$M_{o_\ell}^- = \frac{W_u \ell_n^2}{10} = \frac{(.3475)(16)^2}{10} = 8.88 \text{ ft-k}$$

* 18.17^{1k} is used for reinforcement b/c it is the most critical @ M^-
* 12.49^{1k} used for M^+

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REINFORCEMENT

$$\rho_{max} = 0.85 \beta \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} = (.85)(.8)\left(\frac{5}{60}\right)\frac{.003}{.003 + .004} = .0258$$

$$d^2 = \frac{M_u}{\phi \rho f_y b (1 - 0.59 \rho f_y / f'_c)} = \frac{18.17 \times 12}{(.9)(.0258)(60)(12)(1 - (.59)(.0258)(60/5))}$$

$$d = 3.994 \cong 4.00 < 9.5" \quad \therefore \text{ Use } 9.5"$$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{(18.17)(12)}{(.9)(60)(9.5 - 1.0/2)} \doteq 0.489 \text{ in}^2$$

check a: $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(.489)(60)}{(.85)(5)(12)} = .575"$

$$\frac{200(12)(9.5)}{60000} \doteq .38$$

$$A_{s_{min}} = \frac{(18.17)(12)}{(.9)(60)(9.5 - .575/2)} = \boxed{0.438 \text{ in}^2/\text{ft}} \quad \text{or} \quad \frac{3\sqrt{5000}}{60000}(12)(9.5) \doteq .403$$

for which $a = .575 * (.438/.489) \doteq .515$

$$\phi M_n = \phi A_s f_y (d - a/2) = (.9)(.44)(60)(9.5 - .515/2)/12 = \boxed{18.22 > 18.17}$$
✓ ok

Shrinkage control: $A_s = 0.0018 b h = (.0018)(12)(10.5) \doteq .227 \text{ in}^2/\text{ft}$

$$\boxed{\therefore A_{s_{min}} = .438 \text{ in}^2/\text{ft} \Rightarrow \#6 @ 12" \text{ for top reinforcing}}$$

Crack control:
$$S = 15\left(\frac{40000}{f_s}\right) - 2.5 c_c = 15 - (2.5)(.75) = \boxed{13.125" > 12"} \text{ ✓ ok}$$

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REINFORCEMENT (cont)

<u>bottom reinf. check:</u>

using (a) calculated from before

$$A_{s min} = \frac{(12.49)(12)}{(.9)(60)(9.5 - .515/2)} = .300 \frac{in^2}{ft}$$

$$= \frac{200(12)(9.5)}{60000} = .38 \frac{in^2}{ft}$$

$$= \frac{3\sqrt{5000}(12)(9.5)}{60000} = .403 \frac{in^2}{ft}$$

$$\therefore A_{s min} = .403 \frac{in^2}{ft} \Rightarrow \#6 @ 12" \text{ for bottom reinf}$$

$$V_u = 1.15 \left(\frac{3475 \times 24}{2} \right) - 347.5 \left(\frac{9.5}{12} \right) = 4520^{\#}$$

$$V_n = V_c = 2\sqrt{5000}(12)(9.5) = 16122^{\#}$$

$$\phi V_c = (.75)(16122) = 120.92^{\#} >> 4520^{\#} \quad \checkmark \text{ ok}$$

FINAL LAYOUT

$f'_c = 5 ksi$
$f_y = 60 ksi$

#6 @ 12" top   < Structura reinf
#6 @ 12" bottom   > Structura reinf

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10.5 2-Way Slab Check

DIRECT DESIGN METHOD (ACI 318-08)

$h_{min} = \dfrac{l_n}{36}$ for slab w/out int beams, w/ drop panels, ext w/edge bms

$h_{min} = \dfrac{(32.5)(12)}{36} = 10.8'' < 10.5''$ from structure

Factored load $= 1.2(150 \times {}^{10.5}/_{12} +25) + 1.6(100) = 347.5 \cong .35^k$

E/W Panel (int span)

$M_o = \frac{1}{8} w l_2 l_n^2 = \frac{1}{8}(.35)(24)(32.5)^2 = 1110^{'k}$

$M^- = (.65) = -721.5^{'k}$

$M^+ = (.35) = 388.5^{'k}$

$\alpha = I_b/I_s = 1.0$ assumed b/c no int beams

$\dfrac{l_2}{l_1} = \dfrac{24}{32.5} = .74 \qquad \dfrac{\alpha l_2}{l_1} = .74$

Neg Moment

$(.79)(721.5) = -570^{'k}$ to CS

$(.21)(721.5) = -151.5^{'k}$ to MS

Pos Moment

$(.71)(388.5) = 276^{'k}$ to CS

$(.29)(388.5) = 112.5^{'k}$ to MS

	M^-	M^+
tot	-721.5	388.5
Cs	-570	276
MS	-151.5	112.5

M^-	M^+	M^-
-721.5	388.5	-721.5

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Slab reinforcing
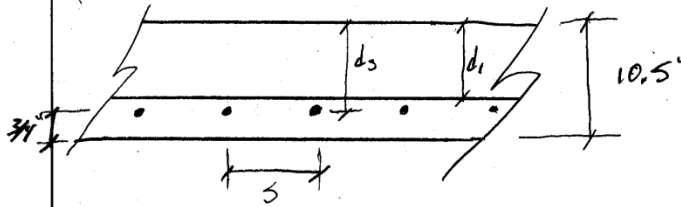
$$S_{max} = 2t = 2(10.5) = 21"$$

#5 $(A = .31 in^2)$ for M^+

$$d_s = 10.5 - 3/4 - (1/2)(.625) = 9.44"$$

$$d_e = 9.44 - .625 = 8.81$$

#6 $(A = .44 in^2)$ for M^-

$$d_s = 10.5 - 3/4 - (1/2)(.75) = 9.375"$$

$$d_e = 9.375 - .75 = 8.63"$$

$$\rho_{max} = .0243 \text{ from table A4 NDS}$$

$$d_{min} = \sqrt{\frac{M_u}{\phi \rho f_y b \left(1 - .59 \rho \frac{f_y}{f_c'}\right)}} \qquad b = 12 \Rightarrow \text{per foot}$$

$$= \sqrt{\frac{633 \times 1000}{(.9)(.0243)(60000)(12)(1 - (.59)(.0243)(60/5))}} = 6.98" < d_{used} \checkmark \text{ ok}$$

@ 330^k $\Rightarrow$ 5.03" $\checkmark$ ok
@ 168 $\Rightarrow$ 3.59" $\checkmark$ ok
@ 125 $\Rightarrow$ 3.10" $\checkmark$ ok

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Col Strip Design

Item #	Description	M^-	M^+
1)	M_n	-570	276
2)	Cs width b	144"	144"
3)	Eff Depth d	8.63	8.81
4)	$M_u = M_n/.9$	-633	307
5)	$M_u * 12/b$	-52.75	25.6
6)	$R = M_u/bd^2$	708.	330
7)	ρ (A-5)	.0130	.0057
8)	$A_s = \rho bd$	16.16	7.23
9)	$A_{smin} = .0018bt$	2.72	2.72
10)	larger 8 or 9/Area	36.73 ⇒ 37	23.3 ⇒ 24
11)	$N_{min} = \frac{slab\ width}{2t}$	6.85 ⇒ 7	7

Middle Strip Design

Item #	Description	M^-	M^+
1)	M_n	-151.5	112.5
2)	b	144"	144"
3)	d	8.63	8.81
4)	$M_u = M_n/.9$	-168	125
5)	$M_u * 12/b$	-14	10.42
6)	$R = M_u/bd^2$	188	134
7)	ρ	.00325	.00225
8)	$A_s = \rho bd$	4.04	2.85
9)	$A_{smin} = .0018bt$	2.72	2.72
10)	8 or 9/area	9.18 ⇒ 10	9.19 ⇒ 10
11)	$N_{min} = \frac{slab\ width}{2t}$	7	7

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N/S Panel (ext Panel)

$$M_o = \tfrac{1}{8}(.35)(32.5)(24)^2 = 820^{1k}$$

$$M_{ext}^- = (.3)(820) = -246^{1k}$$

$$M^+ = (.5)(820) = 410^{1k}$$

$$M_{int}^- = (.7)(820) = -574^{1k}$$

$$\alpha = 1.0 \quad \frac{l_2}{l_1} = \frac{32.5}{24} = 1.35 \qquad \alpha\frac{l_2}{l_1} = 1.35 \qquad \beta_t = 0$$

Ext Neg Moment
$$(100)(-246) = -246^{1k} \text{ to CS}$$
$$(0)(-246) = 0^{1k} \text{ to MS}$$

Pos. Moment
$$(.62)(410) = 254^{1k} \text{ to CS}$$
$$(.38)(410) = 156^{1k} \text{ to MS}$$

Int Neg Moment
$$(.71)(-574) = -408^{1k} \text{ to CS}$$
$$(.29)(-574) = -166^{1k} \text{ to MS}$$

	M_{ext}^-	M^+	M_{int}^-
tot	-246	410	-574
CS	-246	254	-408
MS	0	156	-166

M_{ext}^-	M^+	M_{int}^-
-246	410	-574

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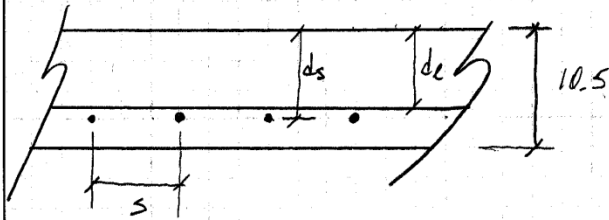
$$S_{max} = 2t = 2(10.5) = 21"$$

#5 $(A = .31 in^2)$ for M^+

$d_s = 9.44$

$d_e = 8.81$

#6 $(A = .44 in^2)$ for M^-

$d_s = 9.375$

$d_e = 8.63$

$\rho_{max} = .0243$

$$d_{min} = \sqrt{\frac{M_u}{\phi \, \rho \, f_y \, b \, (1 - .59 \rho \, f_y/f'_c)}}$$

$$= \sqrt{\frac{273 \times 1000}{(.9)(.0243)(12)(1 - .59)(.0243)(60/5)}} = 4.57" < d_{used} \checkmark ok$$

@ 282 = 4.65" $\checkmark$ ok

@ 453 = 5.89" $\checkmark$ ok

@ 173 = 3.64" $\checkmark$ ok

@ 184 = 3.76" $\checkmark$ ok

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Col Strip Design

Item #	Description	M_{ext}^-	M^+	M_{int}^-
1)	M_n	-246	254	-408
2)	b	195	195	195
3)	d	8.63	8.81	8.63
4)	$M_u = M_n / .9$	-273	282	-453
5)	$M_u * 12/b$	-16.8	17.35	-27.88
6)	$R = M/bd^2$	-225	224	-374
7)	ρ	.0037	.0037	.0065
8)	$As = \rho bd$	6.23	6.36	10.94
9)	$As_{min} = .0018 bt$	3.68	3.68	3.68
10)	larger 8 or /area	14.16 ⇒ 15	20.52 ⇒ 21	24.86 ⇒ 25
11)	$N_{min} = \frac{slab\ width}{2t}$	10	10	10

Middle Strip Design

Item #	Description	M_{ext}^-	M^+	M_{int}^-
1)	M_n	0	156	-166
2)	b	195	195	195
3)	d	8.63	8.81	8.63
4)	$M_u = M_n / .9$	0	173	-184
5)	$M_u * 12/b$	0	10.65	-11.32
6)	$R = M/bd^2$	0	137	-137
7)	ρ	0	.0024	.0024
8)	$As = \rho bd$	0	4.12	4.03
9)	$As_{min} = .0018 bt$	3.68	3.68	3.68
10)	larger 8 or /area	8.36 ⇒ 9	13.3 ⇒ 14	9.16 ⇒ 10
11)	$N_{min} = \frac{slab\ width}{2th}$	9.3 ⇒ 10	10	10

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COMPARISON

Designer

E/W: $\dfrac{CS}{M^-} = 37 \, \#6$ $22 \, \#6$

$M^+ = 24 \, \#5$ NA

$\dfrac{MS}{M^-} = 10 \, \#6$ $18 \, \#6$

$M^+ = 10 \, \#5$ NA

N/S: $\dfrac{CS}{M^-_{ext}} = 15 \, \#6$ NA

$M^+ = 21 \, \#5$ $\#5 @ 8 \cong 18 \, \#5$

$M^-_{int} = 25 \, \#6$ $24 \, \#6$

$\dfrac{MS}{M^-_{ext}} = 10 \, \#6$ NA

$M^+ = 14 \, \#5$ $\#5 @ 8 \cong 18 \, \#5$

$M^-_{int} = 10 \, \#6$ $14 \, \#6$

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Appendix E: Beam Spot Check Calculations

1st Span Analysis of PT

½" ⌀ w/ A = .153 in² 270 ksi strands

As Designed

θ = tan^{-1}($\frac{4(11.5/12)}{44}$) = 5°

Preliminary thickness

h = L/30 = (44)(12)/30 = 17.6" ≯18" = 18" as designed ✓ ok

Loading

DL = 5.8 klf   from col loading plan

A = bh = (72)(18) = 1296 in²

S = $\frac{bh^2}{6}$ = $\frac{(72)(18)^2}{6}$ = 3888 in³

Targeted load balancing (75%) of load on beam

(.75)(966.7 psf) = 725 psf = 4350 plf = 4.35 k/ft

Tendon Ordinate   **Tendon CG Location** (Typ)
anchor ext   4.0"
top int support   7.0"
bot int span   1.0"
end span bottom   1.75"

α int = 18" − 1" = 17"

α end = $\frac{4+18}{2}$ − 1.75 = 9.25"

P = $\frac{w_{bal}L^2}{8}$ = $\frac{(4.35)(44-\frac{30}{12})^2}{8(\frac{11.5}{12})}$ 977 k

Assumed 15 ksi losses in strand

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$$f_{se} = .75 f_{pu} = (.75)(270000) - 15000 = 187.5 \, ksi$$

$$P_{eff} = f_{se} A = (187.5 \, ksi)(.153 \, in^2) = 28.69 \, k$$

$$\# \, tendons = \frac{977^k}{28.69^k/tendon} \cong 34 \, tendons$$

$$P_{actual} = (34)(28.69) = 976^k \cong 991^k \text{ as designed} \checkmark \, ok$$

$\underline{Balanced \ load \ adjustment}$
$$\frac{976}{975} (4.35) = 4.35 \, k/ft$$

$$\frac{P_{actual}}{A} = \frac{976^k}{1296 \, in^2} = 753 \, psi > 125 \, psi$$

However much greater than usually
seen of 200–400

$\underline{Int \ Span \ Force}$
$$P = \frac{(4.35)(30)^2}{(8)(7/12)} = 838^k < 976^k \therefore Less \ force \ in \ int$$

2

Appendix F: Column Spot Check Calculations

COLUMN DESIGN

SI.12H OF COL @ 247 ϕ & 2.1
COL # 247

$f_y = 60$ ksi
(12) bars
4 EF

1.5"max
1.5"
30"
30"
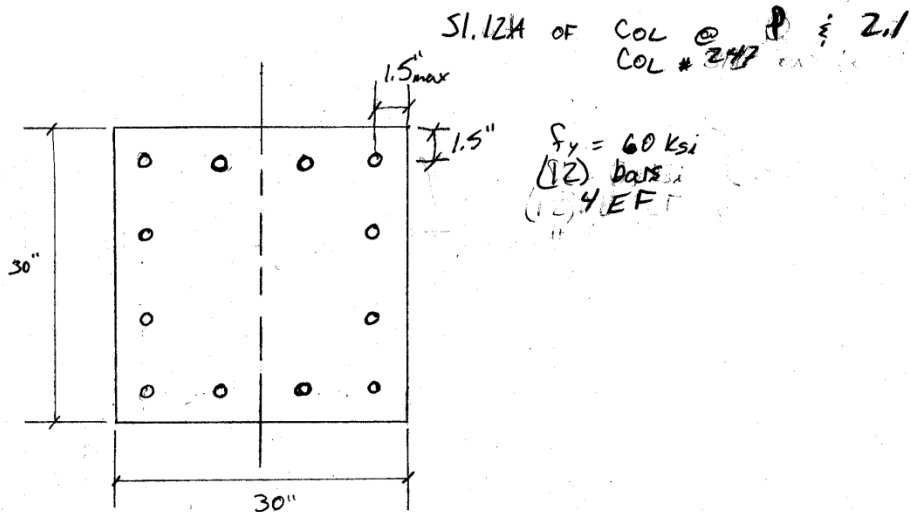
Pure Axial (Level 1-2)

$$P_n = 0.85 f'_c A_c + A_s f_y$$

$$= (0.85)(8)(900 - (12 \times 1.27)) + (12 \times 1.27)(60)$$

$$= 6930.77$$

$$\phi P_n = 0.8(0.65)(6930.77) = \boxed{3604^k}$$

Pure Axial (Level 3)

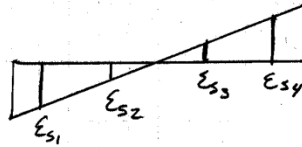$$\phi P_n = (.8)(.65)(.85)(8)[900 - (12 \times 1.00)] + (12 \times 1.00)(60)$$

$$\boxed{= 3860^k}$$

Pure Axial (Level 4)

$$\phi P_n = (.8)(.65)(.85)(6)[900 - (12 \times 1.00)] + (12 \times 1.00)(60)$$

$$\boxed{= 3261^k}$$

Pure Axial (Level 5-10)

$$\phi P_n = (.8)(.65)(.85)(4)[900 - (12 \times .79)] + (12 \times .79)(60)$$

$$\boxed{= 2143.^k}$$

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<u>Balanced strain-Strength @ level 9</u>
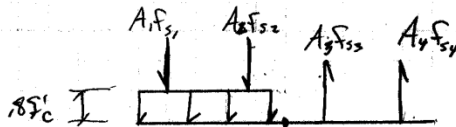
#3 ties

$\varepsilon_c = \varepsilon_u$

$\varepsilon_y = \frac{60}{29000} = .00207$

$d = 30 - 1.5 = 28.5$

$C = \frac{.003}{.003 + \varepsilon_y} d = \frac{.003}{.003 + .00207}(28.5) = 16.86$

$\beta_1 = .85$

$a = \beta_1 C = (.85)(16.86) = 14.33$

$A_1 f_{s_1} \quad A_s f_{s_2} \quad A_3 f_{s_3} \quad A_4 f_{s_4}$

$.85 f_c' \quad \bar{I}$

$\varepsilon_{s_1} = \frac{(.003)(c - d_1)}{c} = \frac{(.003)(16.86 - 1.5)}{16.86} = .00273 > \varepsilon_y$

$f_{s_1} = .00273(29000) = 79.26 \text{ ksi} \rightarrow \boxed{60 \text{ ksi}} \text{ compr}$

$\varepsilon_{s_2} = \frac{(.003)(c - d_2)}{c} = \frac{(.003)(16.86 - 10.5)}{16.86} = .00113 < \varepsilon_y$

$f_{s_2} = (.00113)(29000) = \boxed{32.82 \text{ ksi}} \text{ compr}$

$\varepsilon_{s_3} = \frac{(.003)(c - d_3)}{c} = \frac{(.003)(16.86 - 19.5)}{16.86} = -.00047 < \varepsilon_y$

$f_{s_3} = (.00047)(29000) = \boxed{-13.62 \text{ ksi}} \text{ tension}$

$\varepsilon_{s_4} = \frac{(.003)(c - d_4)}{c} = \frac{(.003)(16.86 - 28.5)}{16.86} = -.00207$

$f_{s_4} = (.00207)(29000) = \boxed{-60 \text{ ksi}} \text{ tension}$

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$$P_b = .85 f'_c \, b \beta_1 c + 4 f_{s_1} + 2 f_{s_2} + 2 f_{s_3} + 4 f_{s_4}$$

$$= (.85)(4 \, ksi)(30)(.85)(16.86)(4)(60) + (2)(32.82) + (2)(-13.62) + (4)(-60)$$

$$= 1461.8 + 240 + 65.64 - 27.24 - 240$$

$$\boxed{= 1500^k}$$

$$M_b = .85 f'_c \, b \beta_1 c \left(\frac{h}{2} - \frac{\beta_1 c}{2} \right)$$

$$= (.85)(4)(30)(.85)(16.86)\left(\frac{30}{2} - \frac{(.85)(16.86)}{2} \right)$$

$$+ (4)(60)(15-1.5) + (2)(32.82)(15-10.5) + (2)(-13.62)(15-19.5) + (4)(60)(15-26.5)$$

954

$$= 954 + 270 + 24.6 + 10.8 + 270$$

$$\phi P_b = (.65)(1500) = \boxed{975 \quad ^k}$$

$$\phi M_b = (.65)(1528) = \boxed{994 \, '^k}$$

PURE BENDING

assume ε_{s_1} & ε_{s_2} do not yield
ε_{s_3} & ε_{s_4} yield

$$f_{s_1} = \frac{.003}{c}(c-1.5)(29000) = \frac{87}{c}(c-1.5)$$

$$f_{s_2} = \frac{.003}{c}(c-10.5)(29000) \quad \frac{87}{c}(c-10.5)$$

$$f_{s_3} = -60$$

$$f_{s_4} = -60$$

$$\Sigma F = 0 = (.85)(4)(30)(.85)c + (4)(f_{s_1}) + (2)(f_{s_2}) + (2)(f_{s_3}) + (4)(f_{s_4})$$

$$= 86.7c + \frac{348}{c}(c-1.5) + 174\frac{(c-10.5)}{c} + (-120) + (-240)$$

$$c = 4.35"$$

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<u>Verify assumption</u>

$$f_{s_1} = \frac{87}{c}(c-1.5) = \frac{87}{4.35}(4.35-1.5) = 57.72$$

$$f_{s_2} = \frac{87}{4.35}(4.35-10.5) = -123 \not> 60 \quad \times \; NG$$

Assume ε_{s_3} does not yield

$$0 = 86.7c + \frac{348}{c}(c-1.5) -120 -120 -240$$

$$c = 3.33$$

$f_{s_1} = 47.81 \quad (c)$

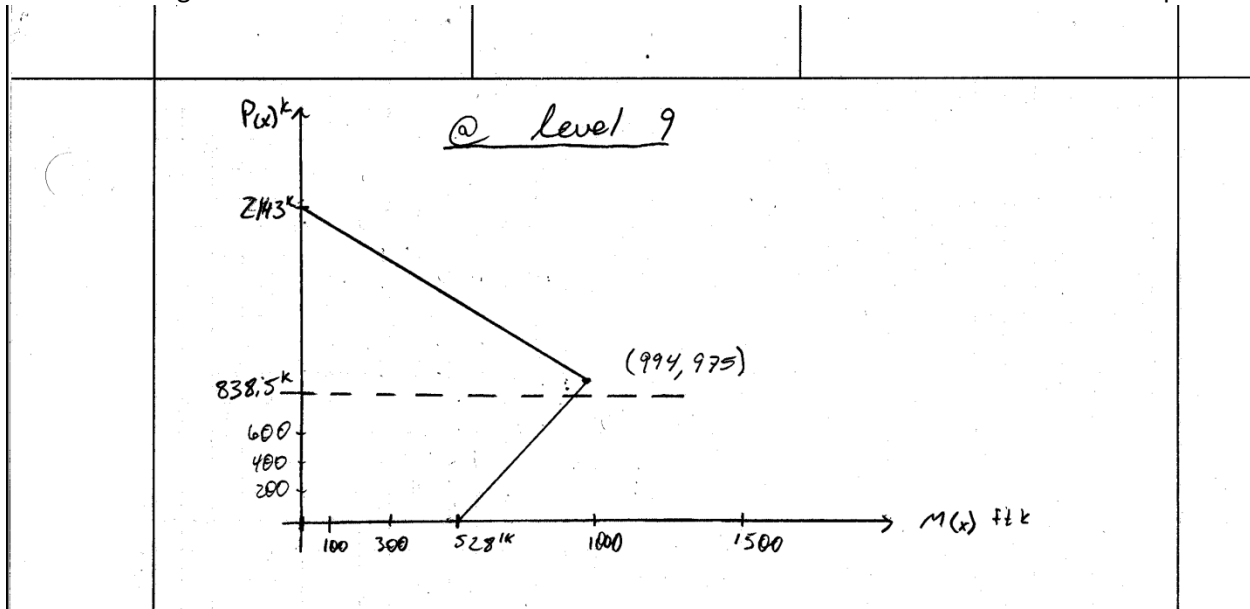$f_{s_2} = -60 \quad (T)$

$f_{s_3} = -60 \quad (T)$

$f_{s_4} = -60 \quad (T)$

$$M_o = (.85)(4)(30)(.85)(3.33)\left[15 - \frac{.85(3.33)}{2}\right]$$

$$+ 4(47.81)(15-1.5) + 2(-60)(15-10.5) + 2(-60)(15-19.5)$$

$$+ (4)(-60)(15-28.5) =$$

$$M_o = 3922.1 + 2581.3 + 3240 = 9744 \; {}^{kin} = 812^{'k}$$

$$\phi M_o = (0.65)(812) = \boxed{528^{'k}}$$

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Floor	Tributary Width (ft)	Dead Load (plf)	Live Load (plf)	Superimposed Dead Load (plf)	Total Dead Load (plf)	Snow Load (plf)	1.4DL	1.2DL + 1.6LL	1.2DL + 1.6S + L (kips)	Total Weight (kips)
Roof	30	4500	3000	750	5250	750	238.9	360.8	341.3	204.8
10	30	3375	3000	750	4125	0	187.7	316.9	258.4	521.6
9	30	3375	3000	750	4125	0	187.7	316.9	258.4	838.5
8	30	3375	3000	750	4125	0	187.7	316.9	258.4	1155.4
7	30	3375	3000	750	4125	0	187.7	316.9	258.4	1472.3
6	30	3375	3000	750	4125	0	187.7	316.9	258.4	1789.1
5	30	3375	3000	750	4125	0	187.7	316.9	258.4	2106.0
4	30	3375	3000	750	4125	0	187.7	316.9	258.4	2422.9
3	30	3375	3000	750	4125	0	187.7	316.9	258.4	2739.8
2	30	3375	3000	750	4125	0	187.7	316.9	258.4	3056.6
1	30	3375	3000	750	4125	0	187.7	316.9	258.4	3373.5

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Appendix G: Floor Plans
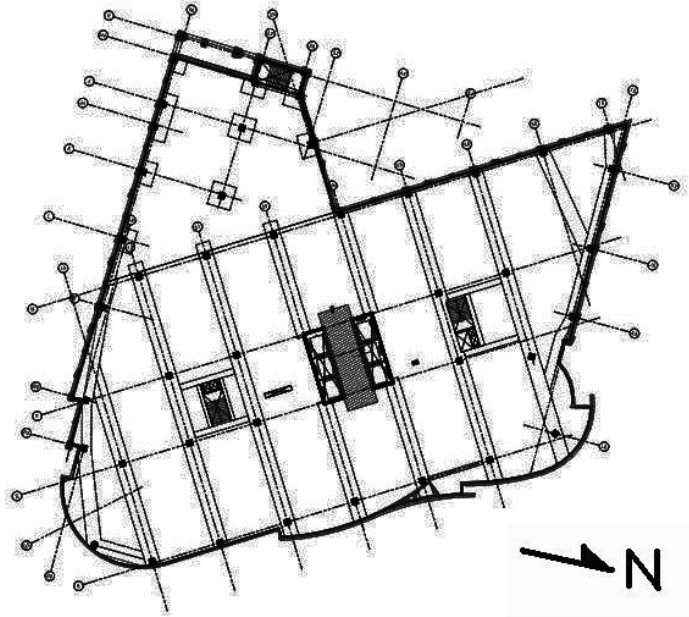
Figure 20: Typical Level 1-3 Plan
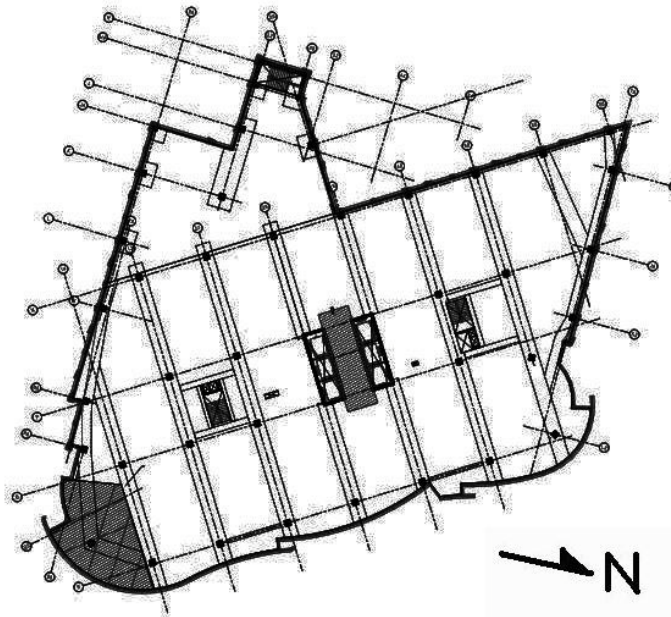
Figure 21: Typical Level 4-6 Plan

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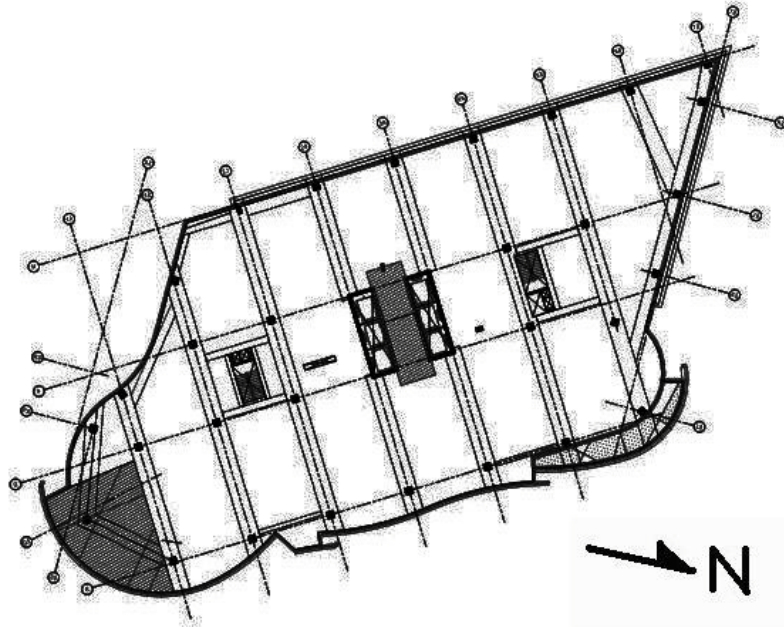
Figure 22: Typical level 7-10 Plan

Figure 23: Roof Plan

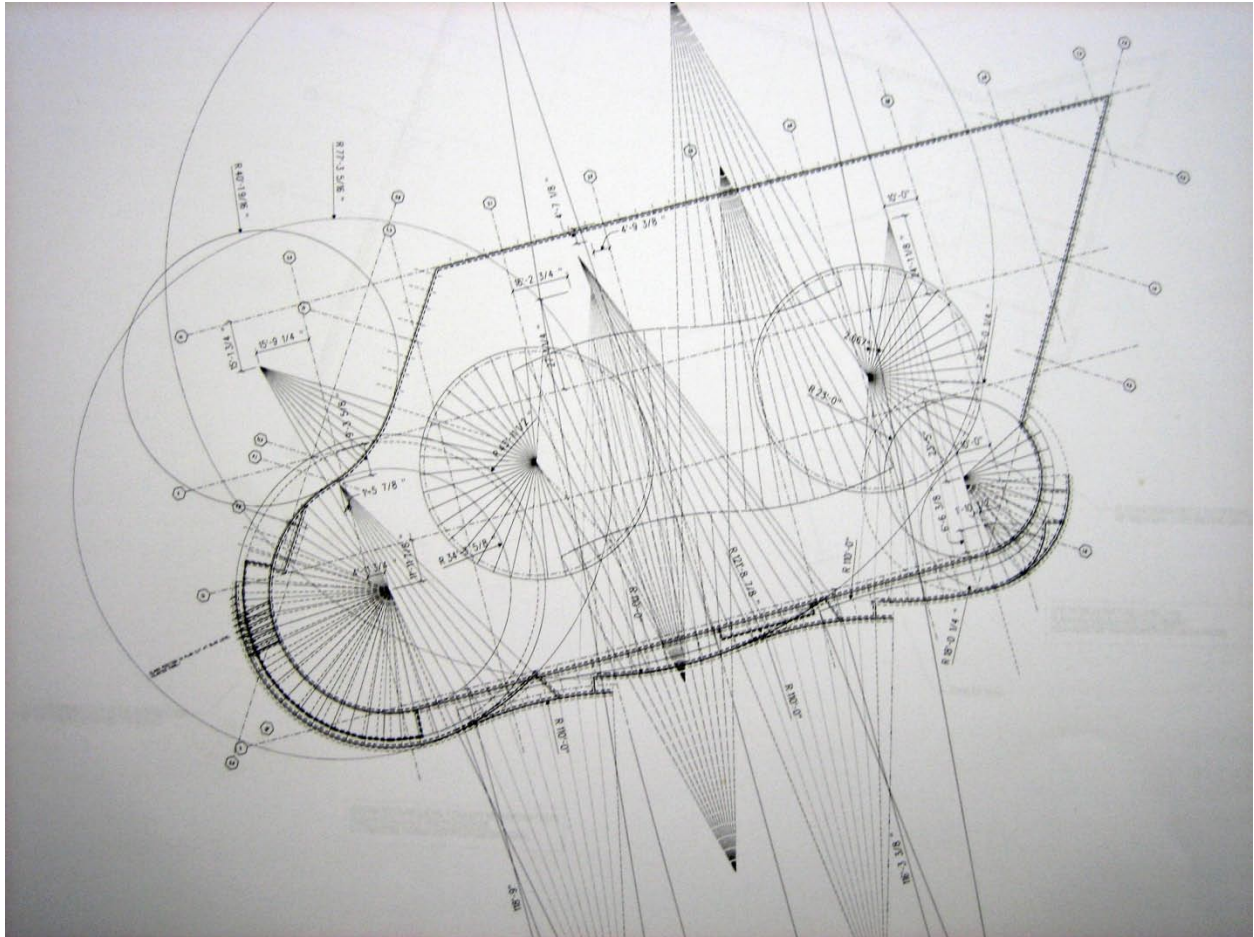Ryan Johnson
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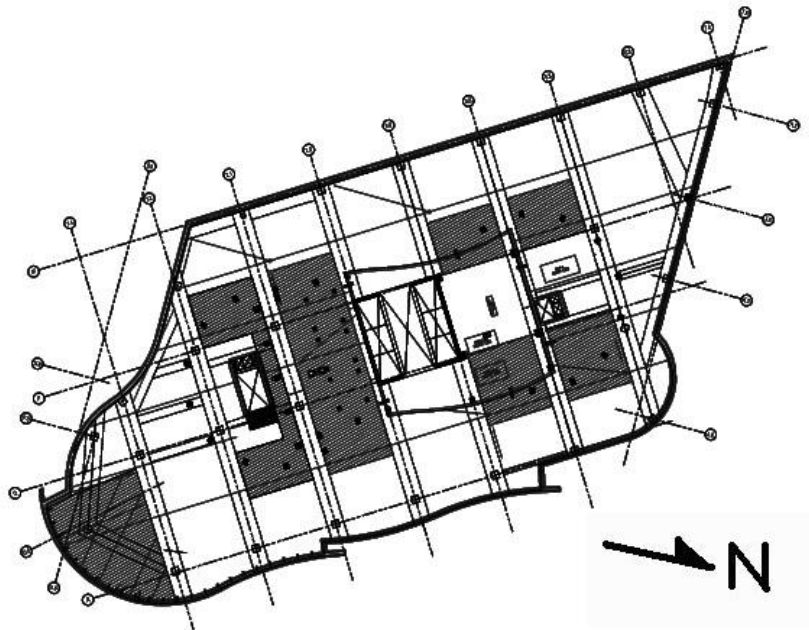
Figure 24: Building Geometry Plan