Technical Report 1

Rendering provided by DCS Design

Kingstowne Section 36A
5680 King Center Drive
Kingstowne, VA 22315

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Structural Option

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EXECUTIVE SUMMARY

The purpose of Technical Report 1 is to describe and analyze the physical existing conditions of Kingstowne Section 36A through information relative to design concepts and required loading. An extensive structural overview and detailed load calculations are included in the scope of this report. Another goal is to compare values calculated within this report to ones used by the engineer of record in the design of this building.

The report will start out with an overview of the building itself and detailed descriptions of the structural systems to familiarize the reader with Kingstowne Section 36A. For this, the building is broken up into foundations, garage floors, office floors, and roof system. Floor systems, framing systems, and lateral systems will be described for both the garage and office floors as the building is split into two completely different structural systems, concrete for the garages and steel for the office levels. Summaries of codes used for design of this building and for the purposes of this report are included next, followed by a summary of the materials used to construct the structure, all of which are included to further clarify the structure of the building.

Gravity loads are then calculated for comparison to the loads used in the design of the building. Through assumptions and estimations used in this report, the building self-weight is only 1.34% above the value used in the design of the building. This shows that including the concrete shear walls, concrete drop panels, and concrete columns in the overall self-weight of the floor slab was a valid assumption.

Lateral load analysis on the building consists of analyzing wind, seismic, and soil loads. Wind loading is the controlling factor over seismic, 828 kips to 671 kips. This controlling base shear value was calculated using a north – south wind direction with the wind blowing to the south. The calculated seismic base shear of 671 k was higher than the value used in design, 595 k, by about 11%. This is a result of being limited on the accuracy of reading the ground motion charts in ASCE 7-10.

BUILDING INTRODUCTION

Kingstowne Section 36A (KT36A) is a 200,000 ft^2, 8 story office building to be located in Fairfax County Virginia. It will contain 4 levels of concrete structure parking garage and 4 levels of composite steel construction office space. Floor space has also been allocated for about 5,000 square feet of retail area on the ground floor (Parking Level 1). KT36A will be 86'-11" in height when measured from the average grade. The reason the building height is measured from average grade is because there is a significant grade elevation change from the south side of the building to the north side, on the order of 26'-8" (See Figure 1). This poses unique challenges in the structural design of the building since the geotechnical report states the soil placing a load of 60psf/ft in depth below grade surface on the structure. This means that there is more than 1600 psf of soil load on the foundation walls at the lowest slab levels. This load alone had enough impact on the building that six 12" thick shear walls had to be constructed at parking level 1 to transfer the loads safely.

When completed, KT36A will be part of a master planned development for retail and office space owned by the Halle Companies. Being a part of a master planned development, the building was designed to match the appearance of the surrounding buildings. This appearance can be characterized by a rectilinear footprint, pink velour brick, aluminum storefront with glass of blue/black appearance, and precast concrete bands around the circumference of the building.

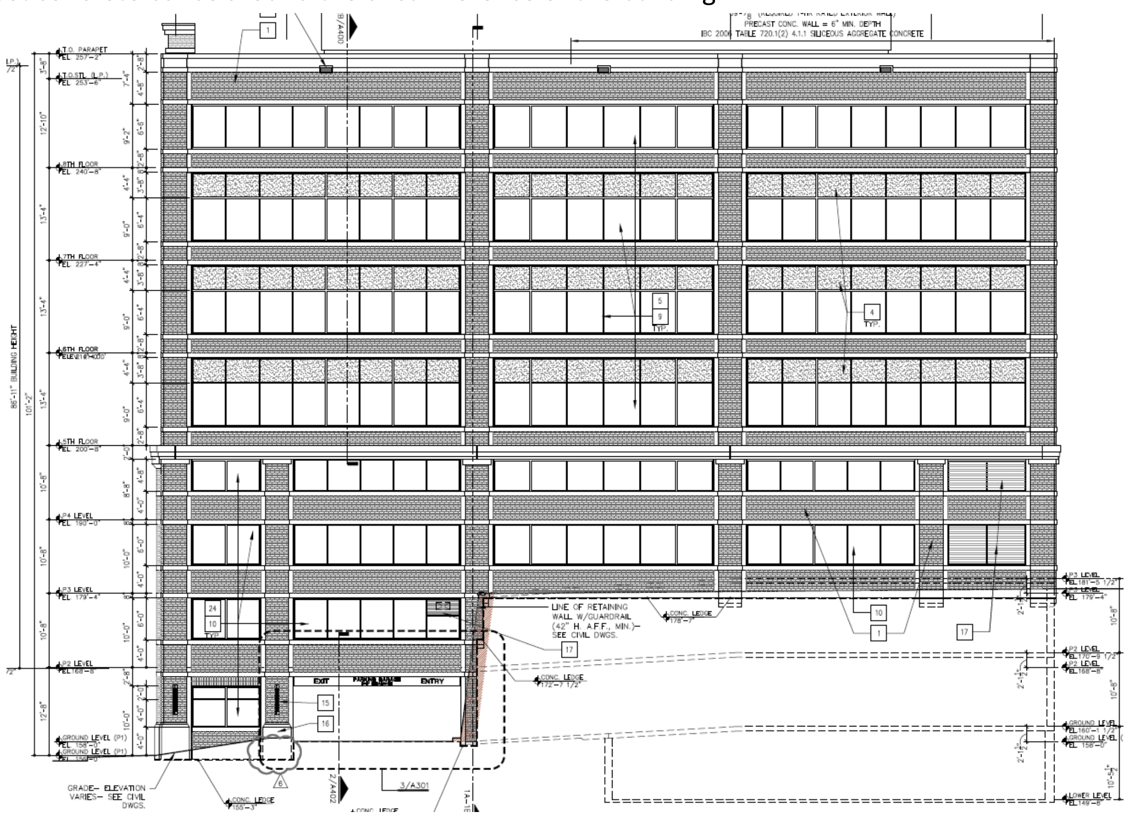


Figure 1: Elevation Looking East Showing Grade Differences (Source: DCS Design Drawing A-301)

STRUCTURAL OVERVIEW

Kingstowne Section 36A consists of two different primary structural systems; cast-in-place concrete for the lowest four floors of the building and a composite steel system for the remaining four floors. The concrete floors are used for the parking garage and retail space while the steel system is used at the office occupancy levels. Lateral forces in the concrete levels are resisted with 12" thick concrete shear walls of varying height. When KT36A transitions to steel construction, lateral forces are transferred to the concrete columns and shear walls through concentrically braced frames, eccentrically braced frames, and rigid moment frames. Per sheet S-001, components such as steel stairs and curtain wall/window systems were not included in the scope for the structural design of this building.

FOUNDATIONS

In their report submitted August of 2009, Burgess & Niple, Inc. (B&N) advised that shallow foundations not be used on this project due to settlement concerns based on subsurface conditions. They performed five new soil test borings, ranging from 45 to 100 feet in depth below the grade surface. In addition, they reviewed 14 borings from previous investigations, ranging in depth from 10 to 55 feet below grade surface.

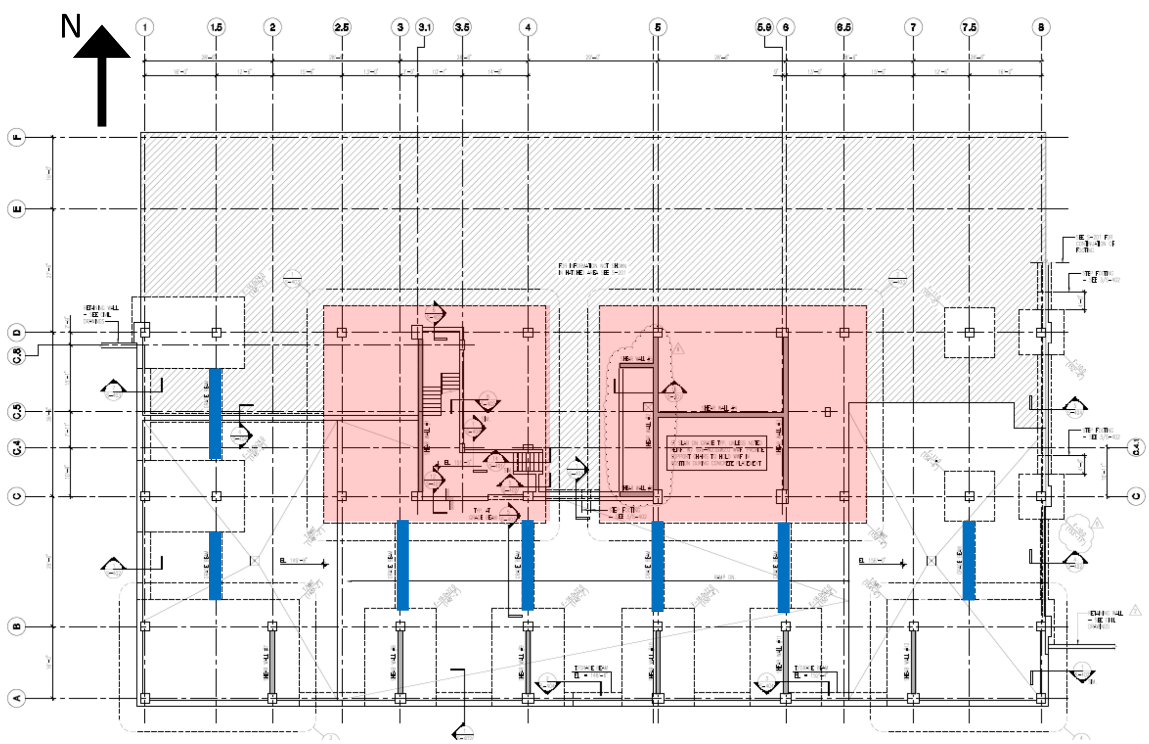


Figure 2: Foundation Plan (Level P0) Showing 48" Thick Mat Foundations Shaded in Red (Source: Cagley & Assoc. Drawing S-200)

Each of the borings consisted of lean clay and fat clay fills with varying amounts of sand, residual soils consisting of lean to fat clay, and clayey to silty sands. Based on the fill materials being encountered between 4 and 48 feet below grade, B&N offered two foundation options. An intermediate foundation system consisting of spread and strip footings bearing on rammed aggregate piers (Geopiers) was chosen for KT36A over the alternate option of a deep system consisting of spread and strip footings bearing on caissons. Geopier diameters typically range from 24 to 36 inches and are compacted using a special high-energy impact hammer with a 45-degree beveled tamper. Per B&N report, footings supported by Geopier elements can be designed using a maximum bearing pressure of 7,000 psf.

Using the information provided by B&N, Cagley & Associates designed spread footings ranging from 27" to 44" in depth to support the columns of KT36A. 48" thick mat foundations are located at the central core of the building to transfer forces in the main shear walls to the soil (See Figure 2). Grade beams (Blue lines in Figure 2) of 30" depth are used throughout level P0 to also transfer forces from the shear walls to the column footings. Foundation walls are supported by continuous wall footings designed for an allowable bearing pressure of 2,500 psf. All foundations are to bear a minimum of 30" below grade unless stated otherwise.

Garage Levels

FLOOR SYSTEM

As previously mentioned, KT36A utilizes cast-in-place concrete for the support structure in the garage. With the exception of the 5" thick slab on grade, this system consists of 8" thick two-way, flat plate construction with drop panels that project 8" below the bottom of structural slab. The drop panels are continuous between grid lines C and D to help the slab span the increased distance of 36'-6" in this bay, otherwise, they are typically 10'-0" x 10'-0" in size. Due to the need for vehicles to circulate vertically throughout the parking garage levels, the floor is sloped on 3 sides of the central core to achieve this.

Since a two-way, flat plate concrete floor system sees both positive and negative moments, reinforcing steel is required in the top and bottom of the slab. The typical bottom mat of reinforcement in KT36A consists of #4 bars spaced at 12" on center in each direction of the slab. Additional bottom reinforcement is also noted on the drawings. Top reinforcement is comprised of both #5 and #6 bars, both oriented in the same fashion as the bottom mat, with the #6 bars typically being used around the columns to resist the larger negative moments present there (see Figure 3 for a typical bay layout). A typical bay size for the concrete levels is 28'-6" x 29'-0".

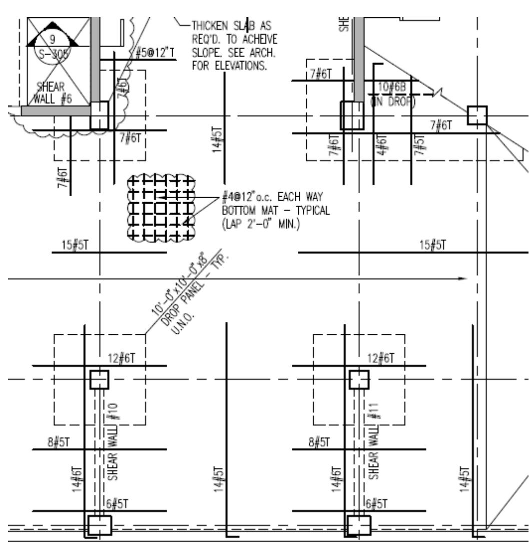


Figure 3: Partial Plan Level P1 (Source: Cagley & Assoc. Drawing S-201)

FRAMING SYSTEM

Supporting the floor slabs are cast-in-place concrete columns constructed of 5000 psi concrete. The most common column size is 24” x 24” reinforced with a varying number of #8 bars and either #3 or #4 stirrups. Columns of this size primarily account for the gravity resisting system of KT36A. The largest columns used are 36” x 30” reinforced with a varying number of #11 bars and #4 stirrups. The larger columns are located at the ends of the large shear walls in the central core of the building. A small number of concrete beams are also present in the project, typically at areas of the perimeter where additional façade support was needed and at large protrusions in the floor slab.

LATERAL SYSTEM

Cast-in-place concrete shear walls resist the lateral forces present in the parking garage levels of KT36A. All of the twelve walls present in the building are 12” thick and cast using 5000 psi concrete. Six of the shear walls (#1 - #6, see Red lines in Figure 4) extend 4-5 stories from the 48” thick mat foundations to office level 1 which is also the 5th elevated floor of the building. Three of the six walls are oriented to resist lateral forces in one direction while the other three walls are oriented in an orthogonal direction. The remaining six walls (#7 - #12, Green lines in Figure 4) are only one story tall and are oriented to best resist the unique lateral soil load placed on KT36A. This load condition is further detailed in the lateral loads section of this report and will be further analyzed in Technical Report 3.

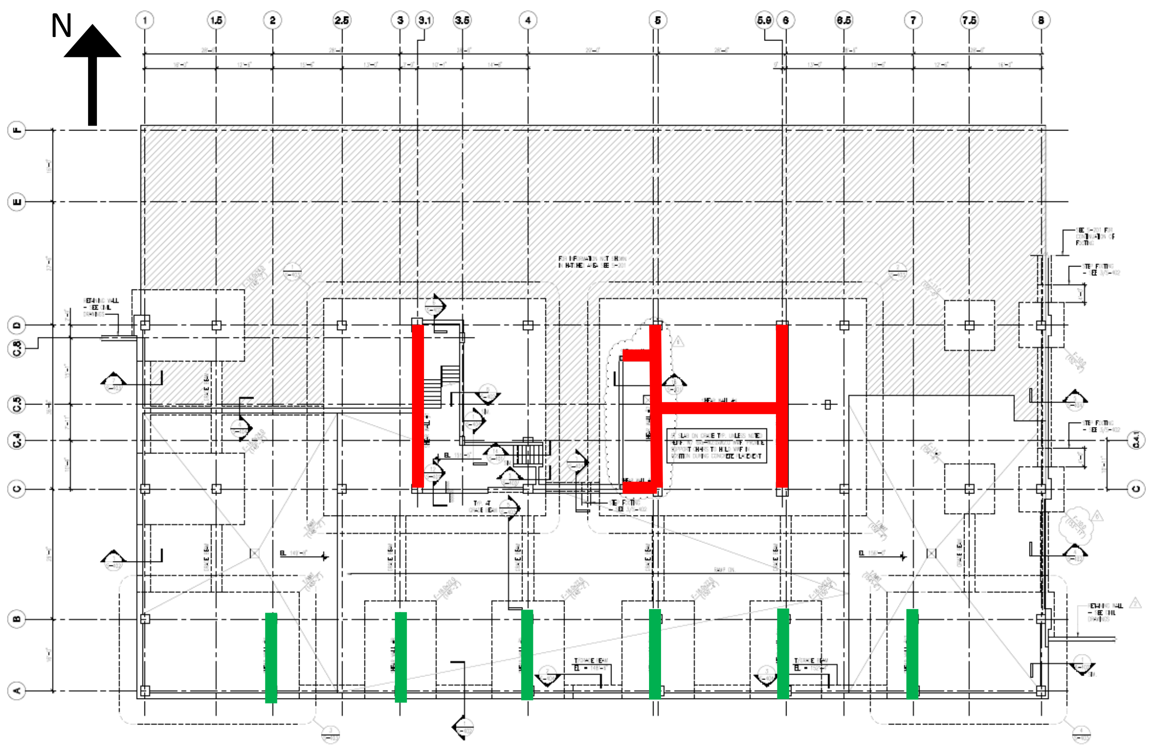


Figure 4: Foundation Plan (Level P0) Showing Shear Walls (Source: Cagley & Assoc. Drawing S-200)

Office Levels

FLOOR SYSTEM

Office level 1 is constructed of the same cast-in-place style of construction as the garage floors below it with the exception of the top of slab elevation being uniform throughout the floor. The remaining floors are constructed using a composite steel system. This system is comprised of 3 ¼" thick concrete on 2" x 18 gage galvanized composite steel decking. The 3000 psi lightweight concrete (115 pcf) coupled with the decking yields a total slab thickness of 5 ¼". Reinforcement for the slab is provided by 6x6-W2.1xW2.1 welded wire fabric.

According to sheet S-001, all decking should meet the three span continuous condition. The decking typically spans 9'-6" perpendicular to cambered beams of varying size. Shear studs of ¾" diameter placed along the length of the beams make this a composite system capable of more efficiently carrying the loads when compared to a non-composite system. The studs must be minimum length of 3 ½" but no longer than 4 ½" to meet designer and code requirements.

FRAMING SYSTEM

The composite floor system mentioned above is supported by structural steel framing comprised of primarily wide flange shapes. W21's and W18's account for most of the beams while the columns range in size from W12x40 to W14x109. A majority of the beams in KT36A are cambered between ¾" and 1 ¼", a function of the span and load demand on the beams. With the exception of four W30x99 sections cambered 1", most of the girders fall within the same size range as the beams. These girders are also not cambered as they typically have much shorter spans than the beams. The columns are all spliced just above the 7th floor (office level 3) where they are reduced in size to more economically carry the lighter axial loads. See Figure 5 below for a typical office floor level layout.

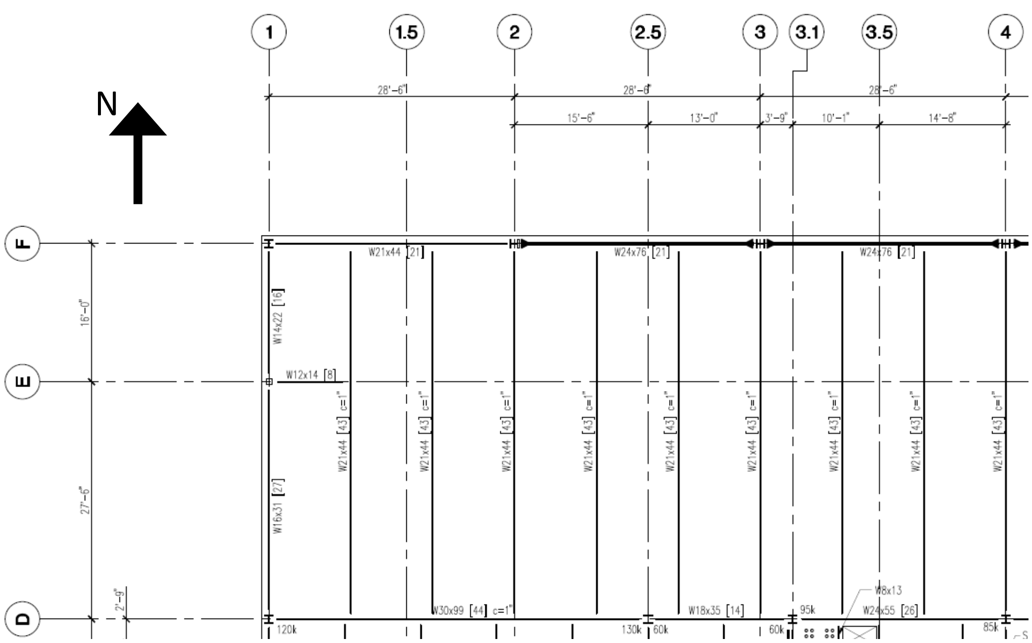


Figure 5: Typical Composite Slab Partial Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

LATERAL SYSTEM

Lateral forces at the office levels are transferred to the concrete shear walls through three different frame systems. Concentrically braced (Green Line) and eccentrically braced frames (Purple Lines) work in the north – south direction while ordinary steel moment frames (Orange Lines) resist the loads in the east – west direction. See Figure 6 for their location and orientation within the building. The eccentrically braced frames were necessary to maintain enough clearance for a corridor in that area of the building. Diagonal bracing for the frames consists of either HSS10x10 or HSS9x9 of varying thickness. Moment frames were most likely chosen for the east – west direction so as not to obstruct the occupants view to the exterior.

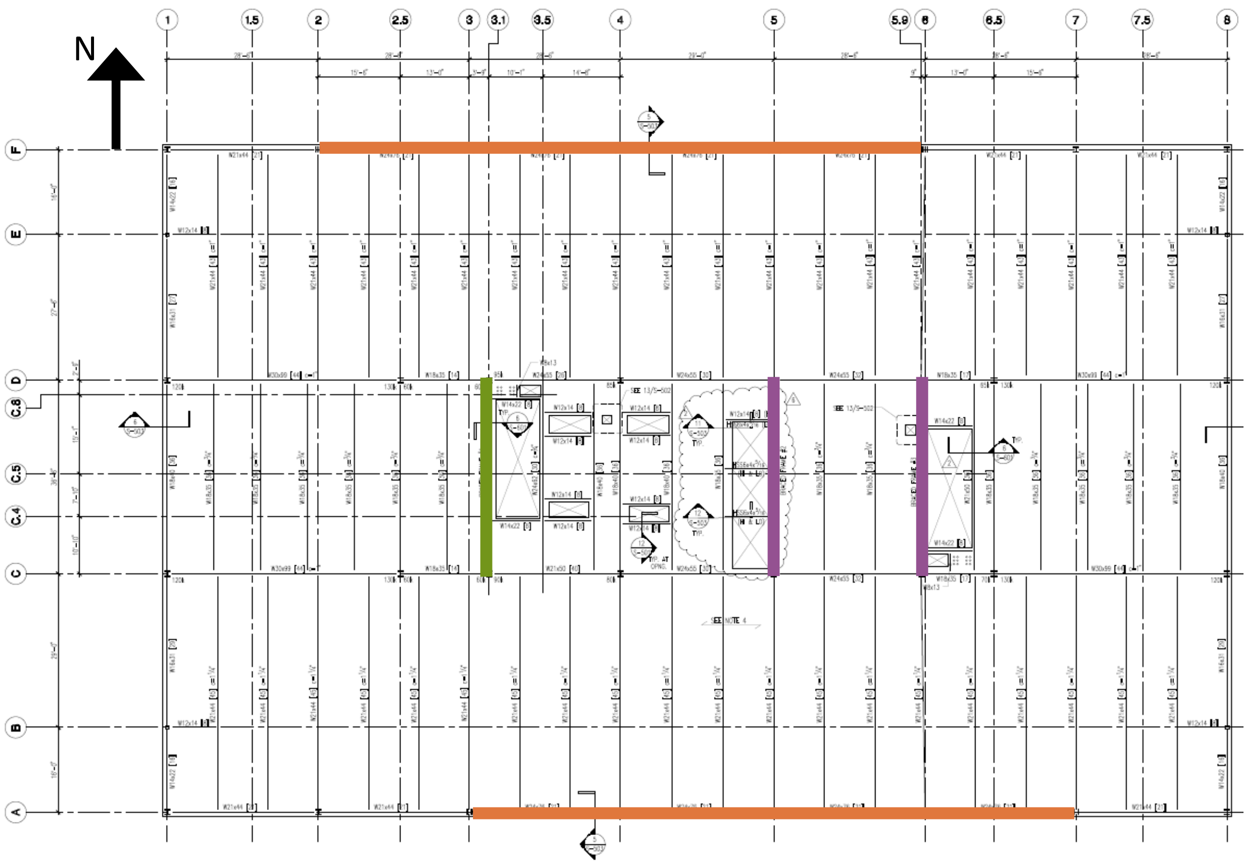
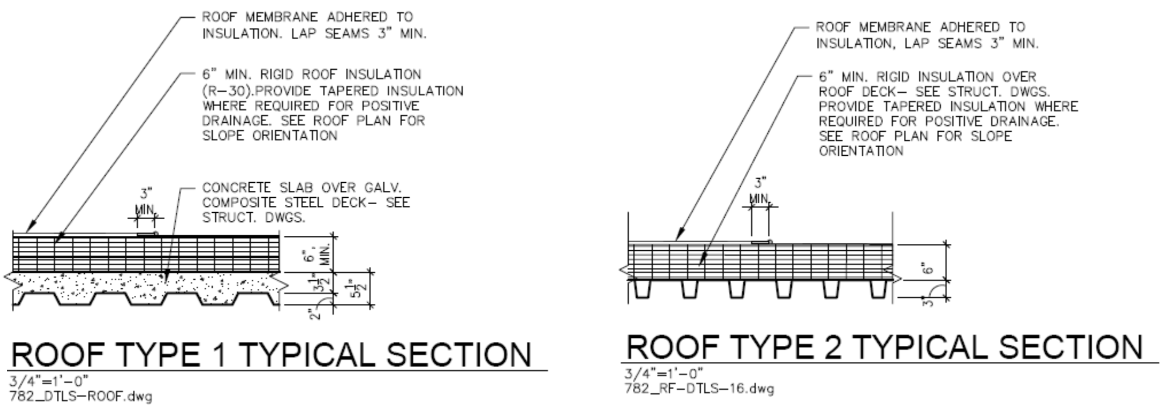


Figure 6: Typical Composite Slab Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

Roof System

The roofing system consists of a white EPDM membrane fully adhered over 6" minimum of R-30 continuous rigid roof insulation. The seams of the membrane must be lapped a minimum of 3" to ensure a watertight seal. Where mechanical equipment is located (see Figure 9), the roofing materials are supported by 2"x 18GA galvanized composite steel deck with a 3.25" thick light-weight concrete topping. The load carrying capacity that this type offers is required to support the four 17,000lb roof top mechanical units needed to condition the air for the building occupants. In all other areas of the roof, the system is supported by 3"x 20GA type N roof deck. Each of the roof types are supported by steel W-shapes that are sloped to achieve proper drainage.



Figures 7 and 8: Typical Roofing Details (Source: DCS Design Drawing A-410)

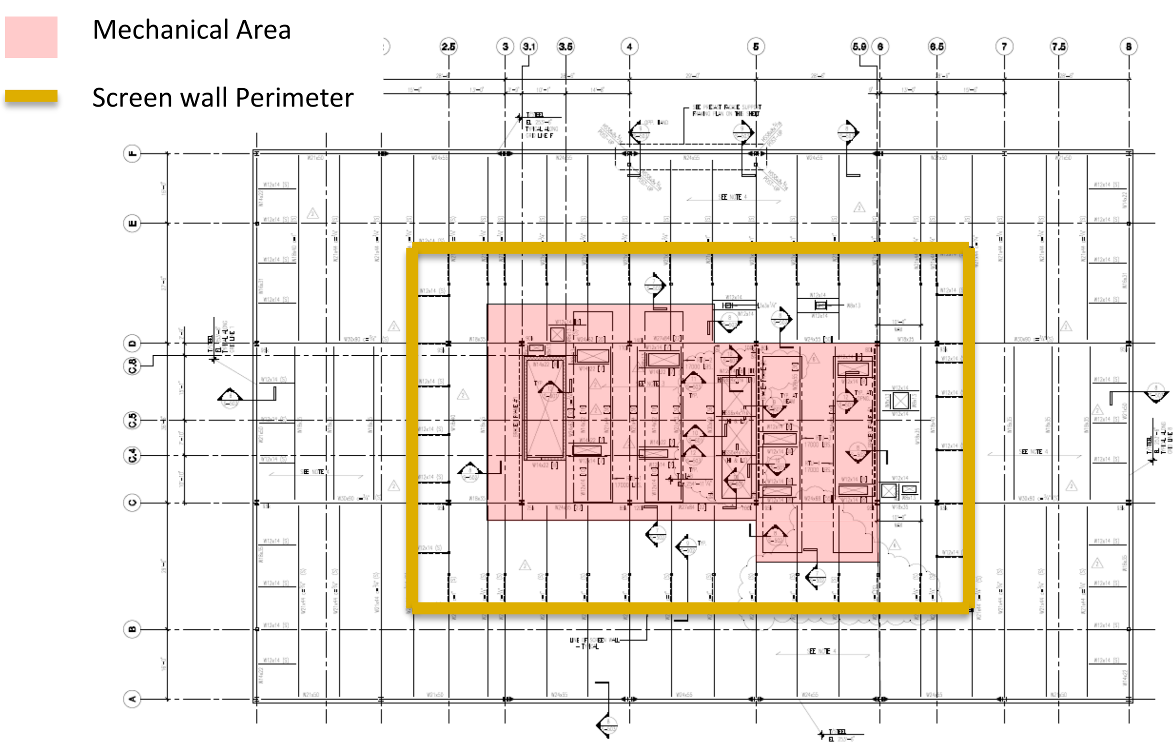


Figure 9: Structural Roof Plan (Source: Cagley & Assoc. Drawing S-209)

DESIGN CODES

Per sheet S-001, Kingstowne Section 36A was designed in accordance with the following codes:

- 2006 International Building Code
- 2006 Virginia Uniform Statewide Building Code (Supplement to 2006 IBC)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- ACI Manual of Concrete Practice, Parts 1 through 5
- Manual of Standard Practice (Concrete Reinforcing Steel Institute)
- Building Code Requirements for Masonry Structures (ACI 530, ASCE 5, TMS 402)
- Specifications for Masonry Structures (ACI 530.1, ASCE 6, TMS 602)
- AISC Manual of Steel Construction, 13^{th} Edition
- Detailing for Steel Construction (AISC)
- Structural Welding Code ANSI/AWS D1.1 (American Welding Society)
- Design Manual for Floor Decks and Roof Decks (Steel Deck Institute)

Codes / Manuals referenced for the purposes of this report:

- 2009 International Building Code
- ASCE 7-10
- ACI 318-11
- AISC Manual of Steel Construction, 14^{th} Edition
- 2008 Vulcraft Decking Manual

Material Properties

Minimum Concrete Compressive Strength	
Location	**28 Day f'c (psi)**
Footings	3000
Grade Beams	3000
Foundation Walls	5000
Shear Walls	5000
Columns	5000
Slabs-on-Grade	3500
Reinforced Slabs	5000
Reinforced Beams	5000
Elevated Parking Floors	5000
Light Weight on Steel Deck	3000

Max. Concrete W/C Ratios	
f'c @ 28 Days (psi)	**W/C (Max)**
$f'c \leq 3500$	0.55
$3500 < f'c < 5000$	0.50
$5000 \leq f'c$	0.45
Elevated Parking	0.40

Reinforcement:

- Deformed Reinforcing Bars — ASTM A615, Grade 60
- Welded Wire Reinforcement — ASTM A185
- Slab Shear Reinforcement — Decon Studrails or Equal

Masonry:

- Concrete Masonry Units — Light weight, Hollow ASTM C90, Min. f'c = 1900 psi
- Mortar — ASTM C270 – Type M (Below Grade)
  Type S (Above Grade)
- Grout — ASTM C476 – Min. f'c @ 28 days = 2000 psi
- Horizontal Joint Reinforcement — ASTM A951 – 9 Gage Truss-type Galvanized

Structural Steel:

- Wide Flange Shapes and Tees — ASTM A992, Grade 50
- Square/ Rectangular HSS — ASTM A500, Grade B, F_y = 46 ksi
- Base Plates and Rigid Frame Continuity Plates — ASTM A572, Grade 50
- All Other Structural Plates and Shapes — ASTM A36, F_y = 36 ksi
- Grout — ASTM C1107, Non-shrink, Non-metallic
  f'c = 5000 psi

Gravity Loads

Dead Loads

Superimposed Dead Loads	
Plan Area	**Load (psf)**
Office Floors	15
Roof	30
Parking Garage Floors	5

Dead loads resulting from system self-weights were calculated and estimated based on the drawings provided. The loads and the assumptions used in their determination are detailed in Appendix A.

Live Loads

Live Loads			
Plan Area	**Design Load (psf)**	**IBC Load (psf)**	**Notes**
Lobbies	100	100	
Mechanical	150	N/A	Non-reducible
Offices	80	80	Corridors used, otherwise 50 psf
Office Partitions	20	15	Minimum per section 1607.5
Parking Garage	50	40	
Retail	100	100	Located on first floor
Stairs and Exitways	100	100	Non-reducible
Storage (Light)	125	125	Non-reducible
Roof Load	30	20	

Snow Loads

Snow loads for KT36A were calculated using ASCE 7-10 for comparison to the snow loads used in the design of the building. According to Figure 7-1 in this code, Kingstowne Virginia is located in a 25 psf ground snow load area. After applying equation 7.3-1 in ASCE 7-10, this equates to a 17.5 psf flat roof snow load which matches the 17.5 psf used in the design of the building. Considering the elevated parapet above the entrance at the north side of the building and the screen wall present on the roof, unbalanced (drift) snow load can be of importance in these areas. Drift on the leeward side of the parapet can add an additional 15" of snow to the roof balanced snow load while a drift occurring on the windward side of the screen wall can add an additional 12" to the balanced snow load. The drift at the screen wall may be further reduced depending on the final decision of how much gap to leave between the bottom of the screen wall and the top of the finished roof. Condition assumptions and calculations are contained in Appendix B of this report.

Gravity System Spot Checks

Steel Decking

A spot check was performed on the steel decking to ensure that it is adequate to carry the loads. Using the 2008 Vulcraft deck catalog, it was determined that the 3 ¼" LWC on 2" x 18GA deck is more than adequate to carry the loads. Capable of carrying a 222 lb. superimposed live load for the 9'-6" span, only 122 lb. is demanded of the slab. This over strength could be attributed to inhibiting vibration concerns and meeting the fire rating without using additional fireproofing.

Composite Beam

A 45' long composite beam in a typical bay of the office floors was chosen to be checked for adequate capacity. The W21x44 beam contains 45 ¾" diameter shear studs evenly spaced along the length of the beam. In going through the analysis, it was determined that the beam was only partially composite due to the limiting case of the shear stud capacity. Partially composite beams are a common design goal in practice that is achieved through limiting the spacing of the studs. The beam passed in strength and deflection without considering the 1 ¼" camber placed on the beam.

Lateral Loads

Wind / Soil

Wind loads for KT36A were calculated using the directional procedure outlined in ASCE 7-10. When designed, the wind loads were calculated using ASCE 7-05, however, only the parameter values used for the calculations are given in the drawing sheets without the base shear values. Thus, a comparison of the calculated loads to the design loads was unattainable. The following table shows the controlling wind scenario on the building. Force / pressure diagrams can be viewed in Appendix D.

North - South (MWFRS) - South Wind									
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Tributary Area (ft2)	Force (k)
Ground (P1)	158	0	0.57	16.40	28.06	16.2		1078	17.5
P2	168.67	10.67	0.57	16.40	28.06	16.2		2155	34.9
P3	179.33	21.33	0.63	18.13	28.06	17.4	-17.0	2153	74.0
P4	190	32	0.712	20.49	28.06	19.0	-17.0	2155	77.5
5 (OL1)	200.67	42.67	0.77	22.16	28.06	20.1	-17.0	2155	79.9
6 (OL2)	214	56	0.83	23.89	28.06	21.3	-17.0	2693	103.0
7 (OL3)	227.33	69.33	0.89	25.61	28.06	22.5	-17.0	2693	106.2
8 (OL4)	240.67	82.67	0.94	27.05	28.06	23.4	-17.0	2695	108.9
Roof	253.5	95.5	0.975	28.06	28.06	24.1	-17.0	2592	106.5
Screen Wall	267	109	1.01	29.07	28.06	43.6	-29.1	1647	119.7

∑=	828	kips
∑ OT Moment=	53198	k*ft

Seismic

Calculating the seismic loads using the equivalent lateral force procedure in chapters 11 and 12 of ASCE 7-10 yielded a seismic base shear of 671 k. This is approximately 11% higher than the value used for design in the drawings, 595 k. Considering the difference in estimated building self-weight was only 1.34%, the difference is most likely attributable to human error in reading S_{ds} and S_{d1} from the ground motion charts located in the code. The table below conveys the results of the analysis. A seismic force distribution diagram can be viewed in Appendix E.

				T=	1.039	s	
				k	1.27		
				V_b=	671	kips	
Floor	Elevation (ft)	Story Height h_x (ft)	Floor Weight w_x (kips)	$w_x * h_x^k$	C_{vx}	Story Force (kips)	Story Shear (kips)
Ground (P1)	158	0	3998	0	0	0	671
P2	168.67	10.67	4250	85932.6	0.0354	23.76	671.00
P3	179.33	21.33	4268	207990.2	0.0857	57.50	647.24
P4	190	32	2261	184434.0	0.0760	50.99	589.74
5 (OL1)	200.67	42.67	4202	493982.8	0.2035	136.56	538.76
6 (OL2)	214	56	1715	284749.5	0.1173	78.72	402.19
7 (OL3)	227.33	69.33	1715	373451.4	0.1539	103.24	323.47
8 (OL4)	240.67	82.67	1709	465343.8	0.1917	128.65	220.23
Roof	253.5	95.5	1013	331294.1	0.1365	91.59	91.59
				Overturning Moment (k*ft)			39886

CONCLUSION

Although KT36A has a simple appearance from the outside, it is really a complex building that requires a respectable analysis. Due to the existing site conditions and how the building is placed in relation them, multiple loading scenarios had to be considered to accurately determine the worst load case for that particular type of loading.

After conducting this report, a more thorough understating of the structural systems and how they work together was gained. In order for this to happen, gravity loads had to be calculated and verified, multiple lateral load scenarios had to be considered and evaluated, and the drawings had to be examined in detail. Gravity dead loads were determined based on the types of construction used and information given on sheet S-001 of the structural drawing set. Live loads for the purpose of this report were determined using IBC 2009, while IBC 2006 was used by the structural engineers in the design of the building. This explains some of the differences seen in the live loading table. Other gravity loads such MEP and a general superimposed dead load were either estimated or tabulated in the structural drawings. Snow loads were calculated considering the additional effects of drifting snow at the parapet and screen wall. In some instances where the unbalanced snow load is higher than the roof live load, snow loads will control the loading condition and therefore had to be considered.

The spot checks performed on the composite decking used and a typical composite beam supporting the decking show that the steel system is adequate for carrying the required loads. These members were checked against strength as well as serviceability requirements, which were both satisfied. The composite beam just barely passed the live load deflection criteria of 1.5” with a deflection of 1.49”. This shows an efficient and economical design on the system even though the beam exceeded strength requirements by about 24%.

Lateral loading on the building was evaluated through consideration of wind, seismic, and soil forces. Since the lateral soil loads are static on the building and always present, only the wind and seismic forces were considered when comparing the loads. Wind loads in the north – south direction with the wind blowing in the south direction were found to control the lateral system design with a base shear of 828 k and an overturning moment of 53,198 k*ft. However, this will likely not be the controlling loading case when the soil loads are considered in conjunction with a wind blowing in the north direction. Lateral loading due to such conditions will be examined further in Technical Report 3.

APPENDIX: A

James Chavanic | Tech 1 | Self Weight Calcs | A.1

Parking Garage Floors and Office Level 1

8" thick flat plate NWC = 150 lb/ft³

$\omega = 150\ \text{lb/ft}^3 \cdot 8''/12''/\text{ft} = 100\ \text{lb/ft}^2$

Drop panel allowance

Average area of drop panels/floor = 5400 ft² (approx)

$150\ \text{lb/ft}^3 \cdot \frac{8''}{12''/\text{ft}} \cdot 5400\ \text{ft}^2 = 540{,}000\ \text{lb}$

Floor Area = 25,654 ft² (127' × 202')

$540{,}000\ \text{lb} / 25{,}654\ \text{ft}^2 = 21\ \text{lb/ft}^2$

Column Allowance

Estimate 8000 lb/column (10' tall)

Approx. 54 columns/floor

$$\frac{8000\ \text{lb/col.} \cdot 54\ \text{columns}}{25{,}654\ \text{ft}^2} = 17\ \text{lb/ft}^2$$

Facade

Estimate 90 psf facade self weight based on 6" thick concrete, ½" thin brick with mortar, ~~and rigid~~ and metal stud wall with insulation and GWB.

Estimate 10 psf for insulated glass curtainwall system

Garage Levels 60% precast 40% glass $0.6 \cdot 90 + 0.4 \cdot 10 = 58$ psf avg.

Office Levels 30% precast 70% glass $0.3 \cdot 90 + 0.7 \cdot 10 = 34$ psf avg.

James Chavanic Tech 1 Self Weight Calcs A.2

Office Levels 2-4

3¼" LWC on 2"x18GA Deck

Theo. Concrete Volume: $0.354\ ft^3/ft^2$

Concrete: $115\ lb/ft^3 \cdot 0.354\ ft^3/ft^2 = 41\ psf$

Decking: 2.61 psf

SW = 44 psf

Framing Allowance

Assume: 7 psf

Roof

3"x20GA TypeN with 6" thick rigid insulation

SW = 2.71 psf

Insulation

1.5 pcf / in. thick · 6" = 9 psf

$\left.\right\}$ Type 2 12 psf

3¼" LWC on 2"x18GA Deck w/ 6" rigid insulation

Conc. + Deck = 44 psf

Insulation = 9 psf

$\left.\right\}$ Type 1 53 psf

Framing Allowance

Assume: 7 psf for both types

APPENDIX: B

James Chavanic | Tech 1 | Snow Loads | B.1

$C_e = 1.0$ Exposure B (Partially) Table(7-1) ASCE 7-10

$C_t = 1.0$ (Table 7-2)

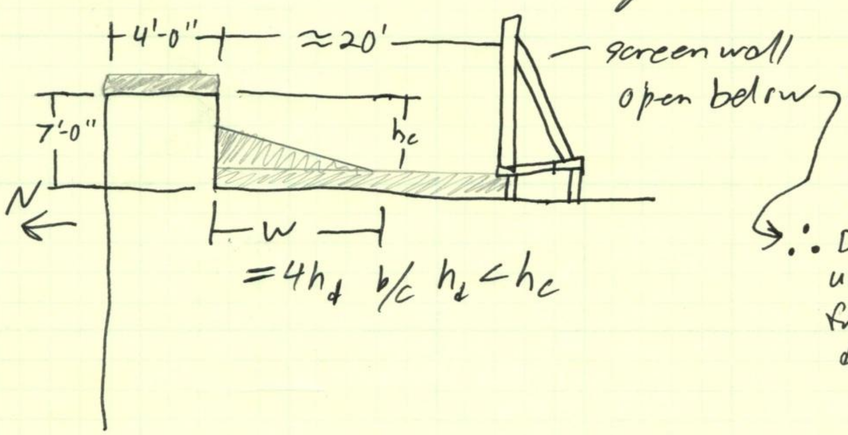
$I_s = (1.0$ risk. cat. 2)

$p_g = 25$ psf (Figure 7-1)

$p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g$

$p_f = 0.7 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 25 = \underline{17.5\ psf}$ flat roof load

One area of concern for drift loading at tall parapet ~~on~~ above main entrance on North side of building.



∴ Doesn't induce unbalanced load from Leeward direction

$h_b = \frac{P_f}{\gamma} = \frac{17.5\ psf}{0.13 \cdot 25 + 14} = 1.014' \approx 13''$

$l_u = 4'\text{-}0''$ => use 20' for calc. Per figure 7.9

$h_d = 1.25'$ for $l_u = 20$ ft and $p_g = 25$ psf

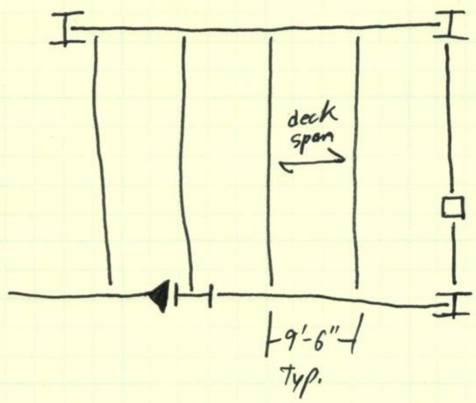
Can argue same windward drift for screen wall since $l_u = 20$ ft for both scenarios

windward h_d would be $0.75 \cdot 1.25' = 0.9375' < 1.25'$

∴ Leeward controls ~~in all cases~~ at elevated parapet
Windward controls at screen wall

APPENDIX: C

James Chavanic | Tech 1 | Decking Spot Check | C.1



Typical bay at office levels 2-4

Span = 4'-6" typical

3 1/4" LWC on 2"x18 GA deck

Using 2VLI18 from 2008 Vulcraft Catalog

SDI Max unshored Clr span

1 span = 10'-6" > 4'-6" ✓

2 span = 12'-7" > 9'-6" ✓

3 span = 12'-7" > 9'-6" ✓

Max Superimposed LL = 222 psf

$SILL_{req} = \underbrace{7psf}_{\text{framing}} + \underbrace{15psf}_{\text{SIDL}} + \underbrace{5psf}_{\text{MEP}} + \underbrace{80psf}_{\text{office}} + \underbrace{15psf}_{\text{partitions}}$

$SILL_{req} = 122 psf$

222 psf > 122 psf ∴ plenty strong enough

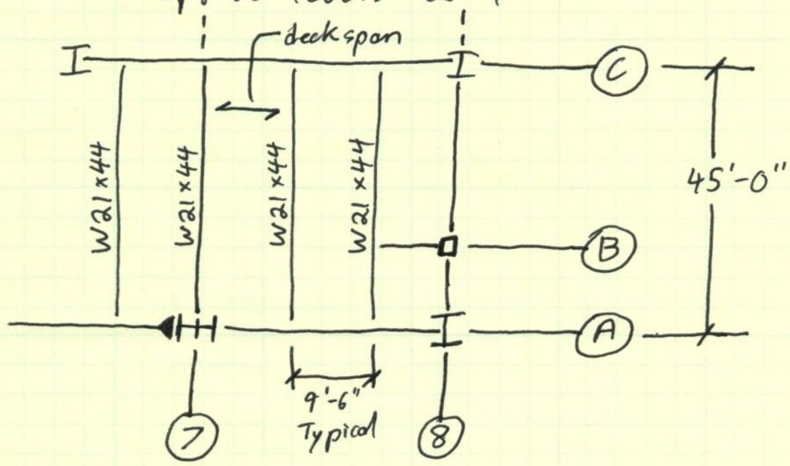
Possibly oversized to carry storage loads

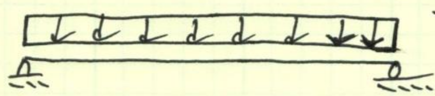
Concrete thickness needed for fire protection

→ Even though S-001 states that decks shall meet the 3 span condition, they still pass if a single span section is needed

James Chavanic | Tech 1 | Beam Spot Check | G2

Typical W21x44 beam between Grids A and C at office levels 2-4





Span: 45'-0"
Trib width: 9'-6"
$f'_c = 3000$ psi LWC
$t = 5.25''$
$A_s = 13.0 \text{ in}^2$ $f_y = 50$ ksi
$d = 20.7''$ $b_f = 6.5''$

Dead Loads
SW: 44 psf
SID: 15 psf
MEP: 5 psf
Beam SW: 44 plf

Live Loads
Office: 80 psf
Partitions: 15 psf minimum

Live Load Reduction

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} \cdot A_T}}\right)$$

$$L = 95\text{psf}\left(0.25 + \frac{15}{\sqrt{2 \cdot 45' \cdot 9.5'}}\right)$$

↳ Int. beam

$$L = 73 \text{ psf}$$

$$w_u = 1.2\, w_D + 1.6\, w_L \quad \boxed{LRFD}$$

$$w_u = 1.2[(44+15+5)9.5+44] + 1.6 \cdot 73 \cdot 9.5$$

$$w_u = 1892 \text{ lb/ft} \approx 1.9 \text{ klf}$$

James Chavanic | Tech 1 | Beam Spot Check (cont.)

C.3

$$M_u = \frac{w_u \ell^2}{8} = \frac{1.9\,klf \cdot (45ft)^2}{8} \qquad \underline{M_u = 481\ k \cdot ft}$$

$$b_{eff} = \min \begin{cases} \text{Trib width} = 9.5' \rightarrow \text{controls} \\ span/4 = 45/4 = 11.25' \end{cases} \qquad b_{eff} = 9.5' = 114''$$

$$V_{cmax} = 0.85 \cdot f'_c \cdot b_{eff} \cdot t \qquad V_{smax} = A_s \cdot f_y$$

$$V_{cmax} = 0.85 \cdot 3000 \cdot 114'' \cdot 5.25'' \qquad V_{smax} = 13.0\,in^2 \cdot 50\,ksi$$

$$V_{cmax} = 1526\ K \qquad V_{smax} = 650\ K$$

$$\Sigma Q_n = \frac{45\,studs}{2} \cdot 17.2\,K/stud \quad \text{(Table 3-21)}$$

$$\Sigma Q_n = 387K$$

↳ ⊥ deck weak stud position

↳ $< V_{smax} < V_{cmax}$ ∴ ~~PNA in concrete~~ Partially Composite

N/A in flange?

$$\cancel{x = \frac{V_{smax} - \Sigma Q_n}{2 \cdot 50\,ksi}} \qquad x = \frac{V_{smax} - \Sigma Q_n}{2 \cdot f_y \cdot b_f}$$

$$x = \frac{650k - 387k}{2 \cdot 50ksi \cdot 6.5''} = 0.4046'' < t_f = 0.45''$$

∴ PNA is in top flange

$$M_n = \Sigma Q_n \cdot (t - a/2) + A_s \cdot f_y \cdot d/2 - 2 \cdot f_y \cdot b_f \cdot x \cdot \frac{x}{2}$$

$$\text{where } a = \frac{\Sigma Q_n}{0.85 \cdot b_{eff} \cdot f'_c} \quad \text{and } x = 0.4046''$$

$$a = \frac{387K}{0.85 \cdot 114'' \cdot 3ksi} = 1.331''$$

$$M_n = 387K \cdot \left(5.25'' - \frac{1.331''}{2}\right) + 650K \cdot \frac{20.7''}{2} - 2 \cdot 50ksi \cdot 6.5'' \cdot \frac{0.4046^2}{2}$$

$$M_n = 704\ k \cdot ft$$

$$\phi M_n = 0.9 \cdot 704\,k \cdot ft$$

$$\phi M_n = 634\ k \cdot ft \implies \text{strength capacity } \underline{OK}$$

$$\phi M_n > M_u$$

James Chavanic | Tech 1 | Beam Spot Check (cont) G4

Check deflections

wet Concrete

$\Delta_{wc\,max} = \frac{l}{240} = \frac{45' \cdot 12''}{240} = 2.25''$ $I_{w21x44} = 843\,in^4$

$w_{wc} = 44psf \cdot 9.5' + 44plf = 0.462\,klf$

$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5 \cdot 0.462klf \cdot (45 \cdot 12)^4 \cdot 1/12}{384 \cdot 29000ksi \cdot 843in^4} = 1.744'' < 2.25''$ ✓

Live Load

Find I_{LB}, need $\bar{y}$ $y_2 = t - a/2 = 5.25'' - \frac{1.331}{2} = 4.585''$

$$\bar{y} = \frac{A_s \cdot d/2 + \frac{\Sigma Q_n}{F_y}(d + y_2)}{A_s + \frac{\Sigma Q_n}{F_y}} = \frac{13.0in^2 \cdot \frac{20.7}{2} + \frac{387k}{50ksi}(20.7'' + 4.585'')}{13in^2 + \frac{387k}{50ksi}}$$

$\bar{y} = 15.924''$

$$I_{LB} = I_x + A_s\left(\bar{y} - d/2\right)^2 + \frac{\Sigma Q_n}{F_y}\left(d + y_2 - \bar{y}\right)^2$$

$$I_{LB} = 843in^4 + 13in^2\left(15.924 - \frac{20.7}{2}\right)^2 + \frac{387k}{50ksi}\left(20.7 + 4.585 - 15.924\right)^2$$

$I_{LB} = 1925\,in^4$

$\Delta_{LL\,max} = \frac{l}{360} = \frac{45 \cdot 12}{360} = 1.5''$ $w_{LL} = 95psf \cdot 9.5' = 0.903\,klf$

$\Delta_{LL} = \frac{5wl^4}{384EI_{LB}} = \frac{5 \cdot 0.903klf/12 \cdot (45 \cdot 12)^4}{384 \cdot 29{,}000ksi \cdot 1925in^4} = 1.49'' < 1.50''$ ✓

Check unshored strength

$\phi M_p = 358\,k \cdot ft$

$w_{u\,DL} = 1.4 \cdot (44 \cdot 9.5 + 44) = 0.647\,klf$

$w_{u\,CL} = 1.2 \cdot (44 \cdot 9.5 + 44) + 1.6\underbrace{(20 \cdot 9.5)}_{\text{construction LL}} = 0.858\,klf$

$M_u = \frac{w_{uCL}\,l^2}{8} = \frac{0.858klf \cdot 45^2}{8}$

$M_u = 217\,k \cdot ft$ $\phi M_p > M_u \Rightarrow$ Good

$\boxed{\text{W21x44 with 45 studs is adequate}}$

Appendix: D

James Chavanic | Tech 1 | Wind Loads | P.1

Calculated using ASCE 7-10

$V = 115$ Mph (Figure 26.5-1A) Risk Cat. II

Exposure: B (Surface Roughness B > 2600 ft)

$K_d = 0.85$ (Table 26.6-1)

$K_{zt} = 1.0$ Section (26.8.2)

$G = 0.85$ (rigid building)

$GC_{pi} = \pm 0.18$ (Enclosed Building)

flexible $= f_{nat} < 1$ Hz

$f_{nat} \approx \frac{1}{T_a} \Rightarrow \frac{1}{0.611\,s} = 1.637\,Hz > 1\,Hz \therefore$ rigid

T_a calculated in seismic Loads

K_z for exposure B

0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04

z	K_z
0	0.57
10.67	0.57
21.33	0.63
32	0.712
42.67	0.77
56	0.83
69.33	0.89
82.67	0.94
95.5	0.975
109	1.01

$q_z = 0.00256 K_z K_{zt} K_d V^2$

$p = q G C_p - q_i (GC_{pi})$

James Chavanic | Tech 1 | Wind Loads Cont. | p.2

N-S Direction (South Wind) or (North Wind)

$L/B = 127/202 = 0.6287$

WW wall $C_p = 0.8$ use w/ q_z

LW wall $C_p = -0.5$ use w/ q_h

Screen wall is ~~95' x 50'~~ 122' x 87'

E-W Direction

$L/B = \frac{202}{127} = 1.59$

WW wall $C_p = 0.8$ use w/ q_z

LW wall $C_p = -0.4$ use w/ q_h

Screen wall is ~~95' x 50'~~ 122' x 87'

North - South (MWFRS) - South Wind									
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Tributary Area (ft2)	Force (k)
Ground (P1)	158	0	0.57	16.40	28.06	16.2		1078	17.5
P2	168.67	10.67	0.57	16.40	28.06	16.2		2155	34.9
P3	179.33	21.33	0.63	18.13	28.06	17.4	-17.0	2153	74.0
P4	190	32	0.712	20.49	28.06	19.0	-17.0	2155	77.5
5 (OL1)	200.67	42.67	0.77	22.16	28.06	20.1	-17.0	2155	79.9
6 (OL2)	214	56	0.83	23.89	28.06	21.3	-17.0	2693	103.0
7 (OL3)	227.33	69.33	0.89	25.61	28.06	22.5	-17.0	2693	106.2
8 (OL4)	240.67	82.67	0.94	27.05	28.06	23.4	-17.0	2695	108.9
Roof	253.5	95.5	0.975	28.06	28.06	24.1	-17.0	2592	106.5
Screen Wall	267	109	1.01	29.07	28.06	43.6	-29.1	1647	119.7

∑=	828	kips
∑ OT Moment=	53198	k*ft

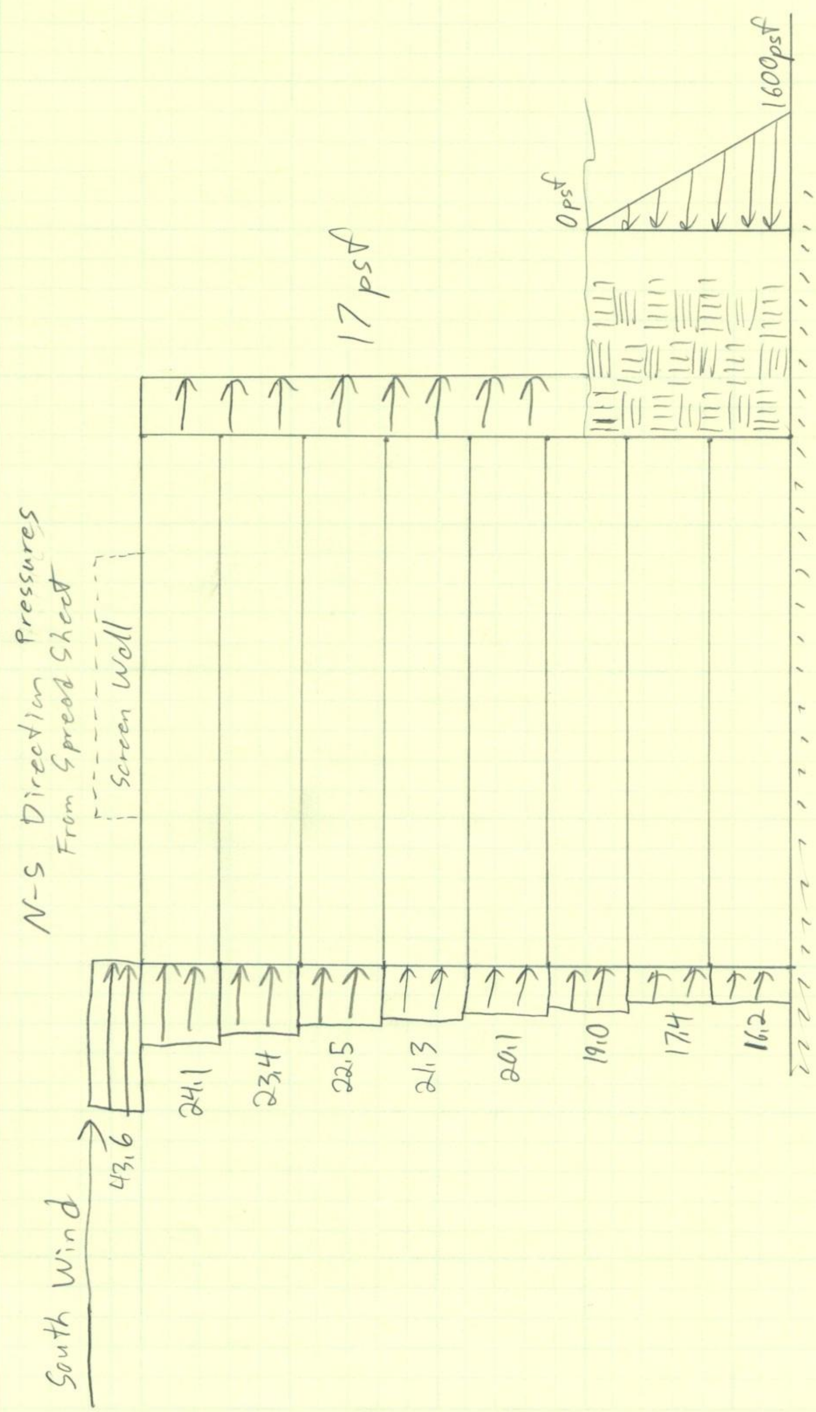
North - South (MWFRS) - North Wind									
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Tributary Area (ft2)	Force (k)
P1	158	0			26.19		-15.8	1078	17.1
P2	168.67	0			26.19		-15.8	2155	34.1
P3	179.33	0	0.57	16.40	26.19	15.9	-15.8	1078	51.2
P4	190	10.67	0.57	16.40	26.19	15.9	-15.8	2155	68.3
5 (OL1)	200.67	21.34	0.63	18.13	26.19	17.0	-15.8	2155	70.9
6 (OL2)	214	34.67	0.73	21.01	26.19	19.0	-15.8	2693	93.8
7 (OL3)	227.33	48	0.8	23.02	26.19	20.4	-15.8	2693	97.5
8 (OL4)	240.67	61.34	0.855	24.60	26.19	21.4	-15.8	2695	100.5
Roof	253.5	74.17	0.91	26.19	26.19	22.5	-15.8	2592	99.4
Screen Wall	267	87.67	0.95	27.34	26.19	41.0	-27.3	1647	112.6

∑=	694	kips
∑ OT Moment=	33582	k*ft

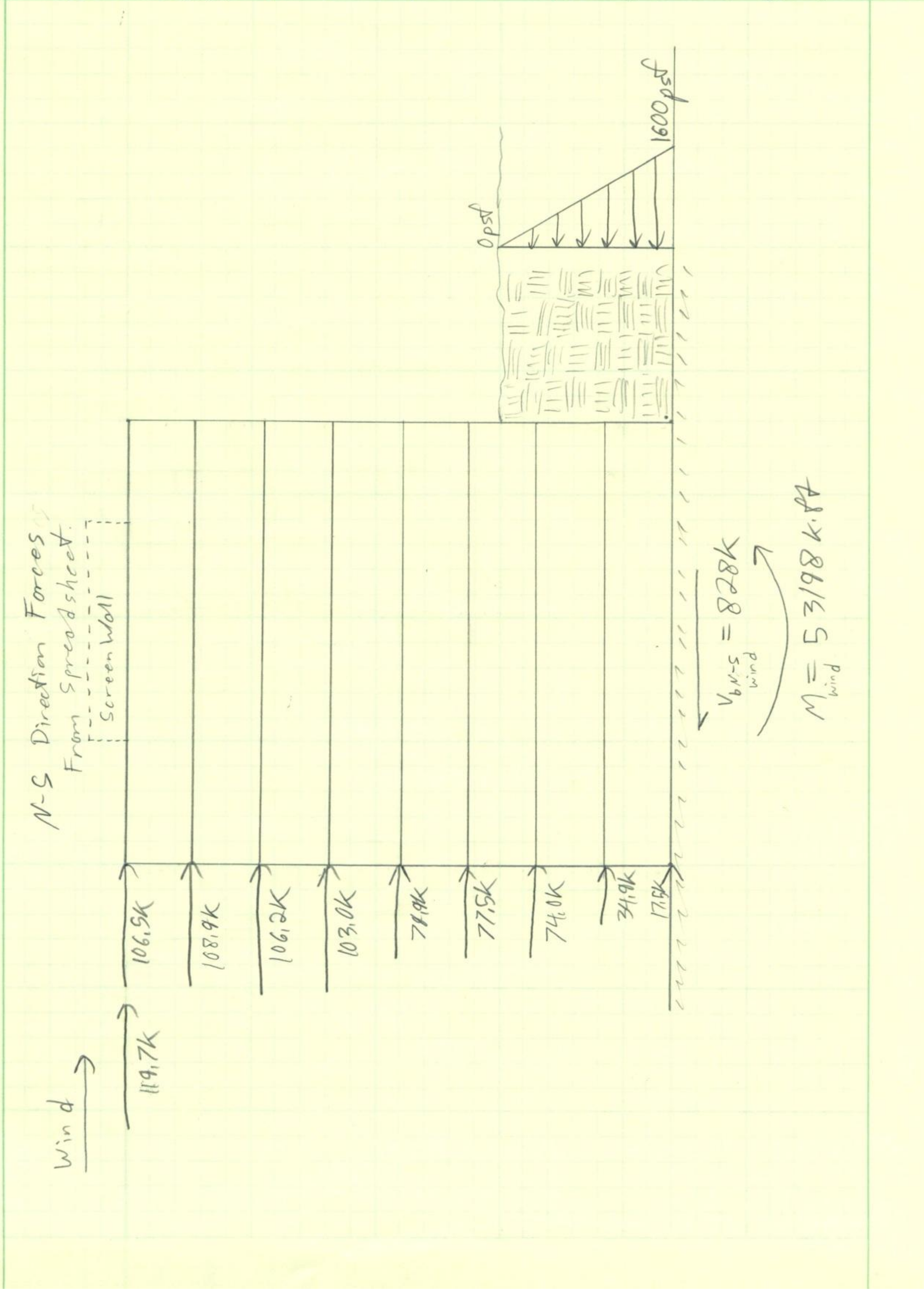
East - West (MWFRS)									
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Tributary Area (ft2)	Force (k)
Ground (P1)	158	0	0.57	16.40	28.06	16.2	-14.6	678	20.9
P2	168.67	10.67	0.57	16.40	28.06	16.2	-14.6	1355	41.7
P3	179.33	21.33	0.63	18.13	28.06	17.4	-14.6	1354	43.3
P4	190	32	0.712	20.49	28.06	19.0	-14.6	1355	45.5
5 (OL1)	200.67	42.67	0.77	22.16	28.06	20.1	-14.6	1355	47.0
6 (OL2)	214	56	0.83	23.89	28.06	21.3	-14.6	1693	60.7
7 (OL3)	227.33	69.33	0.89	25.61	28.06	22.5	-14.6	1693	62.7
8 (OL4)	240.67	82.67	0.94	27.05	28.06	23.4	-14.6	1694	64.4
Roof	253.5	95.5	0.975	28.06	28.06	24.1	-14.6	1629	63.1
Screen Wall	267	109	1.01	29.07	28.06	43.6	-29.1	1175	85.3

∑=	535	kips
∑ OT Moment=	33237	k*ft

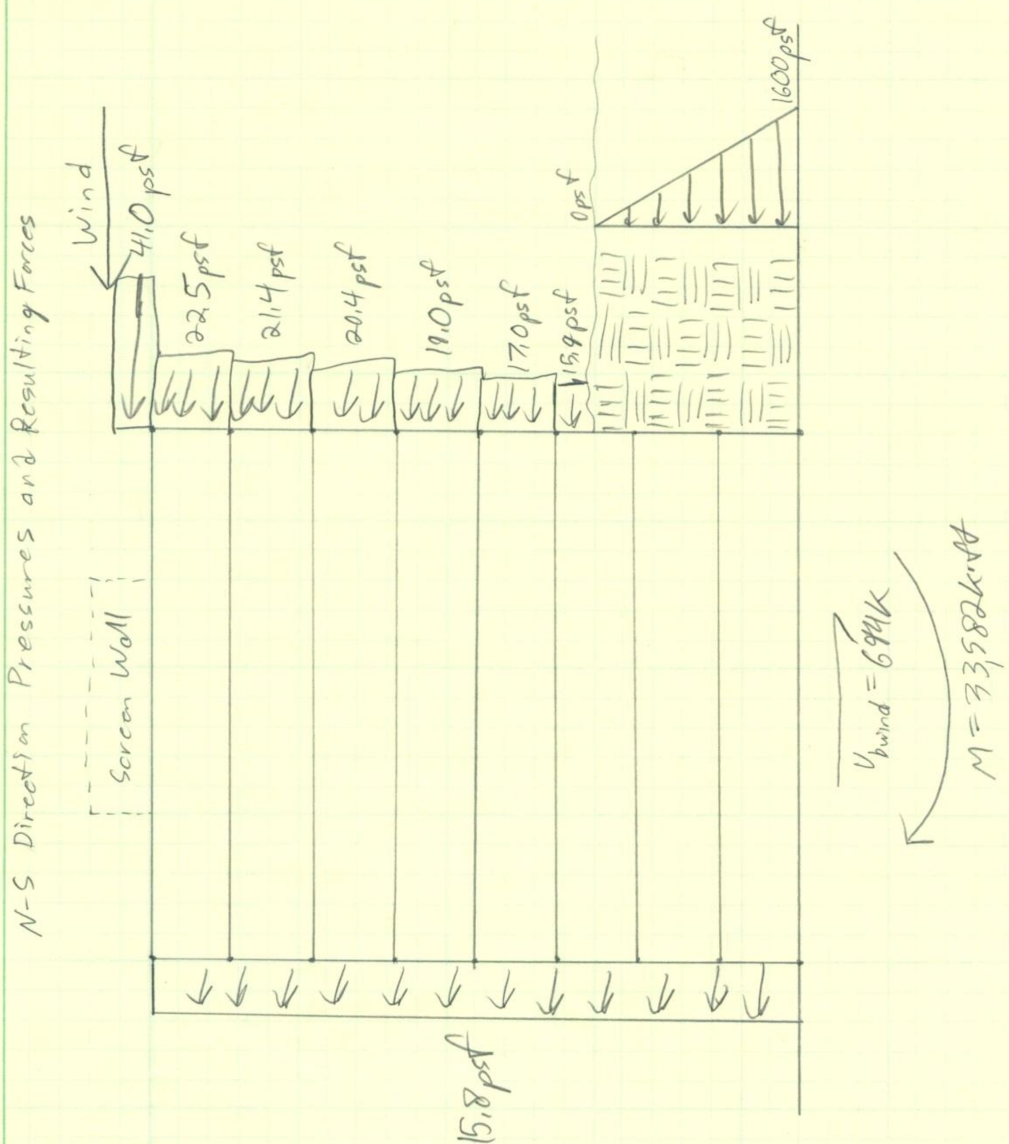
James Chavanic | Tech 1 | Wind loads Cont | D.3



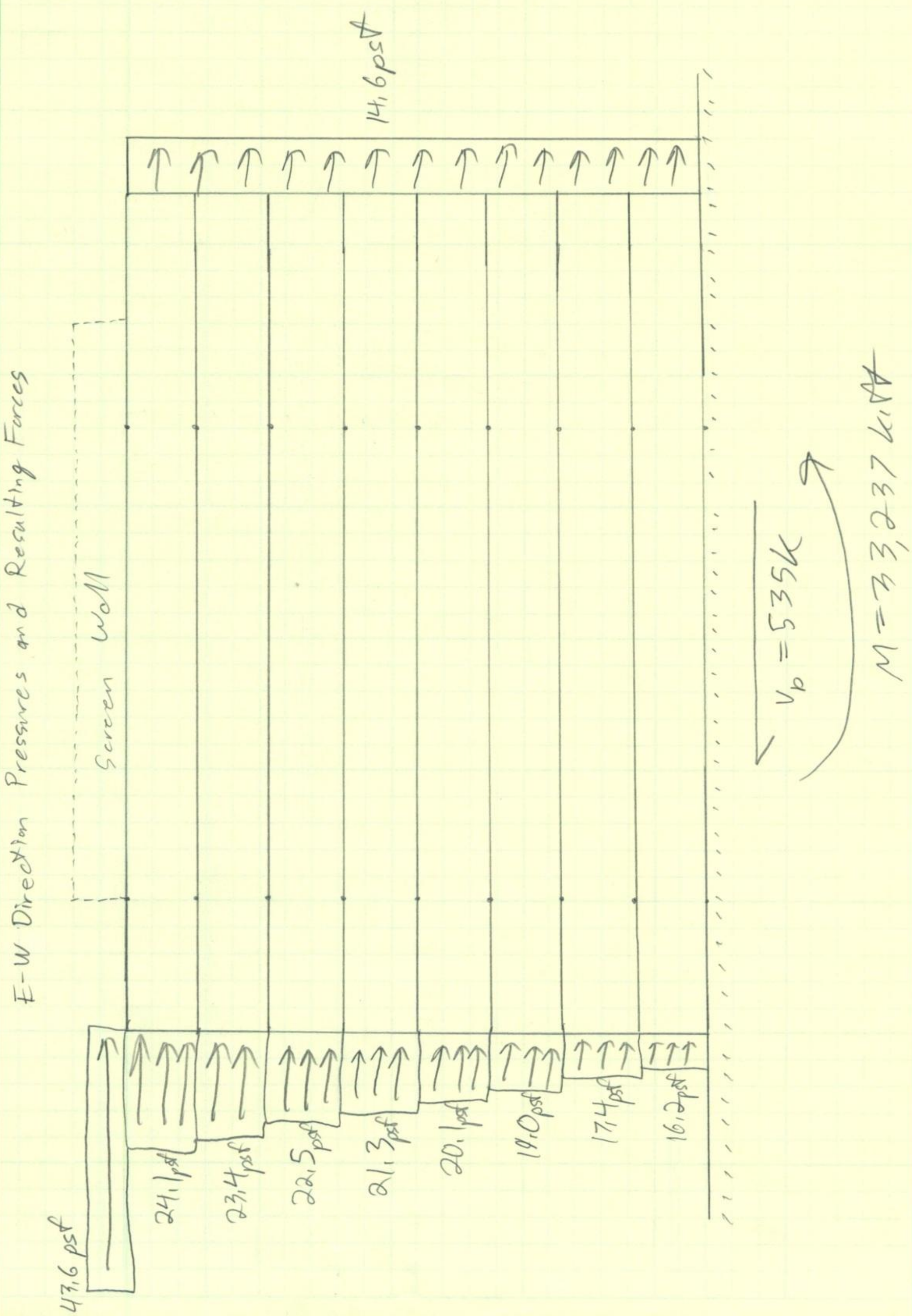
James Chavanic | Tech 1 | Wind Loads Cont. | D.4



James Chavanic | Tech 1 | Wind Loads Cont. | D.S



James Chavanic | Tech 1 | wind Loads Cont. | D.6



APPENDIX: E

James Chavanic | Tech 1 | Seismic Loads | E.1

Site Class D (Geotech) Location: Kingstowne, VA

Risk Category II

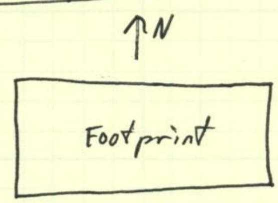
Using ASCE 7-10

$I = 1.0$

R = varies

$S_1 = 0.052$ Fig 22-2

$S_s = 0.13$ Fig 22-1



$S_{ms} = F_a S_s = 1.6 \cdot 0.13 = 0.208$ Table 11.4-1

$S_{m1} = F_v \cdot S_1 = 2.4 \cdot 0.052 = 0.1248$ Table 11.4-2

$S_{DS} = \frac{2}{3} S_{ms} = 0.1387 \rightarrow$ Seis. Cat. A

$S_{D1} = \frac{2}{3} S_{m1} = 0.0832 \rightarrow$ Seis. Cat. B $\rightarrow$ controls

12.2.3.1 Specifies a procedure for vertical combinations of systems

N-S

Shear walls: $R = 4$

Eccentrically Braced Frames: $R = 8$

Concentrically Braced Frames: $R = 3\frac{1}{4}$

E-W

Shear walls: $R = 4$

Steel Moment Frames: $R = 3\frac{1}{2}$

However, according to S-001, the Structural steel is not specifically detailed for seismic resistance $\therefore R = 3$, $\Omega_0 = 3$, $C_d = 3$ for both directions

Per table 12.6-1

Equivalent Lateral Force Analysis is Permitted

James Chavanic | Tech 1 | Seismic Loads Cont. | E.2

$V = C_s \cdot W$

$T_a = C_t h_n^x$ $\quad T_L = 8s$

$C_t = 0.02$
$x = 0.75$ (Table 12.8-2)

$T_a = 0.02 \cdot 45.5^{0.75}$

$T_a = 0.611s$

Coeff for upper limit on T_a

$C_u = 1.7$ (Table 12.8-1)

$T_{max} = C_u \cdot T_a = 1.7 \cdot 0.611s$

$T_{max} = 1.039s$ $\quad T < T_L$

$\therefore C_{s,min} = \begin{cases} \dfrac{S_{DS}}{\frac{R}{I}} = \dfrac{0.1387}{3/1} = 0.0462 \\ \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.0832}{\cancel{0.611} \cdot \frac{3}{1}} = \cancel{0.0454}\ 0.0267 \\ \quad 1.039 \end{cases}$

~~$\therefore C_s = 0.0454 > 0.01$~~ ✓

Base Shear

$V = C_s \cdot W$

W = weight of building

C_s is the same in both directions due to $R = 3$ in both directions

$V = \cancel{0.0454}\ 0.0267 \cdot 25132k = \underline{671k}$

* Cagley Value = 595K

Story Forces

$F_x = C_{vx} \cdot V$

$F_x = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

w_x = weight of each story

h_x = height from base to story of concern

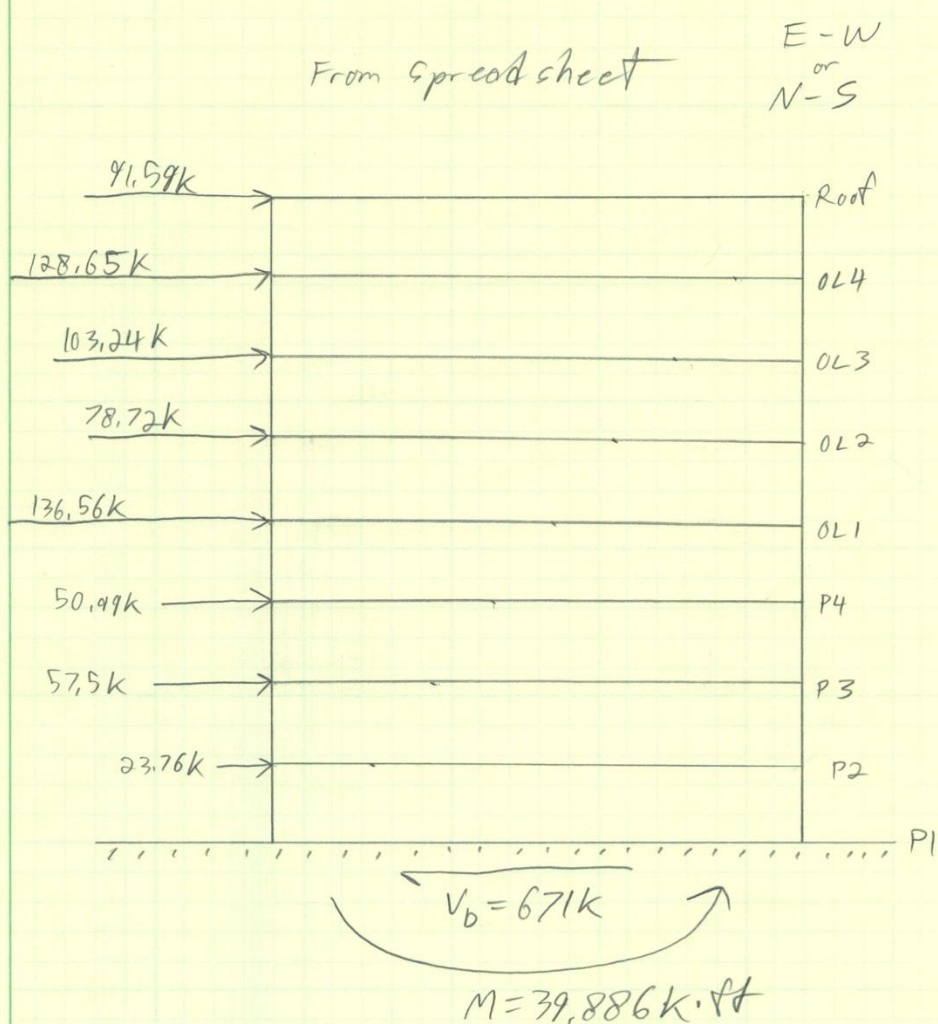
k = exponent related to structure period

– see following spreadsheet –

Floor Self Weight Calcs

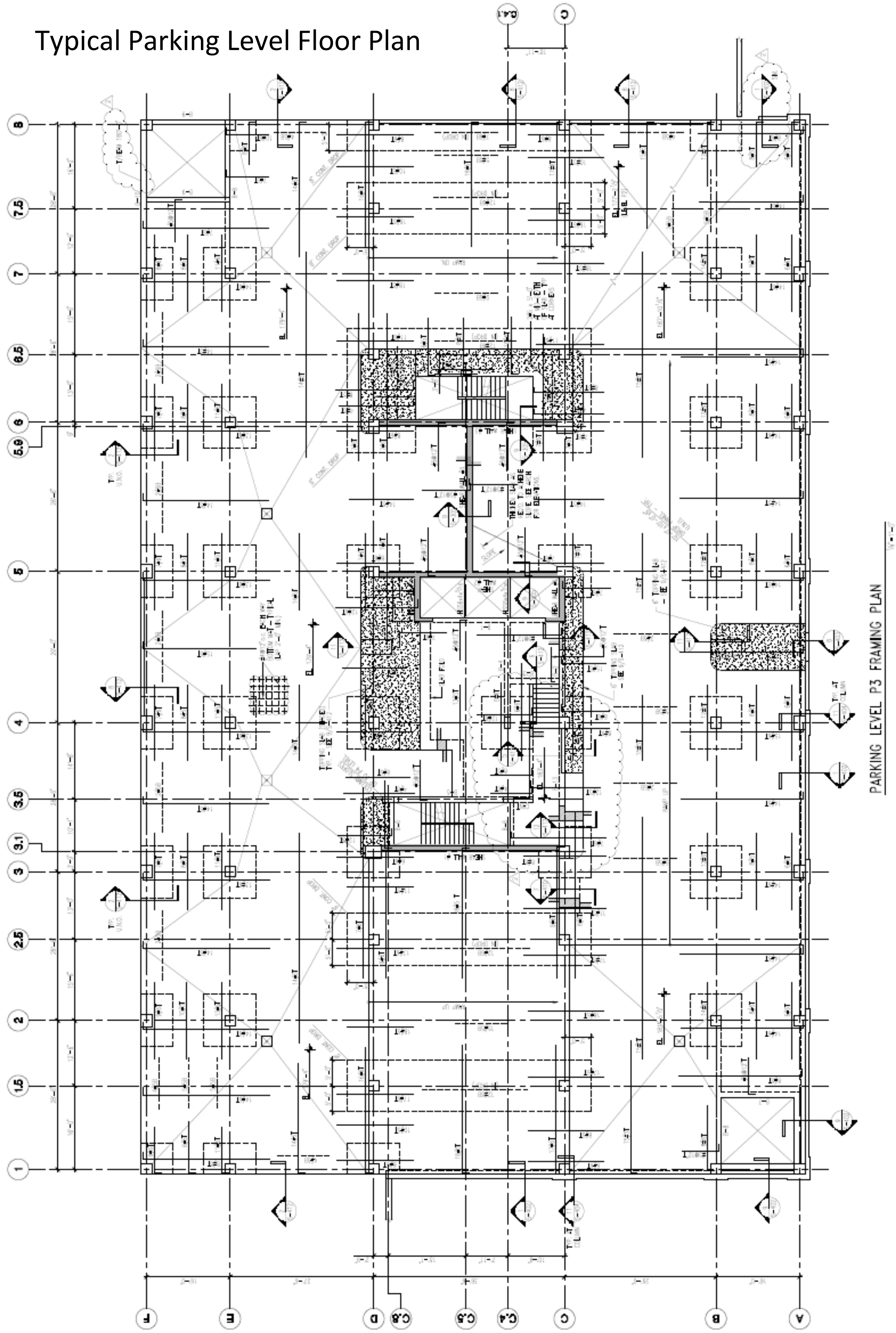
	Area (ft^2)	Perimeter (ft)	Height (ft)	Slab (psf)	Drops (psf)	Framing (psf)	Mech. (psf)	Façade (psf)	Shear Wall (k)	4 RTU @ 17k	Total (kips)
Ground Level (P1)	25116	658	0	100	21	17	5	58	203.3	0	3998
P2	25103	658	10.67	100	21	17	5	58	252.8	0	4250
P3	25235	658	10.66	100	21	17	5	58	252.8	0	4268
P4	11192	658	10.67	100	21	17	5	58	252.9	0	2261
5th Floor (OL1)	25299	658	10.67	100	21	17	5	58	126.4	0	4202
6th Floor (OL2)	25299	658	13.33	44	0	7	5	34	0.0	0	1715
7th Floor (OL3)	25299	658	13.33	44	0	7	5	34	0.0	0	1715
8th Floor (OL4)	25299	658	13.34	44	0	7	5	34	0.0	0	1709
Roof Type 1	4750	658	12.83	53	0	7	5	34	0.0	68	1013
Roof Type 2	20549			12	0	7	5				
										∑=	25132 kips

James Chavanic | Tech 1 | Seismic Loads cont. | E.4

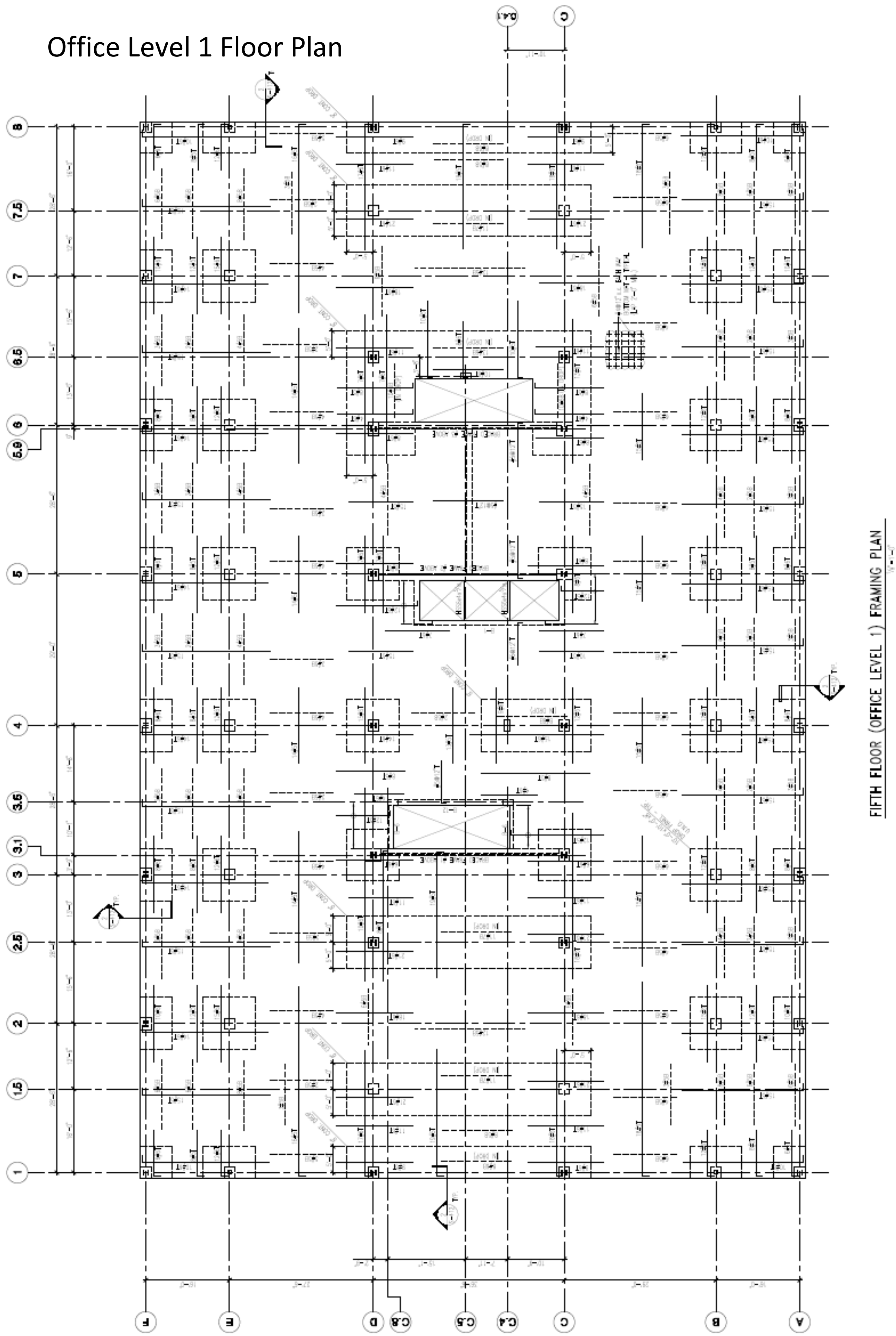


APPENDIX: F

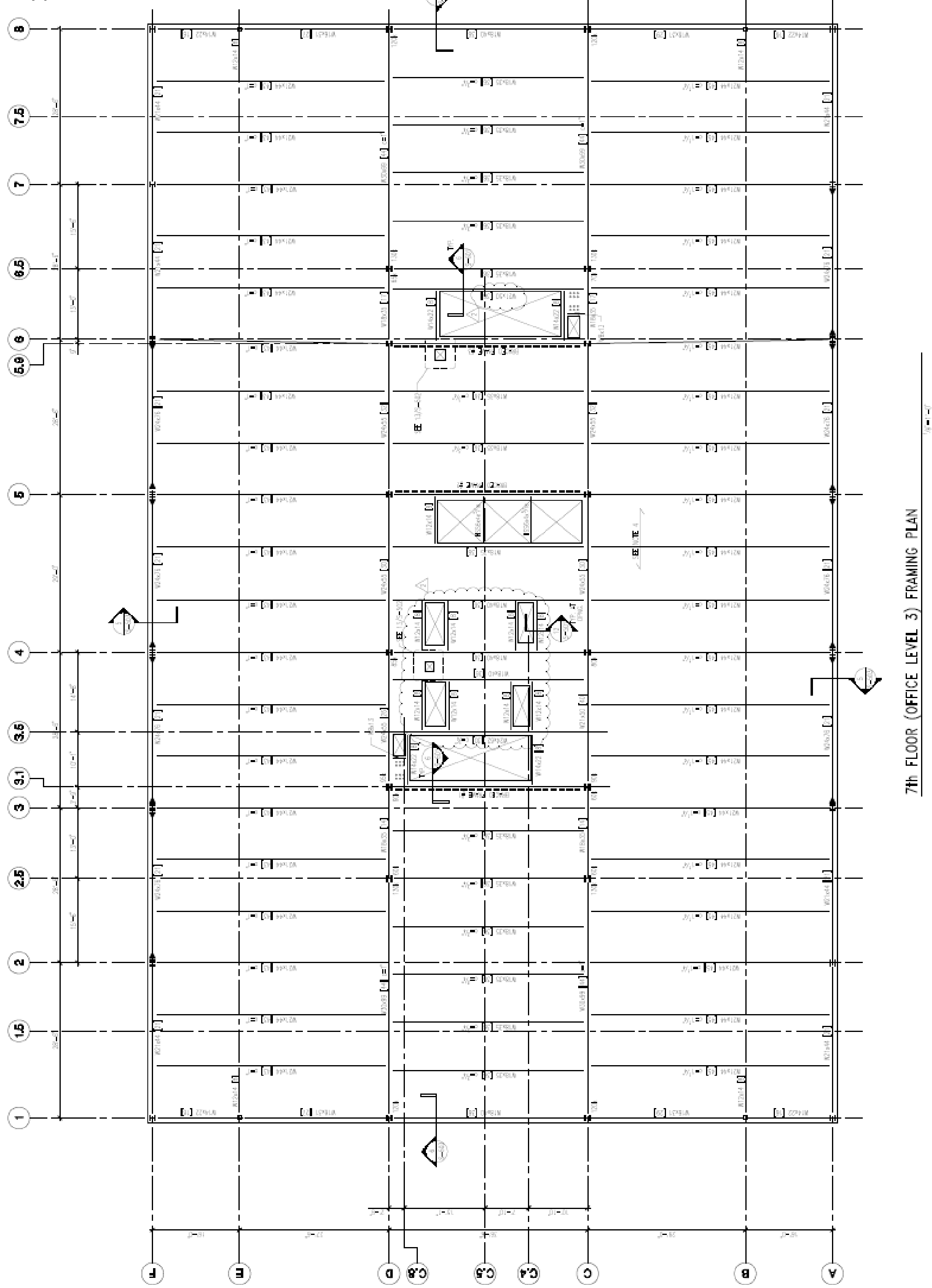
Typical Parking Level Floor Plan



Office Level 1 Floor Plan



Typical Office Level Floor Plan



Roof Level Floor Plan